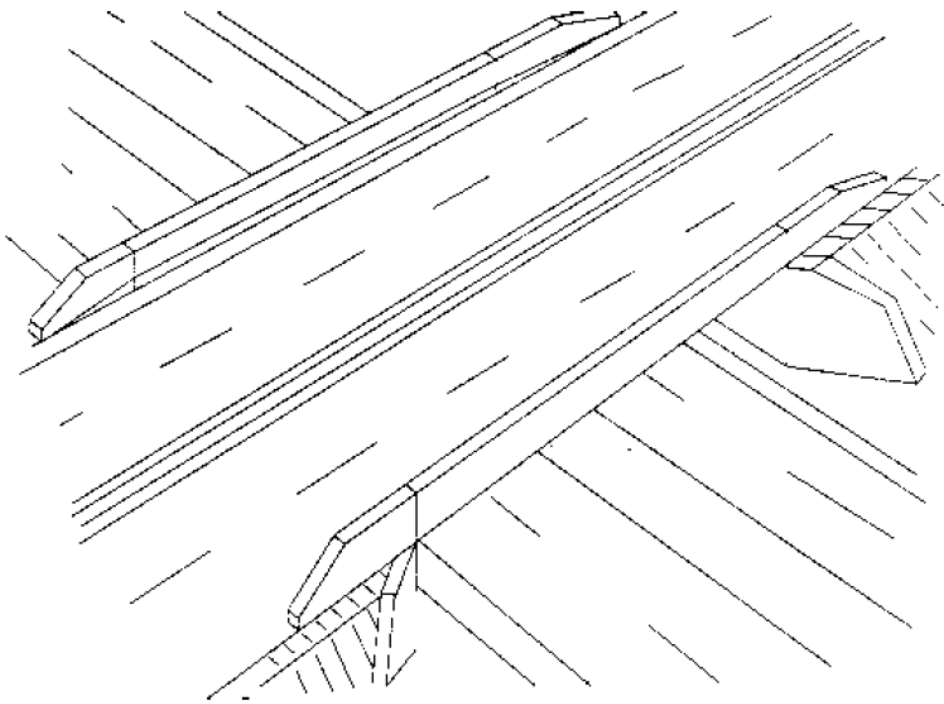
DESIGN MANUAL
PART 4

Volume 1 of 2 Volumes



STRUCTURES

PROCEDURES – DESIGN – PLANS PRESENTATION

PDT – PUB No. 15M

SEPTEMBER 2007 EDITION

(Dual Unit)

(Includes Change No. 1)

COMMONWEALTH OF PENNSYLVANIA
DEPARTMENT OF TRANSPORTATION

PENNDOT

<table>
<tr><th colspan="4">Conversion Factors U.S. Customary to SI</th></tr>
<tr><th colspan="2">English</th><th colspan="2">SI Metric</th></tr>
<tr><td colspan="2">1 IN</td><td colspan="2">25.4 mm</td></tr>
<tr><td colspan="2">1 FT</td><td colspan="2">304.8 mm</td></tr>
<tr><td colspan="2">1 IN^2</td><td colspan="2">645 mm^2</td></tr>
<tr><td colspan="2">1 IN^2/FT</td><td colspan="2">2.12 mm^2/mm</td></tr>
<tr><td colspan="2">1 cubic Yard = 1 CY</td><td colspan="2">0.764 m3</td></tr>
<tr><td colspan="2">1 MPH = 1.467 FT/SEC</td><td colspan="2">1.609 km/hr = 0.447 m/s</td></tr>
<tr><td colspan="2">1 FT/SEC</td><td colspan="2">0.3048 m/s = 1.097 km/hr</td></tr>
<tr><td colspan="2">g = 32.2 FT/SEC^2</td><td colspan="2">9.81 m/s^2</td></tr>
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<tr><td colspan="2">1 lb IN (moment)</td><td colspan="2">113.0 N mm</td></tr>
<tr><td colspan="2">1 lb FT (moment)</td><td colspan="2">1356 N mm</td></tr>
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<tr><td colspan="2">1 lb/FT^2</td><td colspan="2">4.883 kg/m^2</td></tr>
<tr><td colspan="2">1 PSF/FT</td><td colspan="2">1.57×10^{-7} MPa/mm = 0.157 Pa/mm</td></tr>
<tr><td colspan="2">1 FT lb (energy)</td><td colspan="2">1.356 J (joule) or N m</td></tr>
<tr><td colspan="2">1 PSI (lb/IN^2)</td><td colspan="2">6.895×10^{-3} MPa (mega pascal)</td></tr>
<tr><td colspan="2">1 PSF (lb/FT^2)</td><td colspan="2">4.79×10^{-5} MPa = 47.9 Pa</td></tr>
<tr><td colspan="2">1 PSF/FT</td><td colspan="2">4.79×10^{-5} MPa/mm</td></tr>
<tr><td colspan="2">1 PCF (lb/FT^3) (unit weight)</td><td colspan="2">16.02 kg/m^3 = γ (density)</td></tr>
<tr><td colspan="2">1 PCY (lb/yard^3)</td><td colspan="2">0.593 kg/m^3 = γ (density)</td></tr>
<tr><td colspan="2">1 KIP (kilo pounds)</td><td colspan="2">4448 N = 4.448 kN</td></tr>
<tr><td>1 K IN</td><td>1 K IN^2</td><td>1.13×10^5 N • mm</td><td>2.87 × 106 N • mm^2</td></tr>
<tr><td colspan="2">1 K/IN</td><td colspan="2">175.2 N/mm</td></tr>
<tr><td colspan="2">1 KLF (KIP/FT) or KPF</td><td colspan="2">14.6 N/mm</td></tr>
<tr><td colspan="2">1 KSI (KIP/IN^2)</td><td colspan="2">6.895 MPa</td></tr>
<tr><td colspan="2">1 KSF (KIP/FT^2)</td><td colspan="2">0.0479 MPa</td></tr>
<tr><td colspan="2">1 KCF (KIP/FT^3) (unit weight)</td><td colspan="2">1.602 × 104 kg/m^3 = γ (density)</td></tr>
<tr><td colspan="2">1 TSF (ton/FT^2)</td><td colspan="2">0.0958 MPa</td></tr>
<tr><td colspan="2">1 TCF (ton/FT^3) (unit weight)</td><td colspan="2">3.204 × 104 kg/m^3 = γ (density)</td></tr>
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<tr><td colspan="2">1 knot</td><td colspan="2">0.514 m/s = 1.85 km/hr</td></tr>
<tr><td colspan="2">1 tonne (2.205 KIP)</td><td colspan="2">1.00 metric ton (1000 kg)</td></tr>
</table>

To convert 55 KIPs to newtons: 55 KIPs • 4.448 kN/KIP = 244.64 = 245 kN

List of incorporated SOLs

SOL 430-06-28
Bridge Preservation

SOL 431-98-17
Contractor Alternate Design

SOL 431-01-13
Redundancy of Tie-back Soldier Pile Walls w/Precast Concrete Facing Panels

SOL 431-02-09
Enhancement & Rails to Trails Pedestrian Structures and Bridges on Shared Use Trails Signature Authority & Review Responsibility

SOL 431-03-14
Design Manual Part 4 Revisions resulting from Department and APC Task Groups for Temporary Excavation Support and Protection System, Steel Stability, and Pre-drilling of Piles.

SOL 431-04-05
Design-Build: Design Manual Part 4 Policies and Procedures and Standard Special Provisions for Structures

SOL 431-04-06
Active Bridge Design and Construction Standards, Release of Change #2 to BC-700M Series, Change #1 to BD-600M Series, Design Manual Part 4 Revised Pages, Pub. 7 Item Numbers and Special Provisions

SOL 431-05-03
Active Bridge Design and Construction Standards, Release of Change #3 to BC-700M Series, Change #2 to BD-600M Series, Design Manual Part 4 Revised Pages, Pub. 7 Item Numbers and Special Provisions

SOL 431-07-01
Bridge Pro-team Procedures

SOL 431-07-02
Procedure revisions for TS&L Approvals and Foundation Approval

SOL 432-06-08
Lateral clearance requirements

SOL 437-04-01
Usage of 22 X 34" Standard Sheet Sizes for Deliverable Plans

SOL 437-04-02
Release of BRADD Version 3.1.0

SOL 437-05-01
Use of BRADD on PennDOT and Non-PennDOT Pennsylvania Bridge Projects

DM-4 Change No. 1 List of incorporated SOLs and replaced pages

SOL 431-07-11
Publication 15M, September 2007 Edition
Initial release

SOL 431-08-02
Bridge Painting Guidelines
Pages Dated January 2008
TOC ii, A.5 – ii, A.5 – 61 to 67

SOL 431-08-04
Mechanically Stabilized Earth Wall Design Revision
Pages Dated May 2008
B.11 – I, B.11 – 36 to 39

SOL 431-08-14
Eligibility for Federal Funding for Structurally Deficient Bridge Decks
Pages Dated September 2007
A.5 – 25 to 26

SOL 431-09 -04
Appendix J - Approved Commercially Available and Consultant Developed Software
Pages Dated September 2007
Appendix J

SOL 431-09-06
Bridge Preservation and Preventive Maintenance
Eligible for Federal Funding
Pages Dated March 2009
A.5 – ii, A.5 – 60 to 66

SOL 431-09-07
Pre-augering Requirements for Integral Abutment Piles
Pages Dated April 2009
Ap.G-12

SOL 431-09-08
Revisions to Review and Approval Responsibility Tables
Pages Dated June 2009
A.1 – 46 to 50

SOL 431-09-10
Summary of New Bridge and Structure Products
Pages Dated September 2009
Ap.K-1

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PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 1
PART A: POLICIES AND PROCEDURES

PREFACE

All references to AASHTO Division I sections, articles, equations, figures or tables carry the prefix A.

References to AASHTO commentary carry the prefix AC.

References to Design Manual, Part 4, Volume 1, Part A, "Policies and Procedures", carry the prefix PP.

References to Design Manual, Part 4, Volume 1, Part B, "Design Specifications", carry the prefix D.

References to commentary to Design Manual, Part 4 carry the prefix DC.

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 1
PART A: POLICIES AND PROCEDURES

CHAPTER 1 - ADMINISTRATIVE CONSIDERATIONS

CHAPTER 1 - TABLE OF CONTENTS

LIST OF ABBREVIATIONS

AAR	Association of American Railroads
AASHTO	American Association of State Highway and Transportation Officials - Generally "AASHTO" is a reference to LRFD Bridge Design Specifications
ADT	Average Daily Traffic
ADTT	Average Daily Truck Traffic
AITC	American Institute of Timber Construction
AREMA	American Railway Engineering and Maintenance of Way Association
ASTM	American Society of Testing and Materials
AWS	American Welding Society
BC-Stds.	Bridge Construction Standards
BD-Stds.	Bridge Design Standards
BIS	Bureau of Information Systems
BPAA	Bridge Plan Approval Application
BMS	Bridge Management System
BOCM	Bureau of Construction and Materials
BQAD	Bridge Quality Assurance Division
BRADD	Bridge Automated Design and Drafting
Bul. 15	Approved Construction Materials (Bulletin 15)
CADD	Computer-Aided Design and Drafting
C/C	Center-to-Center
CVN	Charpy V-notch (Impact Requirement)
DCNR	Department of Conservation and Natural Resources
DEP	Department of Environmental Protection
DGCE	District Grade Crossing Engineer
DPE	District Project Engineer
DM-4	Design Manual, Part 4
DPM	District Project Manager
FAST	Facilitate Acceleration through Special Techniques
FHWA	Federal Highway Administration

LIST OF ABBREVIATIONS (Continued)

HQAD	Highway Quality Assurance Division
LFTF	Liquid Fuel Tax Funded
LRFD	Load and Resistance Factor Design
MPS	Master Policy Statement
MSE	Mechanically Stabilized Embankment
MTD	Materials Testing Division
NBIS	National Bridge Inspection Standards
PC	Personal Computer
PennDOT	Pennsylvania Department of Transportation
PMC	Project Management Committee
P/S	Prestressed
PS&E	Plans, Specifications and Estimate
PTF	Pennsylvania Traffic Factor
PUC	Public Utility Commission
Q/A	Quality Assurance
RC	Reinforced Concrete
RC-Stds.	Roadway Construction Standards
RQD	Rock Quality Designation
SCEF	Structural Committee for Economical Fabrication
SIRS	Structure Inventory Record System
S. R.	State Route
STStds.	Structural Standards (Predecessor to BC Series)
TC-8700 Series	Traffic Control Signing Standards
TS&L	Type, Size and Location
VEACTT	Value Engineering Accelerated Construction Technology Transfer

INTRODUCTION

Design Manual, Part 4 is part of a series of Department (Pennsylvania Department of Transportation) design manuals which have the specific objective of obtaining uniformity and establishing standard policies and procedures in the preparation of design and construction plans for highway structures. The provisions of the AASHTO LRFD Bridge Design Specifications, SI Units, and the AASHTO LRFD Bridge Design Specifications, Customary U.S. Units, shall govern where applicable, except as specifically modified by the requirements herein.

The inclusion of specified design criteria in this Manual does not imply that existing roadways, which were designed and constructed using different criteria, are either substandard or must be reconstructed to meet the criteria contained herein. Many existing facilities which met the design criteria at the time of their construction are adequate to safely and efficiently accommodate current criteria unless a safety or capacity problem manifests itself.

Since it is not feasible to provide a highway system that is continuously in total compliance with the most current design criteria, it is imperative that both new construction and reconstruction projects are selected from a carefully planned program which identifies those locations in need of improvement and then treats them in priority order. Once a new construction or reconstruction project is selected in this manner, this Manual should be used as a guide in determining the appropriate design criteria to be used.

The Design Manual has precedence over AASHTO design specifications and bridge design and constructions standards.

The Design Manual, which is published in loose-leaf form to facilitate changes and expansion, is divided into three parts: (A) "Policies and Procedures", (B) "Design Specifications" and (C) "Appendices to Design Manual, Part 4".

In Part B, the AASHTO LRFD article numbering system is followed. Where a new article has been added, the suffix P, to designate "Pennsylvania Article", appears at the end of the new article number.

All references to AASHTO LRFD Bridge Design Specifications sections, articles, equations, figures or tables carry the prefix A. References to AASHTO commentary carry the prefix AC. References to Design Manual, Part 4, Volume 1, Part A, "Policies and Procedures", carry the prefix PP. References to Design Manual, Part 4, Volume 1, Part B, "Design Specifications", carry the prefix D. References to commentary to Design Manual, Part 4 carry the prefix DC.

When a DM-4 article modifies and/or adds information to an AASHTO article, the first sentence of the DM-4 article shall read "The following shall supplement Ax.x.x". When a DM-4 article replaces an AASHTO article, the first sentence shall read "The following shall replace Ax.x.x".

Policy and procedure information which does not have a corresponding AASHTO article number will be placed using a "0" attached to the article number.

Maintenance and updating of this Manual is the responsibility of the Bureau of Design.

CHAPTER 1

ADMINISTRATIVE CONSIDERATIONS

1.1 APPLICABLE SPECIFICATIONS AND STANDARDS

1.1.1 Design Specifications

The following specifications, unless otherwise modified or amended in this manual, shall govern the design of highway structures:

(1) AASHTO Design Division, LRFD Bridge Design Specifications, 3rd Edition, 2004

(2) AASHTO/AWS D1.5M/D1.5:2002 - Bridge Welding Code

(3) AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaries and Traffic Signals, 4th Edition 2001 including the 2002 and 2003 Interims.

(4) AREMA Specifications for Railway Bridges, 2004, and other specific requirements by the operating railroad

(5) AITC, American Institute of Timber Construction, Timber Construction Manual, Third Edition, 1985

(6) AASHTO Guide Specifications for the Design of Stress - Laminated Wood Decks, April 1991

(7) AASHTO Guide Specification for Strength Design of Truss Bridges (Load Factor Design), 1985.

1.1.1.1 AASHTO INTERIM SPECIFICATIONS

As AASHTO Interim Specifications are published, BQAD will review the interims and incorporate them into this manual as necessary.

1.1.1.2 DEVIATIONS FROM SPECIFICATIONS

Any deviations from the specifications and standards listed above, or the Department's design criteria described hereafter, require the Chief Bridge Engineer's approval. The approved design criteria shall be shown on the bridge plans.

1.1.1.3 ORDER OF PRECEDENCE

The design criteria given in this Manual supersedes any criteria given in the referenced design specifications in PP1.1.1 and PP1.1.3.

In case of conflict or where clear precedence cannot be established, the Chief Bridge Engineer shall establish governing specifications.

1.1.1.4 INTERPRETATION OF DESIGN SPECIFICATIONS

For Design Manual, Part 4, and AASHTO LRFD Bridge Design Specifications, the final interpretation shall be made by the Chief Bridge Engineer.

1.1.1.5 APPLICABLE SPECIFICATIONS FOR LOCAL PROJECTS

Local projects may be designed using AASHTO specifications only, unless the local authority agrees to use the Department's specifications in addition to the AASHTO specifications. Additionally, Publication 70M, <u>Guidelines for Design of Local Road and Streets</u>, may be used.

1.1.1.6 AASHTO GUIDE SPECIFICATIONS

1. <u>Guide Specifications for Fatigue Design of Steel Bridges, 1989</u>

   This specification shall not be used.

   The current <u>Design Manual, Part 4</u>, in combination with the AASHTO LRFD Bridge Design Specifications provides all needed criteria and directions.

2. <u>Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges, 1989</u>

   This specification shall not be used.

   The current PennDOT Bridge Inspection Manual in combination with the AASHTO Manual for Maintenance Inspection of Bridge provides adequate criteria and directions.

   The Department may ask any designer or bridge strength evaluator to use any of these publications under a special situation. The Chief Bridge Engineer should be contacted prior to authorization of their usage.

3. <u>Guide Specifications for Bridge Railing, 1989</u>

   This specification shall not be used.

   Only use Department established standard details for bridge railing and transition. Any needed modification will be published as a modification to the Department's railing standards. Special designs must match the Department's railing shape and size and ultimate strength.

4. <u>Guide Specifications for Design and Construction of Segmental Concrete Bridges, 1999</u>

   These guide specifications, with the exception of the following items, are to be used in preparation of segmental concrete projects:

   a. Corrosion protection of prestressing steel will be provided in accordance with Publication 408, Section 1108.03(b).

   b. Provision for deck replacement and full-depth repair method must be presented. Provision for a 30 mm {1 1/4 in.} latex overlay.

   c. Do not use unbonded prestressing system as a permanent prestressing system.

   d. Prior to starting design of any component of a segmental bridge, applicable section(s) of these specifications and design methodology must be reviewed and approved by the Chief Bridge Engineer.

5. <u>Guide Specifications for Structural Design of Sound Barriers,</u> 1989 including the 1992 and 2002 Interims.

   Refer to PP3.6.4 for further guidelines.

6. AASHTO <u>Guide Specifications for Horizontally Curved Steel Girder Highway Bridges 2003</u> with Design Examples for I-Girder and Box-Girder Bridges

   Implement these Guide Specifications for Horizontally Curved Highway Bridges where a Load Factor Design is permitted to be used on a project. Permission to use Load Factor Design is to be requested at TS&L stage.

7. Guide Specifications for Alternate Load Factor Design Procedures for Steel Beam Bridges Using Braced Compact Sections, 1991

   The Guide Specifications for alternate load factor design will not be implemented and are not to be used as a design guide on Department projects.

8. Guide Specifications for Aluminum Highway Bridges, 1991

   The Guide Specifications for aluminum design may be used for information only.

   Availability of the Specifications is not an endorsement to use aluminum bridges on Department projects. Special approval of the Chief Bridge Engineer is required for aluminum usage in bridges, as well as specific parts of the specifications on a project-by-project basis.

   A copy of the Guide Specifications for Aluminum Design is available in the BQAD.

9. Guide Specifications for Fatigue Evaluation of Existing Steel Bridges, 1990.

   This Specification shall not be used on Department projects.

1.1.1.7 AASHTO STANDARD SPECIFICATIONS

1. Standard Specifications for Highway Bridges, 15th Edition, 1992, and Interim Specifications 1993 and 1994

   This Specification is in conjunction with AASHTO, Guide Specification for Horizontally Curved Highway Bridges, shall be used for the Department's design of horizontally curved steel girder highway bridges.

   This Specification shall also be used when a service load or load factor design are requested.

2. Standard Specifications for Structural Supports for Highway Signs, Luminaries and Traffic Signals, 4th Edition 2001.

   Implement the 2002 and 2003 interims for these standard specifications.

1.1.2 Construction Specifications

(1) Publication 408, Specifications

(2) Changes to Publication 408

1.1.3 Bridge Standards

The following standard drawings shall be followed and used wherever practicable:

Standards for Bridge Construction, BC Series
Standard Plans for Low Cost Bridges, BLC Series (Only use on local bridge projects and designs. Details must be upgraded to comply with the current policy. Only a portion of plans have been converted to the metric system.)

Standards for Bridge Design, BD Series

Standards for Roadway Construction, RC Series

Traffic Control and Signing Standards, TC-8700 Series

Lighting, TC-8715

Reference to any or all of the Standard Drawings in the BC Series, on the structure design drawings, in lieu of showing specific details is encouraged, provided coordinating information is shown on the design drawings. Reference to applicable RC Standards shall also be made.

1.1.3.1 SUPPLEMENTAL DRAWINGS

The referenced Standard Drawings shall be listed in the Table for Supplemental Drawings and in the General Notes on the Roadway Plans with the other referenced drawings.

Lists of past and current bridge standards are given in Appendix C for information.

1.2 MODIFICATIONS TO DESIGN MANUAL

1.2.1 Need for Modifications

Whenever a user believes that modifications (including additions) to Design Manual, Part 4, would improve the present design practice, the following course of action shall be taken.

1.2.2 Recommended Information for Modifications

The recommended modification shall be transmitted to the Director, Bureau of Design, by completing the revision request form shown in Figure 1.

PENNDOT	**DESIGN MANUAL, PART 4** **REVISION REQUEST**	**TO:** __________________________, **DIRECTOR** BUREAU OF DESIGN
REQUESTING ORGANIZATION/COMPANY	**DATE OF REQUEST**	**SIGNATURE OF PERSON REQUESTING CHANGE**
NAME OF PERSON TO CONTACT	**TELEPHONE NUMBER**	**ADDRESS**

IF YOU REQUIRE MORE SPACE THAN IS PROVIDED, USE SHEETS OF PLAIN PAPER THAT ARE THE SAME SIZE AS THESE SHEETS, NUMBER YOUR ANSWERS TO CORRESPOND WITH THE WAY THE ITEMS ARE NUMBERED BELOW.

1. TITLE, SECTION, NUMBER AND PAGE NUMBER OF THE EXISTING SPECIFICATION.

2. RECOMMENDED MODIFICATION AND THE CHAPTER(S) OR SECTION(S) INTO WHICH IT SHOULD BE INCORPORATED.

3. REASON AND/OR EXPLANATION FOR THE MODIFICATION.

Figure 1.2.2-1 - Revision Request Form

CHECKLIST FOR COMMENTS **COMMENTS**

DIVISIONS

☐ **HIGHWAY**

☐ **BRIDGE**

☐ **DESIGN SERVICES**

☐ **OTHER**

SHEETS MAY BE ATTACHED TO EXPLAIN WHY REQUEST WAS OR WAS NOT RECOMMENDED OR APPROVED.

☐ **RECOMMENDED** ☐ **NOT RECOMMENDED**	☐ **APPROVED** ☐ **DISAPPROVED**
DATE ______________________________	**DATE** ______________________________
______________________________ **SIGNATURE**	______________________________ **SIGNATURE**
DIRECTOR, BUREAU OF DESIGN/CHIEF BRIDGE ENGINEER	**DEPUTY SECRETARY OF HIGHWAY ADMINISTRATION/DIRECTOR, BUREAU OF DESIGN, OR CHIEF HIGHWAY ENGINEER**

Figure 1.2.2-1 - Revision Request Form (Continued)

1.2.3 Processing of Recommended Modifications

Upon receiving the proposed modification, the Director, Bureau of Design, will take the following action:

(a) The Director, Bureau of Design, will review the recommended modification and transmit copies to the various Bureau Directors and District Bridge Engineers involved for their comments.

(b) Upon receiving comments, the Director, Bureau of Design, will finalize the modification and take appropriate action, including obtaining comments by means of a Clearance Transmittal letter and securing FHWA approval if applicable.

(c) If modifications are accepted, the revised or added page(s) will be assigned a revision date which will be noted on the upper right-hand corner of the page. Revised pages will be distributed periodically.

(d) If the proposed modification is not accepted, the Director, Bureau of Design, will notify the originator of the reasons for rejecting it.

1.3 BRIDGE DESIGNS AND REVIEWS BY CONSULTANTS

The Quality Assurance Form for Engineering Agreements, in accordance with Appendix A, must be completed and included as a part of the technical proposal for an Engineering Agreement or as a part of Work Order under an Open-End Contract. This form shall be incorporated in an appendix along with the District's Scope of Work. A separate form must be completed for each structure included under the technical proposal or Work Order.

1.3.1 Scope of Work for New Designs

The scope of work for bridge projects shall contain the following bridge-related items as applicable:

(a) Hydraulic and hydrologic report, if applicable, and acquisition of all needed waterway-related permits, including permits for temporary stream crossing.

(b) Type, size and location studies, cost estimates, foundation exploration and recommendations, final design, and special provision preparation.

(c) Shop drawing review.

(d) Review of alternate design developed by contractors and Design-Build projects.

(e) Consultation during construction.

(f) Constructibility review by an independent consultant for all new major, unusual and complex bridges.

Constructibility is defined as the optimum use of construction knowledge and experience in planning, design, procurement and field operations to achieve overall project objectives and construction without any delay, structural integrity problems, or major claims.

The constructibility review should begin as early as the conceptual design stage (i.e., TS&L stage) to minimize redesign costs, project delays or the inclusion of undesirable details in the contract documents.

1.3.2 Scope of Work for Rehabilitation Projects

The following items shall be included as applicable:

(a) Deck condition survey, or bridge inspection and recommendations if the needed detailed information is not available from the regular NBIS report

(b) Petrographics for concrete

(c) Borings through piers and/or abutments to determine (1) bottom of footings, (2) material type and quality, (3) foundation materials, (4) foundation type, etc., if substructure units are to be reused

(d) Tests for steel to establish chemical composition, yield and ultimate strengths, as well as Charpy tests

(e) Fatigue damage analysis with possible strain gaging if decision on rehabilitation vs. replacement is involved

(f) Rating analysis

(g) Constructibility review by an independent consultant for all new major, unusual and complex bridge rehabilitation projects.

1.3.3 Selection of Design Methodology

The Department shall specify the design method (line girder method, two-dimensional grid analysis, or three-dimensional finite element design) to be used for the project.

For straight girder bridges with skews greater than 70º, a line girder method (with distribution factors as given in A4.6.2.2 and D4.6.2.2) shall be used (see PP3.2.2 for the Department's definition of skew angle).

1.3.4 Bridge Design Review by Consultants

The consultants may be asked to perform one of the following two different levels of review:

(a) Level 1 review (see PP1.3.4.1 for additional details) is applicable mostly to contractor-designed alternates and design-build projects of a complex nature or to those which incorporate "leading edge" technology. Such designs will often be beyond the scope of the criteria covered by the Design Manual and the Department's standards. The review consultants will be required to make a detailed review of the design to ensure that the Contractor's conceptual design approved by the Department is correctly developed and presented by the Contractor. This level of review will also be applicable to "leading edge" designs prepared by other consultants retained by the Department, where an independent review by another consultant would be in the best interest of the Department.

Figure 1, Figure 2 or Figure 3 shall be used for stamping the first sheet of the bridge plans reviewed by the review consultants.

(b) Level 2 review (see PP1.3.4.1 for additional details) is applicable to routine types of structures designed in conformance with Department criteria and standards. Review shall be limited to ensuring general conformance with the Department's design criteria and standards.

Figure 4 or Figure 5 shall be used for stamping the first sheet of the bridge plans reviewed by the review consultants.

The depth of review required of the review consultant shall be described in detail in the Engineering Agreement. When the level of review cannot be clearly determined for contractor-designed alternates and design-build projects, one of the two levels shall be assumed on the basis of the complexity of the as-designed bridge. The level of review will be changed, depending upon the alternate design proposed by the low bidder, either by the District Bridge Engineer or the Chief Bridge Engineer, according to approval responsibility. (Note that if a contractor-designed alternate or a design-build project converts a routine type of structure to a "leading edge" type, the Chief Bridge Engineer becomes responsible for approval.) Since most of the review work assigned to consultants is based upon a specific rate of pay, only the total cost of review would change, depending upon the level of review. When the level of review cannot be readily determined, the cost for both levels may be sought during the agreement stage.

Review may be assigned to consultants when work loads are such that review cannot be done in-house. Most consultant review assignments will be made for the review of contractor-designed alternates and design-build projects, but there may also be some assignments for the review of designs prepared by other consultants during the design phase.

Alternate Design

S x x x x A

PE Seal

Reviewed by:

____

Review Consultant's Name,
Signature and Date

The design review is a detailed review for compliance with the contract documents and for proper development and presentation of the Contractor's conceptual design approved by the Department. It is not intended to relieve the Contractor of full responsibility for accuracy and completeness of the plans or for complete compliance with the contract documents.

Figure 1.3.4-1 Level 1 Review - Applicable to Complex and "Leading-Edge" Contractor-Designed Alternates and Design-Build Projects

Design reviewed by:

PE Seal

____

Review Consultant's Name,
Signature and Date

The design review is a detailed review for proper development and presentation of the concepts in the type, size and location plans approved by the Department. It is not intended to relieve the designer of full responsibility for the proper development and presentation of the design and for the accuracy and completeness of the plans.

Figure 1.3.4-2 Level 1 Review - Applicable to Complex and "Leading-Edge" Designs Prepared by Other Consultants Retained by the Department

Design reviewed by:

PE Seal

____

Review Consultant's Name,
Signature and Date

The design review is a detailed review for compliance with the contract documents, the Department's design and construction criteria and standards, and for proper development and presentation of the concepts in the type, size and location plans approved by the Department. It is not intended to relieve the Contractor of full responsibility for the proper development and presentation of the design, the accuracy and completeness of the plans, or for complete compliance with the contract documents.

Figure 1.3.4-3 Level 1 Review - Applicable to Design-Build projects where the TS&L plan has been provided

Alternate Design S x x x x A PE Seal Reviewed by: ______________________________________ Review Consultant's Name, Signature and Date The design review is for general conformance with the contract documents and the Department's design and construction criteria and standards. It is not intended to relieve the Contractor of full responsibility for the accuracy and completeness of the plans or for complete compliance with the contract documents.

Figure 1.3.4-4 Level 2 Review - Applicable to Non-complex Contractor-Designed Alternates and Design-Build Projects Designed in Accordance with the Department's Criteria and Standards

Design reviewed by: PE Seal ______________________________________ Review Consultant's Name, Signature and Date The design review is for general conformance with the Department's design and construction criteria and standards and is not intended to relieve the designer of full responsibility for the accuracy and completeness of the plans.

Figure 1.3.4-5 Level 2 Review - Applicable to Non-complex Structures Designed by Other Consultants in Accordance with the Department's Criteria and Standards

1.3.4.1 REVIEW LEVELS

(a) Level 1 - Detailed review shall consist of the following, as applicable, and additional requirements which may be unique to a particular bridge:

(1) Evaluation of design methods and design assumptions.

(2) Evaluation of computer program used in design (or check of design using a different computer program acceptable to the Department).

(3) Check of manual calculations.

(4) Check of construction methods, including applicable safety regulations, when required, to ensure that the intent of the design can be realized.

(5) Check of erection stresses, where applicable.

(6) Check of plans to ensure that design information is adequately and correctly presented.

(7) Check of construction dimensions is not required, except as in (6) above unless specified in the engineering agreement.

(8) Quantity check is not required.

(9) Constructibility check is not required for contractor-designed alternates and Design-Build projects, except as noted in (4) above.

(10) Constructibility check is required for review of design prepared by the consultants retained by the Department.

(11) Review for cost-effectiveness when design is prepared by another consultant retained by the Department.

(12) Review for compliance with Department criteria and standards, as applicable.

(b) Level 2 Review for conformance to Department criteria and standards shall consist of the following, as applicable, and additional requirements which may be unique to a particular bridge:

(1) Review for compliance with Department criteria and standards.

(2) Review for constructibility and cost-effectiveness when design is prepared by a consultant retained by the Department.

(3) Check of design calculations only when required (will depend on quality of design, history of design consultant, etc.).

(4) Check of plans to ensure that design information is adequately and correctly shown.

(5) Check of construction dimensions and quantities are not required unless specified in the engineering agreement.

1.4 COMPUTER PROGRAMS

1.4.1 Modification, Acquisition, or Development of Programs

The software outlined in PP1.4.2 and PP1.4.7 shall be used for all PennDOT projects. If software for a particular application is not available from the Department, the designer may use other commercially available software with the approval of the Department. The designer is fully responsible for the entire design and analysis, regardless of the software used.

An Engineering District may request modification to the existing bridge engineering programs or acquisition or development of new engineering programs from the Director, Bureau of Design. Proper justification, benefits, etc. shall be incorporated in the request.

The Bureau of Design in coordination with the Bureau of Information Systems will have the software developed, acquired, or modified, conduct acceptance testing and provide or arrange for training, if needed.

1.4.2 Bridge Automated Design and Drafting System (BRADD)

The BRADD software was written as a tool for the LRFD design of simple span concrete, steel and P/S concrete bridges with or without sidewalks with span lengths ranging from 5500mm (18 ft.) to 60 000mm (200 ft.). The software supports tangent geometry, horizontal curves (chord beams) and vertical curves, with a bridge skew range from 25 degrees to 90 degrees. Available cross-section types are normal, symmetrical, superelevation, and superelevation transition, with a maximum of 8 design lanes or 20 beams in the cross section for a maximum width of 37 680mm (125.54 ft.).

1.4.2.1 DEPARTMENT USE

(a) Each Engineering District shall be responsible for designing bridges using BRADD on its CADD domain. The District Bridge Engineer shall be responsible for keeping a log of the projects for which BRADD is used and the savings realized by using BRADD.

(b) Any programming or technical errors found during use of the software shall be brought, in writing, to the attention of the Chief Bridge Engineer, Bureau of Design. Appropriate changes will be made by the BRADD Manager, Engineering Computing Management Division of the Bureau of Design, with the concurrence of the Chief Bridge Engineer. The Chief Bridge Engineer will notify all the Districts of corrections.

(c) Since BRADD can provide plans for more than one bridge type, the Districts are encouraged to develop designs for several of the most economical types of bridges. Cost analysis for different bridge types may be run on BRADD to select bridge types prior to generating the drafting files. This type of run shall be made if the designer is unsure of the most economical bridge types. Plans for more than one bridge type shall be provided whenever feasible in the bid document package to increase competition, thus reducing construction cost.

1.4.2.2 USE OF BRADD ON CONSULTANT DESIGNED STATE PROJECTS

The Department requires the use of BRADD on all single span bridge projects, including all single span PennDOT Bridge Projects (new single span bridge projects and all single span bridge superstructure replacement projects) unless justified by the designer.

There is a high degree of interest in utilizing BRADD for developing bridge plans for one span bridge replacement projects. However, BRADD can also be partially used for multiple span bridges since it can design and draft abutments for such bridges.

The District Bridge Engineer shall be responsible for keeping a record of the projects for which BRADD is used and the savings realized by using BRADD.

The BRADD Software, based on the Load and Resistance Factor Design method and working in both the SI (metric) and U.S. Customary units, is available only in a Windows XP based (Windows 2000 compatible) version on CD.

The Department will provide this software to consultants and any outside agencies for a fee, provided they sign a license agreement for its use. The Pennsylvania Department of Transportation is the only acquiring agent for this software. Anyone interested in obtaining a copy of the license agreement and fee schedule should contact the BRADD Manager, Engineering Computing Management Division, Bureau of Design at (717) 787-7057 or view information on the Department website at www.dot.state.pa.us.

The department will provide e-mail notification, e-notification, to individuals who register at http://BRADD.ENGRPROGRAMS.com. A company or organization can register as many individuals as desired. E-notifications will deliver timely non-policy issues in a relatively informal manner, such as: Problem reports, work-around solutions, upcoming version releases, answers to frequently asked questions and issues deemed current and relevant to proper implementation of BRADD.

1.4.2.3 USE OF BRADD ON NON-DEPARTMENTAL PENNSYLVANIA BRIDGE PROJECTS

The Department requires the use of BRADD on all non-Departmental Pennsylvania single span bridge projects funded in part by the Department unless justified by the designer. The Department encourages the use of BRADD on all other single span non-PennDOT Pennsylvania Bridge Projects (new single span projects and all single span bridge superstructure replacement projects).

There is a high degree of interest in utilizing BRADD for developing bridge plans for one-span bridge replacement projects. However, BRADD can also be partially used for multiple span bridges since it can design and draft abutments for such bridges.

The District Bridge Engineer shall be responsible for keeping a record of the projects reviewed and approved for which BRADD is used and the savings realized using BRADD.

The BRADD Software, based on the Load and Resistance Factor Design method and working in the SI (metric) and U.S. Customary units, is available only in a Windows XP based (Windows 2000 compatible) version on CD.

The Department will provide this system to consultants and any outside agencies for a fee, provided they sign a license agreement for its use. The Pennsylvania Department of Transportation is the only acquiring agent for this software. Anyone interested in obtaining a copy of the license agreement and fee schedule should contact the BRADD Manager, Engineering Computing Management Division, Bureau of Design at (717) 787-7057 or view information on the Department website at www.dot.state.pa.us.

The department will provide e-mail notification, e-notification, to individuals who register at http://BRADD.ENGRPROGRAMS.com. A company or organization can register as many individuals as desired. E-notifications will deliver timely non-policy issues in a relatively informal manner, such as: Problem reports, work-around solutions, upcoming version releases, answers to frequently asked questions and issues deemed current and relevant to proper implementation of BRADD.

1.4.3 CADD Cells for Bridge Details

The Engineering District may develop a standard bridge detail library (cells) for its own use. However, any detail that has statewide application shall be sent to the Director, Bureau of Design, for approval and permanent storage in the statewide cell library.

1.4.4 Bridge Engineering Software on PCs

If a District desires to develop or secure bridge engineering software (executables) for PC use, the software must be tested and accepted by the Bureau of Design prior to its use in bridge engineering.

1.4.5 Commercial Software

Commercially available or consultant-developed software shall be submitted to the Bureau of Design for review upon request. The submission shall include, as a minimum, design and/or analysis methodology, assumptions, capabilities, limitations, special instructions and comparison with Department-reviewed software. The Department will maintain and provide, upon request, a list of reviewed programs. Additional information for review, including software access if requested, shall be provided to ensure proper evaluation by the Department.

Appendix J contains a list of acceptable girder analysis programs for LFD design.

The Department has the discretion to either accept or reject the use of any commercially available or consultant-developed software proposed for use on any project. The Bureau of Design is available to help the Districts in this matter by providing guidance and recommendations when requested. In any and all cases, the design consultant is responsible for the accuracy of any and all computer software programs utilized on a project.

1.4.6 Computer Programs for Girder Bridges

Any computer program which has not been reviewed by the Department shall be submitted to, and approved by, the Chief Bridge Engineer prior to its use. A sample bridge(s) selected by the Department is to be modeled with the program so that the Department can make comparisons between its reviewed programs and the proposed program. The submission shall contain a description of how the program models the bridge, a discussion of any unique or special features of the program, input data sheets, output data sheets, a discussion of how the live load is treated, and a summary of the results, which should include (but not be limited to) the following:

(a) Dead load longitudinal moment diagram

(b) Positive and negative live load, plus impact longitudinal moment envelope

(c) Dead load shear diagram

(d) Positive and negative live load impact shear envelope

(e) Table of reactions

(f) Table of bottom flange lateral moments which occur at the maximum positive and negative longitudinal moments in each span of a curved bridge

(g) Table of diaphragm loads for curved and/or skewed bridges

This data shall not have load factors applied. The responsibility for obtaining approval of a computer program falls upon the person who has submitted it to the Bureau of Design.

1.4.7 PC Versions of the PennDOT Engineering Programs

The following PC versions of the PennDOT engineering programs, based on the general principle of engineering, will be available for use:

Program Name	Program Title
BSP	Beam Section Properties (dual units)
CAMBR	Field Check of Camber (dual units)
CBA	Continuous Beam Analysis (dual units)
CLLMR	Comparison of Live Load Moments and Reactions (dual units)
EngAsst	Engineering Assistant
GRPRO	Grade Profile (dual units)
HGEO	Horizontal Geometry
BRGEO	Bridge Geometry (dual units)

The following PC versions of the PennDOT engineering programs, based on the Load and Resistance Factor Design method and working in both the SI (metric) and U. S. Customary units, will be available for use:

Program Name	Program Title
ABLRFD	LRFD Abutment and Retaining Wall Analysis and Design
BPLRFD	LRFD Bearing Pad Design and Analysis
BXLRFD	LRFD Box Culvert Design and Rating
FBLRFD	LRFD Floorbeam Analysis and Rating
PAPIER	Pennsylvania Pier Analysis
PSLRFD	LRFD Prestressed Concrete Girder Design and Rating
SPLRFD	LRFD Steel Girder Splice Design and Analysis
STLRFD	LRFD Steel Girder Design and Rating
TRLRFD	LRFD Truss Analysis and Rating (not available yet)

The following PC versions of the PennDOT engineering programs, based on the Service Load and/or Load Factor Design method and work only in the Customary U. S. units, are available for use:

Program Name	Program Title
ABUT5	Abutment and Retaining Wall
ARCH	Arch Analysis and Design
BAR7	Bridge Analysis and Rating
BOX5	Box Culvert Design and Rating
PS3	Prestressed Concrete Girder Design and Rating
SIGN	Sign Structure Analysis

All programs are only available in a PC version running Microsoft Windows (NT, 2000, and XP) operating system. They are available on CD.

The Pennsylvania Department of Transportation is the only acquiring agent for these programs. The Department will provide these programs to consultants, local government (such as county and township) and educational institution for a fee per program, provided they sign a license agreement for the use of these programs. The Department will also provide these programs to federal or state agencies for free, provided they sign the license agreement for the use of these programs. Consultants, local government, and educational institution interested in obtaining a copy of the license agreement and program fees should view the information on the PennDOT Engineering Programs official website at http://penndot.engrprograms.com/home/. The forms for Software Update Request and Request for PennDOT's Engineering Software License can be downloaded from above web site.

The Department will provide e-mail notification, e-notification, to individuals who register at http://Penndot.engrprograms.com. A company or organization can register as many individuals as desired. E-notifications will deliver timely non-policy issues in a relatively informal manner, such as; computer program incident reports, work-around solution, upcoming program version deliveries, answers to frequently asked questions and issues deemed current and relevant to proper implementation of the Department's Engineering Software.

1.5 MAJOR, UNUSUAL OR COMPLEX BRIDGES

1.5.1 Definitions

Major bridges: Major bridges are defined as bridges estimated to cost $15 million or more. This criterion also applies to individual units of separated dual bridges.

Unusual bridges: An unusual bridge is one with difficult or unusual foundation problems, new or complex designs involving unusual structures or operational features, or bridges for which the design standards or criteria may not be applicable. Use of new products and experimental or demonstration projects are also considered as unusual structures.

Complex bridges: Complex bridges are stayed girder bridges, segmental bridges, any structure having a clear unsupported length in excess of 150 000 mm {500 ft.}, or bridges classified as complex by the Chief Bridge Engineer on the basis of TS&L or conceptual review.

1.5.2 Design Requirements

For 100% State funded projects, with the exception of interstate bridges, one complete design shall be developed for a bridge, using either steel or concrete or a combination of both, whichever is determined to be most economical at the type, size and location studies step.

The designer of major, unusual (including new products, experimental, or demonstration features), or complex structures, regardless of funding, shall include inspection and maintenance instructions as a part of the original design. Critical details and special inspection and maintenance requirements shall be stipulated on the plans. The plans shall also include details of access for inspection of the subject details.

For major, unusual (including new products, experimental, or demonstration features), or complex bridges, indicate on the plans or in special provisions the complete erection plan, detailed erection procedures with all needed survey controls, and all required computations.

1.5.3 Early Involvement of BQAD

If the District desires, the Bureau of Design Bridge Quality Assurance Division will provide its assistance in the consultant selection process for major, complex or unusual structures. For this purpose, a major bridge is defined as a bridge or a pair of bridges having an estimated construction cost of $15 million or more. The BQAD will assist in reviewing and commenting on the scope of work up to and including the final prioritization of the short-listed firms by the District.

Early involvement will be beneficial as the BQAD will gain familiarity with the project.

This assistance is voluntary.

1.6 PLAN PRESENTATION

The following shall generally apply to all Department projects. For non-Federally funded municipal projects, the governing municipality may have different requirements. Municipal requirements for Federally funded projects shall comply with stewardship and oversight agreement.

1.6.1 Drawings

Design drawings for highway structures shall be prepared on original reproducible drawings furnished by the Department, or on an approved equivalent Standard ANSI D size 863.6 mm x 558.8 mm (34” x 22”) with a border approximately 38.1mm {1 ½ in.} on the left and 12.7mm {1/2 in.} on the other three sides. A space shall be reserved at the bottom-right corner of each sheet for the title block. (For dimensions and other details see Table 1 and Figure 1.)

Table 1.6.1-1 - Bridge Drawings Block Dimensions

DESCRIPTION	WIDTH (mm)	HEIGHT (mm)
Title Block (Standards for Bridge Construction, Standards for Bridge Design)	200 {8”}	120 {4 3/4”}
Professional Engineer's Seal Block (and Prepared By)	150 {6”}	75 {3”}
Professional Engineer's Seal Block (block only)	75 {3”}*	75 {3”}*
Bridge Supplemental Drawing Block (6 mm {1/4”} line spacing)	200 {8”}	OPEN
Bridge Revision Block	190 {7 ½”}	OPEN

*For BRADD, general plan sheet block will be 60 mm x 75 mm {2 3/8” x 3”}. For all other BRADD sheets, the sheet block will be 60 mm x 60 mm {2 3/8” x 2 3/8”}.

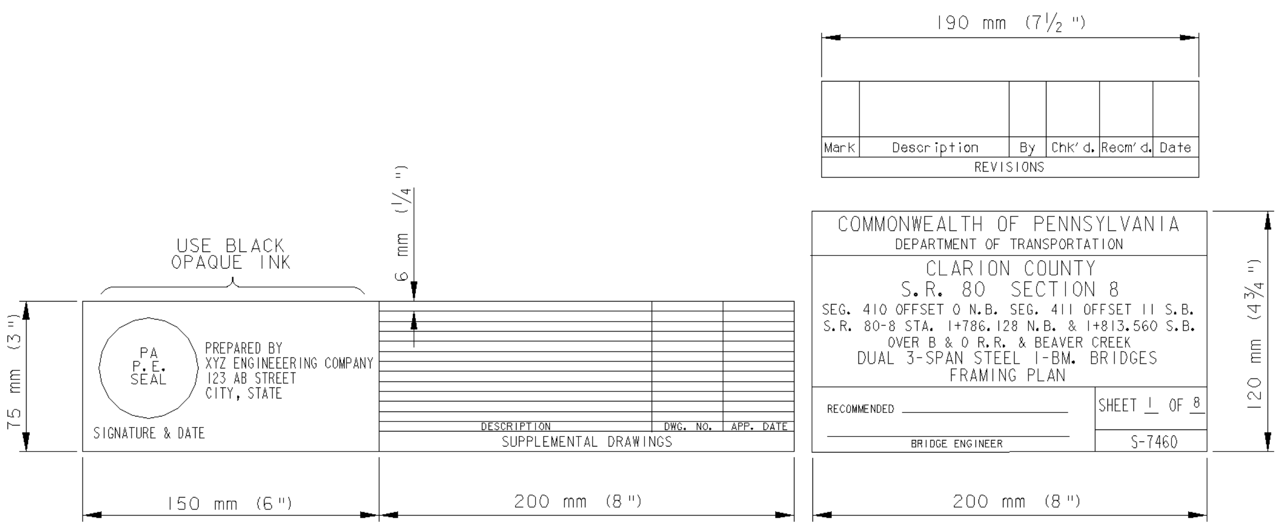


Figure 1.6.1-1 - Example of Dimensions and Details for Design Drawings

1.6.2 Title Blocks

Refer to Figure 1.

Mark	Description	By	Chk'd.	Recm'd.	Date
REVISIONS					

COMMONWEALTH OF PENNSYLVANIA
DEPARTMENT OF TRANSPORTATION

CLARION COUNTY
S.R. 80 SECTION 8
SEG. 410 OFFSET 0 N.B. SEG. 411 OFFSET 11 S.B.
S.R. 80-8 STA. 1+786.128 N.B. & 1+813.560 S.B.
OVER B & O R.R. & BEAVER CREEK
DUAL 3-SPAN STEEL I-BM. BRIDGES
FRAMING PLAN

RECOMMENDED ______________________

____
** BRIDGE ENGINEER

SHEET 1 OF 8
*

S-7460

*Show the following on the first sheet only:

+ SUPPLEMENTAL DRWGS.

**Show BRIDGE ENGINEER or CHIEF BRIDGE ENGINEER on the first sheet only.

Figure 1.6.2-1 - Example of Title Block

1.6.2.1 BASIC INFORMATION

The outline of the title block, dividing lines, and information basic to all projects will be reprinted on the sheets furnished by the Department or CADD generated sheets meeting the requirements of PP1.6.1. The other necessary information shall be added by the designer and shall be shown on each sheet of design drawings as applicable.

The county, route, section, segment, offset, station, features intersected, sheet number and S-number shall be shown in a manner similar to the example in Figure PP1.6.2-1.

The offset is to the beginning of the bridge, and the station shown should be at the approximate center of the bridge or at intersecting base lines as applicable. Segments and offsets for existing bridges shall be consistent with the BMS database.

1.6.2.2 DESCRIPTION OF STRUCTURE

The number of spans and a brief description of the bridge type (steel, prestressed concrete, etc.) shall be shown below the general information in the title block.

1.6.2.3 DESCRIPTION OF DRAWINGS

A brief description of what is contained on the sheet should be shown below the description of the structure for each sheet of drawings, e.g., General Plan, General Notes and Quantities, Abutment No. 1, Pier No. 3, Framing Plan Spans 10 and 11, etc.

1.6.2.4 SHEET NUMBERING

Each sheet of a set of drawings shall be numbered consecutively beginning with No. 1. Sheets shall be arranged in a logical (general plan, stake-out, substructure, superstructure, etc.) and orderly manner beginning with a general plan or an index sheet and ending with boring logs. Sheets closely related to one another shall be grouped together. Sheets that are added as a revision during construction shall bear the same number as the most closely related sheet, with an appropriate suffix (R1, R2, R3, etc.) added.

1.6.2.5 STRUCTURE PLAN NUMBER (S-number)

An automated S-numbering System, S-NUM (S-Number Update Method), is to be used by District personnel to provide a computerized method of securing and recording structure plan numbers (S-numbers).

The District is responsible to access the system and obtain the S-number. To access S-NUM, a TSO User ID Account is required and a system manual is available from BQAD.

BQAD does have the capability to modify S-NUM permanent records, should any misrepresentation of information occur, or if the District changes its mind on a project, or its time schedule, etc.

All structure plan numbers, whether they are designed by Consultant or District, will receive an S-number in the District Office.

Plans do not need an S-number until the Final Plan Stage, permitting time for the District to enter data. However, it is preferable to secure a number at the TS&L approval stage.

For Design-Build projects, the S-number shall be established for the Conceptual TS&L during preliminary design. Districts should contact the BQAD to update the S-NUM permanent records with any final structure information during final design.

Generally, there should be a separate set of drawings for each structure, except for dual structures and in special cases where it is more convenient to include more than one structure on a set of drawings. Rehabilitation or repair drawings, even if they are only sketches made on 297 mm x 210 mm (Metric Size A4) or 8 ½” x 11” (U.S. Customary Letter Size) sheets, shall have S-numbers assigned to them. When a structure has two designs in different materials for the purpose of encouraging competition, each design shall have a complete set of drawings and the same S-number, except for suffixes A and B, respectively, for each set. For a contractor-designed alternate, the S-number will be the same as the original design, except with suffix A added if there was no suffix on the original drawings, and suffix C added if there were two designs with suffixes A and B on the respective original drawings. The S-number will become part of the BMS database through regular input channels.

Structure Drawings with an S-number are required for all culverts having a clear span of 2400 mm {8 ft.} or more. If a multiple pipe, box, or arch culvert exists, the span length is the combined clear span opening.

Regardless of funding source, no S-number is required for local jurisdiction bridges. However, a BPAA (Bridge Plan Approval Application) number shall be obtained from the Bureau of Municipal Services in accordance with Publication 9, Appendix A, and shall be indicated on the bridge plans.

Sign structure plans do not require an S-number and do not get signed by the Bridge Engineer.
VMS/DMS sign structures require S-numbers and are to be signed by the Chief Bridge Engineer. For Title Blocks for sign structures and VMS/DMS sign structures, refer to Design Manual Part 3, Chapter 8.

Sound barrier and proprietary wall plans require an S-number. When contractor-designated alternate plans are submitted for proprietary walls, the S-number, shown on the original design of the proprietary wall, shall be suffixed by the letter P and should be shown on the alternate design plans.

Structure-mounted sound barrier shall be considered an integral part of the structure. Therefore, it will have the same S-number.

1.6.3 Special Requirements

1.6.3.1 ENGINEERING SEAL

For drawings prepared by a consultant, the structure drawings shall be prepared by, or under the direct supervision of, a professional engineer registered in Pennsylvania. The consultant's name and address together with the professional engineer's seal of the responsible professional engineer in charge of the design, his or her signature, and the date shall be shown in black opaque ink near the bottom-middle of the first sheet of each set of structure drawings for the final submission of the tracings to the Department.

For drawings prepared in-house, the structure drawings shall be prepared by, or under the direct supervision of, a professional engineer registered in Pennsylvania. The responsible Assistant District Bridge Engineer professional engineer's seal along with his or her signature and date shall be shown in black opaque ink near the bottom-middle of the first sheet of each set of structure drawings.

All other drawings in the set shall have professional engineer's seal, and it shall be located near the title block. This seal may be either from a black ink rubber stamp seal or a facsimile seal.

The responsible professional geologist or professional engineer shall sign and seal the core boring drawings.

1.6.3.2 SUPPLEMENTAL DRAWINGS

A table for supplemental drawings (standard drawings) shall be placed on the first sheet of each set of structure drawings immediately to the left of the title block whenever any standard drawings are referred to or are otherwise applicable to the construction of the structure. For BRADD the supplemental drawings block is placed in the top right corner of the Quantities sheet. The supplemental drawings shall be identified by drawing number, title and most recent recommendation date. (For an example, see Figure PP1.6.1-1.)

1.6.3.3 REVISIONS

A space shall be reserved immediately above the title block on all sheets of structure drawings for the purpose of listing revisions that are made to the drawings during construction. A blank block is provided for this purpose on the tracings furnished by the Department. Drawings shall be laid out so as not to encroach on this area. Changes made to drawings prior to advertisement for letting shall not be listed as revisions. Changes made to drawings after a project is awarded for construction, including changes that are made by flyers prior to letting, shall be made by crossing out information that is being voided and adding the new information nearby, marked with a revision symbol and number. A brief description of the revision shall be listed in the revision block, together with the initials of those making the revision and checking it, and the date. The District Bridge Engineer has the responsibility of seeing that revisions are properly made.

1.6.3.4 RELEASE OF INFORMATION AND DOCUMENTS TO NON-DEPARTMENT SOURCE

The decision to release Department information in response to request from outside the Department is to be made by the respective Bureau Director, District Executive who has control of the material. It is Department policy that BMS tapes, bridge inspection reports, RMS files and accident reports are not released. Other information such as Preliminary Design Reviews, QA Reports, Material Vendor files, Contractor Pre-Qualification Files, and material related to claims are typically not released.

The Department may release bridge inventory information (such as location, size, type, etc.) only as determined by the Office of Chief Council. The Districts are encouraged to allow public inspection by prospective bidders of specific information for a given construction project during the timeframe from advertisement by a Notice to Contractors to bid opening for the project.

This inspection should consist of the ability of the contractor to view, **but not copy**, information used by the Department to make their engineering decisions which will assist the prospective bidder in making an informed bid. This may consist of the information submitted for foundations in accordance with PP1.9.4.3, PP1.9.4.4 and PP1.9.4.5, and information submitted for the TS&L regarding constructability issues and/or alternatives analyses.

The Department may also release engineering design computations to the successful bidder to assist in developing alternate design or value engineering redesign.

1.6.4 General Requirements

1.6.4.1 LAYOUT

In the preparation of design drawings, every effort shall be made to draw the plans, sections, elevations and details accurately to scale. The scales shall be large enough to show clearly all dimensions and details necessary for construction of the structure. Preferably, plans, sections and elevations should be drawn to a scale not less than 1:50 {1/4" = 1'-0"} and details to a scale not less than 1:30 {3/8" = 1'-0"}. Drawings shall be laid out in such a way that all details fall within the prescribed border lines. All detail views shall be placed on the drawing so as to allow adequate space between them and shall be drawn large enough to be easily read when reduced photographically by 50%. Refer to Design Manual, Part 3, for requirements to provide bar scales. Ensure slope designations are appropriate to the measurement system used (e.g., U.S. Customary pile batter might be 3H:12V, whereas metric pile batter is designated 4V:1H).

1.6.4.2 LINEWORK AND LETTERING

All lines on the drawings shall be of sufficient density and width so as to have some residual density when reduced photographically by 50% and when microfilmed. Minimum height size for lettering, symbols and characters shall be 3 mm (1/8 in.). The metric LeRoy size of 30 {U. S. Customary LeRoy size of 120} should be used for text. All characters shall be open, bold, uniform and formed with a dense, but not wide line. Space between the letters shall be one-half the width of the widest letter, and space between the lines of lettering shall be one-half the height of the tallest letter. For CADD-generated drawings, see Design Manual, Part 3, Chapter 11.6.

1.6.4.3 CHECKING

Each sheet of design drawings shall be thoroughly checked and initialed by the designer and the checker before being submitted for Department review. The designer and checker must be two separate individuals to maintain proper quality control of information shown on each sheet.

1.6.4.4 NORTH ARROW

A north arrow symbol shall be placed on the General Plan and on all plan views of superstructure, substructure, or entire structure layout.

1.6.4.5 DIMENSIONING

Designers shall be particularly careful that sufficient overall and tie-in dimensions and geometric data are given on the plan. Tie-in and overall dimensions shall be arranged in such a way that it will not be necessary when reading the plans to add or deduct dimensions in order to determine the length, width, or height of any element of a structure. If "variable" dimensions are used, maximum and minimum values shall be provided.

1.6.4.6 DUPLICATION OF DETAILS

Showing of details or dimensions in more than one place shall be kept to a minimum. Such duplication is usually unnecessary and increases the risk of errors, particularly when revisions are made. However, such details shall be appropriately cross-referenced so that they may be easily found.

1.6.4.7 CROSS-REFERENCING

If, because of lack of space on a particular sheet or for convenience or other reasons, it is necessary to place a view or a section on another sheet or sheets, all such sheets shall be clearly cross-referenced by adding the appropriate sheet number after the section or detail designation, e.g., Section A-A (Sh __); Detail B (Sh __). Notes shall be used only if absolutely necessary.

1.6.4.8 PAY LIMITS

When misinterpretation is possible, the limits of pay items shall be clearly indicated on the corresponding details of a structure.

1.6.4.9 ABBREVIATIONS

Abbreviations of words shall generally be avoided, and those abbreviations which are not in common use shall be explained in a legend.

1.6.4.10 REINFORCEMENT BAR SCHEDULES

Bar marks should not be repeated. For bar marks that cover varying lengths of bar, the minimum and maximum lengths of bar shall be denoted in the schedules, along with the varying distance per number of bars. For example, S601, 9'-0" to 12'-0", vary 2 EA. by 6".

1.6.4.11 GENERAL PLAN SHEET

The following essential information shall be shown on the first sheet, which is designated as "General Plan". If all of the following items cannot be accommodated on the first sheet, they may be shown on the next or succeeding sheets with proper reference.

(a) Plan

Outlines of substructure above ground and superstructure; length of spans along profile grade of roadway, skew angle(s), stations and grade elevations at intersections of profile grade with centerline bearing at abutment and centerlines at piers; stations at end and beginning of structure (see Figure 1), piers, abutments and wingwall designation (e.g., Pier 5, Abutment 1, Wingwall A); horizontal distance between profile grade lines in the case of dual structures; contours for existing and final groundlines; location of points of minimum actual and required vertical clearances, scuppers, and lighting poles; minimum actual and required horizontal clearances between underpassing highways or railroad tracks and faces of adjacent parts of substructure, and normal horizontal clearances between faces of substructure for drainage structures.

(b) Elevation

Rate and direction of roadway grade, spacing of railing posts, spacing and mounting heights of lighting poles, protective fence location, finished groundline and approximate original groundline along construction centerline of bridge, bottom of footing elevations, estimated pile tip elevations, and required and provided minimum vertical clearances together with the elevations that define the clearances provided. Show the type of joint and a movement classification for each joint on the plans.

For definition and requirements for highway vertical clearance, see D2.3.3.2. For drainage structures, the minimum vertical clearance is the maximum unobstructed design flood flow depth under a bridge.

(c) Typical normal section(s) of superstructure

Roadway width between curbs or sidewalks, overall dimensions, out-to-out of outside faces of barriers, water tables, cross slopes of roadway, minimum slab thickness, girder spacing, girder type, girder size and overhang. All applicable cross-sections shall be shown on the general plan.

(d) Grade data

Horizontal and vertical alignment data, superelevation, run-in/run-out data, points of rotation, and provisions for spirals in accordance with Design Manual, Part 2.

(e) Deck elevations

On structures in which lanes converge or diverge, on structures with transition in superelevation or small skew angles (less than 75 degrees), or on curves or complicated layout, one or all of the following shall be shown at the direction of the District:

(1) Table showing elevations at gutter and breaks in cross slope of the deck at 3000 mm {10 ft.} intervals

(2) Contour lines with framing plan superimposed

(3) Finished deck grade over the centerline of each girder at every tenth point starting from near centerline of the bearing of each girder to the far centerline of bearing.

(f) Summary of quantities

Pay item numbers and units shall be in accordance with the Department Specifications, Publication 408, or as specified and described in the Special Provision. For a list of common pay items, see the Construction Items Catalog (Publication 7M). When two or more structures (such as dual bridges) are presented on the same plan, the quantities shall be tabulated for each structure. For presentation format see Figure 2. When a lump sum item is used, an approximate quantity shall be shown for the item.

All "AND Items", such as reinforcement, piles and pile tip reinforcements items, need not be repeated as shown in Figure 2 for BRADD projects. However, all "AND Items", except reinforcement bars and epoxy-coated reinforcement bars shall be repeated on the summary of quantities sheet (Plate B-II in Chapter 15 of Design Manual, Part 3) for each

Contractor designed alternate option with the quantities to be inserted by the Contractor.

Design-build projects shall have pay items in accordance with PP1.11.2.2.4 and the Standard Special Provisions. The bid item for design is an individual pay item. The bid item for construction will be an "EITHER OR" format to distinguish material types. Refer to Figure 1.6.4.11-3 for sample tabulations.

(g) Index of sheets

Bridge sheet titles and numbers shall be listed when five or more drawing sheets are used.

(h) General notes (For a list of general notes, see PP1.7.)

(i) Hydraulic data (see PP7.1.2.)

For detailed submission items for each bridge submission, see PP1.9.

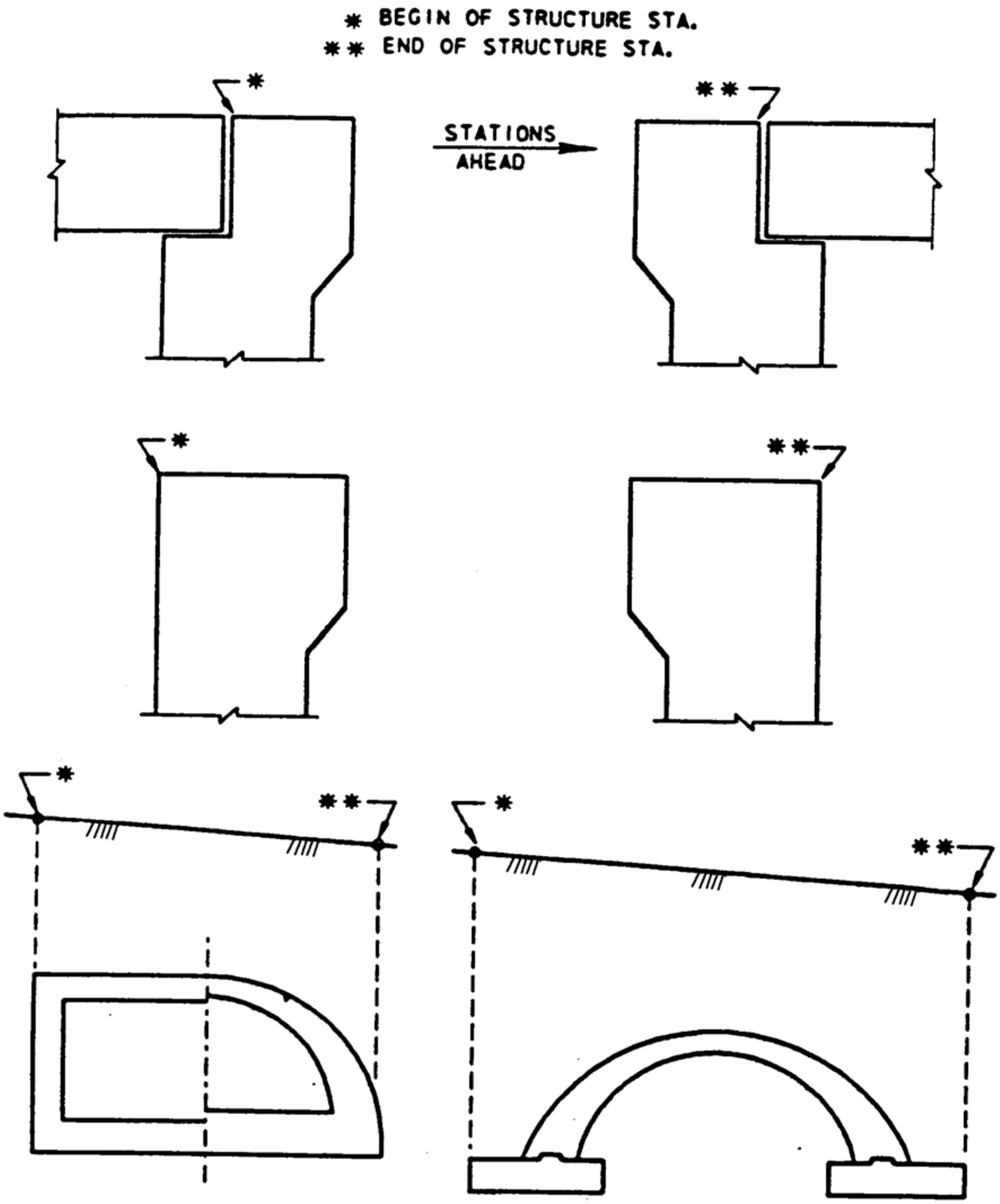


Figure 1.6.4.11-1 - Sections at End and Beginning of Structure

ALTERNATE STRUCTURE ITEMS			
ITEM NO.	ITEM	UNIT	TOTAL
8030-0001	BRIDGE STRUCTURE, AS DESIGNED, S-XXXXX	LS	LUMP SUM
8000-0001	PRESTRESSED CONCRETE BRIDGE STRUCTURE	LS	LUMP SUM
8100-0001	STEEL BRIDGE STRUCTURE	LS	LUMP SUM

APPROXIMATE QUANTITIES - BRIDGE STRUCTURE, AS DESIGNED						
ITEM NO.	ITEM	UNIT (11)	ABUT. 1	ABUT. 2	SUPERSTRUCTURE	TOTAL
8030-0001	BRIDGE STRUCTURE, AS DESIGNED, S-XXXXX	LS				LS
(1)	CLASS 3 EXCAVATION	M3	0	6	---	6
(1)	CLASS AAA CEMENT CONCRETE	M3	---	---	123 (2)	123
(1)	CLASS AA CEMENT CONCRETE	M3	17 (3)	3 (3)	34 (4)	54 (5)
(1)	CLASS A CEMENT CONCRETE	M3	137	79	---	216
(1)	PROTECTIVE COATING FOR REINFORCING CONCRETE SURFACES	M2	---	---	577	577
(1)	SELECTED BORROW EXCAVATION, STRUCTURE BACKFILL	M3	0	462	---	462
(1)	NO. 57 COARSE AGGREGATE	M3	3	3	---	6
(1)	PRESTRESSED CONCRETE BRIDGE BEAMS 715 X 1825	M	---	---	281	281
(1)	ARMORED NEOPRENE STRIP SEAL	M	11	---	---	11
(1)	STEEL BEAM TEST PILES HP 310 X 79 (6)	---	LS (7)	LS (7)	---	---
(1)	GEOTEXTILE, CLASS 2, TYPE B	M2	73	111	---	184
(1)	152 mm STRUCTURE FOUNDATION DRAIN	M	22	41	---	63
(1)	PROTECTIVE COATING FOR REINFORCED CONCRETE SUBSTRUCTURE	M2	102	103	---	205
AND						
3002-0001 (9)	REINFORCEMENT BARS	KG	7518	3947	0	11 465
AND						
3002-0053 (9)	REINFORCEMENT BARS, EPOXY-COATED	KG	853	206	10 281 (8)	11 340
AND						

APPROXIMATE QUANTITIES - BRIDGE STRUCTURE, AS DESIGNED						
ITEM NO.	ITEM	UNIT (11)	ABUT. 1	ABUT. 2	SUPERSTRUCTURE	TOTAL
3005-1103 (10)	STEEL BEAM BEARING PILES, HP 310 x 79	M	288	180	---	468
AND						
3005-1153 (10)	STEEL BEAM PILE TIP REINFORCEMENT, HP 310 X 79	EA	42	24	---	66
3090-0332	EPOXY INJECTION CRACK REPAIR FOR BRIDGE DECKS	M			14	14
3090-0350	CONCRETE BRIDGE DECK CRACK SEALER	M2			84	84

(1) ITEMS IN BRIDGE STRUCTURE LUMP SUM ITEM 8030-0001 GIVEN FOR INFORMATION ONLY
(2) INCLUDES CLASS AAA CONCRETE IN DECK SLAB AND APPROXIMATELY __*__ CUBIC METERS OF CLASS AAA CONCRETE TO ACCOUNT FOR STAY-IN-PLACE FORM TROUGHS
(3) INCLUDES CLASS AA CONCRETE IN SHEAR BLOCKS, ABUTMENT BACKWALLS, CHEEKWALLS AND UWINGS ABOVE THE HORIZONTAL CONSTRUCTION JOINT NEAR THE BRIDGE SEAT
(4) INCLUDES CLASS AA CONCRETE IN CURBS, BARRIERS, SIDEWALKS, DIVISORS AND CONCRETE DIAPHRAGMS
(5) QUANTITY TO BE USED FOR CLASS AA CONCRETE UNDER THE DECK COLUMN HEADING ON THE STRUCTURE COST DATA FORM
(6) IF PILE TIP REINFORCEMENT IS REQUIRED, SHOW FOLLOWING NOTE ON DRAWING:
    INCLUDE _*_ PILE TIP REINFORCEMENT
    (*SHOW NUMBER OF PILE TIP REINFORCEMENT REQUIRED)
(7) SHOW NUMBER AND LENGTH OF TEST PILES ON THE DRAWING (i.e., 2 @ 14 m, 1 @ 14 m or 1 @ 9 m)
(8) INCLUDES _*_ KG OF EPOXY-COATED DOWELS
    (*SHOW QUANTITY OF EPOXY-COATED DOWELS)
(9) FOR AS DESIGNED STRUCTURE, INCLUDED IN BRIDGE BID ITEMS. FOR ALTERNATE DESIGNS, INCLUDED IN BRIDGE STRUCTURE LUMP SUM BID ITEM
(10) INCLUDED IN BRIDGE BID ITEMS
(11) UNIT DESIGNATIONS ARE AS PER STANDARD ITEMS CATALOGUE

Figure 1.6.4.11-2 - Tabulation of Bridge Bid Items and Approximate Quantities

ALTERNATE STRUCTURE ITEMS			
ITEM NO.	ITEM	UNIT	TOTAL
8030-0001	BRIDGE STRUCTURE, AS DESIGNED, S-XXXXX	LS	LUMP SUM
8000-0001	PRESTRESSED CONCRETE BRIDGE STRUCTURE	LS	LUMP SUM
8100-0001	STEEL BRIDGE STRUCTURE	LS	LUMP SUM

APPROXIMATE QUANTITIES - BRIDGE STRUCTURE, AS DESIGNED						
ITEM NO.	ITEM	UNIT	ABUT. 1	ABUT. 2	SUPERSTRUCTURE	TOTAL
8030-0001	BRIDGE STRUCTURE, AS DESIGNED, S-XXXXX	LS				LS
(1)	CLASS 3 EXCAVATION	CY	0	7	---	7
(1)	CLASS AAA CEMENT CONCRETE	CY	---	---	161 (2)	161
(1)	CLASS AA CEMENT CONCRETE	CY	22 (3)	4 (3)	44 (4)	70 (5)
(1)	CLASS A CEMENT CONCRETE	CY	179	103	---	282
(1)	PROTECTIVE COATING FOR REINFORCING CONCRETE SURFACES	SY	---	---	690	690
(1)	SELECTED BORROW EXCAVATION, STRUCTURE BACKFILL	CY	0	604	---	604
(1)	NO. 57 COARSE AGGREGATE	CY	3	3	---	6
(1)	PRESTRESSED CONCRETE BRIDGE BEAMS 28” x 73”	LF	---	---	921	921
(1)	ARMORED NEOPRENE STRIP SEAL	LF	35	---	---	35
(1)	STEEL BEAM TEST PILES HP 12” x 53” (6)	---	LS (7)	LS (7)	---	---
(1)	GEOTEXTILE, CLASS 2, TYPE B	SY	87	132	---	219
(1)	6" STRUCTURE FOUNDATION DRAIN	LF	70	135	---	205
(1)	PROTECTIVE COATING FOR REINFORCED CONCRETE SUBSTRUCTURE	SY	122	123	---	245
AND						
1002-0001 (9)	REINFORCEMENT BARS	LB	16,574	8,701	0	25,275
AND						

APPROXIMATE QUANTITIES - BRIDGE STRUCTURE, AS DESIGNED						
ITEM NO.	ITEM	UNIT	ABUT. 1	ABUT. 2	SUPERSTRUCTURE	TOTAL
1002-0053 (9)	REINFORCEMENT BARS, EPOXY-COATED	LB	1,879	454	22,666 (8)	24,999
AND						
1005-1103 (10)	STEEL BEAM BEARING PILES, HP 12” x 53”	LF	943	590	---	1,533
AND						
1005-1153 (10)	STEEL BEAM PILE TIP REINFORCEMENT,HP 12” x 53”	EA	42	24	---	66
1090-0332	EPOXY INJECTION CRACK REPAIR FOR BRIDGE DECKS	LF			45	45
1090-0350	CONCRETE BRIDGE DECK CRACK SEALER	SY			100	100

(1) ITEMS IN BRIDGE STRUCTURE LUMP SUM ITEM 8030-0001 GIVEN FOR INFORMATION ONLY
(2) INCLUDES CLASS AAA CONCRETE IN DECK SLAB AND APPROXIMATELY _*_ CUBIC YARDS OF CLASS AAA CONCRETE TO ACCOUNT FOR STAY-IN-PLACE FORM TROUGHS
(3) INCLUDES CLASS AA CONCRETE IN SHEAR BLOCKS, ABUTMENT BACKWALLS, CHEEKWALLS AND UWINGS ABOVE THE HORIZONTAL CONSTRUCTION JOINT NEAR THE BRIDGE SEAT
(4) INCLUDES CLASS AA CONCRETE IN CURBS, BARRIERS, SIDEWALKS, DIVISORS AND CONCRETE DIAPHRAGMS
(5) QUANTITY TO BE USED FOR CLASS AA CONCRETE UNDER THE DECK COLUMN HEADING ON THE STRUCTURE COST DATA FORM
(6) IF PILE TIP REINFORCEMENT IS REQUIRED, SHOW FOLLOWING NOTE ON DRAWING:
    INCLUDE _*_ PILE TIP REINFORCEMENT
    (*SHOW NUMBER OF PILE TIP REINFORCEMENT REQUIRED)
(7) SHOW NUMBER AND LENGTH OF TEST PILES ON THE DRAWING (i.e., 2 @ 45 ft., 1 @ 45 ft. or 1 @ 30 ft.)
(8) INCLUDES _*_ LBS. OF EPOXY-COATED DOWELS
    (*SHOW QUANTITY OF EPOXY-COATED DOWELS)
(9) FOR AS DESIGNED STRUCTURE, INCLUDED IN BRIDGE BID ITEMS. FOR ALTERNATE DESIGNS, INCLUDED IN BRIDGE STRUCTURE LUMP SUM BID ITEM
(10) INCLUDED IN BRIDGE BID ITEMS

Figure 1.6.4.11-2 - Tabulation of Bridge Bid Items and Approximate Quantities (Continued)

TABULATION OF STRUCTURE ITEMS			
ITEM NO.	ITEM	UNIT	TOTAL
8210-0001	DESIGN OF BRIDGE STRUCTURE (AS-DESIGNED FOUNDATION PROVIDED), S-XXXXX	LS	LUMP SUM
EITHER			
8250-0001	CONSTRUCTION OF PRESTRESSED CONCRETE STRUCTURE, S-XXXXX	LS	LUMP SUM
OR			
8251-0001	CONSTRUCTION OF STEEL STRUCTURE, S-XXXXX	LS	LUMP SUM
1090-0332	EPOXY INJECTION CRACK REPAIR FOR BRIDGE DECKS	LF	45

TABULATION OF STRUCTURE ITEMS			
ITEM NO.	ITEM	UNIT	TOTAL
8213-0001	DESIGN OF RETAINING WALL (NO AS-DESIGNED FOUNDATION PROVIDED), S-XXXXX	LS	LUMP SUM
EITHER			
8255-0001	CONSTRUCTION OF CONCRETE RETAINING WALL, S-XXXXX	LS	LUMP SUM
OR			
8256-0001	CONSTRUCTION OF PREFABRICATED RETAINING WALL, S-XXXXX	LS	LUMP SUM

TABULATION OF STRUCTURE ITEMS			
ITEM NO.	ITEM	UNIT	TOTAL
8216-0001	DESIGN OF REHABILITATION AND/OR WIDENING (AS-DESIGNED FOUNDATION PROVIDED), S-XXXXX	LS	LUMP SUM
8260-0001	CONSTRUCTION OF REHABILITATION AND/OR WIDENING, S-XXXXX	LS	LUMP SUM
1090-0332	EPOXY INJECTION CRACK REPAIR FOR BRIDGE DECKS	LF	55

Figure 1.6.4.11-3 - Tabulation of Structure Bid Items for Design-Build Projects

1.6.5 Stake-Out Sketch

(a) A stake-out sketch shall be shown, preferably on the first or second sheet of the structure drawings. There should be ample open space outside of the sketch to allow wing and barrier line extensions for stake point recordings. The sketch need not be to scale. Frequently, exaggerations of curvature, angle, etc., are necessary to show the information clearly.

(b) The sketch shall be as simple as possible, but as complete as possible so that the structures will be constructed according to the plans.

(c) All necessary tie-in dimensions between highway alignment, working points, lines of structure, and other control points shall be shown in millimeters {feet to two decimal places} on the sketch.

1.6.5.1 STAKE-OUT GUIDELINES

(a) The stake-out shall be referenced to one straight base line, except in the case of dual structures, where two straight base lines, properly referenced to each other, can be used. The base line will be the centerline of the highway (if tangent), or the long chord connecting the points where the centerline of the highway intersects the face of the abutments on a curved highway, or the tangent line at the point of intersection of highways or the highway and a stream or river. Generally, dimensioning along the long chord is preferred on sketches for viaducts with a long series of spans. In special situations, some other base line can be used if particularly convenient.

(b) The sketch shall show the base line and the shape of the exterior face of the substructure (abutments and wingwalls). All corners shall be referenced by showing work points and distances to the base line. Wingwall angles to the front face of abutments shall also be referenced. Work point coordinates may be shown on the plan.

(c) At intermediate piers, the skew angle between the centerline of the pier and the base line is required. The location of the intersection of pier centerline with base line shall be tied to other parts of the substructure by base line dimensions. The distance from the base line to the centerline of roadway along the centerline of the pier shall be given. The station of the intersection points at the base line shall be shown. Distances between the outside faces of each barrier shall be shown.

(d) For multi-level structures, each level shall be sketched separately, but referenced to the same base line.

(e) The stake-outs for box culverts shall include inside faces of walls, ends of the culvert, and the front face of the wingwalls. Reinforced concrete arch culverts and metal culverts shall be treated similarly.

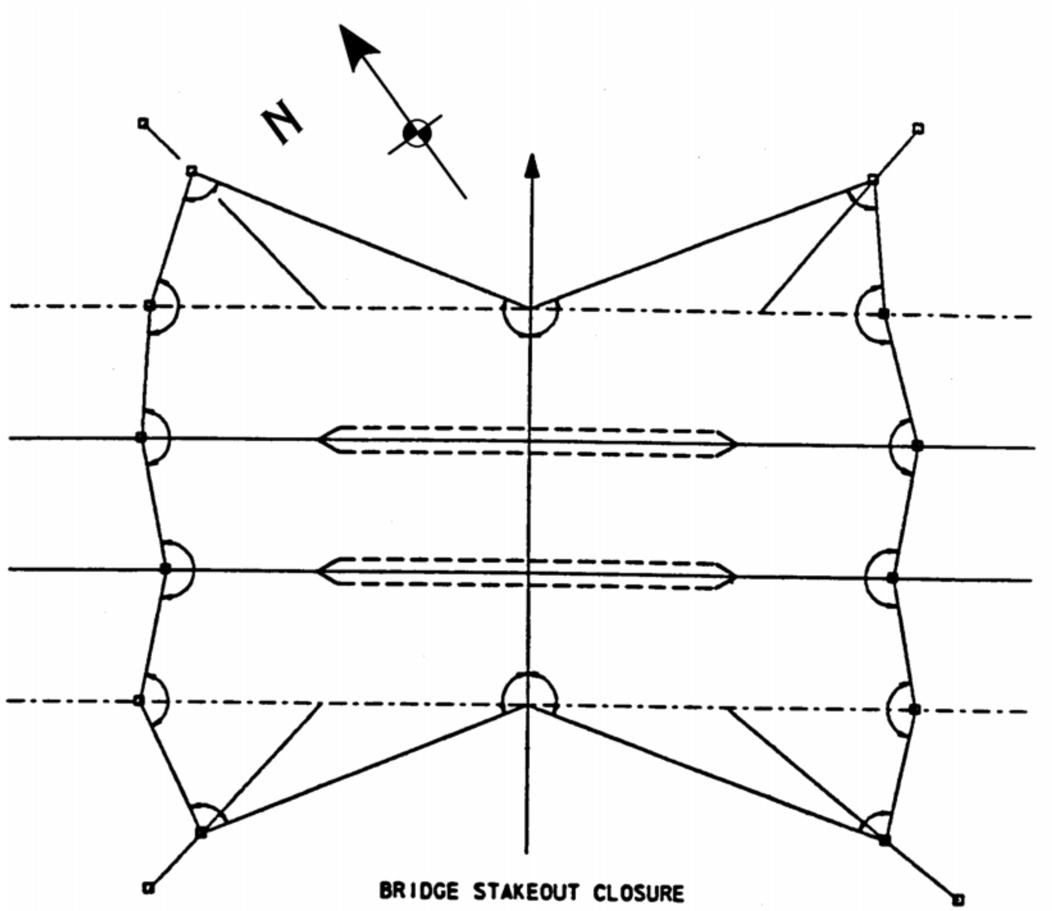
1.6.5.2 PROCEDURE TO ENSURE AGAINST DISCREPANCIES IN BRIDGE STAKE-OUT

(a) The structure stake-out sketch and reference stake locations shall be recorded in a Department survey notebook.

(b) Original stake-out field notes shall be recorded in a survey notebook. When other copies are required, this information shall be taken from the original survey notebook.

(c) An error of closure on the stake-out shall be recorded in the survey notebook. This error of closure shall reflect a comparison between measured and computed angles and distances of a traverse around the near line of the offset stakes of all working lines and shall meet the minimum error of closure of one part in 10 000 (see Figure 1).

(d) A complete centerline tie shall be made at the ends of a structure to ensure proper location.



Run a closed traverse around the near line of offset stakes of all working lines. On projects where various phase stakeout is required, run the intermediate traverse as the stake-out progresses.

Upon completion of an acceptable closure of the perimeter traverse of 1 part in 10 000, compute the individual closures between each pier of piers and abutment lines. These computations may be made in the field by the party Chief or in the office by a member of the District Bridge Unit.

Record this sketch and closure in a survey field book to be made readily available for inspection.

Figure 1.6.5.2-1 - Bridge Stake-Out Closure

1.7 GENERAL NOTES

From the following list, notes related to the type of structure shall be shown on the general plan of the drawings. The notes applicable to specific elements of a structure may be shown on the corresponding sheets. Special notes shall be written, or these notes revised, to meet special conditions on individual projects. Use the imperative mood in writing general notes on the bridge plans.

1.7.1 Design Specifications

1. AASHTO LRFD Bridge Design Specifications*, and as supplemented by Design Manual, Part 4 (including latest revisions**). (Also, include applicable specifications as indicated in PP1.1.1.)

   *specify year, i.e., 2004 **specify month and year

2. Live load distribution to girders is based upon ___ method (see PP1.7.10, Instruction 1).

3. Design is in accordance with the LRFD method.

1.7.2 Design Live Loads

1. PHL-93 or P-82 (910 kN {204 kip} permit load)

2. Fatigue design is based on the following:

   Steel structures: ADTT ___ (year) one-directional)

   Prestressed concrete: ADTT ___ (year) one-directional)

   Maximum allowable tensile stress in precompressed tensile zone:

   Metric Units: $0.25\sqrt{f'_c}$

   U.S. Customary Units: $0.0948\sqrt{f'_c}$

1.7.3 Dead Loads

1. Includes surface area density of 150 kg/m^2 {0.030 ksf} for future wearing surface on the deck slab (see PP1.7.10, Instruction 2).

2. Includes a surface area density of 75 kg/m^2 {0.015 ksf} for permanent metal deck forms which takes into account the weight of the form, plus the weight of the concrete in the valleys of the forms (see PP1.7.10, Instruction 3).

3. Includes ___ kg/m {kip/ft} for utilities, ___ kg/m {kip/ft} for inspection walk and ___ kg/m {kip/ft} for sound barrier.

1.7.4 General

1. Provide materials and perform work in accordance with Specifications, Publication 408*, AASHTO/AWS D1.5M/D1.5 ** Bridge Welding Code, and contract special provisions.

   *Specify year, i.e., 2003
   **Specify applicable year, i.e., 2002 (Use AASHTO/AWS D1.1/D1.1M:2002 for welding not covered in AASHTO/AWS D1.5M/D1.5: 2002). Designers to verify date to be consistent with Pub 408 1105.03 (m).

2. Provide structural steel conforming to AASHTO ***(ASTM ****) designation, except when noted otherwise.

***Specify applicable number, i.e., M 270/M 270M
****Specify ASTM designation, i.e., A 709/A 709M

3. Provide 50 mm {2 in.} concrete cover on reinforcement bars, except as noted.

4. Use Class AAA cement concrete in deck slab, precast channel beams, sidewalks and top slab of concrete box culverts at grade.

   Use Class AA cement concrete in curbs, barriers, divisors, concrete diaphragms, abutment backwalls, cheek walls, shear blocks, U-wings above bridge seat construction joint, footings (when specified) and sound barriers.

   Use Class A cement concrete in piers, abutments below bridge seat, pedestals, wingwalls, retaining walls, footings, arch culverts, spandrel walls, walls and top and bottom slabs of box culverts under fill, and walls and bottom slab of box culverts at grade.

   Use Class C cement concrete below the bottom of footings when specified.

5. A higher class concrete may be substituted for a lower class concrete at no additional cost to the Department.

6. Provide Grade 420 {Grade 60} reinforcing steel bars that meet the requirements of ASTM A 615/A 615M, A 996/A 996M or A 706/A 706M. Do not weld Grade 420 {Grade 60}reinforcing steel bars unless specified. Grade 300 {Grade 40} reinforcing steel bars may be substituted with a proportional increase in cross-sectional area, if approved by the Chief Bridge Engineer. Do not use rail steel A 996/A 996M reinforcement bars in bridge piers, abutments, shear blocks, beams, footings, piles, barriers or where bending or welding of the reinforcement bars is indicated.

7. Use epoxy-coated reinforcement bars in the deck slab, barriers, sidewalk, top flange of non-composite adjacent prestressed box beams, abutment backwalls, Uwings above the construction joint, stirrups protruding from diaphragms and prestressed beams into the deck slab, and pier and abutment seat bars where expansion dams are used. Also epoxy-coat substructure reinforcement bars as indicated.

8. Galvanized reinforcing steel bars may be substituted for epoxy-coated reinforcing steel bars at no additional cost to the Department.

9. Rake-finish all horizontal construction joints, except as indicated.

10. Seismic forces were considered for acceleration coefficient of _*_. (*Insert either 0.05, 0.09 or 0.15)

11. Use retarder admixture conforming to Publication 408 (specify year) in the concrete deck slab (see PP1.7.10, Instruction 4).

12. Verify all dimensions and geometry of the existing structure in the field as necessary for proper fit of the proposed construction (see PP1.7.10, Instruction 5).

13. Construct deck slab transverse construction joints parallel to bridge centerline of bearings.

14. Abutment backwalls may be placed up to a construction joint below the level of the bottom of deck slab prior to construction of the deck.

15. Notify the regional headquarters of the Fish Commission prior to construction and cooperate with Fish Commission during construction. (Include name, address and telephone number for Waterway Conservation Officers)

16. Place cheekwall, concrete shear blocks, and backwall concrete after beams are set in position.

17. Chamfer exposed concrete edges 25 mm by 25 mm {1 in. by 1 in.}, except as noted.

18. All dimensions shown are horizontal, except as noted.

19. Use either permanent metal forms or removable forms to construct the deck slab. (Specify type of forms if required by design conditions.)

20. Deck slab thickness includes a 10 mm {½ in.} integral wearing surface.

21 Superstructure dimensions shown are for a normal temperature of 20º C {68º F}.

22. Before driving piles, place and compact, to footing elevation, specially selected material which contains no rock to interfere with pile driving. Auguring or pre-boring will be permitted to the original ground. (Note to be used if footing is above existing groundline.)

23. Spread footings may be ordered by the Engineer to be at any elevation or of any dimensions necessary to provide a proper foundation. (Not applicable for footing set on piles.)

24. Use corrosion inhibiting admixture in the concrete deck slab (see PP1.7.10, Instruction 8).

1.7.4.1 NOTES FOR PROPRIETARY WALLS (For plans prepared by a proprietor for the Contractor)

1. These drawings are intended for use only at the site for which they are prepared. *__________ disclaims any liability for any other use (*write company name).

2. For additional design information, core borings and other geotechnical information not shown on these plans, refer to the original design plans, S-number______.

3. All design assumptions are validated through either notes to the Contractor or details on these drawings.

1.7.4.2 NOTES FOR BRIDGE REHABILITATION PLANS

1. Do not consider any of the data on the existing structure supplied in the original design drawings or made available to you by the Department or its authorized agents as positive representations of any of the conditions that you will encounter in the field.

2. The information shown on the plans for the existing bridge is not part of the plans, proposal, or contract and is not to be considered a basis for computation of the unit prices used for bidding purposes. There is no expressed or implied agreement that information is correctly shown. The bidder is not to rely on this information, but is to assume the possibility that conditions affecting the cost and/or quantities of work to be performed may differ from those indicated. (List the original design drawings.)

3. The following standard note should appear on each of the existing bridge plans which are to be supplied to the contractors:

   Bidders are advised to field-verify information presented. The data shown herein is not a part of the plans, proposal, or contract, and is not to be considered as a basis for computation for any purpose.

4. After obtaining the laboratory tests results, the following standard note should appear on all bridge painting, demolition, and rehabilitation projects:

   *The existing bridge structural members contain (Or do not contain) lead paint and other toxic materials {such as cadmium, chromium, arsenic, etc.(*Show this detail based on the laboratory test results*)} based on laboratory testing.*

1.7.5 Notes for Piles

1.7.5.1 NOTES FOR PILE DRIVING REQUIREMENTS

The applicable driving method shall be specified in the foundation submission and shown in the general notes on the design drawings. Driving shall be controlled by the wave equation unless the use of the dynamic formula in Publication 408, Section 1005, is permitted in the foundation approval.

(a) Method A - Use when bearing piles are driven to absolute refusal. (Applicable to dynamic formula and wave equation, as noted.)

(1) Dynamic formula only - Control pile driving by the dynamic formula in Publication 408, Section 1005 (specify year).

(2) Wave equation - Control pile driving by the wave equation analysis. (1) Dynamic formula or (2) wave equation - Drive test piles to absolute refusal. The Engineer shall verify, from the test pile driving results, the capability of the pile hammer selected by the Contractor. Drive bearing piles to absolute refusal into the stratum defined by a tip elevation which is predetermined by the Engineer from test piles. The Engineer shall determine the acceptability of the bearing piles which attain absolute refusal above the predetermined tip elevations.

(b) Method B - Use when bearing piles are driven to a capacity determined by the wave equation, but to less than absolute refusal.

Drive test piles to absolute refusal unless otherwise directed by the Engineer. The Engineer shall verify, from the test pile driving results, the capability of the pile hammer selected by the Contractor. Drive bearing piles to a tip elevation and a driving resistance predetermined by the Engineer from a wave equation analysis of the test piles. The Engineer shall determine the acceptability of the bearing piles which attain absolute refusal above the predetermined tip elevations.

(c) Method C - Use when bearing piles are driven to a capacity based on a pile load test result.

(1) Control pile driving by the wave equation analysis.

(2) Drive test piles to absolute refusal unless otherwise directed by the Engineer. The Engineer shall verify from the test pile driving results the capability of the pile hammer selected by the Contractor.

(3) Drive test piles at ____ before driving the load test piles at ____ (indicate locations). Drive all test piles relative to the pile load test first in order to determine the most representative location for the pile load test.

(4) The pile load test at ______ is intended to be representative of the bearing piles at ______ (indicate locations).

(5) Pile load tests at ______ may be conducted concurrently. (Use when there is no need to conduct load tests in consecutive order).

(6) Drive load test piles to a driving resistance and/or a tip elevation predetermined by the Engineer from a wave equation analysis of the test piles.

(7) Apply the load for the pile load test not less than ___ days after the test load pile has been driven.

(8) Do not drive bearing piles before the representative pile load test is completed and the results are evaluated by the Engineer.

(9) Drive bearing piles to a tip elevation and a driving resistance predetermined by the Engineer from the pile load tests. The Engineer shall determine the acceptability of the bearing piles which attain absolute refusal above the predetermined tip elevations.

(10) Show the following pile load test data on the drawings (modify as required):

Pile Load Test Data

In quantities:

Quick pile load test (*)	__ (each)
Additional quick pile load test (**)	__ (each)

In or near general notes:

Pile load test data:

Location (***)	xx
Type and size of pile	xx
Design load	xx
Estimated length	xx
Minimum required resistance factor (φ)	xx
Minimum required ultimate bearing capacity	xx
Minimum required capacity of test equipment (****)	xx

(*) The load test will be the quick type unless otherwise specified in the foundation approval or recommendation.

(**) Include additional pile load test when specified in foundation approval.

(***) Identify by substructure unit.

(****) Equate to nominal structural capacity of the pile, determined as follows:

For H and unfilled tubular steel piles:

$$P_u = A_s f_y$$

For CIP piles:

$$P_u = 0.85 f'_c (A - A_s) + A_s f_y$$

where:

P_u = ultimate structural capacity (KN) {Kips}

A = gross area of concrete (mm^2) {in^2}

A_s = area of steel H or tubular piles, or thick wall shell or rebars for cast-in-place piles (Exclude corrosion allowances and area of thin wall shells)

f'_c = Compressive structural design strength of concrete at 28 days. (MPa) {Ks_i}

f_y = specified minimum yield strength of steel (MPa) {Ks_i}

These ultimates are only for the purpose of sizing the load test equipment for laterally supported piles with zero eccentricity, and are not applicable to the structural design of the piles. Where it is obvious that the soil will develop significantly less than the ultimate structural capacity of the pile, the test equipment should be sized to ensure a plunging failure of the pile based on soil strength parameters.

(d) Method D - Use when driving is controlled by the dynamic formula and the bearing piles are not driven to absolute refusal.

(1) Control driving by the dynamic formula given in Publication 408, Section 1005.3.

(2) Drive test piles to absolute refusal unless otherwise directed by the Engineer.

(3) When timber piles are used, absolute refusal for timber piles is defined as bearing value not less than twice the required safe bearing value determined by the dynamic formula.

(4) Drive bearing piles to a safe bearing value of _____kN {tons} at or below a tip elevation predetermined by the Engineer from test piles. The Engineer shall determine the acceptability of the bearing piles which attain absolute refusal above the predetermined tip elevations.

1.7.5.2 NOTES FOR PILE INSTALLATION INFORMATION

The following table is to be included on the General Notes sheet of the bridge plan and is to be completed for the test piles after installation on the "as-built" plans:

Substructure Unit	Pile Type	Pile Tip (Y or N)	Pile Tip Elevation	Factored Design Load (kip)	Ultimate Pile Capacity at End of Driving (kip)	WEAP or PDA

1.7.6 Notes for Reinforced Concrete Box and Arch Culverts

Do not exceed a 600 mm {2 ft.} difference in fill elevation on the sides during placement of the backfill. Do not allow the wheels of rollers to come closer than 300 mm {1 ft.} to the face of the structure during compaction of the backfill.

1.7.7 Notes for Steel Beams and Girders

1. If beams (girders) cannot be shipped in the lengths shown on the plans, field splice(s) will be permitted at the request of the Contractor, but no compensation will be allowed for the splices (see PP1.7.10, Instruction 6).

2. If beams (girders) can be fabricated in lengths longer than the sections shown on the plans by eliminating field splices, field splice(s) may be omitted at the request of the Contractor. The Contractor assumes full responsibility for securing a hauling permit. Approval for elimination of a field splice at the shop drawing stage does not obligate the Department to issue a hauling permit (see PP1.7.10, Instruction 7).

3. Do not use form support systems that will cause unacceptable overstress or deformation to permanent bridge members.

4. All fasteners are 22.2 mm {7/8 in.} diameter HS bolts, except as noted.

5. Ream subdrilled or subpunched holes for field splices in the fabrication shop.

6. Prepare bearing areas as specified in Publication 408, Section 1001.3(k)9.

7. Do not make welds by manual shielded metal arc process for primary girder welds, such as flange-to-web welds or for shop splices of webs and flanges.

8. Do not weld permanent metal deck forms or other attachments to girder top flanges in tension areas. (Tension areas of top flanges are designated on the plans.) Threaded studs for the support of the overhang deck forming bracket is permitted provided the threaded stud is attached with the same welding processing as the shear studs.

9. Welding of reinforcement bars during fabrication or construction is not permitted unless specified.

10. Provide welded stud shear connectors manufactured from steel conforming to ASTM A 108.

11. Set anchor bolts to template or in preformed holes. Do not drill unless specifically indicated on plans. Fill the preformed holes with non-shrink grout. Fill the clearance between anchor bolts and holes in masonry plates with approved non-hardening caulking compound conforming to Publication 408, Section 705.8.

12. Paint structural steel in accordance with Publication 408, Section 1060.

13. Fabricate all members or member components designated as fracture-critical members (FCM) to conform to the requirements of Design Manual, Part 4, Section 6.6.2, and AASHTO LRFD Bridge Design Specifications, Article 6.6.2, and Publication 408, Sections 1105.02(a)4 and 1105.03(m)9. Meet the base metal Charpy V-notch (CVN) requirements for Zone 2.

14. Metallize structural steel in accordance with the special provision - Shop Metallizing and Painting of New Structural Steel. The flange, bearing stiffener plates and splice plates indicated are oversized in width to accommodate the reduction due to edge grinding.

15. Stability of partial girders and complete girders is to be maintained by the Contractor during erection, until all girders and diaphragms are in-place and all bolts are properly installed. Erection loads including self weight of the steel members, wind loading and construction live load effects are to be evaluated by the contractor for stability, stresses and deflections on the steel members during any stage of erection.

16. An alternate slab placement sequence may be permitted at the request of the Contractor. Submit for review and approval to the Department a revised slab placement sequence with support calculations and computer stress analysis. Satisfy the requirements of the original slab placement sequence. Obtain written approval prior to the use of the revised slab placement sequence and/or camber values. No compensation will be allowed for the development and approval of the revised slab placement sequence and camber values. The Department will be the sole judge of the acceptability of the revised slab placement sequence and camber values.

17. See D6.7.2.2.P, D6.10.3.2.5.2P, D6.13.2.8, and DE6.10.3.2.4.bP for additional notes to be shown on contract drawings.

1.7.8 Welding Notes for Rehabilitation of Structures or Where Field Welding Is Permitted

1. Welding specifications: AASHTO/AWS D1.5M/D1.5 Bridge Welding Code (specify year) consistent with Pub 408 1105.03(m).and the contract special provisions. Do not field-weld on any part of the existing bridge, except where shown on the drawings, without prior approval of the Engineer.

2. Welding of existing structural steel: Use the shielded metal arc process and low hydrogen electrodes which are compatible with the base metal as specified, and in accordance with an approved Weld Procedure Specification.

3. Make tack welds with the same type of electrode and incorporate in the final weld. No other tack welding will be permitted.

4. Do not weld when surfaces to be welded are moist or exposed to rain, snow, or wind, or when welders are exposed to inclement conditions that will adversely affect the quality of the work.

5. Do not weld or burn when the temperature is below -20º C {0º F}. Preheat and maintain the temperature of the metal to at least 20º C {70º F} when the temperature of the metal is between -20º C {0º F} and 0º C {32º F} during welding or burning.

6. Preheat the steel to the specified minimum temperature for a distance equal to the thickness of the part being welded, but not less than 75mm (3 in.) in all directions from the point of welding.

7. Remove by application of heat any moisture present at point of weld. Provide windbreaks for protection from direct wind.

8. Prior to placing the weld, thoroughly clean all portions of new and existing surfaces to receive welds of all foreign matter, including paint film, for a distance of 50 mm {2 in.} from each side of the outside lines of the weld.

9. Test completed welds using visual and nondestructive methods in accordance with AASHTO/AWS D1.5m/D1.5 Bridge Welding Code Chapter 6.

1.7.9 Utility Notes

Coordinate, locate, and conduct all work related to public and private utilities in accordance with Publication 408, Sections 105.06 and 107.12

1.7.10 Instructions

1. Indicate either DM-4 distribution factors, grid analysis method, three-dimensional finite element analysis method, or other.

2. The additional dead load shall not be applied to structures under fill.

3. Metal deck forms with blocked out valleys are commercially available and may be specified when economical for rehabilitation projects or when dead load is a control. Removable forms may also be specified for each case.

4. The note shall be shown on the drawing when retarder admixture is necessary in the concrete deck slab for conditions other than temperature control (mostly in skewed continuous structures).

5. The note shall appear on the plans for repairing, rehabilitating, widening, or extending the existing structures.

6. The design details/splices shall be coordinated with PP1.13, "Hauling Restrictions and Permits", and the standard note adjusted accordingly. If needed, the special provisions shall be expanded or adjusted by the designer. Optional field splice details should not be shown, but the note shall be shown on the drawing for beams or girders between 21 300 mm {70 ft.} and the limiting lengths specified in PP1.13.2.

7. The design details/splices shall be coordinated with PP1.13 and the standard note adjusted accordingly. If needed, the special provisions shall be expanded or adjusted by the designer. For beams or girders exceeding the lengths specified in PP1.13.2, field splice details shall be shown, and the note shall appear on the plans.

8. The note shall appear on the drawing when corrosion inhibiting admixture is to be specified for the project with the approval of the Chief Bridge Engineer. Indicate the time to corrosion.

1.7.11 Timber Notes

At the completion of preservative treatment, clean the treated material by a post-treatment steaming specified for the individual type of material or species. (Use this note for wooden bridges.)

1.7.12 Mechanically Stabilized Earth Walls

I hereby certify that all design assumptions have been validated either through construction details or notes on these drawings or through the contract plans and special provisions. (Show this note above the P.E. seal on structure plan.)

1.7.13 Additional Notes to be shown on Contract Drawings

For additional notes to be shown on contract drawings, see Figure PP1.6.4.11-2 reference (6) for Pile Tip Reinforcement, PP1.9.4.3.1(b)(5) for Geotechnical notes, PP1.10.1.1 for Alternate Design, PP1.10.6.2(b) for Plan Revisions, PP3.3.4.7(l) for Prefabricated Retaining Walls, DB10.5.15 for Micropile Notes, D11.12P3 for Gabion Retaining Walls, and D14.7.6.3.5 for bearing lift-off condition.

1.8 DESIGN COMPUTATIONS

1.8.1 General Requirements

(a) Original design computations shall be made in such a way that they can be microfilmed and will produce clear and legible copies. The minimum font size shall be 10 pt. Computations shall be arranged so that a Reviewer may easily follow the subject and procedures of the design analysis. Each sheet of design computations shall be thoroughly checked and initialed by the designer and the checker. An index sheet shall be provided for easy reference. The computations shall be of good contrast and shall be only on one side of 297 mm x 210 mm (Metric Size A4) or 8 ½” x 11” (U.S. Customary Letter Size) sheets for ease of microfilming. An exception to the minimum font size may be approved by the District on a case-by-case basis if space is limited and all caps are used. For these limited cases, a minimum 8.5 pt size in a font such as ARIAL may be considered.

(b) Design computations are not required for any portion of a structure for which the information is taken verbatim from an applicable current Department standard. In such cases, the standard shall be referenced in the calculations.

(c) In general, the design computations shall consist of these items: geometry calculations, structural analysis, quantity computations and necessary sketches.

Each phase of computer-generated computations should include a table showing the summary of results unless the critical items are obvious, e.g., for "Structural Analysis" the table showing the summary of results shall indicate the actual and allowable stresses for various loading groups.

Preliminary computations are not required, but may be included in the computations if identified and separated from the final design computations.

When methods or formulas which are not in general use are employed, the source shall be given, including title and edition of the book, name of author(s), publisher and page numbers.

(d) Title sheet and cover of original design computations shall show the name of county, project route (S. R. number and section number), station of structure and S-number of the drawings, and be signed and sealed by the responsible engineers. Each sheet of calculations shall be dated by designer and checker.

(e) Original design computations shall be bound in hard covers that completely enclose the parts of the fasteners used. Bindings with exposed fasteners shall not be used. Upon final acceptance of the drawings, the original design computations shall be submitted to the District Office as a permanent record.

1.8.1.1 POLICY FOR DESIGNER'S RESPONSIBILITY FOR CONSTRUCTIBILITY

CASE A: Construction Methods and/or Sequences Specified by the Designer in the Contract Documents

When the design model assumes that the design loads will be applied in a certain sequence or under certain conditions, the designer must provide enough information in the contract documents (including plans) to ensure that construction sequence and/or methods will be consistent with the design assumption. In such cases, the designer is fully responsible and accountable for constructibility due to all temporary and cumulative design loads which eventually become part of the final design loads.

CASE B: Designs Based on New Design Methods which May Affect the Constructibility of a Common Bridge by Decreasing the Size of its Members. Optimized Designs are also Included in Case B.

The Designer is responsible for providing an appropriate notice in the contract documents (including plans) if typical construction methods used successfully in the past will not be permitted because of higher temporary construction stresses that may occur due to design dead loads and/or construction loads. As an alternative, the designer may design for a typical construction method which has been used successfully in the past, if it is economical to do so, and indicate the assumed construction method in the contract documents. The reason for assigning this responsibility to the designer is that the designer is in a better position to be aware of the affect that changes in design specifications and design methods will have on constructibility than would be the Contractor.

Although this policy assigns responsibility to the designer for Case B, the degree of accountability will vary with the circumstances as follows:

(1) If new design specifications or design methods implemented by the Department contain guidelines and/or commentaries which identify potential problems in constructibility, the designer will be held accountable if constructibility problems occur because of a failure to introduce constructibility guidelines in the design.

(2) The absence of constructibility guidelines or commentaries in new design specifications will not relieve the designer of responsibility for addressing constructibility. Accountability, should problems occur, will depend on whether the designer made a reasonable effort to address constructibility. An example of a reasonable effort in this case would be a request for approval of constructibility criteria during design, preferably at TS&L stage.

For contractor-designed alternates and Design-Build projects, the Contractor is completely responsible and accountable for constructibility.

Even with reasonable efforts, some constructibility problems may still occur because of the current state-of-the-art. In such cases, the Department, in the past, has been quite liberal in assuming some degree of accountability and will continue to do so.

The Designer's responsibility for constructibility must be considered in the technical scope of work in consultant engineering proposals.

For bridges under construction, the revised design specifications can often be met by modifying the size and sequence of the deck pours.

1.8.2 Computer Programs

When computations are performed using a computer program, the input and output, with an explanation of terms, and the assumptions and computations used for the determination of the input values, shall accompany the design computations.

For computer programs not available through the Department, a sketch with an explanation of all abbreviations and symbols shall accompany the input and output sheets of the program. All computer programs shall be identified. Design methodology employed by the programs shall be indicated: line girder, grid analysis, finite element, etc.

For software review requirements, see PP1.4.5.

1.8.3 Rating Computations

(a) Computations shall be made showing the inventory and operating ratings in terms of H, HS, ML, TK-527 and PHL-93. Computations shall also be made showing the operating rating for P-82 vehicle. The ratings shall be calculated using the loadings described in A3.6 and D3.6. For steel and prestressed girders, concrete box culverts, steel floorbeams and steel trusses, Tables D3.4.1.1P-1, D3.4.1.1P-2, D3.4.1.1P-4, D3.4.1.1P-5 and D3.4.1.1P-6, respectively, shows the limit state load combination corresponding to the different rating vehicles. The original rating computations are to be included with the original design computations. A description of the method of analysis is to be included in the calculations. The ratings shall be determined by the design method and analysis method (DM-4 distribution factors or a refined method) which were used in design, unless changed by direction of the Chief Bridge Engineer.
Rating calculations shall clearly show the total inventory and operating capacity and the live load capacity, so that the calculations can be used throughout the life of the structure.

(b) The bridge plans for deck replacement, overlays, major rehabilitation and new construction, including Contractor's alternate designed bridges, shall show the bridge load ratings as indicated in the sample chart below:

Table 1.8.3(A) - Bridge Load Ratings
ADTT (at the Time of Design)
Cumulative ADTT (at the Time of Rehabilitation)

		Beam Type and Size					
Span No.		H 20	HS 20	ML-80	TK-527	PHL-93	P-82
Inventory Rating (IR)	Distribution Factor	0.720	0.720	0.720	0.720	0.720	---
	Location	CL	CL	CL	CL	CL	---
	Limit State	SERV III	SERV III	SERV III	SERV III	SERV III	---
	Rating Factor	1.39 M	1.19 M	1.07 M	1.02M	1.04 M	
Operating Rating (OR)	Distribution Factor	1.018	1.018	1.018	1.018	1.018	1.018
	Location	0.05 L	0.45 L	0.45 L	0.45L	0.35 L	0.25 L
	Limit State	STR II	STR II	STR II	STR II	STR IA	STR II
	Rating Factor	2.75 S	2.17 S	1.96 S	1.90 S	2.09 S	1.00 S

Critical Member	Interior
Maximum Factored Flexural Resistance (kNAm) {kAft}	17 993 kN m {13,269 k ft}
Location	0.45 L
Maximum Factored Shear Resistance (kN) {kips}	679 kN {153 kips}

Location 0.45L

Notes:

"M", "S", and "SM" denote that moment, shear, and shear/moment interaction control the rating factor, respectively.
Given distribution factor is the vehicular load distribution factor used to produce the given rating. For the STR-IP limit state, the vehicular live load distribution factor accounts for the presence of pedestrian loads, if applicable.

Include with the chart on the bridge plans the following information:

(1) The force effect controlling the rating.

(2) The limit state used to obtain each of the ratings and each critical moment and shear value.

(3) Live load distribution factors for shear or moment for each load combination used to produce that rating.

(4) IR = Inventory Rating, OR = Operating Rating, ML = Pennsylvania Maximum Legal Load (ML-80), P-82 = Pennsylvania Permit Load

(5) For multiple span structures, identify and provide data for critical span(s) only.

(6) Provide moment and shear influence lines for the structure that is designed using a refined method of analysis.

(7) Identify critical (in moment or shear) member(s) and location.

(8) Load due to future wearing surface is included in the ratings. (The engineering districts may require the rating value with and without future wearing surface.)

(9) The critical moment and shear shall be determined from the operating rating calculations.

(10) Identify whether shear or moment controlled the rating for each rating value.

(11) Identify beam properties used.

(12) Identify continuity assumption used (i.e., whether positive moment reinforcement was considered in ratings).

The table may be modified in order to provide all of the required information. All rating information should be shown together on the first sheet of the bridge plans or indexed on the first sheet.

1.9 BRIDGE SUBMISSIONS – DESIGN PHASE

Refer to Tables 1, 2 and 3 for determining review and approval responsibility for each bridge submission.

1.9.1 General

The chronology of the bridge-related submissions for approval shall be made as follows:

(a) Hydraulics and Hydrologic Report (if applicable) (see PP1.9.2)

(b) Type, Size and Location (TS&L) (see PP1.9.3)

(c) Foundation Submission (see PP1.9.4)

(d) Final Review of Plans (see PP1.9.5)

(e) Final Plans (see PP1.9.6)

(f) Plans, Specifications and Estimate (PS&E) (see PP1.9.7)

All submissions must include pertinent Q/A forms (refer to Appendix A) without which the submission will be returned without any action by the approving office.

1.9.2 Permit Applications and Hydrologic and Hydraulic Report

Permit Applications and Hydrologic and Hydraulic Reports shall be prepared to meet the design requirements of Design Manual, Part 2, Chapter 10; Design Manual, Part 4, Chapter 7; active Strike-Off-Letters (SOLs); and applicable requirements for regulatory permits.

The following procedures and guidelines apply to all PENNDOT projects (Federal-Aid and 100% State) for the submission and review of Hydrologic and Hydraulic Reports:

1. All information required for meeting design and regulatory requirements shall be prepared in hardcopy or electronic copy through the Joint Permit Application (JPA) system. The information developed shall be reviewed by the District Environmental Manager and District technical staff to ensure compliance with all applicable design, environmental, and regulatory requirements. The information developed shall:
   a. include the results of the hydrologic and hydraulic analyses associated with design of the waterway structures or encroachments,
   b. incorporate pertinent prior National Environmental Policy Act (NEPA) documents by reference or excerpt,
   c. address coordination activities with environmental resources agencies, and
   d. reflect any commitments or agreements reached which may affect the processing of the permit application.

2. Hydrologic and Hydraulic Reports for municipal structures using Federal-aid funds also shall be submitted by the Engineering District to Bureau of Design for quality assurance (Q/A) review. This submission may be made at the time of permit application (see Step 6 below). The Bureau of Design will perform the review and provide comments, if any, within fifteen calendar days on selected projects.

3. For projects involving any of the following list of issues, the Engineering District shall submit one additional copy of Hydrologic and Hydraulic Reports to the Bureau of Design for transmittal to the Federal Highway Administration (FHWA) for review and approval:

   a. Significant or controversial channel changes.
   b. Significant or controversial backwater easements.
   c. Significant bridge scour (usually manifested by high stream velocity, severe waterway constriction, deep foundation and/or expensive scour mitigation measures).
   d. Permanent impoundments or causeways involving roadway embankments.
   e. Major bridges with costs of more than $10 million.

4. For Joint Permit Applications, the Engineering District shall perform necessary coordination with the applicable counties and municipalities pursuant to 25 Pa.Code §105.13(d)(1)(v-vi) pertaining to the Stormwater Management Act (32 P.S. §§680.1 et seq.) and the Floodplain Management Act (32 P.S. §§679.101 et seq.). For permit applications involving communities with Stormwater Management Plans, or Floodplain Management Plans, implemented under the Acts, the Engineering District shall request from the local municipality a written statement that the proposed project is consistent with local stormwater management plans and with local floodplain management plans. If the Engineering District cannot obtain a written statement of consistency, the Engineering District must provide sufficient documentation with the permit application to demonstrate consistency with local plans implemented under the Acts.

5. For Joint Permit Applications, the Engineering District shall consider local land use plans.

6. The Engineering District shall complete and sign the appropriate permit application form or letter and send three sets of the permit application packet (including a completed application form or letter, and the required attachments) to the appropriate regulatory reviewing authority or submit the permit application to them through the JPA system. More or fewer sets of the permit application packet may be required by the regulatory review authorities, depending on the type of permits or nature of permit requests involved. For Joint Permit Applications, the primary regulatory review authority will coordinate review of the application with other reviewing agencies when and as necessary (except for the U.S. Coast Guard, or as otherwise notified by the primary review authority on a case by case basis).

7. One copy of the permit application package shall be sent to the Bureau of Design for Quality Assurance (Q/A) review in hardcopy or through the JPA System. The Bureau of Design will perform the review and provide comments, if any, within fifteen calendar days on selected projects.

8. Where there are unusual or controversial special permit conditions specified on the permit by the regulatory authority, the Engineering District should consult with the Bureau of Design prior to acceptance of the permit.

9. The Engineering District shall submit one copy of each permit, or regulatory approval, (including permits or approvals received from the U. S. Army Corps of Engineers and the Pennsylvania Fish and Boat Commission) to the Bureau of Design for microfilming. The Bureau of Design shall be responsible for microfilming all permit documents and will continue to maintain a statewide file of these microfilm records.

Please note that the Engineering District is responsible for processing and obtaining all necessary regulatory permits (such as the U. S. Army Corps of Engineer's Section 10 and U. S. Coast Guard Bridge Permits for proposed activities in navigable waters of the United States). Copies of all issued regulatory permits shall be submitted by the Engineering District to the Bureau of Design for microfilming.

1.9.2.1 HYDROLOGIC AND HYDRAULIC COORDINATORS AND REGULATORY PERMIT COORDINATORS

Each Engineering District shall appoint a Hydrologic and Hydraulics (H&H) Coordinator and a Regulatory Permits Coordinator. These Coordinators are responsible for coordinating the processing of all hydrologic and hydraulic reports and all regulatory permits (or approvals) respectively. Jointly, these coordinators shall be knowledgeable in both administrative and technical aspects of the Hydrologic and Hydraulic Report and the permit application package. It is suggested that these coordinators be selected from the District’s Bridge Unit and the District’s Environmental Unit respectively.

1.9.2.2 RECORDS OF REGULATORY PERMITS

The Engineering District shall maintain complete records of the Hydrologic and Hydraulic Reports and the permit documents sent to regulatory agencies for permit approval. Joint Permit Applications submitted to DEP through the JPA System inherently meet this requirement.

1.9.2.3 PERMIT AMENDMENTS

Regulatory permits for construction activities involving highway structures usually are issued at an early stage in the design process. During later stages of the design process, or during construction, issues may arise that cause changes in the information submitted to regulatory agencies. All changes in design after permits are issued must be weighed carefully against the regulatory permit review criteria (for example, see 25 Pa.Code §105.14, 151, and 161), and the information provided in the permit application. Early consultation and coordination with the regulatory agencies is encouraged. Minor design changes may be eligible for an amendment by letter. For large changes, the regulatory agency may process the amendment request in the same manner that it processes a new application. Primary responsibility for permit amendments during the design or construction phases of project development resides with the party responsible for design or construction respectively.

When design changes or construction changes are proposed that affect, or may affect, a regulatory permit, the following steps shall be followed:

1. The party responsible for any such changes shall carefully evaluate the effects of the proposed changes on the information provided in permit applications, issued permits, and conditions attached to issued permits. If any of these items are, or may be affected, then proceed to step 2.

2. The District’s Regulatory Permit Coordinator will determine whether or not a permit amendment is necessary. Provide all information to the Regulatory Permit Coordinator necessary to enable a sound decision on whether or not an amendment is needed. The Regulatory Permit Coordinator will contact the regulatory agency/ies when necessary. If the Regulatory Permit Coordinator determines that a permit amendment is necessary, then proceed to step 3.

3. During coordination with the regulatory agencies, the Regulatory Permit Coordinator will determine whether or not the proposed amendment is minor. If the amendment is minor, then proceed to Subsection (a) below; otherwise, proceed to Subsection (b).

a. Minor Amendments
   i. Compile all changes in the information submitted with the original permit application.
   ii. Provide a certification with signature and professional seal for the changes as required by 25 Pa. Code §105.13(I).
   iii. Provide a transmittal letter. The letter should be addressed to the regulatory review authority (i.e. DEP or a County Conservation District) from PENNDOT. The letter shall provide a brief summary of the proposed changes; state that the changes are minor; and request a letter of amendment for a minor change.
   iv. Submit a package of the above items to the attention of the Regulatory Permit Coordinator at PENNDOT's District Office. PENNDOT will review the materials.
   v. If satisfactory, the package will be approved, signed, and forwarded to the regulatory review authority; otherwise, revision and resubmission will be necessary.

b. Other Amendments
   i. For non-minor amendments, the approval process at the regulatory agency follows the same process as a new permit application. This includes publication in the Pennsylvania Bulletin for public comment and publication of notice when the amendment is approved.
   ii. Compile all changes in the information submitted with the original permit application. Follow the procedures described in Sections 1.9.2. Paragraphs 1 through 9 above. Be sure to include new notices to local municipalities. Provide replacement pages or replacement sections for the permit application and its attachments for all parts of the permit application affected by the proposed changes.

4. The Engineering District will forward one copy of the amendment package to the Bureau of Design for Q/A review at the same time that it is sent to the regulatory review authority.

5. When a permit amendment is approved, the Engineering District will receive a Letter of Amendment from the regulatory review authority.

6. The Engineering District will forward one copy of the Amendment Letter to the Bureau of Design for microfilming.

7. The Letter of Amendment becomes a part of the original permit and must be permanently attached to the original permit.

1.9.2.4 PERMITS FOR PROJECTS WITH ALTERNATE STRUCTURES AT ONE LOCATION

For major structures (re: Section 1.9.2, Paragraph 3.e), PENNDOT may require regulatory permitting for more than one structure design at one location.

Attachments to the permit application should follow the outline recommended by the permit application forms. For individual items such as the Hydrologic and Hydraulic Report, the sections most affected when analyzing alternate structures at one location will be the hydraulic analysis, the risk assessment, and the summary data tables. With alternate structures, the report outline should not change; however, within each of these sections, each alternate should be analyzed independently. For example, the hydraulic analysis should contain subsections for Alternate 1, Alternate 2, etc. Other affected sections should be handled similarly.

1.9.2.5 PERMITS FOR DESIGN-BUILD PROJECTS

Regulatory permitting for structures does not depend on whether construction contracting uses the traditional procedures or the newer Design-Build procedures. PENNDOT will obtain regulatory permits in either case. Permits for Design-Build projects may be required to include alternate structures at one location as discussed in Section 1.9.2.4 and Section 1.11. Design changes subsequent to permitting must follow the process for permit amendments described in Section 1.9.2.3.

Table 1.9-1 - Review and Approval Responsibility for Federal Oversight Project

FEDERAL OVERSIGHT PROJECTS (7)

(Interstate projects with project cost > $3 million, other NHS projects costing ≥ $15 million, and Appalachian Development Highway System Corridor Completion Projects) in accordance with the June 2007 Stewardship and Oversight Agreement

Item No	Category	TS & L Approval (1)			Final Plans (3), (6)			Remarks
		FHWA	BQAD	District Br. Engr. (2)	FHWA	BQAD	District Br. Engr. (2)	
1. Interstate Pavement Preservation (PPG) (4)	Bridge cost ≤ $3 million (8)	-	-	✓	-	-	✓	
	Bridge cost > $3 million (8)	✓	-	-	-	-	✓	
2. 3R type of work	Deck rehab. ≤ $15 million (4) (8)	-	-	✓	-	-	✓	
	Deck rehab. > $15 million (4) (8)	✓	-	-				
	Culvert extensions	-	-	✓	-	-	✓	
	Wall rehabs /extension	-	-	✓	-	-	✓	
	Deck replacement ≤ $15 million (4)(8)	-	-	✓	-	-	✓ (See remarks)	Final plans for multi-span steel structures, steel structures with skew < 70º, curved girders, and fracture critical structures must be sent to BQAD for approval.
	Deck replacement > $15 million (4)(8)	✓	-	-				
	Superstructure rehabilitation or replacement ≤ $15 million (4) (8)	-	-	✓ (See remarks)	-	-	✓ (See remarks)	TS & L and Final plans for multi-span steel structures, steel structures with skew < 70º, curved girders, and fracture critical structures must be sent to BQAD for approval.
	Superstructure rehabilitation or replacement >$15 million (4) (8)	✓	-	-				
	Substructure rehabilitation ≤ $15 million (4) (8)	-	-	✓	-	-	✓ (See remarks)	Note that District must submit seismic and fatigue retrofit details to BQAD for approval.
	Substructure rehabilitation > $15 million (4) (8)	✓	-	-				
3. New Construction or Reconstruction	New interstate bridge cost ≤ $3 million	-	-	✓	-	-	✓ (See remarks)	Final plans for multi-span steel structures, steel structures with skew < 70º, curved girders, and fracture critical structures must be sent to BQAD for approval.
	New interstate bridge cost > $3 million	✓	-	-				
	New NHS bridge cost ≤ $15 million	-	-	✓				
	New NHS bridge cost > $15 million	✓	-	-				

Table 1.9-1 - Review and Approval Responsibility for Federal Oversight Project (Continued)

FEDERAL OVERSIGHT PROJECTS (7)

(Interstate projects with project cost > $3 million, other NHS projects costing ≥ $15 million, and Appalachian Development Highway System Corridor Completion Projects) in accordance with the June 2007 Stewardship and Oversight Agreement

Item No	Category	TS & L Approval (1)			Final Plans (3), (6)			Remarks
		FHWA	BQAD	District Br. Engr. (2)	FHWA	BQAD	District Br. Engr. (2)	
4. Misc. Str. (4)	Culverts	-	-	✓ (2) (See remarks)	-	-	✓ (See remarks)	Culverts under fills greater than 12 000 mm {40 ft.} must be approved by BQAD.
	Ret. walls < 9000 mm {30 ft.} height	-	-	✓ (2)	-	-	✓	TS&L and Final Plan approval for all MSE walls are to be in the District office.
	Ret. walls ≥ 9000 mm {30 ft.} height	✓	-	-	-	✓	-	All MSE wall submissions are to be approved in the District office.
	Sound Barrier Walls	-	-	✓	-	-	✓	
	Arches: span ≤ 18 000 mm {60 ft.}	-	-	✓ (2)	-	-	✓	
	Arches: span > 18 000 mm {60 ft.}	-	✓	-	-	✓	-	
	Sign Structures	-	-	-	-	-	✓	No formal approval is required.
	DMS Structures	-	-	✓	-	✓	-	
	Rehab of unusual structures	✓	-	-	-	✓	-	
5. Other (4)	All exceptions to HBP program (eligibility) criteria	✓	-	-	-	✓	-	Including deficiency status, sufficiency rating, and 10 year rule
	Design exceptions to structural design criteria and policies	✓	-	-	-	✓	-	Over STRAHNET, coordinate with FHWA for exceptions to 4900 mm {16 ft.} minimum vertical clearance and changes to clearances currently less than 4900 mm {16 ft.}. See DM2, Section 2.21. Other design exceptions must follow guidance as defined in DM-1A, Appendix F and be coordinated with HQAD.
	Non-redundant structures	✓	-	-	-	✓	-	
	3-D analysis	✓	-	-	-	✓	-	Final plans approval authority may be transferred to Dist. Br. Engr., depending upon complexity.
	All experimental or demonstration items	✓	-	-	-	✓	-	
	Unusual structures	✓	-	-	-	✓	-	
	Light Poles	-	-	-	-	-	✓	Review and approve for structural capacity only. See Appendix B. (5)

Note:
(1) Includes Conceptual and Final TS&L submissions for Design-Build projects.
(2) A copy of approvals must be given to BQAD.
(3) Scour, safety, expansion dams and load capacity items must be addressed for all work categories.
(4) Seismic design or assessment must be completed for structures where the work requires removal of the deck, new piers, or new superstructure. For minor maintenance projects, the seismic criteria may be deferred if requested at the TS&L stage.
(5) Light poles approved under a general submission do not require a re-review and are considered pre-approved for other projects.
(6) Includes Final Plans for Design-Build projects
(7) All P.S. & E's for Federal oversight projects are still sent to FHWA for final approval
(8) Cost is based on TS&L estimate and is a per bridge cost for projects with multiple bridges

Table 1.9-2 - Review and Approval Responsibility for Foundation Approval for all Projects

FOUNDATION APPROVAL FOR ALL PROJECTS (1)
Includes Retaining Walls

Category	FHWA	BQAD(2)	Dist. Br. Engineer	Remarks
1. Footings on bedrock	-	-	✓	
2. Culverts with integral bottom on rock or soil	-	-	✓	
3. Footings on soils	-	-	✓	Including MSE wall footings
4. Foundation on pedestals(must be on rock)	-	-	✓	In mining areas approval is from BQAD
5. Foundation on piles not in mining areas	-	-	✓(3)	
6. Anchored walls	-	✓	See Remarks Column	Dist. Br. Engr. may approve anchored walls for local projects.
7. Foundation on piles in mining area, on caissons and on micropiles	✓ Federal Oversight Projects only	✓ PENNDOT Oversight & Non-federally funded projects	-	
10. Unusual foundations	✓ Federal Oversight Projects only	✓ PENNDOT Oversight & Non-federally funded projects	-	
11. Foundation for arches without ties	✓ Federal Oversight Projects only	✓ PENNDOT Oversight & Non-federally funded projects	-	
12. Sound Barrier Walls	-	-	✓	
13. Sign Structure/DMS	-	-	✓	
14. High Mast Lighting	-	-	✓	

Note:
(1) If substructure units of a bridge are supported by different types of foundations, the approval authority will be determined using the most critical foundation type.
(2) On Federal oversight projects when the TS&L requires FHWA approval, a copy of the foundation approval must be sent to FHWA.
(3) Structures with a significant amount of friction piles, e.g., major viaducts, major river crossings and > 300 000 mm {1,000 ft.} of retaining walls, will be submitted to BQAD (and FHWA if Federal Oversight) for approval.

Table 1.9-3 - Review and Approval Responsibility for PENNDOT Oversight Projects

PENNDOT OVERSIGHT PROJECTS
(Interstate projects with cost ≤ $3 million, other NHS projects costing < $15 million and Off NHS Projects)
in accordance with the June 2007 Stewardship and Oversight Agreement
AND NON-FEDERALLY FUNDED PROJECTS INCLUDING LOCAL PROJECTS

Item/Category	TS&L (1)		Final Plan Approvals (2)		Remarks
	BQAD	District Br. Engr.	BQAD	District Br. Engr.	
1. Deck rehabilitation	-	✓	-	✓	
2. Superstructure rehabilitation	-	✓	-	✓	
3. Substructure rehabilitation	-	✓	-	✓	
4. Culvert extension	-	✓	-	✓	
5. Wall rehabilitation	-	✓	-	✓	
6. Deck & superstructure replacement	-	✓ See remark column	-	✓ See remark column	BQAD is only responsible for multi-span steel bridges, steel bridges having skew < 70°, curved girders, complex and unusual bridges, bridges designed using 3D analysis, and fracture-critical bridges.
7. New bridges & bridge replacements with bridge cost < $15 million	-	✓ See remark column	-	✓ See remark column	BQAD is only responsible for multi-span steel bridges, steel bridges having skew < 70°, curved girders, complex and unusual bridges, bridges designed using 3D analysis, and fracture-critical bridges.
8. New bridges & bridge replacements with bridge cost ≥ $15 million	✓	-	✓	-	Final plan approval authority may be transferred to Dist. Br. Engr. on case-by-case basis.
9. Culverts	-	✓	-	✓	
10. Retaining walls < 9000 mm {30 ft.} height	-	✓	-	✓	
11. Arches ≤ 18 000 mm {60 ft.} Span L	-	✓	-	✓	
12. Mechanically stabilized earth walls	-	✓	-	✓	
13. Sound Barrier Walls	-	✓	-	✓	
14. Retaining walls ≥ 9000 mm {30 ft.}	✓	-	✓	-	Final plan approval authority may be transferred to Dist. Br. Engr. on case-by-case basis.
15. Arches > 18 000 mm {60 ft.}	✓	-	✓	-	Final plan approval authority may be transferred to Dist. Br. Engr. on case-by-case basis.
16. Designs which use non-AASHTO load distribution factors	✓	-	✓	-	Final plan approval authority may be transferred to Dist. Br. Engr. on case-by-case basis.
17. Unusual structures	✓	-	✓	-	Final plan approval authority may be transferred to Dist. Br. Engr. on case-by-case basis.
18. Anchored walls	✓	-	✓	-	Final plan approval authority may be transferred to Dist. Br. Engr. on case-by-case basis.
19. <u>All</u> design exceptions to structural design criteria and policies	✓	-	✓	-	Over STRAHNET, coordinate with FHWA for exceptions to 4900 mm {16 ft.} minimum vertical clearance and changes to clearances currently less than 4900 mm {16 ft.}. See DM2, Section 2.21. Other design exceptions must follow guidance as defined in DM-1A, Appendix F and be coordinated with HQAD.
20. Non-redundant structures	✓	-	✓	-	
21. <u>All</u> experimental or demonstration items	✓	-	✓	-	
22. Light poles	-	-	-	✓	Review and approve for structural capacity only. See Appendix B.
23. Sign structures	-	-	-	✓	No formal approval is required.
24. DMS structures	-	✓	✓	-	

Note: (1) Includes Conceptual and Final TS&L submissions for Design-Build projects.

(2) Includes Final Plans for Design-Build projects.

A copy of all approvals by Dist. Br. Engr. and completed Q/A forms must be sent to BQAD. Scour, seismic retrofit, safety, expansion dams, and load capacity items must be addressed for all work categories. All design exceptions for structures must be approved by BQAD and/or FHWA. Exceptions or waivers of HBP program eligibility criteria must be approved by FHWA Division.

1.9.3 Type, Size and Location (TS&L)

1.9.3.1 GENERAL

The investigation of a proposed structure shall be sufficiently intense to discriminatingly select and justify type, size and location on the basis of the information available from the various phases of study outlined in Design Manual, Part 1, including any foundation information obtained. Preliminary cost comparisons shall be made to support TS&L recommendations.

The District Bridge Engineer or his representative shall attend the Design Field View for all bridge projects to provide input in finalizing location, horizontal and vertical alignment for the project, taking into account site specific conditions, such as slide or scour potential and railroad clearances. Whenever an existing foundation is to be reused and new loads are to be applied, the existing foundations must be re-evaluated to assure adequate foundation carrying capacity. For bridge projects over railroad facilities, early coordination and involvement with the Railroad shall be in accordance with the latest edition of Publication 371, Grade Crossing Manual, Chapter 4. This would allow for the Railroad's input to various design parameters being considered.

TS&L for any structure supported on proprietary walls shall not be approved unless adequate foundation information including scour evaluation (if applicable) is available or foundation investigation is completed and recommendations are available.

Type of substructure will be approved during foundation approval.

Formal TS&L approval is required for in-house designed BRADD projects.

Submit the TS&L Report and Structure Geotechnical Foundation Report for Retaining Walls, and Sound Barrier Walls concurrently.

1.9.3.2 RESPONSIBILITY

Refer to Tables PP1.9-1 and PP1.9-3 for the review and approval responsibility for TS&L. For new bridge designs having a deck joint at a substructure unit, the TS&L must be submitted to BQAD for approval.

1.9.3.2.1 Responsibility of District

The designer shall submit two sets of TS&L plans and related information (see PP1.9.3.3) to the pertinent District for approval when the District is responsible for TS&L approval.

The District shall send to the BQAD an informational copy of the final TS&L approval letter, with road plans, applicable Q/A forms and preliminary bridge plans showing core boring layout.

1.9.3.2.2 Responsibility of BQAD

The District Executive shall submit to BQAD, for approval, two sets of TS&L plans and related information (see PP1.9.3.3) for approval when BQAD is responsible for TS&L Approval.

If the District desires to revise a Consultant's submission, the revision shall be marked on the plans in red and forwarded to BQAD with an explanation where necessary.

BQAD will review the submission and will approve it if it is found satisfactory and after obtaining necessary FHWA approval. Submission of revised preliminary plans will be requested, if necessary.

1.9.3.3 SUBMISSION REQUIREMENT

TS&L Submission requirements are divided into two categories, Standard and Streamlined. Standard TS&L submissions are required on all projects unless a Streamlined TS&L submission is agreed upon by the District, BQAD and FHWA (if applicable).

1.9.3.3.1 Standard TS&L

The following information shall be included for TS&L submission:

(a) TS&L submission letter

The letter of transmittal shall include the following:

(1) Location - Over or under S. R. or local road, segment, offset, and station (and/or stream name, railroad name, or road name)

(2) Type of superstructure recommended - Girder size and spacing and deck overhang dimensions

1. Indicate the type of superstructure coating system for steel girders (i.e. Painting, Galvanizing, Metallizing or None [for weathering steel bridges]).

(3) Span C/C bearing and/or C/C piers

(4) Roadway width Out-to-out, curb-to-curb and sidewalk width where applicable

(5) Skew angle or range of skew angles and direction (left or right)

(6) Vertical and horizontal clearance: Minimum required, actual provided

(7) Type of substructure recommended

(8) Location, type and movement classification of proposed deck joints

(9) Bearing type and location (defer designation of bearing fixity in multi-pier structures until final design)

(10) Deck and off structure drainage

(11) Design methodology to be used for superstructure design

(b) TS&L plans

The following information shall be shown on TS&L plans:

(1) Plan view, including controlling clearances, span length, skew, existing contours and finished contours (excluding BRADD plans), scupper locations, and end structure drainage, where required

(2) Elevation view showing controlling clearances, span length, existing and finished ground line, continuity, fix-expansion support condition, type and movement classification of expansion dams, and type of bearings

(3) Cross-section showing out-to-out (O/O) dimension, traffic lanes, shoulder widths, beam type, size and spacing, overhangs, cross slope, superelevation, minimum slab thickness, type of traffic or pedestrian barrier, and thickness of wearing surfaces

(4) Typical sections showing limits of individual construction stages where staging is required for construction of the bridge. Locations of longitudinal joints in the deck, locations and the type of temporary barrier, and traffic lane locations and widths shall be shown.

(5) Elevation view of pier(s) showing proposed configuration

(6) Deck protective system (for rehabilitation projects only)

(7) Loading, design and analysis method; non-standard details

(8) Core boring layout

(9) Hydraulic information including design flood data, flood of record and date, slope protection, where required, and preliminary scour information

(10) Horizontal and vertical curve data for all roadways shown

(11) For retaining walls, the length and height for each segment (Note that the TS&L for walls will not be approved until foundation recommendation is provided.)

(12) Bridge-mounted lighting poles, sound barriers and signs, if required.

(c) Report on alternate studies and justification for the recommended bridge types

(1) Cost comparison for all types considered during type, size and location study. (The cost estimate shall be arranged so as to indicate total cost per substructure unit and major portion of superstructure, e.g., rolled beam span, plate girder span.)

(2) Justification for recommended alternate

(3) Address the need to account for future widening and future redecking requirements into the recommended bridge

(4) Design Requirements for Contractor-Designed Alternate Structures and Design-Build Projects.

- Permissible changes to the bridge geometrics (span, bridge width, abutments, and piers) and vertical and horizontal alignment.
- Permissible Material Types (e.g., weathering steel, proprietary walls, etc.).
- Permissible Number of Deck Joints (typically, this will be the number of deck expansion joints provided in the as-designed structure; however, this limitation should not be so restrictive that it eliminates the use of individual superstructure material types for the alternate).
- Future Redecking Requirements (as applicable)
  - Maximum Number of Permissible Construction Stages.
  - Number of Required Lanes.
  - Minimum Lane Width(s).
  - Lane Location Limitations (if any).
  - Need to Maintain Pedestrian Traffic.
  - Minimum Number of Beams.
  - Design requirements for the individual stages.
- Future Widening Requirements (as applicable).
- Environmental Requirements Related to the Structure (as specified in the environmental clearance document - EIS, EA, CEE, or EER).
- Other.

(d) Soil reconnaissance and foundation exploration plans

Submission requirements are discussed in PP6.3.1.

(e) Additional information to be supplied by the designer

(1) Route and section number, index map and segment/offset of limits

(2) Program under which project will be financed (Federal-aid classifications, 100% State-funded, Department Force, or special program) , the WBS code and MPMS number.

(3) Name of designer (Consultant or District Office)

(4) List of proposed structures by station and type

(5) Design traffic data including current and projected ADTT and class of highways on relevant roads

(6) Date of line and grade approval and design speed

(7) Statement on balance of earthwork for project

(8) Statement whether project is designed for free or controlled access

(9) Prints or roadway plans showing approved typical sections; also pavement-type approval when available

(10) Copy of waterway approval (from Department of Environmental Protection) and results of acidity tests of water and soil, if applicable

(11) Copy of the minutes of the Design Field View approval as defined in Design Manual, Part 1, Chapter 2, and available road plans.

(12) For rehabilitation projects, the following information shall be provided:

1. Age of existing structure, present and cumulative ADTT, portion to be replaced, type of steel-for-steel bridges, date of last inspection, type of diaphragm connections, i.e., welded or riveted, type and location of deterioration, deck drainage, expansion dam type, barrier type, and other pertinent items.

2. Live load ratings of the bridge at present and after rehabilitation.

3. Fatigue-prone details, such as out-of-plane bending problem areas, cover-plated beams, remaining fatigue life with and without retrofit, fatigue problems observed during inspection, recommended retrofit for existing fatigue-prone details, and other pertinent items.

4. Proposed scope of work.

(13) For structures involving railroads, the following information shall be provided:

1. Completed Form D-4279 "Railroad Crossing Data for Design", as well as railroad right-of-way cross-sections, 152 000 mm {500 ft.} each side of proposed structure, degree of track curvature and rate of superelevation, if applicable.

2. Existing railroad drainage facilities and conditions in the vicinity of the structure site shall be investigated and described.

3. For situations in which railroads are overpassed by a highway structure, the procedures to determine track clearances are discussed in Design Manual, Part 1A, Chapter 7, Section 12 "Clearance of Track where Railroads are Overpassed by a Highway Structure", D2.3.3.4, and Publication 371, Grade Crossing Manual.

4. All contacts with the railroad companies shall be through the District Executive unless authorization is given to consultants, in which case copies of all correspondence and memoranda of meetings shall accompany submission of plans to the District.

5. A copy of the railroad company's letter of approval of acceptance regarding horizontal and vertical clearances, type of design live loading, type of steel and allowable stresses for various structural members shall be submitted with TS&L submission, as well as a request for temporary support for railroad tracks, if needed.

6. Demolition procedures including a schematic plan shall be provided for the removal of structures over or adjacent to railroads. The procedures and schematic must be coordinated with railroad representatives (see note 4 above).

(14) Copies of all available structure foundation exploration from Design Manual, Part 1, Chapters 3 and 4. (Provide a statement concerning mining in the area and any previous foundation problems, if any.)

(15) Pedestrian count and information concerning possible future development which might warrant need for sidewalks and/or pedestrian protective fence.

(16) Address problem areas so that there are no surprises at the final plan submission (kink in girders rather than curved girders, etc.). If problems or questions arise after approval is given, they should be brought to the attention of the Department.

(17) Address safety areas which are structure related and were noted at the Design Field View.

(f) Completed applicable Q/A Forms D-501, D-502, D-503 and/or D-504 (refer to Appendix A).

1.9.3.3.2 Streamlined TS&L

A Streamlined Submission, as a result of a Bridge Pro-Team meeting, shall include the information outlined in PP1.9.3.3.1(a) with signature blocks, (b) and (f).

The submission shall also include meeting minutes from the Pro-Team Bridge Scoping capturing all alternates discussed with reasoning behind decisions to pursue or exclude. All involved parties including the appropriate reviewing authorities, not just meeting attendees, should review and approve the minutes. The minutes should be made available to consultants for review during agreement advertisement for projects where consultants are used for design.

1.9.4 Foundations

1.9.4.1 GENERAL

The foundation exploration and report preparation shall be done as outlined in Chapter 6, and outlined herein.

Submit the TS&L Report and Structure Geotechnical Foundation Report for Retaining Walls, and Sound Barrier Walls concurrently.

1.9.4.2 RESPONSIBILITY

Refer to Table PP1.9-2 for the review and approval responsibility for foundations.

1.9.4.2.1 Responsibility of District

The foundation approval may be granted by the District Bridge Engineer or designee: However, input from the District Geotechnical Engineer should be considered. The District may consult BQAD and the Geotechnical Engineers and Geotechnical Engineering Section of the Materials and Testing Division about unusual cases.

The designer shall submit two sets of foundation plans to the District for approval.

1.9.4.2.2 Responsibility of BQAD

The designer shall submit to the District three sets of foundation plans for PENNDOT oversight projects and four sets for Federal oversight projects.

The District Bridge Engineer and the District Soils Engineer shall review the submission. The District Executive shall forward the submission to BQAD with the District's recommendation. The District shall submit to BQAD one set of the foundation submission for BQAD approval responsibility and two sets for FHWA approval responsibility projects.

If the District desires to revise the Consultant's recommendations, the revision shall be marked on the plans in red, with an explanation where necessary.

BQAD will review the submission and, after obtaining necessary FHWA approval, will approve it if it is found satisfactory. Submission of revised plans will be requested if necessary. The Soils and Geotechnical Engineering Section of the Materials and Testing Division may be consulted about unusual or complex foundations.

1.9.4.3 SUBMISSION REQUIREMENT

Foundation Submission requirements are divided into two categories, Standard and Streamlined. Standard Foundation submissions are required on all projects unless a Streamlined Submission is agreed upon by the District, BQAD and FHWA (if applicable).

1.9.4.3.1 Standard Foundation

The following information shall be included in the foundation submission:

(a) Foundation submission letter

The letter shall include the following for each substructure unit:

(1) Proposed bottom of footing elevation.

(2) Applicable core borings - B1, B2, etc. Identify bearing stratum. Show percentage of gross recovery and RQD for rock stratum and reasons for low gross recovery and/or RQD when applicable. For spread footing on soils, show average N value below the footing elevation to a depth equal to 1.5 times the width of the footing. For footings on piles, show average N value at least 3 m {10 ft} below the estimated pile tip.

(3) Rock and/or soil data for each layer below the footing, listed on Figures 1 and 2, used to compute the nominal foundation bearing resistance in MPa {tsf}. (See also QA Form D505)

(4) Pile type and size, pile tip reinforcement when required, rock and/or soil data for each layer below the footing, listed on Figure 3, used to compute the pile load resistance and driving method.

(5) Estimated pile tip elevation, bottom of pedestal, drilled shaft and length of socket into bedrock for each substructure unit when applicable.

(6) Caisson type, size and the rock and/or soil data for each layer below footing, listed on Figure 3, used to compute the resistance.

(7) Estimated settlement for footings on soils, fill material settlement that may affect the foundation, etc.

(8) Scour depth for each substructure unit, if stream crossing.

(9) If piles are in a corrosive environment, submit information as stated in D10.7.5.6.

(b) Foundation plans

The following information shall be shown on foundation plans:

(1) Preliminary plans (dated), including plans and elevation showing type and elevation of the bottom of the footing and elevation of pile tip, recommended for each substructure unit.

(2) Soil profile along the substructure units and longitudinal profile along the centerline of the structure (for uneven bearing stratum or when requested by the engineer).

(3) Plotted logs of core borings and boring layouts, grouped for each substructure.

(4) If settlement is a problem, a settlement control scheme or mitigation plan, after approval, shall be included in the final plans or in the proposal.

(5) Foundation investigation information: The subsurface exploration data which are used in making recommendations concerning foundations shall include an *endorsement*, including the date and signature of a qualified geotechnical engineer or engineering geologist, stating that the information, as submitted, accurately represents the conditions encountered by the test boring program, including boring logs, earth samples, rock cores, classification of materials, and depth of borings.

Furthermore, the boring log sheets that are part of the bridge plans shall have the following note and initials of the geotechnical engineer or engineering geologist on each sheet: "The classifications of the materials encountered have been verified." (Initials).

For bridge construction plans prepared by District personnel, the District Soils Engineer shall verify the accuracy of foundation data secured by drilling contractors or Department forces.

Endorsement shall be shown on the first sheet of the plotted test borings. In addition, the following note shall be shown on each test boring sheet: "This sheet is included for the convenience of the Department. Refer to Publication 408 Section 102.05 for further information."

The plotted test borings shall include all information contained on the boring logs.

(6) Pile type, size and tip reinforcement for pile-supported footings. (See also QA Form D505)

(7) Nominal size of drilled shafts, including the rock socket for footings supported on caissons.

(8) Identification of substructure unit at each test boring, elevation of bottom of footings at each test boring, and elevation of bottom of pedestals and/or drilled shafts and/or pile tip.

(9) Finished ground elevation at face of abutments or piers along the roadway, stream, or railroad.

(10) Approved "Contour Grading and Drainage Plan" for interchange areas and other areas when applicable.

(11) For foundations of structures over or along a stream, scour computation for abutments, piers and retaining walls and proposed scour countermeasures (including size and extent of riprap) calculated using finalized hydraulic data shall be included in the report.

(12) Foundation information of existing or nearby structures (type of foundation, footings elevations, sign of settlement due to scour, etc.).

(13) For bridge replacement structures over a stream or river, submit the following information on the *existing* structure:

1. Date built
2. Type of superstructure
3. Type of substructures
4. Type of foundation and piles if known and applicable
5. Bottom of footing elevation
6. Stream bed elevation
7. Waterway opening larger or smaller than proposed structure
8. Any scour or settlement due to scour
9. Debris accumulation problem
10. Containment within banks or lack of containment of upstream flood water
11. Reason for structure being replaced
12. Reason for substructure failure, if applicable
13. Approximate value of bearing pressure or pile load

For additional information concerning release of information and documents, see PP1.6.3.4.

(c) Geotechnical Report

In addition to the requirements specified in PP6.3.4.2.8, include foundation alternates studied (spread footings, piles, caissons, pedestals, etc.) including possible use of prefabricated proprietary walls, cost comparison when applicable, available driller's and Engineer's logs, and justification for the recommended foundation type, including allowable settlement or ultimate strength. If settlement is a problem, a settlement control scheme or mitigation plan, after approval, shall be included in the final plans or in the proposal.

(d) Completed Q/A Form D-505 for Foundations (refer to Appendix A).

SOIL BEARING RESISTANCE - THEORETICAL, SPT OR CPT METHODS FOR SPREAD FOOTINGS

			ABUT. 1	ABUT. 2	PIER 1	PIER 2	PIER 3
		Soil Type:					
		% Gross Recovery:					
		Average N Value:					
		Unconfined Comp. Test Result (Mpa) {tsf}:					
		Coeff. of Friction between soil & footing:					
		Soil Density:					
		D_f (mm) {ft}:					
		B (mm) {ft}:					
Show the appropriate information for A, B or C	(A) Theoretical Estimation Method:	c:					
		φ_f: f:					
		$N_\gamma N_\gamma$:					
		N_c					
		$N_q N_q$:					
	(B) SPT Method:	$\bar{N}_{corr}$:					
	(C) CPT Method:	q_c					
	q_{ult}*-STRN (MPa) {tsf}:						
	Bearing Resistance Factor φ: :						
	q_u - SERV at anticipated settlement (MPa) {tsf}:q_u - SERV						
	Estimated. Total Settlement (mm) {in}:						
	Estimated Settlement After Beam Erection (mm) {in}:Estima						
	Tolerable Net Settlement (mm) {in}:Tolerable Settlement:						

*q_{ult} is based on the assumed parameters as appropriate for cohesive or non-cohesive soils:

For all Methods:

B/L = 1.0 $(H/V)_{unfactored} = 0.25$

ground water at 1.50 B + D_f below surface

Footing in General Shear Failure Mode

These approved parameters will be used to calculate q_{ult} by the Department's Substructure Programs using the appropriate modification factors and load combinations.

Figure 1.9.4.3-1 - Soil Bearing Resistance Form

ROCK BEARING RESISTANCE - SEMI-EMPIRICAL OR ANALYTIC FOR SPREAD FOOTINGS

					ABUT. 1	ABUT. 2	PIER 1	PIER 2	PIER 3
				Bearing Stratum:					
				RQD:					
				Coeff. of Friction between rock & footing:					
				D_f (mm) {ft}:					
				B (mm) {ft}:					
Show the appropriate information for A or B	(A) Semi-Empirical Method	Use a or b		(a) RMR:					
				(b) NGI:					
				C_o					
				N_m					
	(B) Analytic Method			c:					
				φ_{fm}					
				Joint Spacing,S:					
		Closely Spaced Joints (D10.6.3.2.3aP)	Open Joints	K_e					
			Closed Joints	N_c					
				N_q:					
				N_γ:					
				γ:					
		Widely Spaced Joints (D10.6.3.2.3bP)		J:					
				N_{cr}:					
				H_p:					
				K_e:					
				q_{ult}*-STRN (MPa) {tsf}:					
				Sliding Resistance Factor ϕ_s:					
				Bearing Resistance Factorφ:					
				q_o – SERV at anticipated settlement (MPa){tsf}:					
				Estimated Total Settlement (mm) {in}:					
				Tolerable Settlement (mm) {in}:					

For Rocks defined as very poor quality in the Semi-Empirical Method use Soil Bearing Form with an equivalent soil mass.

These approved parameters will be used to calculate q_{ult} by the Department's Substructure Programs

Figure 1.9.4.3-2 - Rock Bearing Resistance Form

PILE RESISTANCE - SEMI-EMPIRICAL, SPT OR CPT METHODS FOR PILES IN SOIL

				ABUT. 1	ABUT. 2	PIER 1	PIER 2	PIER 3
			Soil Type:					
			Static Analysis Method:					
			Undrained ShearStrength, S_u:					
Show the appropriate information for A, B or C for friction and end bearing piles and caissons	(A) Semi-empirical Method:	Show the appropriate information for a, b, c or d	(a) Nordlund					
			K_δ:					
			P_d:					
			C_F:					
			δ:					
			(b) α:α:					
			(c) β:β:					
			σ'_v:σ'_v:					
			(d) $\lambda\sigma'_v$:					
	(B) SPT Method:		$\bar{N}$:					
			N_{corr}:N_{corr}:					
			D_b:D_b:					
			D:D:					
	(C) CPT Method: (Requires approval of Chief Bridge Engineer)		q_{c1}:q_{c1}:					
			q_{c2}:q_{c2}:					
			Q_s:Q_s:					

Figure 1.9.4.3-3 - Pile Resistance Form

	ABUT. 1	ABUT. 2	PIER 1	PIER 2	PIER 3
Pile Length:	_________	_________	________	________	________
Shaft Resistance:	_________	_________	________	________	________
Tip Resistance:	_________	_________	________	________	________
Total Resistance:	_________	_________	________	________	________
φ:	_________	_________	________	________	________
Group Resistance:	_________	_________	________	________	________
Is pile buckling a consideration?:	_________	_________	________	________	________
Pile Spacing:	_________	_________	________	________	________
Maximum Factored Vertical Load per Pile:	_________	_________	________	________	________
Unfactored Vertical Load per Pile:	_________	_________	________	________	________
Maximum Factored Lateral Load per Pile:	_________	_________	________	________	________
Unfactored Lateral Load per Pile:	_________	_________	________	________	________
Estimated Total Settlement:	_________	_________	________	________	________
Estimated Lateral Settlement:	_________	_________	________	________	________
Estimated Settlement Before Beam Erection:	_________	_________	________	________	________
Tolerable Settlement:	_________	_________	________	________	________

*φ may be increased to a value between that shown in Table D10.5.5-2 for the method used to calculate total resistance and the value shown for "load tests" based on the data in DM-4, Appendix F, if it applies to the site conditions under consideration.

Figure 1.9.4.3-3 - Pile Resistance Form (Continued)

	ABUT. 1	ABUT. 2	PIER 1	PIER 2	PIER 3
Pile Length (mm) {ft}	___________	____________	__________	__________	__________
Bearing Stratum:	___________	____________	__________	__________	__________
Boring Number	___________	____________	__________	__________	__________
Bottom of Footing Elevation (mm)	___________	____________	__________	__________	__________
Pile Type & Batter:	___________	____________	__________	__________	__________
Estimated Pile Length (mm) {ft}	___________	____________	__________	__________	__________
Estimated Pile Tip Elevation (mm) {ft}:	___________	____________	__________	__________	__________
Strength Resistance Factor φ:	___________	____________	__________	__________	__________
Tip resistance (kN) {kips}:	___________	____________	__________	__________	__________
Is pile buckling a consideration?:	___________	____________	__________	__________	__________
Pile Spacing (greater than 3*D?) (mm) {ft}:	___________	____________	__________	__________	__________
Maximum Factored Vertical Load per Pile:	___________	____________	__________	__________	__________
Unfactored Vertical Load per Pile:	___________	____________	__________	__________	__________
Maximum Factored Lateral Load per Pile:	___________	____________	__________	__________	__________
Unfactored Lateral Load per Pile:	___________	___________	__________	__________	__________

Figure 1.9.4.3-4 – Point Bearing Pile Resistance Form

1.9.4.3.2 Streamlined Foundation

A Streamlined Submission, as a result of a Bridge Pro-Team meeting, shall include the information outlined in PP1.9.4.3.1(a), (b), (c) and (d). PP1.9.4.3.1(c) references additional requirements in PP6.3.4.2.8. PP6.3.4.2.8(a), (b), and (d)(2) need not be furnished if borings are provided. PP6.3.4.2.8(d)(5) may be omitted if discussed in the Pro-Team meeting and documented in Pro-Team minutes

The submission shall also include a Foundation approval letter with signature blocks, to the District, listing items from DM4 PP 1.9.4.3(a) and meeting minutes capturing all alternates discussed with reasoning behind decisions to pursue or exclude. All involved parties including the appropriate reviewing authorities, not just meeting attendees, should review and approve the minutes. The minutes should be made available to consultants for review during agreement advertisement for projects where consultants are used for design.

1.9.4.4 SPECIAL CONSIDERATIONS

Based upon past experience, the following list of precautionary items is provided:

(a) Foundation in limestone/dolomite area shall be evaluated conservatively, i.e., use a smaller resistance per pile, provide grouting if necessary, etc. History of sinkhole activity must be checked.

(b) Piles or other deep foundations shall be recommended for substructure units in flood plain unless the footing will be supported on bedrock. Exceptions must be evaluated with extreme caution.

(c) Interference of inclined piles of the same and adjoining substructure units must be checked.

(d) For structure widening, watch for undercutting of existing foundation. Foundation column alternates may be considered. Similarly, foundation adjoining operating railroad or other property must be evaluated for the use of foundations column, caissons, etc., to eliminate cost of sheet piling or other similar costly measures.

(e) Foundations for non-flexible walls or substructure units must be set below the frost depth.

(f) Pile overdrive requirements may be needed for Conemaugh (clay stone and clay shale), decomposed mica schist and similar formations, if load test history indicates such a need.

1.9.4.5 FOUNDATION APPROVAL

The following items shall be included, as a minimum, in the foundation approval letter:

(a) All data outlined in PP1.9.4.3(a).

(b) Reasons for lower than normal allowable foundation pressures, pile loads, etc.

(c) Specific pile-driving method.

(d) Precautionary notes (for example, "Note that piles will terminate on limestone bedrock and considerable variation in the pile tip elevations may result").

(e) A note to the effect that a copy of the foundation approval letter is to be given to the field office for the Inspector's guidance during construction.

1.9.5 Final Review of Plans

1.9.5.1 RESPONSIBILITY

Refer to Tables PP1.9-1 and PP1.9-3 for review and approval responsibility for final plans.

The review of the final plans shall be conducted by either the District or BQAD, whichever has the final plan approval responsibility.

1.9.5.2 SUBMISSIONS

Two sets of prints of final plans, special provisions and Quality Assurance forms shall be submitted for review and approval to the District Executive for the bridge types for which the District is responsible. Three sets of prints (five sets for major, unusual, or complex bridges) of final plans, and special provisions and Quality Assurance Forms shall be submitted to the District for the bridge types for which the Chief Bridge Engineer has the responsibility for approval. The District shall submit two sets (four sets for major, unusual, or complex bridges) to BQAD for review and comments. If the District desires to revise the Consultant's submission, the revision shall be marked on the prints in red prior to being forwarded to BQAD. BQAD will send a set (three sets for major, unusual, or complex bridges) to FHWA for its review and comments when applicable.

For FAST and special projects, partial submissions, such as superstructure or substructure, may be made for early input and comments so that major items are resolved before plans are finalized.

The Quality Assurance forms submitted shall be all the required applicable formed in accordance with Appendix A. A copy of all calculations pages required by the form shall also be attached to each form. Additional calculations may be required upon request of the reviewing office.

1.9.5.2.1 Plan Review by Consulting Engineers

The District shall provide one set of review plans, special provisions, and design computations and the required applicable Quality Assurance forms in accordance with Appendix A to the review consulting engineer. Direct communication and correspondence between the design and review consultants shall be permitted, provided that copies of correspondence are forwarded to both the District and BQAD. Any deviation from standard design practices, design criteria and standards shall be approved by the Chief Bridge Engineer prior to its acceptance by the review engineer.

1.9.5.3 CHECKLIST OF MINIMUM ITEMS

In addition to the items included in PP1.6, the following list of minimum items is provided for uniformity and as a reminder:

(a) All pertinent items included in TS&L and Foundation submissions. The sheet or sheets of plotted core borings shall be the last sheet or sheets in the set of structure plans.

(b) Applicable general notes, quantities in the prescribed format (See Standard Construction Items Catalogue for appropriate unit measures), including utility installation items and alternate bid items, table of deck elevations, etc. All bridges shall have alternate bid item unless prior approval for one design is secured at the TS&L stage.

(c) All new bridges and new bridge superstructures (for rehabilitation projects) shall be bid lump sum. However, items below footings shall be bid on a unit price basis, except test piles, which will be a lump sum item. In addition, where quantities can be well defined, items may be bid lump sum. In case of conflict, the Chief Bridge Engineer shall be contacted.

(d) All design computations shall be submitted at this stage, and shall be completely checked with an index.

(e) All drawings shall be thoroughly checked for correctness and accuracy and shall be initialed by the designer and checker.

(f) Bridge type, size, location and foundation details shall match approvals.

(g) Foundation bearing pressures, axial and lateral pile/caisson loads, and the horizontal force for checking against sliding shall be shown for the controlling condition for each substructure unit. Indicate the controlling limit state, whether maximum values control, whether temporary or final conditions control, and the factored force effects (i.e., factored bearing pressure, factored pile axial load) and resistance values associated with the controlling conditions. A summary of soil/rock properties at each layer used for design shall also be shown, including, as applicable, undrained shear strength, mass unit density, saturated unit density, cohesion, effective friction angle, and empirical rock bearing capacity.

EXAMPLE:

Factored Pile Axial Load = ________________ kip (Strength 1)*
Factored Pile Axial Resistance = ________________ kip (Strength 1)*
Factored Pile Lateral Load = ________________ kip (Strength 1)*
Factored Pile Lateral Resistance = ________________ kip (Strength 1)*

*Show actual controlling limit state.

(h) If a construction item is not a standard item covered by Publication 408, a special provision shall be prepared and submitted. Construction item terminology shall match the construction item catalog.

(i) Utility occupancy data, transportability of prefabricated structure components, inclusion of special provisions for hauling permit and review of routes for accessibility shall be provided.

(j) Moment and shear envelopes, section properties for composite designs, prestressed notes, details for live load continuity for prestressed beams including continuity diaphragms, additional deck steel in negative moment area, dowel details, no keying of continuity diaphragms, deck pouring sequence, camber diagrams, etc. shall be shown in drawings.

(k) Bearing type and size shall be provided. The tolerance values used for the bearing pad design shall be shown on the construction plans. Provisions for future superstructure jacking shall be considered. The construction plans should clearly indicate where and when jacking is required, and provisions for jacking points must be included in the design and detailing of the superstructure and substructure. The jacking forces should also be specified.

(l) When consecutively fixed piers are utilized in a design, instructions for jacking the required deflection into the piers for proper positioning of the bearings under the beams shall be shown on the drawings. A table of dimensions shall be included showing the relative distance that each pier must be moved for each 5º C {10º F} temperature variation from the mid-range of the anticipated temperature extremes.

The theoretical fixed point on the bridge, based on the relative stiffness and heights of the piers that are fixed, shall also be shown on the drawings.

(m) Details for expansion dams, manufacturing and installation, shall match the standards unless approved by the Chief Bridge Engineer.

(n) Prestressed adjacent box beams: Bearing area should follow the deck cross slope; however, possibility of beam twisting should be watched by comparing seat cross slope at each end of the beam. Longitudinal slope should match the combination of grade and camber.

Prestressed spread box beams: Bearing area can be level transversely. Longitudinal slope should match the combination of grade and camber. Beveled sole plate shall be provided when longitudinal slope of the beam seat exceeds 4%, and the beam seat shall be level in the longitudinal direction. Special care shall be exercised when the bearings are parallel to a substructure unit with sharp skew; in such instances bevel shall be in two directions.

For both prestressed adjacent and spread box beams, D14.7.6.3 provides bearing area and sole plate requirements. For box beams having a transverse beam seat slope, $s_{t,}$ exceeding 5% and placed on neoprene bearing pads thicker than 90 mm {3.5 in.}, provide a note on the design drawings requiring the contractor to provide temporary lateral support to the beam during construction until the end diaphragms are cast and the shear blocks or dowel bars are installed per D14.7.6.3.8d1.2P

If an exception has been given for the deck slab overhang, include a note on the plans per D9.7.1.5.1P.

(o) Steel structures: Deck pouring sequence (watch for lateral support for compression flange); fracture control plan; Charpy V-notch requirements; identification of tension flange; diaphragm connections to girders (watch for fatigue details); no out-of-plane bending details; end rotation on skewed bridges (compensate in expansion dam movement classification unless deck block-out detail is used for the dam); direction of deck placement (skewed placement) to eliminate corner uplift; camber diagram (including differential camber between fascia and interior girders), appropriate overhang notes (see D6.10.3.2.4.2P and D9.7.1.5.1P), note per D14.7.6.3.5 if a lift-off condition is expected when beam is initially set on bearing pad, weld joint symbols, etc.

(p) Prefabricated walls: Typical foundation detail, conceptual drawing with all needed locations, dimensions and elevations, allowable foundation pressure with settlement control plan, if any, construction procedure, where required, barrier connection details, general notes, temporary shoring, where required, drainage details, abutment details, if applicable, concrete wall abutting details, and other site-specific requirements.

(q) Wall design: Clarify the use of wet or dry soil condition for wall design at the very first submission. Also, clarify how the designer validated assumptions on the construction or contract plans.

(r) Completed applicable Q/A Forms D-506 through D-518 (refer to Appendix A).

(s) Demolition procedures including a schematic plan as approved by the railroad at TS&L submission shall be provided for the removal of structures over or adjacent to railroads.

1.9.6 Final Plans

1.9.6.1 RESPONSIBILITY

Refer to Tables PP1.9-1 and PP1.9-3 for final plans.

1.9.6.1.1 Responsibility of District Bridge Engineer

The District may consult BQAD about unusual cases. When Federal funds are used in any phase of a project, PENNDOT oversight project procedure shall be followed.

Bridge-mounted sound barrier plans shall be approved for structural adequacy only.

1.9.6.1.2 Responsibility of BQAD

The District Executive shall submit the tracings, special provisions, one set of prints and the review prints with BQAD's comments to BQAD for approval of the bridge plans.

BQAD will review and approve the plans (tracings) and Special Provisions after satisfactory resolution of all comments, and will send prints of the approved plans, if necessary, or when requested by the District for preparing PS&E submission.

Bridge-mounted noise barrier plans shall be approved for structural adequacy only.

1.9.6.2 PLAN PRESENTATION

See PP1.6, “Plan Presentation”. If the plans are prepared by a consulting engineer, the first sheet shall be signed and stamped by a Professional Engineer registered in Pennsylvania.

For design review performed by consultants, see PP1.3.4, which provides additional requirements.

All comments from the review of the final plans (PP1.9.5) shall be addressed before the final plans are approved.

1.9.6.3 SIGNING OF BRIDGE PLANS

The first sheet shall be dated and signed by the Chief Bridge Engineer or the District Bridge Engineer, depending on whose office has signature authority. All other sheets, except core boring sheets, shall be dated only. The core boring sheets shall neither be signed nor dated, except as indicated in PP1.9.4.3(b)(5).

The following procedure shall be followed for the approval of structure plans for local projects:

1. Federally-Funded Local Projects:

   The structure plans shall be processed using appropriate (PENNDOT oversight or Federal oversight) procedures. The District Bridge Engineer should sign the first sheet of the structure plans and indicate the approval date on the remaining sheets.

2. State-Funded Local Projects:

   The structure plans shall be processed as specified in Appendix A, Publication 9. The District Bridge Engineer should sign the first sheet of structure drawings "For Structural Adequacy Only" and indicate the approval date on the remaining sheets.

If separate structure plans are not prepared, all structure-related drawings shall be distinctly separated (preferably at the end) from other drawings, such as highway plans or traffic plans, etc. In such case, the District Bridge Engineer shall sign the sheet where the structure drawings begin and indicate the approval date on the remaining structure sheets.

1.9.7 PS&E Submission

1.9.7.1 GENERAL

See Design Manual, Part 1A, Chapter 7. The bridge plans must be signed before submitting PS&E submission to the Contract Management Division.

For Bridge rehabilitation or replacement projects incorporate the following with the PS&E submission to Central Office:

Indicate the availability of the existing bridge plans on the Title Sheet of the contract plans, as shown:

Sample Title Sheet
Also Included

Interconnection Plans	4 Sheets
Traffic Signal Plans	21 Sheets
Traffic Control Plans	5 Sheets
Pavement markings & Signing Plans	9 Sheets
Structure Plans S-21004	2 Sheets
Landscaping Plans	10 Sheets
Existing Bridge Plans (Upon Request)	
S-xxxxx	__ Sheets
S-zzzzz	__ Sheets

Include one set of existing bridge plans (half-size), as well as plans of any interim work done on the existing structures. Include only those plans and/or shop drawings which have been used and are appropriate in preparing the proposed contract bridge plans.

Stamp sheet “For Reference Only” on each existing bridge plan submitted.

The existing plans (half-size) and the construction plans will be sold to the prospective bidders, upon request, by the Contract Sales Store.

1.9.7.2 RESPONSIBILITY OF DISTRICT BRIDGE ENGINEER

The District Bridge Engineer is responsible for correct item numbers, descriptions and quantities shown on the bridge plans, and this information shall be cross checked by the District with roadway plans. The Bridge Quantity Summary shall be shown on the roadway plans.

1.9.7.3 RESPONSIBILITY OF BQAD

BQAD is responsible for reviewing the structure portion of the PS&E submission in accordance with Table PP1.9-1.

1.9.8 Revisions to Contract Drawings and As-Built Plans

Refer to PP1.10.6.

1.10 BRIDGE SUBMISSIONS - CONSTRUCTION PHASE

1.10.1 Alternate Design by Contractors

Submission of applicable Q/A forms per Appendix A is mandatory without which the submission will be returned without any action by the approving office. Any delay caused by such non-compliance will be the Contractor's responsibility.

1.10.1.1 GENERAL

Alternate bridge design by contractors shall be permitted for all bridge projects unless approval is secured for one bridge type at TS&L stage from the Director, Bureau of Design. Any constraint requirements shall be included in the "AlternateBridge" special provision. See PP3.3.3 for submission requirements regarding prefabricated walls. Alternate bridge plans are considered a new set of design plans. Write the following note on the title sheet in the area of the structure (S) numbers:

"DRAWING S - ____ SUPERSEDES DRAWING S - ____"

Using the following policy for all alternate designs:

1. Significant changes to the "as-designed" structure, i.e., both superstructure and/or substructure redesign, requires the alternate designer to sign and seal the entire set of alternate drawings with no restrictions in responsibility.

2. In cases where only minor items are modified, i.e., change to specific substructure units, etc., the following note and seal combination is acceptable on the first sheet of alternate designs:

ALTERNATE DESIGN	
This alternate encompasses a Redesign of (items redesigned) Sheets (affected sheets) and all related items	
(Seal)	
(Signature)	Prepared by:

1.10.1.2 DETAILS FOR ALTERNATE DESIGN BY CONTRACTOR

When the Contractor chooses to bid an alternate bridge based on his/her own design, the tracings of the original design shall be provided by the District Office, after the project is awarded, upon request. After the conceptual approval is secured, the Contractor may change the original tracings to reflect the alternate design, or may develop completely new drawings on reproducible linens or drafting film. The redesign plans will go through an approval process before actual construction begins, as explained in the bid proposal. The original design computations shall be loaned to the Contractor, upon request, when the tracings of the original design are obtained. These computations shall be returned when the alternate design computations are submitted with the alternate design plans. The alternate design by the Contractor shall be submitted to the pertinent Engineering District Executive. Based upon the approval authority for the original design, the District Executive shall process the submission as shown in the following chart:

Chart for Review of Alternate Design Developed by Contractors

Original plans signed by Chief Bridge Engineer		Original plans signed by District Bridge Engineer	
PP1.10.1.2.1.1 In-house review and approval	PP1.10.1.2.1.2 Consultant review and approval	PP1.10.1.2.2.1 In-house review and approval	PP1.10.1.2.2.2 Consultant review and approval

1.10.1.2.1 Federal Oversight Projects and Major, Unusual or Complex Projects

1.10.1.2.1.1 In-House Review and Approval

The following procedure shall be used:

(a) The Contractor shall submit three sets (five sets for major, unusual and complex projects) of the conceptual design plans to the District Executive, of which two sets (four sets for major, unusual or complex projects) shall be sent to the Chief Bridge Engineer for approval, and one copy will be retained in the District Office. One set (three sets for major, unusual or complex projects) shall be sent to the FHWA for review and concurrence. The conceptual design shall include all basic details for the proposed bridge, plus the design methodology, including the type of computer program that will be used in the design, construction sequencing and any concepts or details not covered in design and construction specifications or standards, or practice not commonly used in Pennsylvania. A list of major items that deviate from the "as-designed" plans shall be attached to the submission. All applicable Q/A forms, D-501 through D-505, shall also be completed and submitted (refer to Appendix A).

(b) The District Office shall submit general comments to the Chief Bridge Engineer to help in the decision on acceptance of the alternate. After the conceptual design is approved by the Chief Bridge Engineer, two sets (four sets for major, unusual, or complex projects) of detailed design plans and computations shall be submitted (when requested) to the Chief Bridge Engineer for review and approval. For major, unusual or complex projects, three sets shall be forwarded to FHWA for review and concurrence. Partial design plans and computations may be submitted for approval to expedite the project. All applicable Q/A forms, D-506 through D-518, shall also be completed and submitted (refer to Appendix A).

(c) After each partial submission is approved, the Contractor shall submit 12 sets of approved drawings for distribution. These 12 sets shall be stamped "Recommended for Construction", signed by the Chief Bridge Engineer or his designee, and distributed as follows:

1. six sets to Contractor
2. three sets to District
3. one set to consultant that developed the alternate design
4. two sets for Bridge Division file, including one set for FHWA

(d) Upon design completion, the Contractor shall submit the tracings and computations to the Chief Bridge Engineer for signature, including all applicable Q/A forms completed and submitted under (a) and (b) above. The first sheet of the tracings shall bear the name of the consultant that developed the alternate design and shall bear the professional seal and signature of the engineer responsible for the design.

(e) Once the plans have been signed, one set of half-size prints shall be sent to the Contractor and one set (three sets for major, unusual and complex projects) shall be sent to FHWA. The Bridge Division will keep one set for the files. A set of microfilms will be made and kept by the Bureau of Design, Plans, Records and Reproductions section.

(f) The tracings and computations shall be sent to the pertinent District Office for additional copy distribution and holding until construction is completed.

(g) After the project is completed and tracings are revised "as constructed", the District shall send the tracings to the Bureau of Design for microfilming.

(h) The Bureau of Design will send microfilms and tracings back to the District for permanent storage.

1.10.1.2.1.2 Consultant Review and Approval

The following procedure shall be used:

(a) The Contractor shall submit four sets (six sets for major, unusual and complex projects) of the conceptual design plans to the District Executive, of which two sets (four sets for major, unusual or complex projects) shall be sent to the Chief Bridge Engineer, one set to the review consultant, and one set will be retained in the District. The BQAD will send one set (three sets for major, unusual or complex projects) to FHWA for review and concurrence. See PP1.10.1.2.1.1(a) for items to be included in the submission by the Contractor.

(b) The Consultant will review and recommend the conceptual design to the Chief Bridge Engineer for approval. The District Office shall send general comments to the Chief Bridge Engineer to help in the final decision on alternate acceptance.

(c) After the conceptual design has been approved by the Chief Bridge Engineer, the Contractor shall submit four sets (six sets for major, unusual or complex projects) of detailed plans and computations (if requested): one set to the Chief Bridge Engineer, one set to the District Office, one set to FHWA (three sets for major, unusual or complex projects) and one set to the review consultant for approval. Partial plans and computations may be submitted to expedite the project. All applicable Q/A forms, D-506 through D-518, shall also be completed and submitted (refer to Appendix A).

(d) The review consultant will review and approve the partial submission. Upon approval by the consultant, the Contractor shall submit to the Chief Bridge Engineer 13 sets for Department acceptance and distribution. These 13 sets shall be stamped "Recommended for Construction", signed by the Chief Bridge Engineer, and distributed as follows:

1. six sets to Contractor
2. three sets to District
3. one set to consultant that developed the alternate design
4. one set to review consultant
5. two sets for Bridge Division file, including one set for FHWA

(e) After all partial submissions have been reviewed and approved, the Contractor shall submit tracings and computations to the review consultant for final verification.

(f) The review consultant shall send the tracings, after stamping them according to PP1.3.4, to the Chief Bridge Engineer with a letter indicating that the alternate design is satisfactory and recommending approval of the tracings. The first sheet of the tracings shall bear the name of the consultant who developed the alternate design and shall bear the professional seal and signature of the engineer responsible for the design.

The computations and all applicable Q/A forms completed and submitted under (a) and (c) above shall be sent directly to the District by the review consultant.

(g) The Chief Bridge Engineer, upon approval of the plans, will send one set of half-size prints to the Contractor and one set (three sets for major, unusual or complex projects) to FHWA, and will keep one set for the office. A set of microfilms will be made and kept by the Bureau of Design, Engineering Support Division. The tracings will be sent to the District for distribution of additional copies and storage until the project is completed.

(h) After the project is completed and tracings are revised to "as constructed", the District shall send the tracings to the Bureau of Design for microfilming.

(i) The Bureau of Design will send microfilms and tracings back to the District for permanent storage.

1.10.1.2.2 PENNDOT Oversight Projects

1.10.1.2.2.1 In-House Review and Approval

The following procedure shall be used:

(a) The Contractor shall submit two sets of the conceptual design plans to the District Executive for approval. See PP1.10.1.2.1.1(a) for items to be included in the submission by the Contractor.

If the Contractor's design employs a sophisticated design method or software, or includes superbeams or any unusual features or unusual foundation, the submission shall be sent to the Chief Bridge Engineer for approval. The approval procedure for Federal oversight projects shall be followed thereafter, unless directed otherwise by the Chief Bridge Engineer.

(b) Upon approval of the conceptual design by the District Executive, one copy of the conceptual approval shall be sent to BQAD for information and the file, and the Contractor shall submit two sets of prints of the final design and computations to the District for review and approval. All applicable Q/A forms, D-506 through D-518, shall also be completed and submitted (refer to Appendix A).

(c) After the final plans have been approved, the Contractor shall submit tracings and computations to the District Executive for signature by the District Bridge Engineer. The first sheet of the tracings shall bear the name of the consultant that developed the alternate design and shall bear the professional seal and signature of the engineer responsible for the design.

(d) After the tracings have been approved, the District shall distribute 11 sets of prints of the plans as follows:

1. six sets to Contractor
2. three sets for District use
3. one set to consultant that developed the alternate design
4. one set to BQAD for the file

The District, at this point, can make a set of copies for its use and send the tracings to Bureau of Design for microfilming. After microfilming, the tracings will be sent back to the District for revision during construction and for completion to "as constructed" plans. The Bureau of Design will hold the microfilms until "as constructed" plans are microfilmed as indicated below.

(e) After the project is completed and tracings are revised "as constructed", the District shall send the tracings to the Bureau of Design for microfilming.

(f) The Bureau of Design will send the microfilms and tracings back to the District for permanent storage.

1.10.1.2.2.2 Consultant Review and Approval

The following procedure shall be used:

(a) The Contractor shall submit three sets of the conceptual design plans to the District Executive, of which one set shall be sent by the District to the review consultant. See PP1.10.1.2.1.1(a) for items to be included in the submission by the Contractor.

If the Contractor's design employs a sophisticated design method or software, or includes superbeams or any unusual features or unusual foundations, the submission shall be sent to the Chief Bridge Engineer for approval. The approval procedure for Federal oversight projects shall be followed thereafter, unless otherwise directed by the Chief Bridge Engineer.

(b) The Consultant shall review and, if acceptable, recommend the conceptual design for approval to the District Executive. One copy of the conceptual approval shall be sent to BQAD for information and the file.

(c) After the conceptual design has been approved by the District Executive, the Contractor shall submit two sets of the final plans and design computations to the review consultant and one set to the District. Partial submissions may be made to expedite the review process. All applicable Q/A forms, D-506 through D-518, shall also be completed and submitted (refer to Appendix A).

(d) The review consultant will review and approve partial submissions. Upon approval by the review consultant of the partial submission, the Contractor shall submit 12 sets to the District Executive for Department acceptance and distribution. These sets will be stamped "Recommended for Construction", signed by the District Bridge Engineer, and distributed as outlined in Step (f) below.

(e) After all partial submissions have been reviewed and approved, the Contractor shall submit tracings and computations to the review consultant for final verification.

(f) The review consultant shall submit the tracings, stamped according to PP1.3.4, and the original computations and all applicable Q/A forms completed and submitted under (a) and (c) above to the District Executive with a letter stating that the alternate design is satisfactory and that the review consultant recommends approval of the tracings.

(g) After the tracings have been approved, the District shall distribute 12 sets of prints of the plans as follows:

1. six sets to Contractor
2. three sets for District use
3. one set to consultant that developed the alternate design
4. one set to review consultant
5. one set to BQAD for the file

The District, at this point, can make a set of copies for its use and send the tracings to the Bureau of Design for microfilming. After microfilming, the tracings will be sent back to the District for revision during construction and for completing to "as constructed" plans. The Bureau of Design will hold the microfilms until "as constructed" plans are microfilmed as indicated below.

(h) After the project is completed and tracings are revised "as constructed", the District shall send the tracings to the Bureau of Design for microfilming.

(i) The Bureau of Design will send the microfilms and tracings back to the District for storage.

1.10.1.3 DISPOSITION OF UNUSED TRACINGS FOR ALTERNATE BRIDGE DESIGNS

While an unused alternate design is of no future use to the Department, it nevertheless represents evidence of a product received for design fees expended. It is the opinion of the Department's Chief Counsel that the microfilm of the unused alternate design plan is sufficient evidence of a product received for design fees expended. In addition, the microfilm of the unused design may be useful in future litigation.

Therefore, unused design plans shall be microfilmed and kept indefinitely. Design computations for unused designs need not be kept.

Mylar of an alternate design developed by a Contractor and used to build the project shall be sent to the Bureau of Design, to the attention of Plans, Records and Reproduction, for microfilming once the contract is completed. A 297 mm x 210 mm (Metric Size A4) or 8 ½” x 11”. (U.S. Customary Letter Size) sheet is to be included showing county, S. R., section, station, S-number, letting date, and the statement: "These plans were used for bridge construction in lieu of S____ and S___". This sheet will be microfilmed and placed on the front of the package. A different colored microfilm card will be used to identify Contractor's alternate.

When a contractor has used one of two designs supplied by the Department, only a 297 mm x 210 mm (Metric Size A4) or 8 ½” x 11”. (U.S. Customary Letter Size) sheet, similar to that specified above, should be sent to the Bureau of Design for the design used.

1.10.1.4 CHANGES IN PRESTRESSING BY CONTRACTOR

The contractor is not permitted to modify the number, size or spacing of the beams or the design methodology without bidding an alternate design. For any other deviations from the original beam design, such as changes in the prestressing, the plan revision procedures found in PP1.10.6 are to be followed during the construction phase of the project.

1.10.2 Shop Drawings

Shop drawings submitted for acceptance shall be prepared by the Contractor in accordance with the requirements of the design drawings, Department standards, contract special provisions and Publication 408, Section 105.02(d). Shop drawings shall be properly reviewed and accepted before fabrication begins. The Department requires initial submissions to be electronic portable document format (PDF) files and requires final submissions for distribution to be prints with matching PDF files, as described in Publication 408, Section 105.02(d).

1.10.2.1 SHOP DRAWING REVIEW

Technical guidelines for shop drawing review are given in Appendix B.

1.10.2.1.1 In-House Design

For all in-house designs developed by the Districts, the shop drawings shall be reviewed and approved by the District Bridge Engineer.

1.10.2.1.2 Consultant Design

The shop drawings for the Consultant's design shall be reviewed and approved as follows:

(a) By the same consultant, if the shop drawing review item is included in the original engineering agreement or in a supplemental engineering agreement

(b) By a different consultant using an open-end engineering agreement with the Districts or using a statewide open-end contract with Central Office

(c) By the District Bridge Unit Staff

Consideration shall be given to retaining the original consultant for shop drawing review when the design is complex and particularly when fracture-critical members are involved.

1.10.2.2 PURPOSE

Shop drawing review is conducted to ensure that fabrication of items is in accordance with the intent of the contract, i.e., the design drawings, standards, specifications and special provisions.

1.10.2.3 REQUIRED SHOP DRAWINGS

Unless stated otherwise in the contract special provisions, the following items routinely require submission of shop drawings:

(a) Fabricated structural steel including, but not limited to, the following:

  (1) Primary and secondary members, such as girders, trusses, beams, framing systems, cross bracing, diaphragms and stringers

  (2) Grid floors

  (3) Expansion dams and fixed dams

  (4) Railings and/or barrier, sidewalk or protective barrier

  (5) Bearings (complex and simple bearing devices)

  (6) Sign structures

(b) Pre-tensioned, pre-post-tensioned and post-tensioned concrete beams and panels

(c) Permanent metal deck forms

(d) Metal plate culverts

(e) Precast concrete culverts

(f) Precast channel beams

(g) Precast deck sections - Pretensioned, post-tensioned, or reinforced concrete

(h) Precast bridge barriers and curbs (only if permitted by the contract)

(I) Timber bridges

(j) Impact attenuators

(k) Reinforced concrete cribbing

(l) Proprietary retaining walls (reinforced earth, retained earth, doublewall, etc.)

(m) Anchored pile walls

(n) Stud details

(o) Sound barriers

(p) Light poles

(q) Protective Fence

1.10.2.4 STATEWIDE STANDARDIZATION AND QUALITY CONTROL

The Bureau of Design, Bridge Quality Assurance Division, is responsible for developing and maintaining guidelines for the review and approval of shop drawings. Unique features or deviations from standard practice should be brought to the attention of BQAD, preferably through the District Bridge Engineer. When deviations from standard practices become repetitive, BQAD will distribute acceptable deviations to the Districts for uniformity. The use of District standards is discouraged.

1.10.2.5 REVIEW PROCEDURE

The General Contractor has the responsibility to inform the Subcontractors of the name of the Department's agent for shop drawing review. Prior to commencing review, the Reviewers shall ascertain that the contract has been awarded. Furthermore, it is important that the Reviewers are in possession of all the latest contract documents, i.e., design drawings, special provisions and supplements to Publication 408. The District is responsible for furnishing the Reviewers with all pertinent contract documents. The Reviewers must also have in their possession general reference material, such as Design Manual, Part 4, Bulletin 15, and Standard Drawings.

Generally, shop drawings shall be submitted for each structure individually (structures with the same S-number). This procedure will facilitate bookkeeping and avoid confusion when microfilm records are made. Each drawing must contain a title block in the lower right-hand corner indicating the county, route, section number, segment and offset, station, contract number, name of Contractor, name of Fabricator, title of drawing, sheet number, design structure number (S-number), initials of the drawer, initials of the checker, and date of the drawing.

The Contractor is responsible for furnishing shop drawings to the Department in accordance with Publication 408, Section 105.02(d). Unless the Contractor insists on being the liaison through which the shop drawings flow from the Subcontractors to the Reviewer, the respective Fabricators may submit their shop drawings directly to the reviewing engineer contracted to do the shop drawing review.

The Contractor shall be kept informed about the progress of the review by copies of transmittal letters between the Reviewer and the respective Fabricators.

The following procedure shall be used:

(a) Review the initial submission against design plans, contract with revisions, and all addenda up to the date of review. It is recommended that a print of each electronic drawing be produced and used for review, instead of reviewing the drawings on computer screen only, to improve cross-checking. When the review is complete add comments, corrections and status label directly to PDF file (either using writer software or using reader software if the commenting and signature features were enabled by the author), or scan marked-up print to produce new PDF file. Return files to Fabricator on compact disc or upload to the Fabricator supplied FTP web site (if available). As an alternate, files may be returned by e-mail provided attachments total to no more than 2.0 MB per e-mail message. Provide electronic transmittal forms regardless of return method. Provide a status label on each drawing in one of the following forms:

1. "Initial Submission - Accepted" or

2. "Initial Submission - Accepted as Noted" or

3. " Initial Submission - Returned for Correction"

(a) Files returned to the Fabricator marked either "Accepted" or "Accepted as Noted" shall be corrected by the Fabricator as required, and prints thereof resubmitted for final distribution. Files returned to the Fabricator marked "Returned for Correction" shall be corrected by the Fabricator and then resubmitted to the Reviewer in electronic form for further review. Continue procedure until files are returned marked either “Accepted” or “Accepted as Noted” and then prints thereof shall be resubmitted for final distribution after corrections, if any, have been made.

(b) Review of prints submitted for final distribution should be conducted against design plans, contract with revisions, and all addenda up to the date of review. When the review is complete, transfer any comments and corrections to the remaining sets of prints and stamp the drawings in one of the following ways:

1. “Accepted” or

2. “Accepted as Noted” or

3. “Returned for Correction”

Only after all drawings are stamped either “Accepted” or “Accepted as Noted” are they to be distributed in accordance with Figure 1. If any of the drawings are stamped “Returned for Correction”, notify the Fabricator of the situation and arrange to have sufficient sets of replacement prints submitted and reviewed until all drawings are found satisfactory. Then distribute the drawings in accordance with Figure 1. If an ftp site folder is available, all shop drawings of a submission are to be combined into a single PDF file which is then uploaded.

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(d) Distribution of accepted shop drawings shall be in accordance with Figure 1.

SHOP DRAWING DISTRIBUTION					
ITEM	FABRICATIONS	CONTRACTORS	DIVISION OF MATERIALS AND TESTING	PRESTRESSED CONCRETE FABRICATORS	DISTRICT EXECUTIVE
Metal Deck Forms	As many sets as desired	1 set*	---	---	3 sets [a]
All Other Shop Drawings	As many sets as desired	1 set*	2 sets**	---	3 sets [a]

Δ 1 - Str. Control Engineer
1 - District Bridge Engineer
1 - Construction Project Engineer

*If a Contractor desires to have additional sets, may require the Fabricator to supply the additional sets, either directly or via the Department

**1 set is to be sent directly to the Shop Inspector

Figure 1.10.2.5-1 - Shop Drawings Distribution

(e) When there is no shop inspection, as is the case for metal plate culverts or precast concrete culverts, small rehabilitation and repair jobs, etc., copies of correspondence and/or distribution of shop drawings shall be adjusted accordingly.

(f) When a drawing is stamped accepted the only other note the reviewer may place on the drawings is as follows: This shop drawing has been reviewed for the compliance with the design concept and strict adherence to all material, technical requirements and details indicated in the contract drawings, standards, and specifications. Approval does not relieve Contractor from their responsibility for design (if applicable), detail, workmanship, dimensions, and full conformance to the contract documents.

The following additional requirements shall be met:

(a) The Fabricator shall deal directly with the Reviewer (unless the Contractor wants to be the liaison through which the work flows).

(b) For outside review, the Fabricator and Reviewer shall furnish copies of pertinent correspondence and transmittal letters to the District for monitoring purposes.

(c) All distribution prints of accepted shop drawings shall be affixed with the Reviewers acceptance stamp and distributed as required according to Figure 1.

(d) The District shall keep a log of the activities related to review and distribution of shop drawings.

1.10.2.6 PERMANENT RECORDS OF SHOP DRAWINGS

Permanent records are required of the following shop drawings:

(a) Structural steel

(b) Grid floors

(c) Expansion dams

(d) Sign structures

(e) Pretensioned and post-tensioned concrete beams

(f) Timber bridges

(g) Sound Barrier Walls

Shop drawing files shall be maintained until the job is paid off, or as the case may be, a prospective claim has been settled.

"As-built" shop drawing tracings or microfilms thereof shall be furnished by the Contractor in accordance with Publication 408, Section 105.02(d), and shall be in the Department's possession before the job is paid off.

1.10.3 Pile Hammer Approvals

Districts shall submit to BQAD, for approval, the hammer operating specifications and all other additional data specified in Publication 408. The District Structural Control Engineer or the District Bridge Engineer is responsible for approval of the steel pile tip reinforcement attachment details and splice details based upon Standard Drawing BC-757M. This standard should be included as a supplement drawing for design projects where piles are used. If a submission for pile tip reinforcement attachment details and splice details are outside the scope of the Standard Drawing BC-757M, the approval should be sent to the Chief Bridge Engineer for approval.

1.10.4 Pile Load Test Evaluations

When a pile load test is conducted, continuous liaison shall be established between the field and BQAD, at which time the test results will be evaluated.

All pertinent test results shall be forwarded to BQAD as specified in the proposal.

1.10.5 Construction Problems

Design-related bridge construction problems shall be brought to the attention of the District Bridge Engineer. The District Soils Engineer shall also be notified if the problem involves the geotechnical area of the bridge. All problems of structural integrity related to fabrication, erection, or construction items shall also be brought to the attention of the District Bridge Engineer.

The District Bridge Engineer shall approve modifications or corrections including change in foundation type, according to the approval authority of the original design. For all Federal oversight projects and major, complex or unusual structures, BQAD shall be contacted with detailed information for approval. BQAD will secure FHWA approval prior to approving the modifications.

If design error is suspected, the procedure outlined in Publication 93-C, Chapter 4.4, "Consultant Highway Design Errors", shall be followed.

If a Contractor must perform a reanalysis of a design due to deficient strength concrete, then the required applicable Quality Assurance Form D-518 (refer to Appendix A) must be submitted along with their analysis.

1.10.6 Revisions to Contract Drawings and "As-Built" Plans

Once the contract drawings are signed and sealed by the designer-of-record, no changes are permitted unless the plan revision procedures described below are followed.

1.10.6.1 REVISIONS DURING BIDDING PHASE

The District is to revise the plans to reflect changes identified in addenda during the bidding stage. Issue the necessary copies of the revised plans to the successful contractor and all affected parties.

1.10.6.2 REVISIONS DURING CONSTRUCTION PHASE

1.10.6.2(a) Revisions Due to Errors/Omissions or Field Conditions

The District will revise the contract drawings, as necessary, to correct errors or omissions uncovered during construction or to document changes necessitated by field conditions. The District will make the necessary copies and distribute the revised plans to all affected parties.

1.10.6.2(b) Revisions Due to Minor Design Changes Requested by the Contractor

For revisions to the contract drawings required due to minor, acceptable modifications of the original design (e.g., change in prestressing of a prestressed concrete bridge beam) at the request of the contractor during construction, the contractor will be responsible for making the plan revisions. In such cases, the following procedure is to be used:

- The District is to provide the tracings of the contract drawing and copies of the relevant design computations to the contractor upon request.

- The contractor must submit three sets of prints of the proposed revisions to the original drawings and three sets of supporting calculations of the modified design (if applicable), signed and sealed by a professional engineer licensed in the Commonwealth of Pennsylvania, to the District Bridge Engineer or his/her designated agent for review and acceptance. The design must be in complete compliance with the current editions of the AASHTO LRFD Bridge Design Specifications, PennDOT Publication 15M, Design Manual, Part 4, and the BD- and BC- standard drawings.

In addition, the following note and seal/signature block is to appear on the first sheet of revised contract drawings:

PLAN REVISION DESIGN	
Plan Revision R (insert revision number) encompasses a modification to the design of (insert modified component, e.g., “the prestressed concrete beams”). Sheets (list all affected sheets) and all related items.	
(Seal)	
(Signature)	Prepared by:

- The shop drawing review procedure found in PP1.10.2.5 is to be followed with regards to acceptance of the plan revision by the Department.

- Upon the Department's acceptance of the proposed plan revisions, the contractor is to return the original tracings, with the plan revisions made, to the District Office.

- The District is to make the necessary copies and distribute the revised plans to all affected parties.

- Any design computations required as part of the plan revision are to be amended to the original design computation package and recorded accordingly.

  For a plan revision involving prestressed concrete bridge beams, the items to be addressed in the submission by the contractor include, but are not limited to, revisions of the strand pattern (e.g., fully-bonded straight, straight debonded or draped), debonding lengths, drape point locations, shear stirrup reinforcement bar size and spacing, beam cambers, beam daps, structure load ratings and beam seat elevations, as necessary and as applicable.

1.10.6.3 ACCEPTANCE OF PLAN REVISIONS

The District Bridge Engineer, his designated agent, or the original designer-of-record may accept the plan revision. Note, however, that all major structure revisions for Federal oversight projects must be processed through BQAD prior to revising the plan.

1.10.6.4 DRAFTING PROCEDURES FOR PLAN REVISIONS

The original tracing are transmitted to Central Office as part of the PS&E package prior to bidding. Once a project has been awarded to the successful contractor, the tracings are returned to the District Office for the purpose of reproducing and distributing the required number of copies to the appropriate parties.

Following receipt of the original tracings from Central Office, make all revisions on the drawings by crossing out erroneous or modified information, circling the correct information and marking the corrections with an R1, R2, etc. Follow standard drafting practices for linework, lettering, etc., as outlined in Design Manual, Part 3. Use the revision block, which is typically located above the title block to denote the nature of the revision, the initials of the responsible persons for drafting, checking and recommending (usually the District Bridge Engineer) the revision, as well as the recommendation date. Refer to Figure PP1.6.1-1 or PP1.6.2-1. If required to add additional sheets, locate within original set and designate as sheet #A, B, etc.

1.10.6.5 MICROFILMING AND STORAGE REQUIREMENTS

Refer to PP1.10.1.2.2.1(e) and (f) for microfilming and storage requirements for "as-built" plans.

1.11 STRUCTURE SUBMISSIONS – DESIGN-BUILD ONE-STEP, LOW BID

1.11.1 General

Design for Design-Build One-Step Low Bid projects shall be developed in two phases, preliminary design and final design. Preliminary design will be performed by the Department or the Department's Consultant. Preliminary design will include hydrology and hydraulics; conceptual type, size and location; foundation exploration; foundation recommendations or foundation design guidelines; and specifications. Final design will be performed by the Design-Build Team which is comprised of the Contractor and the Contractor's Design Engineer. Final design will include final type, size and location; subsurface exploration and foundation design for some projects; final structure plans and specifications; and as-built plans. Amendments to the waterway permits and alternate foundation design, as applicable, will also be included in final design.

The chronology of the structure-related submissions for review and approval during preliminary and final design shall be made as follows:

Preliminary Design

1. Hydrologic and Hydraulics Report (as applicable)

2. Conceptual Type, Size and Location (Conceptual TS&L)
3. Foundation Submission
4. Bid Plans and Special Provisions

Final Design

1. Hydrologic and Hydraulics Report Amendment (as applicable)
2. Final Type, Size and Location (Final TS&L)
3. Foundation Submission (as applicable)
4. Final Structure Plans and Specifications
5. As-Built Plans

Refer to Figure 1.11.1(1) for a graphic showing the project development process for structures.

1.11.2 Preliminary Design

1.11.2.1 HYDROLOGIC AND HYDRAULICS REPORT

In accordance with PP1.9.2 and as follows:

The hydraulic analysis shall be based on the structure recommended in the Conceptual TS&L. When more than one recommended structure type is included in the Conceptual TS&L, hydraulic analyses shall be performed on the recommended structure types and the JPA should be submitted with a request to obtain a waterway permit that allows for construction of either structure.

1.11.2.2 CONCEPTUAL TS&L

1.11.2.2.1 General

The District Bridge Engineer or the DBE's representative shall attend the Design Field View for all structure projects to provide input in finalizing the location and horizontal and vertical alignment for the project, taking into account site specific conditions such as scour potential. Where complex geotechnical conditions are anticipated, the District Geotechnical Engineer/Manager shall also attend.

Design Build One-Step, Low Bid

Project Development Process for Structures

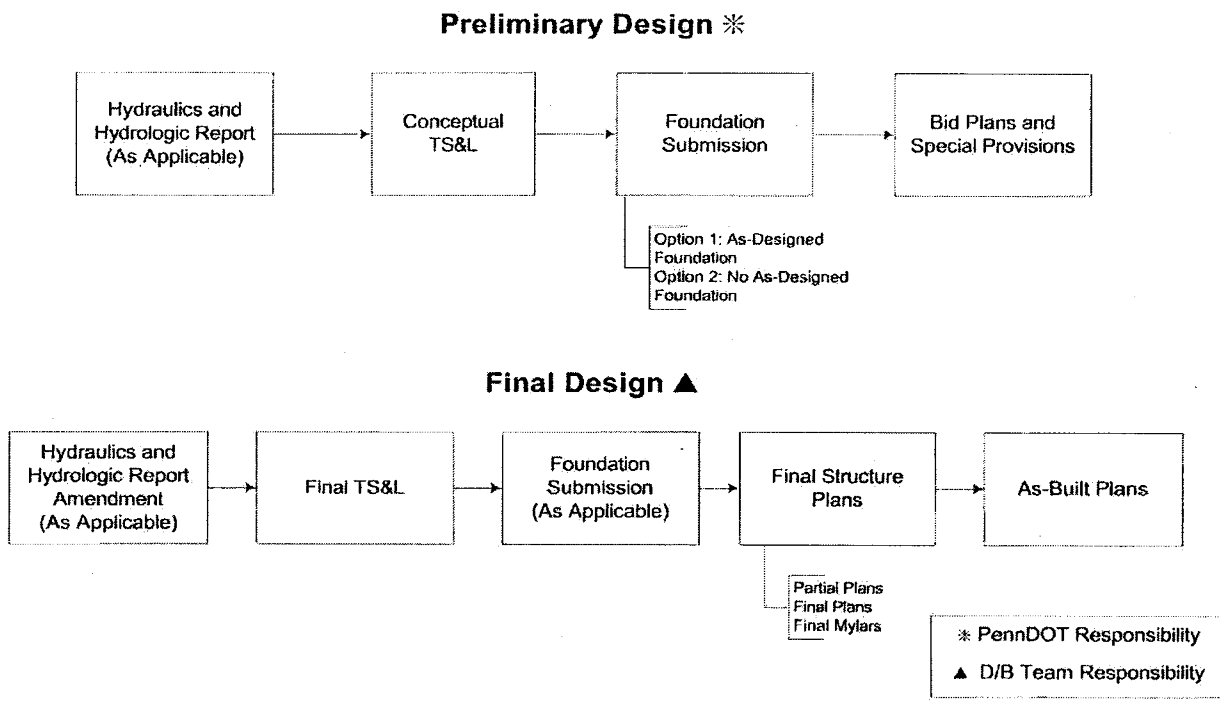

Figure 1.11.1(1) Project Development Process for Structures

The investigation of a proposed structure shall be sufficiently detailed to select and justify the type, size and location on the basis of the information available from the various phases of study outlined in Design Manual, Part 1A, including any foundation information obtained. Preliminary cost comparisons shall be made to support the Conceptual TS&L recommendations.

In general, the Conceptual TS&L submission shall be developed in accordance with P1.9.3.3 for TS&L submissions. However, certain details that are normally finalized at the TS&L stage may not need to be finalized in the Conceptual TS&L plans. For example, if a multiple span bridge is proposed with multiple pier fixity, the number of fixed piers does not need to be finalized in the Conceptual TS&L plans. In this case the Conceptual TS&L plans and/or special provisions should indicate that the number of fixed piers shall be established by the Design-Build Team subject to the design requirements in Design Manual Part 4 and the contract special provisions. Districts should contact the BQAD to obtain guidance on the level of detail where uncertainties occur on individual projects.

Conceptual TS&L plans shall be prepared for both steel and concrete structures where cost differences are insignificant, and where other project specific requirements do not justify the selection of an individual material type. Conceptual TS&L plans shall be prepared for both steel and concrete structures for all major structures.

When existing substructure units are to be retained and new loads are to be applied, the existing substructure units shall be evaluated during the Conceptual TS&L stage to assure adequate load carrying capacity for proposed conditions.

Conceptual TS&L for any structure supported on proprietary walls will not be approved until adequate foundation information including scour evaluation (if applicable) is available or the foundation investigation is completed and recommendations are available.

1.11.2.2.2 Review and Approval Responsibility

Refer to Tables PP1.9-1 and PP1.9-3 for the review and approval responsibility for the Conceptual TS&L.

1.11.2.2.2.1 District Approval

The designer shall submit two copies of the Conceptual TS&L submission to the pertinent District for approval when the District is responsible for TS&L approval.

The District shall send to the BQAD an informational copy of the Conceptual TS&L approval letter, with roadway plans, applicable Quality Assurance Forms and Conceptual TS&L plans showing the core-boring layout.

1.11.2.2.2.2 BQAD/FHWA Approval

The District Executive shall submit to BQAD, for approval, one copy of the Conceptual TS&L submission for an estimated structure cost of less than $10 million, and two sets for a structure cost over $10 million. For 100% State-funded projects, only one set is required.

District review comments to a Consultant's submission shall be marked on the plans in red and forwarded to BQAD with an explanation as appropriate. Alternatively, comments may be prepared and transmitted in a written itemized form.

BQAD will review the submission and will approve it if it is found satisfactory and after obtaining FHWA approval, when required. Submission of revised preliminary plans will be requested, if necessary.

1.11.2.2.3 Submission Requirements

Prepare a Conceptual TS&L submission for the proposed bridge. Prepare and transmit the submission in accordance with DM 4, Policies and Procedures, Chapter 1.9.3.3, other applicable sections of DM 4, Policies and Procedures. All bullet items listed in PP1.9.3.3(c)(4) must be specifically addressed. Additionally, the report shall contain a summary of design requirements for the structure shown on the Conceptual TS&L plan and for alternate structures. The summary shall be in a format such that this data can be directly integrated into the "DESIGN OF _______ , S-XXXXX" and "CONSTRUCTION OF ______, S-XXXXX" standard special provisions. Refer to PP1.11.2.4.2 for a listing of available structure-related standard special provisions for MTK projects.

Where utility facilities are to be located on a structure, the Conceptual TS&L plans shall provide sufficient details for the Design-Build Team's use in preparing the final structure plans. All materials required for each utility and the party who is to furnish and install the material shall be indicated in tabular form on the Conceptual TS&L plans. Refer to Design Manual, Part 5, Utility Relocation, Publication 16M, Chapter 7, for Utility Occupancy of Highways and Bridges, for general guidelines coordination procedure and guidelines for accommodation of utilities on structures.

1.11.2.2.4 Method of Payment for Structure Items

The method of payment for construction of the structure shall be developed as follows:

(a) New Construction, Superstructure Replacements, and Bridge Widenings

All new construction, new bridge superstructures, and bridge widenings shall be bid lump sum, except that items for which quantities are subject to site conditions and/or the Department's discretion should be bid on a unit price basis. Foundations should be paid for in accordance with PP1.11.2.3.5.

(b) Rehabilitations

All items for which quantities can be defined at the Conceptual TS&L stage shall be included in a lump sum item. This includes items such as removal of portion of existing bridge, concrete for a new deck and parapets, reinforcement bars, etc. Items for which quantities cannot be defined at the Conceptual TS&L stage shall be bid on a unit price basis. This includes items such as deck repairs, repair of deteriorated substructures, crack repair, etc. Refer to PP1.11.2.3.5 for payment methodology for foundations.

1.11.2.3 FOUNDATIONS

1.11.2.3.1 General

Two foundation development options are available for Design-Build One-Step, Low Bid projects. Either of these options may be used.

The first option involves the preparation of a complete foundation submission as per PP 1.9.4 during preliminary design. This includes a complete subsurface exploration program, recommendations for a foundation type(s), and preparation of geotechnical design parameters for use by the Design-Build Team. Recommendations for permissible alternate foundation types and the use of the as-designed foundations at relocated substructure must be provided. This design process, identified as "Option 1: As Designed Foundation Design Prepared During Preliminary Design," is defined in PP1.11.2.3.3.

The second option involves a limited foundation investigation during preliminary design and completion of the foundation investigation during final design. During preliminary design, a limited geotechnical exploration, recommendations for permissible foundation types, recommendations for geotechnical design parameter limitations, and a Foundation Design Guidance Report as per PP1.11.2.3.4.4 must be completed. During final design, the subsurface exploration must be completed and a foundation report shall be prepared by the Design-Build Team. The foundation report must include the proposed foundation type and recommended geotechnical design parameters. This process, which is intended to expedite preliminary design, is identified as "Option 2: No As-Designed Foundation Prepared During Preliminary Design." This process is defined in PP1.11.2.3.4 and its use is subject to certain limitations as described in PP1.11.2.3.4.2.

Refer to Figure 1.11.2.3.1(1) for a graphic showing the foundation development process.

1.11.2.3.2 Review and Approval Responsibility

Refer to Table PP1.9-2 for the review and approval responsibilities for foundations.

The BQAD and, as applicable, FHWA will be responsible for approving Foundation Design Guidance Reports.

1.11.2.3.2.1 District Approval

The designer shall submit two copies of the Foundation Submission to the pertinent District for approval when the District is responsible for Foundation approval. The foundation approval shall not be granted without consulting the District Geotechnical Engineer/Manager. The District may consult with the BQAD, the Geotechnical Engineers/Managers, and Geotechnical Engineering Section of the Materials and Testing Division about unusual cases.

1.11.2.3.2.2 BQAD/FHWA Approval

The designer shall submit three copies of the foundation submission for PENNDOT oversight projects and four sets for FHWA oversight projects.

The District Bridge Engineer and the District Geotechnical Engineer/Manager shall review the submission. The District shall forward the submission to BQAD with the District's recommendation. The District shall submit to BQAD one copy of the foundation submission for BQAD approval responsibility and two copies for FHWA approval responsibility projects.

District review comments to a Consultant's submission shall be marked on the plans in red and forwarded to BQAD with an explanation as appropriate. Alternatively, comments may be prepared and transmitted in a written itemized form.

BQAD will review the submission and approve it if it is found satisfactory and after obtaining necessary FHWA approval. Submission of revised data and information will be requested, if necessary. The Soils and Geotechnical Engineering Section of the Materials and Testing Division may be consulted about unusual or complex foundations.

Design Build One-Step, Low Bid

Foundation Development Process

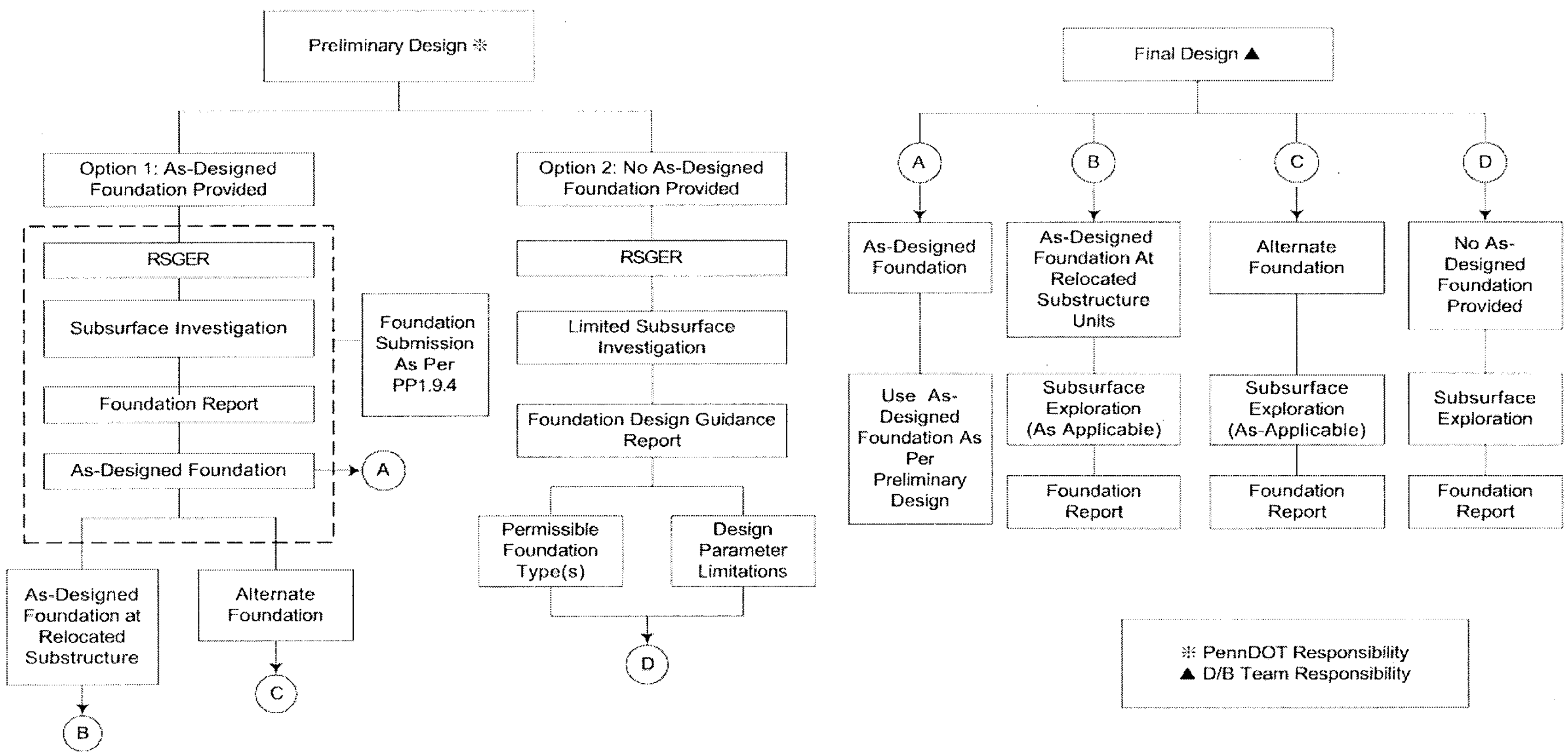

Figure 1.11.2.3.1(1) Foundation Development Process

1.11.2.3.3 Option 1: As-Designed Foundation Design Prepared During Preliminary Design

1.11.2.3.3.1 General

A subsurface exploration and foundation recommendation shall be prepared in accordance with PP1.9.4.

Foundation recommendations in the foundation report shall be summarized to allow for direct integration of this information into the applicable "DESIGN OF ________(AS-DESIGNED FOUNDATION PROVIDED), S-XXXXX" standard special provision. Foundation design parameters that are required input items in the Department's computer programs shall be provided for each substructure unit. For sound barriers, foundation design parameters required for use with the Bridge Design Standard Drawings shall be provided. Guidelines on the use of as-designed foundations at relocated substructure units and the use of alternate foundations shall be addressed in the report as follows:

<u>Relocated Substructure Units using As-Designed Foundations</u>

1. Provide recommendation on the use of as-designed foundations at substructure units relocated from the positions shown on the Conceptual TS&L plan.

2. Provide listing of project limitations. Typically, this will consist of the design and construction requirements specified for the as-designed foundation locations.

3. Provide recommendation on the need for additional test borings. Typically, it is expected that a minimum of two borings shall be provided within the footprint of each substructure unit. These can be borings from the preliminary design or additional borings drilled by the Design-Build Team. Where uniform subsurface conditions are expected, consideration can be given to reducing the extent of exploration and testing required during final design. For example, if piles are specified for the as-designed foundation and the borings taken during preliminary design show the presence of bedrock at elevations that can be interpolated between borings, the need for additional borings may be limited to that necessary to verify the top of rock elevation and rock competency at the relocated substructure unit.

<u>Alternate Foundations</u>

1. Provide recommendation for all permissible alternate foundation types.

2. Provide a listing of limitations that apply to the alternate foundations. Limitations that supercede or are not covered in AASHTO and DM 4 should be covered.

3. Provide recommendation on the need for additional test borings and laboratory testing. Typically, it is expected that a minimum of two borings shall be provided within the footprint of each substructure unit. These can be borings taken during the preliminary design or additional borings drilled by the Design-Build Team.

4. Provide recommendations for geotechnical design parameter limitations for the permissible alternate foundations. The limitations should contain sufficient information to help prevent situations where interpretation of AASHTO and/or DM 4 could result in the development of controversial or unacceptable design parameters by the Design-Build design engineer. For example, if spread footings on soil are permitted, the maximum soil friction angle that would be permitted for the design should be established. Or if a spread footing on rock is permitted, the maximum ultimate bearing capacity that would be permitted for the rock should be established.

5. Provide recommendations for subgrade preparation and construction monitoring (pile dynamic analysis, load tests, settlement monitoring, etc.).

1.11.2.3.4 Option 2: No As-Designed Foundation Design Prepared During Preliminary Design

1.11.2.3.4.1 General

The process where geotechnical design parameters are established during final design rather than preliminary design shall typically be used only where favorable subsurface conditions are present. The use of this process is subject to the limitations specified in PP1.11.2.3.4.2.

The following is required during preliminary design:

1. Reconnaissance in accordance with PP6.2, for single as well as multiple-span bridges.

2. Subsurface Exploration. Refer to PP1.11.2.3.4.3.

3. Foundation Design Guidance Report. Refer to PP1.11.2.3.4.4.

1.11.2.3.4.2 Limitations

The process where foundation design is prepared during final design rather than preliminary design shall not be used where any of the following conditions are encountered unless otherwise approved by the Chief Bridge Engineer:

1. Problematic subsurface conditions as per D10.4.4P are identified in the reconnaissance or are discovered in the preliminary design test-boring program.

2. Unusual scour conditions are present or are anticipated at the project site.

3. Structure Carrying or Crossing Over a Railroad

4. Retaining Wall Located along a Railroad

5. Mechanically Stabilized Earth Wall Abutments are proposed in the Conceptual TS&L

6. Proprietary Retaining Wall is proposed in the Conceptual TS&L

7. Culverts

In cases where special approval is requested from the Chief Bridge Engineer, the method of payment for foundations must be addressed. Consideration must be given to specifying a foundation payment method that will avoid significant risk taking by the Contractor during the bid process.

1.11.2.3.4.3 Subsurface Exploration

A limited foundation exploration shall be conducted to establish subsurface conditions within the limits of the bridge structure. As a minimum, one boring at each abutment should be obtained. For multiple span bridges, additional borings at a maximum interval of 45 meters {150 feet} between abutments shall be obtained. Preferably, a minimum of one boring shall be taken at each substructure unit as shown on the Conceptual TS&L plan.

The exploration should be performed in accordance with PP6.3, except that a Foundation Design Guidance Report as per PP1.11.2.3.4.4 shall be prepared in lieu of the report indicated in PP6.3(e). Soil and water testing as required to establish the corrosion potential at the structure site must be performed.

1.11.2.3.4.4 Foundation Submission

A Foundation Design Guidance Report shall be prepared. The report shall provide results of the subsurface exploration and recommendations of permissible foundation types. The following items shall be included in the report:

1. Brief description of each site including history, surface features, geological formation and items identified during the final exploration meeting.

2. Plotted logs of core borings and boring layouts.

3, Typewritten Engineer's logs.

4. Results of the professional engineer certified laboratory tests.

5. Endorsement of foundation investigation information and notes as per PP1.9.4.3(b)(5).

6. Recommended additional subsurface exploration to be performed during final design, including:

   (a) Minimum number of borings per substructure unit. (Typically, it is expected that a minimum of two borings shall be provided within the footprint of each substructure unit. These can be borings from the preliminary design or additional borings drilled by the Design-Build Team.)

   (b) Laboratory testing.

7. Recommended permissible foundation types and recommended geotechnical design parameter limitations for each foundation type. The geotechnical design parameter limitations should contain sufficient information to help prevent situations where interpretation of AASHTO and/or DM 4 could result in the development of controversial or unacceptable design parameters by the Design-Build design engineer. For example, if spread footings on soil are permitted, the maximum soil friction angle that would be permitted for the design should be established. If a spread footing on rock is permitted, the maximum ultimate bearing capacity that would be permitted for the rock should be established.

8. Scour Analysis

9. Recommended minimum number of test piles.

10. Recommended construction monitoring (e.g., pile dynamic analysis, load tests, settlement monitoring, etc.)

Foundation recommendations in the report shall be summarized to allow for direct integration of this information into the applicable "DESIGN OF ________(NO AS-DESIGNED FOUNDATION PROVIDED), S-XXXXX" standard special provision.

1.11.2.3.5 Method of Payment for Foundations

The method of payment for as-designed foundations, alternate foundations, and Design-Build team designed foundations shall be in accordance with the methodology specified in the "CONSTRUCTION OF ______, S-XXXXX" standard special provisions or in accordance with the methodology established in the foundation report.

1.11.2.4 BID PLANS AND SPECIAL PROVISIONS

1.11.2.4.1 Bid Plans

The Conceptual TS&L plans shall incorporate information from the foundation exploration and foundation submission.

Where an as-designed foundation is developed during preliminary design, the plans shall include information from the foundation plans, including test-boring locations, bottom of footing elevations, pile size and estimated pile tip elevations, caisson sizes, etc. Foundation design parameters for the as-designed foundations and alternate foundations shall be provided in the applicable "DESIGN OF __________ (AS-DESIGNED FOUNDATION PROVIDED), S-XXXXX" standard special provision. Test boring logs shall also be incorporated into the plans.

Where an as-designed foundation is not developed during preliminary design, the plans shall include test boring locations and test boring logs. Foundation design guidelines shall be provided in the applicable "DESIGN OF_________(NO AS-DESIGNED FOUNDATION PROVIDED), S-XXXXX" standard special provision.

1.11.2.4.2 Special Provisions

Separate design and construction special provisions shall be developed for the structure using the following standard special provisions:

New Design

- c82100 ITEM 8210-XXXX DESIGN OF BRIDGE STRUCTURE (AS-DESIGNED FOUNDATION PROVIDED), S-XXXXX
- c82110 ITEM 8211-XXXX DESIGN OF BRIDGE STRUCTURE (NO AS-DESIGNED FOUNDATION PROVIDED), S-XXXXX
- c82310 CONSTRUCTION OF BRIDGE STRUCTURE, S-XXXXX
- c82120 ITEM 8212-XXXX DESIGN OF RETAINING WALL (AS-DESIGNED FOUNDATION PROVIDED), S-XXXXX
- c82130 ITEM 8213-XXXX DESIGN OF RETAINING WALL (NO AS-DESIGNED FOUNDATION PROVIDED), S-XXXXX
- c82320 CONSTRUCTION OF RETAINING WALL, S-XXXXX
- c82140 ITEM 8214-XXXX DESIGN OF CULVERT (AS-DESIGNED FOUNDATION PROVIDED), S-XXXXX
- c82330 CONSTRUCTION OF CULVERT, S-XXXXX
- c82150 ITEM 8215-XXXX DESIGN OF SOUND BARRIER WALL (AS-DESIGNED FOUNDATION PROVIDED), S-XXXXX
- c82151 ITEM 8219-XXXX DESIGN OF SOUND BARRIER WALL (NO AS-DESIGNED FOUNDATION PROVIDED), S-XXXXX
- c82340 CONSTRUCTION OF SOUND BARRIER WALL, S-XXXXX

Bridge Rehabilitation/Widening

- c82160 ITEM 8216-XXXX DESIGN OF REHABILITATION AND/OR WIDENING OF BRIDGE STRUCTURE (AS-DESIGNED FOUNDATION PROVIDED), S-XXXXX
- c82170 ITEM 8217-XXXX DESIGN OF REHABILITATION AND/OR WIDENING OF BRIDGE STRUCTURE (NO AS-DESIGNED FOUNDATION PROVIDED), S-XXXXX
- c82350 CONSTRUCTION OF REHABILITATION AND/OR WIDENING OF BRIDGE STRUCTURE, S-XXXXX
- c82180 ITEM 8218-XXXX DESIGN OF SUPERSTRUCTURE REPLACEMENT, S-XXXXX
- c82360 CONSTRUCTION OF SUPERSTRUCTURE REPLACEMENT, S-XXXXX

Other Department standard special provisions shall be included as applicable. For example, if mechanically stabilized earth retaining walls are either specified in the Conceptual TS&L plans or permitted as an alternative wall type, then the Mechanically Stabilized Earth Retaining Wall System standard special provision shall be included in the contract special provisions.

Additional special provisions, as applicable, shall be prepared in accordance with PP 3.7.

1.11.3 Final Design

1.11.3.1 HYDROLOGIC AND HYDRAULICS REPORT AMENDMENT

Where the proposed structure span arrangement and/or substructure configuration differs from that used in the hydraulic studies prepared during preliminary design and specified in the waterway permit, the Hydrologic and Hydraulic Report and other applicable documents must be revised and approved by the Department and other agencies. Refer to PP1.9.2.3 for the amendment process.

1.11.3.2 FINAL TYPE, SIZE AND LOCATION

1.11.3.2.1 General

The Final Type, Size and Location (TS&L) submission shall be prepared during final design by the Design-Build Team. The Final TS&L structure type may be based on the Conceptual TS&L plan developed by the Department during preliminary design or another structure subject to the limitations specified in the "DESIGN OF ______, S-XXXXX" special provision.

Structure types, concepts, construction sequencing, or other details that are not covered in the design and construction specifications or standards, or practice not commonly used in Pennsylvania are permitted only when specifically indicated in the "DESIGN OF ______, S-XXXXX" special provision. Where construction that deviates from standard practice is proposed, a conceptual design shall be submitted prior to the Final TS&L for review and approval. The submittal shall contain conceptual plans, a list of items that deviate from standard design and construction, including but not limited to design methodology, the computer program that will be used in the design, construction sequencing, and any specialized construction techniques.

The following information shall be included in the TS&L submission:

1. Final TS&L submission letter: In accordance with PP1.9.3.3.1(a).

2. Final TS&L plans: In accordance with PP1.9.3.3.1(b).

3. Additional information to be supplied by the designer

   (a) Route and section number, index map and segment/offset of limits

   (b) Name of designer

   (c) Design traffic data including current and projected ADTT and class of highways on relevant roads

   (d) Date of line and grade approval and design speed (if changes are made to the as-designed vertical and horizontal alignments)

   (e) Copy of waterway approval (from Department of Environmental Protection) if an amendment to the waterway permit is necessary

   (f) Address problem areas so that there are no surprises at the final plan submission stage(kink in girders rather than curved girders, etc.). If problems or questions arise after approval is given, they should be brought to the attention of the Department.

4. Completed applicable Q/A Forms D-501, D-502, D-503 and/or D-504 (refer to Appendix A)

1.11.3.2.2 Review and Approval Responsibility

Refer to Tables PP1.9-1 and PP1.9-3 for the review and approval responsibility for the Final TS&L. Refer to PP1.11.3.5 for the review and approval process.

1.11.3.2.3 Submission Requirements

In accordance with PP1.9.3.3, except PP1.9.3.3(c) is not required.

1.11.3.3 FOUNDATIONS

As applicable, foundation design performed in final design will be covered in one of the following four categories:

1. As-Designed Foundation. Refer to PP1.11.3.3.2

2. Relocated Substructure Units Using As-Designed Foundations. Refer to PP1.11.3.3.3

3. Alternate Foundations. Refer to PP1.11.3.3.4

4. No As-Designed Foundation Provided. Refer to PP1.11.3.3.5

1.11.3.3.1 Review and Approval Responsibility

Refer to Table PP1.9-2 for the review and approval responsibility for foundations. Refer to PP1.11.3.5 for the review and approval process.

1.11.3.3.2 As-Designed Foundation

This is applicable to projects where an as-designed foundation was prepared during preliminary design. If the Design-Build Team uses the as-designed foundation with substructure units located at positions shown on the Conceptual TS&L plan, then no foundation submission is required during final design. The Design-Build Team must comply with the foundation design parameters and construction requirements specified in the DESIGN OF ______(AS-DESIGNED FOUNDATION PROVIDED), S-XXXXX and CONSTRUCTION OF ______, S-XXXXX special provisions.

1.11.3.3.3 Relocated Substructure Units Using As-Designed Foundations

This is applicable to projects where the Design-Build Team proposes to use the as-designed foundations for substructure units relocated from the positions shown in the Conceptual TS&L plans. The use of relocated substructure units using as-designed foundations will be subject to limitations established during preliminary design and specified in the "DESIGN OF ______(AS-DESIGNED FOUNDATION PROVIDED), S-XXXXX" and "CONSTRUCTION OF ______, S-XXXXX" special provisions.

1.11.3.3.4 Alternate Foundations

Alternate foundations are foundation types proposed by the Design-Build Team that differ from the as-designed foundations developed during preliminary design. Alternate foundations will be subject to the limitations established during preliminary design and specified in the "DESIGN OF ______(AS-DESIGNED FOUNDATION PROVIDED), S-XXXXX" and "CONSTRUCTION OF ______, S-XXXXX" special provisions.

1.11.3.3.5 No As-Designed Foundation Provided

This is applicable to projects where no as-designed foundation was prepared during preliminary design. Subsurface exploration, a foundation report, and construction monitoring and testing shall be performed as established during preliminary design and specified in the "DESIGN OF ______(NO AS-DESIGNED FOUNDATION PROVIDED), S-XXXXX" and "CONSTRUCTION OF ______, S-XXXXX" special provisions.

1.11.3.4 FINAL STRUCTURE PLANS AND SPECIFICATIONS

1.11.3.4.1 General

In accordance with applicable sections of DM4.

1.11.3.4.2 Review and Approval Responsibility

Refer to Tables PP1.9-1 and PP1.9-3 for the review and approval responsibility for the Final Plans. Refer to PP1.11.3.5 for the review and approval process.

1.11.3.5 SUBMITTAL REVIEW AND DISTRIBUTION

1.11.3.5.1 Submissions Approved by BQAD/FHWA

Submissions requiring BQAD and/or FHWA approval in accordance with PP1.9, Tables 1.9-1, 1.9-2, and 1.9-3, shall be submitted and reviewed as follows:

1.11.3.5.1.1 In-House Review and Approval

(a) General

Unless indicated otherwise herein or in the "DESIGN OF ______, S-XXXXX" special provision, the Design-Build Team shall make concurrent submissions (plans, reports, etc.) as follows:

1. one set to the District

2. two sets to the Chief Bridge Engineer

3. two sets to FHWA (for projects requiring FHWA approval)

The names and addresses of the persons to receive the submissions will either be listed in the "DESIGN OF ______, S-XXXXX" special provision or established at the preconstruction meeting.

(b) Coordination of Review Comments

The District, BQAD, and FHWA (as applicable) shall perform independent reviews of the submission and prepare written comments. Comments from reviewers shall be exchanged with each other, preferably via email or fax. Reviewers will discuss the comments via meeting or teleconference, and a single set of comments comprised of comments agreed upon by the reviewers will be prepared by the District. The District will send the comments to the Design-Build Team.

(c) Partial Final Structure Plans

Partial final structure plans and computations may be submitted for approval to expedite the project. Partial submission requirements and limitations, if any, will be listed in the "DESIGN OF ______, S-XXXXX" special provision. All applicable Q/A forms, D-506 through D-518, shall also be completed and submitted with each partial submission.

After each partial submission is approved, the Design-Build Team shall submit 12 sets of prints of approved drawings to the BQAD for signature and distribution. Each sheet of the prints shall bear the seal of the Design-Build Team's professional engineer responsible for the design. The first sheet of each set of prints shall also bear the signature of the professional engineer. Each sheet of the 12 sets shall be stamped "Recommended for Construction" by the BQAD. The first sheet shall be signed and dated by the Chief Bridge Engineer or his designee, and remaining sheets shall be dated. The plans will be returned to the Design-Build Team for distribution as follows:

1. six sets to Contractor

2. three sets to District

3. one set to Design-Build Team Design Engineer

4. two sets for BQAD file, including one set for FHWA

The partial final plans distribution process described above may be modified at the preconstruction meeting to meet the needs of individual projects.

(d) Final Tracings and Computations

Upon completion of design, the Design-Build Team shall submit the tracings to the Chief Bridge Engineer for signature. Each sheet of the tracings shall bear the seal of the Design-Build Team Design's professional engineer responsible for the design. The first sheet of the tracings and computations shall also bear the signature of the professional engineer. Computations, including all applicable Q/A forms shall be submitted to the District.

After the plans have been signed by the Chief Bridge Engineer, the BQAD will return the plans to the Design-Build Team. Half-size prints will be made by the Design-Build Team and distributed as follows:

1. one copy for BQAD file
2. two copies for FHWA (major, unusual, and complex projects)
3. one copy for District Bridge Unit
4. two copies for District Construction Unit

The tracings and computations shall be sent to the pertinent District Office for storage until the structure is completed.

Within three months following completion of the structure, the tracings should be retrieved by the Design-Build Team from the District. The Design-Build Team Design Engineer shall revise the tracings to incorporate as-built conditions in accordance with PP1.11.4.5. Upon acceptance of the plan revisions the District shall transmit the tracings to the Bureau of Design, Plans, Records, and Reproductions Section for archiving. The Bureau of Design will then send the archived files and tracings back to the District for permanent storage.

1.11.3.5.1.2 Consultant Review and Approval

(a) General

Unless indicated otherwise herein or in the "DESIGN OF ______, S-XXXXX" special provision, the Design-Build Team shall make concurrent submissions (plans, reports, etc.) as follows:

1. one set to the District
2. two sets to the BQAD, Chief Bridge Engineer
3. two sets to FHWA (major, unusual and complex projects)
4. two sets to the review consultant

The names and addresses of the persons to receive the submissions will either be listed in the "DESIGN OF ______, S-XXXXX" special provision or established at the preconstruction meeting.

(b) Coordination of Review Comments

The District, BQAD, FHWA (as applicable), and the review consultant shall perform independent reviews of the submission and prepare written comments. Comments from reviewers shall be exchanged with each other, preferably via email or fax. Reviewers will discuss the comments via meeting or teleconference, and a single set of comments comprised of comments agreed upon by the reviewers will be prepared by the review consultant. The review consultant will send the comments to the Design-Build Team.

(c) Partial Final Structure Plans

Partial final structure plans and computations may be submitted for approval to expedite the project. Partial submission requirements and limitations, if any, will be listed in the "DESIGN OF ______, S-XXXXX" special provision. All applicable Q/A forms, D-506 through D-518, shall also be completed and submitted with each partial submission.

The review consultant will review the partial submission in accordance with PP1.3.4. After each partial submission is approved by the review consultant and Department review personnel, the Design-Build Team shall submit 13 sets of prints of approved drawings to the review consultant. Each sheet of the prints shall bear the seal of the Design-Build Team's professional engineer responsible for the design. The first sheet of each set of prints shall also bear the signature of the professional engineer. Each sheet of the 13 sets shall be stamped by the review consultant with the appropriate stamp in accordance with PP1.3.4. The 13 sets of prints will then be transmitted to the BQAD for stamping and signature. Each sheet of the 13 sets shall be stamped "Recommended for Construction" by the BQAD. The first sheet shall be signed and dated by the Chief Bridge Engineer or his designee, and remaining sheets shall be dated. The plans will be returned to the Design-Build Team for distribution as follows:

1. six sets to Contractor
2. three sets to District
3. one copy to the Design-Build Team Design Engineer
4. one set to the review consultant
5. two sets for BQAD file, which includes one set for FHWA

The partial final plans distribution process described above may be modified at the preconstruction meeting to meet the needs of individual projects.

(d) Final Tracings and Computations

Upon completion of design, the Design-Build Team shall submit the tracings and computations to the review consultant for final verification. Each sheet of the tracings shall bear the seal of the Design-Build Team Design's professional engineer responsible for the design. The first sheet of the tracings and computations shall also bear the signature of the professional engineer.

The review consultant shall stamp the tracings according to PP1.3.4 and then transmit them to the Chief Bridge Engineer with a letter indicating that the design is satisfactory and recommending approval of the tracings.

The computations and all applicable Q/A forms completed shall be sent directly to the District by the review consultant.

After the plans have been signed by the Chief Bridge Engineer, the BQAD will return the plans to the Design-Build Team. Half-size prints will be made by the Design-Build Team and distributed as follows:

1. one copy for BQAD file
2. two copies for FHWA (major, unusual, and complex projects)
3. one copy for District Bridge Unit
4. two copies for District Construction Unit
5. one copy for the review consultant

The tracings shall be sent to the pertinent District Office for storage until the structure is completed.

Within three months following completion of the structure, the tracings should be retrieved by the Design-Build Team from the District. The Design-Build Team Design Engineer shall revise the tracings to incorporate as-built conditions in accordance with PP1.11.4.5. Upon acceptance of the plan revisions the District shall transmit the tracings to the Bureau of Design, Plans, Records, and Reproductions Section for archiving. The Bureau of Design will then send the archived files and tracings back to the District for permanent storage.

1.11.3.5.2 Submissions Approved by Districts

Submissions requiring District Bridge Engineer approval in accordance with PP1.9, Tables 1.9-1, 1.9-2, and 1.9-3, shall be submitted and reviewed as follows:

1.11.3.5.2.1 In-House Review and Approval

(a) General

Unless indicated otherwise herein or in the “DESIGN OF ______, S-XXXXX” special provision, the Design-Build Team shall make two copies of all submissions (plans, reports, etc.) to the District. The name and address of the person to receive the submissions will either be listed in the “DESIGN OF ______, S-XXXXX” special provision or established at the preconstruction meeting.

(b) Coordination of Review Comments

The District Bridge Unit shall review and provide comments to the Design-Build Team on the submission.

(c) Partial Final Structure Plans

Partial final structure plans and computations may be submitted for approval to expedite the project. Partial submission requirements and limitations, if any, will be listed in the “DESIGN OF ______, S-XXXXX” special provision. All applicable Q/A forms, D-506 through D-518, shall also be completed and submitted with each partial submission.

After each partial submission is approved, the Design-Build Team shall submit 10 sets of prints of approved drawings to the District for signature and distribution. Each sheet of the prints shall bear the seal of the Design-Build Team’s professional engineer responsible for the design. The first sheet of each set of prints shall also bear the signature of the professional engineer. Each sheet of the 10 sets shall be stamped "Recommended for Construction" by the District Bridge Unit. The first sheet shall be signed and dated by the District Bridge Engineer, and remaining sheets shall be dated. The plans will be returned to the Design-Build Team for distribution as follows:

1. six sets to Contractor

2. three sets to District

3. one set to Design-Build Team Design Engineer

The partial final plans distribution process described above may be modified at the preconstruction meeting to meet the needs of individual projects.

(d) Final Tracings and Computations

Upon completion of design, the Design-Build Team shall submit the tracings and computations to the District Bridge Engineer for signature, including all applicable completed Q/A forms. Each sheet of the tracings shall bear the seal of the Design-Build Team Design’s professional engineer responsible for the design. The first sheet of the tracings and computations shall also bear the signature of the professional engineer.

After the plans have been signed by the District Bridge Engineer, the District will return the plans to the Design-Build Team. Half-size prints will be made by the Design-Build Team and distributed as follows:

1. one copy for District Bridge Unit

2. two copies for District Construction Unit

The tracings and computations shall be sent to the pertinent District Office for storage until the structure is completed.

Within three months following completion of the structure, the tracings should be retrieved by the Design-Build Team from the District. The Design-Build Team Design Engineer shall revise the tracings to incorporate as-built conditions in accordance with PP1.11.4.5. Upon acceptance of the plan revisions the District shall transmit the tracings to the Bureau of Design, Plans, Records, and Reproductions Section for archiving. The Bureau of Design will then send the archived files and tracings back to the

District for permanent storage.

1.11.3.5.2.2 Consultant Review and Approval

(a) General

Unless indicated otherwise herein or in the “DESIGN OF ______, S-XXXXX” special provision, the Design-Build Team shall make concurrent submissions (plans, reports, etc.) as follows:

1. two sets to the District

2. two sets to the review consultant

The names and addresses of the persons to receive the submissions will either be listed in the “DESIGN OF ______, S-XXXXX” special provision or established at the preconstruction meeting.

(b) Coordination of Review Comments

The District and the review consultant shall perform independent reviews of the submission and prepare written comments. Comments from reviewers shall be exchanged with each other, preferably via email or fax. Reviewers will discuss the comments via meeting or teleconference, and a single set of comments comprised of comments agreed upon by the reviewers will be prepared by the review consultant. The review consultant will send the comments to the Design-Build Team.

(c) Partial Final Structure Plans

Partial final structure plans and computations may be submitted for approval to expedite the project. Partial submission requirements and limitations, if any, will be listed in the “DESIGN OF ______, S-XXXXX” special provision. All applicable Q/A forms, D-506 through D-518, shall also be completed and submitted with each partial submission.
The review consultant will review the partial submission in accordance with PP1.3.4. After each partial submission is approved by the review consultant and Department review personnel, the Design-Build Team shall submit 11 sets of prints of approved drawings to the review consultant. Each sheet of the prints shall bear the seal of the Design-Build Team’s professional engineer responsible for the design. The first sheet of each set of prints shall also bear the signature of the professional engineer. Each sheet of these 11 sets shall be stamped by the review consultant with the appropriate stamp in accordance with PP1.3.4 . The 11 sets of prints will then be transmitted to the District for stamping and signature. Each sheet of the 11 sets shall be stamped "Recommended for Construction" by the District Bridge Unit. The first sheet shall be signed and dated by the District Bridge Engineer or his designee, and remaining sheets shall be dated. The plans will be returned to the Design-Build Team for distribution as follows:

1. six sets to Contractor

2. three sets to District

3. one set to the review consultant

4. one set to the Design-Build Team Design Engineer

The partial final plans distribution process described above may be modified at the preconstruction meeting to meet the needs of individual projects.

(d) Final Tracings and Computations

Upon completion of design, the Design-Build Team shall submit the tracings and computations, including completed applicable Q/A forms, to the review consultant for final verification. Each sheet of the tracings and computations shall bear the seal of the Design-Build Team Design’s professional engineer responsible for the design. The first sheet of the tracings and computations shall also bear the signature of the professional engineer.

The review consultant shall stamp the tracings according to PP1.3.4 and then transmit them, along with the computations, to the District Bridge Engineer with a letter indicating that the design is satisfactory and recommending approval of the tracings. The first sheet of the tracings shall bear the professional seal and signature of the Design-Build Team Design Engineer responsible for the design.

After the plans have been signed by the District Bridge Engineer, the BQAD will return the plans to the Design-Build Team. Half-size prints will be made by the Design-Build Team and distributed as follows:

1. one copy for District Bridge Unit

2. two copies for District Construction Unit

3. one copy for the review consultant

The tracings shall be then be maintained at the District Office for storage until the structure is completed.

Within three months following completion of the structure, the tracings should be retrieved by the Design-Build Team from the District. The Design-Build Team Design Engineer shall revise the tracings to incorporate as-built conditions in accordance with PP1.11.4.5. Upon acceptance of the plan revisions the District shall transmit the tracings to the Bureau of Design, Plans, Records, and Reproductions Section for archiving. The Bureau of Design will then send the archived files and tracings back to the District for permanent storage.

1.11.3.5.3 Review Times

Review times for design submittals (Final TS&L, Final Structure Plans, etc.) will be specified in the applicable "DESIGN OF ______, S-XXXXX" special provision.

1.11.4 Construction

1.11.4.1 SHOP DRAWINGS

In accordance with PP1.10.2.

1.11.4.2 PILE HAMMER APPROVALS

In accordance with PP1.10.3

1.11.4.3 PILE LOAD TEST EVALUATIONS

In accordance with PP1.10.4

1.11.4.4 CONSTRUCTION PROBLEMS

Refer to PP1.10.5, modified as follows:

If a design error occurs, the Contractor is fully responsible for the costs associated with providing additional design analysis and construction modifications, acceptable to the Department, to correct the problem.

The Department may require reimbursement for design errors to cover engineering review costs. This amount shall be deducted from the lump sum cost for the construction of structure item via work order.

1.11.4.5 REVISIONS DURING CONSTRUCTION AND AS-BUILT PLANS

In accordance with applicable sections of DM 4, Chapter 1.10.6, except that the Design-Build Team is responsible for making changes to the contract drawings and making and distributing necessary copies of revised plans to all affected parties.

1.12 STRUCTURE SUBMISSIONS – DESIGN-BUILD BEST VALUE

1.12.1 General

Editor's Note: The policy regarding the contracting type in Section PP1.12 (name also changing) is currently under review by the Department. Until further direction is given, this contract type shall not be utilized.

The structure submission process for Design-Build Best Value (D-BBV) projects shall be established on a project-by-project basis. Guidelines for Department requirements for D-BBV projects can be found in Publication 448, Innovative Bidding Toolkit. Project specific requirements and the submittal process should be developed in conjunction with the efforts of a technical review committee. It is anticipated that much of PP1.11 Structure Submissions – Design-Build One-Step, Low Bid can be applied to D-BBV projects. Additionally, it is expected that the “DESIGN OF ______, S-XXXXX” and “CONSTRUCTION OF ______, S-XXXXX” standard special provisions as listed in PP1.11.2.4.2 can be used, in modified form, for D-BBV projects.

1.13 HAULING RESTRICTIONS AND PERMITS

1.13.1 General

Bridge members are designed in lengths, depths and widths that can be transported from the fabrication source to the project. Field splices, if required, are designed for specific locations shown on the structure drawings. The addition of field splices to shorten members, or the elimination of field splices to lengthen members, for shipping purposes, will not be approved on shop drawings without prior written approval by the District Bridge Engineer. In such cases, a sketch is required, and design computations prepared by a Registered Professional Engineer may be required. The Contractor assumes full responsibility for securing a hauling permit. Approval for elimination of a field splice at the shop drawing stage does not obligate the Department to issue a hauling permit. In either instance, no additional compensation to the Contractor will be allowed for the splice revisions.

1.13.2 Permit - Legal Loads

Note: Until State laws are changed, permit and legal load requirements are still officially in the "English" system.

State laws controlling legal size loads restrict total length to 70 feet [21 300 mm] for "any LOAD non-divisible as to length hauled on a combination of vehicles". However, a non-divisible load can exceed 70 feet [21 300 mm] if a permit is obtained. For beams in excess of 110 feet [33 500 mm] in length, the following applies:

(a) Special hauling permits may be issued for the movement on two-lane highways of overlength articles impossible to dismember, provided that the total length does not exceed 120 feet [36 500 mm].

(b) All movements having a total length in excess of 120 feet [36 500 mm] may be limited to multi-lane highways. Consideration shall be given only for short distances on two-lane highways (e.g., from the nearest port or railroad siding to the job site).

(c) Movements longer than 160 feet [48 800 mm] total length or in excess of 201 000 pounds [894 kN] gross weight are defined as superloads in Department regulations (Title 67 Pennsylvania Code, Chapter 179). Approval for such movement may be secured only by submitting a preliminary application to the Central Permit Office for preliminary approval in accordance with Title 67 Pennsylvania Code, Chapter 179.8(4)(iii).

No axle on any transporting equipment shall be permitted to exceed 27 000 pounds [120 kN], regardless of gross weight.

No request for waiver of any provisions of Title 67, Chapter 179, shall be approved, except as noted in Appendix E of this Manual. In the event that the design requires beams (non-divisible loads) longer than 110 feet [33 500 mm], the designer shall make sure that the Central Permit Office will issue a permit for such a load. Therefore, for beams over 110 feet [33 500 mm] in length, the Designer shall include in the project special provisions any special hauling restrictions applicable to that particular design, thus alerting the Contractor to those requirements.

Travel over superelevated roads or bridges shall be weighed carefully, and needed lateral shoring to a deep beam shall be provided to avoid the possibility of overturning, which can jeopardize public safety.

1.14 SYSTEMS APPROVAL

Any proprietary system must undergo a Departmental evaluation and approval process, as specified in Master Policy Statement (MPS) 418, prior to inclusion as an alternate system during the design phase or as a value engineering alternate during construction. Do not permit any new wall system during construction.

The types of wall systems permitted will be determined when the foundations are approved. When prefabricated walls are approved, the Contractor should be given an option to construct a conventional wall with the condition that secures a new foundation approval from the BQAD.

The Department bases its approval of the system and the supplier on the following considerations:

(a) The system has a sound theoretical and practical basis for the engineer to evaluate and anticipate its claimed performance. For this purpose, the supplier or his representative must submit a package containing:

1. The theory, its proponent, and the year it was proposed.

2. Where and how it was developed.

3. Experiments supporting the theory.

4. Field tests.

5. Practical application, descriptions and photographs.

6. A list of users including names, addresses, and telephone numbers.

7. Names and qualifications of the designer(s). The Designer must be a Professional Engineer.

8. Sample details of all elements, design calculations, strength and service limit state, factored force effects and nominal and factored resistance, estimated life, method of installation, and procedures for field and laboratory evaluation including instrumentation and special requirements, if any.

9. Sample specifications showing material type, quality, certifications, fabrication requirements and method, field testing and acceptance criteria.

10. Typical unit costs supported by data from actual construction. Refer to Figure 1 for procedures for new structure-related product.

(b) If precast or prefabricated material is used in the system, the Precaster or Fabricator must be approved by the Department prior to approval for construction on Department projects.

(c) The system will not be approved for frequent use unless it goes through the experimental installation and evaluation process.

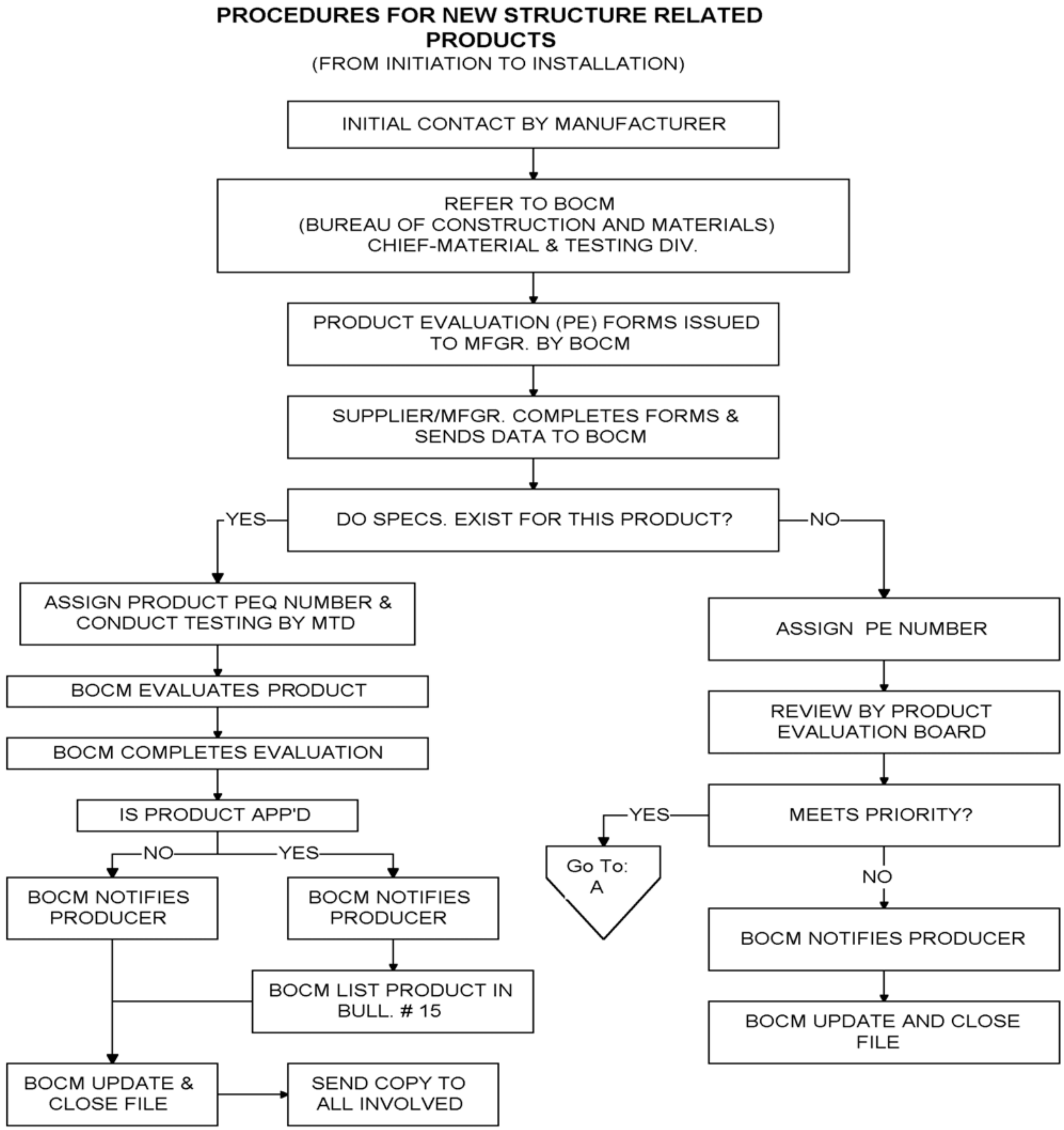

Figure 1.14-1 - Procedures for New Structure-Related Products

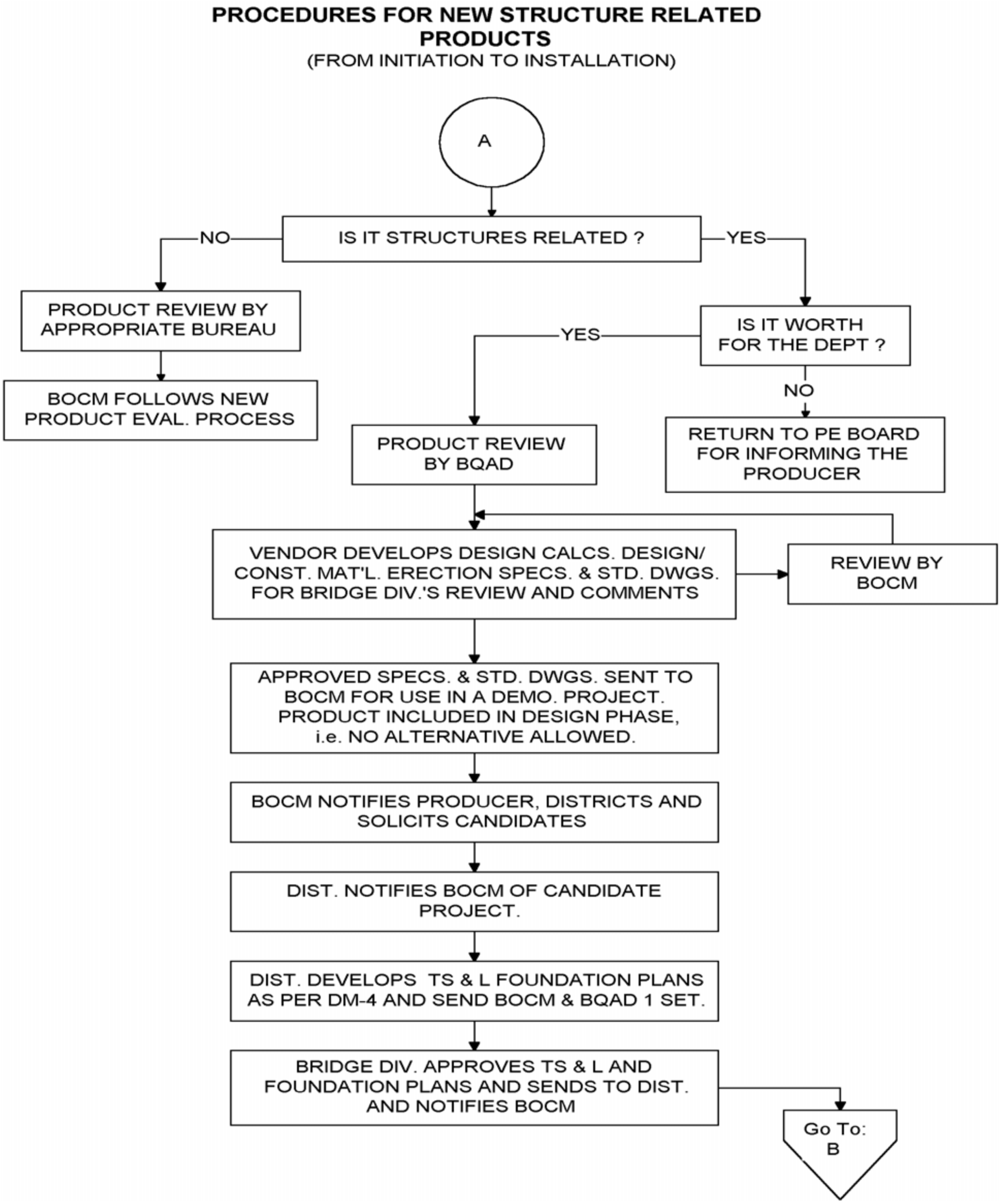

Figure 1.14-1 - Procedures for New Structure-Related Products (Continued)

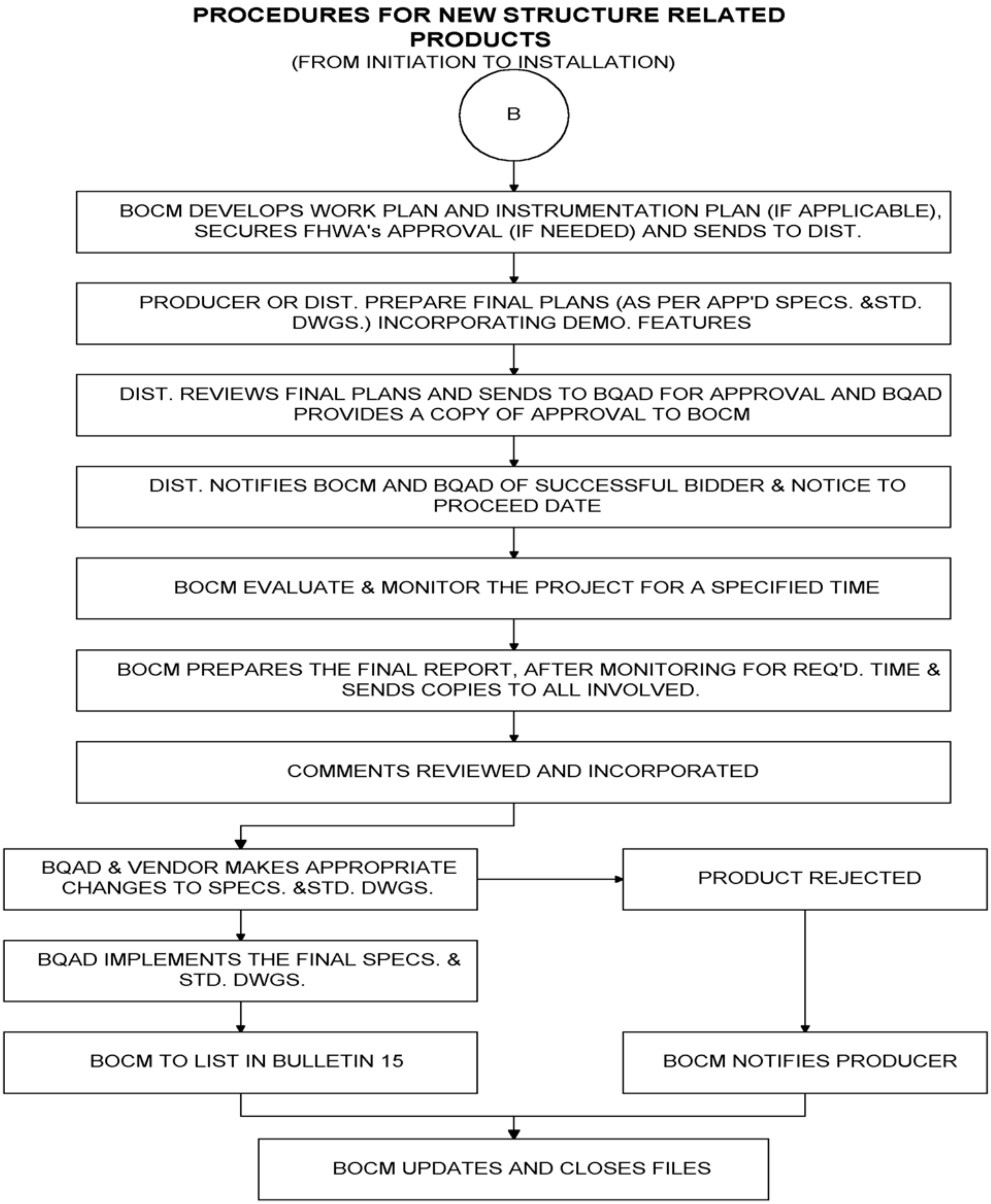

Figure 1.14-1 - Procedures for New Structure-Related Products (Continued)

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 1
PART A: POLICIES AND PROCEDURES

CHAPTER 2 - SELECTION OF BRIDGE TYPES

CHAPTER 2 - TABLE OF CONTENTS

2.0 GENERAL

Selection of bridge type is a part of the type, size and location cost study which is based on foundation information obtained from a preliminary geotechnical engineering investigation, drainage conditions, highway limitations and environmental impact both present and future in accordance with PP1.9.3.

Unless other requirements (such as environmental commitments) govern, the most economical bridge type shall be selected. Lowering the design criteria is not acceptable. Therefore, low cost can only be achieved by careful selection of details and method of construction. Short spans and low cost bridges are reflected in various standards. The minimum number of spans, joints and beams should always be used. All efforts shall be made to minimize skew as much as and whenever possible.

Unless disapproved by the Deputy Secretary for Highway Administration, alternate designs by contractors are permitted in all cases. Whenever alternate designs are not allowed, justification must be provided.

Waterway permit requests shall include a permit for a temporary bridge or a temporary crossing for construction where applicable.

Structure-related environmental commitments shall be carefully considered and justified. Consideration shall be given to citizens, groups or other interest groups which, during the environmental clearance phase, may demand restricting a structure to only one material type. In these instances, the project manager must offer industry representatives the opportunity to develop and present alternate material concepts to the public for consideration. The Department's policy is to allow a contractor alternate design for bridges in every possible situation. Depicting only a steel or a concrete or a timber structure in public meetings or environmental documents could jeopardize the alternate design/bid policy. Design options chosen for display on graphics should be labeled, "preliminary, subject to change" or other similar wording. In a situation where a commitment may be appropriate, District Bridge Engineer's input shall be obtained prior to committing a bridge type. Keep all possible options open for the designer to provide a structurally sound and economical bridge design during the final design phase. Where it is necessary to deviate from the established standards, justifications and special approvals should be well documented.

Where practical, do not commit to stone lining. Preferable alternates are colored concrete and/or texture developed by form liners. Stone lining not only restricts structural inspection of the bridge members, but also is costly to construct and maintain.

In the broad definition of prestressing of concrete members, it can indicate both pretensioning and post-tensioning of concrete members. However, in this manual, prestressing will imply pretensioning, unless otherwise noted.

2.1 STEEL BRIDGES

2.1.1 Typical Steel Bridges Used in Pennsylvania

(a) Composite steel multi-I-beam
(b) Composite steel multi-plate girder

2.1.2 Restricted Types of Construction

Do not use pin-hanger and avoid piggy-back-type of construction.

2.1.3 General Requirements

Design bolted field splices in accordance with BD-616M. For lateral bracing requirements, see BD-620M.

2.2 CONCRETE BRIDGES

2.2.1 R.C. Slab Bridges

Precast slab bridges may be used as a replacement on low class highways up to 6700 mm {22 ft.} span in accordance with Standard Drawing BD-606M.

This type of superstructure construction shall be used for widening of existing reinforced concrete slab bridges only when concrete box culvert or prestressed plank superstructure or concrete channel beam superstructure is not feasible or economical.

2.2.2 R.C. T-Beam Bridges

This type of superstructure construction may be used for widening of T-beam bridges, but is not recommended for new construction. Even for widening, consider using precast channel or prestressed box beams since cast-in-place R.C. T-Beams widening would be costly unless the work is done by Department forces.

2.3 PRESTRESSED CONCRETE BRIDGES

2.3.1 Typical Prestressed Concrete Bridges in Pennsylvania

(a) P/S adjacent box beam
(b) P/S spread box beam
(c) P/S I-beam
(d) P/S segmental (with removable deck design and bonded post-tensioning features)

2.3.2 General Requirements

Unbonded post-tensioning system is not permitted for prestressed bridges, except as a corrective measure for existing bridges with the approval of the Chief Bridge Engineer.

Prestressed beam cross-sections must be selected from Standard Drawing BD-652M. Beam cross-sections and section properties that deviate from the current standards will require specific approval from the Chief Bridge Engineer.

Some longer span P/S I-beams will be in the superbeam category as defined in PP1.13.2. The use of superbeams should be investigated during the TS&L stage and approval for their use must be obtained from the Chief Bridge Engineer. Use of superbeams is permitted in alternate designs by the Contractor if it is not restricted by contract, all hauling restrictions are obeyed and a hauling permit can be issued by the Department.

With approval of the Chief Bridge Engineer, a P/S beam fabricator may be permitted to decrease the size of box beam void to simplify fabrication or to increase the section modulus of a beam used for structures with limited underclearance, or reduce lines of beams (beds) during fabrication. This may also be utilized in original designs where utilities or other requirements dictate maximum beam spacing, provided a thorough investigation of the special beam design has been made, the outside beam dimensions have not been altered, and the stresses are within allowable limits. In order to maintain design plan dimensions in box beams and to fabricate beams conforming to the tolerance requirements of P/S beams, fabricators are encouraged to specify 12 mm {1/2 in.} undersize voids on the shop drawings. The additional 6 mm {1/4 in.} concrete around the inner perimeter of voids for box beams is not considered for section properties. However, the weight of this additional concrete shall be used in the original design.

2.4 TIMBER BRIDGES

The policy, procedure and criteria outlined in PennDOT Publication 9, Appendix A "Policies and Procedures for the Administration of the County Liquid Fuel Tax Act of 1931 and the Liquid Fuels Tax Act 655", Publication 70, "Guidelines for Design of Local Roads and Streets", and "Local Bridge Program Operational Manual", Page 7, require that AASHTO and PennDOT bridge design criteria be used. Therefore, if Federal or State or liquid fuels funding is used in any part of the project, AASHTO and PennDOT bridge design criteria must be used. Timber bridge plans must be reviewed by BQAD before final approval.

Do not approve timber bridge plans without a review by the BQAD to ensure compliance with our design criteria and avoid omissions. A variety of timber bridges are being proposed, and the design criteria of these bridges is in the developmental stage.

See A8 and D8 wood structures for design criteria.

Timber bridges may be used for:

(a) Low-speed, low-truck-volume roads (ADTT less than 25 or ADT less than 750, whichever governs)
(b) Locations where no debris exists for a condition where a timber pile pier is needed.
(c) Locations where a timber pile substructure can be constructed economically, i.e., timber piles can be driven as friction piles to sufficient depth below potential scour depth and without being damaged, to get adequate lateral support. Generally, this is a problem because of the geology of Pennsylvania.

2.4.1 Typical Timber Bridges Used in Pennsylvania

(a) Glulam - hardwood bridges
(b) Other types if approved by the Chief Bridge Engineer.

2.4.2 Geometry

Bridge width shall be in accordance with PP3.1.1.

One-lane bridges may be permitted on local roads with PMC approval if no Federal funds are used, if advanced posting of one-lane bridge is provided, if the bridge width is not less than the approach pavement width, and if the owner absolves the Department from any legal responsibility resulting from the narrow width of the bridge.

2.5 CULVERTS

The following culvert types are generally used in Pennsylvania.

2.5.1 Metal Culverts

a) Corrugated steel/aluminum pipe culvert
b) Corrugated steel/aluminum pipe arch culvert
c) Steel/aluminum plate pipe culvert
d) Steel/aluminum plate pipe arch
e) Steel/aluminum box culverts

2.5.2 Precast Concrete Culverts

a) R.C.C. pipes
b) R.C.C. elliptical pipes
c) R.C.C. box culverts

2.5.3 Cast-in-Place Box Culverts

One-cell or multiple cells

2.5.4 Cast-in-Place Frame Structures

Use when unyielding foundation is available.

2.5.5 Precast Concrete System

a) CON/SPAN Bridge System
Permitted on NHS provided the precast units are post-tensioned

b) Bebo Precast Arch Bridge System
Permit usage of all possible culvert types.

2.6 OTHER BRIDGE TYPES

2.6.1 Unusual Bridges

(a) Steel Box Girder
(b) Through or deck truss
(c) Steel or concrete arch
(d) Cable-stayed
(e) Tied-arch
(f) Suspension
(g) Two girder system (Permitted only in special condition or as a short-term temporary bridge)

(h) Rigid Frame

2.7 SELECTION OF BRIDGE TYPES

A. Small bridges (up to C15 000 mm {50 ft.} span)

1. Single or multi-unit culverts
2. Slab bridges
3. Composite concrete channel bridges
4. R.C. T-beam bridges (rarely used)
5. Timber bridges
6. Prestressed concrete planks - composite
7. Prestressed concrete adjacent box beam bridges - composite
8. Prestressed concrete spread box beam bridges
9. Prestressed concrete I-beam bridges
10. Composite steel multi-I-beam
11. Composite steel multi-plate girder (generally for larger spans)

B. Medium size bridges (up to 46 000 mm {150 ft.} clear span)

1. P/S concrete adjacent box beam-composite
2. P/S concrete spread box beam
3. P/S concrete I-Beam
4. Composite steel multi-I-beam
5. Composite steel multi-plate girder
6. Deck and through steel truss bridges (rarely used for this span range for new construction)
7. Rigid Frame

C. Large Span Bridges (from 46 000 mm to 152 000 mm {150 ft. to 500 ft.} span)

1. Steel multi-girder
2. P/S concrete segmental <u>(Removable deck and internally bonded design feature must be included)</u>
3. Concrete arch
4. Steel arch
5. Steel deck truss
6. Steel through truss
7. Steel box girder (Generally not permitted)

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 1
PART A: POLICIES AND PROCEDURES

CHAPTER 3 - DESIGN CONSIDERATIONS

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3.1 BRIDGE GEOMETRY

Considerations for bridge geometry shall take into account issues of highway safety, including sight distance, adequate clearances, and bridge widths compatible with the approach roadway.

3.1.1 Bridge Width

Unless otherwise directed, bridge width shall conform to Design Manual, Part 2, Chapter 1. This includes criteria for bridges on Very Low Volume Roads.

3.1.2 Vertical Clearance

See D2.3.3.2.

3.1.3 Lateral Clearance

See D2.3.2.2.1.

3.2 SUPERSTRUCTURE

3.2.1 Girder Bridges

Girder bridges shall have a minimum of four girders unless approved otherwise by the Chief Bridge Engineer. The girder (beam) spacing shall be selected for optimum economy. Maximum girder spacing shall be 4600 mm {15 ft.}. Four girders provide redundancy in the superstructure and will facilitate possible future redecking operations by allowing maintenance of traffic on one-half of the bridge.

A three-girder bridge with more than one traffic lane and without a floor system (floorbeams and stringers) is prohibited, except that a three-girder system for prestressed box beam superstructures may be permitted.

For three or more girders, the lateral live load distribution factors may be used as described in A4.6.2.2 and D4.6.2.2. In this case, the provisions of A3.6.1.1.2 and D3.6.1.1.2 (the multiple-lane presence factors) are already included in the factors and shall *not* be applied separately. When the lever rule is used for a three girder system, the multiple presence factor, as specified in A3.6.1.1.2, shall be applied.

In the special case where a two-girder system has been approved for design by the Chief Bridge Engineer, the fraction of live load distributed to each girder line shall be calculated by placing the loads on the bridge and summing moments about the opposite girder line. The provisions of A3.6.1.1.2 regarding multiple lanes of live load shall apply in this calculation.

3.2.2 Skew Angle

PennDOT defines skew angle as the smaller angle between the highway centerline (or a tangent thereto) and a line parallel to the support (wall, abutment, pier, etc.) or to the centerline of culverts (see Figure 1). AASHTO defines skew angle as the angle between the centerline of a support and a line normal to the roadway centerline. The sum of PennDOT's and AASHTO's skew angle is 90°, i.e.,

$$\theta_{PA} = 90 - \theta_{AASHTO} \qquad (3.2.2\text{-}1)$$

$$\theta_{AASHTO} = 90 - \theta_{PA} \qquad (3.2.2\text{-}2)$$

Except where noted, the AASHTO definition shall be used with AASHTO Specifications, and the PennDOT definition shall be used in all PennDOT documents and correspondence with the Department.

For bridges whose skew angle is less than 70°, the Engineer must submit, with the project proposal, the method to be used to analyze the structure. The analysis method selected will be part of the technical review and should be justified in the proposal.

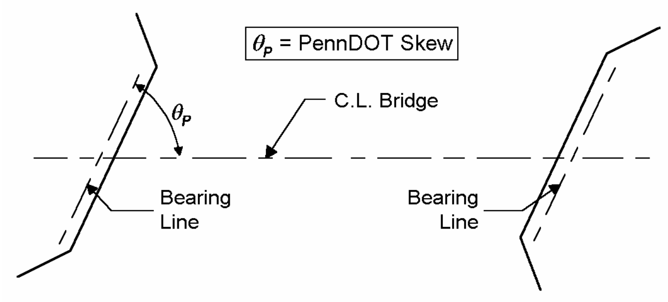


Figure 3.2.2-1 - PennDOT Skew Angle

3.2.3 Superstructure Drainage

3.2.3.1 SCUPPER LOCATION

Scuppers shall be provided where hydraulic computations show that they are needed. Type 1 or Type 2 scuppers (see applicable BC Standards) may be provided at a minimum spacing of 15 000 mm {50 ft.} for the purpose of removing saltwater and anti-skid material on flat grades less than 2%. Scuppers shall also be provided at an approximate spacing of 122 000 mm {400 ft.} for long bridges with a profile grade of 2% and over, even if they are not needed hydraulically. Type 2 scuppers shall be used only if Type 1 scuppers cannot be accommodated.

For decks supported on adjacent prestressed concrete beams, the same criteria apply, except that standard rectangular metal curb drains shall be used. For decks with sidewalks supported on prestressed adjacent box beams, downspouting through box beams may be considered in such a way that the downspouting does not adversely affect the strand pattern. For decks supported on spread box beams, either Type 1 or Type 2 scuppers or curb drains may be provided, depending upon the overhang. Metal curb drains shall not be used on decks supported on steel or prestressed concrete I-beams to avoid collection of contaminants on bottom flanges.

Free-falling scuppers shall be spaced so that the discharge is not adjacent to a substructure unit. Scuppers shall be omitted or the spacing shall be adjusted where the discharge would fall on roadways, railroad tracks, walkways, unprotected embankments, or other developed or highly erodible areas, or may be omitted entirely where their location outside the fascia beams is aesthetically unacceptable.

Flat pan scuppers should not be used because they collect debris and anti-skid material, and grow vegetation.

3.2.3.2 SCUPPER TYPES

When hydrologic and hydraulic calculations indicate the need for deck inlets, Type 1 scuppers shall be used exclusively. Since the hydraulic capacity of Type 2 scuppers is negligible, they should not be used where scuppers are needed hydraulically.

The Type 2 scuppers detailed in Standard Drawing BC-751M shall not be used unless approved by the Chief Bridge Engineer. If Type 2 scuppers are permitted, they shall be used with 203 mm {8 in.} diameter downspouting to eliminate debris accumulation and subsequent vegetative growth, and to ensure the downspouting capacity is greater than the scupper interception capacity.

3.2.3.3 END STRUCTURE DRAINAGE

Roadway inlets shall be placed off the structure at the low end(s) of bridges. Inlets shall also be placed at the high end if the approach roadway is in cut, if the possibility of drawing significant roadway water over the bridge exists, or if the roadway is curbed. These facilities shall be included with the roadway pay items and shall be shown both on bridge design drawings and on roadway plans. On the bridge drawings, these inlets shall be indicated as Roadway Pay Items.

Locating a bridge in a sag curve or in a flat grade less than 0.5% shall be avoided to the maximum extent possible, since such geometry causes clogging of inlets. If such a location cannot be avoided, the roadway runoff shall be intercepted before it reaches the bridge.

3.2.3.4 DOWNSPOUTING

Bridge scuppers shall be located to avoid long pipe runs. Pipe bends less than 135º shall be avoided when possible. For pipes sloped less than 45º, cleanout plugs shall be provided to ensure adequate maintenance. Downspouts shall be located preferably on the face of the pier which has the least exposure to the public view, but in no case shall downspouts interfere with specified vertical and horizontal clearances. Downspouts shall not be embedded in pier stems, pier columns, abutments, wingwalls, or retaining walls. Redundant systems may be utilized at the discretion of the Bridge Engineer. Longitudinal runs shall not extend below the superstructure. A minimum slope of 8% shall be provided for longitudinal pipes between scuppers or from scuppers to point of discharge. Downspouting for free-fall condition shall be extended 150 mm {6 in.} below the adjoining beam. Refer to Standard Drawing BC-751M when designing downspouting for bridge drainage.

3.2.3.5 SPLASH BLOCK

When discharge into storm drains is not practical or available, cast-in-place concrete blocks (splash blocks) 150 mm to 230 mm {6 in. to 9 in.} thick, approximately 900 mm by 900 mm {3 ft. by 3 ft.} in size, or any shape and size required, shall be placed in finished ground below the bottom of the vertical bridge drain pipe (downspout). The quantity of concrete shall be included in Class A cement concrete. Elevation at the top of the splash block at the centerline of the bridge drainpipe shall be shown on the drawings. The splash block shall be contoured and dished to contain and direct the flow.

3.2.3.6 DRAINAGE FOR REHABILITATION PROJECTS

Treatment of bridge drainage in rehabilitation projects shall be as follows:

(a) Existing flat pan scuppers shall be replaced with Type 1 scuppers in accordance with the criteria in PP3.2.3.1 and PP3.2.3.2. Flat pan scuppers shall be eliminated or their number reduced wherever possible. Existing flat pan scuppers may be retained only if Type 1 scuppers cannot be used.

(b) If drains can be located near simple supports of a span, modification of the steel beam flanges may be considered to accommodate Type 1 scuppers. In some instances, turning a Type 1 scupper by 90º with the grate remaining parallel to the flow of water, or offsetting the pipe on one side of the scupper, may also be considered to avoid interference with the flanges.

(c) For bridges rehabilitated using safety curbs and barriers:

    If the existing curb-to-curb width is adequate, the same width should be maintained and overhang cut back to keep the same gutter lines for scuppers, if possible.

    If existing scuppers must be retained and existing shoulders are widened by 150 mm {6 in.} using the safety curb and barrier, the deck shall be finished to drain into the scuppers. The grate seat shall be adjusted as needed.

3.2.3.7 DRAINAGE SYSTEM DESIGN (Hydrologic and Hydraulic Calculations)

The following items are general comments on the articles in this section:

- The scupper interception capacity formula is specified on the basis of the updated information included in FHWA Hydraulic Engineering Circular No. 22.

- Scupper spacing procedures are specified to incorporate the procedures suggested in the FHWA Report Bridge Deck Drainage Guidelines and in FHWA Hydraulic Engineering Circular No. 22.

- PP3.2.3.7.2 permits water accumulation over the pavement of a maximum 600 mm {2 ft.} width. Generally, vehicles travel beyond or about that line. This practice is permitted for narrow shoulders to utilize practically unused pavement for water and/or snow storage.

The superstructure drainage shall be designed as follows:

3.2.3.7.1 Scupper Types

Details for various types of scuppers are shown on appropriate standard drawings.

3.2.3.7.2 Scupper Location

The spacing of scuppers shall be based on the following criterion:

The width of flow in the gutter for a ten-year frequency, five-minute duration, shall not exceed the width of shoulder. (For shoulders less than 1800 mm {6 ft.} wide, a portion of the adjacent traffic lane, up to a maximum of one-fifth of the lane, may be included in the width of flow, if directed by the Department.)

3.2.3.7.3 Notation

QR = Maximum runoff on bridge deck (m^3/s) {cfs}
QG = Maximum gutter storage capacity (m^3/s) {cfs}
Q_i = Scupper interception capacity (m^3/s) {cfs}
n = Roughness coefficient of deck surface = 0.016

3.2.3.7.4 Values of QR, QG and Q

(a) Value of QR

Metric Units: $$QR = \frac{CIA}{\left(3.6 \times 10^6\right)} \qquad (3.2.3.7.4\text{-}1)$$

U.S. Customary Units: $$QR = \frac{CIA}{(43{,}560)}$$

where:

C = Runoff factor = 0.90 for bridge decks

I = Rate of rainfall for a given storm (mm/hr) {in/hr}. (This rate shall be determined in accordance with Publication 584, *PennDOT Drainage Manual,* Section 7.5.D)

A = Drainage area (m^2) {ft^2} = WL, in which W and L are the width and length of deck to be drained by the scuppers, both in meters {feet}

(b) Value of QG

The maximum gutter storage capacity shall be determined assuming the full shoulder as the width of gutter (see Figure 1). Given the roadway grade (slope of channel) and the shoulder cross slope, determine Q = QG from Figure 2.

where:

S_x = Cross slope of deck or deck shoulder

S = Grade as a function of location on bridge

T = Design spread (m) {ft.}

(c) Value of Q_i

The scupper interception capacity of the grate inlet on continuous grades shall be computed from the following relationship: relationship:

$$Q_i = (E)(Q) = Q[(R_f)(E_0) + R_S(1 - E_0)] \qquad (3.2.3.7.4\text{-}2)$$

where:

Q_i = Intercepted flow (m^3/s) {cfs}

E = Grate efficiency, dimensionless

Q = Total gutter flow (m^3/s) {cfs}

R_f = Grate inlet frontal flow interception efficiency, dimensionless (see Figures 2, 3 and 4)

R_s = Grate inlet side flow interception efficiency, dimensionless (see Figure 5)

E_o = Ratio of front flow to total gutter flow, dimensionless (see Figure 6)

When using Figure 4, use curve P-50 for grate without cross bars. Where bicycle traffic is contemplated, cross-rods should be incorporated (P-50-100 grate) in the grate.

For further information on the above relationship, refer to Hydraulic Engineering Circular No. 22, Urban Drainage Design Manual, Report No. FHWA-SA-96-078, November 1996.

The capacity of the 254 mm {10 in.} diameter downspout has been determined to be much greater than the scupper interception capacity and need not be calculated.

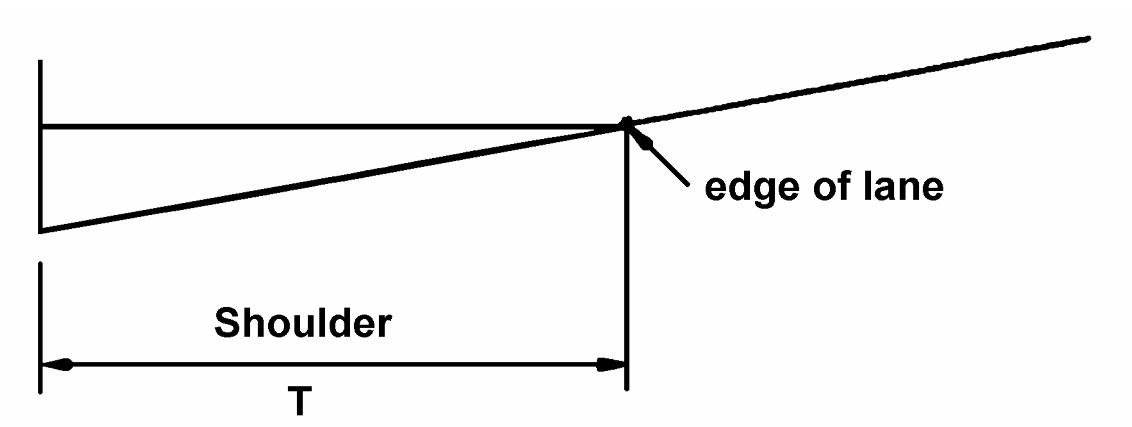


Figure 3.2.3.7.4-1 - Maximum Gutter Storage Capacity

METRIC UNITS

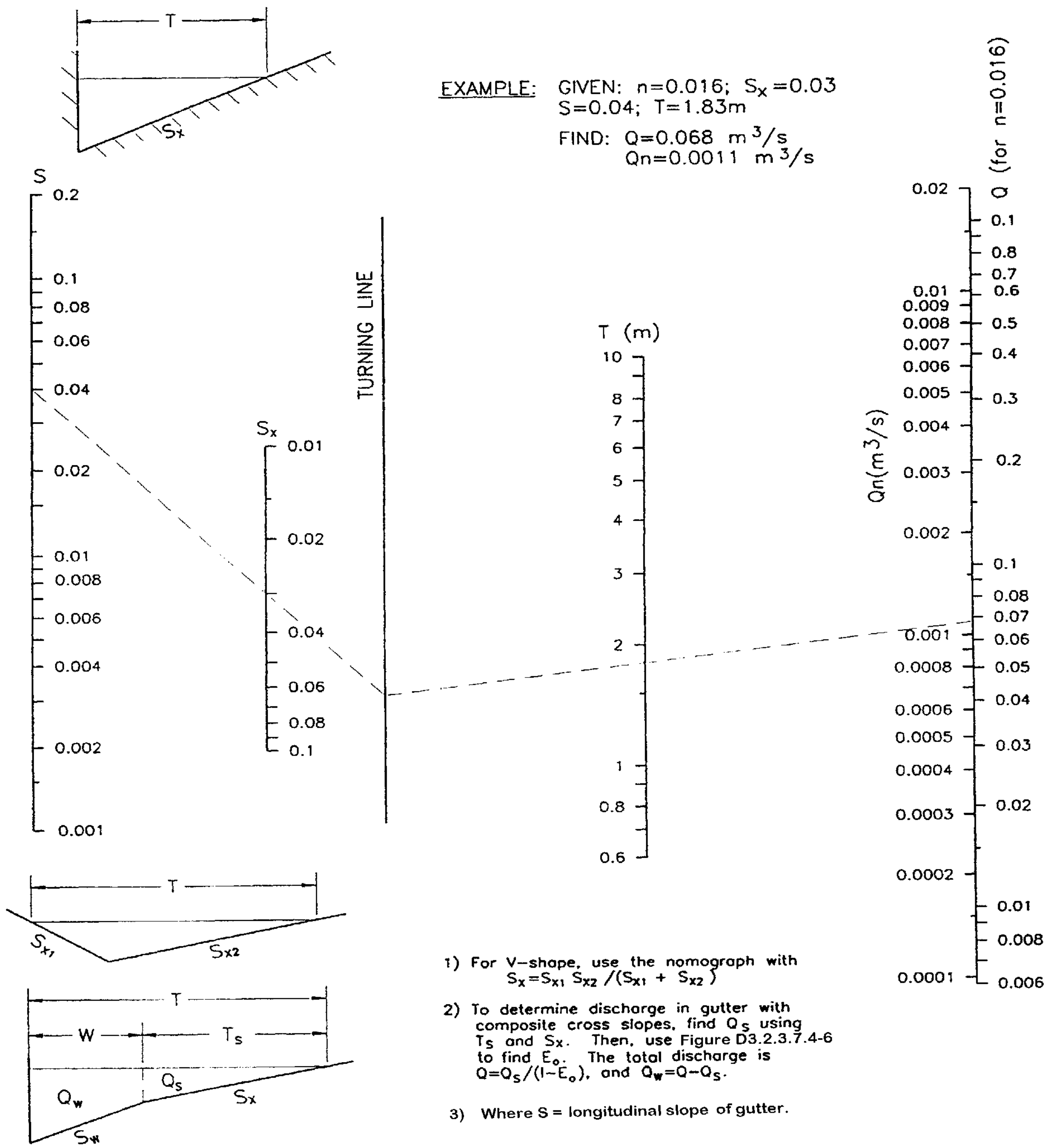


Figure 3.2.3.7.4-2 - Flow in Triangular Gutter Sections

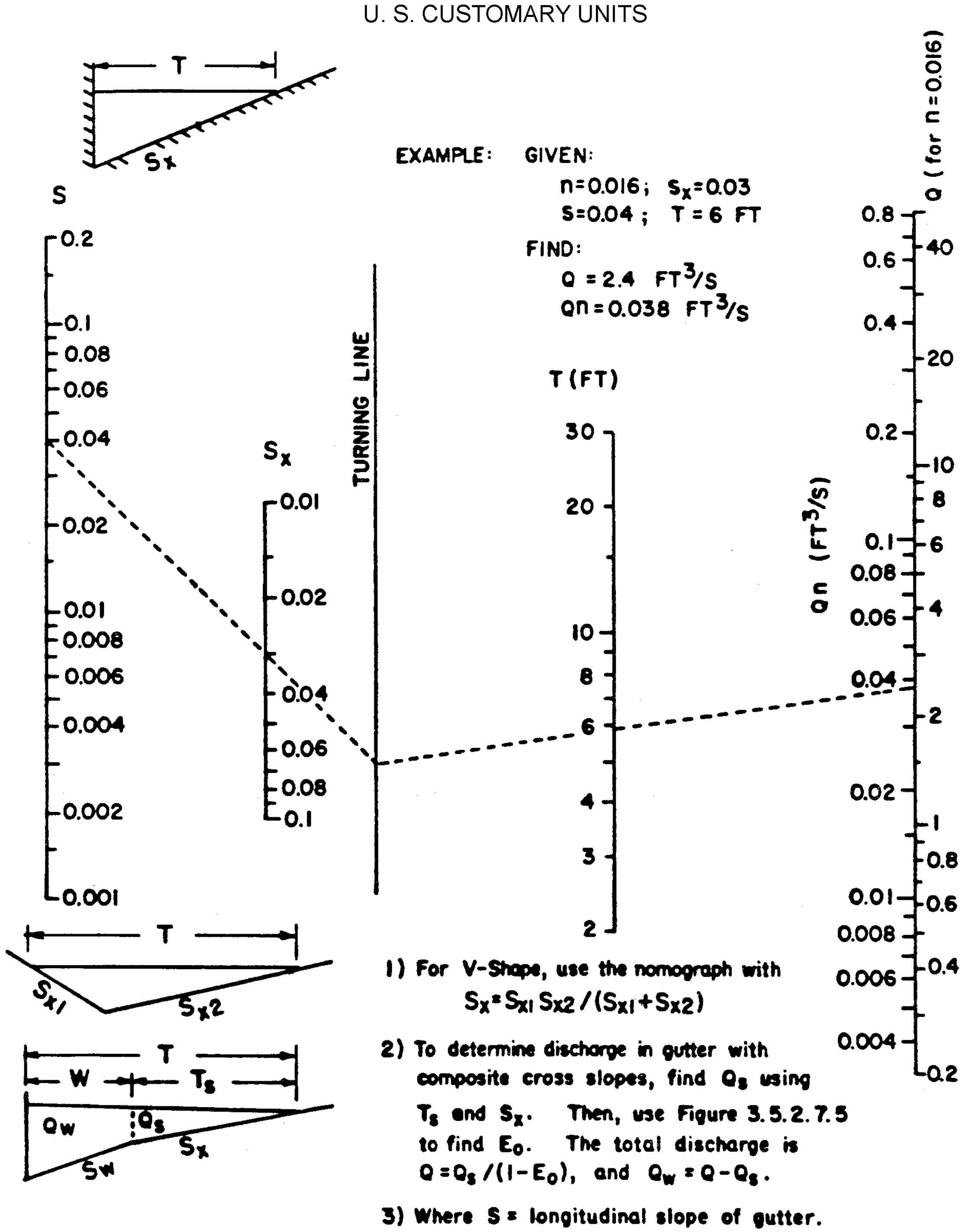


Figure 3.2.3.7.4-2 - Flow in Triangular Gutter Sections (Continued)

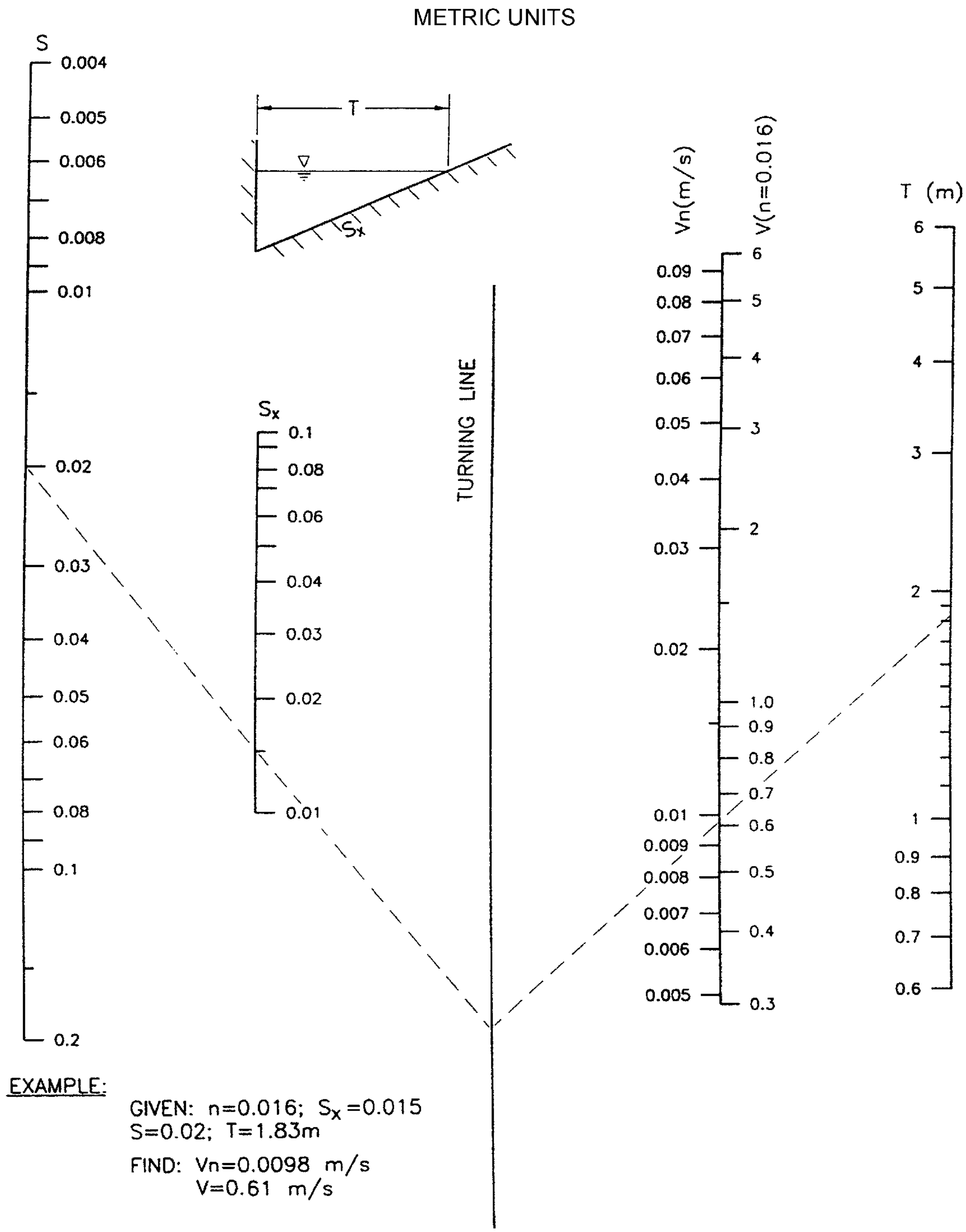


Figure 3.2.3.7.4-3 - Velocity in Triangular Gutter Sections

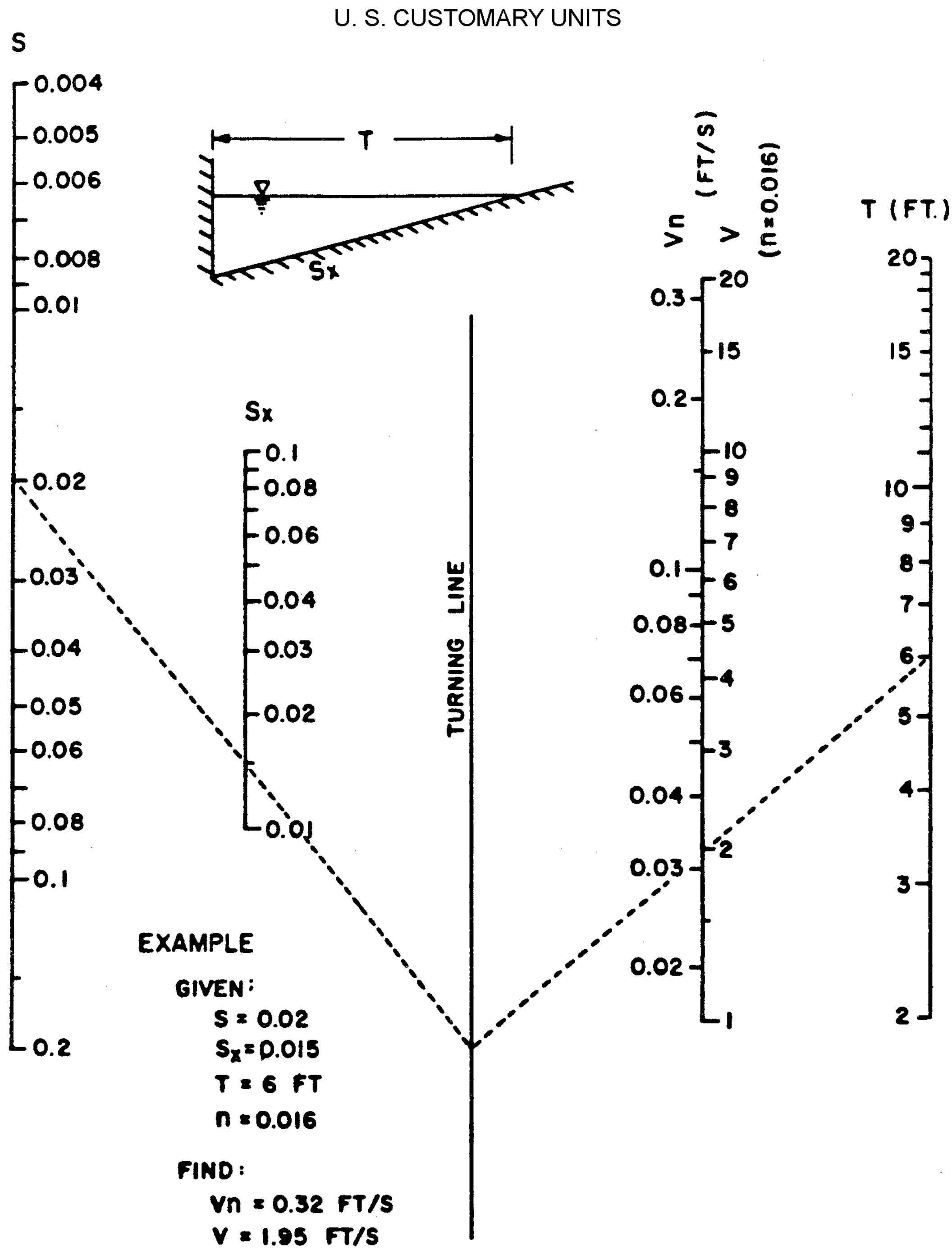
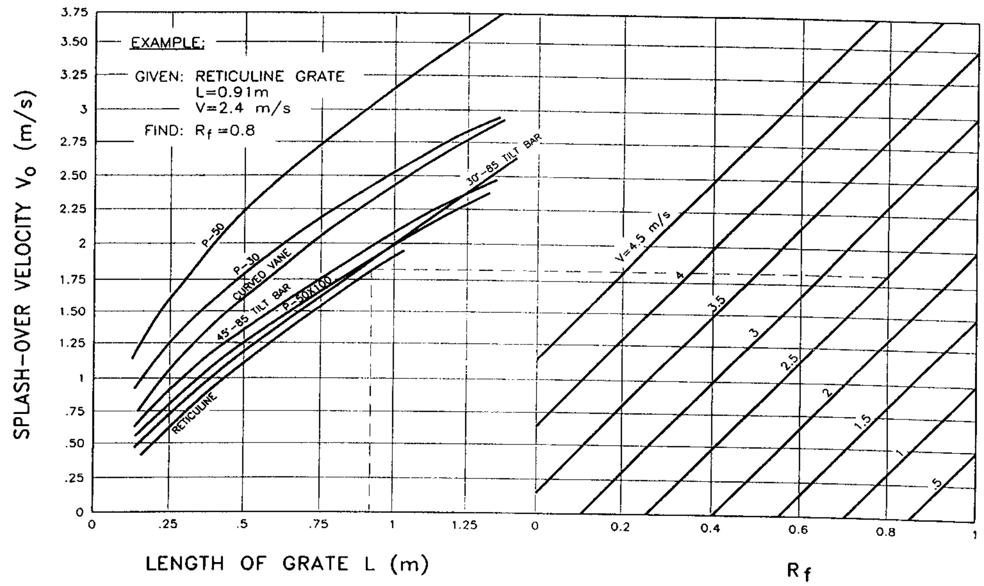


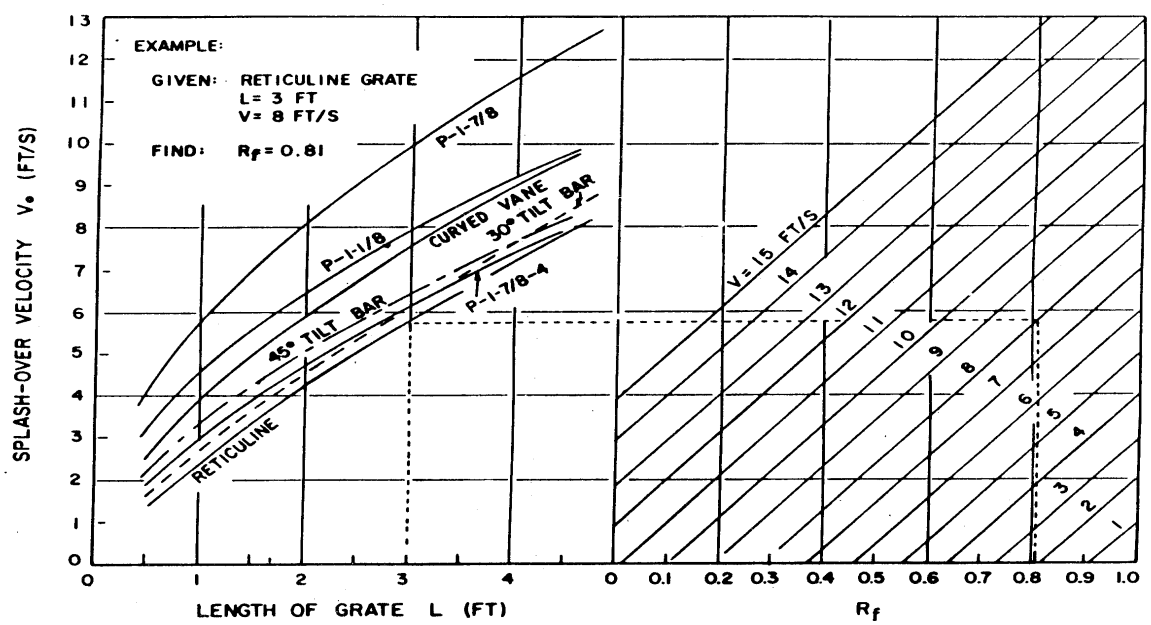
Figure 3.2.3.7.4-3 - Velocity in Triangular Gutter Sections (Continued)

METRIC UNITS



V = VELOCITY OF FLOW IN THE GUTTER.

U. S. CUSTOMARY UNITS



V = VELOCITY OF FLOW IN THE GUTTER.

Figure 3.2.3.7.4-4 - Grate Inlet Frontal Flow Interception Efficiency

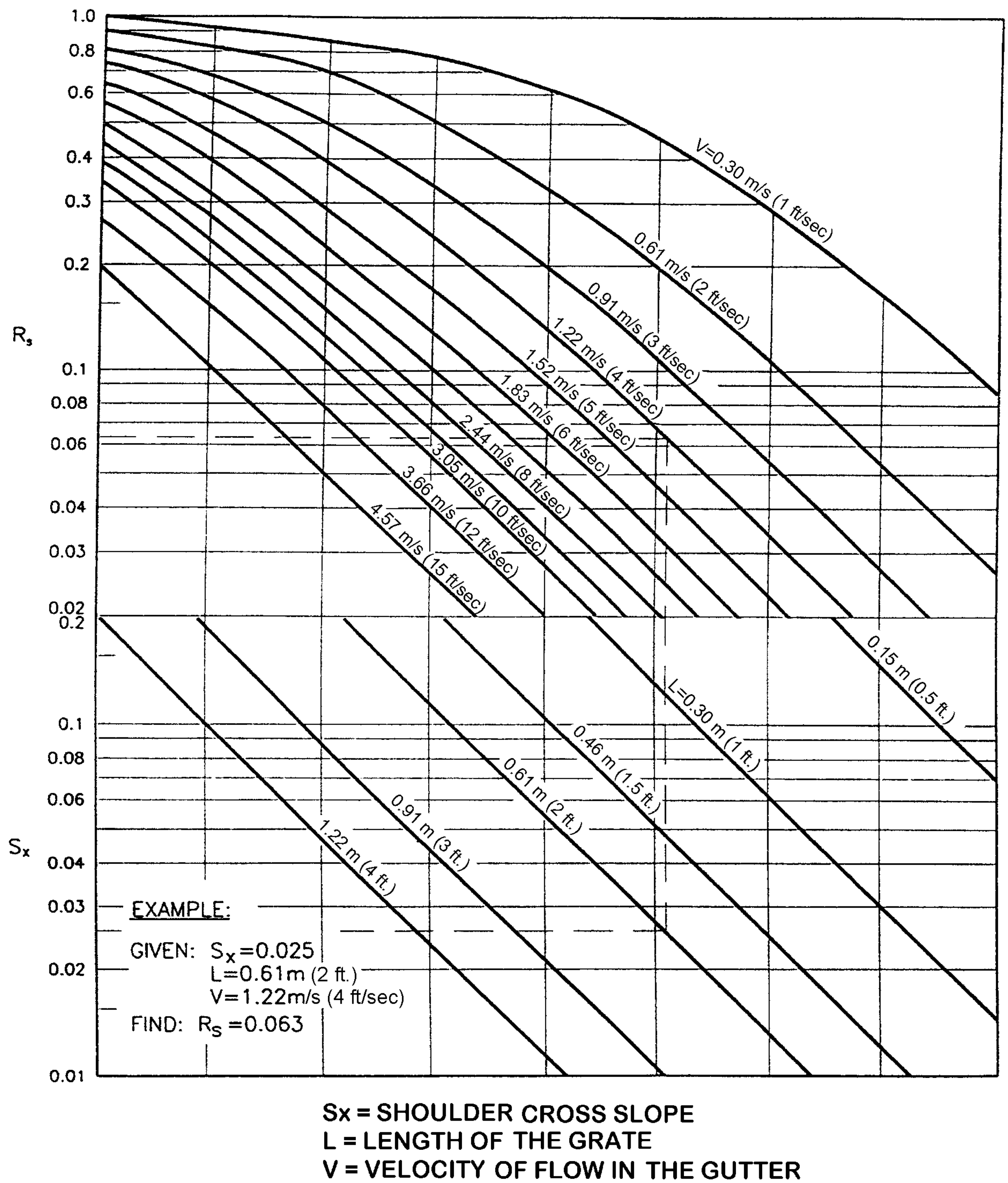
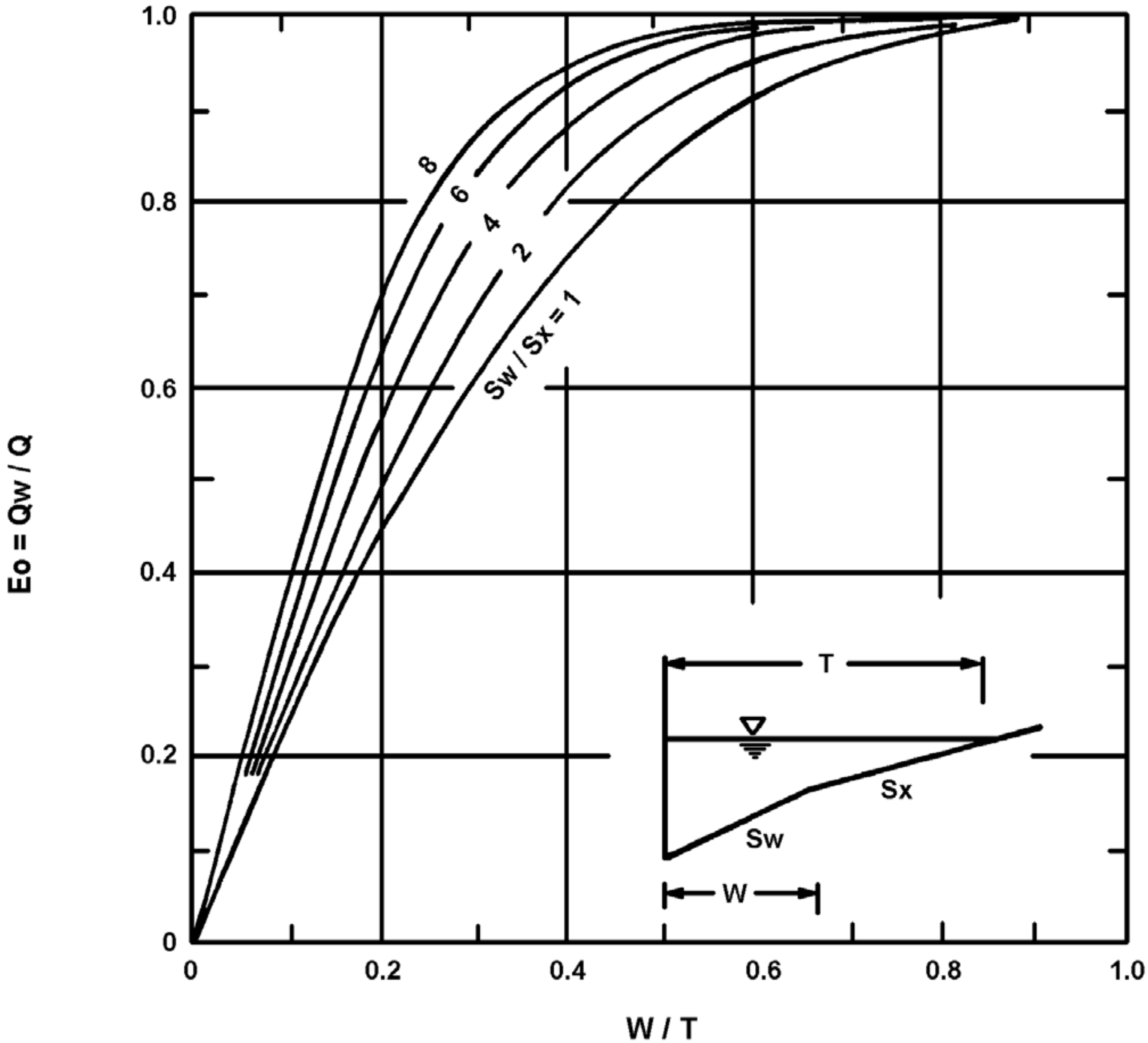


Figure 3.2.3.7.4-5 - Grate Inlet Side Flow Interception Efficiency



T = PERMITTED SPREAD WIDTH
W = GRATE WIDTH

Figure 3.2.3.7.4-6 - Ratio of Frontal Flow to Total Gutter Flow

3.2.3.7.5 Determination of Allowable Length of Bridge Without Scuppers

The allowable length of bridge without scuppers, L shall be determined by the use of Equation 1.

Metric Units: $$L = \frac{1.35 * 10^6}{C\,n}\left(\frac{S_x^{1.67}\,S^{0.5}\,T^{2.67}}{I\,W}\right) \qquad (3.2.3.7.5\text{-}1)$$

U.S. Customary Units: $$L = \frac{24{,}393.6}{C\,n}\left(\frac{S_x^{1.67}\,S^{0.5}\,T^{2.67}}{I\,W}\right)$$

where:

C = 0.9 for bridge decks

n = Roughness coefficient of deck surface = 0.016

S_x = Cross slope of deck

S = Grade as a function of location on bridge

T = Design spread (m) {ft.}

I = Rate of rainfall (mm/hr) {in/hr}

W = Width of deck to be drained (m) {ft.}

If the value of L as determined from Equation 1 is greater than the length of the bridge deck, scuppers are not needed hydraulically.

3.2.3.7.6 Scupper Spacing on Continuous Grades

If L, as determined from PP3.2.3.7.5, is less than the bridge deck length, scuppers are necessary. The following procedure for determining spacing is theoretical. Practical consideration may lead to constant spacing. The theoretical spacings may be revised for ease of placement, maintenance, etc.

(a) Determine the distance from the high point to the first scupper (by trial and error if the bridge is within a vertical curve):

The distance, L, from the high point to the first scupper can be determined from Equation PP3.2.3.7.5-1.

(b) Determine the distance from the first scupper to the second scupper:

(1) Determine the quantity of the intercepted flow at the first scupper (q_1). (This location has grade S_1.) Solve for the total gutter flow:

Metric Units: $$QR_1 = \frac{CIWL}{3.6x10^6} \qquad (3.2.3.7.6\text{-}1)$$

U.S. Customary Units: $$QR_1 = \frac{CIWL}{43{,}560}$$

where:

I = Rate of Rainfall (mm/hr) {in/hr}

L = Distance from high point to first scupper (m) {ft.}

C = 0.9 for bridge decks

W = width of drained deck (m) {ft.}

Solve for the intercepted flow:

$q_1 = (E)\ (QR_1)$ (3.2.3.7.6-2)

in accordance with PP3.2.3.7.4(c).

(2) Select M_1, the distance from the first to the second scupper. (This establishes S_2 as the grade at $L + M_1$.)

(3) Find total gutter flow at $L + M_1$:

Metric Units: $$QR_2 = \left(\frac{CIW}{3.6 \ x \ 10_6} \ (L + M_1) \right) - q_1$$ (3.2.3.7.6-3)

U.S. Customary Units: $$QR_2 = \left(\frac{CIW}{3.6 \ x \ 10^6} \ (L + M_1) \right) - q_1$$

(4) Determine the gutter flow by using the nomogram in Figure PP3.2.3.7.4-2.

(5) Find actual spread width, t, by using Figure PP3.2.3.7.4-2 to find this spread.

(6) If t is equal to the permitted spread width T, the spacing is right. (If t is less than T, increase M_1. If t is greater than T, decrease M_1. Repeat the procedure above until the spacing is right.)

(c) Determine the distance from the second scupper to the third scupper:

(1) Determine the quantity of the intercepted flow at the second scupper q_2. (This location has Grade S_2.)

Use QR_2 computed from the preceding run. Solve for the intercepted flow:

$q_2 = (E)\ (QR_2)$ (3.2.3.7.6-4)

in accordance with PP3.2.3.7.4(c).

(2) Select M_2, the distance from the second to the third scupper. (This establishes S_3 as the grade at $L + M_1 + M_2$.)

(3) Metric Units: $$QR_3 = \left(\frac{CIW}{3.6 \ x \ 10^6} \ (L + M_1 + M_2) \right) - q_1 - q_2$$ (3.2.3.7.6-5)

U.S. Customary Units: $$QR_3 = \left(\frac{CIW}{3.6 \ x \ 10^6} \ (L + M_1 + M_2) \right) - q_1 - q_2$$

(4) Determine the gutter flow by using the nomogram in Figure PP3.2.3.7.4-2:

(5) Find actual spread width, t, by using Figure PP3.2.3.7.4-2 to find this spread.

(6) If t is equal to the permitted spread width T, the spacing is optimum. (If t is less than T, increase M_2. If t is greater that T, decrease M_2. Repeat the procedure until these values are equal.)

(d) Determine the distance from the third to the fourth, the fourth to the fifth scupper, etc.

Continue with M_3, M_4, etc. until L, plus the sum of the spacing, equals the bridge length.

For detailed example problems and further information on the above spacing procedure, refer to the FHWA Report Bridge Deck Drainage Guidelines.

3.2.3.7.7 Spacing of Scuppers in a Sag

Scuppers shall be required in a sag vertical curve, one at the low point and one on each side of this point, where the grade elevation is approximately 60 mm {0.20 ft.} higher than that at the low point. The two adjacent scuppers (one at each side of the low point) are to be provided for contingency and not for hydraulic reasons. The capacity of the scupper at the low point shall be computed by neglecting the existence of the two adjacent scuppers.

The capacity of scuppers in a sag can be calculated from the following formula, assuming 50% efficiency:

$$Q_i = 0.5\,(C_w)\,(P)\,(d)^{1.5} \qquad (3.2.3.7.7\text{-}1)$$

where:

P = Perimeter of the grate (m) {ft.}, disregarding bars and the side against the curb

C_w = 1.66 {U.S. Customary Units: 3.0}

d = Depth of flow at the curb (m) {ft.}

A grate inlet in a sag location may operate as a weir or an orifice, depending on the grate properties (bar configuration and size) and depth of flow. Grates of larger dimension and grates with more open area, i.e., with less space occupied by lateral and longitudinal bars, will operate as weirs to greater depths than will smaller grates or grates with less open area.

Equation 1 applies to the weir condition, where shallow water depth generally governs. This equation should be adequate for the determination of the scupper capacity at a low point, since the allowable water depth for the superstructure drainage is generally shallow.

For further information regarding the interception capacity of inlets (scuppers) in sag locations, refer to FHWA Hydraulic Engineering Circular No. 22.

A suggested procedure for determining the actual spread width and the necessity of additional scuppers is indicated as follows:

(a) Determine Q_i (which equals QR) from the selected length and width of the bridge deck to be drained, in accordance with PP3.2.3.7.4(a).

(b) Determined from:

$$d^{1.5} = \frac{Q_i}{0.5(C_W)\,P} \qquad (3.2.3.7.7\text{-}2)$$

(c) Obtain the actual spread width, t, using d computed as above and the known gutter cross slope.

(d) If t is equal to or less than the permitted spread width T, no additional scuppers are needed for the selected length from the hydraulic point of view. If t is greater than T, additional scuppers are needed.

3.2.4 Paving Notch and Bridge Approach Slab

Provide a paving notch or an attached approach slab on structures from curb-to-curb on all interstates and expressways and all locations with concrete approach pavements. A paving notch is not required on structures with ADT less than 750 or when a concrete pavement is not anticipated in the future unless differential settlement between the approach fill and the structure are anticipated or providing an approach slab will eliminate a joint on the structure.

A bridge approach slab is not required on structures with ADT less than 750 unless providing an approach slab will eliminate a joint on the structure. However, approach slabs are always required with integral abutments and shall be the same curb-to-curb width of the bridge. The U-wings must be designed in such a manner as to permit approach slab movement independent of the U-wings.

3.3 SUBSTRUCTURE

3.3.1 Slope Walls

Slope walls under the end spans are required, when directed by the District Engineer/District Administrator, to protect the slopes from erosion and to eliminate unsightly appearance of barren slopes where aesthetics is a factor or where maintenance is impractical. However, slope walls may not be necessary in rock cut areas.

Cement concrete slope walls, either precast or cast-in-place (see Standard Drawing BC-731M), and random stone slope walls (see Standard Drawing BC-781M) can be used alternatively. However, random stone slope wall is low in cost and suitable for use in rural areas where vandalism is not a great concern.

Stone slope wall may be used in scenic areas for aesthetic purposes (see Standard Drawing RC-40M).

3.3.2 Substructure Drainage

Underdrain pipes which extend from the end or from the face of walls to the highway drainage system shall be identified on the structure drawings as a Roadway Pay Item and shall be included in the Required List on the Roadway Plans.

3.3.3 Abutments and Cast-in-Place Retaining Walls

Abutments, wingwalls and retaining walls shall be of the reinforced concrete cantilever type, except for 100% state-funded projects, where, if specified, wingwalls and retaining walls of overall height of 3600 mm {12 ft.} or less may be of the type shown on Standard Drawing BD-631M.

Use of approved proprietary walls, tie-back walls, or concrete cantilever walls shall be evaluated in light of economics and site suitability.

Approved alternate walls shall be permitted unless only one type is practical at a particular site. The wall types (on the approved list) that are not permitted for a site shall be indicated or specified in the contract.

3.3.4 Prefabricated Retaining Walls

3.3.4.1 GENERAL

Prefabricated walls, including approved proprietary systems, shall be used in competition with conventional reinforced concrete walls where conventional wall design is provided. If conventional walls are clearly not competitive, they shall be excluded from the alternates, thereby saving time and costs provided that two or more prefabricated systems are available.

Preferably none of the suppliers should be contacted during the design stage. However, if during the design of a project, it becomes necessary for the designer to obtain detailed information on any of the proprietary walls, the suppliers of all types permitted in the foundation approval shall be contacted and offered the same degree of involvement so as not to give a particular supplier a time advantage in preparing his or her bid.

3.3.4.2 SYSTEMS APPROVAL

For systems approval guidelines and consideration, see PP1.14.

3.3.4.3 SELECTION PROCEDURE

All feasible, innovative, cost-saving alternates must be considered as follows:

(a) For use as an alternate system during design phase: Consider and permit all feasible alternates. If only one proprietary system is feasible, it shall be specified only as an alternate to a conventional design. Use of only one system must be justified and approval must be secured from the Chief Bridge Engineer before the system is specified.

(b) Value engineering alternate during construction: Contractors may propose any cost-saving, equivalent, approved alternate, with adequate justification.

(c) Experimental use: An experimental system will not be permitted as an alternate.

3.3.4.4 ECONOMIC CONSIDERATIONS FOR PROJECT SELECTION

The decision to designate a prefabricated retaining wall for a particular project requires the determination of its technical feasibility and its economy compared with conventional construction. With respect to economy, the following guidelines are provided:

(a) Mechanically stabilized earth (MSE) walls are generally more economical than conventional cast-in-place retaining walls in fill situations under the following conditions:

(1) Where the retaining wall has a total area greater than 186 m^2 {2,000 ft^2}

(2) For average wall heights greater than 3000 mm {10 ft.} with no traffic barrier

(3) For average wall heights greater than 4500 mm {15 ft.} when traffic barriers are required

(4) For walls of any height, where a rigid conventional wall system requires a deep foundation for support, provided that settlements of MSE walls are tolerable.

(b) Concrete modular systems and soldier pile walls are generally more economical than conventional cast-in-place retaining walls in cut situations, where substantial excavation is necessary for average wall heights greater than 2400 mm {8 ft.} and where the retaining wall area is greater than 46 m^2. {500 ft^2}

(c) Specific project conditions, as outlined below, may reduce the cost-effectiveness of prefabricated wall systems:

(1) Limited availability and high cost of select backfill

(2) Complicated horizontal alignment requiring many turning points and highly irregular finished grades

(3) The necessity of providing temporary excavation support systems during construction

3.3.4.5 PLAN PREPARATION

Where prefabricated retaining walls are permitted as alternates, the conceptual design used for bidding purposes shall contain the following minimum information:

(a) Beginning and end wall stations

(b) Elevations on top of wall at beginning and end of wall, all profile break points, and roadway profile data at wall line

(c) Original and proposed ground profiles in front of and behind retaining wall

(d) Cross-sections at retaining wall location, showing limits of excavation and backfill

(e) Horizontal alignment of wall

(f) Details of wall appurtenances such as traffic barriers, copings, drainage outlets, location and configurations of overhead signs, lighting including conduit locations, and all affected utilities

(g) Right-of-way limits and all affected utilities

(h) Construction sequence requirements, if applicable, including traffic control, access, and staged construction sequences (particularly for abutments); how much settlement must have occurred per stage, and any jacking requirements due to settlement

(i) Elevation of bottom of wall and maximum allowable bearing capacity at this level for each type of wall; location, depth, and extent of any unsuitable material to be removed and replaced

(j) Magnitude, location, and direction of external loads due to bridges, overhead signs, and lighting structures

(k) Architectural treatment for facing panel or module finishes and colors

(l) Quantities table showing estimated wall area, abutment and wingwall quantities, as well as appurtenances and traffic barriers

(m) At abutments, elevation of bearing pads, location of bridge seats, skew angle, and all horizontal and vertical survey control data including clearances; complete design and details of stub abutment for MSE system. Show dead load, live load and longitudinal loads perpendicular to the abutment per linear millimeter {per linear foot} on the plans for abutments on prefabricated walls

(n) Limits and requirements for drainage features beneath, on top of, and behind retaining structure

(o) At stream locations, extreme high-water and normal-water levels and scour protection

(p) Governing construction specifications and special provisions

(q) Limit for epoxy-coated bars in the panels (refer to D5.4.3.6P)

(r) Estimated settlement, factored bearing resistance, and slope stability analysis where applicable (to be submitted with the foundation approval submission)

(s) Reference to D11.10 governing design requirements and allowable deviations, if any

(t) Foundation information (to be submitted with foundation approval submission): for a depth equal to 150 percent of the anticipated length of soil reinforcement at any wall location, for all soil strata

  (1) Drained angle of internal friction, φ

  (2) Undrained shear strength, Cu, for saturated cohesive materials

  (3) Total density of the material

  (4) Boring logs

  (5) Location of watertable

  (6) Random backfill information

  - Drained angle of internal friction
  - Cohesive strength
  - Total density

  If the above information is not provided, the appropriate values given in D10 and D11 shall be used.

  As a part of the foundation submission, the designer shall estimate the applied or design bearing pressure and compare it with the calculated factored bearing resistance. Where an abutment is supported by the prefabricated wall, pressure applied by abutments shall be included in the calculation of the total applied pressure. If the estimated applied pressure exceeds the factored bearing resistance, the designer shall consider such remedial measures as overexcavation and replacement with granular backfill and shall indicate in the plans the depth and lateral extent of the foundation modifications.

(u) External stability, including overturning, sliding, settlement, and overall slope stability shall be checked using an approved analysis method. For analysis purposes, a base width equal to 0.7 of the height or the minimum specified width shall be used for mechanically stabilized earthwalls.

(v) All MSE wall plans (if applicable) must show pile locations and proposed location and arrangement of MSE wall soil-reinforcing elements to avoid interference with the piles. In some instances, pile locations interfere with soil-reinforcing grids (VSL walls) or soil-reinforcing strips (RE walls) behind the MSE walls. Specific method should be developed for the field installation to avoid and circumvent interference with piles. Simply cutting soil-reinforcing elements (grids or strips) is not acceptable.

(w) For abutments, wingwalls and retaining walls, eliminate flexible open cell polyethyer or polyurethane foam strip fillers for horizontal joints between panels; cover all joints between panels on back side of the wall with geotextile Class 2, Type A fabric. Apply adhesive coating on panels only and not on geotextile fabric. Do not apply adhesive within 50 mm {2 in.} of the joint.

For Precast Modular Unit Retaining Walls, T-walls, and any prefabricated wall, provide complete drainage behind the wall and provide weepholes in the front wall as given in Standard Drawing BC-799M.

These procedures will provide free drainage and minimize hydrostatic pressure.

3.3.4.6 PROPOSAL PREPARATION AND BIDDING INSTRUCTIONS

(a) Proposal Preparation

Where prefabricated walls are used for retaining walls and conventional retaining walls are permitted as alternates, the Lump Sum bid items will include "Retaining Wall, S-xxxxx", "Mechanically Stabilized Retaining Wall, S-xxxxxP" and "Precast Modular Retaining Wall, S-xxxxxP". Note that the same S-number will appear in all items.

When conventional retaining walls are NOT permitted as alternate designs, conceptual drawings for the prefabricated walls should be shown on the structure drawings with specific S-number. However, the Lump Sum bid items will include "Mechanically Stabilized Retaining Wall, S-xxxxxP" and "Precast Modular Retaining Wall, S-xxxxxP".

Where prefabricated walls are not permitted as alternates for retaining walls, the lump sum bid items will include "Retaining Walls, As-Designed, S-xxxxx".

Prefabricated walls shown in accordance with PP3.3.4.5 are not considered as "as-designed" walls. Therefore, a detailed break down of the prefabricated wall quantities for the "Component Item Schedule" is not required. However, the successful bidder will be required to submit a "Component Item Schedule" for the prefabricated wall in accordance with the Special Provisions (Part A).

Figure 1 through Figure 6 are provided for your guidance. They cover the procedures for preparing the PS&E package, where the proposal includes prefabricated walls and alternate bridge structures. For typical tabulations of quantities, refer to Figures 7 through 13. Note that Figure 6 should be avoided, if conventional walls are clearly not competitive, by using Figure 5.

(b) Bidding Instructions

In order to give suppliers of prefabricated walls sufficient time to prepare bids, the presence of these items in forthcoming projects must be described in the D-407 (given in Design Manual, Part 2) cover sheet submittal, so that the necessary information can be included in the Notice to Contractors. This is especially important for prefabricated walls because designs of sufficient depth to permit reliable pricing must be developed by the suppliers during the advertising period.

When prefabricated walls are used for the wingwalls and/or abutments of a bridge, they will be considered as separate lump sum items, for both as-designed bridges and contractor-designed alternates. Only general layout and elevation information for the various prefabricated walls permitted will be shown for the as-designed bridge. The design criteria will be in accordance with this Manual or as modified in the Special Provisions.

The bidders will be required to indicate the type of prefabricated wall intended for construction. The Contractor shall submit, for approval, a design of the prefabricated wall as would be done for any contractor design alternate.

WHERE PREFABRICATED WALLS ARE USED AS RETAINING WALLS AND CONVENTIONAL R.C. WALLS ARE PERMITTED:

**ITEM 8610-00xx - CONCRETE RETAINING WALL, S-xxxxx
*ITEM 8621-00xx - MECHANICALLY STABILIZED RETAINING WALL, S-xxxxxP
*ITEM 8622-00xx - PRECAST MODULAR RETAINING WALL, S-xxxxxP
____

Construct one of the above at S.R. _________,
Section ______, Segment ________, Offset _________.

ALTERNATE WALL PART A: S00(ID80101A)

Include the appropriate retaining wall system Standard Special Provisions, e.g. S00(NM00411A), S00(NE00411A), S00(ND00571A).

*List only applicable retaining wall systems.
**Include "Component Item Schedule" for as-designed wall

Figure 3.3.4.6-1 - Prefabricated Walls are used as Retaining Walls and Conventional R.C. Walls are Permitted

WHERE PREFABRICATED WALLS ARE USED AS RETAINING WALLS AND CONVENTIONAL R.C. WALLS ARE NOT PERMITTED (STRUCTURE DRAWINGS REQUIRED):

*ITEM 8621-00xx - MECHANICALLY STABILIZED RETAINING WALL, S-xxxxxP
*ITEM 8622-00xx - PRECAST MODULAR RETAINING WALL, S-xxxxxP
____

Construct one of the above at S.R. _______,
Section ______, Segment_______, Offset _______.

ALTERNATE WALL PART A: S00(ID80121A)

Include the appropriate retaining wall system Standard Special Provisions, e.g. S00(NM00411A), S00(NE00411A), S00(ND00571A).

*List only applicable retaining wall systems.

NOTE: Refer to CMS Index document BDW1(INDEX00I) for the current document number of all Standard Special Provisions.

Figure 3.3.4.6-2 - Prefabricated Walls are used as Retaining Walls and Conventional R.C. Walls are Not Permitted

WHERE CONVENTIONAL R.C. WALLS ARE USED AS RETAINING WALLS AND PREFABRICATED RETAINING WALLS ARE NOT PERMITTED:

**ITEM 8610-00xx - CONCRETE RETAINING WALL, AS-DESIGNED, S-xxxxx

____

Construct the above at S.R. _________,
Section ______, Segment ________, Offset _________.

**Include "Component Item Schedule" for as-designed wall

Figure 3.3.4.6-3 - Conventional R.C. Walls are used as Retaining Walls and Prefabricated Retaining Walls are not Permitted

LUMP SUM BRIDGE STRUCTURE WITH CONVENTIONAL ABUTMENTS AND CONVENTIONAL WINGWALLS WHERE PREFABRICATED WALLS ARE NOT PERMITTED:

**ITEM 8xxx-00xx - BRIDGE STRUCTURE, AS-DESIGNED, S-xxxxx
ITEM 8100-00xx - STEEL BRIDGE STRUCTURE
ITEM 8000-00xx - PRESTRESSED CONCRETE BRIDGE STRUCTURE
____

Construct one of the above at S.R. _______,
Section ______, Segment _______, Offset ______.

ALTERNATE BRIDGE STRUCTURE: S00(ID80041A)

Special Drawings and Special Design Requirements (PART B)

Piles (when as-designed bridge includes piles)

NOTE: Refer to CMS Index document BDW1(INDEX00I) for the current document number of all Standard Special Provisions.

**Include "Component Item Schedule" for as-designed structure.

Figure 3.3.4.6-4 - Lump Sum Bridge Structure with Conventional Abutments and Conventional Wingwalls where Prefabricated Walls are not Permitted

LUMP SUM BRIDGE STRUCTURE WITH PREFABRICATED WALL ABUTMENTS AND/OR PREFABRICATED WINGWALLS WHERE CONVENTIONAL ABUTMENTS AND WINGWALLS ARE NOT PERMITTED (BEING UNECONOMICAL):

**ITEM 8xxx-00xx - BRIDGE STRUCTURE, AS-DESIGNED, S-xxxxx
ITEM 8100-00xx - STEEL BRIDGE STRUCTURE
ITEM 8000-00xx - PRESTRESSED CONCRETE BRIDGE STRUCTURE

Construct one of the above at S.R. _______,
Section _______, Segment _______, Offset _______.

ALTERNATE BRIDGE STRUCTURE: S00(ID80041A)

Special Drawings and Special Design Requirements (PART B)

Piles (when as designed bridge includes piles)

NOTE:

- Provide a separate pay item in the schedule of prices for prefabricated abutments and/or wingwalls (do not provide separate S-number for prefabricated wall layout drawing).

- Stub abutments, structure backfill behind stub abutments should be included with bridge structure lump sum.

*ITEM 8641-00xx - MECHANICALLY STABILIZED ABUTMENT OR WINGWALL, S-xxxxxP
*ITEM 8642-00xx - PRECAST MODULAR ABUTMENT OR WINGWALL, S-xxxxxP

Construct one of the above at S.R. ________,
Section ______, Segment ______, Offset _______.

ALTERNATE WALL PART A: S00(ID80121A)

Include appropriate prefabricated wall system Standard Special Provisions, e.g., S00(NM00411A), S00(NE00411A), S00(ND00571A).

NOTE: Refer to CMS Index document BDW1(INDEX00I) for the current document number for all Standard Special Provisions.

**Include "Component Item Schedule" for as-designed bridge structure.

*List only applicable retaining wall systems

Figure 3.3.4.6-5 - Lump Sum Bridge Structure with Prefabricated Wall Abutments and/or Prefabricated Wingwalls where Conventional Abutments and Wingwalls are not Permitted

LUMP SUM BRIDGE STRUCTURE WITH CONVENTIONAL ABUTMENTS AND CONVENTIONAL RETAINING WALLS AND WINGWALLS WHERE PREFABRICATED ABUTMENTS AND/OR WINGWALLS ARE PERMITTED (WHEN CONVENTIONAL ABUTMENTS ARE COMPETITIVE):

NOTE: If possible, avoid this figure by using Figure 4. However, if included on PS&E package, use the following procedure:
+ITEM 8xxx-00xx - BRIDGE STRUCTURE, AS-DESIGNED
(INCLUDES CONVENTIONAL ABUTMENT AND WINGWALL DESIGN), S-xxxxx
++ITEM 8xxx-00xx - BRIDGE STRUCTURE, AS DESIGNED (DOES NOT INCLUDE CONVENTIONAL ABUTMENT AND WINGWALL DESIGN), S-xxxxx
ITEM 8100-00xx - STEEL BRIDGE STRUCTURE (CONVENTIONAL ABUTMENTS)
ITEM 8100-00xx - STEEL BRIDGE STRUCTURE (PREFABRICATED ABUTMENTS)
ITEM 8000-00xx - PRESTRESSED CONCRETE BRIDGE STRUCTURE (CONVENTIONAL ABUTMENTS)
ITEM 8000-00xx - PRESTRESSED CONCRETE BRIDGE STRUCTURE (PREFABRICATED ABUTMENTS)

Construct one of the above at S.R. ______,
Section ______, Segment ______, Offset ______.

ALTERNATE BRIDGE STRUCTURE: S00(ID80041A)

Special Drawings and Special Design Requirements
(PART B).
Piles (when as-designed bridge includes piles)

ITEM 8640-00xx PREFABRICATED ABUTMENT OR WINGWALL, S-xxxxxP

Construct one of the following at S.R. _______,
Section ______, Segment _______, Offset _______:

*1. Mechanically Stabilized Retaining Wall System.

*2. Precast Modular Retaining Wall System.

ALTERNATE WALL PART A: S00(ID80121A)

Include appropriate prefabricated wall system Standard Special Provisions, e.g. (S00(NM00411A), S00(NE00411A), S00(ND00571A)).

NOTE: Refer to CMS Index document BDW1(INDEX00I) for the current document number of all Standard Special Provisions.

+Include "Component Item Schedule" for as-designed structure

++Include "Component Item Schedule" but reduce conventional abutment related quantities (Class 3 Excavation; Class A Concrete; Class AA Concrete; Reinforcement Bars; Structure Backfill Material and other items related to conventional abutments)

*List only applicable retaining wall systems

Figure 3.3.4.6-6 - Lump Sum Bridge Structure with Conventional Abutments and Conventional Retaining Walls and Wingwalls where Prefabricated Abutments and/or Wingwalls are Permitted

When Prefabricated Walls are used as Retaining Walls and Conventional R.C. Walls are Permitted:
(Refer to Figure 1):

Items shown on this table are incomplete for an actual report and are shown merely as an example of the proposed items layout.

TABULATION OF BRIDGE BID ITEMS & APPROXIMATE QUANTITIES								
QUANTITY	ITEM NUMBER	INFORMAL QUANTITY & UNIT	DESCRIPTION	RETAINING WALL OR WINGWALL				
	Unit							
EITHER LS	8610 0001	RETAINING WALL, S-XXXXX		---				
	LS							
		468 M3	CLASS 3 EXCAVATION	221				
		930 M3	SELECTED BORROW EXCAVATION, STRUCTURE BACKFILL	489				
		300 M3	CLASS A CEMENT CONCRETE	137				
AND 14 318	3002 0053	REINFORCEMENT BARS, EPOXY-COATED		1030				
	kg							
OR LS	8621 0002	MECHANICALLY STABILIZED RETAINING WALL, S-XXXXXP		(---) M2				
	LS							
OR LS	8622 0002	PRECAST MODULAR RETAINING WALL, S-XXXXXP		(---) M2				
	LS							

(List Applicable Prefabricated Wall Systems on the Drawing)

Figure 3.3.4.6-7 - Prefabricated Walls are used as Retaining Walls and Conventional R.C. Walls are Permitted

Items shown on this table are incomplete for an actual report and are shown merely as an example of the proposed items layout.

TABULATION OF BRIDGE BID ITEMS & APPROXIMATE QUANTITIES								
QUANTITY	ITEM NUMBER Unit	INFORMAL QUANTITY & UNIT	DESCRIPTION	RETAINING WALL OR WINGWALL				
EITHER LS	8610 0001 LS	RETAINING WALL, S-XXXXX		---				
		613 CY	CLASS 3 EXCAVATION	289				
		1, 217 CY	SELECTED BORROW EXCAVATION, STRUCTURE BACKFILL	640				
		393 CY	CLASS A CEMENT CONCRETE	179				
AND 31,563	3002 0053 LB	REINFORCEMENT BARS, EPOXY-COATED		2,270				
OR LS	8621 0002 LS	MECHANICALLY STABILIZED RETAINING WALL, S-XXXXXP		(---) SF				
OR LS	8622 0002 LS	PRECAST MODULAR RETAINING WALL, S-XXXXXP		(---) SF				

(List Applicable Prefabricated Wall Systems on the Drawing)

Figure 3.3.4.6-7 - Prefabricated Walls are used as Retaining Walls and Conventional R.C. Walls are Permitted (Continued)

When Prefabricated Walls are used as Retaining Walls and Conventional R.C. Walls are not Permitted:
(Refer to Figure 2):

Items shown on this table are incomplete for an actual report and are shown merely as an example of the proposed items layout.

TABULATION OF BRIDGE BID ITEMS							
QUANTITY	ITEM NUMBER / UNIT	DESCRIPTION	RETAINING WALL OR WINGWALL				
EITHER LS	8621 0001 / LS	MECHANICALLY STABILIZED RETAINING WALL, S-XXXXXP	(---) M2				
OR LS	8622 0002 / LS	PRECAST MODULAR RETAINING WALL, S-XXXXXP	(---) M2				

(List Applicable Prefabricated Wall Systems on the Drawing)

Figure 3.3.4.6-8 - Prefabricated Walls are used as Retaining Walls and Conventional R.C. Walls are not Permitted

Items shown on this table are incomplete for an actual report and are shown merely as an example of the proposed items layout.

TABULATION OF BRIDGE BID ITEMS							
QUANTITY	ITEM NUMBER UNIT	DESCRIPTION	RETAINING WALL OR WINGWALL				
EITHER LS	8621 0001 LS	MECHANICALLY STABILIZED RETAINING WALL, S-XXXXXP	(---) SF				
OR LS	8622 0002 LS	PRECAST MODULAR RETAINING WALL, S-XXXXXP	(---) SF				

(List Applicable Prefabricated Wall Systems on the Drawing)

Figure 3.3.4.6-8 - Prefabricated Walls are used as Retaining Walls and Conventional R.C. Walls are not Permitted (Continued)

When Conventional R.C. Walls are used as Retaining Walls and Prefabricated Retaining Walls are not Permitted: (Refer to Figure 3):

Items shown on this table are incomplete for an actual report and are shown merely as an example of the proposed items layout.

TABULATION OF BRIDGE BID ITEMS & APPROXIMATE QUANTITIES								
QUANTITY	ITEM NUMBER Unit	INFORMAL QUANTITY & UNIT	DESCRIPTION	RETAINING WALL OR WINGWALL				
EITHER LS	8610 0001 LS	RETAINING WALL, AS DESIGNED, S-XXXXX		---				
		468 M3	CLASS 3 EXCAVATION	221				
		930 M3	SELECTED BORROW EXCAVATION, STRUCTURE BACKFILL	489				
		300 M3	CLASS A CEMENT CONCRETE	137				
AND 14 318	3002 0053 kg	REINFORCEMENT BARS, EPOXY-COATED		1030				

(List Applicable Prefabricated Wall Systems on the Drawing)

Figure 3.3.4.6-9 - Conventional R.C. Walls are used as Retaining Walls and Prefabricated Retaining Walls are not Permitted

Items shown on this table are incomplete for an actual report and are shown merely as an example of the proposed items layout.

TABULATION OF BRIDGE BID ITEMS & APPROXIMATE QUANTITIES								
QUANTITY	ITEM NUMBER / Unit	INFORMAL QUANTITY & UNIT	DESCRIPTION	RETAINING WALL OR WINGWALL				
EITHER LS	8610 0001 / LS	RETAINING WALL, AS DESIGNED, S-XXXXX		---				
		613 CY	CLASS 3 EXCAVATION	289				
		1,217 CY	SELECTED BORROW EXCAVATION, STRUCTURE BACKFILL	640				
		393 CY	CLASS A CEMENT CONCRETE	179				
AND 31,563	3002 0053 / LB	REINFORCEMENT BARS, EPOXY-COATED		2,270				

(List Applicable Prefabricated Wall Systems on the Drawing)

Figure 3.3.4.6-9 - Conventional R.C. Walls are used as Retaining Walls and Prefabricated Retaining Walls are not Permitted (Continued)

Lump Sum Bridge Structure with Conventional Abutments and Conventional Wingwalls where Prefabricated Abutments and Wingwalls are Not Permitted (Refer to Figure 4):

Items shown on this table are incomplete for an actual report and are shown merely as an example of the proposed items layout.

TABULATION OF BRIDGE BID ITEMS & APPROXIMATE QUANTITIES								
QUANTITY	ITEM NO. / UNIT	INFORMAL QUANTITY & UNIT	DESCRIPTION	NEAR ABUTMENT	PIER	FAR ABUTMENT	SUPERSTR.	DECK
EITHER LS	8010 0001 / LS	BRIDGE SUPERSTRUCTURE, AS DESIGNED, S-XXXXX						
		468 M3	CLASS 3 EXCAVATION	221	76	171		
		930 M3	SELECTED BORROW EXCAVATION, STRUCTURE BACKFILL	489	76	365		
		300 M3	CLASS A CEMENT CONCRETE	137	76	87		
		LS	STEEL BEAM TEST PILES, HP 310 X 79	2 @ 11 m				
		LS	STEEL BEAM TEST PILES, HP 310 X 79			1 @ 7 m 1 @ 11 m		
AND 14 318	3002 0053 / Kg	REINFORCEMENT BARS, EPOXY-COATED		1030		84	454	12 750
AND 509	3005 1103 / M	STEEL BEAM BEARING PILES, HP 310 X 79		277		232		
AND 75	3005 1153 / EACH	STEEL BEAM PILE TIP REINFORCEMENT, HP 310 X 79		42		33		
OR LS	8100 0002 / LS	STEEL BRIDGE STRUCTURE						
AND (---)	3005 1103 / M	STEEL BEAM BEARING PILES, HP 310 X 79		(---)		(---)		

Figure 3.3.4.6-10 - Lump Sum Bridge Structure with Conventional Abutments and Conventional Wingwalls where Prefabricated Abutments and Wingwalls are Not Permitted

TABULATION OF BRIDGE BID ITEMS & APPROXIMATE QUANTITIES							
AND (---)	3005 1103 EACH	STEEL BEAM PILE TIP REINFORCEMENTHP 310 X 79	(---)		(---)		
OR LS	8000 0003 LS	PRESTRESSED CONCRETE BRIDGE STRUCTURE					
AND (---)	3005 1103 M	STEEL BEAM BEARING PILES,HP 310 X 79	(---)		(---)		
AND (---)	3005 1153 EACH	STEEL BEAM PILE TIP REINFORCEMENT,HP 310 X 79	(---)		(---)		

Figure 3.3.4.6-10 - Lump Sum Bridge Structure with Conventional Abutments and Conventional Wingwalls where Prefabricated Abutments and Wingwalls are <u>Not</u> Permitted (Continued)

Items shown on this table are incomplete for an actual report and are shown merely as an example of the proposed items layout.

TABULATION OF BRIDGE BID ITEMS & APPROXIMATE QUANTITIES								
QUANTITY	ITEM NO. UNIT	INFORMAL QUANTITY & UNIT	DESCRIPTION	NEAR ABUTMENT	PIER	FAR ABUTMENT	SUPERSTR.	DECK
EITHER LS	8010 0001 LS		BRIDGE SUPERSTRUCTURE, AS DESIGNED, S-XXXXX					
		613 CY	CLASS 3 EXCAVATION	289	100	224		
		1,217 CY	SELECTED BORROW EXCAVATION, STRUCTURE BACKFILL	640	100	477		
		393 M3	CLASS A CEMENT CONCRETE	179	100	114		
		LS	STEEL BEAM TEST PILES, HP 12 X 53	2 @ 37’				
		LS	STEEL BEAM TEST PILES, HP 12 X 53			1 @ 24’ 1 @ 35’		
AND 31,563	1002 0053 LB		REINFORCEMENT BARS, EPOXY-COATED	2,270		186	1,000	28,107
AND 1,672	1005 1103 LF		STEEL BEAM BEARING PILES,HP 12 X 53	910		762		
AND 75	1005 1153 EACH		STEEL BEAM PILE TIP REINFORCEMENT, HP 12 X 53	42		33		
OR LS	8100 0002 LS		STEEL BRIDGE STRUCTURE					
AND (---)	1005 1103 LF		STEEL BEAM BEARING PILES, HP 12 X 53	(---)		(---)		

Figure 3.3.4.6-10 - Lump Sum Bridge Structure with Conventional Abutments and Conventional Wingwalls where Prefabricated Abutments and Wingwalls are Not Permitted (Continued)

TABULATION OF BRIDGE BID ITEMS & APPROXIMATE QUANTITIES							
AND (---)	1005 1103 EACH	STEEL BEAM PILE TIP REINFORCEMENT HP 12 X 53	(---)		(---)		
OR LS	8000 0003 LS	PRESTRESSED CONCRETE BRIDGE STRUCTURE					
AND (---)	1005 1103 LF	STEEL BEAM BEARING PILES, HP 12 X 53	(---)		(---)		
AND (---)	1005 1153 EACH	STEEL BEAM PILE TIP REINFORCEMENT, HP 12 X 53	(---)		(---)		

Figure 3.3.4.6-10 - Lump Sum Bridge Structure with Conventional Abutments and Conventional Wingwalls where Prefabricated Abutments and Wingwalls are Not Permitted (Continued)

Lump Sum Bridge Structure with Prefabricated Wall Abutments and/or Wingwalls where Conventional Abutments and Wingwalls are Not Permitted (Refer to Figure 5):

Items shown on this table are incomplete for an actual report and are shown merely as an example of the proposed items layout.

TABULATION OF BRIDGE BID ITEMS & APPROXIMATE QUANTITIES								
QUANTITY	ITEM NO. Unit	INFORMAL QUANTITY & UNIT	DESCRIPTION	NEAR ABUTMENT	PIER	FAR ABUTMENT	SUPERSTR.	DECK
EITHER LS	8010 0001 LS	BRIDGE SUPERSTRUCTURE, AS DESIGNED, S-xxxxx		---				
		468 M3	CLASS 3 EXCAVATION	221	76	171		
		930 M3	SELECTED BORROW EXCAVATION, STRUCTURE BACKFILL	489	76	365		
		300 M3	CLASS A CEMENT CONCRETE	137	76	87		
		LS	STEEL BEAM TEST PILES, HP 310 X 79	2 @ 11 m				
		LS	STEEL BEAM TEST PILES, HP 310 X 79			1 @ 7 m 1 @ 11 m		
AND 14 318	3002 0053 kg	REINFORCEMENT BARS, EPOXY-COATED		1030		84	454	12 750
AND 509	3005 1103 M	STEEL BEAM BEARING PILES, HP 310 X 79		277		232		
AND 75	3005 1153 EACH	STEEL BEAM PILE TIP REINFORCEMENT, HP 310 X 79		42		33		
OR LS	8100 0002 LS	STEEL BRIDGE STRUCTURE						
AND (---)	3005 1103 M	STEEL BEAM BEARING PILES, HP 310 X 79		(---)		(---)		
AND (---)	3005 1103 EACH	STEEL BEAM PILE TIP REINFORCEMENT		(---)		(---)		

Figure 3.3.4.6-11 - Lump Sum Bridge Structure with Prefabricated Wall Abutments and/or Wingwalls where Conventional Abutments and Wingwalls are Not Permitted

TABULATION OF BRIDGE BID ITEMS & APPROXIMATE QUANTITIES							
OR LS	8000 0003 LS	PRESTRESSED CONCRETE BRIDGE STRUCTURE					
AND (---)	3005 1103 M	STEEL BEAM BEARING PILES, HP 310 X 79	(---)		(---)		
AND (---)	3005 1153 EACH	STEEL BEAM PILE TIP REINFORCEMENT, HP 310 X 79	(---)		(---)		
LS	8641 0001 LS	MECHANICALLY STABILIZED ABUTMENT OR WINGWALL, S-XXXXXP	(---) M2		(---) M2		
OR LS	8642 0001 LS	PRECAST MODULAR ABUTMENT OR WINGWALL, S-XXXXXP	(---) M2		(---) M2		

Figure 3.3.4.6-11 - Lump Sum Bridge Structure with Prefabricated Wall Abutments and/or Wingwalls where Conventional Abutments and Wingwalls are Not Permitted (Continued)

Items shown on this table are incomplete for an actual report and are shown merely as an example of the proposed items layout.

TABULATION OF BRIDGE BID ITEMS & APPROXIMATE QUANTITIES								
QUANTITY	ITEM NO. Unit	INFORMAL QUANTITY & UNIT	DESCRIPTION	NEAR ABUTMENT	PIER	FAR ABUTMENT	SUPERSTR.	DECK
EITHER LS	8010 0001 LS		BRIDGE SUPERSTRUCTURE, AS DESIGNED, S-xxxxx	---				
		613 CY	CLASS 3 EXCAVATION	289	100	224		
		1,217 CY	SELECTED BORROW EXCAVATION, STRUCTURE BACKFILL	640	100	477		
		393 CY	CLASS A CEMENT CONCRETE	179	100	114		
		LS	STEEL BEAM TEST PILES,HP 12 X 53	2 @ 37’				
		LS	STEEL BEAM TEST PILES, HP 12 X 53			1 @ 24’ 1 @ 35’		
AND 31,563	1002 0053 LB		REINFORCEMENT BARS, EPOXY-COATED	2,270		186	1,000	28,107
AND 1,672	1005 1103 LF		STEEL BEAM BEARING PILES, HP 12 X 53	910		762		
AND 75	1005 1153 EACH		STEEL BEAM PILE TIP REINFORCEMENT, HP 12 X 53	42		33		
OR LS	8100 0002 LS		STEEL BRIDGE STRUCTURE					
AND (---)	1005 1103 LF		STEEL BEAM BEARING PILES, HP 12 X 53	(---)		(---)		
AND (---)	3005 1103 EACH		STEEL BEAM PILE TIP REINFORCEMENT	(---)		(---)		

Figure 3.3.4.6-11 - Lump Sum Bridge Structure with Prefabricated Wall Abutments and/or Wingwalls where Conventional Abutments and Wingwalls are Not Permitted (Continued)

TABULATION OF BRIDGE BID ITEMS & APPROXIMATE QUANTITIES							
OR LS	8000 0003 LS	PRESTRESSED CONCRETE BRIDGE STRUCTURE					
AND (---)	1005 1103 LF	STEEL BEAM BEARING PILES, HP 12 X 53	(---)		(---)		
AND (---)	1005 1153 EACH	STEEL BEAM PILE TIP REINFORCEMENT, HP 12 X 53	(---)		(---)		
LS	8641 0001 LS	MECHANICALLY STABILIZED ABUTMENT OR WINGWALL, S-XXXXXP	(---) SF		(---) SF		
OR LS	8642 0001 LS	PRECAST MODULAR ABUTMENT OR WINGWALL, S-XXXXXP	(---) SF		(---) SF		

Figure 3.3.4.6-11 - Lump Sum Bridge Structure with Prefabricated Wall Abutments and/or Wingwalls where Conventional Abutments and Wingwalls are Not Permitted (Continued)

Include Conventional Abutment and Wingwall Design (Refer to Figure 6):

Items shown on this table are incomplete for an actual report and are shown merely as an example of the proposed items layout.

TABULATION OF BRIDGE BID ITEMS & APPROXIMATE QUANTITIES								
QUANTITY	ITEM NO UNIT	INFORMAL QUANTITY & UNIT	DESCRIPTION	NEAR ABUTMENT	PIER	FAR ABUTMENT	SUPERSTR	DECK
EITHER LS	8010 0001 LS		BRIDGE STRUCTURE, AS DESIGNED, S-XXXXX					
		468 M3	CLASS 3 EXCAVATION	221	76	171		
		930 M3	SELECTED BORROW EXCAVATION, STRUCTURE BACKFILL	489	76	365		
		300 M3	CLASS A CEMENT CONCRETE	137	76	87		
		LS	STEEL BEAM TEST PILES,HP 310 X 79	2 @ 11 m				
		LS	STEEL BEAM TEST PILES,HP 310 X 79			1 @ 7 m 1 @ 11 m		
AND 14 318	3002 0053 kg		REINFORCEMENT BARS,EPOXY-COATED	1030		84	454	12750
AND 509	3005 1103 M		STEEL BEAM BEARING PILES, HP 310 X 79	737		232		
AND 75	3005 1153 EACH		STEEL BEAM PILE TIP REINFORCEMENT, HP 310 X 79	42		33		
OR LS	8100 0002 LS		STEEL BRIDGE STRUCTURE					
AND (---)	3005 1103 M		STEEL BEAM BEARING PILES, HP 310 X 79	(---)		(---)		

Figure 3.3.4.6-12 - Conventional Abutment and Wingwall Design

TABULATION OF BRIDGE BID ITEMS & APPROXIMATE QUANTITIES							
AND (---)	3005 1103 EACH	STEEL BEAM PILE TIP REINFORCEMENT	(---)		(---)		
OR LS	8000 0003 LS	PRESTRESSED CONCRETE BRIDGE STRUCTURE					
AND (---)	3005 1103 M	STEEL BEAM BEARING PILES, HP 310 X 79	(---)		(---)		
AND (---)	3005 1153 EACH	STEEL BEAM PILE TIP REINFORCEMENT, HP 310 X 79	(---)		(---)		

Figure 3.3.4.6-12 - Conventional Abutment and Wingwall Design (Continued)

Items shown on this table are incomplete for an actual report and are shown merely as an example of the proposed items layout.

TABULATION OF BRIDGE BID ITEMS & APPROXIMATE QUANTITIES								
QUANTITY	ITEM NO UNIT	INFORMAL QUANTITY & UNIT	DESCRIPTION	NEAR ABUTMENT	PIER	FAR ABUTMENT	SUPERSTR	DECK
EITHER LS	8010 0001 LS	BRIDGE STRUCTURE, AS DESIGNED, S-XXXXX						
		613 CY	CLASS 3 EXCAVATION	289	100	224		
		1,217 CY3	SELECTED BORROW EXCAVATION, STRUCTURE BACKFILL	640	100	477		
		393 CY	CLASS A CEMENT CONCRETE	179	100	114		
		LS	STEEL BEAM TEST PILES, HP 12 X 53	2 @ 37’				
		LS	STEEL BEAM TEST PILES, HP 12 X 53			1 @ 24’ 1 @ 35’		
AND 14318	1002 0053 LB	REINFORCEMENT BARS,EPOXY-COATED		2,270		186	1,000	28,107
AND 1,672	1005 1103 LF	STEEL BEAM BEARING PILES,HP 12 X 53		910		762		
AND 75	1005 1153 EACH	STEEL BEAM PILE TIP REINFORCEMENT, HP 12 X 53		42		33		
OR LS	8100 0002 LS	STEEL BRIDGE STRUCTURE						
AND (---)	1105 1103 LF	STEEL BEAM BEARING PILES, HP 12 X 53		(---)		(---)		

Figure 3.3.4.6-12 - Conventional Abutment and Wingwall Design (Continued)

TABULATION OF BRIDGE BID ITEMS & APPROXIMATE QUANTITIES							
AND (---)	1005 1103 EACH	STEEL BEAM PILE TIP REINFORCEMENT	(---)		(---)		
OR LS	8000 0003 LS	PRESTRESSED CONCRETE BRIDGE STRUCTURE					
AND (---)	1005 1103 LF	STEEL BEAM BEARING PILES, HP 12 X 53	(---)		(---)		
AND (---)	1005 1153 EACH	STEEL BEAM PILE TIP REINFORCEMENT, HP 12 X 53	(---)		(---)		

Figure 3.3.4.6-12 - Conventional Abutment and Wingwall Design (Continued)

Does Not Include Conventional High Abutment and Wingwall Design (Refer to Figure 6):

Items shown on this table are incomplete for an actual report and are shown merely as an example of the proposed items layout.

TABULATION OF BRIDGE BID ITEMS & APPROXIMATE QUANTITIES								
QUANTITY	ITEM NO. / UNIT	INFORMAL QUANTITY & UNIT	DESCRIPTION	NEAR ABUTMENT	PIER	FAR ABUTMENT	SUPERSTR.	DECK
EITHER LS	8010 0001 / LS	BRIDGE STRUCTURE, AS DESIGNED, S-XXXXX						
		468 M3	CLASS 3 EXCAVATION	221*	76	171*		
		930 M3	SELECTED BORROW EXCAVATION, STRUCTURE BACKFILL	489*	76	365*		
		300 M3	CLASS A CEMENT CONCRETE	137*	76	87*		
		LS	STEEL BEAM TEST PILES, HP 310 X 79	2 @ 11 m				
		LS	STEEL BEAM TEST PILES, HP 310 X 79			1 @ 7 m 1 @ 11 m		
AND 14318	3002 0053 / KG	REINFORCEMENT BARS, EPOXY-COATED		1030		84	454	12 750
AND 509	3005 1103 / M	STEEL BEAM BEARING PILES, HP 310 X 79		277		232		
AND 75	3005 1153 / EACH	STEEL BEAM PILE TIP REINFORCEMENT, HP 310 X 79		42		33		
OR LS	8100 0005 / LS	STEEL BRIDGE STRUCTURE (PREFABRICATED ABUTMENTS)						
AND (---)	3005 1103 / M	STEEL BEAM BEARING PILES, HP 310 X 79		(---)		(---)		

Figure 3.3.4.6-13 - Does Not Include Conventional High Abutment and Wingwall Design

TABULATION OF BRIDGE BID ITEMS & APPROXIMATE QUANTITIES							
AND (---)	3005 1153 EACH	STEEL BEAM PILE TIP REINFORCEMENT, HP 310 X 79	(---)		(---)		
OR LS	8000 0006 LS	PRESTRESSED CONCRETE BRIDGE STRUCTURE (PREFABRICATED ABUTMENTS)					
AND (---)	3005 1103 M	STEEL BEAM BEARING PILES,HP 310 X 79	(---)		(---)		
AND (---)	3005 1153 EACH	STEEL BEAM PILE TIP REINFORCEMENT, HP 310 X 79	(---)		(---)		
LS	8640 0001 LS	PREFABRICATED ABUTMENT OR WINGWALL, S-XXXXXP	(---) M2		(---) M2		

Figure 3.3.4.6-13 - Does Not Include Conventional High Abutment and Wingwall Design (Continued)

*Adjust Quantities

(List applicable prefabricated wall systems on the drawing)

Items shown on this table are incomplete for an actual report and are shown merely as an example of the proposed items layout.

TABULATION OF BRIDGE BID ITEMS & APPROXIMATE QUANTITIES								
QUANTITY	ITEM NO. / UNIT	INFORMAL QUANTITY & UNIT	DESCRIPTION	NEAR ABUTMENT	PIER	FAR ABUTMENT	SUPERSTR.	DECK
EITHER LS	8010 0001 / LS	BRIDGE STRUCTURE, AS DESIGNED, S-XXXXX						
		613 CY	CLASS 3 EXCAVATION	289*	100	224*		
		1,217 CY	SELECTED BORROW EXCAVATION, STRUCTURE BACKFILL	640*	100	477*		
		393 CY	CLASS A CEMENT CONCRETE	179*	100	114*		
		LS	STEEL BEAM TEST PILES, HP 12 X 53	2 @ 37’				
		LS	STEEL BEAM TEST PILES, HP 12 X 53			1 @ 24’ 1 @ 35’		
AND	1002 0053 / LB	REINFORCEMENT BARS, EPOXY-COATED		2,270		186	1,000	28,107
AND	1005 1103 / LF	STEEL BEAM BEARING PILES,HP 12 X 53		910		762		
AND 75	1005 1153 / EACH	STEEL BEAM PILE TIP REINFORCEMENT, HP 12 X 53		42		33		
OR LS	8100 0005 / LS	STEEL BRIDGE STRUCTURE (PREFABRICATED ABUTMENTS)						
AND (---)	1005 1103 / LF	STEEL BEAM BEARING PILES,HP 12 X 53		(---)		(---)		

Figure 3.3.4.6-13 - Does Not Include Conventional High Abutment and Wingwall Design (Continued)

TABULATION OF BRIDGE BID ITEMS & APPROXIMATE QUANTITIES							
AND (---)	1005 1153 EACH	STEEL BEAM PILE TIP REINFORCEMENT, HP 12 X 53	(---)		(---)		
OR LS	8000 0006 LS	PRESTRESSED CONCRETE BRIDGE STRUCTURE (PREFABRICATED ABUTMENTS)					
AND (---)	1005 1103 LF	STEEL BEAM BEARING PILES,HP 12 X 53	(---)		(---)		
AND (---)	1005 1153 EACH	STEEL BEAM PILE TIP REINFORCEMENT, HP 12 X 53	(---)		(---)		
LS	8640 0001 LS	PREFABRICATED ABUTMENT OR WINGWALL, S-XXXXXP	(---) SF		(---) SF		

Figure 3.3.4.6-13 - Does Not Include Conventional High Abutment and Wingwall Design (Continued)

*Adjust Quantities

(List applicable prefabricated wall systems on the drawing)

3.3.4.7 REQUIREMENTS FOR CONTRACTOR PREPARED PLANS

The drawings shall include all details, dimensions, quantities, and cross-sections necessary to construct the wall. The plans shall be prepared to Department standards (see PP1.6) and shall include, but not be limited to, the following items:

(a) A plan and elevation sheet or sheets for each wall, containing the following:

(1) An elevation view of the wall which shall indicate the elevation at the top of the wall at all horizontal and vertical break points, and at least every 15 000 mm {50 ft.} along the wall; elevations at the top of leveling pads and footings; the distance along the face of the wall to all steps in the footings and leveling pads; the designation of the type of panel or module; the length, size, and number of mesh or strips, and the distance along the face of the wall where changes in length of the mesh or strips occur; and the location of the original and final ground line

(2) A plan view of the wall which shall indicate the offset from the construction centerline to the face of the wall at all changes in horizontal alignment, limit of widest module, mesh, or strip, and the centerline of any drainage structure or drainage pipe behind or passing through or under the wall

(3) Any general notes required for constructing the wall

(4) All horizontal and vertical curve data affecting wall construction

(5) A listing of the summary of quantities provided on the elevation sheet of each wall for all items including incidental items

(6) A cross-section showing the limits of construction and, in fill sections, limits and extent of granular material placed above original ground

(7) Name of the material supplier

(b) All details including reinforcing bar bending details

(c) All details for foundations and leveling pads, including details for steps in the footings or leveling pads, as well as the factored bearing resistance and the maximum design bearing pressures

(d) All details for panels and modules, showing all dimensions necessary to construct the element, all reinforcing steel in the element, identification of panels with epoxy-coated reinforcement, and the location of reinforcement attachment devices embedded in the panels

(e) All details for construction of walls around drainage facilities, sign footings, and abutment piles

(f) All details of the architectural treatment

(g) All details for connections to barriers, copings, noise walls, and attached lighting

(h) Detailed erection plan, particularly construction sequencing for the wall

(i) Detailed computations for internal and external stability and life expectancy for reinforcement and hardware

(j) A design summary in the tabulation format shown in Figures 1 and 2, as applicable.

(k) When contractor designed alternate plans are submitted for proprietary walls, the S-number shown on the original design of the proprietary wall shall be suffixed by the letter P and appear on the alternate design plans.

(l) The following note shall appear on sheet one of the alternate design plans (above title block):

For additional design information, core borings and other geotechnical information not shown on these plans, refer to the original design plans, (S-number of original design plans).

The plans shall be prepared and signed by a Professional Engineer registered in the Commonwealth of Pennsylvania.

The number of sets of design drawings and computations and who they shall be submitted to are given below:

- MSE Walls:

  Two sets of design drawings and computations shall be submitted to the District Engineer/Administrator for review and approval.

- Precast Modular Walls:

  Four sets of design drawings and computations shall be submitted to the District Engineer, who shall submit three sets to the Bridge Quality Assurance Division (BQAD) for approval. The district shall retain one set. BQAD will send one set to the Materials and Testing Division (MTD). The District and MTD shall provide comments to BQAD.

Any related shop drawings shall be reviewed according to Appendix B by the District and compared with the approved design drawings for the proprietary walls. Approval of the shop drawings and notifications shall be made by the District.

The computations shall be legible and shall include an explanation of any symbols and computer programs used in the design of overturning. As specified in PP3.3.4.5, bearing pressure beneath the wall footing or mechanically stabilized embankment, and estimated settlement shall be clearly indicated.

COUNTY: ________
LR/SR: ________
SECTION: ________
STATION: ________
S. NO.: ________

MSE WALL DESIGN SUMMARY

(NOTE: FOR DEFINITION OF TERMS REFER TO A11.9)

(METRIC UNITS)

DATE: _______

WALL HEIGHT = H = ________ . **REINFORCEMENT LENGTH = L = ________ .** **WALL NO. _________ .**

REINF. LAYER NO.	DEPTH (mm)	σ_v (MPa)	K	f*	Np	$\sigma_H = \gamma_p \sigma_v K$	Le (mm)	SELECT STRIP/ MESH PANEL	$F_H = \sigma_H(A)$ (N)	F_y STRIP= $F_{H/A'}$ (MPa)	fy CONN.= ΔΔ (MPa)	$P_n = \varphi P_{fg}$ OR $P_n = \varphi P_{fs}$	N= ΔΔΔ	P_i	REMARKS

Maximum Design Bearing Pressure = ______.
Factored Bearing Resistance = ______.
Maximum Design Eccentricity = ______.
Maximum Allowable Eccentricity = ______.
Design Horizontal Force = ______.
Maximum Design Horizontal Resistance = ______.

Calculated Maximum Settlement = ______.

ΔΔ = 85% of max. tie tension to a depth of factored $0.6H_1$, increasing to 100% at the toe maximum of wall.
ΔΔΔ = No. of cross bars or strips in Maximum Design Failure Plane
A = $\dfrac{\text{A panel}}{\text{No. of strips or mesh}}$
A' = Area of strip or mesh

Figure 3.3.4.7-1 - Design Summary for Mechanically Stabilized Earth (MSE) Walls

COUNTY: ________
LR/SR: ________
SECTION: ________
STATION: ________
S. NO.: ________

MSE WALL DESIGN SUMMARY

(NOTE: FOR DEFINITION OF TERMS REFER TO A11.9)

(U.S. CUSTOMARY UNITS)

DATE: _______

WALL HEIGHT = H = ________ . **REINFORCEMENT LENGTH = L = ________ .** **WALL NO. _________ .**

REINF. LAYER NO.	DEPTH (ft.)	σ_v (ksf)	K	f*	Np	$\sigma_H=\gamma_p\sigma_vK$	Le (ft.)	SELECT STRIP/ MESH PANEL	$F_H=\sigma_H(A)$ (kips)	F_y STRIP= $F_{H/A'}$ (ksi)	Fy CONN.= ΔΔ (ksi)	$P_n=\varphi P_{fg}$ OR $P_n=\varphi P_{fs}$	N= ΔΔΔ	P_i	REMARKS

Maximum Design Bearing Pressure = ______.
Factored Bearing Resistance = ______.
Maximum Design Eccentricity = ______.
Maximum Allowable Eccentricity = ______.
Design Horizontal Force = ______.
Maximum Design Horizontal Resistance = ______.

Calculated Maximum Settlement = ______.

ΔΔ = 85% of max. tie tension to a depth of factored $0.6H_1$, increasing to 100% at the toe maximum of wall.
ΔΔΔ = No. of cross bars or strips in Maximum Design Failure Plane
A = $\frac{\text{A panel}}{\text{No. of strips or mesh}}$
A' = Area of strip or mesh

Figure 3.3.4.7-1 - Design Summary for Mechanically Stabilized Earth (MSE) Walls (Continued)

MODULAR WALL DESIGN SUMMARY

DATE: _______

COUNTY: ________
LR/SR: ________
SECTION: ________
STATION: ________
S. NO.: ________

(METRIC UNITS)

WALL HEIGHT = H = ________ . **WALL BATTER ________ .** **WALL NO. _________ .**
(Excluding Barrier)
FRICTION ANGLE - BACKFILL = ____________. **SURCHARGE:** LEVEL _______. SLOPING _______, ABUTMENT _______

Course From Top	**MODULE SIZE** (mm)					**WALL REINF.** (Size/Spacing)			Design Eccentricity	REMARKS
	Height	Width	Length	Wall Thickness (mm)	Internal Pressure (MPa)	Hoop Bars	Vertical Bars	Epoxy-Coated Y/N		

Design Footing Size: Toe (Width x Thickness) _____________.
Longit. Bars ____________.
Transv. Bars ____________.
Provide Footing Size: Toe (Width x Thickness) _____________.

Heel (Width x Thickness) _____________.
Longit. Bars ____________.
Transv. Bars ____________.
Heel (Width x Thickness) _____________.

Calculated Maximum Settlement = ________.
Maximum Design Horizontal Force = _______.
Maximum Design Horizontal Resistance = ______.

Maximum Design Bearing Pressure = _______.
Factored Bearing Resistance = ______.

Figure 3.3.4.7-2 - Design Summary for Modular Walls

MODULAR WALL DESIGN SUMMARY

DATE: _______

COUNTY: ________
LR/SR: ________
SECTION: ________
STATION: ________
S. NO.: ________

(U.S. CUSTOMARY UNITS)

WALL HEIGHT = H = ________ . **WALL BATTER** ________ . **WALL NO.** _________ .
(Excluding Barrier)
FRICTION ANGLE - BACKFILL = ____________**.** **SURCHARGE:** LEVEL _______. SLOPING _______, ABUTMENT _______

Course From Top	**MODULE SIZE** (in.)					**WALL REINF.** (Size/Spacing)			Design Eccentricity	REMARKS
	Height	Width	Length	Wall Thickness (in.)	Internal Pressure (ksi)	Hoop Bars	Vertical Bars	Epoxy-Coated Y/N		

Design Footing Size: Toe (Width x Thickness) _____________.
Longit. Bars ____________.
Transv. Bars ____________.
Provide Footing Size: Toe (Width x Thickness) _____________.

Heel (Width x Thickness) _____________.
Longit. Bars ____________.
Transv. Bars ____________.
Heel (Width x Thickness) _____________.

Calculated Maximum Settlement = ________.
Maximum Design Horizontal Force = _______.
Maximum Design Horizontal Resistance = ______.

Maximum Design Bearing Pressure = _______.
Factored Bearing Resistance = ______.

Figure 3.3.4.7-2 - Design Summary for Modular Walls (Continued)

3.4 BRIDGE PROTECTIVE SYSTEMS

3.4.1 Overlays

Generally, overlays should not be provided for new construction. However, if it becomes necessary to provide an overlay to correct for a poor riding surface or for other reasons, a 40 mm {1 1/2 in.} thick latex-modified concrete overlay is preferred.

For treatment of existing decks, refer to PP5.5.2.3.

3.4.2 Protective Coatings for Concrete Surfaces

(a) Superstructure

For bridge decks that are poured and opened to traffic between September 1 and March 1, a general note shall be added to the design plans specifying the application of a concrete sealant in accordance with Publication 408, Section 1001.3(k)6.

(b) Substructure

Concrete sealants shall be specified for substructure units in accordance with D5.4.3.6P(b).

3.4.3 Cathodic Protection

Cathodic protection is considered an effective means of stopping and preventing corrosion of reinforcement bars in concrete. Refer to PP5.5.2.7.

3.4.4 Protective Coatings for Steel

Generally, all new steel bridges shall be painted with an inorganic zinc-rich painting system as specified in Publication 408 for painting fabricated structural steel. Consideration may also be given to galvanizing or metallizing of steel members. When considering a galvanizing protection system, fabrication issues such as galvanizing tank sizes and lifting capacities of galvanizing facilities need to be evaluated. Galvanizers listed in Bulletin 15 should be consulted as to their capabilities and the feasibility of the process for the specific steel member(s) being considered. For metallizing of steel members see D6.7.3.

Certain rural locations in the state may have low rates of air pollution and, therefore, a steel superstructure would not be exposed to spray generated from deicing chemicals,. For such a location, the use of unpainted weathering steel may be considered, provided there is little potential for industrial development or significant increases in traffic in the area (see D6.4.1).

For epoxy-coated rebar see D5.4.3.6P.

3.4.5 Vitrified-Clay Liner Plates

The use of vitrified-clay liner plates shall be specified for any concrete structure that is located in a stream having a hydrogen ion concentration (pH) of 5.0 or less as determined by a laboratory analysis of a representative sample of water taken from the stream during a period of normal water level. The liner plates shall be placed on the face of the concrete from approximately 450 mm {18 in.} below the streambed to approximately 450 mm {18 in.} above normal water elevation. Grade SA sewer bricks meeting the requirements of AASHTO M 91 may be used for curved portions of the structure when the use of vitrified-clay liner plates is not practical.

The samples of water should be obtained when a representative degree of acidity is present. In recommending the use of vitrified-clay liner plates, the worthiness of the sample should be taken into consideration. A pH factor of 7.0 indicates that the water is neutral; a pH factor less than 7.0 shows the degree of acidity of the water; and a pH factor greater than 7.0 shows the degree of alkalinity.

3.4.6 Other Protective Systems

Other protective systems which are applicable to specific items of construction (e.g., piles, buried structures, MSE walls, and permanently anchored walls) may be found in the sections of the Manual that cover the respective items.

3.5 UTILITIES

3.5.1 Service Utilities

The design and review of bridge attachments and responsibility to assure compliance with attachment and license requirements are basic functions of District Bridge Units.

The Bridge Quality Assurance Division of Bureau of Design, Central Office, will provide additional guidance if necessary for the particular bridge attachment.

District Utility Units will act as the liaison between utility companies and District Bridge Units.

Refer to Design Manual, Part 5, Utility Relocation, Publication 16M, Chapter 7, for Utility Occupancy of Highways and Bridges, for general guidelines coordination procedure and guidelines for accommodation of utilities on structures.

3.6 MISCELLANEOUS

3.6.1 Bridge Lighting and Navigational Lighting

For lighting of bridges, see Design Manual, Part 1A, Chapter 7. Bridge-mounted highway lighting shall be avoided wherever possible. The designer shall investigate the possibility of mounting the lighting on an extended pier cap. If bridge-mounted lighting cannot be avoided, it shall be located as close to a pier as is practical. Lighting poles shall be installed upon completion of the entire superstructure to prevent damage due to traffic on half-width of deck slab. The designer shall insure that vibration of the lighting pole at the luminare is not exceeded by the vibration induced by traffic on the as-designed bridge by providing computational proof.

3.6.2 Energy-Absorbing Devices

For policy on impact attenuators, refer to Design Manual, Part 2, Chapter 12.

3.6.3 Sign Structures

For implementation of the design, design review and fabrication control of sign structures, see Publication 10A, Design Manual Part 1A, Chapter 7.

Standard Drawings BD-641M, BD-642M, BD-643M, BD-644M and BD-645M shall be followed for design of sign structures. Standard Drawings BC-741M, BC-742M, BC-743M, BC-744M and BC-745M shall be followed for fabrication and construction of sign structures. For design specifications, refer to AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaries and Traffic Signals, 4th edition, 2001 and AASHTO/AWS D1.5M/D1.5:2002 – Bridge Welding Code (refer to AWS D1.1 for welding not covered in D1.5).

Refer to ITS-1003M (Pub. 647M) for preparation of DMS structure layouts and contract drawings.

Selection of the type of sign structure should consider initial construction cost, type of sign (aluminum or VMS), foundation sizes, uniformity/consistency of sign structure types, and the ability to galvanize the structure.

It is preferable to galvanize a field section of a sign structure in a single dip as opposed to double dipping the field sections. Double dipping of field sections has caused component failures during the galvanizing process. To determine if a field section can be galvanized in a single dip, the designer should discuss with local galvanizers listed in Publication 35 Bulletin 15 the depth of a field section (out-to-out of the chord splice plates), the weight of a field section and length of a field section. If the structure cannot be galvanized in a single dip by a local galvanizer, then a different sign structure type should be evaluated, e.g. for a trichord structure which cannot be galvanized in a single dip, then a 4 chord structure should be evaluated.

All sign supports located within the clear zone must be shielded with a crashworthy barrier. If a barrier is required or used, the sign bridge shall be located just beyond the design deflection distance of the barrier to minimize the required span length.

For caisson foundation, core borings are recommended unless geology is known, such as roadway borings show consistency.

Sign structure shop drawings are to be reviewed and accepted at the District level.

3.6.3.1 BRIDGE-MOUNTED SIGNS

Signs shall be aligned parallel to the bridge if the skew angle is 80 or more. Otherwise, the signs shall be perpendicular to the traveling lanes underneath. For horizontally curved roadway for which the sign is placed on a bridge, the sign shall be perpendicular to the chord joining a point 107 000 mm {350 ft.} away from the sign and the sign location. This value may be reduced on low speed roads. The bottom of the luminaries or sign shall be 75 mm {3 in.} above the bottom of the fascia girder. The sign support brackets may be attached to barrier and/or girders using standard, approved details.. Lock nuts or lock washers shall be used to compensate for bridge vibration effect.

3.6.3.2 BRIDGE-MOUNTED SIGN STRUCTURE (OVERHEAD SIGNS)

Bridge-mounted sign structures shall be avoided wherever possible. If they cannot be avoided, they should be located as close to a pier support as is practical. The affected bridge components shall be designed to carry the additional loads due to the weight of the sign structures and wind. All sign supports located within the clear zone must be shielded with a crashworthy barrier. If a barrier is required or used, the sign bridge shall be located just beyond the design deflection distance of the barrier to minimize the required span length.

3.6.4 Sound Barriers Walls

3.6.4.1 GENERAL

(a) For acoustic requirements of barriers, wall sizing and location, see Design Manual, Part 1A, Chapter 7.

(b) Wall heights must equal or exceed the acoustical profile.

(c) Design sound barrier walls in accordance with the Working Stress Design (WSD) method.

(d) Construct sound barrier walls using precast concrete or steel posts and precast concrete panels in accordance with the Standard Drawings. Alternate wall types using steel, concrete, timber, masonry, plastic, or any other material must be approved by the Department prior to bidding. The use of aluminum is not permitted.

(e) Any proprietary sound barrier wall system used on Department projects must be approved through the Department evaluation and approval process, as specified in the Master Policy Statement (MPS) 418, prior to use during the design or construction phase (see PP1.14, Systems Approval). Only accepted sound barrier systems (refer to Publication 35, Bulletin 15 for approved suppliers) shall be used.

(f) Refer to Standard Drawings BC-776M, BC-777M, BC-778M, BC-779M, and BC-780M for fabrication and construction of sound barrier walls.

(g) Refer to Standard Drawings BD-676M, BD-677M, BD-678M, BD-679M, and BD-680M for design of sound barriers walls.

(h) Structure-related environmental commitments shall be carefully considered and justified. Where practical, avoid sound barriers or minimize their size by other mitigative measures including earth berms. If possible, refrain from making commitment on material type, i.e., steel, concrete, timber, masonry, etc., until after the final design stage, when all reasonable options can be considered by the designer for a structurally and economical sound barrier. Where it is necessary to deviate from standards, justifications and special approvals must be well documented.

(i) Provide fire hydrant openings or other highway access as required. Provide additional reinforcement around openings in accordance with the Standard Drawings.

(j) Provide access doors in the wall if required by local fire departments, for inspection, or if directed by the District Bridge Engineer, in accordance with the Standard Drawings.

(k) Provide a constant post spacing for the entire length of wall. The constant post spacing may be interrupted to miss drainage pipes, utilities, expansion joints, construction joints, and/or any other physical features. Variations must be accepted by the District Bridge Engineer.

(l) Sound Absorptive Panels: Indicate if sound absorptive panels are required to reduce the reflective sound when sound barriers are on both sides of the highway and are spaced closer than 30 000 mm {100'-0"} and if required by the noise study. If a sound absorptive material is required, only use a system that is Pre-Approved by the Department.

(m) Provide construction and fabrication tolerances in accordance with Publication 408, Section 1086.

(n) Provide S-Number in accordance with PP1.6.2.5.

(o) Plan presentation shall be in accordance with PP1.6 and PP3.6.4.12.

(p) Provide uncoated reinforcement bars in the footings and caissons, unless otherwise specified.

(q) Provide epoxy coated or galvanized reinforcement bars in the precast concrete posts, precast concrete panels, pedestals, and raised panel seats where the wall is within 4300 mm {14'-0"} of the edge of travel lane. Epoxy coated or galvanized reinforcement bars may be required if future widening is anticipated.

3.6.4.2 WALL TYPES

(a) Ground Mounted Sound Barrier Walls

(1) General

- Provide a minimum panel ground embedment of 150mm {6 inch}, unless otherwise specified.
- Grade ground to drain surface water away from wall. Provide slopes so water will not pond at or near the sound barrier.
- Ground mounted sound barrier walls must be protected by concrete barriers, unless the wall is located beyond the horizontal clear zone or if the bottom of the panels are a minimum of 1500 mm (5'-0") above the edge of pavement.
- Earth berms are permitted to be used in conjunction with Linear and Offset Walls. The top of the earth berm must be wide enough to permit construction of the wall.

(2) Linear Walls (Post and Panel)

- Minimum Wall Height (Top of wall to bottom of bottom panel) = 1219 mm {4'-0"}
- Maximum Wall Height (Top of wall to bottom of bottom panel) = 9144 mm {30'-0"}
- Maximum Post Spacing = 6096 mm {20'-0}
- Precast concrete posts with precast concrete panels (Refer to Standard Drawings BC-776M, BC-777M, BD-676M, and BD-677M)
- Steel posts with precast concrete panels (Refer to Standard Drawings BC-776M, BC-778M, BD-676M, and BD-678M)
- Timber posts with timber panels
- Masonry block
- or any other pre-approved wall system.

(3) Offset Walls

- Offset walls are defined as freestanding walls undulating to create a "fan" type appearance supported on a cast-in-place spread footing.
- Minimum Wall Height (Top of wall to top of footing) = 1829 mm {6'-0"}
- Maximum Wall Height (Top of wall to top of footing) = 9144 mm {30'-0"}
- Provide a minimum panel ground embedment of 610 mm {2'-0"}.
- Refer to Standard Drawing BC-780M and BD-680M for additional information.

(4) Earth Berms

- Earth berms are defined as berms constructed from natural earthen materials to act as a "natural" sound barrier. These types of barriers are typically constructed with surplus materials available from the project or materials transported from an off-site location.

- Construct earth berms in accordance with the Department's criteria.

- The following factors shall be considered when selecting earth berms:
  - Right-of-Way requirements
  - Visual implications
  - Maintenance and accessibility
  - Drainage implications

(b) Structure Mounted Sound Barrier Walls

(1) General

- Precast concrete posts are not permitted for structure mounted sound barrier walls, provide steel posts.

- Maximum Post Spacing = 3658 mm {12'-0"} unless otherwise specified.

- Provide lock nuts or lock washers due to structure vibrations in accordance with the Standard Drawings.

- Steel cables are required in all structure mounted precast concrete panels unless both sides of the panel are located beyond the horizontal clear zone or if the bottom of the panels are a minimum of 1500 mm {5'-0"} above the edge of pavement. Steel cables are always required in the bridge mounted precast concrete panels. (Refer to BC-779 for steel cable details.)

- For non-concrete panels, alternate means of positive connection must be provided if used in applications requiring cables, as stated above. The connections must be approved by the Chief Bridge Engineer prior to bidding.

- Mount sound barriers to structures in accordance with Standard Drawing BC-779M and BD-679M.

- Structure mounted sound barrier walls shall be designed and detailed to maintain bridge inspectability. For special conditions, the inspectability shall be determined by the Chief Bridge Engineer.

- Slip forming is not permitted for concrete traffic barriers when sound barriers are required.

(2) Sound Barrier Mounted on Traffic Barrier on Bridges (Steel Posts with Precast Concrete Panels)

- Minimum Wall Height (Top of wall to top of barrier) = 1219 mm {4'-0"}

- Maximum Wall Height (Top of wall to top of barrier) = 3048 mm {10'-0"}

- Minimum Traffic Barrier Height = 1070 mm {3'-6")

- Provide low-density (light weight) precast concrete panels on bridges when required in accordance with the requirements of Standard Drawing BD-601M.

- Stacked panels are not permitted on bridges.

- Design the traffic barrier, deck slab, and beams/girders considering the additional loads due to the sound barrier wall.

- Make provisions to allow for bridge movement in the design of bridge mounted sound barriers. (Refer to BD-679M for details)

- Provide expansion panels over bridge expansion joints. (Refer to BD-679M for details.)

(3) Sound Barrier Mounted on Traffic Barrier on Retaining Walls or Moment Slabs (Steel Posts with Precast Concrete Panels)

- Minimum Wall Height (Top of wall to top of barrier) = 1219 mm {4'-0"}

- Maximum Wall Height (Top of wall to top of barrier) = 4572 mm {15'-0"}

- Minimum Traffic Barrier Height = 1070 mm {3'-6")

- Design the traffic barrier, retaining wall, and/or moment slab considering the additional loads due to the sound barrier wall.

- Provide expansion panels when shoulder relief joint is required in the moment slab. (Refer to BD-679M for details.)

(4) Sound Barrier Mounted on Retaining Wall (Steel Posts with Precast Concrete Panels)

- Minimum Wall Height (Top of wall to top of retaining wall) = 1219 mm {4'-0"}

- Maximum Wall Height (Top of wall to top of retaining wall) = 6096 mm {20'-0"}

- Design the retaining wall considering the additional loads due to the sound barrier wall.

3.6.4.3 GEOMETRY AND LAYOUT

(a) All sound barrier walls located within the clear zone must be protected with a concrete barrier in accordance with the requirements of Design Manual, Part 2, Chapter 12. The positive protection from vehicles preclude the need for designing the sound barrier wall for a traffic impact load.

(b) Generally the alignment of the sound barrier shall be constructed at a continuously uniform distance from the roadway.

(c) Minimum horizontal distances (sight and stopping) shall be considered when determining sound barrier alignment.

(d) Avoid abrupt changes in the horizontal and vertical alignment of the sound barrier.

(e) If sound barrier is located along a curved roadway alignment, the alignment of the sound barrier is permitted to be broken up into chorded sections in order to simplify the layout of the sound barrier.

(f) Provide angled or corner posts when required. Refer to BC-777M and BC-778M for angle limitations.

(g) Locate sound barriers to avoid conflicts with utilities, drainage pipes, and/or any other physical feature.

(h) If the sound barrier height exceeds 4600 {15'-0"} consider using a sound barrier in combination with an earth berm to reduce the structure height of the sound barrier.

(i) Sound barriers will obstruct light as well as sound. Special consideration shall be given to possible roadway icing and other induced environmental conditions caused by the placement of the wall.

(j) Access to the residential side of the sound barrier shall be considered for inspection and maintenance.

(k) Ends of Sound Barriers Walls

- Ends of the sound barrier should be reduced in height (top of barrier to ground line) from their acoustically required height to a height of approximately 1500 mm {5’-0”} (to match Right-of-Way fence height). This may be accomplished by using sloped end panels or by increasing the sound barrier length and sloping or stepping the panels to create a more aesthetically pleasing sound barrier. Options are to be discussed with the Department and accepted by the District Bridge Engineer.

- Ends of the sound barrier could also be buried into existing or proposed slopes if the topography permits.

- If using precast concrete posts, provide end posts in accordance with BC-777M and BD-677M.

(l) Overlapping Sound Barriers Walls

- Sound barriers which overlap each other are usually constructed to allow access gaps for maintenance, inspection, or safety purposes. The general rule-of-thumb is that the ratio between the overlap distance and gap width shall be at least 4:1 to ensure negligible degradation of sound barrier performance. The location of the access gaps, if required, shall be coordinated with the Department.

(m) Structure Mounted to Ground Mounted Connection

- Refer to BD-679M for transition details from a structure mounted sound barrier wall to a ground mounted sound barrier wall.

3.6.4.4 PUBLIC INVOLVEMENT AND AESTHETICS

(a) Refer to PENNDOT Publication No. 24, Project Level Highway Traffic Noise Handbook.

(b) Refer to PENNDOT Publication No. 21, Making Sound Decisions about Highway Noise Abatement.

(c) Determine aesthetics considering the surrounding landscape and local architectural features. Aesthetics shall also be coordinated and approved by the District Environmental Manager.

(d) When meeting with the public only present wall types which are in accordance with the Standard Drawings or Pre-Approved by the Department as alternates.

(1) Consider and permit all feasible alternates. The Sound Barrier Standard Drawings are to be used unless restricted by the following requirements:

- Acoustical profile requirements
- Sound absorptive panels requirements
- Architectural surface treatments requirements
- Wall alignment
- or any other requirement

The above restrictions must be discussed and accepted by the Department prior to public involvement.

(2) If only one proprietary type is feasible it shall be specified, provided that justification is given for this selection (economic, noise abatement) and that approval has been obtained from the Chief Bridge Engineer prior to bidding.

(e) Architectural Surface Treatments (texture) and Color:

(1) The Department will decide the color and texture on the highway side of the proposed sound barrier wall unless there is third party funding involved.

(2) The public will be presented options for the color and texture on the residential side of the proposed sound barrier. The final decision shall be determined by the Project Team based on the opinions of the public.

(3) The color and texture choices shall be limited to prevent numerous different colors and textures on a given project. The sound barrier walls shall be consistent on a wall-to-wall basis, on the project, to simplify construction and to be economical.

(4) In some urban areas, sound barriers may be subjected to graffiti on their surfaces. In these locations, the surface texture selected shall be such that it is difficult to place the graffiti or such that the graffiti is easily removed. Sound barriers with rough textures and dark colors tend to discourage graffiti.

(5) Color on concrete surfaces may be obtained by using penetrating concrete stain and/or integral concrete stain.

(6) Color on steel surfaces shall be obtained by painting after galvanizing.

(7) Avoid using form liner finishes on both sides of the precast concrete panels. A form liner finish, on one side of the panel, along with a stamped finish, on the other side of the panel is permitted. Preferred option is to only use a form liner finish on one side.

(8) Raked or broom finishes are only permitted to be on one side of the panel.

(9) Stamped finishes may be permitted if accepted by the District Bridge Engineer.

(10) Form liner finishes are not recommended on the precast concrete posts.

(11) The architectural surface treatment thickness, on each side of precast concrete panel, is permitted to vary from 0 to 38 mm {1½ inch}, but the total average architectural surface treatment thickness, on both sides of the precast concrete panel, must not be greater than 38 mm {1½ inch}. Thicker architectural surface treatments may be permitted if accepted by the District Bridge Engineer and accounted for in the design of the precast concrete panel.

3.6.4.5 DESIGN SPECIFICATIONS AND DESIGN LOADS

(a) AASHTO Guide Specifications for Structural Design of Sound Barriers, 1989, and Interim Specifications 1992 and 2002.

Design loads and loading combinations of dead load, lateral earth pressure, live load surcharge, wind load, seismic load, ice load, and traffic impact load shall be in accordance with this Guide Specifications and as modified herein.

Modify the following sections as follows:

SECTION 2 - LOADS

1-2.1 Applied Loads

1-2.1.1 Dead Loads

Add the following:

Density of Normal Concrete {Unit Weight of Concrete}:
Density of Concrete = 2400 kg/m^3
{Unit Weight of Concrete = 150 pounds per cubic foot}

Density of Low-Density Concrete {Unit Weight of Light Weight Concrete}:
Density of Concrete = 1840 kg/m^3
{Unit Weight of Concrete = 115 pounds per cubic foot}

Density of Soil above top of Footing/Drilled Caisson {Unit Weight of Soil}:
Density of Soil = 1600 kg/m^3
{Unit Weight of Soil = 100 pounds per cubic foot}

1-2.1.2 Wind Load

Delete this section and replace with the following:

The design wind pressure for Ground Mounted and Structure Mounted Sound Barrier Walls shall be as indicated below.

The wind load includes a gust factor of 1.3 and a drag factor of 1.2 and is based on a maximum 50-year Mean Wind Velocity at 9144 mm {30'-0"} above the ground surface of 130 km/hr {80 mph}.

Height Zone is defined as the distance from the adjoining (adjacent) average ground line (streambed or lower roadway level for grade separation structures) to the centroid of the sound barrier.

(a) Ground Mounted Sound Barrier Walls (including Offset Walls)

Height Zone (ft.)	Wind Pressure (psf)
0 - 14.0	20
Over 14.0	28

Height Zone (mm)	Wind Pressure (kPa)
0 - 4267	0.96
Over 4267	1.34

(b) Structure Mounted Sound Barrier Walls

Design structure mounted sound barrier walls for a wind pressure equal to 1.77 kPa (37 psf).

1-2.1.3 Seismic Load

Add the following:

The acceleration coefficient, A, shall be taken from Figure D3.10.2-1.

1-2.1.5 Traffic Loads

Add the following:

Sound barrier walls are to be designed and detailed so they will not be impacted by traffic loads.

1-2.1.7 Ice & Snow Loads

Delete this section and replace with the following:

Ice load shall be based on a pressure on the wall of 0.14×10^{-3} MPa (0.14 kPa) {0.003 ksf (3 psf)} applied at only one face of the wall.

1-2.2 Load Combinations

1-2.2.1 Working Stress Design (WSD)

Use the following Group Loads and Allowable Overstresses:

Group I:	D + E + SC	100%
Group II:	D + W + E + SC	100%
Group III:	D + EQD + E	133%
Group IV:	D + W + E + I	100%

1-2.2.2 Load Factor Design (LFD)

Delete this Section.

1-2.2.3 Strength Reduction Factors, (phi)

Delete this Section.

SECTION 3 - CONCRETE

Use fully reinforced sections only, including all foundation components. In case of conflict between this Guide Specification and Department criteria in DM-4, the Department criteria will govern.

SECTION 8 – FOUNDATION DESIGN

1-8.2 Spread Footings

Delete the Minimum Factors of Safety for Overturning and Sliding and replace with the following:

Minimum Factors of Safety for Overturning for Footings supported on Soil:
Group I = 2.0
Group II = 2.0
Group III = 1.50
Group IV = 2.0

Minimum Factors of Safety for Overturning for Footings supported on or embedded in Rock:
Group I = 1.50
Group II = 1.50
Group III = 1.125
Group IV = 1.50

Minimum Factors of Safety for Sliding:
Group I = 1.50
Group II = 1.50
Group III = 1.125
Group IV = 1.50

Add the following:

Minimum Factor of Safety for Bearing Capacity for Footing supported on soil and rock:
Group I = 3.0
Group II = 3.0
Group III = 1.5
Group IV = 3.0

(b) AASHTO Standard Specifications for Highway Bridges, 15th Edition, 1992 and Interim Specifications 1993 and 1994.

Modify the following sections as follows:

SECTION 8 - REINFORCED CONCRETE

8.1 APPLICATION

8.1.1 General

Add the following:
Design sound barrier walls in accordance with the Working Stress Design (WSD) Method.

8.1.2 Notations

Revise notation of f'c as follows:
f'c = structural design strength, psi

8.2 CONCRETE

Replace A8.2 as follows:

Minimum mix design compressive strength (psi) shall be in accordance with Section 704.1(b) of Publication 408.

The following classes of cement concrete with the corresponding f'c are to be used for structural designs:

Class of Cement Concrete	f'c Structural Design Strength
AA, Modified	35 MPa (5,000 psi)
A	21 MPa (3,000 psi)
C	14 MPa (2,000 psi)

The use of different classes of cement concrete shall be as follows:

<u>Class AA Cement Concrete, Modified</u>
- Precast Concrete Panels
- Precast Concrete Posts

<u>Class A Cement Concrete</u>
- Pedestals
- Raise Panel Seats
- Footings
- Caissons

<u>Class C Cement Concrete</u>
- below bottom of footings when specified

Show the structural design strength (f'c) of the concrete for each part of the structure on the plans.

8.3 REINFORCEMENT

Supplement A8.3 as follows:

Provide Grade 420 {Grade 60} [fs = 160 MPa {24,000 psi}] deformed reinforcement bars. Do not weld reinforcement bars.

Provide Grade 450 {Grade 65} [fs = 160 MPa {24,000 psi}] plain welded wire fabric in the precast concrete panels.

Provide Grade 480 {Grade 70} [fs = 160 MPa {24,000 psi}] deformed welded wire fabric in the precast concrete posts.

8.3.5

Delete A8.3.5

8.5 EXPANSION AND CONTRACTION

8.5.3

Add the following:

The coefficient of thermal expansion and contraction for low-density {light weight} concrete shall be taken as 9.0 x $10^{-6}/^{o}$ C {5.0 x $10^{-6}/^{o}$ F}.

8.6 STIFFNESS

Supplement A8.6 as follows:

8.6.3P Moment of Inertia

The value of the moment of inertia for the computation of flexural stiffness of slabs, beams, columns, etc., shall be based on gross concrete section with the effect of reinforcement neglected.

8.19 LIMITS FOR SHEAR REINFORCEMENT

8.19.1 Minimum Shear Reinforcement

8.19.1.3

Minimum Shear reinforcement requirements are not to be waived on any projects in Pennsylvania.

8.20 SHRINKAGE AND TEMPERATURE REINFORCEMENT

8.20.1

Supplement A8.20.1 as follows:

The minimum temperature steel reinforcement of 265 mm^2/m {0.125 in^2 / foot} must always be met when the member is subject to temperature variation. Any member subject to loading or stress shall have minimum steel reinforcement of No. 13 bars at 300 mm {No. 4 bars at 12 inches) of No. 16 bars at 450 mm {No. 5 bars at 18 inches}. For ties in reinforced concrete I-posts for sound walls, the minimum steel reinforcing shall be No. 10 bars at 225 mm {No. 3 bars at 9 inches}. The temperature reinforcement requirements are also met with this steel provided the spacing does not exceed three times the member thickness. Any exception to these criteria must be approved by the Chief Bridge Engineer.

8.20.3P Minimum Temperature Reinforcement

The requirements of AASHTO Article 8.20 must be met when any member is subject to temperature variation. Parts permanently embedded below 900 mm {3'-0"} in the ground or below frost level may be considered not subject to temperature variation.

8.22 PROTECTION AGAINST CORROSION

8.22.1

The following shall replace A8.22.1.

The following minimum concrete cover shall be provided for reinforcement:

- Concrete cast against and permanently exposed to earth: 100 mm {4 in.}
- Concrete exposed to earth: 75 mm {3 in.}
  (Except 50 mm {2 in.} may be used to in raised panel seats)

- Concrete exposed to weather: 50 mm {2 in.}
  (Except 75 mm {3 in.} for ties in drilled caissons and pedestals)

- Precast Concrete Panels: 38 mm {1½ in.}

- Precast Concrete Posts: 44 mm {1¾ in.}

SECTION 10 - STRUCTURAL STEEL

10.1 APPLICATION

10.1.2P Design Method

Design sound barrier walls in accordance with the Working Stress Design (WSD) Method.

10.8 MINIMUM THICKNESS OF METAL

10.8.1 Structural Steel

Replace A10.8.1 with the following:

The web thickness of rolled beams shall not be less than 6 mm {0.23 inch}. The minimum base plate thickness shall be 19 mm {3/4 inch}.

10.24 FASTENERS (Rivets and Bolts)

10.24.5 Spacing of Fasteners

10.24.5.3

Replace A10.24.5.3 with the following:

When oversize or slotted holes are used, the minimum clear distance between edges of adjacent bolt holes in the direction of force and transverse to the direction of the force shall not be less than twice the diameter of the bolt.

10.24.7 Edge Distance of Fasteners

10.24.7.3

Replace A10.24.7.3. with the following:

When oversize or slotted holes are used, the clear distance between edges of hole and edges of members shall not be less than the diameter of the bolt

(c) AASHTO Standard Specifications for Structural Supports of Highway Signs, Luminaries and Traffic Signals, 4th Edition, 2001 and Interim Specifications 2002 and 2003.

3.6.4.6 PRECAST CONCRETE PANELS

(a) Provide Class AA Cement Concrete, Modified in the precast concrete panels.

(b) Provide normal density {weight} concrete for the ground mounted sound barrier panels.

(c) Provide normal density {weight} concrete panels for the structure mounted panels for the barriers mounted on retaining walls and moment slabs.

(d) Provide either normal density (weight) concrete or low-density {light weight} concrete panels for the structure mounted panels for the barriers mounted bridges. Refer to BD-601M for requirements.

(e) Panels shall be designed as simply supported beams using a 305 mm {1'-0"} strip width.

(f) Minimum Structural Panel Thickness:

(1) Panels for Linear Ground Mounted and Structure Mounted Walls = 114 mm {4 1/2 inch}
(Note: Panels in the Standard Drawings are designed using a 127 mm {5 inch} structural thickness.)

(2) Panels for Offset Ground Mounted Walls = 254 mm (10 inch)

(g) Architectural Surface Treatment Thickness: Refer to PP3.6.4.4(e)(11).

(h) Wind Load for Panel Design:

(1) Ground Mounted Sound Barriers Panels: Design all panels using a wind pressure equal to 1.34 kPa {28 psf}.

(2) Structure Mounted Sound Barrier Panels: Design all panels using a wind pressure equal to 1.77 kPa {37 psf}.

(i) Design panels to include the additional weight due to the architectural surface treatment thicknesses. Architectural surface treatment thicknesses are not permitted to be considered as a load carrying element. Stresses shall be calculated using only the structural panel thickness.

(j) The minimum horizontal and vertical reinforcement shall not be less than 0.001 times the gross area of the panel. The gross area shall include the structural panel thickness plus the average architectural surface treatment thickness.

(k) In addition to the group loads indicated in PP3.6.4.5(a), design panels for stresses due to stripping, handling, erection, and transportation in accordance with the Precast/Prestressed Concrete Institute (PCI) Design Handbook, Precast and Prestressed Concrete, 5th Edition, 1999, Chapter 5.

(1) Design panels both horizontally and vertically using the following equivalent static load multipliers in accordance with Table 5.2.1:

- Stripping caused by Form Suction and Impact = 1.50
  The 1.50 factor is for the condition when the panels are cast horizontally and lifted/stripped from the form. If panels are cast using a tilt-up table, the 1.50 factor may not be applicable and a reduced factor may be more appropriate. The reduced factor must be accepted by the Chief Bridge Engineer.

- Yard Handling = 1.20

- Erection = 1.20

- Transportation = 1.50

The equivalent static load multiplier is applied to the weight of the panel and used as an equivalent static service load.

(2) For stripping and yard handling provide a minimum concrete compressive strength of 80% of the 28-day minimum design compressive strength.

(3) Design panel thickness and reinforcement for stripping and lifting using a two-point pickup or four point pickup in accordance with Figure 5.2.4. Place lifting inserts at the locations indicated in Figure 5.2.4. Lifting inserts are permitted to be on the top and one side of the panel as required for stripping and erection.

(4) In accordance with Section 5.2.4.1, the structural panel thickness shall be adequate such that the flexure tensile stress in the concrete using the uncracked gross section, while neglecting the reinforcement, is less than the Modulus of Rupture reduced by a safety factor of 1.50.

(l) Maximum permitted panel deflection equals panel length in millimeters {inches} divided by 360.

(m) Welded wire fabric is the preferred type of reinforcement in the precast concrete panel. #13 (#4) reinforcement bars may be substituted for welded wire fabric with an equivalent area at no additional cost to the Department.

(n) Provide perimeter reinforcement in the panel in accordance with the Standard Drawings. Minimum bar size shall be #13 {#4} for linear and structure mounted panels. Minimum bar size shall be #16 {#5} for offset panels.

(o) Minimize the number of horizontal panel joints and provide uniform steps. If steps are required, the elevation difference between adjacent panels is not permitted to be less than 150 mm {6 inches} or greater than 610 mm (2'-0"). Stacked panels are not permitted for bridge mounted barriers.

(p) Provide a 610 mm {2'-0"} maximum step between adjoining panels when stepping panels is required.

(q) Indicate if the top of panels are stepped or sloped. Sloped panels are preferred.

(r) Install panels truly vertical.

3.6.4.7 POSTS

(a) General

(1) Posts shall be designed as vertical cantilever beams.

(2) Maximum permitted post deflection equals post height (cantilever length) in millimeters {inches} divided by 360. The effects of rotation and deflection at the top of the drill caisson shall be ignored and only the relative displacement between the top of the caisson and top of post due to the applied loads shall be considered when calculating the deflections.

(3) Install posts truly vertical.

(b) Precast Concrete Post and Connections

(1) General

- Provide Class AA Cement Concrete, Modified in the precast concrete posts.

- Provide normal density {weight} concrete for the ground mounted sound barrier posts.

- The minimum vertical post reinforcement ratio shall be in accordance with the following equation:

$$\rho_{min} = \left[10 + \frac{(I/y_t)}{(bd^2)}\right]\left[\frac{(I/y_t)}{(bd^2)}\right]\frac{\sqrt{f'_c}}{f_y}$$

(2) Precast Concrete Post with Steel Base Plate and Anchor Bolts supported on Drilled Caissons or Spread Footings

- Provide epoxy coated or galvanized vertical post reinforcement bars.

- Provide threads on one end of the bar. Provide either normal threads on bar or provide upset threads. Specify type of bar to be used on the plans.

  - Normal Threads: Specify bar size and cut threads at one end.

    - Minimum vertical post reinforcement bar size shall be #16 (#5).

  - Upset Threads: Oversize bar with specially forged end for the length of threads.

- Minimum vertical post reinforcement bar size shall be #13 (#4).

- Design threaded reinforcement bars using the net tensile stress area of the threaded bar.

- Galvanize the steel base plate and anchor bolts.

- Paint exposed galvanized components to match wall color.

- Refer to PP3.6.4.8 for base plate design.

- Refer to PP3.6.4.9 for anchor bolt design.

(3) Precast Concrete Post Embedded in Drilled Caisson or Spread Footing (with or without pedestal)

- Provide plain, galvanized, or epoxy coated vertical post reinforcement bars as required.

- Minimum vertical post reinforcement bar size shall be #13 (#4).

- Post embedded in Drilled Caisson

  - The precast concrete post must be embedded to a depth where the caisson reinforcement is fully developed.

  - Vertical post reinforcement bars must extend a development length beyond the end of the post into the drilled caisson, except when the post is embedded to the bottom of the drilled caisson.

- Post embedded in Spread Footing

  - The precast concrete post must be embedded a minimum of 150 mm {6 inches}into the spread footing.

  - Provide 90 degree hooks on the vertical post reinforcement bars. The bar must extend a hooked development length beyond the end of the post. The hook length shall be based on a standard 90 degree hook.

(4) Precast Concrete Angled and Corner Post Embedded in Drilled Caisson or Spread Footing (with or without pedestal)

- Provide angle posts when the intersecting angle between adjacent panels is greater than 138 degrees and less than 162 degrees.

- Provide corner posts when the intersecting angle between adjacent panels is greater than 78 degrees and less than 102 degrees.

- Design Conditions:

  - Design Condition 1:
    Design post for a wind direction normal to the post with an effective width equal to the post spacing multiplied by the sine of the one-half the angle between the centerline of the panels. Design must investigate wind from both directions. The depth of the compression block must be verified so it does not extend beyond the flange of the post and that the tensile reinforcement is adequate. The post must also be designed for flexure, shear, deflection, minimum shear reinforcement, minimum flexural reinforcement, and flange bending stresses in accordance with the requirements of AASHTO and Design Manual, Part 4.

  - Design Condition 2:
    Design post for a wind direction normal to the panel with an effective width equal to one-half the post spacing applied to only one side of the post. Design must investigate wind from both directions and the combined torsion and shear capacity of the post must be verified. Checked torsion in accordance with ACI 318-99, Section 11.6.1. The post must also be designed for flexure, shear, deflection, minimum shear reinforcement, minimum flexural reinforcement, and flange bending stresses in accordance with the

requirements of AASHTO and Design Manual, Part 4.

- For additional information refer to PP3.6.4.7(b)(3).

(c) Steel Posts and Connections

(1) General

- Provide structural steel conforming to AASHTO M270M Grade 250 {36} [ASTM A709M {A709}, Grade 250 {36}], unless otherwise noted.

- The structural steel designed in the Sound Barrier Standard Drawings use AASHTO M270M Grade 250 {36}. Grade 345 {50} is permitted to be used as an alternate.

- Galvanize and paint the steel posts, base plates and anchor bolts.

- Weathering Steel (A588) is not permitted.

- Allowable Bending Stress shall be in accordance with AASHTO Standard Specifications for Highway Bridges, 15th Edition, 1992 and Interim Specifications 1993 and 1994, Section 10.32.1

(2) Steel Post with Steel Base Plate and Anchor Bolts supported on Drilled Caissons, Spread Footings, or Structures

- Refer to PP3.6.4.8 for base plate design.

- Refer to PP3.6.4.9 for anchor bolt design.

- Minimum fillet weld size = 10 mm {3/8 inch}

(3) Steel Post Embedded in Drilled Caisson or Spread Footing with Pedestal

- Fatigue design for welded studs, for ground mounted walls, shall be evaluated at two million cycles, non-redundant, Category C. Refer to AASHTO Standard Specifications for Highway Bridges, 15th Edition, 1992 and Interim Specifications 1993 and 1994, Section 10.38.5.1.1 and 10.38.5.1.2.

- Post embedded in Drilled Caisson

    - The centerline of the top welded stud must be located at a depth where the caisson reinforcement is fully developed.

    - The post and welded studs must extend beyond the centerline of the top stud so the applied forces can be transferred into the drilled caisson.

- Post embedded in Spread Footing

    - The centerline of the top welded stud must be located a minimum of 150 mm {6 inches} below the top of spread footing.

    - The post and welded studs must extend beyond the centerline of the top stud so the applied forces can be transferred into the spread footing.

(4) Steel Pipe Post Embedded in Drilled Caisson or Spread Footing (with or without pedestal)

- Provide steel pipe posts when the intersecting angle between adjacent panels is greater than 78 degrees and less than 160 degrees.

- Provide structural steel tubing conforming to ASTM A53M (A53), Grade B, Type E. [Fy = 240 MPa {35 ksi}]

- Design Conditions:

  - Design Condition 1:
    Design post for a wind direction normal to the post with an effective width equal to the post spacing multiplied by the sine of the one-half the angle between the centerline of the panels. Design the pipe post for flexure, shear, and deflection in accordance with the requirements of AASHTO and Design Manual, Part 4. In addition, the attached flange plates and weld must be checked for combined bending and shear stresses.

  - Design Condition 2:
    Design post for a wind direction normal to the panel with an effective width equal to one-half the post spacing applied to only one side of the post. Design the pipe post for flexure, shear, combined torsion and shear, and deflection in accordance with the requirements of AASHTO and Design Manual, Part 4. In addition, the attached flange plates and weld must be checked for combined bending and shear stresses.

- For additional information refer to PP3.6.4.7(c)(3).

3.6.4.8 BASE PLATES

(a) Base plates shall be designed for bending due to applied compression and tension anchor bolt forces. The anchor bolt force may be assumed to be distributed out at an angle of 45 degrees from the center of the anchor bolt.

(b) Determine base plate thickness in accordance with AASHTO Standard Specifications for Structural Supports of Highway Signs, Luminaries and Traffic Signals, 4th Edition, 2001 and Interim Specifications 2002 and 2003, Section 5.8.

(c) Minimum base plate thickness shall be 19 mm (3/4 inch).

(d) Support all base plates on washers and leveling nuts. Place non-shrink grout between the bottom of the base plate and supporting component. Pack grout into place, do not pour or inject. Due to construction issues related to the acceptable placement of the grout, the non-shrink grout shall not be considered as a load-carrying element.

(e) Base plates for ground mounted sound barrier walls designed and detailed in the Sound Barrier Standard Drawings are not designed to support the vertical load of the precast concrete panels.

(f) Base plates for structure mounted sound barrier walls designed and detailed in the Sound Barrier Standard Drawings are not designed to support the vertical load of the precast concrete panels even though the panels rests on the base plate. The reason for this is that the panel is continuously supported on top of the barrier or retaining wall, thus the base plate is not inducedd to any additional loading.

(g) Alternate base plate designs are permitted which include the vertical load of the panel supported on the base plate. Alternate designs must be accepted by the Chief Bridge Engineer.

3.6.4.9 ANCHOR BOLTS

(a) Anchor Bolts shall be designed in accordance with AASHTO Standard Specifications for Structural Supports of Highway Signs, Luminaries and Traffic Signals, 4th Edition, 2001 and Interim Specifications 2002 and 2003, Section 5.17 and 5.12.

  - Allowable Compression Stress = 0.5Fy (Maximum)

(b) Minimum anchor bolt diameter shall be 19.1 mm (3/4 inch).

(c) Anchor bolts shall be embedded to a depth sufficient to develop the yield strength of the anchor bolt. The minimum anchor bolt embedment length must extend to a depth where the supporting component reinforcement is fully developed, above and below the embedment length, and capable of transferring the applied forces.

(d) Anchor bolts shall be cast-in-place and placed before placing concrete. Preformed holes and drilled holes are not permitted.

(e) Adhesive anchors are not permitted.

3.6.4.10 FOUNDATIONS

(a) Spread Footings

(1) Spread footings shall be designed bearing on soil or rock in accordance with PP3.6.4.5.

(2) Design of spread footings shall consider if the proposed ground line is level or sloping.

(3) Design spread footings to include live load surcharge, if required.

(4) Provide a minimum soil depth of 457 mm {1'-6"} above the top of footing.

(5) Provide a minimum footing thickness of 457 mm {1'-6"}.

(6) Design footings for no uplift if supported on soil.

(7) Spread footings designed and detailed in the Sound Barrier Standard Drawings use the following parameters:

- Spread footings are designed bearing on soil with an allowable bearing pressure equal to 0.140 MPa {1.50 tons/square foot} and a coefficient of sliding friction equal to 0.30.
- Spread footings are designed for wind pressures equal to 0.96 kPa {20 psf} and 1.34 kPa {28 psf}.
- Spread footings are designed for level ground.
- Spread footings are designed for no uplift.
- Spread footings are designed for no live load surcharge.

(8) Alternate spread footing designs are permitted and are required if the foundation design parameters and site conditions differ from those indicated in the Standard Drawings. Foundation parameters must be accepted by the Department.

(b) Drilled Caissons (Shafts)

(1) The design of laterally loaded drilled caissons shall account for effects of soil layering, variable groundwater level, loss of lateral support, cyclic loading, combined axial and lateral loading and sloping ground.

(2) Drilled Caisson lengths shall be determined using COM624P or LPILE 5.0 computer program based on the site specific soil properties. An additional 914 mm {3'-0"} length must be added to the minimum caisson length determined by COM624P or LPILE 5.0 to account for freezing and thawing, weathering, and other shallow ground disturbances.

(3) Maximum lateral design displacement at top of caisson = 13 mm {1/2 inch}

(4) Maximum allowable vertical displacement = 25 mm {1 inch}

(5) Design of caissons shall consider if the proposed ground line is level or sloping.

(6) Design caissons to include live load surcharge, if required.

(7) Provide a minimum factor of safety against overturning of 2.0.

(8) Provide a minimum drilled caisson diameter of 762 mm {2'-6"}.

(9) Provide a minimum caisson length in soil equal to 3 times the caisson diameter.

(10) Drilled caissons designed and detailed in the Sound Barrier Standard Drawings use the following parameters:

- Drilled caissons are designed using the soil properties for the four soil types indicated.
- Drilled caissons are designed for a wind pressures equal to 1.34 kPa {28 psf}.
- Drilled caissons are designed for level ground.
- Drilled caissons are designed for no live load surcharge.

(11) Alternate drilled caisson designs are permitted and are required if the foundation design parameters and site conditions differ from those indicated in the Standard Drawings. Foundation parameters must be accepted by the Department.

- The designer should take full advantage of the site specific conditions rather than rely on the limited soil types indicated in the Standard Drawings in order to reduce the required caisson lengths and overall construction cost.

3.6.4.11 OFFSET WALLS

(a) Support offset wall panels on a cast-in-place spread footing.

(b) The offset wall stability analysis shall be based on a four-panel unit length. The stability analysis shall not include the spread footing since the wall is not rigidly attached to the spread footing. The stability analysis shall be made at the interface of the panel and footing. The minimum factor of safety against overturning and sliding shall be in accordance with PP3.6.4.5 for soil. Passive soil resistance is to be neglected.

(c) The bearing stress, fb, at the interface of the panel and footing shall not exceed 0.30 f'c. Negative bearing stress (i.e. uplift) is not permitted.

(d) Precast concrete panels shall be designed in accordance with PP3.6.4.6. Panels shall be designed for individual backfill lift heights and/or maximum soil height differential between the two sides of the panel. Individual backfill heights shall be indicated and shown on the contract drawings.

(e) Panel Connections

(1) Connect the panels together using 9.5 mm (3/8 inch) 7 x 19 stainless steel (Type 302 or 304) flexible wire rope (aircraft cable) with a minimum breaking strength of 53.4 kN (12 kips) and 12.7 mm (1/2 inch) stainless steel cap screws.

(2) Design the connection for the Group Loads indicated in PP3.6.4.5(a) in conjunction with a two degree wall tilt.

(3) Provide a minimum of two cable connections for each panel to panel connection. Provide a minimum of three cables connections for the end panel to adjacent panel connection.

(4) End panel shall be connected to the spread footing in accordance with the details shown on BC-780M.

(f) Spread Footings

(1) Spread footings shall be designed bearing on soil or rock in accordance with PP3.6.4.5.

(2) Design of spread footings shall consider if the proposed ground line is level or sloping.

(3) Design spread footings to include live load surcharge, if required.

(4) Provide a minimum soil depth (panel embedment depth) of 610 mm {2'-0"} above the top of footing.

(5) Provide a minimum footing thickness of 305 mm {1'-0"}.

(6) Provide a minimum footing width of 900 mm {3'-0"}.

(7) Design footings for no uplift if supported on soil.

(8) Provide footing steps as required.

- Minimum step height = 150 mm {6 inches}
- Maximum step height = 600 mm {2'-0"}

(9) Spread footings designed and detailed in the Sound Barrier Standard Drawings use the following parameters:

- Spread footings are designed bearing on soil with an allowable bearing pressure equal to 0.140 MPa {1.50 tons/square foot} and a coefficient of sliding friction equal to 0.30.
- Spread footings are designed for wind pressures equal to 0.96 kPa {20 psf} and 1.34 kPa {28 psf}.
- Spread footings are designed for level ground.
- Spread footings are designed for no uplift.
- Spread footings are designed for no live load surcharge.

(10) Alternate spread footing designs are permitted if the foundation design parameters and site conditions differ from those indicated in the Standard Drawings. Foundation parameters must be accepted by the Department.

3.6.4.12 PLAN PRESENTATION AND DESIGN ITEMS

Designs shall conform to Standards Drawings. The following information and details must be part of each submission:

(a) Beginning and end wall stations.

(b) Overall wall length.

(c) Horizontal alignment of sound barrier.

(d) Vertical alignment of sound barrier.

(e) Stake-out sketch including work point coordinates.

(f) Post spacing and type of post.

(g) Elevations (as required) indicating the acoustic profile, existing ground line, proposed ground line, top of traffic barrier and top of retaining wall. Elevations shall be given at a minimum 20 meter {50'-0"} interval. Provide additional elevations as required.

(h) Elevations of top and bottom of spread footings and drilled caissons, type of foundation, spread footing maximum allowable and design bearing pressures, drainage location; depth, and extent of any unsuitable material to be removed and replaced.

(i) Right-of-way limits.

(j) Construction Sequence.

(k) Quantity table showing the estimated sound barrier area in square meters {square foot} (show method of payment).

(l) Boring logs, when applicable.

(m) Approximate top of rock elevations, when applicable.

(n) Approximate ground water elevations, when applicable.

(o) Prepare design calculations for structural design, foundation design, post to foundation design or post to barrier anchor design (post to panel securing). Design calculations are not required if the designs and details are taken directly from the Standard Drawings.

(p) Design and dimension tables.

(q) Details of fit between panels and posts.

(r) Details of post connections to supporting components (i.e. spread footing, drilled caisson, traffic barrier, and retaining wall).

(s) Emergency access, maintenance access.

(t) If no approved wall type exists, all key structural foundation and acoustic items shall either be detailed on the contract plans or clearly specified in the construction specifications.

(u) Provide a signed statement from the District Environmental Manager indicating that the acoustic requirements of the proposed sound barrier have been reviewed and accepted.

(v) Limits of penetrating concrete stain.

(w) Any other information required to construct the sound barrier wall.

3.6.4.13 SPECIAL PROVISIONS

The following information must be specified in the contract special provisions:

(a) Type of permitted alternate wall types.

(b) Type of permitted post types. (Precast concrete or steel)

(c) Type of Architectural Surface Treatments on the residential and roadway sides on the precast concrete sound barrier panels and posts.

(d) Color of the Integral Pigmentation for the Precast Concrete sound barrier panels and posts.

(e) Color of joint sealing material and/or caulking compound and non-shrink grout.

(f) Color of Penetrating Concrete Stain.

(g) Paint Color of Steel components.

(h) Provide Federal Color Numbers in accordance with Federal Standard Number 595A or 595B.

(i) and any other information that may be required to construct the sound barrier wall.

3.6.4.14 ACOUSTIC PERFORMANCE SPECIFICATIONS

(a) Sound barrier panels shall achieve a minimum Sound Transmission Class (STC) of 25 as measured in accordance with ASTM E90-99.

(1) Precast concrete sound barriers panels with a minimum structural thickness equal to or greater than 100 mm (4 inch) will achieve a Sound Transmission Class (STC) of at least 25 and therefore do not require an Independent Laboratory Test Report.

(2) Precast concrete panels with a structural thickness less than 100 mm (4 inch) or panels which are constructed from other materials must be approved by the Department using the New Product Evaluation Process, prior to bidding, and must include an Independent Laboratory Test Report indicating the Sound Transmission Class (STC) achieves a value of 25 or more.

(b) Sound absorptive panels shall achieve a minimum Noise Reduction Coefficient (NRC) of 0.70 as measured in accordance with ASTM C423-02.

(1) Sound absorptive panels must be approved by the Department using the New Product Evaluation Process, prior to bidding, and must include an Independent Laboratory Test Report indicating the Noise Reduction Coefficient (NRC) achieves a value of 0.70 or greater.

3.6.4.15 SUBMISSIONS

Design and construction submissions shall be in accordance with PP1.9 and PP1.10 for ground and structure mounted sound barriers with the following modifications:

(a) The TS&L Report and Structure Geotechnical Foundation Report for ground mounted, retaining wall mounted, and moment slab mounted sound barrier walls shall be submitted concurrently. The submission requirements shall be in accordance with PP1.9.3 for TS&L and PP1.9.4 for Foundations.

(b) Each submission shall show the appropriate data (PP1.9.4 Foundations omitted) for Bridge Mounted sound barrier walls.

(c) Responsibilities as delineated for Bridge Submission in PP1.9 and PP1.10 are also applied to ground mounted sound barrier walls.

(d) Shop drawings are required in accordance with PP1.10.2.3.

3.6.4.16 DESIGN BUILD: MODIFIED TURNKEY

(a) Design Build Modified Turnkey projects shall be prepared, if directed by the Department, for the Ground Mounted Post and Panel Sound Barrier Walls which may allow the Contractor to determine the appropriate post type, post spacing, and foundation type based on the information provided in the contract documents. For additional information refer to PP1.11.

(b) Design Build Modified Turnkey projects shall be not be prepared for Ground Mounted Offset Sound Barrier Walls unless directed by the Department.

(c) Design Build Modified Turnkey projects shall be not be prepared for Structure Mounted Sound Barrier Walls since the post spacing and wall elements shall be set by the Design Engineer unless directed by the Department.

3.6.5 Pedestrian Structures and Bridges on Shared Use Trails

Enhancement and rails to trails pedestrian structures, involving both new construction and rehabilitation, shall be reviewed for critical areas such as the applicable material specifications, deflections, design loads, member dimensions, fabrication details, connections, and special provisions for erection and construction. If a proprietary product does not meet the criteria, it must be submitted and evaluated through the Department's Product Evaluation Process. Pedestrian and shared use trail structures may be produced only by Bulletin 15 approved fabricators meeting the requirements listed herein. The Department will provide full time in-plant quality assurance inspection during fabrication.

All pedestrian and shared use trail structures may be categorized in one of three groups as follows:

(a) Group I – Structures located on or over Department right-of-way

(b) Group II – Structures not located on or over Department right-of-way but crossing a public roadway (roadway owned by another local or state agency)

(c) Group III – Structures not on or crossing any public roadway (i.e.; structures in parks or crossing railroads)

3.6.5.1 GROUP I – STRUCTURES LOCATED ON OR OVER DEPARTMENT RIGHT-OF-WAY

All pedestrian and shared use trail structures located on or over Department final right-of-way shall conform to the DM-4 policies, procedures, and specifications, including appropriate design submissions. Certification acceptance (Stewardship and Oversight Agreement) procedures shall be followed. In all cases the designer shall stamp and seal the structure plans as per DM-4 PP1.6.3.1, and the Bridge Engineer shall review and approve the plans "For Structural Adequacy Only." Publication 408 specifications shall be used for construction and materials.

In every case, the structure must be competitively bid and allow multiple manufacturer's bridge types. These structures may be bid as designed with alternates or they may be bid as a Modified Turnkey (design-build) project. Highlights and exceptions to the specifications for these bridges are as follows:

(a) Department criteria must be followed. Note the following requirements for:

(1) Redundancy (DM-4 D1.3.4) – A redundancy analysis will be required for non-redundant structures

(2) Deflection (DM-4 D2.5.2.6.2) – L/1000 for metal bridges

(3) Live Load (DM-4 D3.6.1.6 and AASHTO LRFD A3.6.1.6) – Pedestrian load of 85 psf to be applied

(4) Inspection requirements (Pub 238) – Comprehensive inspection at 2-year maximum intervals

(5) Fatigue detail categories restrictions (DM-4 D6.6.1.2.4) – Category C or better detail must be provided

(6) Bearings and Joints (DM-4 D14) – Method A used for laminated neoprene bearings

(7) Fracture Critical Members (DM-4 PP1.7.7 (13))

(8) Construction and Fabrication (Publication 408) – Bridge fabricator must have current AISC certification to the Major Steel Bridge category with Fracture Critical endorsement.

(b) ASTM A500 (indocate Grade) and A847 materials may be used.

(c) ANSI/AWS/D1.1 is applicable for welding structural shapes to tubular members. (Also note Item 1.e above.)

(d) 100% of welds on main load carrying tubular members shall be non destructively tested as follows:

(1) Full penetration groove welds in butt joints shall be radiographically tested.

(2) Full penetration groove welds in T and corner joints shall be ultrasonically tested (UT). For material less than 5/16 in. (8mm) thick, UT procedures shall be submitted to the Chief Structural Materials Engineer for approval prior to use.

(3) Partial penetration groove welds and fillet welds shall be magnetic particle tested.

(e) Main load carrying member components of A709 steel subject to tensile stress shall meet the supplementary notch toughness requirements for the longitudinal Charpy V-notch test specified for Zone 2 in Table S1.2 (non-fracture critical) or S1.3 (fracture critical) of the applicable ASTM material specifications. A500 and A847 tubular members shall meet the requirements stipulated in the Tables for A709, Grade 50 material. Tubular members shall be tested at "P" piece frequency (sampled at one end of each length of tubing supplied) for fracture critical members, and at "H" (heat lot) frequency for non-fracture critical members, all in accordance with ASTM A673/A673M.

(f) SMAW, SAW, FCAW, and GMAW are approved welding processes, except that FCAW-S (self-shielding) and GMAW-S (short circuit arc transfer) will not be accepted for any welding.

(g) All Weld Procedure Specifications (WPS's) shall be submitted to, and approved by, the Chief Structural Materials Engineer prior to production welding, including tack welding. Prequalification of weld procedure specifications for welds on tubular members will be determined in strict compliance with Chapter 3 and Annex H of the latest edition of ANSI/AWS/D1.1 For welded non-tubular structures, welding and weld procedure qualification test should conform to ANSI/AWS/D1.5.

3.6.5.2 GROUP II – STRUCTURES NOT LOCATED ON OR OVER DEPARTMENT RIGHT-OF-WAY BUT CROSSING A PUBLIC ROADWAY

Review, approval and bidding requirements are the same as Group I. Highlights and exceptions to design criteria for these structures not on or over Department right-of-way but crossing a public roadway are as follows:

(a) A comprehensive structure inspection is completed as per Publication 238M every two years.

(b) ASTM A500 (indicate Grade) and A847 materials may be used.

(c) ANSI/AWS/D1.1 is applicable for welding structural shapes to tubular members.

(d) 100% of welds on main load carrying tubular members shall be non-destructively tested as follows:

(1) Full penetration groove welds in butt joints shall be radiographically tested.

(2) Full penetration groove welds in T and corner joints shall be ultrasonically tested (UT). For material lessthan 5/16 in. (8mm) thick , UT procedures shall be submitted to the Chief Structural Materials Engineer for approval prior to use.

(3) Partial penetration groove welds and fillet welds shall be magnetic particle tested.

(e) Main load carrying member components of A709 steel subject to tensile stress shall meet the supplementary notch toughness requirements for the longitudinal Charpy V-notch test specified for Zone 2 in Table S1.2 (non-fracture critical) or S1.3 (fracture critical) of the applicable ASTM material specifications. A500 and A847 tubular members shall meet the requirements stipulated in the Tables for A709, Grade 50 material. Tubular members shall be tested at "P" piece frequency (sampled at one end of each length of tubing supplied) for fracture critical members, and at "H" (heat lot) frequency for non-fracture critical members, all in accordance with ASTM A673/A673M.

(f) SMAW, SAW, FCAW, and GMAW are approved welding processes, except that FCAW-S (self-shielding) and GMAW-S (short circuit arc transfer) will not be accepted for any welding.

(g) All Weld Procedure Specifications (WPS's) shall be submitted to, and approved by, the Chief Structural Materials Engineer prior to production welding, including tack welding. Prequalification of weld procedure specifications for welds on tubular members will be determined in strict compliance with Chapter 3 and Annex H of the latest edition of ANSI/AWS D1.1. For welded non-tubular structures, welding and weld procedure qualification test should conform to ANSI/AWS/D1.5.

(h) The redundancy requirement (DM-4 D1.3.4) may be waived

(i) PennDOT specifications for neoprene bearings and expansion joints shall be used.

(j) Bridge fabricator must have current AISC certification to either the Simple Steel Bridge Structure category or the Major Steel Bridges Category.

(k) All fatigue details must be designed in accordance with AASHTO. (The Category C requirement may be waived.)

(l) Deflection must meet the criteria contained in the AASHTO Guide Specifications (L/500). DM-4 deflection criteria may be waived.

3.6.5.3 GROUP III – STRUCTURES NOT ON OR CROSSING ANY PUBLIC ROADWAY (I.E.: STRUCTURES IN PARKS OR CROSSING RAILROADS)

For locally sponsored and owned pedestrian structures involving state and/or federal funding which are located off of the Department final right-of-way and which do not cross a public road, the local owner may accept review responsibility. Examples of this would be a pedestrian trail bridge in a state or local park over a small creek or a pedestrian bridge over a railroad. In these cases, the AASHTO minimum criteria for design (see AASHTO Guide Specifications for Design of Pedestrian Bridges) may be used provided an independent check of the plans and computations for conformance to design criteria and structural adequacy is completed by a licensed Professional Engineer provided by the local owner. The designer shall stamp and seal the structure plans, and the review engineer shall sign and seal the plans using the following format:

Design reviewed by: PE Seal __________________________ Review Consultant’s Name, Signature and Date The design review is for general conformance with AASHTO design and construction criteria and is not intended to relieve the designer of full responsibility for the accuracy and completeness of the plans.

If the local owner does not accept review responsibility, the Department may be asked to provide a review. In this case, Group II criteria for pedestrian structures must be followed, and the District Bridge Engineer will approve the plans “For Structural Adequacy Only.” In these cases, inspection will be required from the local owner on a two-year cycle.

3.6.5.4 Designers and reviewers should be aware that the PennDOT Publication 408 is very specific in its specifications for construction, and may require prequalification for fabricators and/or specific fabrication practices. If a local project is designed using AASHTO only, the designer must provide special provisions to allow construction practices which deviate from Publication 408. In addition, if the structure crosses over a private entity such as a Railroad, all supplemental design requirements of that entity must be met.

3.6.5.5 For those pedestrian bridges with fracture critical members (FCM), FCM provisions (See DM-4, PP1.7.7, Note 13) will continue to be required for structures over public roadways and significant water crossings (waterway not able to be traversed by foot during normal flow). Please note that special provisions will need to be developed by the designer to allow any construction and material exceptions selected by the local municipality.

3.6.5.6 Recycled (used) bridges may be acceptable provided that the structure meets the following conditions:

- A complete inspection has been performed
- The material certifications are acceptable, or physical testing has been completed
- New connection material is utilized if the bridge is reconstructed or reassembled (new bolts, etc.), and
- The bridge is accepted by the District Bridge Engineer.

3.6.6 Usage of Unapproved Products

Any item not covered by the established standards, criteria or specifications and seeking approval in Bulletin 15 must go through the Bureau of Construction and Materials, Engineering Technology Division, New Product Evaluation Process.

With the increasing volume of new products being marketed for incorporation into Department construction projects, it is important that established procedures for evaluation of new products be followed. See Publication 51, Chapter 3C.

Use Master Policy Statement No. 418 for orderly product evaluation.

MPS No. 418 has been in effect for many years and was revised in August 1989 only to reflect Act 101 (1988), which requires mandatory recycling.

It is mandatory that new products should not be incorporated into Department projects unless approved for inclusion by the Chief Bridge Engineer.

3.6.7 Bridge Inspectability

For design requirements see D2.5.2.2.

To assist in reviewing deck girder/truss designs for inspectability using PennDOT underbridge crane, use the following guidelines and Figure 1.

1. Provide adequate lateral clearance from bridge superstructure to obstructions to permit crane boom deployment.

   (a) Horizontal clear distance between dual bridges or an obstruction "A" > 3000 mm {10 ft.} minimum (> 4600 mm {15 ft.} desirable). Clearances from electric power lines are critical.

2. For maximum horizontal reach of the crane under the bridge:

   Crane boom B-3 (telescoping) must be deployed horizontally. Therefore:

   (a) Vertical distance "D" < 6100 mm {20 ft.} and/or

   (b) Barrier/fences/sound barrier height "B" < 2700 mm {9 ft.} and/or

   (c) Depth to bottom of superstructure "C" < 3400 mm {11 ft.}

   (d) Horizontal distance from center of truck to outside edge of structure "E" < 4900 mm {16 ft.}

   With the above instructions:

   Maximum Horizontal Reach "H" = 13 400 mm {44 ft.}

3. Many bridge configurations outside the above restrictions are inspectable with a crane, but reach is compromised. For example, a deep girder with:

   (a) Barrier Height < 1070 mm {42 in.}

   (b) Depth to bottom of superstructure "C" < 6700 mm {22 ft.}

   (c) Truck adjacent to barrier "E" = 2100 mm {7 ft.}

   The maximum horizontal reach "H" is limited to 10 700 mm {35 ft.}

4. The bridge should be considered inspectable if <u>reasonable</u> access is provided within each bay of girders. Currently, it is not required to be able to touch every square millimeter of bridge superstructure from the crane for an inspection. Provide other means of access for inspection and maintenance when it is necessary or prudent and cost effective.

5. Contractor alternates and Value Engineering Proposals must provide the same level of inspectability as original designs.

6. These guidelines are based on current PennDOT Crane #4 (the second largest of PennDOT's current fleet of three cranes) to ensure that the bridge's inspectability is not limited to a single crane and that generic design specifications are prepared.

7. For through-truss bridges or those with other obstructions (power lines, buildings, etc.), restricted boom deployment may limit crane use. For some situations, inspectability by crane may have to be determined in the field.

8. If the District would like the Bridge Quality Assurance Division to review a specific bridge configuration, especially at the TS&L stage, submit a scaled drawing (1:50) {1/4” = 1’-0”} of the bridge with obstructions.

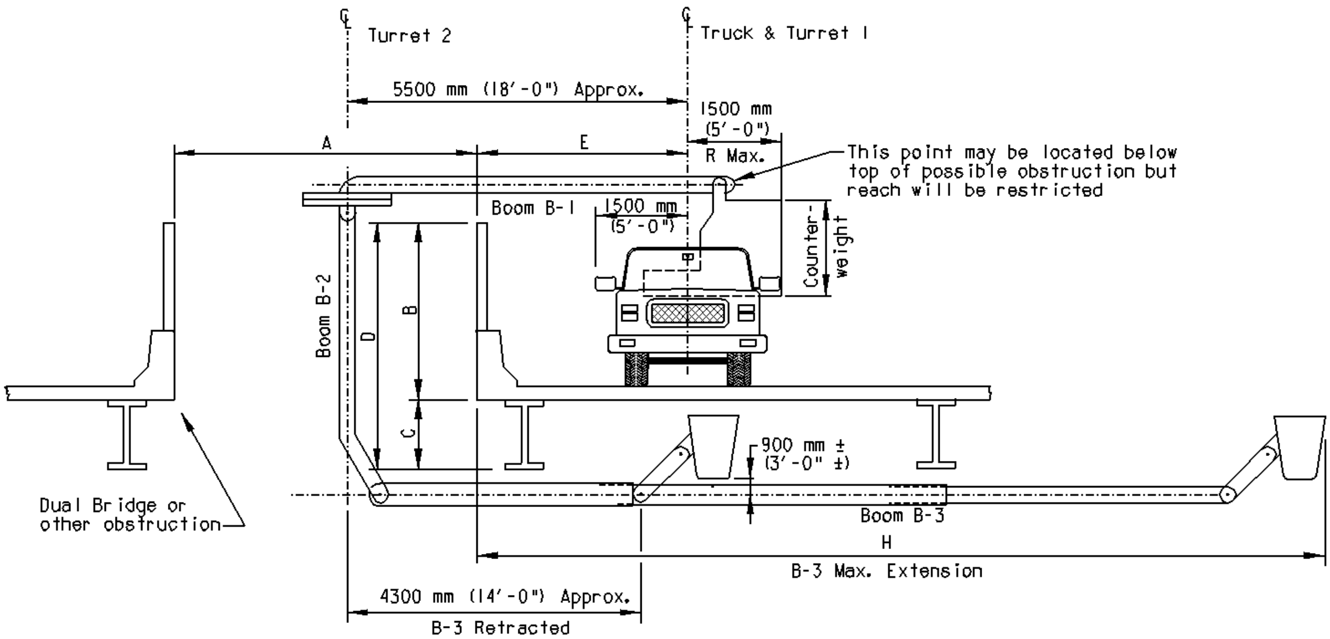


Figure 3.6.6-1 - PennDOT Underbridge Inspection Crane

3.6.8 Adhesive Anchors

Do not use adhesive anchors in a tension application for permanent installations.

3.6.9 Self Consolidating Concrete

Consideration can be given to the use of self consolidating concrete in piers, caissons, rehabilitation projects to aid in constructability and in precast members to aid in fabrication.

3.7 SPECIAL PROVISIONS

In the event that a design requirement or direction applying to any item of a project is not contained in, or deviates from, Publication 408, its supplements, or this Manual, the designer of the project shall write a special provision for its inclusion in the proposal. Such special provisions shall be in the imperative mood.

When alternates by contractors are permitted and the designer deviates from established design criteria (LRFD, Standard Drawings or this Manual), a special provision specifying the design criteria used shall be included in the contract documents. Any deviation from the established design criteria shall be approved by the Chief Bridge Engineer.

Refer to Publication 51 for guidelines concerning the preparation of contract proposals.

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 1
PART A: POLICIES AND PROCEDURES

CHAPTER 4 - BRIDGE ECONOMICS

CHAPTER 4 - TABLE OF CONTENTS

4.1 BRIDGE ECONOMICS - GENERAL

4.1.1 Cost Effective Bridges

There are many factors which will influence the cost of a bridge, such as type of superstructure, type of substructure, construction material, fabrication, transportation, erection, maintenance, traffic protection, etc. The cost of these factors changes with time, along with the cost relationship among them. The combined cost for superstructure and substructure for a given bridge site will determine the most economical bridge.

4.1.2 Economic Consideration

The following factors should be considered in determining the bridge cost.

1. Geometry:

   Work with the roadway designer in providing the simplest possible geometry on the bridge. A 90º skew bridge will not only simplify the design, but will offer the most economical construction. Similarly, variable width ramps, superelevation or superelevation transition, and simple, compound or spiral curves on the bridge should be avoided where practicable. Consideration of future widening for increased traffic demands may be considered if desired.

   Keep bridge geometry, including beam framing, as simple as possible.

   Whenever possible, consideration should be given to replacing part of the bridge length with an embankment. Sometimes, combination of several short-span bridges and embankments may be more economical than one long-span bridge.

2. Hydraulic Requirement:

   Hydraulic and regulatory requirements will dictate the minimum span length and minimum bottom of the beam elevation for bridges crossing streams or rivers.

3. Type of Structure:

   A precast or prefabricated bridge is usually more economical than a cast-in-place bridge.

   In selecting a bridge type, the designer should use the systematic elimination process. Start this process with all possible options as stated in PP4.1.3(A)1, then proceed to next level in PP4.1.3(A). Continue this process until the most economical and practicable type of structure for the required bridge length is found. This should not be construed as a requirement that all possible types of culverts or bridges should be cost analyzed in selecting a bridge type for a bridge location. Some of the options can be eliminated due to impracticality, proven uneconomical bridge type or known problems related to the bridge type(s) under consideration. A preliminary cost estimate may be performed to screen the first two or three most economical bridge types that survive the process of logical elimination.

   Where an automated design and drafting system, such as BRADD, is used, the designer should strive to provide additional contract drawings for different bridge types for the site. However, the District Bridge Engineer may determine the extent to which these additional contract drawings are needed based on past contractor alternate bid records, new technology, geometry, right-of-way, under clearance, utilities, etc.

   Refer to PP4.1.3 for superstructure selection.

   Refer to PP4.1.4 for substructure selection.

4. Maintenance Cost:

   Maintenance costs such as painting, repairs and other anticipated costs should be anticipated and may be considered in addition to the initial bridge construction cost.

5. Transportation, erection and fabrication cost:

   Consider costs which are particularly critical for special bridge types. For example, superbeams, with inherent transportation problems, will require a police escort.

6. Social Need:

   Local community needs should be considered. These needs should be identified at the environmental clearance stage. All possible bridge types which can be constructed at the site under consideration should be offered to the public. At public meetings, a commitment should not be made for one specific bridge type.

7. Delivery and Availability:

   Consider delivery time of alternate components and evaluate resulting cost impacts on community.

4.1.3 Selection of Superstructure Types

(a) The following structure types should be considered in developing a bridge superstructure.

1. Span less than 6000 mm {20 ft.}:

   In this span range generally, precast R.C. culverts or pipes, metal culverts or pipes, precast R.C. boxes, or precast R.C. slab bridges are considered more economical structures than cast-in-place R.C. box culverts. Cast-in-place arch culverts are rarely economical, but precast arches may be considered. Alternatives may be limited by class of highway, ADT, or ADTT. Where bedrock is within 900 mm to 1500 mm {3 ft. to 5 ft.} of the streambed or that of the lower roadway for the grade separation structures, cast-in-place rigid frame structures may be economical.

2. Spans 6000 mm to 9000 mm {20 ft. to 30 ft.}:

   In this span range, P/S plank beam, P/S box beam, or P/S channel beam bridges are generally more economical than steel I-beam bridge. Consideration should also be given to multiple precast R.C. boxes or culverts or multiple span cast-in-place box culverts in lieu of a single span bridge, but physical constraints and characteristics of the project site, such as debris potential and aquatic habitat, may need to be considered.

3. Spans 9000 mm to 27 000 mm {30 ft. to 90 ft.}:

   In this span range, P/S I-beam or box beam bridges with composite deck slab are generally more economical superstructures. P/S spread box beam with composite deck slab bridges are usually more economical than P/S I-beam or P/S adjacent box beam bridges. In the recent history, steel structures in this span range have not been proven economical. However, changing market conditions and bridge site conditions (low under clearance, sharp skew with large width variation, etc.) could make steel bridges in this span range economically attractive.

4. Spans 27 000 mm to 46 000 mm {90 ft. to 150 ft.}:

   In this span range, steel multi-girder bridge, P/S I-beam bridge and P/S box beam bridge may be equally cost effective. The final selection should be based on the cost analysis for each bridge type for each location.

5. Spans over 46 000 mm {150 ft.}:

   Bridges with span length over 46 000 mm {150 ft.} are more complex structures. Process of selecting the most economical type of structure will require that the designer develop preliminary design using different superstructure types, span arrangement and substructure types. Generally, for spans up to 75 000 mm {250 ft.}, multi-girder steel bridge may be an economical type of bridge. Refer to PP4.3.1 for economy of steel structures. However, consideration should also be given to spliced pre-post-tension concrete member bridge, drop-in span, or segmental bridges, etc. Note that Chief Bridge Engineer's approval is required at TS&L stage for the bridge types not covered in the Department standards or for which Department design specifications are not explicit. Bridges with fracture-critical non-redundant members should not be used.

(b) The following guidelines may be of further help to the designer in selecting the most economical bridge:

1. Previous designs for the similar bridge environment may be used as a guide by the designers.

2. Use of approved sophisticated design methods or softwares may aid in reducing cost of the structure. Refer to PP1.3.3 for selection of Design Methodology.

3. Minimize the number of girders at the bridge cross-section. Refer to PP3.2.1 for minimum number of girders required.

4. Designers should take advantage of higher material strength whenever possible.

5. Optimize the number of spans.

6. Use the maximum beam depth allowed by the underclearance.

7. Optimize the dead load of the bridge.

4.1.4 Selection of Substructure Types

a. Abutments can be reinforced concrete cantilever type, tie-back wall type, or approved proprietary wall type. Prefabricated walls and abutments are generally more economical than cast-in-place concrete walls and abutments. Refer to PP3.3.3 for economic consideration for substructure type selection. In most cases, the cost of a prefabricated wall abutment is generally about 50% or more of the conventional reinforced concrete abutment. Note that long-term settlement and service life must be considered in any substructure type selected.

b. When a reinforced concrete cantilever substructure is used, shallow spread footing on rock or good founding material is usually the most economical foundation. However, potential settlement and potential scour depth may require a deeper foundation.

c. When suitable rock is available at an average depth of less than 3000 mm {10 ft.} below the proposed bottom of footing, pedestal foundation or foundation which is made possible by removal of the overburden and backfilling with lean concrete or suitable material may be more economical than piling or drilled shafts. For depths greater than 3000 mm {10 ft.}, the piling is usually more economical than the drilled shafts. However, in special situations (where piles cannot be driven due to site conditions), augured or drilled concrete shafts have been proven to be more economical for 3000 mm to 9000 mm {10 ft. to 30 ft.} lengths. Where practical, an option of different foundation types shall be given to the contractor.

d. Integral abutments supported by piles may be economical and advantageous from future maintenance point of view, since they eliminate the joints at the abutments. Refer to Appendix G.

e. Minimizing the number of substructure units is usually more economical where a deep foundation is contemplated.

f. Substructure units should be optimized in shape and size to ease construction and economize quantity. Special forms should be avoided unless it is for aesthetic or other special reasons. However, site conditions must be satisfied.

4.1.5 Cost Analysis

A life-cycle cost analysis may be employed in determining the cost of the bridge. Approximate cost analysis may be used to screen the types of bridges on the preliminary list.

The following list includes some (but not all) of the most common factors that should be considered in the life-cycle cost analysis.

1. First cost of the structure.

2. Salvage value of the structure. Note that in most cases salvage value is negative (i.e., removal cost).

3. Design and engineering cost.

4. Maintenance costs.

5. Traffic maintenance and protection costs, during maintenance, rehabilitation or replacement.

6. Painting cost.

7. Deck replacement cost.

4.1.6 Final List of Bridge Type

At least three bridge types which survive the logical selection process should be submitted in the TS&L submission, together with life-cycle cost analysis and a final recommended bridge type. A submission containing less than three bridge types may be accepted if justification is provided.

For a major bridge, a minimum of two bridge types should be studied for each steel and concrete alternate designs. One bridge type per alternate may be accepted if a reasonable explanation is provided.

The Department policy is to encourage all possible contractor alternate designs in order to reduce bridge construction costs (PP1.10.1.1). Therefore, designers should submit (in the TS&L submission) the design and construction restriction requirement of these alternate designs for the later use in the development of special provisions for alternate designs. Designers may refer to Appendix D for important items which have to be addressed in alternate design special provisions.

4.1.7 Value Engineering

For value engineering concepts and theory, refer to Design Manual, Part 1A, Chapter 9, Section D. Value engineering is applied after TS&L, foundation or final plans are developed using common and best engineering concepts and materials known at the time of the plan development. The new substituted item must provide equal quality, service and longevity with the same or less maintenance need as for the item it replaces.

Value Engineering/Acceleration Construction Technology Transfer (VE/ACTT) is a workshop to develop cost effective and constructible projects through: deploying value engineering concepts prior to the start of final design or developing final contract documents; construction of a project on paper prior to final design to develop design around constructability; evaluating concepts and opportunities to expedite construction, and making preliminary design decisions to minimize permitting, design and construction time and costs. For projects where a Design VE is required and a VE/ACTT session has been held, the VE/ACTT satisfies the Design VE requirement per Design Manual Part 1A, Section H.1.2.

4.2 LIFE-CYCLE COSTS

Currently the Department utilizes the “First Cost” of the structure (for all materials) as the Life Cycle cost.

For bridges on Very Low Volume Roads, it is permissible to perform a life cycle cost analysis of the decking to justify use of alternate deck types.

4.3 STEEL GIRDER BRIDGES

4.3.1 Guidelines for Economical Steel Girder Bridges

Many factors influence the cost of a steel girder bridge, including (but not limited to) type of material, type of substructure, amount of material, fabrication time (the number of detail pieces and shop operations involved), transportation and erection. The cost of these factors changes with time, along with the cost relationship between them. Therefore, the rules used to determine the most economical type of steel girder on one bridge must be reviewed and updated for the next bridge.

The guidelines listed below are taken from the summary of "Economical Steel Plate Girder Bridges", Engineering Journal, AISC, 2nd Quarter, 1984, pp. 89-93. The designer must evaluate each of these guidelines for validity as it pertains to the specific structure in question. The designer should be aware that these guidelines are subject to change for numerous reasons. Also see NSBA document 12.1-2003 (Guidelines for Design for Constructability) for additional guidance.

These guidelines apply to spans up to 61 000 mm {200 ft.} in length, representing a majority of the bridge population, although some also apply to longer spans.

(a) Unpainted ASTM A 709/A 709M, Grade 345W {50W}, weathering steel is the most economical design. Properly designed in the appropriate environment, weathering steel bridges are more economical than those requiring painting of the whole structure. Note that the use of unpainted weathering steel requires special evaluation and the Chief Bridge Engineer's approval.

(b) Designs should use the fewest number of girders compatible with deck design and other factors. A girder spacing of 3000 mm {10 ft.} is suggested as the minimum for economical results. *Note that a minimum of four girders per bridge is required*.

(c) Web design can have a significant impact on the overall cost of a plate girder. From the standpoint of material costs, it is usually desirable to make girder webs as thin as design considerations will permit. However, this may not always produce the greatest economy since fabricating and installing stiffeners is one of the most labor intensive of shop operations. The following guidelines are provided for the use of stiffeners:

1. Transverse stiffeners (except diaphragm connections) should be placed on only one side of the web.

2. Longitudinal stiffeners used in conjunction with transverse stiffeners on longer spans with deeper webs should preferably be placed on the opposite side of the web from the transverse stiffener. Where this is not possible, such as at intersections with cross-frame connection plates, the longitudinal stiffener should not be interrupted for the transverse stiffener.

(d) Designs with web thickness which varies by field section are suggested.

(e) Longitudinally stiffened designs should not be considered for spans less than 90 000 mm {300 ft.}.

(f) Use no more than three plates (two shop splices) in the top or bottom flange of field sections up to 39 000 mm {130 ft.} long. In some cases, a single flange plate size should be carried through the full-length of the field section.

(g) Flange plates represent a significant portion of material costs. The amount of labor involved in fabricating flanges can vary significantly as a result of design. The most efficient way to make flanges is to butt weld together several wide plates of varying thickness received from the mill. After welding and non-destructive testing, the individual flanges are "stripped" from the full plate. This reduces the number of welds, individual run-off tabs to both start and stop welds, the amount of material waste and the number of x-rays for non-destructive testing. Therefore, it is preferable to keep flange widths constant within an individual shipping length by varying material thickness as required. This also makes it easier to use metal stay-in-place deck forms. This may not always be possible in girder spans over 105 000 mm {350 ft.} where a flange width transition may be required in the negative bending regions. Because plate is most economically purchased in widths of at least 1200 mm {48 in.}, it is best to repeat plate thickness as much as possible. An average of approximately 320 kg {0.700 kips} of flange material should be saved to justify the introduction of a flange splice.

When making flange transitions, consider two additional items:

1. It is good design practice to reduce the flange cross-sectional area by no more than approximately one-half of the area of the heavier flange plate to reduce the build-up of a stress at the transition.

2. If a transition in width must be provided, shift the butt splice a minimum of 75 mm {3 in.} from the transition. This makes it much easier to fit run-off tabs, weld and test the splice and then grind off the run-off tabs.

(h) Haunched girder designs should not be considered for most conventional cross-sections until spans exceed 120 000 mm {400 ft.}.

(i) Omit bottom lateral bracing where such omission is permitted by LRFD.

(j) Use elastomeric bearings or pot bearings in lieu of custom-fabricated steel bearings.

4.3.2 Rolled Beams

Unless otherwise directed, rolled beams shall be used where economical.

4.3.3 Composite Beams

Composite design is preferred for simple span and continuous bridges. Non-composite design may be used if it is more economical than composite design.

4.3.4 Intermediate Transverse Stiffeners

Straight girders may be designed with or without intermediate transverse stiffeners. For stiffened webs, the fabrication and handling costs must be considered in addition to material costs when determining the most economical design. Intermediate stiffeners may be designed on one or both sides of the web plates, but generally it is more economical to place stiffeners on one side of the web while placing on the opposite side only those stiffeners used also as connection plates.

4.3.5 Flange Plates

In the design of a welded plate girder, changes in flange area can be accomplished by varying the thickness and/or the width of the flange plate by appropriate increments. A constant top flange width simplifies the forming of the deck. This should be weighed against economies accrued by varying the top flange width.

4.3.6 Girder Depth

When economically practical, the depth of fascia girders in multiple spans shall be as similar as possible. The girder depth shall be selected from an evaluation of girder depths considered in the preliminary design.

4.3.7 Field Section Length of Girders

The field section lengths of girders shall be determined by the designer through a thorough evaluation of site conditions, economy and hauling considerations. Before initiating final design using lengths greater than 21 000 mm {70 ft.}, the designer must ensure that a hauling permit can be secured for the proposed length.

The designer is advised that superloads require special permits. A superload is defined as a vehicle with a gross weight over 201 kips {201,000 pounds) [894 kN], a length greater than 48 800 mm {160 ft.}, or a width greater than 4800 mm {16 ft.}. Therefore, a girder or group of girders with a weight of approximately 654 kN {147 kips} or greater, or a length of approximately 44 000 mm {144 ft.} or greater, would require vehicular transportation which is classified as a superload. Additional information concerning superloads can be found in PP1.13.2.

4.3.8 Structural Steels

There has been considerable discussion about the use of unpainted ASTM A 709/A 709M, Grade 345W or HPS-485W {50W or HPS-70W}, steel in bridges. It is obviously important to be extremely cautious in areas of constant wetting, such as overpasses in urban areas. However, in areas where deicing salts are not a significant factor, or where structures are over streams or other areas where tunnel-effect entrapment of salt spray is not likely, ASTM A 709/A 709M, Grade 345W or HPS-485W {50W or HPS-70W}, steel can probably still be of service. While the use of weathering steel can have a benefit with respect to long-term maintenance costs, it can also have a benefit in first-cost, which is the deciding factor in the current alternative bid situation. The first-cost savings come not only from the saving of the cost of the paint, but also savings in the surface treatment and handling of the finished girder in the fabrication process. For requirements that apply to the use of unpainted ASTM A 709/A 709M, Grade 345W or HPS-485W {50W or HPS-70W}, steel, see D6.4.1.

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 1
PART A: POLICIES AND PROCEDURES

CHAPTER 5 - REHABILITATION STRATEGIES

CHAPTER 5 - TABLE OF CONTENTS

5.0 DEFINITIONS AND NOTATIONS

The following definitions and notations are used in this chapter:

(a) Actual Stress Range - Fluctuation in stress experienced by the in-service member

(b) Constant Amplitude Stress Ranges - Stress ranges of equal minimum and maximum stress and, therefore, stress range magnitude

(c) Effective Stress Range - Constant amplitude stress range which is representative of a variable amplitude stress range spectrum

(d) Fatigue Life - Number of cycles to failure at a specified stress range

(e) GVM - Gross Vehicle Mass (formerly this was GVW - Gross Vehicle Weight)

(f) PTF - Pennsylvania Traffic Factor

(g) Probability of Failure - Statistical likelihood of a design criterion being exceeded

(h) Refined Methods of Analysis - Analysis in which live load bending moments carried by each girder are computed by techniques as follows: three-dimensional finite element method, two-dimensional grillage analogy, or a simple line girder analysis using sophisticated live load distribution factors

(i) Regression Analysis - Statistical analysis in which a curve is generated to approximate a random phenomenon

NOTATIONS

A = Design fatigue life coefficient (PP5.1.1.1.3)

$(\Delta f)_D$ = Calculated fatigue stress range (according to PP5.1.1.1.2.1)

$(\Delta f)_e$ = Effective stress range (PP5.1.1.1.2)

$(\Delta f)_i$ = Actual stress range of a random truck (PP5.1.1.1.2)

$(\Delta f)_{ic}$ = Calculated stress range of a random truck

N_{design} = Design fatigue life (PP5.1.1.1.3)

α = Ratio of the actual stress range caused by the passage of a particular vehicle to the calculated fatigue stress range caused by the passage of the same vehicle (PP5.1.1.1.2) (This value is to be taken as 1.0. If a value different than 1.0 is to be used, it must be approved by the Chief Bridge Engineer.)

v_i = Frequency of occurrence i (PP5.1.1.1.2) (decimal less than 1.0)

φ_i = $(\Delta f)_{ic}/(\Delta f)_D$ (PP5.1.1.1.2) (Ratio of calculated stress range of a truck to the calculated design stress range)

5.1 FATIGUE DAMAGE OF STEEL BRIDGES

5.1.1 Load-Induced Fatigue

5.1.1.1 LOAD-INDUCED FATIGUE DAMAGE ASSESSMENT

Cumulative fatigue damage of uncracked members and fasteners subject to repeated variations or reversals of load-induced stress shall be assessed according to the provisions of PP5.1.1.1.1 through PP5.1.1.1.5. The lists of detail categories and illustrative examples to consider in a fatigue damage assessment are shown in Table A6.6.1.2.3-1 and A6.6.1.2.3-2 and Figure A6.6.1.2.3-1.

If cracks have already been visually detected, a more complex fracture mechanics approach for load-induced fatigue is required instead of the procedure outlined here. The fracture mechanics approach is beyond the scope of this presentation. Further, the expense and trouble of a fracture mechanics analysis may not be warranted. Generally, upon visual detection of cracking, the vast proportion (perhaps over 80%) of the fatigue life has been exhausted and retrofitting measures should be initiated.

5.1.1.1.1 Infinite Fatigue Life

If the factored live load stress range, $\gamma(PTF)(\Delta f)$, produced by the method described in PP5.1.1.1.2.1 is less than one-half of the constant amplitude fatigue threshold given in Table A6.6.1.2.5-3, the detail shall be considered to have infinite life.

5.1.1.1.2 Finite Fatigue Life

5.1.1.1.2.1 Calculated Fatigue Stress Range

5.1.1.1.2.1.1 Approximate Method of Analysis

The factored live load stress range, $\gamma(PTF)(\Delta f)$, produced by the method given in the LRFD and the DM-4 (which is given in A3.4.1, D3.4.1, A3.6.1.4.1, A3.6.1.4.3b and D6.6.1.2.2) is considered the approximate method of analysis.

5.1.1.1.2.1.2 Refined Methods of Analysis

Before a refined method of analysis can be used for a finite fatigue life evaluation, the approval of Chief Bridge Engineer must be obtained.

When using refined methods of analysis for finite fatigue life evaluation, the stress range shall be calculated by moving the loading condition, given in A3.6.1.4.1, across the bridge in the critical transverse position.

5.1.1.1.2.2 Effective Stress Range

The entire collection of stress ranges ($(\Delta f)_i$) caused by actual truck traffic that a structural detail will experience during its life is called the stress range spectrum. If all of the stress ranges are of equal magnitude (i.e., the amplitude from minimum stress to maximum stress is constant), the spectrum is called a constant-amplitude stress range spectrum. Since vehicles crossing the bridge are of various weights and axle configurations, the stress range spectrum experienced by a bridge detail is not of constant amplitude, but is a variable-amplitude stress range spectrum.

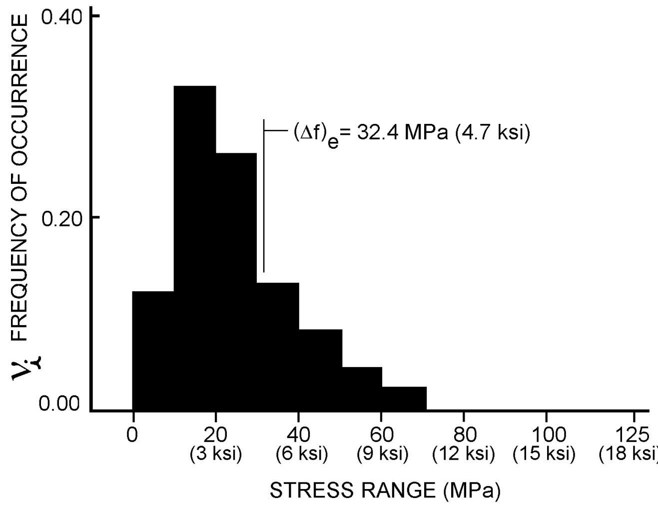
In terms of cumulative fatigue damage, an equivalent constant-amplitude spectrum will be used to represent the actual variable-amplitude spectrum experienced by a structural detail. Each type of spectrum consists of the same number of total stress range cycles (number of cycles = average daily truck traffic ($ADTT_{SL}$), as defined in A3.6.1.4.2 and D3.6.1.4.2, times the period of interest in days). The constant-amplitude stress range hypothesized to produce the same cumulative fatigue damage in the same total number of cycles as the variable-amplitude stress range spectrum is called that spectrum's effective stress range.

The effective stress range of a bridge detail shall be considered equal to

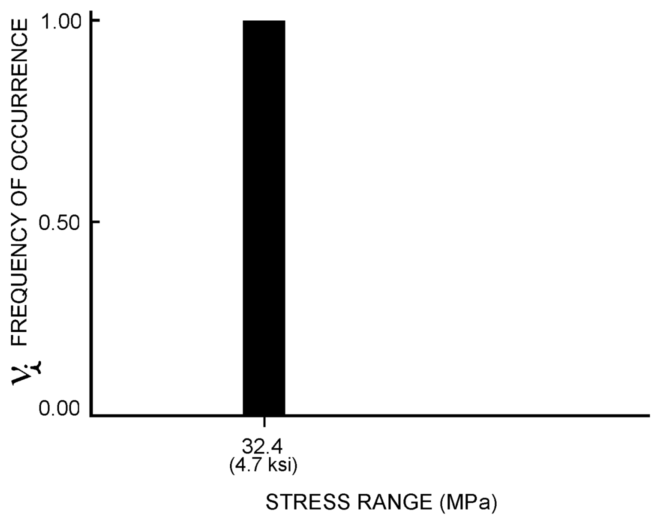
$$(\Delta f)_e = \left[\Sigma \nu_i (\Delta f)_i^3\right]^{1/3} \qquad (5.1.1.1.2.2\text{-}1)$$

where v_i is the frequency of occurrence (a decimal less than 1.0) of stress range $(\Delta f)_i$, and $(\Delta f)_i$ is an experimentally measured (not calculated) value. This equation was developed from the fatigue damage accumulation model most commonly used for bridge engineering applications.

The concept of effective stress range is illustrated in Figure 1. The histogram (or bar graph) shown in Part (a) of this figure represents the distribution of actual stress range that a particular bridge detail will experience during a period of time. Calculating an effective stress range according to Equation 1 yields 32.4 MPa {4.7 ksi}. The summation of the numbers of cycles represented in Part (a) of this figure is assumed to be equal to the $ADTT_{SL}$ times the period of interest in days.



(a) VARIABLE-AMPLITUDE HISTOGRAM



(b) CONSTANT-AMPLITUDE HISTOGRAM

Figure 5.1.1.1.2.2-1 - Effective Stress Range Concept

In Part (b) of this figure, a histogram is shown of the same total number of cycles, but all with amplitude equal to 32.4 MPa {4.7 ksi}. The accumulated fatigue damage which the distribution in Part (a) represents is assumed to be equal to that of the distribution in Part (b). In both histograms, the numbers of cycles represented is equal. In terms of accumulated fatigue damage, all of the variable amplitude stress range cycles of Part (a) can be replaced with an equal number of stress range cycles of constant amplitude equal to the effective stress range, as indicated in Part (b).

If the distribution of stress range (the v_i and $(\Delta f)_i$) is unknown, $(\Delta f)_e$ can be approximated for bridges in Pennsylvania as

$$\gamma(\mathrm{PTF})\,(\Delta\,\mathrm{f})_e = \gamma(\mathrm{PTF})\alpha\,(\Delta \mathrm{f})_D \qquad (5.1.1.1.2.2\text{-}2)$$

where:

$(\Delta f)_D$ = calculated fatigue stress range (according to PP5.1.1.1.2.1)

α = ratio of actual stress range to calculated fatigue stress range; the value is to be taken as 1.0. If a value different than 1.0 is to be used, it must be approved by the Chief Bridge Engineer.

PTF = Pennsylvania Traffic Factor as given in D6.6.1.2.2

γ = load factor for fatigue given in A3.4.1 and D3.4.1.1

The approximation for effective stress range given in Equation 2 is based upon the results of gross vehicle mass (GVM) surveys for Pennsylvania. The effective stress range can be more rigorously defined as follows:

$$\gamma(\Delta f)_{e'} = \left[\Sigma v_i \phi_i^{\,3}\right]^{1/3} \alpha(\Delta f)_D \qquad (5.1.1.1.2.2\text{-}3)$$

where:

v_i = frequency of occurrence of $(\Delta f)_i$

φ_i = $(\Delta f)_i/(\Delta f)_D$

α = ratio of actual stress range to calculated fatigue stress range; the value is to be taken as 1.0. If a value different than 1.0 is to be used, it must be approved by the Chief Bridge Engineer.

$(\Delta)_D$ = calculated fatigue stress range according to PP5.1.1.1.2.1

The 1970 FHWA Nationwide Loadometer Survey suggests that the summation of $v_i\ \varphi_i^3$ be taken as 0.35, as a national average. This value is the basis for the past AASHTO (pre-LRFD) design provisions for fatigue. In recognition that GVM have been increasing since the 1970 FHWA survey, the LRFD design provisions for fatigue are based on an assumed summation of $v_i\ \varphi_i^3$ equal to 0.422. However, the value can increase significantly for major truck arteries.

The most recent 1993 Pennsylvania GVM surveys of six locations indicated that for the worst location, I-80 in Clearfield County, the summation of $v_i\ \varphi_i^3$ is 0.76. The Department has decided to use this worst case value of the summation of $v_i\ \varphi_i^3$ equal to 0.76 (which will provide a PTF equal to 1.2) for freeways, expressways, major highways and streets with ADTT greater than 2500. A comparison of the 1970 FHWA, the 1973 Statewide PennDOT GVM surveys and 1993 PennDOT GVM survey of I-80 in Clearfield County is shown in Figure 2. For major truck arteries, the value of 0.76 may be unconservative and detailed development using the principles outlined here is recommended. GVM distribution and corresponding cumulative damage tabulations of the 1973 Statewide Survey and 1993 I-80 in Clearfield County survey are given in Tables 1, 2 and 3.

The factor α has been taken as 1.0. This value is inherent in the LRFD design provisions.

Table 5.1.1.1.2.2-1 - Pennsylvania Statewide Cumulative Damage by Truck Type (1973 Data)

Gross Vehicle Mass Range		φ_i (GVM_i/ GVM_D)	φ_i^3	Single-Unit Trucks				Tractor Semi-Trailer Combinations					
				2 axle		3 axle		3 axle		4 axle		5 axle or more	
(tonne)	(kips)			v_i(%)	$v_i\ \varphi_i^3$	v_i(%)	$v_i\ \varphi_i^3$	v_i(%)	$v_i\ \varphi_i^3$	v_i(%)	$v_i\ \varphi_i^3$	v_i(%)	$v_i\ \varphi_i^3$
9.1-13.6	20.0-29.9	t	0.042	85.4	0.03573	48.11	0.02013	55.78	0.02334	24.52	0.01026	8.28	0.00346
13.7-18.1	30.0-39.9	0.486	0.115	14.14	0.01624	21.62	0.02482	38.45	0.04415	32.76	0.03762	15.48	0.01777
18.2-22.7	40.0-49.9	0.625	0.244	0.44	0.00107	14.06	0.03431	4.48	0.01093	24.82	0.06058	12.68	0.03095
22.8-27.2	50.0-59.9	0.764	0.446	0	0.00000	8.65	0.03855	1.28	0.00570	12.17	0.05423	15.16	0.06756
27.3-31.7	60.0-69.9	0.903	0.736	0	0.00000	4.86	0.03575	0	0.00000	4.53	0.03332	24.15	0.17765
31.8-36.3	70.0-79.9	1.042	1.130	0	0.00000	1.08	0.01220	0	0.00000	1.1	0.01243	19.43	0.21957
36.4-40.8	80.0-89.9	1.180	1.645	0	0.00000	1.62	0.02665	0	0.00000	0	0.00000	4.04	0.06646
40.9-45.4	90.0-99.9	1.319	2.297	0	0.00000	0	0.00000	0	0.00000	0.1	0.00230	0.71	0.01631
45.5-49.9	100.0-109.9	1.458	3.101	0	0.00000	0	0.00000	0	0.00000	0	0.00000	0.07	0.00217
50.0-over	110.0-over	1.528	3.566	0	0.00000	0	0.00000	0	0.00000	0	0.00000	0.02	0.00071
Number of Trucks per Type				226		185		156		995		4361	
Percentage of Truck Type				3.8		3.1		2.6		16.8		73.6	
$\Sigma v_i\varphi_i^3$ by Truck Type* =				0.05304		0.19241		0.08412		0.21074		0.60261	

Total Cumulative Damage:

$\Sigma v_i\varphi_i^3$ = Σ [(Percentage of Truck Type) ($\Sigma v_i\varphi_i^3$ by Truck Type)]
= 0.038(0.05304) + 0.031(0.19241) + 0.026(0.08412) + 0.168(0.21074) + 0.736(0.60261)
= 0.489

*The quantity $\Sigma v_i\varphi_i^3$ shown for each truck type represents the cumulative fatigue damage caused by the distribution of trucks in each truck type as a percentage of that caused by a like number of vehicles, all of the design gross vehicle mass.

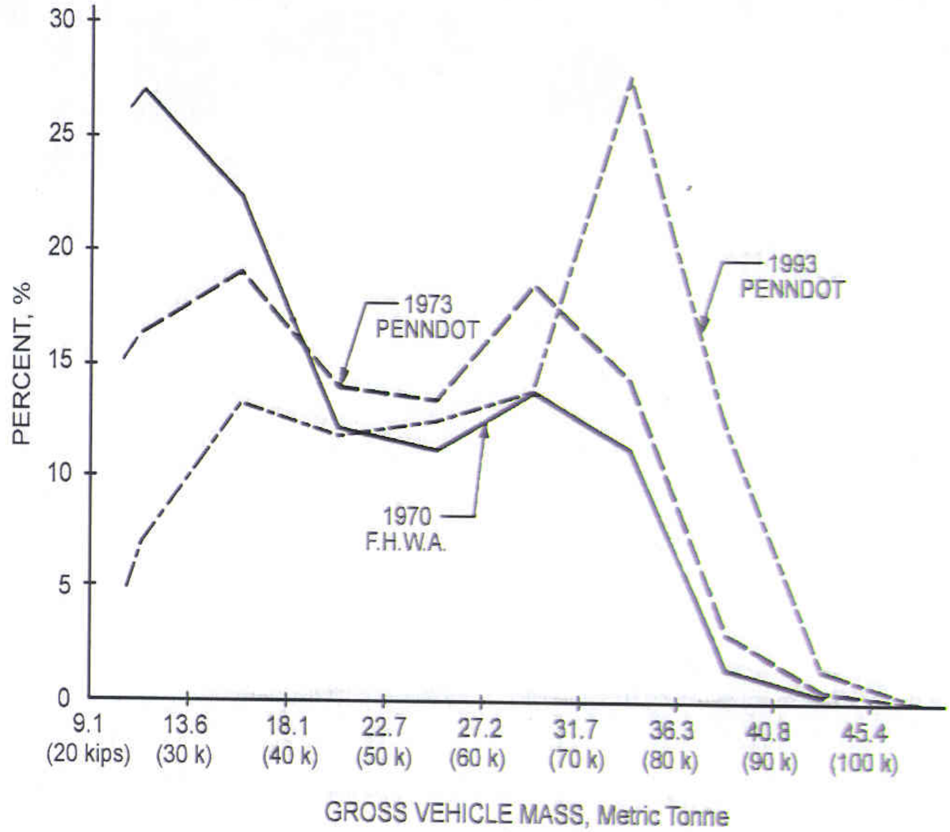


Figure 5.1.1.1.2.2-2 – Loadometer Surveys

Table 5.1.1.1.2.2-2 - GVM Distribution by Class of Vehicle (1993 Data)

Gross Vehicle Mass Range		Class of Vehicle											
		2		3		4		5		6		7 plus	
(tonne)	(kips)	Number	v_i(%)	Number	v_i(%)	Number	v_i(%)	Number	v_i(%)	Number	v_i(%)	Number	v_i(%)
9.1-13.6	20.0-29.9	9,370	74.98	240	12.27	2,213	49.47	520	3.05	6,666	31.97	34,455	4.88
13.7-18.1	30.0-39.9	2,352	18.82	1,182	60.43	1,472	32.91	497	2.92	6,176	29.62	89,760	12.70
18.2-22.7	40.0-49.9	544	4.35	500	25.56	622	13.91	442	2.59	4,544	21.79	83,393	11.80
22.8-27.2	50.0-59.9	190	1.52	30	1.53	136	3.04	1,010	5.93	2,084	9.99	92,286	13.06
27.3-31.7	60.0-69.9	21	0.17	3	0.15	22	0.49	4,020	23.58	777	3.73	101,121	14.31
31.8-36.3	70.0-79.9	18	0.14	0	0	4	0.09	9,575	56.17	529	2.54	201,183	28.47
36.4-40.8	80.0-89.9	0	0	1	0.05	2	0.05	936	5.49	68	0.33	93,431	13.22
40.9-45.4	90.0-99.9	1	0.01	0	0	1	0.02	42	0.25	6	0.03	9,911	1.40
45.5-49.9	100.0-109.9	0	0	0	0	1	0.02	3	0.02	1	0	671	0.09
50.0-54.3	110.0-119.9	0	0	0	0	0	0	0	0	0	0	284	0.04
54.4-58.9	120.0-129.9	0	0	0	0	0	0	0	0	0	0	107	0.02
59.0-63.4	130.0-139.9	0	0	0	0	0	0	0	0	0	0	54	0.01
63.5 plus	140.0 plus	0	0	0	0	0	0	0	0	0	0	59	0.01
Sum		12,496	100.0	1,956	100.0	4,473	100.0	17,045	100.0	20,851	100.0	706,715	100.0
% in each class		1.64		0.26		0.58		2.23		2.73		92.56	

Table 5.1.1.1.2.2-3 - Cumulative Damage by Truck Type (1993 Data)

GVM		φ_i (GVM_i/ GVM_o)	φ_i^3	$v_i\ \varphi_i^3$ by Class of Vehicle					
(tonne)	(kips)			2	3	4	5	6	7 plus
9.1-13.6	20.0-29.9	0.347	0.042	0.0314	0.0051	0.0207	0.0013	0.0134	0.0020
13.7-18.1	30.0-39.9	0.486	0.115	0.0216	0.0694	0.0378	0.0034	0.0340	0.0146
18.2-22.7	40.0-49.9	0.625	0.244	0.0106	0.0624	0.0340	0.0063	0.0532	0.0288
22.8-27.2	50.0-59.9	0.764	0.446	0.0068	0.0068	0.0136	0.0264	0.0445	0.0582
27.3-31.7	60.0-69.9	0.903	0.736	0.0013	0.0011	0.0036	0.1735	0.0274	0.1053
31.8-36.3	70.0-79.9	1.042	1.130	0.0016	0	0.0010	0.6349	0.0287	0.3218
36.4-40.8	80.0-89.9	1.181	1.645	0	0.0008	0.0008	0.0903	0.0054	0.2175
40.9-45.4	90.0-99.9	1.319	2.297	0.0002	0	0.0005	0.0057	0.0007	0.0322
45.5-49.9	100.0-109.9	1.458	3.101	0	0	0.0006	0.0006	0	0.0028
50.0-54.3	110.0-119.9	1.597	4.075	0	0	0	0	0	0.0016
54.4-58.9	120.0-129.9	1.736	5.233	0	0	0	0	0	0.0010
59.0-63.4	130.0-139.9	1.875	6.592	0	0	0	0	0	0.0007
63.5 plus	140.0 plus	2.014	8.168	0	0	0	0	0	0.0008
$\Sigma v_i\varphi_i^3$ by Class of Vehicle				0.0735	0.1456	0.1126	0.9424	0.2073	0.7873

Total Cumulative Damage:

$\Sigma v_i\varphi_i^3$ = Σ [(Percentage by Class of Vehicle) ($\Sigma v_i\varphi_i^3$ by Class of Vehicle)]
= 0.0735*0.0164+0.1456*0.0026+0.1126*0.0058+0.9424*0.0223+0.2073*0.0273+0.7873*0.9256
= 0.76

PTF = $(0.76)^{1/3}/0.75 = 1.22$, Use PTF 1.2

5.1.1.1.3 Design Fatigue Life (in Cycles)

The number of cycles to failure (fatigue life, based on a probability of failure of 2.3%) can be determined from

$$N_{design} = \left[A\gamma(PTF)\ (\Delta\ f)_e\right]^{-3} \text{ or } A\left[\gamma\ (\Delta\ f)_{e'}\right]^{-3} \qquad (5.1.1.1.3\text{-}1)$$

where:

Ndesign = estimated minimum number of cycles to failure

γ(PTF)(Δf)e = calculated effective stress range, based on Equation PP5.1.1.1.2.2-2 (MPa)

A = constant (values given in Table A6.6.1.2.5-1)
γ(Δf)e' = calculated effective stress range, based on Equation PP5.1.1.1.2.2-3 (MPa)
PTF = Pennsylvania Traffic Factor as given in D6.6.1.2.2

The probability of failure associated with the LRFD design fatigue life is 2.3%.

These equations are plotted in Figure AC6.6.1.2.5-1, along with the constant amplitude fatigue thresholds. The design equations were developed by observing laboratory fatigue failures of various details. Typically, fatigue failures fall within a wide scatterband. This phenomenon can be observed in Figure 1. In this figure are plotted the observed laboratory fatigue failures of welded beams without any welded attachments, used in the development of Category B. Also shown is a solid line through the data points which represents the mean, on the basis of the principle of least squares. A distance of two standard errors of estimate (in this case, the standard error of estimate approximates the standard deviation) from either side of this mean line is shown as parallel dashed lines; these lines form an envelope which statistically should contain about 95% of all Category B fatigue failures. About 2.3% of all Category B fatigue failures can be expected to occur at a number of cycles greater than that indicated by the envelope. Analogously, about 2.3% can be expected at a number of cycles less than that indicated by the envelope. The lower bound of this "95% confidence limit" envelope has been used as the Category B LRFD design curve. Using this lower bound of the wide scatterband as a design equation is reasonable and yields an adequate margin of safety against fatigue failures.

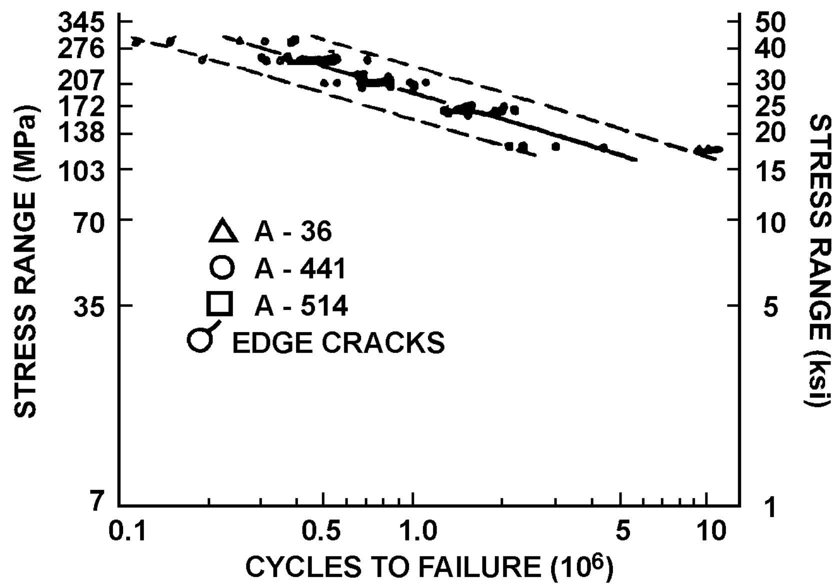


Figure 5.1.1.1.3-1 - Laboratory Fatigue Failures of Welded Beams

Using the lower bound to estimate remaining fatigue life in an evaluation context is another matter. The large majority of Category B details will exhibit fatigue lives in excess of those predicted by the design fatigue life equation using Table A6.6.1.2.5-1. For example, at an effective stress range of 138 MPa {20 ksi}, the design equation derived for Category B predicts a fatigue life of 1.5 million cycles. This calculated value of fatigue life, however, is a lower bound. At the other end of the 95% confidence limit envelope, the calculated fatigue life is just under 6 million cycles. Thus, it can be expected that 95% of all Category B details cycled at a stress range of 138 MPa {20 ksi} will fail between 1.5 and 6 million cycles. The width of the scatterband of resultant fatigue life is very large.

The probability of failure associated with this design fatigue life is about 2.3%, or 1 in 43.5. This means that 2.3% of the test data fall on each side of the so-called 95% confidence limits. In other words, only 2.3% of all details would be expected to fail prior to this calculated life. Another 2.3% would be expected to fail after the higher fatigue life indicated by the upper 95% confidence limit.

In summary, the design fatigue life equations, represented by Equation 1 with the use of Table A6.6.1.2.5-1, can be used to estimate the fatigue life of welded, bolted or riveted details. They provide lower bounds on life. For non-redundant or less redundant load path structures, where cracking can have dire consequences, it may be prudent to use these equations to predict safe life. However, for more redundant structures, these equations may prove too conservative for practical purposes.

5.1.1.1.4 Evaluation Fatigue Life

When the design fatigue life equations (with the use of Table A6.6.1.2.5-1) prove to be too conservative (which will probably be the case more often than not), Table 1 may be used to convert calculated design fatigue life into an expected fatigue life with a greater (more realistic) probability of failure, for use in evaluation. To determine the evaluation fatigue life, the design fatigue life calculated according to PP5.1.1.1.3, shall be multiplied by the multiplier in Table 1 corresponding to an acceptable probability of failure.

The probability of failure relates to the initiation of cracking at a welded, bolted, or riveted detail. This probability should not be confused with that of the collapse of the bridge. Cracking at a fatigue-sensitive detail of a non-redundant cross-section (e.g., the tie of a tied-arch) could certainly precipitate rapid collapse of the bridge, but failure of one Category E' cover plate end welded detail in a multi-girder bridge is not necessarily tantamount to the collapse of that bridge. Thus, the acceptable probability of failure should be a function of the bridge's redundancy.

The period when the bridge was constructed may also play a role in the assignment of an acceptable probability of failure. Bridges fabricated prior to the adoption of AASHTO's Fracture Control Plan (1978) may have lower fracture toughness levels than are currently deemed acceptable. Without destructive material testing to ascertain toughness levels, the acceptance of a probability of failure greater than that used for design (greater than 2.3%) is questionable. An even lower value than this may be appropriate.

If there are poor details (i.e., those explicitly excluded from current design practice), such as intersecting fillet welds, a lower acceptable probability of failure should be selected.

Final approval of the acceptable probability of failure is the responsibility of the Chief Bridge Engineer.

Table 5.1.1.1.4-1 - Multiplier to Convert Design Fatigue Life toEvaluation Fatigue Life

	Probability of Failure						
Category	0.5%	1.0%	2.3%	5%	10%	25%	50%
A	0.75	0.85	1.00	1.20	1.44	1.96	2.77
B	0.82	0.89	1.00	1.13	1.27	1.57	1.97
B'	0.78	0.87	1.00	1.16	1.36	1.76	2.37
C & C'	0.92	0.95	1.00	1.05	1.11	1.21	1.34
D	0.87	0.93	1.00	1.09	1.18	1.36	1.59
E	0.87	0.93	1.00	1.09	1.18	1.36	1.59
E'	0.76	0.86	1.00	1.18	1.39	1.84	2.51

If the Engineer wishes to assign an acceptable probability of failure other than those shown in Table 1, the following general equation can be used to assess accumulated load-induced fatigue damage for use in evaluation:

$$N = CD^{-n}\left[\gamma(PTF)(\Delta f)_e\right]^{-3} \quad \text{or} \quad CD^{-1}\left[\gamma(\Delta f)_{e'}\right]^{-3} \qquad (5.1.1.1.4\text{-}1)$$

where:

N	=	estimated number of cycles to failure
C	=	fatigue life coefficient as given in Table 2
D	=	probability of failure coefficient as given in Table 2
n	=	probability of failure exponent as given in Table 3
$\gamma(PTF)(\Delta f)_e$	=	effective stress range based on Equation PP5.1.1.1.2.2-2
$\gamma(\Delta f)_{e'}$	=	effective stress range based on Equation PP5.1.1.1.2.2-3
PTF	=	Pennsylvania Traffic Factor as given in D6.6.1.2.2

Table 5.1.1.1.4-2 - Values for General Fatigue Life Equations

Category I	Constant C	Constant D
A	2.27×10^{13}	1.66
B	7.67×10^{12}	1.40
B'	4.72×10^{12}	1.54
C & C'	1.95×10^{12}	1.16
D	1.13×10^{12}	1.26
E	5.61×10^{11}	1.26
E'	3.22×10^{11}	1.59

Table 5.1.1.1.4-3 - Probability of Failure

Probability of Failure (%)	Exponent
98.0%	-2.0
75.0%	-0.675
50.0%	0.0
25.0%	0.675
10.0%	1.285
5.0%	1.645
2.3%	2.0
1.0%	2.327
.5%	2.576
.25%	2.810
.1%	3.090

All of the above estimates represent total life. The residual life of an existing detail is equal to this total life minus its life to-date.

5.1.1.1.5 Example Fatigue Damage Assessment

The following threefold example of fatigue damage assessment is taken in part from a Department study of the Clarion River Bridge along I-80. At the time of the study, the bridge was in service for about 15 years.

The example is based on using a fatigue stress range, $(\Delta f)_D$ at a Category E' cover plate detail on a longitudinal member of 13.3 MPa {1.93 ksi}. The average past ADTT to the time of the assessment was determined from Department data to be 3,900 trucks per day, and based on Department projections, a reasonable average future ADTT for the bridge is 8,650 trucks per day. The current ADTT was 5,000 trucks per day.

5.1.1.1.5.1 Simplified Damage Assessment

The simplified fatigue damage assessment is based upon the use of Equation PP5.1.1.1.2.2-2. The effective stress range can be estimated from the calculated fatigue stress range using Equation PP5.1.1.1.2.2-2:

$$\gamma(PTF)(\Delta f\ )_e = \gamma(PTF)\ \alpha\ (\Delta f\ )_D$$
$$= 0.75(1.2)(1.0)(13.3\ \text{MPa})$$
$$= 12.0\ \text{MPa}\ \{1.74\ \text{ksi}\}$$

The fatigue life shall be estimated using Equation PP5.1.1.1.3-1

$$N_{design} = A[\gamma(PTF)(\Delta f\ )_e]^{-3}$$
$$= (1.28 \times 10^{11})[12]^{-3}$$
$$= 74.1 \times 10^6 \text{ cycles}$$

Up to the present, the structural detail has experienced 21.4 x 10^6 cycles (assuming one cycle per truck, 3,900 trucks per day for 15 years). Thus, significant residual life remains about 53 million cycles, or, at the future ADTT of 8,650 trucks per day, about 17 years.

If a less conservative probability of failure than that in to the LRFD design provisions is justified, the multiplier in Table PP5.1.1.1.4-1 can be used. If a probability of failure as high as 50% could be justified due to multiple redundancy, a multiplier of 2.51 could be applied to the design fatigue life:

$$N_{evaluation} = 2.51\ N_{design}$$
$$= 2.51\ (74.1 \times 10^6 \text{ cycles})$$
$$= 186 \times 10^6 \text{ cycles}$$

Now the residual life is about 165 million cycles or, at the projected future ADTT of 8,650 trucks per year, 52 years.

5.1.1.1.5.2 Assessment Using Traffic Study

Traffic studies can be used to add a degree of refinement to the fatigue damage assessment. While the most readily implementable data is GVM distribution, the acquisition of such data would require mass stations specific to the bridge site. Site-specific distributions of truck traffic by truck type are more readily obtainable than distributions of traffic by gross vehicle mass. Generation of distributions by type requires mere observation and counting of trucks; determining distribution by mass requires determining the actual mass of the trucks.

If the assumption is made that the distribution of gross vehicle masses within each truck type (as indicated by statewide weighing) is constant, the cumulative damages by truck type can be proportioned on the basis of the observed distribution by truck type to obtain a site-specific GVM distribution.

On the basis of the sampling of 5,923 significant vehicles (i.e., those over 9 tonne in mass) {20 kip in weight}, shown in Table 1, the 1973 statewide break-down of traffic by truck type can be assumed to be as follows:

Truck Type		Traffic (%)
Single Unit Trucks	- 2 axles	3.8
	- 3 axles	3.1
Tractor Semi-Trailer Combinations	- 3 axles	2.6
	- 4 axles	16.8
	- 5 axles or more	73.6

Using this same sampling, the cumulative damage ($\Sigma v_i \varphi_i^3$) by truck type can be calculated to be as follows:

Truck Type		$\Sigma v_i \varphi_i^3$
Single Unit Trucks	- 2 axles	0.0530
	- 3 axles	0.19241
Tractor Semi-Trailer Combinations	- 3 axles	0.08412
	- 4 axles	0.21074
	- 5 axles or more	0.60261

Combining these summations of cumulative damage with the percentages of truck types above yields the following equation:

$$\Sigma v_i \varphi_i^3 = 0.038(0.05304) + 0.031(0.19241) + 0.026(0.08412) + 0.168(0.21074) + 0.736(0.60261) = 0.489$$

The foregoing calculation led to the value of $\Sigma v_i \varphi_i^3$ of 0.50, which was used before the 1993 data was available.

The breakdown of cumulative damage by truck type, as shown above, allows for improvements in estimated damage where different distributions of traffic by truck type are encountered. For the preceding example, a site-specific distribution by truck type was also available, as shown in Table 1. More reliable estimates for fatigue damage are obtained by utilizing the site-specific data. Combining the statewide summations of cumulative damage by site-specific percentages of truck types yields the following estimate of total damage:

$$\Sigma v_i \varphi_i^3 = \Sigma\ [(\text{Percentage of truck type from Table 1})\ (\Sigma v_i \varphi_i^3 \text{ by truck type from Table PP5.1.1.1.2.2-1})]$$

$$\Sigma v_i \varphi_i^3 = 0.065(0.05304) + 0.017(0.19241) + 0.014(0.08412) + 0.083(0.21074) + 0.822(0.60261) = 0.521$$

This example of the Clarion River Bridge along I-80 (based on 1982 data) suggests that the cumulative damage along I-80 may be less than that suggested by the 1993 GVM survey.

This more accurate value can now be used in Equation PP5.1.1.1.2.2-3 in conjunction with a value of α consistent with the LRFD design provisions.

$$\begin{aligned} \gamma\,(\Delta f\,)_{e'} &= [\Sigma\ v_i\ \varphi_i^3]^{1/3}\ \alpha\ (\Delta f\,)_D \\ &= [0.521]^{1/3}\ (1.0)\ 13.3\ \text{MPa}\ \{1.93\ \text{ksi}\} \\ &= 10.7\ \text{MPa}\ \{1.55\ \text{ksi}\} \end{aligned}$$

The 10.7 MPa {1.55 ksi} compares with 12.0 MPa {1.74 ksi} using the simplified approach (see PP5.1.1.1.5.1).

In this case, the results of the assessment using a traffic study will not be significantly different from the simplified approach, but more faith can be placed in the assessment because of the use of the site-specific traffic study.

Table 5.1.1.1.5.2-1 - Clarion River Bridge Truck Type Distribution

Year	ADTT	Single-Unit Trucks				Tractor Semi-Trailer Combinations					
		2 Axle		3 Axle		3 Axle		4 Axle		5 Axle or More	
		%	#	%	#	%	#	%	#	%	#
68	350	14	49	4	14	5	18	42	147	35	123
69	550	14	77	4	22	5	28	38	209	39	215
70	1750	10	175	3	53	3	53	27	473	57	998
71	3800	6	228	2	76	2	76	13	494	77	2926
72	3900	7	273	3	117	2	78	11	429	78	3042
73	3950	6	237	1	40	1	40	10	395	82	3239
74	4450	6	267	1	45	1	45	10	445	82	3649
75	4550	6	273	1	46	2	91	8	364	83	3777
76	4800	6	288	1	48	2	96	7	336	84	4032
77	5000	5	250	1	50	1	50	7	350	86	4300
78	5200	5	260	1	52	1	52	6	312	87	4524
79	5200	5	260	2	104	1	52	5	260	87	4524
80	5100	5	255	2	102	1	51	5	255	87	4437
81	5150	8	412	2	103	1	52	4	206	85	4378
82	4950	10	495	3	149	1	50	4	198	82	4059
	58 700		3799		1021		832		4873		48 223
Weighed Average %			6.5%		1.7%		1.4%		8.3%		82.2%

5.1.1.1.5.3 Assessment Using Measured Stresses

The stress range spectrum can be estimated by measuring strains at critical structural details over an extended period of time to capture a representative sample of truck traffic. In this case, the observed effective stress range, $\gamma(\Delta f\ )_{e'}$, was just under 6.9 MPa {1.0 ksi}, while the observed maximum stress range during the period of observation was just under 20.7 MPa {3.0 ksi}.

Since the maximum stress range of 20.7 MPa {3.0 ksi} is greater than infinite life threshold value of 17.9 MPa {2.6 ksi} for Category E' detail from Table A6.6.1.2.5-3, the estimated design fatigue life using Equation PP5.1.1.1.3-1 results in:

$$N_{design} = 1.28 \times 10^{11} (6.9)^{-3}$$
$$= 390 \times 10^{6} \text{ cycles}$$

Even at the low probability of failure associated with the LRFD design provisions (2.3%), the calculated residual life of about 370 million cycles (or 117 years at 8,650 trucks per year) is in excess of any specified life for the bridge. Varying the probability of failure to determine an evaluation fatigue life is not necessary in this case.

5.1.1.1.5.4 Effect of Permit Vehicles

To illustrate the potential effect of permit vehicles on accumulated fatigue damage, the following example is presented. Gross vehicle mass recorded for all the overmass permits issued by the Department for one calendar year were reduced to a value of the summation of $v_i\varphi_i^3$ equal to 3.21 (see Table 1) (as compared with 0.76 based on the 1993 GVM surveys, as stated earlier). In the 1993 GVM distribution, approximately 14% of the vehicles exceeded 36.4 tonne {80.2 kips}. This figure represents the vehicles observed during a particular period, potentially including any permit vehicles. If it is assumed that 4% of all trucks in the through lane are permit vehicles, and they are added to the observed distribution which already includes 14% of the vehicles over 36.4 tonne {80.2 kips}, then the combined summation of $v_i\varphi_i^3$ is equal to

$$(0.96)(0.76) + (0.04)(3.21) = 0.86$$

Using this value in the example of a simplified assessment in PP5.1.1.1.5.1 yields

$$\gamma\,(\Delta f\,)_{e'} = [\Sigma\, v_i\, \varphi_i^{\,3}]^{1/3}\, \alpha\, (\Delta f)_D$$
$$= [0.86]^{1/3}\, (1.0)\; 13.3 \text{ MPa } \{1.93 \text{ ksi}\}$$
$$= 12.6 \text{ MPa } \{1.83 \text{ ksi}\}$$

as opposed to 12.0 MPa {1.74 ksi} without permit vehicles included. Now,

$$N_{design} = 1.28 \times 10^{11}\, (12.6)^{-3}$$
$$= 63.3 \times 10^{6} \text{ cycles}$$

as opposed to about 74 million cycles when permit vehicles are excluded. In other words, when permit vehicles are factored into the analysis as 4% of the truck traffic distribution, the number of anticipated cycles to cracking is reduced to about 85% of that when permit vehicles are neglected.

Table 5.1.1.1.5.4-1 - Permit Load Cumulative Damage

Mass/Weight		φ_i	#	v_i	$v_i\varphi_i^3$
(tonne)	(kips)				
33.2 - 43.1	73.281 - 95.0	1.169	24,547	0.4744	0.7579
43.2 - 55.8	95.1 - 123.0	1.514	18,989	0.3670	1.2736
55.9 - 68.0	123.1 - 150.0	1.896	7,557	0.1460	0.9951
68.1 - 80.3	150.1 - 177.0	2.271	435	0.0035	0.0648
80.4 - 92.5	177.1 - 204.0	2.646	179	0.0035	0.0648
over 92.5	over 204.0	2.833	39	0.0008	0.0182
			51,746	1.0000	3.2080

5.1.2 Displacement-Induced Fatigue

Displacement-induced fatigue is usually associated with relatively small out-of-plane displacements. In many cases, local relative movements measured in only hundredths of a millimeter have been sufficient to cause displacement-induced fatigue damage. Such displacement-induced fatigue cracking is epidemic on steel bridges throughout America.

Two conditions are necessary for displacement-induced fatigue damage to occur:

(a) A periodic out-of-plane force or displacement.

(b) An abrupt local change in stiffness where the force/displacement is applied.

Without the combination of stiffness changes and driving force, displacement-induced fatigue cracking will not develop. The presence of one of the above-mentioned conditions is not in itself sufficient to cause displacement-induced cracking. For example, the end of a transverse stiffener (not a transverse connection plate) of a plate girder cut short of the tension flange (for load-induced fatigue considerations) constitutes an abrupt change in stiffness in the out-of-plane direction of the plate girder web. After erection and placement into service, no out-of-plane force exists to oppose the change in out-of-plane stiffness. The potential for in-service displacement-induced cracking at the cut-short stiffener end does not exist, since both conditions (stiffness change and driving force) are not present. In some documented cases, cracks have been observed at cut-short ends of transverse stiffeners before the girder was put into service. It has been shown that such cracking was the result of inadequate blocking for shipment by rail. The periodic rocking of the railway car induced an inertial force of the tension flange in opposition to the stiffness change. In this case, both conditions were present and the potential for cracking was realized.

5.1.2.1 DISPLACEMENT-INDUCED FATIGUE DAMAGE ASSESSMENT

Cumulative fatigue damage of members and fasteners subject to repeated variations or reversals of stress due to out-of-plane deformations or secondary forces shall not be assessed.

The time-honored simplifying assumptions made to analyze bridges does not provide the tools necessary to quantify these out-of-plane displacements or the stresses that result from them. In the past, much of the effort in bridge research dealt with developing simplified tools by which a complex three-dimensional structure could be analyzed as a simple one-dimensional element. For example, in the design process for a simple multi-girder bridge, this highly interconnected assemblage of deck, girders, cross-frames and laterals is idealized as a single line element and analyzed as an isolated beam. Any ability to calculate forces or displacements normal to the plane of the idealized line element is lost.

Refined methods of analysis enable designers to take some of the three-dimensionality of the bridge into account. For example, in the multi-girder type of bridge discussed above, it is now practical to use grillage or finite element methods to obtain very realistic load distribution among the several girders and to quantify forces in laterals and cross-frames. As large an improvement as this is, it still does not necessarily lead to practical design-office evaluation of displacement-induced stresses because an entirely different order of magnitude in element sizes (hence in numbers of nodes and elements) is required to calculate these very localized stresses.

5.1.3 Fracture-Critical Cross Girder Pier Caps Cracking at Internal Diaphragms

Box girder pier caps are classified as fracture-critical. The connection detail of the internal diaphragm of a box girder pier cap which provides continuity through nested steel beams is highly susceptible to fatigue cracking unless the diaphragms are connected to both flanges and webs of the box girder.

An NBIS inspection on a steel box cross girder pier cap with a multi-girder superstructure framing into it revealed that the box girder's interior diaphragm was welded on three sides, but did not connect to the bottom tension flange as shown in Figure 1. A "U-shaped" crack in the web at the bottom of one diaphragm was detected visually and then was verified with dye-penetrant. Unchecked, the crack could have extended into the flanges and caused the eleven year old girder to fail.

The detail of providing only a partial depth internal diaphragm, as shown in Figure 1, was formerly believed to avoid problems associated with welding tension flanges. However, this gap allows out-of-plane bending and creates possibility of web cracking at the end of the partial depth diaphragm near both tension and compression flanges. This cracking may extend completely through the web or only partially if lamellar tearing also occurs. Because these cross-girders are fracture-critical, appropriate measures must be taken to avoid a sudden failure and to retrofit the girder for continued use. Do not use this type of detail on future projects.

Retrofit this type of detail as follows:

1. Connect the diaphragm to the flange with bolted steel angles (see Figure 2).

2. Provide friction connection with high strength bolts.

3. Bolts placed at maximum pitch will suffice.

4. Because the new bolt holes will reduce the cross-sectional area of the tension flanges, the capacity of the cross girder must be checked. Maintain a minimum strength of PHL-93 inventory and P-82 permit loading with LRFD method. Stagger bolts on tension flanges as necessary to maintain girder strength.

If the above retrofit cannot be performed in a timely manner, and if cracking is observed, an interim retrofit as shown in Figure 3 would suffice. This was used in District 6-0, and has performed well for over 15 years.

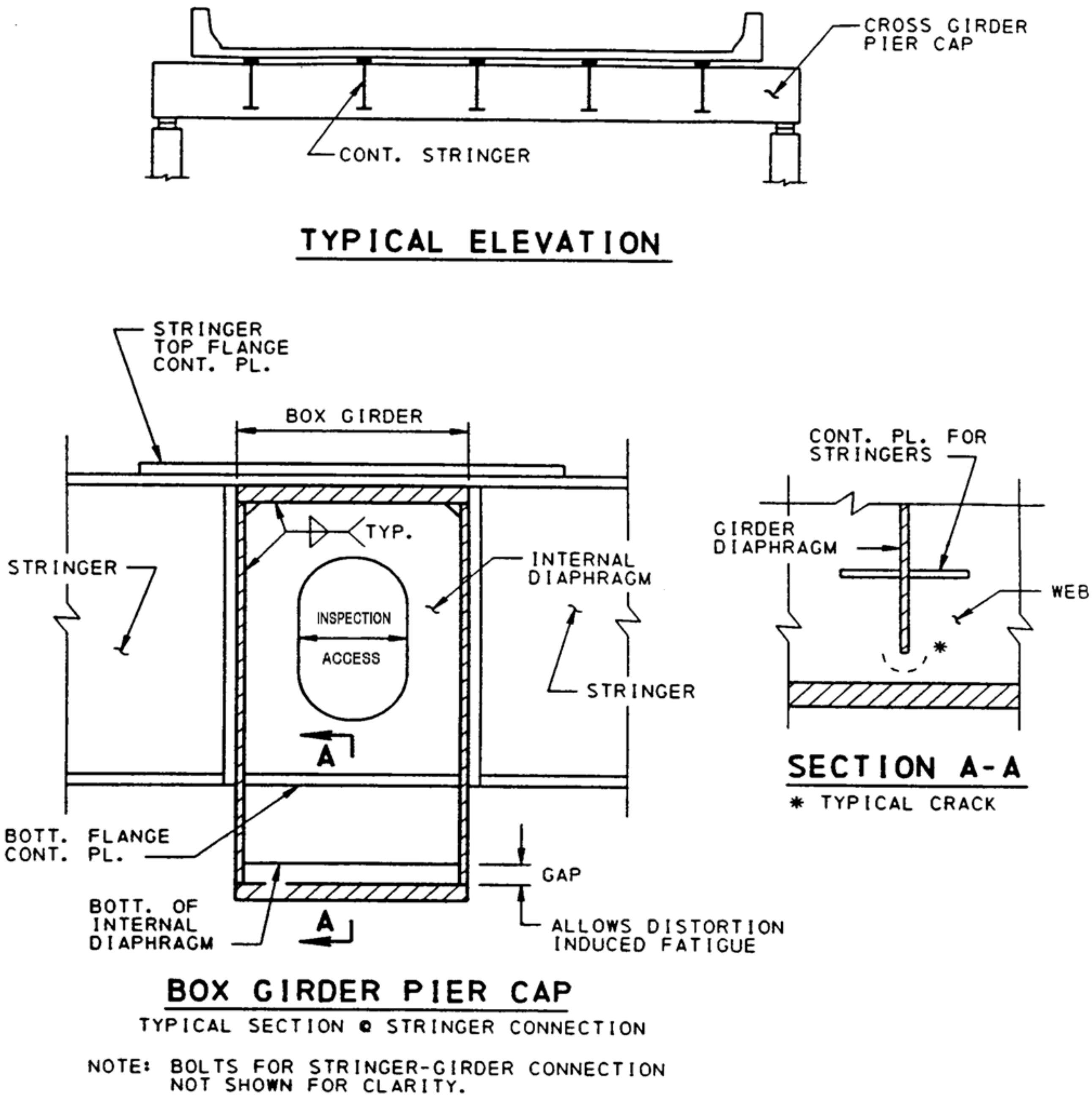


Figure 5.1.3-1 - Fracture-Critical Girder Pier Caps

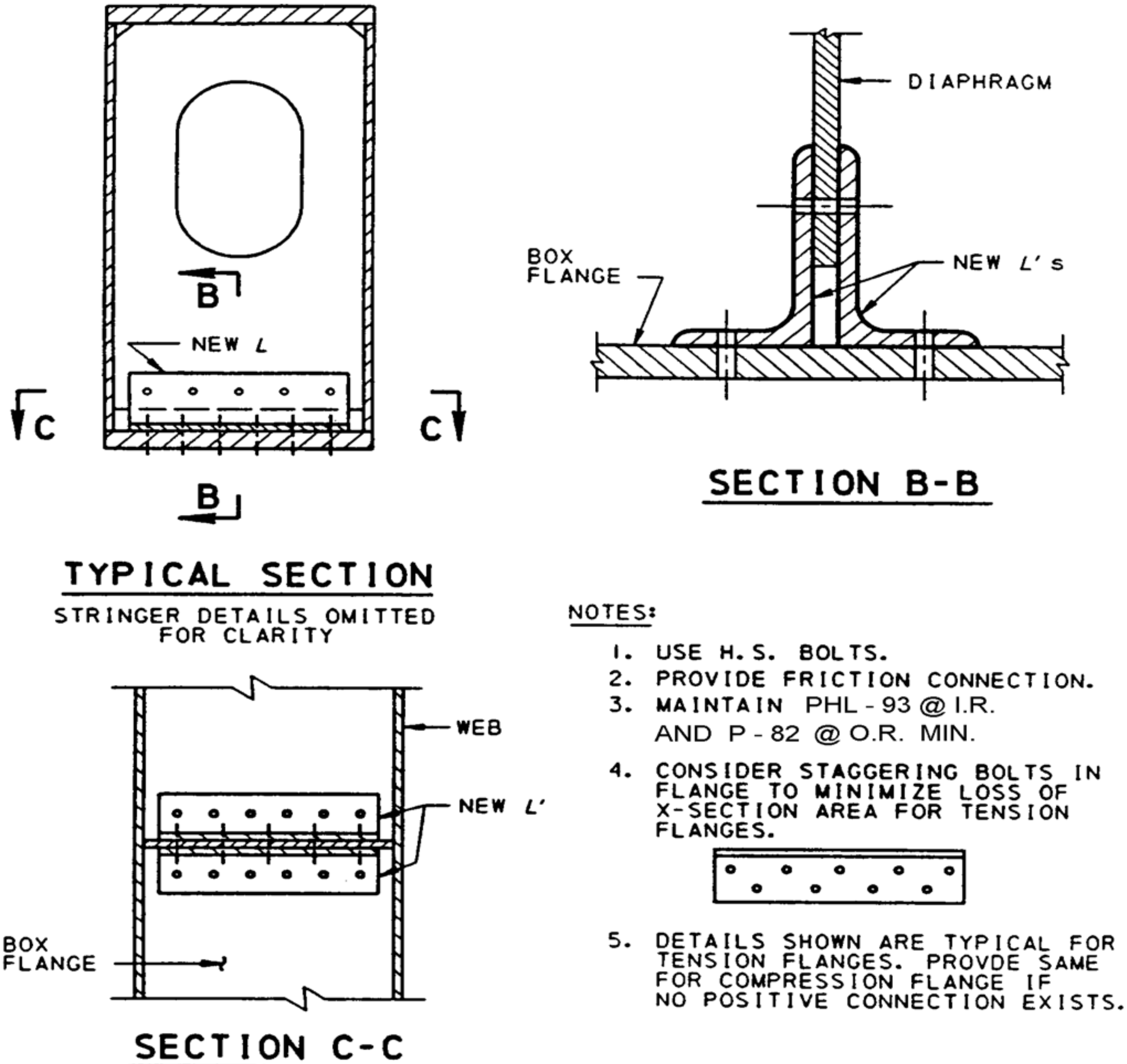


Figure 5.1.3-2 - Preferred Retrofit Details

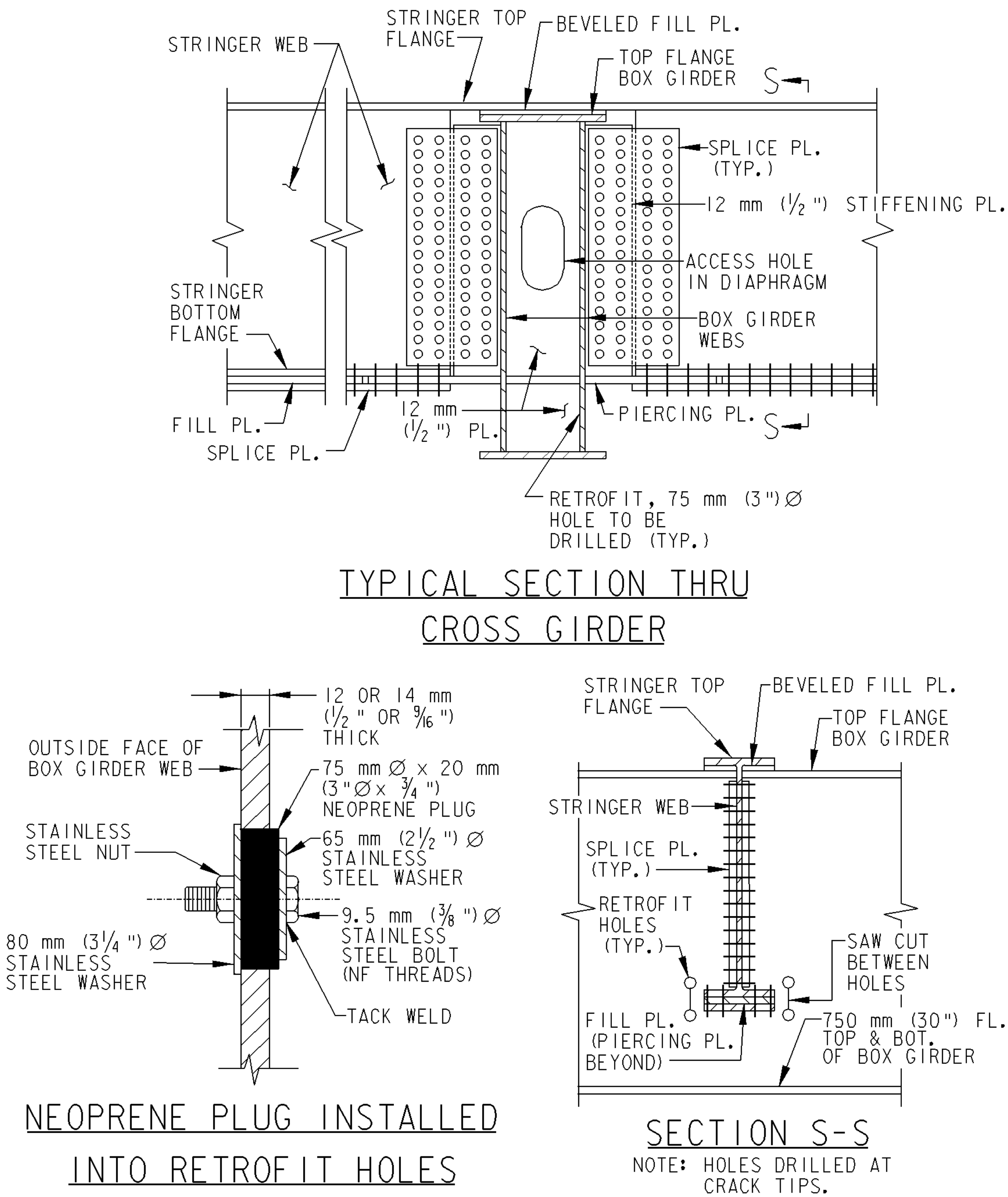


Figure 5.1.3-3 - Alternate Retrofit Details

5.2 COVER PLATE CUTOFFS

The articles in this section are intended for rehabilitation projects in which the use of partial-length cover plates may be necessary.

5.2.1 General

Any cover plate that is not required to be full-length shall extend beyond the theoretical cutoff point far enough to develop the capacity of the plate or shall extend beyond a section where the stress in the remainder of the girder flange is equal to the nominal fatigue resistance, whichever is greater. The theoretical cutoff point of the cover plate is the section at which the stress in the flange without the cover plate equals the flange resistance, exclusive of fatigue considerations.

5.2.1.1 WELDED COVER PLATES

Any partial-length welded cover plate shall extend beyond the theoretical end by the terminal distance, or it shall extend beyond a section where the stress range in the beam flange without the cover plate is equal to the nominal fatigue resistance for base metal adjacent to or connected to by fillet welds. The theoretical end of the cover plate is the section at which the stress in the flange without the cover plate equals the flange resistance, exclusive of fatigue considerations. The terminal distance is two times the nominal cover plate width for cover plates not welded across their ends and one and one-half times for cover plates welded across their ends. The width at ends of tapered cover plates shall not be less than 75 mm {3 in.}. The weld connecting the cover plate shall be of sufficient size to develop a total stress of not less than the computed stress in the cover plate at its theoretical end.

5.2.1.2 RETROFITTING OF EXISTING WELDED COVER PLATES

Rolled beam steel girders with welded partial length cover plates have experienced fatigue cracking at the end of cover plates. Research shows that end bolting of the cover plates appreciably increases the fatigue life at the end of cover plates. This is very significant in case of retrofitting partial length cover plated beams. New cover plated rolled beams shall be designed in accordance with A6.10.9 and D6.10.9.

Whenever retrofitting is required on existing bridges, the cover plates must be end bolted as detailed in Figure 1. The number and size of high strength bolts may vary. The design procedure is as follows:

1. For non-cracked flanges at cover plate ends:

   Design Moment M

   a. Calculate area of bolt holes, Δ_A, in excess of 15% of gross flange area.

   b. Deduct Δ_A from gross flange area.

   c. Calculate net moment of inertia of beam with bolt holes, I_{net}.

   $$I_{net} = I - \Delta_A \left(\frac{d}{2} - \frac{t_f}{2} \right)^2$$

   d. Calculate, $S_{net} = \dfrac{I_{net}}{\frac{d}{2}}$

   e. Calculate design moment, $M = S_{net} F_y$

Moment Carried by Web

Stress at extreme fiber of web, $f = \dfrac{F_y\left(\dfrac{d}{2} - t_f\right)}{\dfrac{d}{2}}$

Section modulus of web, $S_w = \dfrac{1}{6} t_w \left(d - 2t_f\right)^2$

Moment carried by web, $M_w = S_w f$

Force carried by flange, $T = \dfrac{M_f}{d}$

Required Number of Bolts

The force carried by a bolt shall not exceed the nominal slip resistance (R_n) specified in A6.13.2.8 and D6.13.2.8:

Number of bolts required with two slip planes = $\dfrac{T}{R_n}$ (Round to the next even number)

Required Splice Plate Area

$$A_{reqd} = \frac{T}{F_y}$$

Select the plate size so that $^{*}A_{reqd}$ = gross area minus area of bolt holes in excess of 15% of gross area.
$^{*}A_{reqd}$ should not be less than the flange area.

2. For cracked flanges at the cover plate ends, design the splice plates for the design moment, M.

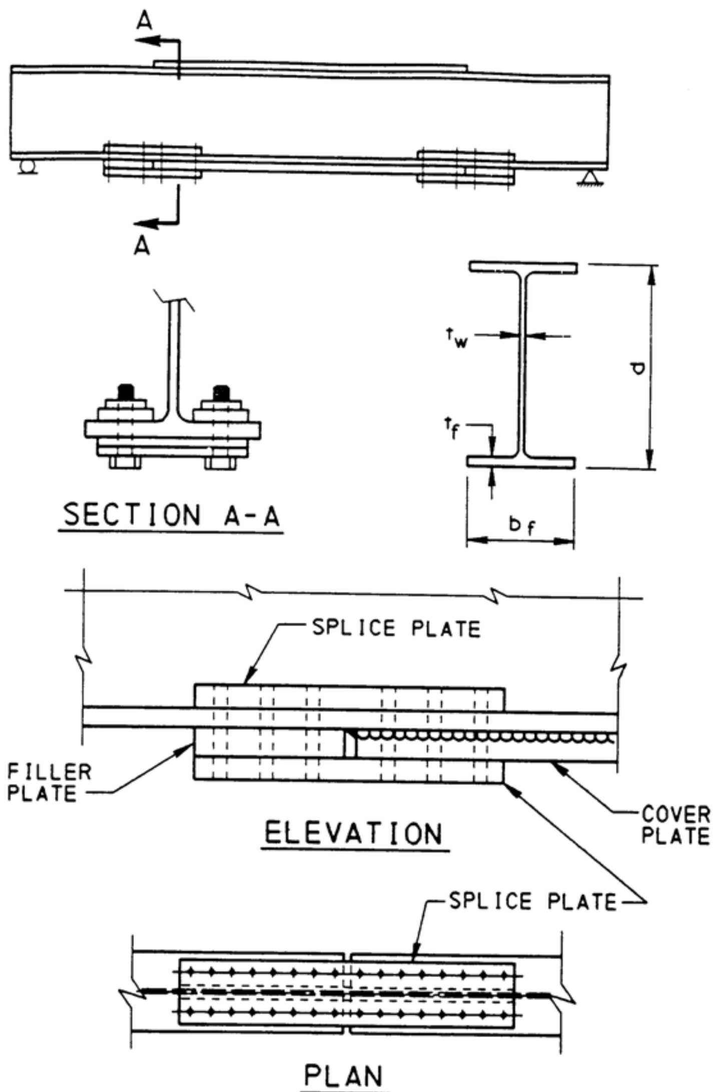


Figure 5.2.1.2-1 - Partial Length Cover Plates

The following is a metric example of above-outlined bolt design procedure.

Assumed Beam size	=	W920 x 238, flange cracked and A 36/A 36M steel
Area of flange A_f	=	$b_f\ t_f$ = 305 x 25.9 = 7900 mm^2 {12.2 in^2}
Area of two bolt holes in the flange	=	$2t_f\ d_h$ = 2 x 25.9 x 28.6 = 1481 mm^2 {2.3 in^2}
Assuming 25.4 mm dia. bolts, d_h	=	bolt diameter, 3.2 mm larger than bolt size.

Area of bolt holes in excess of 15% of flange area, Δ_A = $2t_f\ d_h - 0.15A_f$
= 1481 - (0.15 x 7900) = 296 mm^2 {0.5 in^2}

Therefore, 296 mm^2 is to be deducted from the gross area for calculation of stresses.

$$I_{net} = I - \Delta_A = \left(\frac{d}{2} - \frac{t_f}{2}\right)^2 = 4060\left(10^6\right) - 296\left(\frac{915}{2} - \frac{25.9}{2}\right)^2 = 4002\left(10^6\right)mm^4 \left\{9{,}615 in^4\right\}$$

$$S_{net} = \frac{I_{net}}{\left(\frac{d}{2}\right)} = \left(\frac{4002\left(10^6\right)}{\frac{915}{2}}\right) = 8748\left(10^3\right)\ mm^3 \left\{534\ \ in^3\right\}$$

Design stress F_y = 250 MPa {36 ksi}

Therefore, Design moment = S_{net} F_y = 8748(10^3) x 250 = 2187(10^6) N•mm {19.4 (10^3) k•in}

<u>Calculate moment carried by web</u>:

Stress at extreme fiber of web, $f = F_y \frac{\left(\frac{d}{2} - t_f\right)}{\left(\frac{d}{2}\right)} = (250)\frac{\left(\frac{915}{2}\right) - 25.9}{\left(\frac{915}{2}\right)}$ = 235.8MPa{34.2ksi}

Section modulus of web $S_w = \frac{1}{6} t_w (d - 2t_f)^2$

S_w = (1/6) x 16.50 (915 - 2 x 25.9)^2 = 2049(10^3) mm^3 {125 in^3}

Moment carried by web = S_w f = 2049(10^3) x 235.8 = 483(10^6) N•mm {4.3(10^3) k•in}

Moment carried by flange, M_f = 2187(10^6) – 483(10^6) = 1704(10^6) N•mm {15.1(10^3) k•in}

Force carried by flange, $T = \frac{M_f}{d} = \frac{1704(10^6)}{915} = 1862kN\{419kips\}$

<u>Required number of bolts</u>:

R_n = K_h K_s N_s P_t

K_h = 1.0 (for standard holes from Table A6.13.2.8-2)

K_s = 0.33 (for Class A surface conditions from Table A6.13.2.8-3)

N_s = 2 (number of slip planes)

P_t = 227 kN (for 25.4 mm bolt from Table D6.13.2.8-1)

Therefore, R_n = 1.0 x (0.33) x 2 x 227 = 149.8 kN {33.7 kips}

N_b = number of bolts in the joint

or $N_b = \frac{T}{R_n} = \frac{1862}{149.8} = 12.4$

14-25.4 mm dia. A 325 HS bolts {14-1 in. dia. A325 HS bolts}

<u>Required Splice Plate Area</u>

$$A_{reqd} = \frac{T}{F_y} = \frac{1862(10^3)}{250} = 7448\text{mm}^2\{11.5\text{in}^2\}$$

Assuming outer plate 250 x 20 and two inside plates 100 x 20

$A_{provided}$ = (250 x 20) + 2(100 x 20) = 9000 mm^2 {14.0 in^2}

15% of $A_{provided}$ = 9000 x 0.15 = 1350 mm^2 {2.1 in^2}

Area of four holes for 25.4 mm dia. bolts = 4t d_h = 4 x 29 x 28.6 = 2288 mm^2 {3.5 in^2}

Area to be deducted from gross area = 2288-1350 = 938 mm^2 {1.4 in^2}

A_{net} = 9000 - 938 = 8062 mm^2 > 7448 mm^2 required {12.6 in^2 > 11.5 in^2}

The retrofit shall be designed for fatigue using the constant amplitude fatigue thresholds of LRFD Category B, regardless of whether the flange is non-cracked or cracked.

The tip of any crack that has entered the web shall be arrested by drilling a hole, cleaning the area, installing a high-strength bolt of the same diameter as is used for the splice, and torquing it to the prescribed initial tension. Extra care shall be taken that the hole does not miss the crack tip.

If the web crack is longer than one sixth of the beam depth, the web shall be considered fully cracked. Therefore, the splice shall be designed as a full flange and web splice using A6.13 and D6.13.

5.3 FASTENERS

For rehabilitation of riveted members carrying calculated stress, 15.9 mm {5/8 in.} diameter fasteners shall be used only if 19.1 mm {3/4 in.} diameter or larger fasteners will not fit.

5.4 WELD REPAIRS OF TENSION FLANGES OF A-7 STEEL BRIDGES

Sometimes numerous nicks and cuts are inflicted on the top flanges of beams of simple and continuous girder spans when the contractors saw-cut the bridge deck transversely to facilitate its removal. Saw-cut may also go through the rivet heads and edges of flanges. Some damages can be inflicted by blows from a jackhammer. Consequently, it becomes necessary to establish the weldability of the steel by performing a chemical analysis and determining the carbon equivalent of the material, in order to approve the method of weld repairs, especially for the large saw cuts in the tension area of the top flanges of the girders, which are considered as critical members that are subject to fatigue consideration due to the creation of various geometrical notches. Welding on the tension flange should be avoided whenever possible. All weld repairs shall be treated with Ultrasonic Impact Treatment (UIT) process. Weld repairs should be considered only for major cuts otherwise nicks and cuts should be ground to bright metal in the direction of stress for both tension flanges, compression flanges or webs. The excavation shall be smooth and free of irregularities. Material shall be faired with a slope not to exceed one in ten. It may be necessary to verify the member has adequate cross sectional area due to the removal of the nicks and gouges.

Use the following criteria when welding in the field becomes necessary to repair damages inflicted to critical members of A-7 steel bridges in field operations:

1. Perform the chemical analysis of a plate to determine the feasibility of welding.

   The current specifications for A 36/A 36M, A 242/A 242M, Type 2, A 572/A 572M and A500 steels limit the sulfur and phosphorus to a maximum of 0.05% and 0.04%, respectively. If the chemical analysis of a plate reveals higher sulfur and/or phosphorus, do not perform welding. Phosphorus and sulfur are undesirable impurities which embrittle the steel and weld metal.

   With regard to cold cracking, based on the expression

   $C + M_n/6 = CE$

The carbon equivalent (CE) must not exceed 0.40%. For steel with CE equal to or less than 0.40, use ANSI/AASHTO/AWS D1.5-2002 Chapter 12 including preheat Section 12.17.6(8)(a) and post heat Section 12.15. If the CE of any plate to be weld repaired exceeds 0.40, then particular attention must be given to low-hydrogen practice and the time between welding and inspection (per Section 12.16.4) should be increased to 48 hours for all plates. The welds should be ground flush before inspection. After grinding and prior to RT/UT, the welds should be dye-penetrant (DT) inspected. In UT, the scanning should include pattern D, as well as pattern E (ANSI/AASHTO/AWS D1.5 – 2002), with UT from both top and bottom surfaces of the flanges.

2. Place proper documentation of the damages, such as: location, type, size, stress range at damaged location, method of weld repair, method of N.D.T., etc., in the District's Structure Inventory and Appraisal (inspection) file.

3. Treat weld repair areas with Ultrasonic Impact Treatment process.

5.5 BRIDGE REHABILITATION STRATEGIES

The items included herein are not all inclusive. Depending upon the type of rehabilitation needs, some or all of the items may be applicable.

In evaluating the rehabilitation or replacement of any component of a bridge, every component, as well as structural capacity, deck geometry, scour, seismic adequacy, and other deficiencies need to be assessed.

5.5.1 Needs Establishment

Rehabilitation of an existing bridge may be warranted by its condition, special situation, or by inclusion on a program, such as the Interstate Preventive Maintenance (IPM), highway capacity improvement, safety improvement, or other structural improvement program.

A Pennsylvania condition rating (refer to Publication 100A, Bridge Management System 2 (BMS2), Coding Manual) of six or less for the entire bridge or its components would indicate the need for rehabilitation.

If the sufficiency rating (SR) of a bridge is less than 50.0 and the bridge is structurally deficient or functionally obsolete, it is eligible for Highway Bridge Program (HBP) funds for rehabilitation or replacement. If the SR is greater than or equal to 50.0, but less than or equal to 80, and the bridge is structurally deficient or functionally obsolete, it is eligible for HBP funds for rehabilitation only. The bridge must not have been replaced or rehabilitated within the last 10 years utilizing any federal funding source.

The rehabilitation or replacement of structurally deficient bridge decks (those with Publication 100A, BMS2, Item 1A01 – Deck ≤ 4) is eligible for HBP funding. Such bridge work is to be considered eligible regardless of the bridge's sufficiency rating. The 10-year rule will not prevent a current structurally deficient deck from being replaced or rehabilitated; however, once the deck has been replaced or rehabilitated, the 10-year rule will apply.

Review the bridge inspection file and address all the deficiencies and problems identified in the file.

Pre-plan preparation and post-plan preparation bridge inspections are warranted for major (deck replacement and significant modifications to main load carrying members) rehabilitation projects. The bridge designer or design consultant must be present for these inspections. Depending on available data and complexity of the work, either of the two inspections may be waived by the District Bridge Engineer.

5.5.2 List of Items to be Included

For IPM projects, specific bridge items will be evaluated and corrected. Items to be evaluated include: crack sealing, joint repair, seismic retrofit, scour countermeasures and painting structural steel. IPM projects should correct bridge deficiencies contributing to accident clusters. IPM will include the upgrade of guide rail connections to the bridges, guide rail protection at bridge piers and bridge safety walks to prevent vaulting.

When the repair needs are extensive, the bridge portions of an IPM project may be upgraded using normal rehabilitation criteria. The following list of items includes a description of items included in rehabilitation strategies.

5.5.2.1 GEOMETRY

Bridge width: Refer to DM-2, Chapter 1. This includes criteria for Bridges on Very Low Volume Roads.

Vertical clearance: Refer to DM-2, Section 2.21

Barrier: Based on the Test Level (TL) required (See DM-2, Chapter 12.9) follow Standard Drawings BD-601M, BD-614M, BD-615M, BD-617M or BD-618M for barrier rehabilitation.

Guide rail transition: Follow Standard Drawing BC-739M, BC-703M, BC-706M or BC-708M for Interstate Preventative Maintenance Projects with the detail on Sheet 4, BC-799M for CIP barrier details located off of the structure.

5.5.2.2 DRAINAGE

(a) Superstructure

Refer to PP3.2.3.6. Ensure that the existing scuppers and downspouting are repaired, cleaned or replaced and splash blocks are provided if none exist.

(b) Substructure

For MSE wall abutments, provide drainage according to Standard Drawing BC-799M. For other abutment types, provide drainage as necessary.

(c) Off Structure Drainage

Provide an appropriate roadway inlet to eliminate shoulder washouts in accordance with Standard Drawing RC-50M.

5.5.2.3 DECKS

The deck replacement decision shall be made based on whether the bridge structure geometry is substandard or is functionally obsolete, or if other major 3R/4R or IPM-type of work on the associated segment of the highway is to be undertaken. Otherwise, the following guidelines shall be used in the decision making process.

(a) Concrete decks without overlays

The following guidelines are provided for general purpose use only. For comparative values for bridge decks, see Figure 1.

Collect the following information prior to determining the type of rehabilitation needed for a bridge.

(1) Extent of concrete spalls, in percentage of deck area.

(2) Extent of delamination in percentage of deck area and/or traveled lanes. Use infrared thermography, chain dragging, or other approved methods.

(3) Salt content in top 25 mm {1 in.} and in the last 25 mm {1 in.} of concrete immediately above the top mat of reinforcing bars.

(4) Air content. The lower the air content, the more important other deck quality factors become.

(5) Concrete strength. Use concrete coring and testing, and Windsor probe results, if permitted.

(6) Type of deck steel (black vs. coated reinforcement bar) concrete cover over the top rebar mat.

(7) Half-cell electric potential or other methods of determining active corrosion, as permitted by the Department.

(8) Age of the deck.

(9) Type and location of major cracking indicating superstructure flexibility problems.

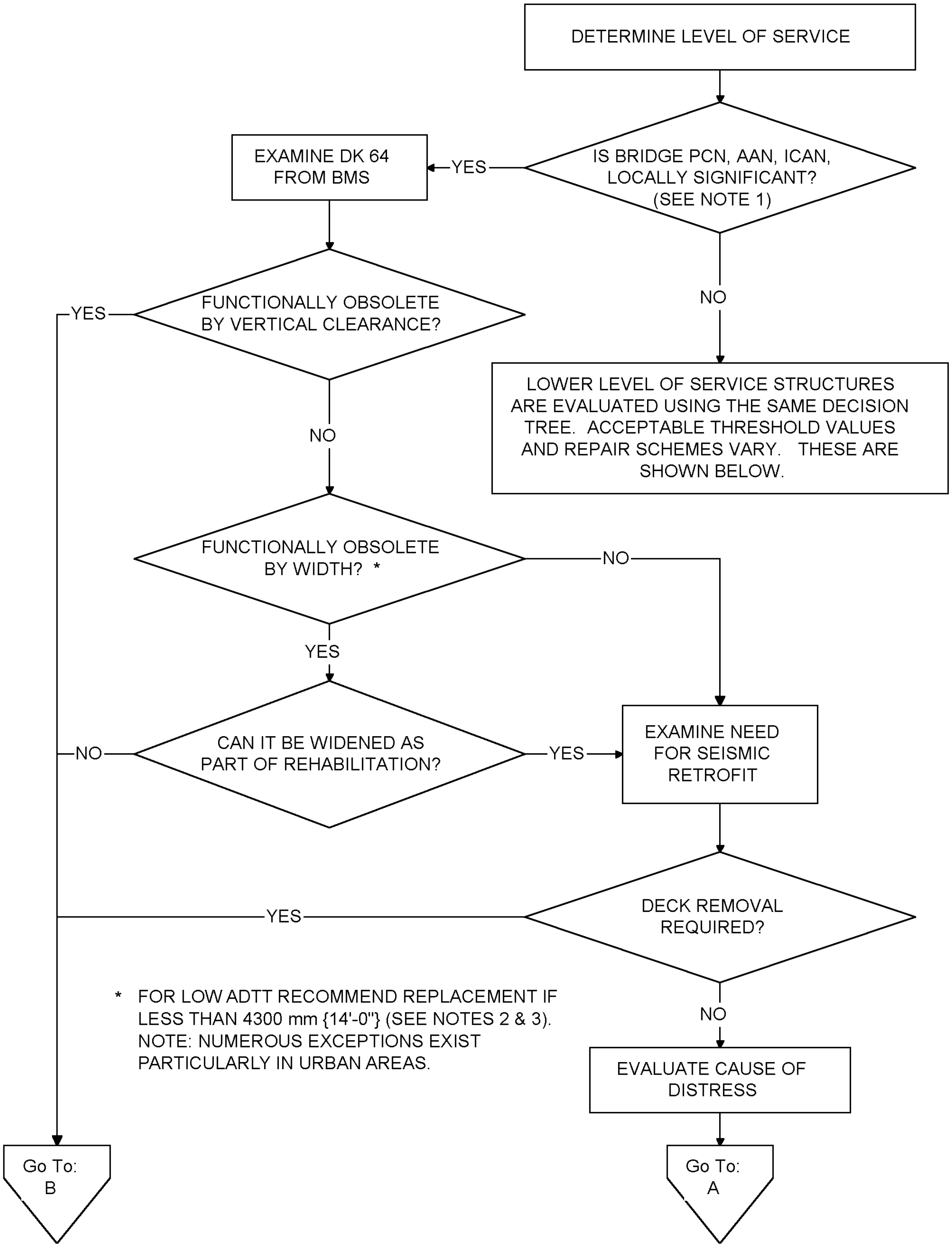


Figure 5.5.2.3-1 - Comparative Values for Bridge Deck

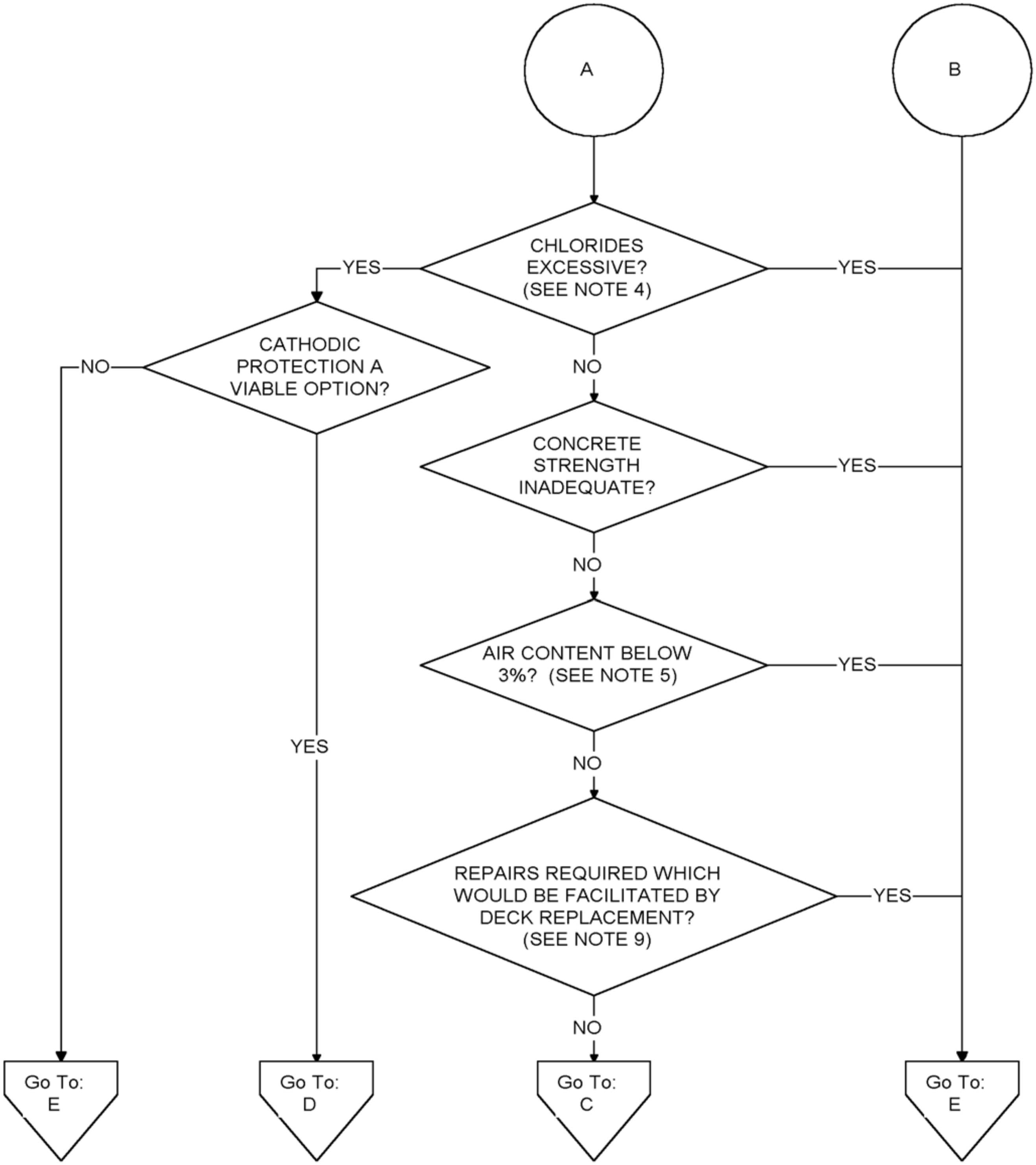


Figure 5.5.2.3-1 - Comparative Values for Bridge Deck (Continued)

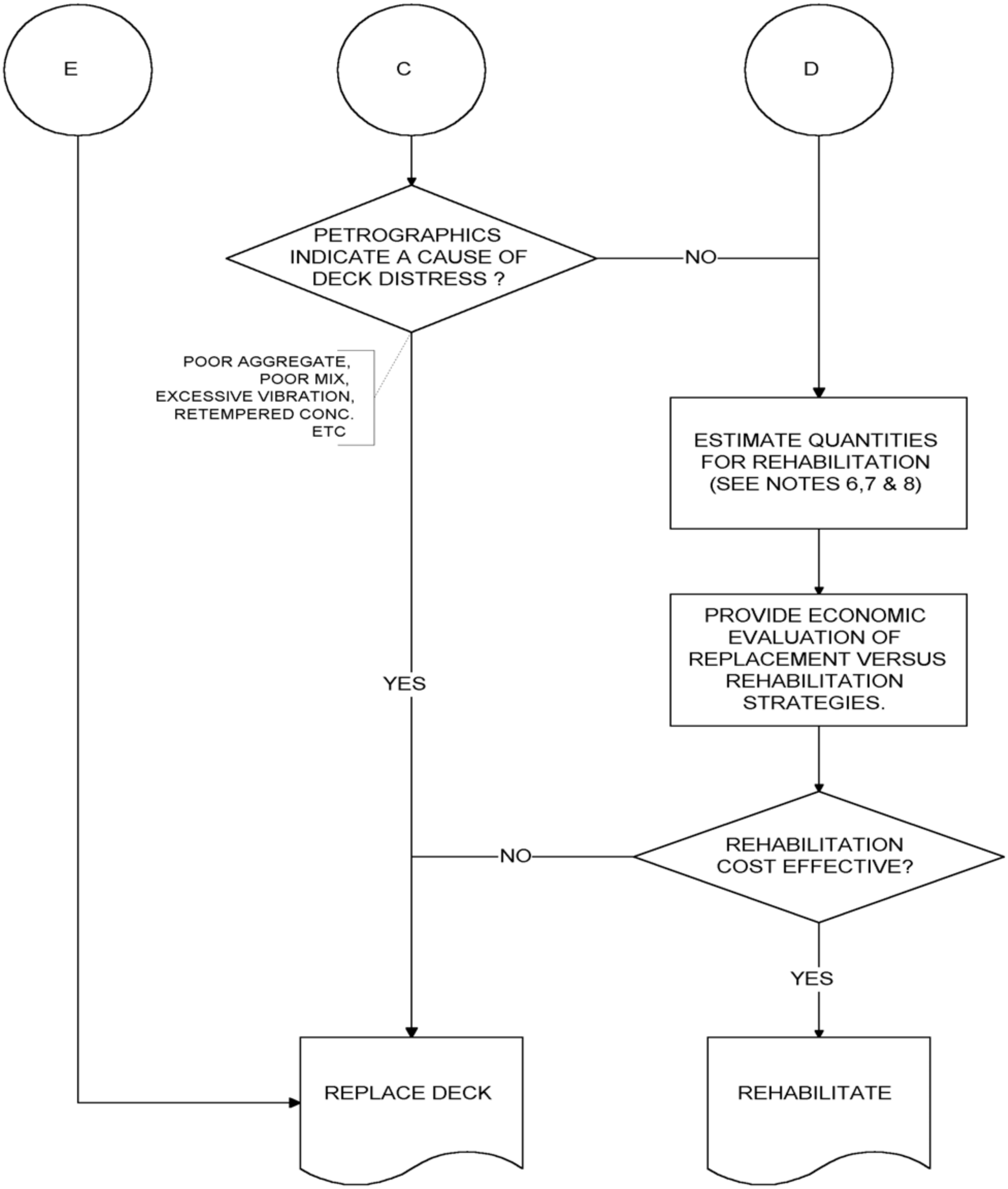


Figure 5.5.2.3-1 - Comparative Values for Bridge Deck (Continued)

NOTES:

(1) Bridges which have significant impact on the local economy should be included in this group.

(2) Reference, Ahlskog, O'Connor, "Level of Service Approach to Estimating National Bridge Needs", IBC-89-71.

(3) Condition rating of 3 per BMS.

(4) Research studies have found chloride contamination accelerates corrosion when chloride levels are between 0.6 and 0.8 kg/m^3. {1.0 and 1.4 pcy}. FHWA has not mandated threshold values for acceptable levels of chloride concentration. Contamination is often deemed as significant at the 0.9 to 1.2 kg/m^3 {1.5 to 2 pcy} level. However, bridge decks have continued to perform well with 3.6 to 5.3 kg/m^3 {6 to 9 pcy}. Higher levels of chloride contamination will be acceptable on lower level routes when programming rehabilitation/replacement strategies are considered. Each bridge will be evaluated independently.

(5) Air content will be a factor when determining the salvage value of a bridge deck. On PCN routes, low air less than 3% will be justification for replacement. Also consider air characteristics. A surface area of air voids (specific surface) < 24 mm^2/mm^3 {600 in^2/in^3}or a spacing factor of > 0.2 mm {0.008 in.} would also be justification for replacement. On lower level systems, low air content will be acceptable for certain structures, arches, etc. on lower level routes, low air content may be acceptable for other contents.

(6) Estimated quantities for rehabilitation should reflect the anticipated quantities in the year of construction. Recommended rehabilitation schemes for high and medium ADTT routes include cathodic and latex modified concrete.

(7) Estimated quantities for rehabilitation should reflect the anticipated quantities in the year of construction. Recommended rehabilitation schemes for low volume ADTT routes include bituminous overlays with membrane and latex modified concrete.

(8) Bituminous overlays without membranes are viable options on all routes when the bridge deck has been programmed for replacement, and the overlay is used to provide a smooth riding surface until replacement can be accomplished.

(9) Repairs would include seismic retrofit, bearing/pedestals, pier cap repairs, beam repairs, fatigue retrofits, etc.

Figure 5.5.2.3-1 - Comparative Values for Bridge Deck (Continued)

Items 1, 2, 3, 4, 6 and 8 are required in all cases. Collection of these information items may be stopped at any point when it becomes obvious that the deck must be replaced. The remaining items will be required depending upon the deck size, condition, roadway network and scope of the project. Any deviation must be approved jointly by the District Bridge Engineer and the Chief Bridge Engineer.

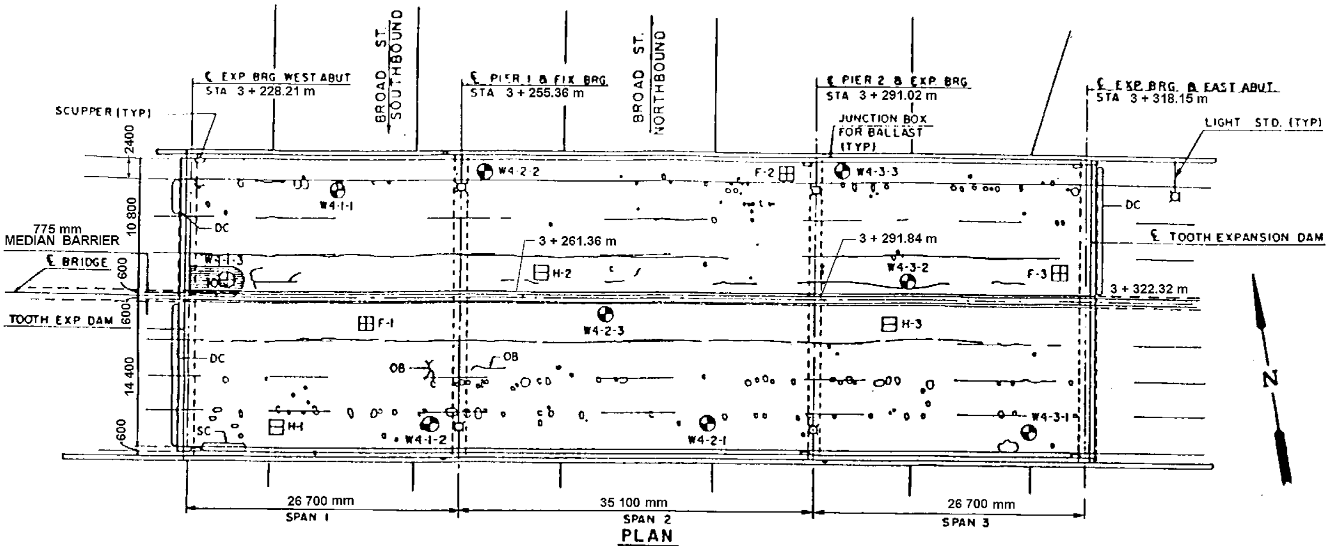
Provide deck survey and other results in the format shown in Figure 2.

Use Figure 3 "Bridge Deck Rehabilitation Guide". The table shown is based upon the assumption that the concrete deck has black steel. For concrete decks with epoxy or galvanized rebars, see PP5.5.2.3(b). The Chief Bridge Engineer will decide on any deviations for Federal oversight projects, and the District Bridge Engineer will decide on any deviations for PENNDOT oversight projects.

Where latex modified concrete overlay is to be provided, the deck shall be scarified. Scarify the deck in 5 mm {1/4 in.} deep passes to avoid structure damage (cracking and spalling) of the remaining deck and to eliminate pulverization of the concrete around the reinforcement due to the high pressures needed for more than a 5 mm {1/4 in.} pass. If Hydrodemolition removal concept is permitted, the required depth may be removed in one pass. If Hydrodemolition equipment is utilized, the equipment shall be a computerized, self-propelled machine that utilizes a high pressure water jet stream to provide a rough and bondable surface while removing all unsound concrete, rust, and concrete particles from any exposed reinforcement during the initial pass. Initial experience with Hydrodemolition has indicated that this treatment should provide the expected service life.

Refer to Standard Drawing BC-783M for Type 2 and Type 3 repairs, latex concrete overlay transition, and expansion dam retrofit for the latex concrete overlay concept, and BC-788M for scupper retrofit details.

METRIC UNITS



LEGEND

- CORE LOCATION
- CORE LOCATION TO CONFIRM THERMOGRAPHIC RESULTS
- FULL DEPTH REMOVAL
- HALF DEPTH REMOVAL
- ~ SEALED CRACK, UNLESS OTHERWISE NOTED
- OB OPEN CRACK
- OL OPEN CRACK WITH LEACHING
- AC ALLIGATOR CRACKS
- SC SLIPPAGE CRACKS
- DC "D" CRACKS
- PH POT HOLE
- RW RAVELING AND WEATHERING

GENERAL CONDITIONS

1. SHALLOW RUTTING CAN BE SEEN IN WHEEL PATHS.
2. AREAS ADJACENT TO SCUPPERS, CURB LINES AND EXPANSION DAMS HAVE BEEN SEALED.
3. LOW SEVERITY RAVELING AND WEATHERING, APPROXIMATELY 300 to 600 mm IN WIDTH, IS PRESENT ALONG THE EXTERIOR CURB LINES.

TOP OF DECK DEFECTS

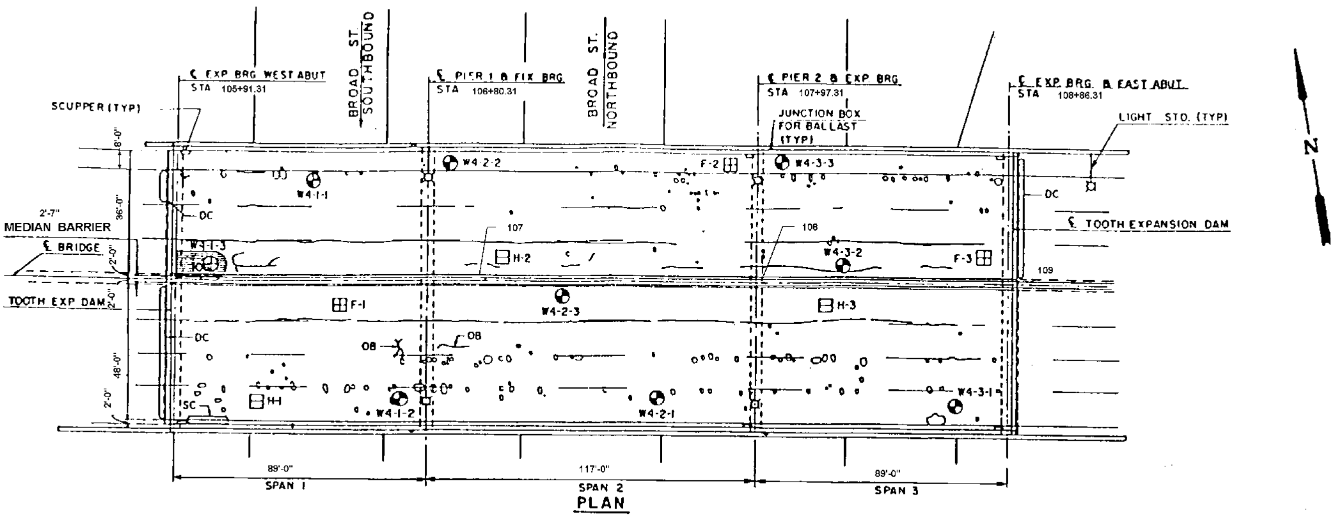
CURB TO CURB AREA (SPANS 1 TO 3): 1370 m² EASTBOUND RDWY. = 1206 m² WESTBOUND RDWY.

SYM.	TYPE	EASTBOUND ROADWAY AREA (m²)	EASTBOUND ROADWAY % OF DECK SPANS 1-3	WESTBOUND ROADWAY AREA (m²)	WESTBOUND ROADWAY % OF DECK SPANS 1-3
	DELAMINATION	20.1	1.5	9.3	0.8
	PATCH	0	0.0	0	0.0
	DEBONDED PATCH	0	0.0	15.4	1.3

EXPANSION DAM OPENINGS

LOCATION	EASTBOUND ROADWAY THEOR. (mm)	EASTBOUND ROADWAY ACT. (mm)	EASTBOUND ROADWAY TEMP. (C°)	WESTBOUND ROADWAY THEOR. (mm)	WESTBOUND ROADWAY ACT. (mm)	WESTBOUND ROADWAY TEMP. (C°)
WEST ABUT.	5	5	27	5	5	12
EAST ABUT.	5	5	27	5	5	12

Figure 5.5.2.3-2 - Format for Deck Survey and Other Results



LEGEND

Symbol	Description
	CORE LOCATION
	CORE LOCATION TO CONFIRM THERMO-GRAPHIC RESULTS
	FULL DEPTH REMOVAL
	PARTIAL DEPTH REMOVAL
~	SEALED CRACK, UNLESS OTHERWISE NOTED
OB	OPEN CRACK
OL	OPEN CRACK WITH LEACHING
AC	ALLIGATOR CRACKS
SC	SLIPPAGE CRACKS
DC	"D" CRACKS
PH	POT HOLE
RW	RAVELING AND WEATHERING

GENERAL CONDITIONS

1. SHALLOW RUTTING CAN BE SEEN IN WHEEL PATHS.
2. AREAS ADJACENT TO SCUPPERS, CURB LINES AND EXPANSION DAMS HAVE BEEN SEALED.
3. LOW SEVERITY RAVELING AND WEATHERING, APPROXIMATELY 1" TO 2" IN WIDTH, IS PRESENT ALONG THE EXTERIOR CURB LINES.

TOP OF DECK DEFECTS

CURB TO CURB AREA (SPANS 1 TO 3) 14,750 SF EASTBOUND RDWY. = 12,980 SF WESTBOUND RDWY.

SYM.	TYPE	EASTBOUND ROADWAY AREA (Sq. Ft.)	EASTBOUND ROADWAY % OF DECK SPANS 1-3	WESTBOUND ROADWAY AREA (SQ.FT.)	WESTBOUND ROADWAY % OF DECK SPANS 1-3
	DELAMINATION	216	1.5	100	0.8
	PATCH	0	0.0	0	0.0
	DEBONDED PATCH	0	0.0	165	1.3

EXPANSION DAM OPENINGS

LOCATION	EASTBOUND ROADWAY THEOR. (IN.)	EASTBOUND ROADWAY ACT. (IN.)	EASTBOUND ROADWAY TEMP. (°F)	WESTBOUND ROADWAY THEOR. (IN.)	WESTBOUND ROADWAY ACT. (IN.)	WESTBOUND ROADWAY TEMP. (°F)
WEST ABUT.	2 7/16	2 1/2	80	2 9/16	2 1/4	55
EAST ABUT.	2 5/16	2 3/8	80	2 11/16	2 3/8	55

Figure 5.5.2.3-2 - Format for Deck Survey and Other Results (Continued)

Table 5.5.2.3-1 - Concrete Decks with Epoxy or Galvanized Rebars

MAIN STRUCTURE – GLOUCESTER ANCHORAGE (BRIDGE MS-GA)								
Concrete Core Results							Windsor Probe Results	
Location	Chloride Content				Compressive Strength		Location	Compressive Strength
	Depth 1 (mm)	kg/m^3	Depth 2 (mm)	kg/m^3	h/d	MPa		MPa
MS-GA1-1	25	1.0	670	0.3			1	55
MS-GA1-2	25	3.3	670	2.0	1.64	40	2	50
MS-GA1-3	25	0.7	670	0.3	1.74	40	3	50
MS-GA2-1	25	0.9	670	0.3			4	60
MS-GA2-2	25	0.6	670	0.2	1.80	40	5	50
MS-GA2-3	25	3.0	670	2.0	1.49	35	6	50
MS-GA3-1	25	0.9	670	0.5			7	55
MS-GA3-2	25	1.2	670	0.9	1.37	35	8	60
MS-GA3-3	25	1.5	670	1.4	1.54	35	9	65

MAIN STRUCTURE - GLOUCESTER ANCHORAGE (BRIDGE MS-GA)								
Concrete Core Results							Windsor Probe Results	
Location	Chloride Content				Compressive Strength		Location	Compressive Strength
	Depth 1 (in)	pcy	Depth 2 (in)	pcy	h/d	ksi		ksi
MS-GA1-1	0’-1”	1.7	2’-2 1/2”	0.55			1	8.200
MS-GA1-2	0’-1”	5.5	2’-2 1/2”	3.4	1.64	5.720	2	7.500
MS-GA1-3	0’-1”	1.1	2’-2 1/2”	0.51	1.74	5.970	3	7.800
MS-GA2-1	0’-1”	1.5	2’-2 1/2”	0.47			4	8.850
MS-GA2-2	0’-1”	0.94	2’-2 1/2”	0.27	1.80	6.250	5	7.600
MS-GA2-3	0’-1”	5.1	2’-2 1/2”	3.3	1.49	5.060	6	7.600
MS-GA3-1	0’-1”	1.5	2’-2 1/2”	0.78			7	8.025
MS-GA3-2	0’-1”	2.1	2’-2 1/2”	1.6	1.37	5.180	8	8.600
MS-GA3-3	0’-1”	2.6	2’-2 1/2”	2.4	1.54	5.410	9	9.500

BRIDGE DECK REHABILITATION GUIDE (1)

DECK COND (2)	9 - 7						6 (6)						5						4						<=3					
% AIR (3)	<3%			=>3%			<3%			=>3%			<3%			=>3%			<3%			=>3%			<3%			=>3%		
DECK AGE (4) REHAB/ REPLACE STRATEGIES (5)	POST 1976	1971- 1976	PRE 1971	POST 1976	1971- 1976	PRE 1971	POST 1976	1971- 1976	PRE 1971	POST 1976	1971- 1976	PRE 1971	POST 1976	1971- 1976	PRE 1971	POST 1976	1971- 1976	PRE 1971	POST 1976	1971- 1976	PRE 1971	POST 1976	1971- 1976	PRE 1971	POST 1976	1971- 1976	PRE 1971	POST 1976	1971- 1976	PRE 1971
(LO)	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●												
(BP)																									•	•	•	•	•	•
(BO)																									•	•	•	•	•	•
(RO)								L	L																					
(RMO)								M	M		L	L																		
(R)							●			●																				
(RL)								H	H		M,H	M,H										M,H	M,H	M,H						
(RC)												H											M,H	M,H						
(RCMO)																							M,H	M,H						
(RCL)																							M,H	M,H						
(RPL)																			H					H	●	●	●	●	●	●

KEY

Where options appear in more than one row, an economic evaluation is required and/or additional test data to support a decision.

● Denotes action for all occurrences (level of service) of a given condition rating.

* With concurrence of the District Bridge Engineer.

L On low volume routes (ADTT < 100)

M On medium volume routes (100 ≤ ADTT ≤ 500)

H On high volume routes (PCN + Exceptions) (ADTT > 500)

Shaded area is "gray area" where other factors must be considered.

Follow procedures in Figure PP5.5.2.3

NOTES:

(1) The Bridge Deck Rehabilitation Guide as such is for generalized usage. Exceptions will exist. These may include: monolithic deck systems, arches, concrete tee-beams, etc.
(2) Refers to deck condition rating established using BMS Item E17.
(3) Refers to measured entrained air content.
(4) Dates were established based on coated steel versus uncoated steel. Pre-1971 black steel was used in PA. 1971-1976 both epoxy-coated and galvanized, Post 1976 epoxy-coated steel was utilized.
(5) Rehabilitation, maintenance and replacement options are listed per the following code:

LO = Linseed oil or other surface treatment (silane, etc.) at prescribed maintenance intervals (maintenance activity)
BP = Bituminous patching only (maintenance activity)
BO = Bituminous overlay only (maintenance activity)
RO = Repair Types II and III, plus bituminous overlay
RMO = Repair Types II and III membrane waterproofing, plus bituminous overlay (maintenance activity)
R = Repair Types II and III
RL = Repair Types II and III, plus latex concrete overlay
RC = Repair Types II and III, plus cathodic protection
RCMO = RC plus membrane waterproofing and bituminous overlay
RCL = RC plus latex modified concrete
RPL = Replace the deck

(6) Intuitively the Benefit/Cost ratio on lower volume roads will not support high cost rehabilitation options.

Figure 5.5.2.3-3 – Bridge Deck Rehabilitation Guide

(b) Concrete Decks with Epoxy or Galvanized Reinforcement Bars, with Spalls

Provide Type 2 or Type 3 repairs. Surface deterioration due to loading, abrasion and other similar activities can be corrected by concrete repairs and latex modified concrete overlays. Bituminous overlays are not recommended since they entrap contaminated moisture beneath the overlay and accelerate deterioration.

(c) Concrete Decks with Bituminous Overlay and without Membrane Waterproofing

Replace such decks when they have a condition rating of three or less. For preservation or rehabilitation projects, cores shall be taken and assessed per PP5.5.2.3(a) to determine scope of work.

(d) Concrete Decks with Bituminous Overlay and Membrane Waterproofing

Patch deteriorated concrete and replace affected membrane and overlay until the repaired area is anticipated to be 50% or more of the deck area. Follow Figure 3 thereafter.

(e) Concrete Decks with Latex Modified Mortar or Concrete Overlay

Follow Figure 3. Consider latex modified mortar or concrete overlaid decks the same as regular concrete deck when using this table. If a new overlay is required, the existing overlay and deteriorated deck areas must be removed, and appropriate patching and overlay should be provided as per Figure 3. Pressurized water jet concrete removal may be cost effective in such instances.

(f) Open Grid Steel Decks

Open grid steel decks have a poor performance record. Fatigue cracking of welded grid members have been a continual maintenance problem, even on routes with low ADTT. For existing bridges where continuous maintenance welding is experienced, replace deck with concrete filled grid steel deck or fill the existing deck with concrete.

Mechanical connections between the deck and the supporting beams is preferred over welded connections for ease of construction and minimization of weld cracking. This should minimize continued maintenance costs. Also specify shims to eliminate residual construction load stresses induced by the placement of a load over the grid.

Install anti-skid studs when needed.

Open end steel grid decking shall not be used for new construction, or deck replacement when the anticipated total accumulated truck traffic will exceed 300,000 trucks (exceeding 2.7 tonne) {3 tons} per lane during the expected life of the flooring. It has a poor performance record and promotes rapid deterioration of the supporting members. Such decks may be used for a temporary bridge or as a temporary deck.

(g) Concrete Filled Grid Steel Decks

The use of grid reinforced concrete bridge decks is permitted within the limitations imposed by Standard Drawing BD-604M.

Generally, these decks last for a long time (40 to 50 years). However, in older designs the filled concrete may cup out after ten years or so, depending upon the quality of the original work and whether or not the concrete was placed above the deck steel.

Remove deteriorated concrete and fill the cups with concrete if the full-depth concrete is deteriorating. Use of calcium nitrate may be considered as a rust inhibitor. Fill the cups with overlay material if only the top quarter or less of concrete is deteriorated, and overlay the deck with either bituminous or asphalt as a short-term (i.e., < 5 years) or latex modified concrete overlay as a mid-term (10 $\pm$ years) or other approved polymer resin concrete overlay as a long-term (15 to 20 years) solution.

Also with older designs, longitudinal creeping problems of concrete filled grid decks is well documented, particularly with decks having large (more than three) aspect (span length/width) ratios. Timely deck cut-off and releasing the deck for expansion will minimize chances for secondary stresses in other bridge members. Provision for closely spaced expansion joints would reduce deck creeping. However, it can increase expansion joint maintenance and the probability of subfloor and substructure deterioration.

This deck type may be used for deck replacement in special situations only, since it is very expensive and longitudinal creeping is a major concern.

(h) Timber Decks

Replace the deteriorated members with treated lumber. If deck deterioration is over 25%, replace the entire deck with treated timber or other material. Bituminous overlay may be provided to improve riding quality. If overlaid, use a leveling course prior to providing the surface course.

(i) Other Deck Types

Rehabilitation of other deck types shall be evaluated and rehabilitation strategies shall be developed based upon the characteristic, performance and condition of such decks. In some instances, special studies will be needed.

For determining construction quantities, anticipate further deck deterioration from the day of field investigation to the actual deck work and anticipate considerably more Type 2 and 3 repairs than field observations indicate.

Deck repairs and rigid deck overlay or overlay repairs are eligible work under IPM criteria.

5.5.2.4 EXPANSION JOINTS

Eliminate expansion joints at substructure units whenever practical. The design life for bridge joints shall be compatible with deck life, which is currently considered to be between 40 and 50 years for decks with epoxy-coated bars or decks with similar deck protective systems and grid reinforced concrete bridge decks built in accordance with BD-604M. The deck joints incorporated in Standard Drawings BC-767M and BC-762M are expected to provide 40 to 50 years of life if properly fabricated and constructed. For maintenance projects with an expected service life up to 5 years, a two-part silicone joint system may be used. The expansion joint indicated in Standard Drawing BC-766M would provide reasonable life for low volume (ADT < 1000) roads. No other type of deck joint shall be incorporated without specific approval by the Chief Bridge Engineer.

Provide a trough or similar device under the existing tooth dams to protect the beams and substructure units from salt contamination and water.

Replace existing plate dams using either deck continuity or an appropriate joint specified in this section.

(a) Deck or Superstructure Replacement Projects

Minimize the number of joints by providing deck continuity over the existing joints and/or fixing the abutments, thereby eliminating backwalls if the criteria outlined in D11.6.1.7P is applicable. Refer to D14.5 for selection of the type of expansion joint.

The secondary effects of deck continuity, if done in conjunction with beam continuity, must be evaluated and corrective retrofits, if warranted, incorporated into the contract. Such effects may include effects on the type and size of bearings, superstructure to substructure connections, and structural capacity of substructure units including foundations.

Eliminate all pin-hanger joints and provide structural continuity. Effects of structural continuity on all superstructure and substructure elements, including foundation elements, shall be analyzed and appropriate structural modifications made.

For existing short-span prestressed concrete beam bridges (each span less than 24 000 mm {80 ft.}) having beam depth differences between the adjoining spans of 160 mm {6 in.} or less, consideration may be given to using only deck continuity. Full depth diaphragms shall be provided as per BD-664M and BD-665M at pier locations. In such instances No. 13 bars at 125 mm {No. 4 bars at 5 in.} spacing longitudinal reinforcing steel in the top and bottom of the deck shall be provided as minimum steel for distribution to prevent transverse cracks.

For existing multi-span steel beam bridges, the designer shall consider providing continuity using flange and web connection plates at pier locations. In such situations, analysis of the girder as a continuous member shall be performed. The skew effect behavior of steel bridges and their relative flexibility when compared with prestressed concrete bridges must be evaluated and considered in the analysis.

At each pier, the beam end fixity must be the same at each of the beams. See Figure 1 for allowable configuration. Mixing of fixity (Expansion/fixed) is not allowed at these locations. Beam end restraints may require changes. Replacement of dual bearing lines with a single bearing should be considered particularly if replacement of the bearings is required.

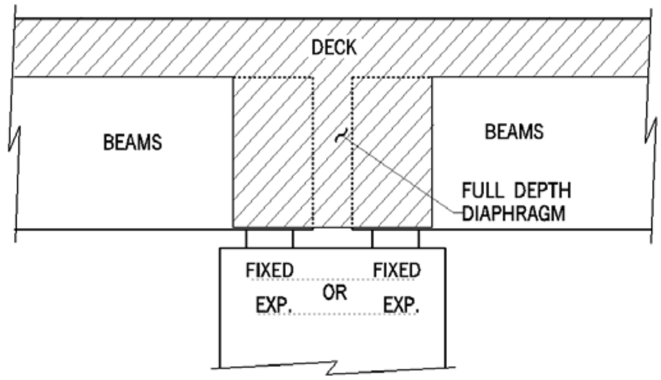


Figure 5.5.2.4-1 - Full Depth Diaphragms

(b) Deck Joint Replacement

Elastomeric deck joints that are found not to be repairable shall be eliminated by providing deck continuity or replaced with the joint types specified in Standard Drawings BC-767M, BC-762M or BC-766M.

In special situations shallow depth (40 mm or 50 mm {1 1/2 in. or 2 in.} deep) strip seal dams may be used with a specific approval from the Chief Bridge Engineer. Refer to Figure 2 for general details.

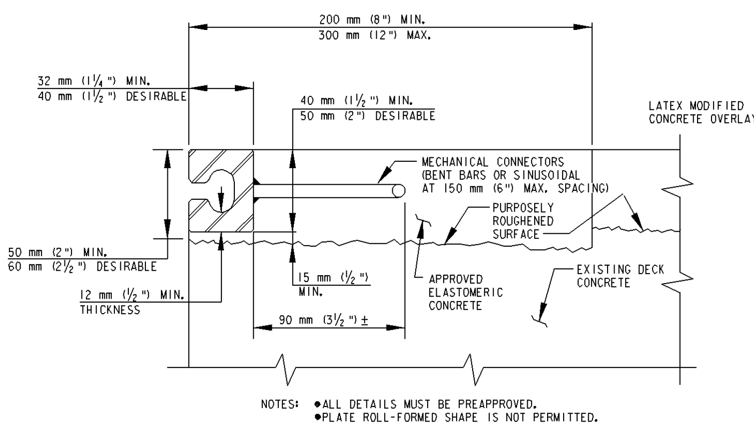


Figure 5.5.2.4-2 - Strip Seal Dams Detail

For bituminous overlays use Figure 3.

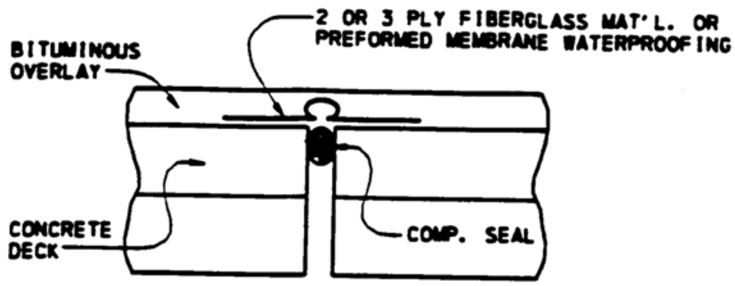


Figure 5.5.2.4-3 - Deck Joint for Bituminous Overlays

Replace damaged gland or troughs as necessary. Clean troughs. Non-performing expansion dams should be replaced with strip seal or other approved expansion dams.

(c) Elastomeric Expansion Dams

An analysis of the data for expansion joints using elastomeric expansion dams (received by May 31, 1994) revealed the following:

1. The first elastomeric expansion dams were installed in 1972.

2. Distress in these joints, in order of highest incidence were as follows:

   a. Anchorage Failure
   b. Concrete spalling at joint
   c. Neoprene Failure

3. There was some indication that durability of the joints was related to truck volume. However, some joints on bridges having large truck volumes were outlasting those joints on bridges with small truck volumes.

4. Some dams had been replaced, others had been repaired, and many others required maintenance or replacement. A few were performing well.

5. A total of six suppliers furnished all the elastomeric expansion dams used in the State. They were:

   a. Watson-Bowman Association
   b. Fel-Pro
   c. D. S. Brown Co.
   d. General Tire
   e. A. H. Harris
   f. Royston

   Appropriate rehabilitation strategies for these deck joints based on the type of rehabilitation required are as follows:

   - Deck Replacement

     When complete deck replacement is anticipated, consider elimination of deck joints first, where feasible. The decision to eliminate existing joints should be based on length of structure, type of bearings and substructure/foundation compatibility (see Section (a) of the article).

     Where elimination of joints is not feasible, expansion devices, as shown in Standard Drawing BC-767M (Neoprene Strip Seal Dam, armored, for movements up to 100 mm {4 in.}), should be used. Tooth dams with 3 mm {1/8 in.} thick reinforced sheet neoprene trough (50 durometer hardness) shall be specified for movement over 100 mm {4 in.}. Standard Drawing BC-766M (Preformed Neoprene Compression Seal Joint, unarmored) is discouraged but may be used for structures having ADT less than 1000 and ADTT less than 100 with caution because its success in terms of providing a leakproof joint is highly dependent upon perfect construction.

   - Joint Repair

     There were many existing elastomeric dams that were performing well. However, some required maintenance such as anchor bolt replacement, full or partial seal replacement, hold-down plates or section replacement.

     If a cost analysis shows that repair is cost effective, <u>and the repair will restore the joint to water tightness</u>, every effort should be made to schedule the maintenance.

     Except for anchor bolts, replacement parts should be obtained from the appropriate suppliers to assure compatibility with the existing in-place dams. The use of steel plates to make temporary or makeshift repairs to damaged dams is not encouraged since the function of the dam to provide a water tight joint is not restored.

Generally, it has been found that the anchor bolt arrangements for the various dams were inadequate, particularly in areas of high stress (wheel paths). Consequently, bolts that have sheared off or pulled out should be replaced with new anchor bolts epoxied into pre-drilled holes. See Figure 4 for repair scheme.

Some elastomeric expansion dam failures originated from poor initial installation, particularly poorly consolidated concrete or uneven bedding in the block-outs. It is recommended that these areas be inspected during repairs to determine if other improvements should be made.

If temporary expansion dams are essential, asphaltic plug joints, elastomeric concrete with a two-part cold applied polymeric seal, or preformed silicone strip seals, as applicable, may be used as a stop gap measure.

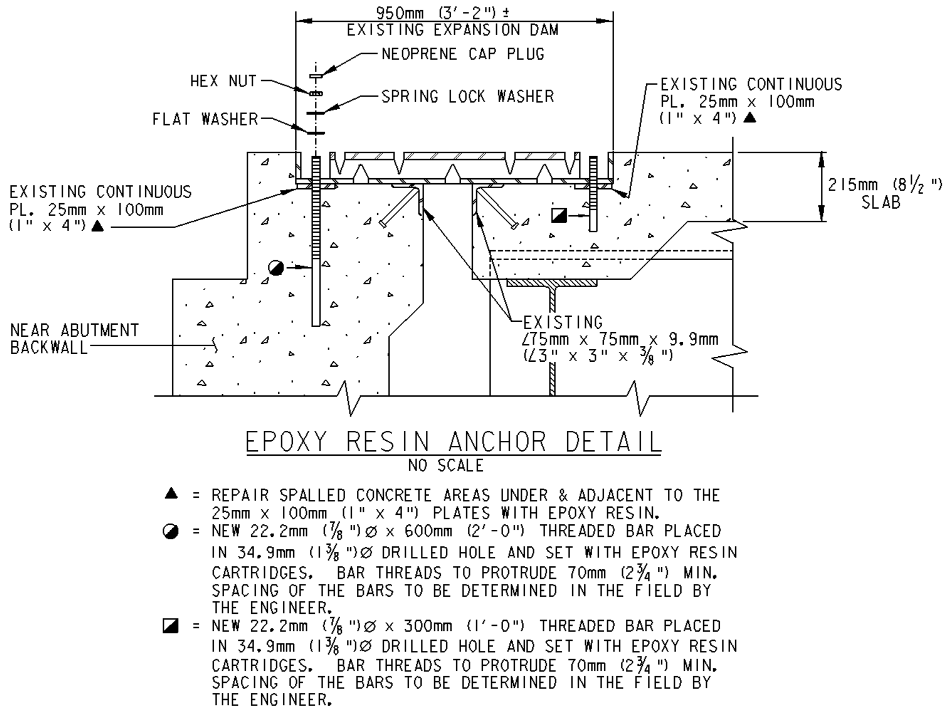


Figure 5.5.2.4-4 - Epoxy Resin Anchor Detail

- Joint Replacement

Elastomeric deck joints that cannot be repaired should be scheduled for replacement.

Several manufacturers have developed <u>strip seal joints</u> that are embedded in elastomeric concrete which can be used to bond the total joint to the concrete blockout. These proprietary systems include Delcrete, Wabocrete and Cevacrete, and are particularly well suited as replacement systems for existing elastomeric expansion dams because the required blockout depth is between 50 mm and 65 mm {2 in. and 2 1/2 in.}, and the required blockout widths are comparable to the existing blockouts that accommodate elastomeric seals.

The above tradenames apply to elastomeric concretes furnished by the various manufacturers, i.e., D. S. Brown Co., Watson-Bowman Association and Epoxy Industries, Inc., respectively. The joints should be a combination of the elastomeric concretes, appropriate <u>extrusions</u> and compatible waterproof neoprene strip seals.

Strip seals are available for movements up to 125 mm {5 in.}. However, they should not be used for more than 100 mm {4 in.} movement classification due to our concern for load carrying capacity of wider openings.

Some of these systems use heat fusion bonding which will heat cure in about 2 hours, will bond to irregularities, such as spalls, in the blockout area, and are suitable for stage construction.

5.5.2.5 BEARINGS

(a) Rocker and Roller Bearings

For minor bridge rehabilitation work, unless replacement is warranted due to seismic requirements or functional obsolescence, readjust all rocker and roller bearings to restore their required function. Clean, paint and lubricate (roller bearings only) as warranted. For deck replacement or other major bridge rehabilitation projects, rocker bearings should be replaced and roller bearings should be considered for replacement unless seismic criteria is met.

(b) Other Metal Bearings

Restore the required function of these bearings, as warranted, by repairing or replacing worn-out parts. Special attention and analysis may be needed for bronze or similar friction parts.

(c) Pot Bearings

Ensure that the neoprene material is adequately contained in the pot and the gap between the top of the pot and the piston bearing plate is fairly uniform under dead load. Also, sufficient end distance should exist to the stainless steel plate (mirror plate) for expansion and contraction at extreme temperatures. If any deficiencies exist, corrective measures must be incorporated into the contract plans.

(d) Other Multi-Rotation Bearings

If adverse functional conditions exist, corrective measures must be incorporated into the contract plan.

(e) Neoprene Bearings

Problems such as major uneven deformation or walk-out shall be corrected.

(f) Bearings for Temporary Construction Condition

Appropriate bearing type and restraining connections shall be designed to endure construction and traffic loads. Expected life of the temporary bearing shall be five years.

5.5.2.6 OTHER SUPERSTRUCTURE ELEMENTS AND FATIGUE

Establish material parameters based upon existing plans or previous testing. If data is not available, samples may be taken and tested to establish needed parameters. All construction details must be inspectable and maintainable.

(a) Redundancy

For non-redundant superstructure, ensure that all elements are structurally sound and will provide prescribed service life as specified in PP5.5.4. Where possible, an alternate load path should be provided if economically feasible. All pin-hanger connections shall be removed and replaced with continuity when replacing the deck as specified in PP5.5.2.4(a).

For redundant superstructures, the pin-hanger connections shall also be replaced when replacing the deck.

(b) Deteriorated Beam Ends and Painting

Deteriorated concrete beam ends shall be cleaned, repaired for structural integrity and protected from future deterioration either by deck continuity, encasing in concrete (diaphragm), or providing leakproof joints, and applying a breathable coating.

Deteriorated steel beam ends shall be cleaned, strengthened if needed, painted and protected from future deterioration by providing continuity or leakproof joints.

Where needed, spot and zone or total bridge painting shall be incorporated in the rehabilitation project to achieve the targeted life specified in PP5.5.4, unless a special painting contract is to follow very shortly after the rehabilitation project. Either the contract plans or special provisions must indicate whether or not the existing paint contains lead and other toxic materials such as cadmium, chromium, arsenic, etc., in order to alert the contractor. Paint coating coupons from different bridge members must be laboratory tested for lead content and other toxic materials such as cadmium, chromium, arsenic, etc. To determine cleaning and painting strategy, evaluate the thickness of the paint to be retained, adhesiveness and compatibility of the existing paint to the proposed paint system. For small span steel bridges with lead base paint, it may be cost effective to replace the superstructure.

To determine cost effectiveness, compare the remaining fatigue life, load carrying capacity, steel repair costs, cleaning and painting costs and other associated costs for the existing bridge, to the longer life and relatively minimal maintenance costs associated with a new superstructure. In borderline cases, permit a Contractor's alternate for a new superstructure.

(c) Cable Bridges

For cable-stayed and suspension bridges, cable condition must be thoroughly evaluated to ensure the targeted service life. Cables in the anchoring zone and splash zone are the most vulnerable. Deteriorated cables shall be replaced or reconstructed.

(d) External Post-Tensioning

External post-tensioning may be utilized to provide adequate load carrying capacity for short-term rehabilitation subject to special approval of the Chief Bridge Engineer. External post-tensioning is to be used as a last resort since the longevity of an unbonded system is questionable without periodic inspection. If external post-tensioning is employed, double corrosion protection to the prestressing tendons and end anchors shall be provided where possible.

(e) Jacking of Superstructure

Where superstructure jacking is required, at least one constructible option must be shown in the contract documents. All related analysis, including the effects of jacking on all connections, superstructure and (rarely) substructure elements must be evaluated. If strengthening is required, all details shall be shown in the contract documents. A Contractor's alternate may be permitted through a special provision or notes on contract drawings.

(f) Curb and Barrier

If the existing deck is to be replaced, construct standard curb and barrier.

Guide rail transition to the bridge barriers (F-shaped barrier, PA HT, PA Type 10M and PA vertical wall barriers) shall be made as per Standard Drawings BC-739M, BC-707M, BC-708M and BC-703M. Guide rail transition to older style barriers (New Jersey shaped barrier) shall be developed using Standard Drawing BC-739M as the basis. Any exception shall be approved by the Director, Bureau of Design.

If the existing deck is to remain in place, it may be possible to modify the existing curb and barrier to meet the current standards. This decision should be made based on accident history, ADT and ADTT, approach geometry and sight distance, severity of the condition and the cost of improvements, and follow the details in Standard Drawing BD-614M.

(g) Fatigue Evaluation and Retrofit

Determine remaining fatigue life of all critical details as specified in PP5.1, 5.2 and 5.3 and provide appropriate corrective measures.

For cover plate retrofit, refer to PP5.2.1.2. Retrofit all critical load and displacement-induced fatigue details. For displacement-induced fatigue retrofit details and load-induced retrofit details refer to PennDOT Research Project 83-21, "Deformation Induced Cracking in Steel-girder Bridges and Retrofit Guidelines", published in July 1987, and FHWA, March 1990 Publication, "Fatigue Cracking of Steel Structures", Volume II, Publication No. FHWA-RD-89-167.

(h) Utility Supports

Verify that all utility supports are structurally and functionally sound. They shall be either galvanized or coated with non-staining coating. If not, incorporate appropriate corrective measures.

(i) Structure-Mounted Signs

Ensure that all signs and sign connections are structurally sound. Specify repairs or modifications as necessary.

5.5.2.7 SUBSTRUCTURE ELEMENTS AND SCOUR

(a) Crack Sealing

Specify repair and/or rehabilitation of all deteriorated or damaged components. Special attention should be paid to cracked concrete pier caps, since the critical reinforcement for the pier caps may be vulnerable to corrosion if the concrete is crushed and exposed to contaminants from leaking joints. All such cracks shall be sealed with appropriate epoxy compounds.

If the cracking is caused by differential settlement, the situation shall be evaluated and corrected.

(b) Concrete Repair

Surface spalls of the concrete elements shall be cleaned to sound concrete and repaired with epoxy mortar. If deteriorated concrete extends beyond the primary reinforcement, the concrete shall be removed to at least 25 mm {1 in.} below the reinforcement and repaired with either concrete (if space permits) or lifts of epoxy mortar. An epoxy bonding compound shall be specified between the old and the new concrete and concrete lifts if needed. If significant deterioration exists, provide a temporary support to the superstructure and specify the needed repairs or replacement.

(c) Abutment Spalling

Spalling of the abutment stem under full-depth concrete diaphragm shall be repaired as depicted on Figure 1. Pavement migration should be corrected by providing a pavement relief joint.

(d) Hammerhead Pier Caps

External post-tensioning, as specified in PP5.5.2.6(d), may be specified for hammerhead pier caps to restore structural integrity. If a need for external post-tensioning is established, the system shall be designed to carry the entire load assuming the existing cap steel is ineffective. Adequate and uniform bearing between the concrete face and the post-tensioning system bearing plate shall be provided.

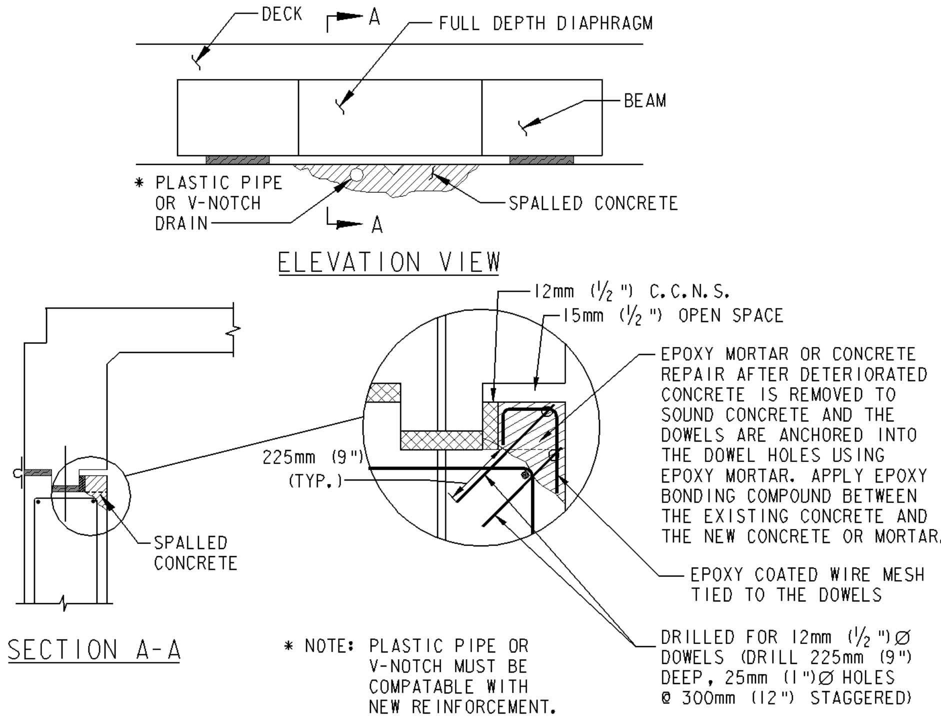


Figure 5.5.2.7-1 - Repair of Abutment

Other alternatives, such as casting wall or other support beneath the cantilever, may be feasible and may offer a long-term solution.

(e) Backwall Deterioration

Repair deteriorated backwalls. If the backwall is severely damaged due to pavement migration, it shall be replaced and a pavement relief joint installed.

(f) Structural Stability

If the existing substructure unit is determined to be marginal in overturning and/or sliding or overall slippage stability, the situation shall be corrected by appropriate measures. Generally, abutments on steep slopes are of concern. Such situations can be corrected by slope stabilization and the use of tie backs, replacement of the substructure unit, or by adding a span.

If settlement is a problem, the root cause(s) shall be determined and corrective measures shall be incorporated. Corrective measures may include underpinning, revising superstructure bearings if settlement has ceased, other appropriate corrective measures or replacement.

If settlement of a substructure unit founded on steel piles is evident, the integrity of the piles could be suspect. Settlement of point bearing piles in karst conditions may indicate sinkhole activity. Settlement of steel friction and/or bearing piles may indicate subsidence of substrata or corrosion activities of the piles. Settlement of friction or end bearing steel piles passing through fill materials may indicate corrosion, negative friction, overloading of piles, and settling substrata, or a combination thereof. Each situation should be evaluated on its own merit and corresponding corrective action shall be specified.

(g) Scour

Evaluate scour susceptibility and provide remedial measures as per HEC-18 and PP7.2.

(h) Cathodic Protection

For existing substructures with evident salt damage, the use of cathodic protection should be considered. Even though the technology of cathodic protection has advanced significantly over the past several years, it is still considered experimental, particularly for substructures. Cathodic protection of substructure units must be approved by the Bureau of Construction, Material and Testing Division.

5.5.2.8 SEISMIC CONSIDERATIONS

5.5.2.8.1 General

At this time, it is the policy of the Department to require an analysis and complete seismic assessment on all rehabilitation projects. As part of this assessment, the Designer must indicate all deficient seismic items and provide preliminary details for any needed seismic retrofits. The Chief Bridge Engineer may permit a waiver of seismic retrofits on a project-by-project basis, if requested from the District Engineer. The request must include a detailed justification for waiving the retrofit and explain when the structure will be completely upgraded to handle seismic loads.

Please note that the Department has not developed standard retrofit details. Schematic examples indicated herein are acceptable. FHWA Research Reports FHWA-IP-87-6, FHWA-RD-83-007 and FHWA-RD-94-052 also contain acceptable references for retrofit details.

The following lists and sketches provide additional direction and guidance into common and uncommon retrofit details to be used in Pennsylvania. These items are not to be considered exhaustive nor should they exclude sound engineering practice.

5.5.2.8.2 Common Retrofit Concepts

1. Replace high rocker and roller bearings. See Figures 1 and 2 for examples of these bearing types and an example of a typical replacement of these bearings, respectively.

2. Extend bearing seats. Bearing seat lengths must meet the minimum support lengths as per the design specifications. This must be addressed on rehabilitation projects. Seat extensions, as per Figure 3, should be provided.

3. Provide cribbing for vulnerable bearings. While it is desirable to eliminate vulnerable bearings (i.e., rocker and roller bearings), the Department recognizes that this is not always possible or cost effective. Cribbing to assure support of the superstructure in the event of a bearing collapse is an acceptable alternate to complete bearing replacement. Cribbing is expected to be used as a temporary measure until an economical bearing replacement can be performed (i.e., during a deck or bridge replacement). See Figure 4. Note that the approval of the Chief Bridge Engineer is required if rocker and roller bearings are not replaced.

4. Add Shear Blocks and/or Pedestals - Structures which are deficient in areas such as seat length and bearing instability, or have inadequate superstructure to substructure connections may be retrofitted by addition of shear blocks and/or dowel bars, or by construction of concrete pedestals which will act as shear blocks and alleviate bearing instability. See Figures 9 and 10.

5. Department sponsored research has shown that piers and columns built according to pre-1992 AASHTO criteria should have acceptable seismic performance for all regions of Pennsylvania provided confinement reinforcement consists of a minimum of No. 13 bars at 300 mm {No. 4 bars at 12 in.} and development and splice lengths meet current AASHTO requirements. If these conditions are met, no retrofit to the columns or piers is required. This does not mean that no damage will occur in the event or a ground acceleration of 0.15g, but the damage should not be life threatening and should be repairable (Memari et al. 2001). In cases where this minimum reinforcement is not present, see Figures 5, 6, 7 and 8 as acceptable means to retrofit this deficiency.

   Another less costly seismic retrofit method is to perform a more elaborate non-linear analysis including lateral strength or/capacity/demand methodology on a case by case basis.

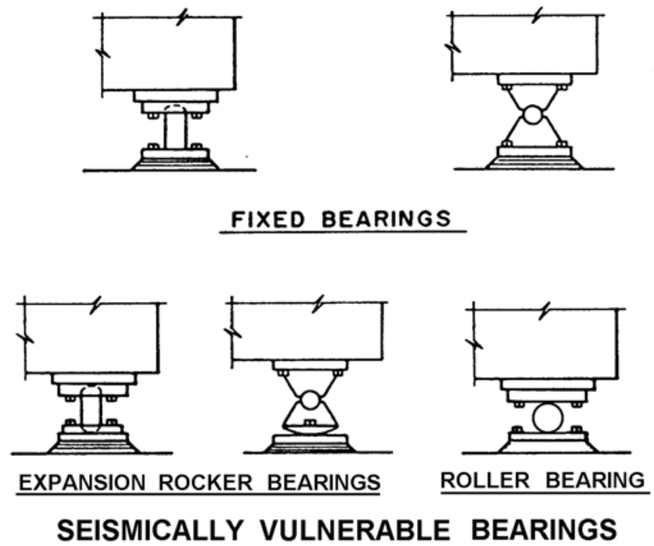


Figure 5.5.2.8.2-1 - Seismically Vulnerable Bearings

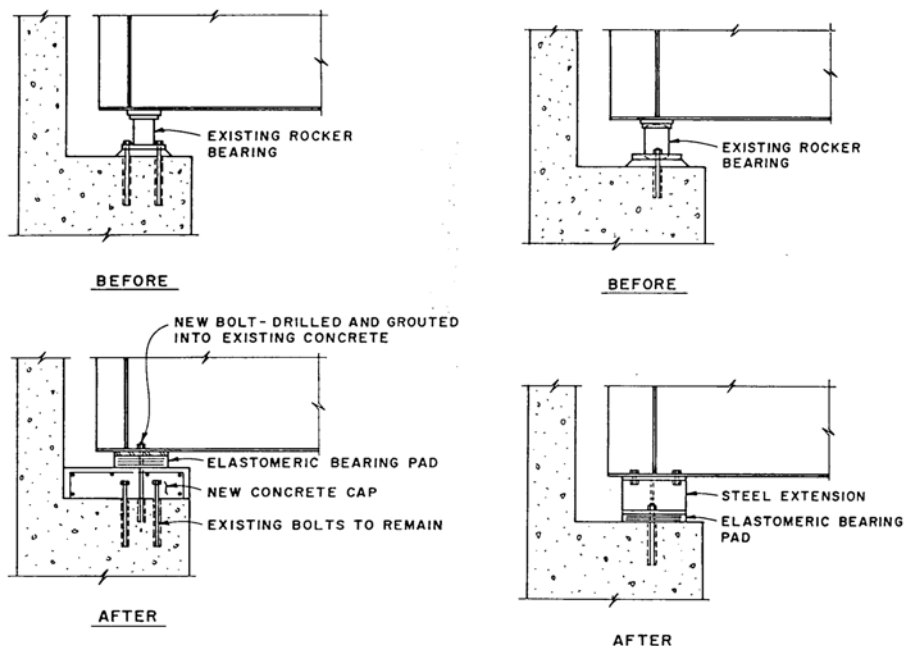


Figure 5.5.2.8.2-2 - Examples of Rocker Bearing Replacement

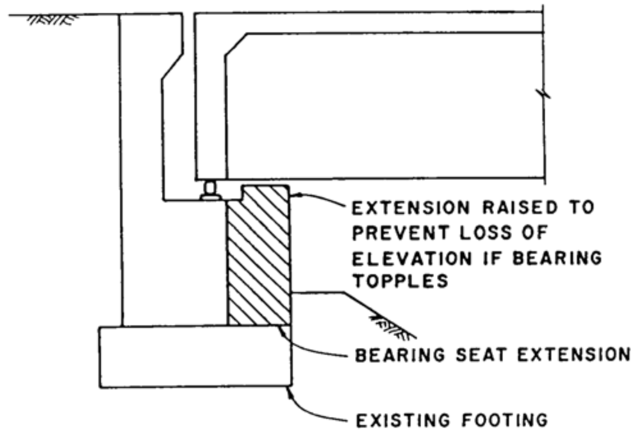


Figure 5.5.2.8.2-3 - Bearing Seat Extensions

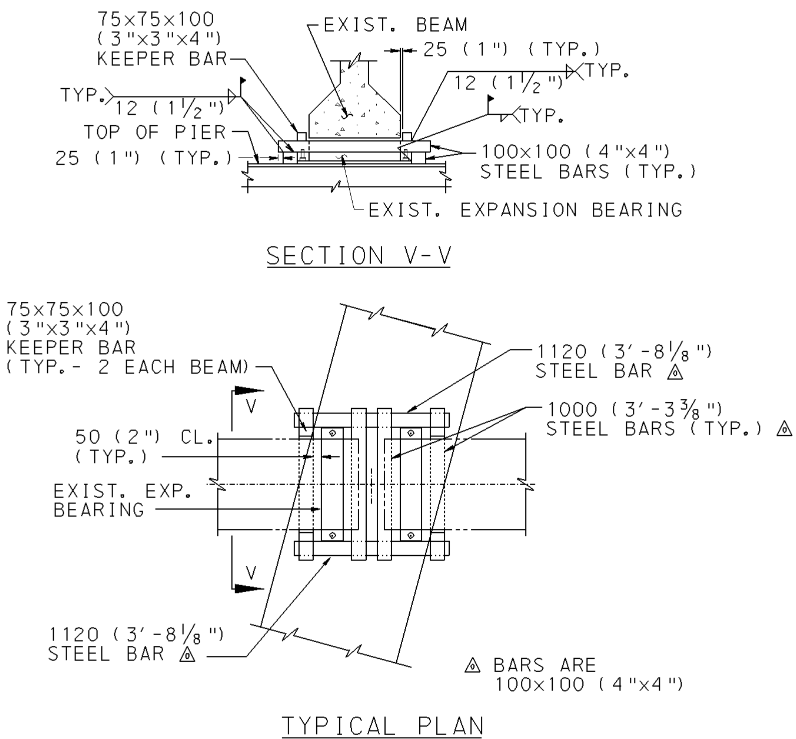


Figure 5.5.2.8.2-4 - Example of Cribbing for Bearings

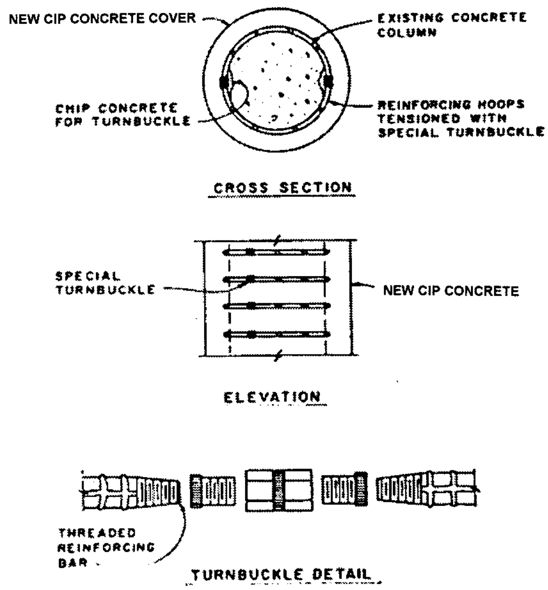


Figure 5.5.2.8.2-5 - Reinforced Concrete Column Retrofit-1

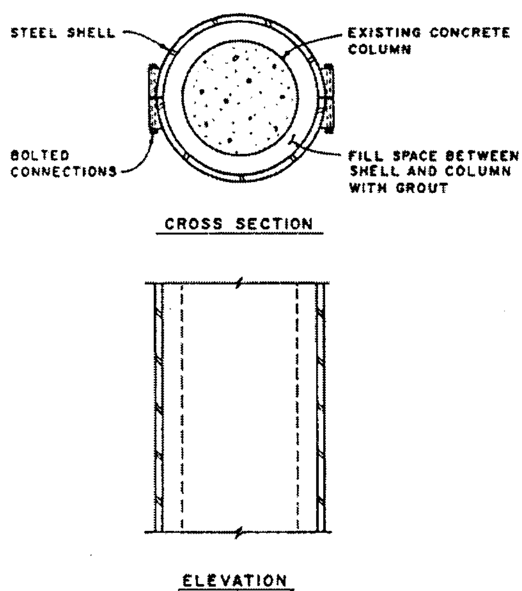


Figure 5.5.2.8.2-6 - Reinforced Concrete Column Retrofit-2

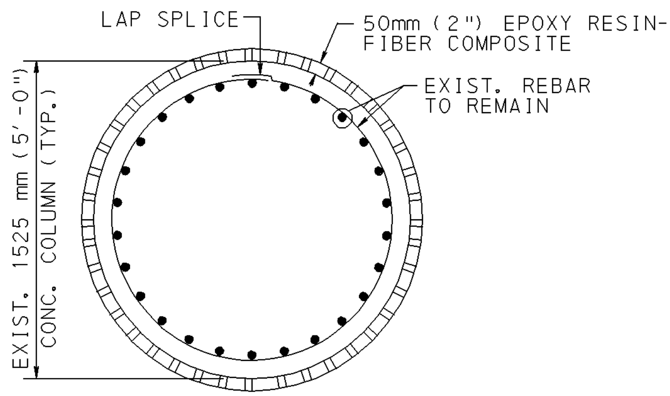


Figure 5.5.2.8.2-7 - Reinforced Concrete Column Retrofit-3

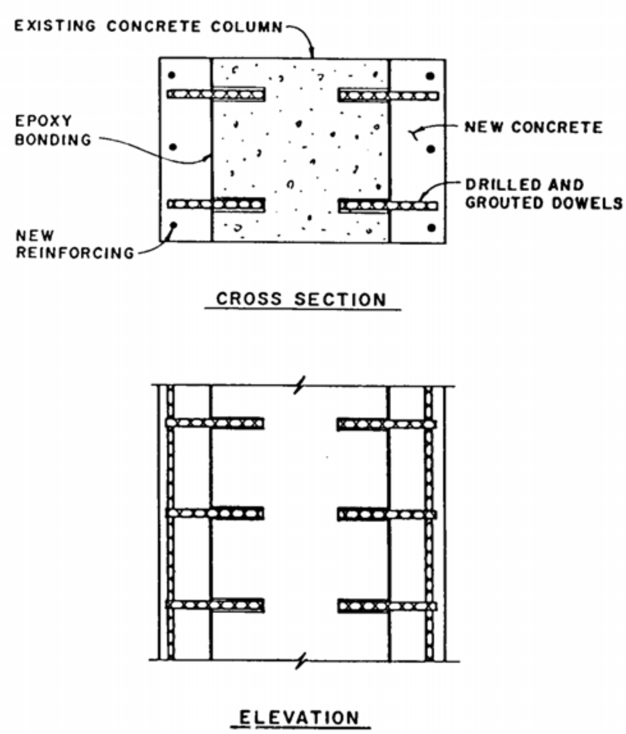


Figure 5.5.2.8.2-8 - Reinforced Concrete Column Retrofit-4

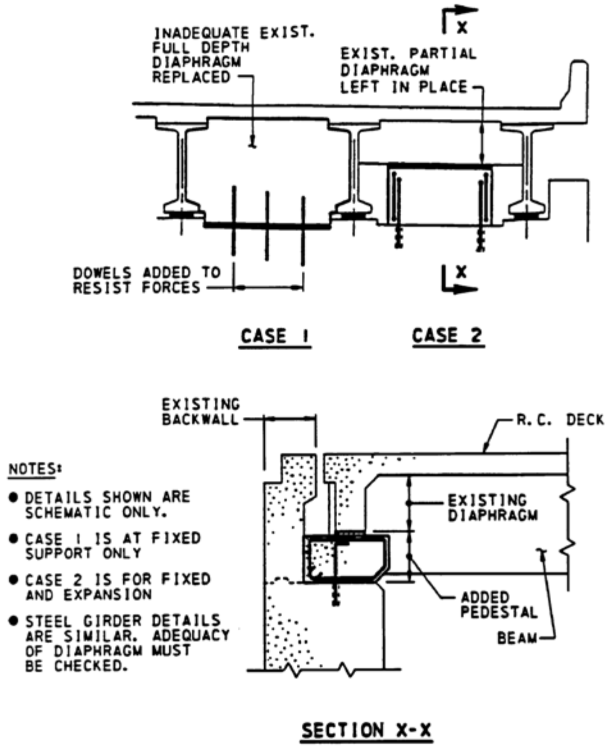


Figure 5.5.2.8.2-9 - Retrofit Using Shear Blocks or Additional Dowel Bars

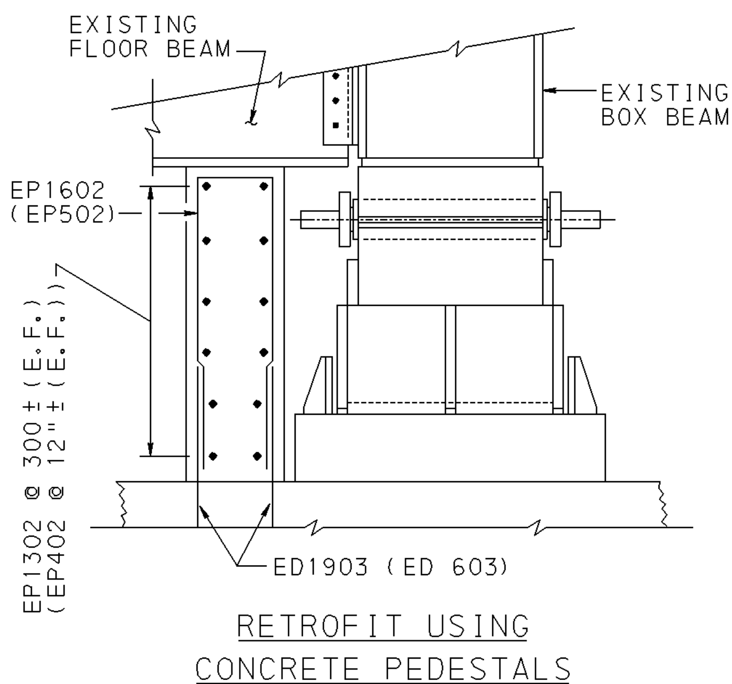


Figure 5.5.2.8.2-10 - Retrofit Using Concrete Pedestals

5.5.2.8.3 Uncommon Retrofit Concepts

The following concepts are considered to be valid retrofit concepts; however, these concepts are not to be considered as a first choice. Approval of the Chief Bridge Engineer is required before these concepts are included on any project.

1. Cable Restrainers - PennDOT has used a few cable restrainers. Cable retrofits are not typically recommended since more workable alternate concepts are available. Because standard cable retrofit details and special provisions have not been developed, they should not be considered as first choice at this time. See Figures 1, 2 and 3.

   [Note that this should not be construed to indicate that the Department is not satisfied with the documented performance of cable restrainers. Moreover, this only represents that seismic retrofits in Pennsylvania are typically done on major rehabilitation projects where seismic rehabilitation needs can be specifically addressed and cable restrainers may not be needed.]

2. Footing Retrofits - The need for this type of retrofit should be identified; however, the Chief Bridge Engineer will make the final decision regarding inclusion in final design. See Figure 4.

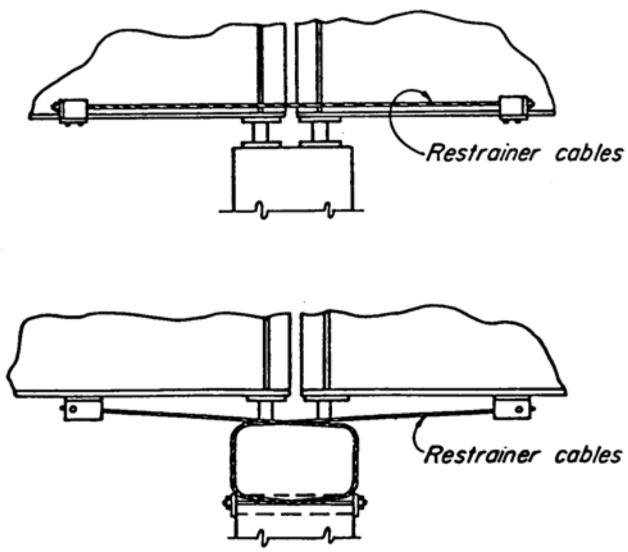


Figure 5.5.2.8.3-1 - Typical Cable Restrainer

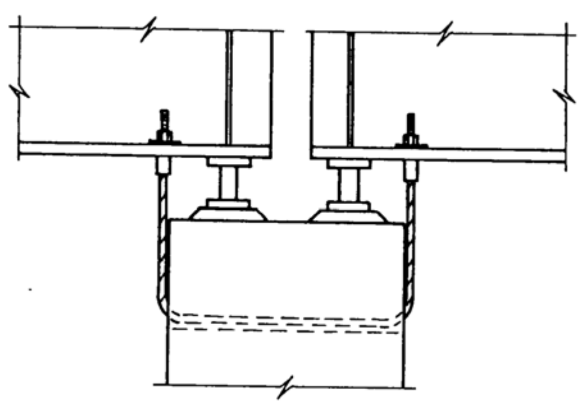


Figure 5.5.2.8.3-2 - Typical Cable Restrainer for Uplift

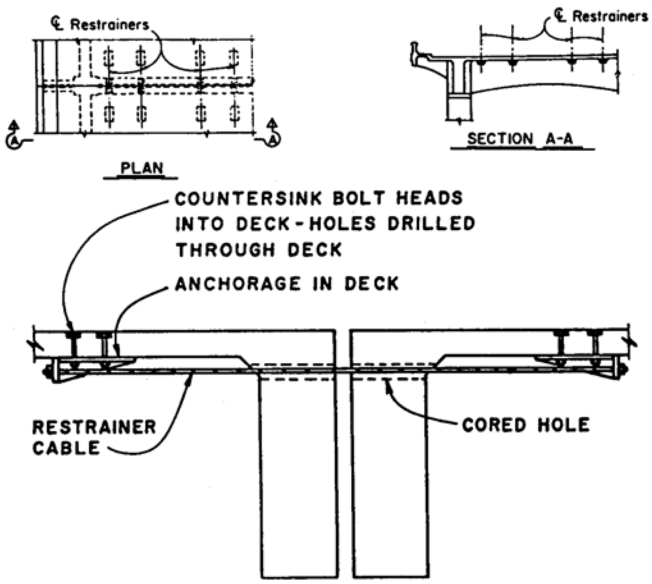


Figure 5.5.2.8.3-3 - Expansion Joint Restrainer

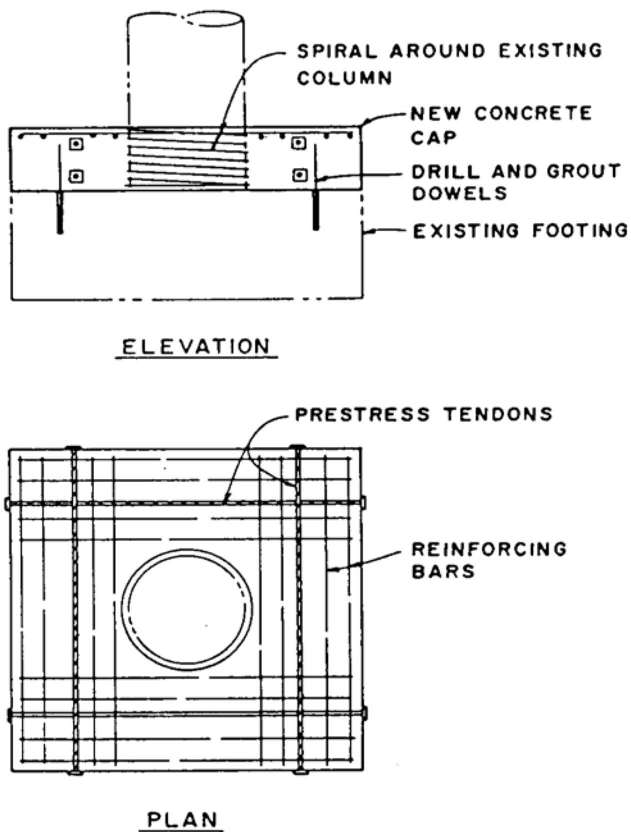


Figure 5.5.2.8.3-4 - Footing Retrofit

5.5.2.9 BRIDGE APPROACH SLAB AND PAVEMENT RELIEF JOINTS

Replace or rehabilitate the bridge approach slab to help minimize the impact loading on the bridge.

Service the existing pavement relief joint as needed to ensure a smooth ride.

Provide a pavement relief joint for all concrete pavement types, if one has not been provided previously, except between two bridges which are less than 300 m {1,000 ft.} apart, or when expansion length of the concrete pavement is less than 300 m {1,000 ft.} apart.

5.5.2.10 FLARED SAFETY WINGS

Flared safety wings are eligible work under the IPM criteria.

5.5.3 Construction Related Items

5.5.3.1 MAINTENANCE AND PROTECTION OF TRAFFIC

When an existing bridge is to be replaced or is to undergo major rehabilitation, the decision of whether to maintain traffic in the proximity of the existing bridge or to detour traffic must be made. This decision is based upon consideration of many conditions, including engineering feasibility, cost effectiveness, ADT/truck traffic, impact on local economy and emergency services, environmental impact and obtaining right-of-way.

The options for maintenance of traffic (including non-motorized modes as applicable) for bridge projects are to be evaluated and the decision is to be made during the Preliminary Design/Environmental study stage. Adequate public coordination must be performed in order to minimize adverse impact. If a temporary bridge is recommended, Program Management Committee (PMC) approval must be obtained prior to making a public commitment. A decision to use a temporary bridge at <u>any</u> time during the project is subject to PMC approval.

If, after completion of the Preliminary Design/Environmental Phase, a decision is made to use a maintenance of traffic option that was not reflected in the original environmental evaluation process, re-evaluation of the environmental impacts may be necessary.

Generally, maintenance and protection of traffic will be based on the following hierarchy of options. Refer to current Department policy for a complete discussion of these options.

1. Detour

2. Half Width Construction

3. New Bridge Adjoining the Existing Bridge (use existing bridge for maintenance of traffic)

4. Temporary Stream Crossing and Approaches (stream crossing using multiple pipes/fill material)

5. Temporary Bridge and Approaches (Temporary Bridge Structure)

Off-peak traffic hours construction schedule and/or incentive/disincentive clauses may be prescribed for bridges carrying extremely high traffic volume. Precast concrete deck elements with longitudinal post-tensioning or prefabricated steel decks may be utilized if warranted and approved by the Chief Bridge Engineer.

Traffic maintenance and related bridge construction items shall be depicted on plans. If temporary barriers are utilized, locate the temporary barrier on the structure plan. Based on the construction traffic patterns, select the barrier type from Standard Drawing BC-719M, and use the barrier to deck connections as shown on Standard Drawing BC-719M for the type selected.

5.5.3.2 ENVIRONMENTAL RELATED ITEMS

Requirements for paint removal, containment and disposal of contaminants shall be incorporated as per the current Department's policy.

Generally, it is the Department's policy not to construct sound abatement walls for existing highways. The need for bridge-mounted sound walls shall be determined in conjunction with the need for sound walls on the adjoining roadway.

The proposed construction should neither damage an existing wetland nor adversely affect the historical significance of the bridge itself or its surroundings, except as permitted through the environmental evaluation process.

5.5.3.3 CONSTRUCTIBILITY AND STRUCTURAL STABILITY

Constructibility shall include, but not be limited to:

- Material availability at reasonable cost
- Fabrication and erection requirements
- Site accessibility and material transportability
- Erection feasibility
- Construction risk
- Effect of the selected construction alternate on the project
- Construction sequencing of different operations
- Environmental impact of proposed construction method (including lead based paint issues)
- Impact on construction schedule

Each of the above items shall be evaluated to ensure constructibility and to minimize or eliminate "surprises" during construction.

For redecking projects, particularly when the new deck overhang may be larger than the existing overhang, structural stability of the fascia girder shall be evaluated using current design criteria.

For jacking requirements refer to PP5.5.2.6(e). If traffic is to be maintained on the temporarily jacked superstructure, the stability of the jacks or temporary bearing must be ensured by using needed restrainers and redundancy. Extra longitudinal and transverse forces due to traffic and other forces shall be shown on the contract plans for Contractor's use in proposing an alternate.

5.5.3.4 TEMPORARY BRIDGES

If the need for a temporary bridge is established, each temporary road shall be designed to be compatible with the existing site conditions, volume and an acceptable operating speed for the temporary condition. Engineering judgment is to be used with these guidelines.

5.5.3.4.1 Definitions

For the purpose of these guidelines, the following definitions apply:

(a) Temporary Stream Crossing

A temporary crossing of a stream with multiple pipes, pipe arches or similar conduits covered with fill material.

(b) Temporary Bridge

A temporary crossing of a stream or other topographic feature consisting of a bridge superstructure with an appropriate substructure.

(c) Temporary Road

A temporary roadway forming the approaches to a temporary stream crossing or temporary bridge.

5.5.3.4.2 Temporary Road, Traffic Control, etc.

For a temporary road, geometry and composition, traffic controls, permit requirements, environmental policies and other related requirements, refer to current Department guidelines.

5.5.3.4.3 Temporary Bridge Design Guidelines

Temporary bridges for public use will be designed using current Department bridge design methodology including consideration of geometric constraints (truck turning patterns, etc.) as per DM-2.

Temporary bridges can be specified to be constructed and removed by the Contractor or to be leased proprietary temporary structures if they meet the design requirements.

Normally, temporary bridges are a short-term installation, meant to be in use for a period ranging from a few months to two years. If a temporary bridge is being considered to be in place for more than two years, approval of the design by the Bureau of

Design, BQAD must be obtained.

(a) Bridge Width

The minimum recommended clear width between curbs or bridges railing is:

- 4300 mm {14 ft.} for single lane bridge
  (A minimum width of 5500 mm {18 ft.} may be required for farm equipment)

- 7300 mm {24 ft.} for two-lane bridge

Bridge clear widths should not be less than the combined width of the temporary roadway and shoulders.

If proprietary temporary structures are specified (with special approval from PMC), clear bridge widths slightly less than those recommended above may be used.

(b) Bridge Railing

Bridge railing may consist of single face concrete F-shape barrier, or other crash tested and approved railing system appropriate to the roadway (see DM-2, Chapter 12.9). Under most conditions, the concrete F-shape barrier provides the highest and least costly level of protection. Therefore, the concrete F-shape barriers are generally the preferred alternative. If the temporary bridge could cause hydraulics problems during flooding conditions, the use of an open metal railing (such as the PA 10M) is encouraged to minimize restrictions to water flow during high water episodes.

The connections between the bridge railing and any guide rail on the approaches is to be smooth and of adequate strength so that no "pockets" will be created if impacted by vehicles.

(c) Minimum Design Loads

The following are considered minimum design loads. If a temporary bridge must carry construction equipment, the appropriate loads must be considered in the design.

(1) PHL-93 at Strength IA limit state

- If the bridge is expected to carry heavy truck traffic and/or heavier loads, it must be designed for heavier loads (such as P-82 permit load at Strength II limit state).

- Use load factor of 1.35 for Contractor's heavy equipment.

- The Contractor shall be required to re-analyze the structure to ensure safety using these load factors if the bridge is to be used for construction equipment. Include a special provision in the PS&E requiring structural analysis to ensure prescribed factor of safety and bridge strength, if warranted.

- The temporary bridge must be posted in accordance with the current posting policy.

(2) No seismic loads.

(3) Other loads (wind, ice, etc.) as per LRFD or DM-4.

(4) Allowable live load deflection of L/500.

(5) Debris loading shall be as per A3.7.3.1. A Q_{10} flood will be used to calculate forces on the bridge.

(d) Substructure Design

The load factors for substructure design shall be multiplied by 0.90.

(e) Proprietary Temporary Bridge-Deck Surface

If proprietary temporary bridges are an option, specify that the bridge deck is to be coated to provide acceptable skid resistance (treated timber or steel plate decks do not provide acceptable skid resistance).

(f) Waterway Opening

A Q_{10} flood will normally be adequate. If warranted by site conditions and engineering analysis, the recurrence interval can be reduced (never lower than $Q_{2.33}$) or increased.

(g) Scour Protection

Scour protection will be provided in accordance with the scour analysis. Use Q_{25} velocity to determine scour depth.

Include a special provision in the PS&E requiring the Contractor to close the bridge during high water. In the special provision, define high water as a specific water surface elevation. This water surface elevation will normally be based on a Q_{10} flood, but can be based on a larger flood (not to exceed Q_{20} if the designer feels the superstructure can safely tolerate the forces imposed by high water). Specify that the bridge may be reopened after inspection and adequate mitigation measures are taken (if warranted), and the bridge is determined to be structurally sound.

5.5.4 Targeted Service Life for Rehabilitated Bridges and Superstructures

(a) Estimated Deck Service Life

(1) Terminal decks (condition rating 3 or less) with minor patching and bituminous overlay...... 2 to 5 years.

(2) Deck to remain in place with protective measures:..........20 years for deck

- Membrane waterproofing and bituminous overlay. The life of the bituminous overlay may be 6 to 8 years. The membrane may need to be replaced each time the overlay is replaced.

(3) Latex modified concrete overlays, cathodic protection and rehabilitation of other deck types........20 years

(4) New concrete deck with epoxy-coated reinforcement........40 to 50 years

(b) Expansion Dams

Same as deck - periodic replacement of glands or trough should be expected.

(c) Beam end repairs and/or rehabilitation

Minimum: Same as deck
Desirable: 50 years

(d) Substructure repairs and rehabilitation

Minimum: Same as deck
Desirable: 50 years

(e) Repair and/or rehabilitation of other superstructure types and their elements

Minimum: Same as deck
Desirable: 50 years

(f) Bearings

Same as beams

(g) New superstructure

Minimum: 50 years
Desirable: 100 years

(h) Substructure rehabilitation

Same as superstructure

(i) Retaining Walls

Minimum: 25 years
Desirable: 50 years

(j) Culverts

Minimum: 15 years
Desirable: 50 years

New extension
Minimum: 50 years
Desirable: 100 years

(k) Sign Structures

Minimum: 25 years
Desirable: 50 years

(l) Ground-Mounted Sound Barriers

Minimum: 15 years
Desirable: 40 years

(m) Structure-Mounted Sound Barriers

Same as deck

(n) Temporary Bridges

3 to 5 years

5.5.5 Load Carrying Capacity

5.5.5.1a SUPERSTRUCTURES

Minimum load carrying capacity for all rehabilitated bridges shall be same as for a new design using LRFD/DM-4 method. Analysis should include 150 kg/m^2 {0.030 ksf} for future wearing surface. A latex overlay is not considered structurally effective. Special approval of the Chief Bridge Engineer is required for any deviation.

Figure 1 shall be used for decision making for deck replacement projects.

If existing beams do not adequately rate for shear using the current LRFD shear criteria, the beams should be rated using the criteria used for the original design. A note on the rating table should indicate which criteria was used in determining the shear rating. Any design errors in the original design should be brought to the attention of the Chief Bridge Engineer.

All superstructure components must be checked for the remaining fatigue life. The remaining fatigue life must be at least the expected life of the type of rehabilitation being considered.

5.5.5.1b SUBSTRUCTURES

Substructures should be analyzed for adequacy for the following conditions:

1. Superstructure Replacement
2. Change in Bearing Fixities
3. Evidence of Substructure Distress

The desirable load carrying capacity of substructures for rehabilitated bridges shall be HS-25. The minimum load carrying capacity is HS-20.

AASHTO criteria supplemented by DM-4 shall be used for the analysis.

Where the analysis of concrete substructures shows overstress using current AASHTO LRFD Bridge Design Specifications, the criteria used during the original design may be used. For shear analyses using working stress methods, do not exceed the stress limitations contained in the 1973 AASHTO Standard Specifications. Where no transverse reinforcement is provided (except footings), the current AASHTO Specifications for shear must be used.

Foundation bearing resistance should be determined based on available test boring data. The need for a detailed foundation investigation including the drilling of borings should be determined considering existing conditions, the size and complexity of the structure, and the extent of the proposed rehabilitation. If no boring data is available and no borings are planned for the project, as a minimum an assessment of the adequacy of the foundations to sustain the bridge in its rehabilitated condition should be made based on known substructure information and local geologic data.

Where end or point bearing piles are present, the pile resistance may be based on current design criteria. For friction piles, a static analysis should be performed using the available soil and pile data. In cases where the actual pile length or soil data is not known, an assessment should be made based on existing conditions and available data.

Deviations to this policy must be approved by the Chief Bridge Engineer.

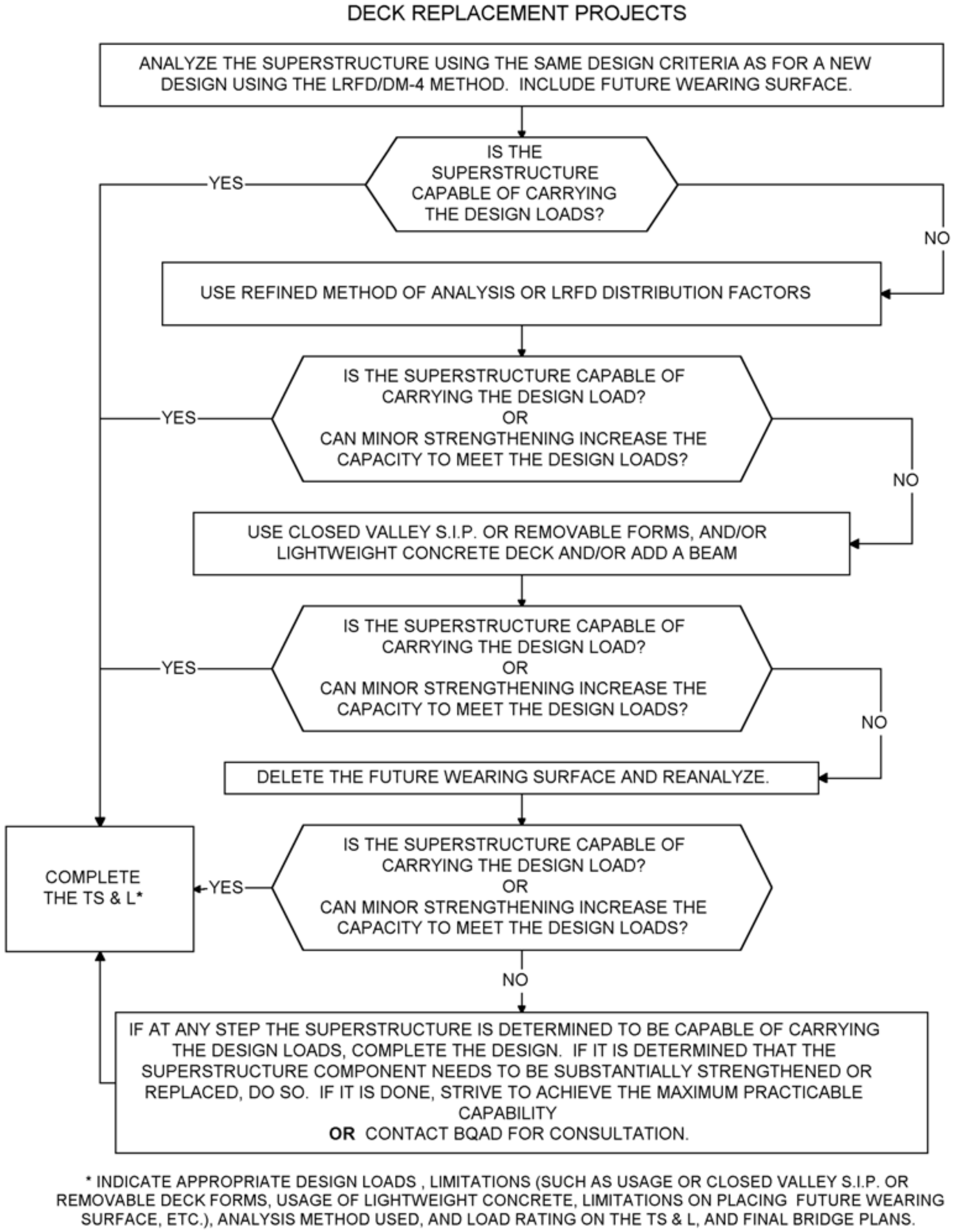


Figure 5.5.5-1 - Load Carrying Capacity for Deck Replacement Project

5.5.6 Plan Preparation and Presentation

For plan presentation follow PP1.9.6.2. The existing plans for rehabilitation projects may be obtained from the contract sales store. When developing bridge rehabilitation plans, all pertinent details should be shown on the contract drawings. In addition, the District should make the existing bridge plans available to the contractors during the bidding stage. Refer to PP1.3.2 for type of details needed for the existing bridge proposed to be rehabilitated. The pertinent notes from PP1.7.4.2 shall be shown on the proposed contract plans.

In plan preparation, actual field measurements should be considered more reliable than the drawings, and the shop drawings should be considered more reliable than existing bridge plans.

The contract plans shall be sufficiently detailed to provide an overall view of the bridge indicating the existing and proposed geometric dimensions, limitations and restrictions, extent and type of work to be performed, construction stages, material information, and all related information needed to rehabilitate the bridge. Pay limits, quantities and pay items should be adequately defined to eliminate ambiguity or confusion. All work shall be accounted for by specific pay items and no work shall be hidden under "incidental" to other work item(s) unless for extremely minor work. The "incidental" work should be the exception rather than the rule.

Where applicable, reasons for critical limitations and restrictions should be explained to assist the Contractor and the field inspector in adjusting to the field conditions.

For submittal requirements and approval responsibilities, refer to PP1.9.

For rehabilitation projects, the TS&L plans shall have the normal TS&L plan details, plus a complete scope and extent of work, and the anticipated bridge rating after the proposed work is incorporated.

5.5.7 Retaining of Existing Beams by the Department

For bridge or superstructure replacement projects, FHWA has no objection to the Department retaining the existing beams if it so desires. Previous procedures of salvage value creditation has been discontinued. In fact, FHWA would encourage the Department to retain the existing beams whenever it would serve a useful purpose.

5.5.8 Bridge Rehabilitation Projects Requiring Bridge Painting or Superstructure Jacking

From a review of some project lettings, it is evident that, instead of cleaning and painting of the existing steel beams containing lead based paint, their replacement is sometimes cost effective. The cost of lead based paint removal and disposal is constantly increasing. To provide cost effectiveness, the following guidelines are provided for bridge rehabilitation projects involving bridge painting:

1. For projects requiring bridge painting, refer to PP5.5.2.6(b).

2. If the existing bridge paint contains lead or toxic materials, it must be indicated, either on the plans or in the special provision, to alert the contractor. If you are not sure of the lead or toxic material content in the paint system, take a few samples and send them to the Bureau of Construction, Materials and Testing Division, for analysis. Based upon the test results, inform the prospective bidders, through bidding documents, whether or not lead is present in the paint.

For bridge rehabilitation projects where superstructure jacking is required, at least one constructible scheme must be shown in the contract documents. All related analyses, including the effects of the jacking on all connections and superstructure and (rarely) substructure elements, must be performed. Consider the following jacking design guidelines:

1. Bridge deck should be closed to traffic.

2. Do not include L + I loads to design jacking force requirements. Assume bridge is closed to traffic until jacking is done and bearings are completed.

3. If shims and blocks are used for temporary supports under traffic, their design must include L + I.

If strengthening is required, all details must be shown in the contract documents. A contractor may be allowed to submit alternate schemes through a special provision or notes on contract drawings.

5.6 BRIDGE PRESERVATION AND PREVENTIVE MAINTENANCE

The 2005 Safe, Accountable, Flexible, Efficient Transportation Equity Act (SAFETEA-LU) amended Title 23 United States Code Section 144 “Highway Bridge Program” (HBP) enabling systematic bridge preservation activities to be funded from the HBP. FHWA has determined that this HBP funding may also be used for systematic bridge preventative maintenance.

HBP funds may be used for these activities on any NBIS bridge. Other Federal funds may be used for these activities on NBIS and non-NBIS length bridges carrying roadways with functional classifications that are eligible for the specific fund category, but shall not be performed on bridges carrying local roads or rural minor collectors.

To obtain Federal funding the project must be competitively bid through ECMS.

5.6.1 Bridge Preservation

Eligible work items for bridge preservation are as follows:

1. Scour Countermeasures: Scour countermeasures including underpinning, riprap placement, streambed paving, etc. properly designed for predicted scour.

2. Expansion dams: Repair, replace or install new expansion dams to ensure leakproof joints. Where economically feasible, eliminate the deck joints. Repairs to deck drainage or downspouting may also be included. Replacement of seals is also permitted, provided other items, if any, relative to leakage are also addressed.

3. Beam end repairs and restoration: Restore steel, concrete or P/S concrete beam-ends to extend their service life.

4. Fatigue and Fracture Retrofits: Retrofits or repairs to fatigue-prone details of steel bridges.

5. Bridge bearings and supports: Restore or replace the existing bearings to make them functional and repair or rehabilitate substructure units to extend their service life. If bearings are replaced, they must meet seismic requirements. However, no seismic analysis is to be performed.

6. Spot/Zone painting: Spot/zone painting can be used as a stand-alone measure or with other steel repair items. Preservation of zinc-rich paint systems should be considered. Cleaning and waste disposal is included in this item. Spot/Zone painting to be completed in accordance with Pub 408, Section 1071.

7. Deck restoration and overlays: Concrete deck patching (Repair Types I, II, or III) and waterproofing overlays (i.e., latex concrete, bituminous with membrane) needed to extend deck life and improve rideability are eligible. Full deck replacements are not eligible. Bituminous deck patching alone is not eligible.

8. Painting: Full overcoats or complete repainting, with cleaning, waste disposal, and steel repairs in accordance with Pub 408, Section 1070.

9. Approach slabs: Repair the approach slab as necessary where the condition of the approach slab is affecting the performance of the bridge. Where practical and needed, repair or replace approach slabs, pavement relief joints, and other high spots adjacent to bridge to restore functionality and/or improve rideability.

10. Where practical, bridge preservation projects in close proximity should be grouped together to economize traffic control and other incidental costs. Bridges within limits of other highway work should be evaluated for opportunities for simultaneous bridge preservation work

11. Other bridge preservation items not mentioned in the above categories may be included, but must be properly justified.

12. Safety items such as bridge parapet replacement are not eligible work items for preservation. However, safety improvements funded using other than bridge preservation funds may be included in such projects to take advantage of traffic control and other incidental project

Once preservation activities are completed at a candidate bridge, this structure should not be revisited for rehabilitation or preservation for 10 years, except for routine maintenance activities.

5.6.2 Bridge Painting Guidelines

The corrosion of structural steel bridge members is an ongoing concern that must be addressed to prolong the life of the bridges in Pennsylvania. Not only does corrosion change the aesthetics of the bridge, it can seriously jeopardize the structural integrity of the entire structure. An efficient and economical method to provide corrosion protection to existing steel bridge members is painting.

The Bridge Painting Guidelines were developed to provide a baseline for programming painting projects to extend the life of steel bridges. The Bridge Painting Guidelines are primarily intended for use on bridges greater than 100 feet long. Packaging multiple bridges into one contract for structures less than 100 feet may be appropriate. For smaller bridges, the proportionally higher cost of environmental controls for cleaning may outweigh the benefits of painting. For larger bridges (in excess of 500 feet) or complex bridges, paint preservation should be prioritized due to the high replacement cost of the bridge.

Bridge painting is weather sensitive. The air temperature must be warm and the humidity must be low. Therefore, work/letting need to be scheduled when there is low probability of inappropriate weather conditions. Typically, May through September is the ideal time to accomplish bridge painting. If a painting project occurs outside of this range, a controlled environment is required in accordance with Pub 408 section 1070.

Painting projects should be coordinated with roadway projects, especially on Business Plan Networks (BPN) 1 and 2. The necessary time for design and analysis of a containment system by a professional engineer registered in the state of Pennsylvania should be included in the project schedule between the notice to proceed and the physical start of work. Also, consider the necessary time required for the industrial hygienist/certified professional to develop/review the lead safety plan and other submittals. A typical lead removal painting project cost is in the range of $14-18/SF (includes containment, lead removal, disposal, and repainting).

When repainting existing bridges over high ADT roadways where roadway restrictions must be minimized, use of a rapid deployment strategy should be considered. Rapid deployment is a viable option primarily designed for use on these highway overpasses where the structural steel is easily accessible from the roadway below using a mobile work platform. This mobile work unit includes a containment device, dust collector, and blast equipment. Rapid Deployment methodologies may be specified using Special Provisions. For field painting activities, use a three-coat system with an organic primer as per Pub 408, Section 1070. At this time, two-coat paint systems are not approved. Further research is necessary for single-coat systems.

5.6.2.1 ZINC RICH PAINT SYSTEMS

For a properly shop-installed zinc rich paint system, the following painting activities and frequencies are general rules of thumb to establish painting guidelines to maintain and preserve the life of the steel bridges in Pennsylvania. Wide spread use of these zinc rich paint systems began in the 1980's. Environmental factors (e.g., under a leaking deck joint, within "splash zone", exposed to salt spray) will have a detrimental effect on the life of the paint system which will require an increased frequency of painting activities. Correct leaking deck joints and other bridge deficiencies affecting paint system performance prior to completing any new painting activities. Consideration must also be given to bridges that are on a program to be improved, rehabilitated or replaced. Bridges on a program must be evaluated to determine if a painting activity is still warranted. Due to the high cost of containment and mobilization, a cost/feasibility estimate must be completed to determine the most economic work scope for any given structure. (i.e., Use of spot/zone painting versus a full re-paint for any given structure or entire component replacement must be evaluated.) This work scope should include aesthetic considerations for the visible portions of the bridge, such as fascia beams. An overcoat painting activity is generally not an economically viable option for modern paint systems.

Painting Activity	Frequency
Spot/Zone Painting	10-18 years
Full Re-paint	30-40 years

Note: Maintaining the paint system on a bridge may require a series of spot/zone painting activities throughout the life of the paint system.

5.6.2.2 LEAD BASED AND NON-ZINC RICH PAINT SYSTEMS

The flow chart in Figure 5.6.2-1 is intended to assist in the selection of the painting activity for various paint condition ratings. For each activity, the candidate bridges must be prioritized and programmed. Funding level is an important consideration. For a description of the paint condition ratings, refer to Publication 100A, Bridge Management System 2 (BMS2) Coding Manual.

Some of the terminology used in Figure 5.6.2-1 is as follows:

<u>6B36</u> – BMS2 Item for Paint Condition Rating.

<u>Criticality Evaluation</u> – This analysis/review is intended to determine if bridge painting is needed presently to preserve the bridge until the bridge is rehabilitated or replaced. This is primarily a structural safety issue.

<u>Rehab / Replace</u> – This option refers to the bridge being rehabilitated or replaced under a funding program, such as Billion Dollar Bridge, Betterment, I-4R, etc. This implies that the decision to paint or replace the steel will be included and implemented as part of that project.

<u>Full Re-paint</u> – This option involves total removal/cleaning and repainting of the entire bridge with a new zinc rich system.

<u>Overcoat</u> – This option involves the application of intermediate and/or top coats over existing paint with minimal removal of old paint. Compatibility patches are required at least one year in advance to determine the suitability of the proposed paint system.

<u>Spot / Zone</u> – This option is the re-painting of specific parts of the bridge, such as splash zones, near expansion dams, etc. Paint removal and cleaning will be thorough in those areas.

<u>Do Nothing</u> – No painting at this time.

Practicable – The term practicable means available and capable of being done after taking into consideration cost, existing technology, and logistics in light of overall project purposes. There may also be considerations (e.g., structural safety, historical preservation) that could over-ride pure economics.

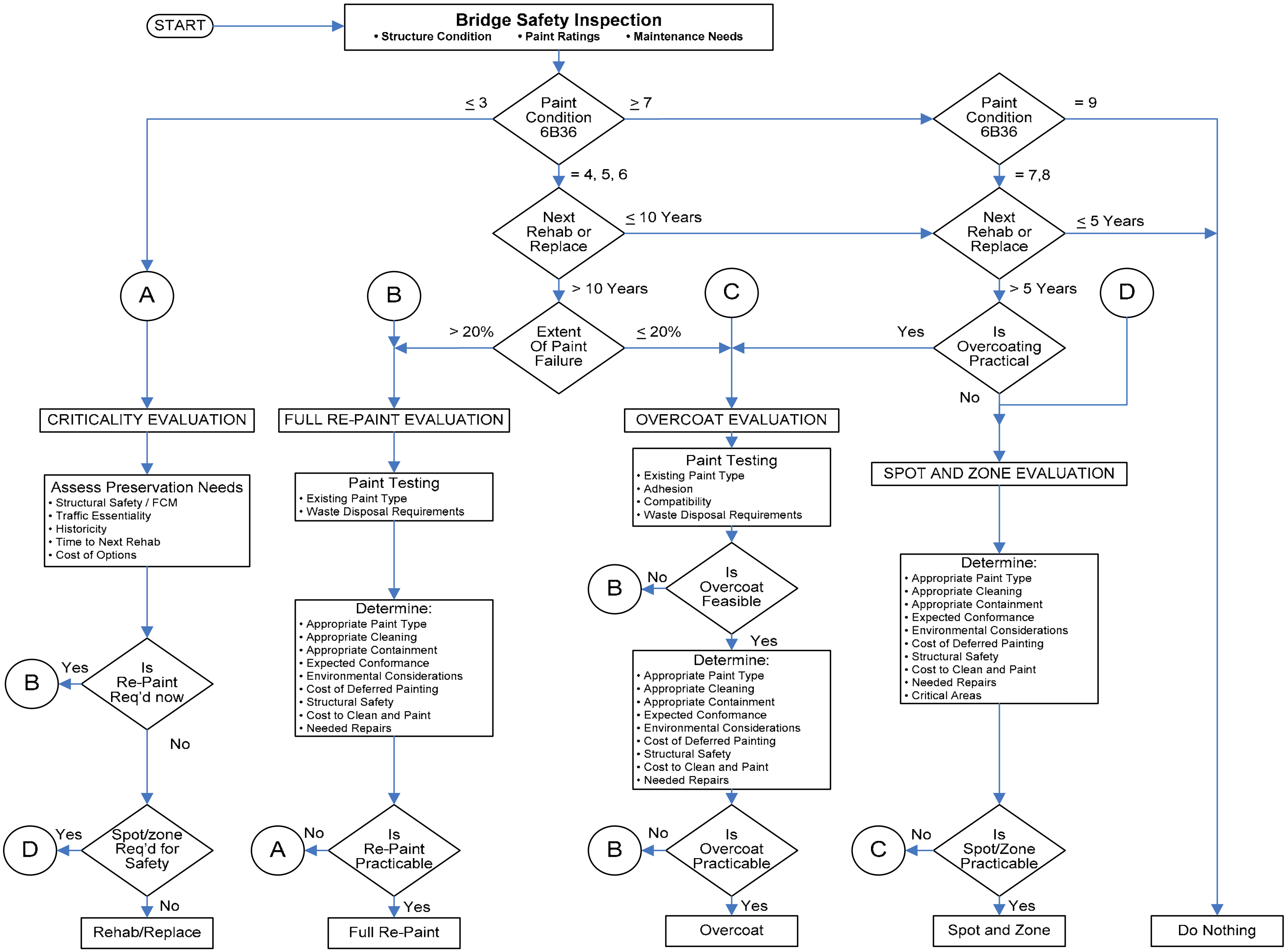

START
Bridge Safety Inspection
• Structure Condition
• Paint Ratings
• Maintenance Needs
≤ 3
Paint Condition 6B36
≥ 7
= 4, 5, 6
Paint Condition 6B36
= 9
= 7,8
Next Rehab or Replace
≤ 10 Years
> 10 Years
Next Rehab or Replace
≤ 5 Years
> 5 Years
A
B
C
D
> 20%
Extent Of Paint Failure
≤ 20%
Yes
Is Overcoating Practical
No
CRITICALITY EVALUATION
FULL RE-PAINT EVALUATION
OVERCOAT EVALUATION
SPOT AND ZONE EVALUATION
Assess Preservation Needs
• Structural Safety / FCM
• Traffic Essentiality
• Historicity
• Time to Next Rehab
• Cost of Options
Paint Testing
• Existing Paint Type
• Waste Disposal Requirements
Paint Testing
• Existing Paint Type
• Adhesion
• Compatibility
• Waste Disposal Requirements
Determine:
• Appropriate Paint Type
• Appropriate Cleaning
• Appropriate Containment
• Expected Conformance
• Environmental Considerations
• Cost of Deferred Painting
• Structural Safety
• Cost to Clean and Paint
• Needed Repairs
• Critical Areas
B
No
Is Overcoat Feasible
Yes
Determine:
• Appropriate Paint Type
• Appropriate Cleaning
• Appropriate Containment
• Expected Conformance
• Environmental Considerations
• Cost of Deferred Painting
• Structural Safety
• Cost to Clean and Paint
• Needed Repairs
Determine:
• Appropriate Paint Type
• Appropriate Cleaning
• Appropriate Containment
• Expected Conformance
• Environmental Considerations
• Cost of Deferred Painting
• Structural Safety
• Cost to Clean and Paint
• Needed Repairs
B
Yes
Is Re-Paint Req'd now
No
D
Yes
Spot/zone Req'd for Safety
No
Rehab/Replace
A
No
Is Re-Paint Practicable
Yes
Full Re-Paint
B
No
Is Overcoat Practicable
Yes
Overcoat
C
No
Is Spot/Zone Practicable
Yes
Spot and Zone
Do Nothing

Figure 5.6.2-1

5.6.3 Bridge Preventive Maintenance

5.6.3.1 BRIDGE WASHING

The existing Title 23 Section 116 "Maintenance" prescribes that a preventive maintenance activity shall be eligible for Federal assistance if it is demonstrated that the activity is a cost-effective means of extending useful life. Therefore, preventive maintenance activities shall be demonstrated as cost-effective and shall be performed using a systematic process.

PennDOT has been performing bridge washing and cleaning for many years because it is a cost effective means to preserve our bridges and extend useful life. Bridge washing has been demonstrated as cost effective by New York State. Their bridges are exposed to similar environmental conditions as Pennsylvania's including application of deicing salts. A report prepared by NYSDOT demonstrated a decline in deterioration rate from conducting the core preventive maintenance activities washing, joint repair and concrete sealing. A 40 percent decline in deterioration rate over a nine year period was denoted. The FHWA's 2000 "Survey Results of Bridge Maintenance Practices in the Midwest" demonstrates that seventeen Midwestern States routinely perform washing because they believe it is cost-effective.

PennDOT's methodology, systematic process, for selecting washing and cleaning candidates entails reviewing the maintenance screens in our Bridge Management System (BMS2) and reviewing inspection reports. Inspections are conducted on a maximum 24 month cycle and condition and maintenance needs updated. The inspection reports typically have a section that is dedicated to identifying specific detailed bridge washing needs. Eligible bridge washing and cleaning activities are identified in Table 5.6.3.1.1-1.

5.6.3.1.1 Bridge Washing Systematic Process

A systematic process shall be used to select bridges and their elements where washing and cleaning is performed as preventive maintenance.

Bridge washing and cleaning shall be performed on a specified cyclical basis or as-identified. Regardless if performed cyclically or as-identified, it shall be a consistent and continual program applied to the full inventory or a specified subset of the inventory. The effectiveness of this continual program shall be demonstrated by performance goals and measurements.

Cyclical activities will best occur following the cold weather deicing season. This is also an appropriate time as it improves surface cleanliness thereby decreasing safety inspection effort. As-identified (i.e. non-cyclical) activities shall occur based on findings discovered during inspection or maintenance activities or findings contained in inspection or maintenance reports.

Bridges programmed for complete replacement or superstructure replacement should be excluded from washing and cleaning unless necessary to prevent deterioration or debris/rust buildup from further affecting safe load capacity or functionality.

Table 5.6.3.1.1-1 - Eligible Washing and Cleaning Activities

Eligible Washing and Cleaning Activities				
Bridge Element	Element Location	Frequency	Benefit	Limitations
Deck surface	Shoulder or gutter line	Cyclical or as need identified	Prevent scupper/downspout blockages which contribute to system failure directing water & debris to deck joint or beam beneath scupper	Washing full deck width generally not cost effective especially on large widths given there is less permanent debris & contaminants in travel way
Deck joint	Top of deck joint	Cyclical or as need identified	Prevent premature joint failure from debris compaction &/or obstruction of movement	Strip seals, compression seals, modulars, sliding plates
Deck joint trough	Beneath deck joint	Cyclical or as need identified	Prevent premature trough failure from debris accumulation	Generally not needed if self cleaning performance demonstrated

Table 5.6.3.1.1-1 - Eligible Washing and Cleaning Activities (Continued)

Eligible Washing and Cleaning Activities				
Bridge Element	Element Location	Frequency	Benefit	Limitations
Superstructure elements beneath deck joint	Beneath deck joint	Cyclical or as need identified	Prevent corrosion or paint failure from visible & non-visible contaminants; necessary because deck joint integrity cannot be guaranteed	Generally steel members including beam ends, cross frames, diaphragms, floor beams, etc.
Seats & pier/abutment tops	Beneath deck joint	Cyclical or as need identified	Prevent corrosion of bearings, anchorages & reinforcement steel	Beneath deck joints unless need identified (low waterway crossings, etc.); may not be required for tops with intact coatings
Bearings	Beneath deck joint	Cyclical or as need identified	Prevent premature failure from corrosion; prevent obstruction of movement from pack rust or debris	Metal bearings or bearings with metal components (ex. pot bearings) beneath deck joints unless need identified (low waterway crossings, etc.)
Superstructure members	In vehicle spray zone	Cyclical	Prevent corrosion or paint failure from non-visible contaminants	Generally steel members; roadway beneath bridge is salted, has speed ≥ 30 mph & vertical clearance ≤25'; include all steel beams & framing
Open grid deck	Open grid and elements beneath	Cyclical or as need identified	Prevent corrosion of grid decks, superstructure and substructure members from visible & non-visible debris & contaminants	If no visible contaminants cyclical frequency may be warranted when roadway is salted
Trusses, thru arches & other complex bridges	Members exposed to debris or salt spray; members prone to debris collection	Cyclical or as need identified	Prevent corrosion or paint failure from visible & non-visible contaminants	Generally steel members; many members are exposed to debris or salt spray including overhead & side members that form clearance envelope & members to the side & beneath the deck; many members are prone to debris collection including gusset connections, horizontal members, built-up open members, built up closed members with intermittent openings, etc.; most deck trusses & arches have less exposure

5.6.3.1.2 Bridge Washing Administration

The Scope of Work, Special Provisions, systematic selection process and list of bridges to be washed are to be submitted to BQAD for approval three weeks prior to advertisement.

5.7 PRESTRESSED CONCRETE BRIDGES

5.7.1 Repair of Prestressed Concrete Bridges

Utilize the November 1988, NCHRP Synthesis of Highway Practice 140, "Durability of Prestressed Concrete Highway Structures", when addressing items such as:

1. Repair of Prestressed Bridges
2. Method of Detecting Deterioration
3. Maintenance and Repair Techniques
4. Methods of Insuring Durability in New Construction

Two of the more significant and practical references of the synthesis are:

1. NCHRP Report 226, "Damage Evaluation and Repair Methods for Prestressed Concrete Bridge Members"

2. NCHRP Synthesis Report 280, "Guidelines for Evaluation and Repair of Prestressed Concrete Bridge Member"

Reports 226 and 280 contain a considerable amount of data on how to repair spalls, splice severed strands and strengthen beams through the addition of strands.

Repair and rehabilitation of damaged or deteriorated prestressed concrete beams, especially the repair of beams that were damaged by oversize vehicles is required on all rehabilitation projects.

The above references provide practical information on how to repair the most commonly occurring damages. However, another matter of concern is the observed deterioration of the bearing areas of some prestressed concrete box beams, generally found below leaky deck joints on structures usually more than 25 years old.

Spalling of the bearing areas is primarily caused by the infiltration of salt-laden runoff through leaky joints, with subsequent chloride saturation of the beam ends and resulting rusting of mild steel in the beam ends, and worse, rusting and debonding of prestressing strands.

Additionally, the beam seats at those deficient joints may be buried by 100 mm {4 in.} or more of salt-laden cinders and other flushed roadway debris. Nothing will stand up under such an environment. Structures thus affected should receive priority treatment under the Bridge Preventive Maintenance Program.

As an initial measure, the bearing seats and beam ends should be flushed clean and the joints should be repaired (by installing strip seal joints or eliminating the joint by providing a continuous deck as part of a rehabilitation project).

The affected structures should be monitored regularly as part of the inspection process, and joints, particularly on prestressed concrete box beam structures, should be kept watertight.

5.7.2 Procedure for Rating Existing Prestressed Concrete Bridges

The following steps shall be used in rating prestressed concrete bridges:

1. Analyze the bridge using PennDOT's LRFD Prestressed Girder Design and Rating computer program using the rating option.

2. Review analysis, does the bridge meet the operating rating levels?

    - Yes, summarize and submit findings

    - No, continue

3. Analyze the bridge using PennDOT's LRFD Prestressed Girder Design and Rating computer program using the rating option with this modification.

    - f’c value increased by 20%

4. Review analysis, does the bridge meet the operating rating levels?

    - Yes, summarize and submit findings

    - No, is the P-82 rating vehicle the only vehicle which is not meeting the operating rating level?

        - Yes, continue

        - No, go to Step 7

5. Analyze the bridge using PennDOT's LRFD Prestressed Girder Design and Rating computer program using the rating option with these modifications.

    - f'c value increased by 20%

    - P-82 live load which is comprised of the P-82 in the design lane that produces the largest load effects with the design truck in the other design lanes

6. Review analysis, does the bridge meet the operating rating levels?

    - Yes, summarize and submit findings

    - No, continue

7. Perform rating of the bridge using the original design method.

8. Review analysis, does the bridge meet the operating rating levels?

    - Yes, summarize and submit findings

    - No, contact PennDOT immediately so that posting procedure can be started

REFERENCES

Keating, P. B., and Fisher, J. W., Review of Fatigue Tests and Design Criteria on Welded Details, Fritz Engineering Laboratory, Report No. 488-1 85, Table 4

Memari, A., H. Harris, A. Hamid, and A. Scanlon. Seismic Column Reinforcement Study. Final Report, prepared for the Pennsylvania Department of Transportation, 2001.

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 1
PART A: POLICIES AND PROCEDURES

CHAPTER 6 - PROCEDURES FOR GEOTECHNICAL EXPLORATIONS

CHAPTER 6 - TABLE OF CONTENTS

6.1 GENERAL

Procedures followed for geotechnical explorations shall meet the requirements of this section, plus the following publications:

Design Manual, Part 3, Chapter 5, Sections 0 through 3

Design Manual, Part 4, Design Specifications, D10

Design Manual, Part 4, Policies and Procedures, PP1.9.4

Refer to Publication 293M, Geotechnical Engineering Manual, for the technical aspects of the exploration, including potential problem areas and geologic hazards.

6.2 RECONNAISSANCE

For other than single-structure replacements, reconnaissance for soils and geological engineering exploration shall be performed as a part of the Preliminary Engineering Study (see Design Manual, Part 1, Chapter 3). Major features and pertinent data may be presented on topographic maps of the site areas. For single-structure replacements and similar small projects, the reconnaissance may be conducted during the hydraulic study.

The reconnaissance shall generally consist of the following five phases:

(a) Search of published and unpublished information

(b) Visual site inspection

(c) Development of subsurface exploration program

(d) Subsurface exploration

(e) Preliminary Geotechnical Engineering Report

The following guidelines are provided to implement a core boring contract effectively:

1. Use one inspector for every two rigs, where practical, rather than one inspector for each rig as specified in PP6.3.4.2. Do not compromise the duties nor the responsibilities of the inspector.

2. Department personnel or a consultant working for the Department may provide the required drilling inspectors.

3. The contract document is extremely flexible. Hence, modify contract specifications on an as needed basis to secure information. Do not ask for information that does not contribute to determining the bearing capacity. Even though geotechnical studies are required during the preliminary design stage, ask for only minimum information at this stage to help to evaluate alternates. Seek detailed information for determining the bearing capacity or other soils parameters during the final design stage.

4. The standard contract requires filling each drilled hole with cement concrete or mortar. Use this clause judiciously. If use of other economical materials serve the intent, modify the contract requirement. In some instances (particularly NX coring in wilderness), filling holes may not be warranted. At the time of this writing, it is the Department's understanding that DEP does not require these holes to be filled.
5. The following items provide guidance on the geotechnical information required for various foundation types:

    a) If a spread footing is contemplated to be placed on bedrock (preliminary study would give some idea), detailed properties of the overburden material may not be of much importance, hence, only blow count, soil composition, gradation, water level, etc., would suffice for the overburden. However, all needed properties, including seam size, rock and seam quality, cementation, voids, solution cavities, reason for low RQD, or core recovery, etc., must be secured for the bedrock strata in accordance with the D10.

b) If a spread footing is contemplated to be placed on soils, all required data of the soil strata below the anticipated bottom of the footing up to prescribed depth should be secured to help in determining bearing capacity, settlement, and other pertinent values (e.g., corrosion resistivity for MSE walls). For bridges over streams, scour depth may control the bottom of the footing location and type of foundation.

c) For footings to be supported on point bearing piles (on bedrock), follow the same guidelines as for the spread footing on rock.

d) For footings on end bearings piles, secure detailed data of the pile end (last 3000 mm to 4600 mm {10 ft. to 15 ft.} of piles) supporting strata and the strata immediately below the supporting strata. Ask for minimum data (blow count, driveability constraints, corrosion resistivity, etc.) for the overburden.

e) For footings on friction piles, get all required detailed information for most strata.

f) For footings on caissons, secure all data for rock socket substrata and areas below it, bearing strata for bell type of caissons, and needed data for overburden for lateral stability similar to end bearing piles.

The above guidelines are shown for uniformity. However, areas where special conditions, such as solution cavities, mines, sinkhole activities, etc., are expected will require more core borings than the normal requirement. Collection of sufficient data should not be compromised. Similarly, for tie-back walls, additional holes are needed in the tie bearing area and in the wall area. These guidelines will reduce construction problems, delays and work orders that have been experienced during the last several years.

6.2.1 Search of Published and Unpublished Information

Review of available published and unpublished information shall include evaluation of the following:

(a) Preliminary plans of the proposed construction

(b) Surface features on topographic maps

(c) Geologic maps and other sources of geologic information

(d) Soil survey maps

(e) Aerial photographs

(f) Mine maps and subsidence information

(g) Previous geotechnical explorations in the vicinity of the project, including any soil studies done during the environmental evaluation for the project

(h) Logs of existing borings and water wells

(i) Records and photographs regarding the construction and behavior of nearby structures relative to planned structures

In addition to review of the above information sources, consult with local residents, consultants, contractors and Department personnel who may be knowledgeable about local surface and subsurface conditions.

6.2.2 Visual Site Inspection

The information obtained in PP6.2.1 shall be verified and supplemented with a site inspection, preferably with the District Soils Engineer. Observations of the following shall be included:

(a) Proposed locations of structures

(b) Existing structures (type and condition)

(c) Surface soils

(d) Topography and vegetation

(e) Drainage features

(f) Rock outcrops, excavations, visible indications of subsurface conditions

(g) Existing problem areas, such as slope movements, subsidence, mine shafts, or sinkholes

(h) Utility locations

6.2.3 Development of Subsurface Exploration Program

The Geotechnical/Foundation Engineer shall meet with the District Soils Engineer and District Bridge Engineer to identify and summarize potential problem areas and to outline the planned subsurface exploration work. The District Soils Engineer shall notify the BOCM of the meeting if it is anticipated that the expertise of the Geotechnical Section will be needed. A written submission is not required for the meeting.

A letter outlining the work shall be prepared by the Geotechnical Engineer and submitted to the District for review and concurrence of the District Soils Engineer before the work is begun.

6.2.4 Subsurface Exploration

For major projects, subsurface exploration shall be conducted at each proposed site to identify potential design or construction problems and to allow comparison of the suitability of the various sites. For single site replacements, foundation records from the existing structure will usually suffice, and subsurface exploration will not be required provided that no evidence is available indicating structural distress from the foundation. For structures where records are not available, a literature and file search should provide sufficient information.

A limited drilling and sampling (or test pit excavation) program shall be included, with at least one boring or test pit for each structure.

Laboratory testing of samples will rarely be required in the reconnaissance exploration. The scope of the investigation shall be sufficient to anticipate the type and depth of foundations, especially as related to structure type and span. For this work, subsurface exploration shall be considered a professional service.

The District Engineer may approve modification to, or elimination of, the subsurface exploration requirement if sufficient information is available from other evaluations obtained in the reconnaissance program to determine an economical bridge foundation.

6.2.5 Preliminary Geotechnical Engineering Report

Sufficient information about each site shall be presented to the Department in a Preliminary Geotechnical Engineering Report, developed in accordance with Publication 293M, to permit a comparison of approximate constructions costs for each site under consideration. Four copies of the report shall be provided to the District Soils and Bridge Engineers by attachment to the Type, Size and Location (TS&L) submission. As a minimum, the report sections (s) shall include the following::

(a) Proposed construction for which the exploration is made

(b) Data obtained during the search of published and unpublished information, including information from available geomorphological studies conducted during environmental studies.

(c) Subsurface features and structures observed during the visual site inspection and, if appropriate, air photo interpretation

(d) Scope of the subsurface exploration, if conducted (see PP6.2.4)

(e) Identification of geological formations in the area

(f) Conditions encountered in the subsurface exploration

(g) Evaluation of the effect of the condition encountered or anticipated on the proposed construction

(h) Relative advantages and limitations of each site under consideration

(i) Topographic map showing the proposed sites and locations of important site features

(j) Indication of further exploration requirements (including boring layout)

Space shall be provided in the report for endorsement by the District Soils Engineer.

6.3 FOUNDATION EXPLORATION - FINAL DESIGN

Following acceptance of the appropriate sections of the Preliminary Geotechnical Engineering Report, the foundation exploration shall be initiated as part of the final design in accordance with PP6.3.1 through PP6.3.5 and Publication 293M. Publication 293M also provides guidance regarding selection of the number and depth of borings, drilling techniques, sampling methods, in situ testing, and laboratory testing.

The following steps should be taken as outlined in Publication 293M and herein:

(a) Foundation exploration plan

(b) Foundation exploration meeting

(c) Preparation of proposal/contract

(d) Administration of the contract

(e) Report - Subsurface Conditions at Structure Locations (Note that this report is part of the Preliminary Geotechnical Engineering Report

6.3.1 Foundation Exploration Plan

After reviewing all available information from the appropriate sections of the Preliminary Geotechnical Engineering Report and receiving approval for initiating the final design, the Geotechnical/Foundation Engineer shall prepare and submit to the District four copies of the preliminary plan. This submission shall be made at least two weeks before the final exploration plan meeting. For each site, the following shall be included:

(a) The number of test borings at each structure unit.* For limestone and other similar problem areas, the following coverage shall be incorporated as a minimum:

- (1) Test borings must "enclose" foundation.
- (2) Additional borings shall be provided, as necessary, between substructure units to obtain an overall picture of the area at the bridge site.
- (3) Selected holes shall extend to a sufficient depth to locate mines, caverns, or other underground openings which could affect foundation performance.

(b) A discussion of possible adverse and positive subsurface conditions*

(c) Profile sheet noting existing and design grades

(d) Recommended methods of subsurface exploration

(e) Plan sheet noting suggested test locations*

(f) Recommended laboratory testing program

(g) Copy of pertinent foundation information from Preliminary Geotechnical Engineering Report

*These items and the appropriate sections of the Preliminary Geotechnical Report shall be included with the TS&L Submission (see PP1.9.3).

6.3.2 Foundation Exploration Meeting

The Geotechnical/Foundation Engineer shall meet with the District Soils Engineer and the District Bridge Engineer to present and discuss the Foundation Exploration Plan. The District Soils Engineer and the District Bridge Engineer may request assistance from the BQAD and BOCM if the foundation investigation will be complex and involve special testing.

A field view shall be held if deemed necessary by the District Soils Engineer. The Geotechnical/Foundation Engineer shall finalize the exploration plan on the basis of the conclusions of the meeting and submit the final exploration plan to the District as a part of the TS&L submission. The final exploration plan is the draft Subsurface Boring, Sampling and Testing Contract.

The Foundation Exploration Plan may be modified if unanticipated subsurface conditions are encountered during the drilling operation.

6.3.3 Entry onto Railroad Right-of-Way

6.3.3.1 RIGHT-OF-ENTRY PERMIT

Any entry onto a railroad right-of-way requires a right-of-entry permit; generally one permit can be obtained to cover all entries during the design of a project. The following activities are required to obtain a permit:

(a) Project Management shall inform the District Grade Crossing Engineer (DGCE) of the nature of the project. At this time, the DGCE shall inform the District Project Manager (DPM) or the District Liaison Engineer (DLE) of any special requirements that the Railroad may have. There is the possibility that the Railroad's requirements may change during the work; the DGCE shall inform the DPM or DLE of any change.

(b) DGCE shall amend the insurance agreement, process an inspection agreement and request an estimate from the Railroad of its costs.

(c) The District (in-house projects) or the Design Consultant shall check the status of the insurance and inspection agreements, and then apply for the permit, which is made out in the name of the Department of Transportation. Generally, one can apply for the permit before agreements are finalized.

(d) Information for each individual entry (for all entries including those of any subconsultant or subcontractor) shall be supplied when applying for the permit. This shall include exact location of the entry, estimated cost of work to be performed on the Railroad property, distance from the outermost track where the activities will take place, anticipated date of entry, number of people and length of time.

(e) Railroad shall send a right-of-entry form to the applicant. Design consultant shall complete, execute and send the form to the DGCE. The applicant for in-house projects shall forward the form directly to the DGCE. DGCE will review and complete the form, arrange for Department execution and return the form to the Railroad. Department personnel shall not sign the form. The Railroad shall forward the right-of-entry permit to the applicant. The DGCE will advise the applicant of any necessary modifications due to a specific Railroad's policy.

6.3.3.2 ACTUAL ENTRY ONTO RIGHT-OF-WAY

The following activities are required for actual entry onto the right-of-way for geotechnical exploration:

(a) Determine an exact scheduled starting date and ending date for the entry.

(b) Incorporate these dates in the letter seeking bidder interest and the proposal.

(c) Where District policy allows an alternative to a defined schedule, note the desired method of scheduling in the letter seeking bidder interest and in the proposal.

(d) Notify the DGCE of the scheduled starting date at least 30 days before that date. The Railroad will require at least 21 days notice.

(e) Maintain an independent record of personnel making the entry, date of entry, purpose of entry, Railroad personnel and equipment present and their activities, etc. Review and countersign Railroad time sheets(s) daily; do not countersign if there is disagreement - notify the DPM or DLE. Forward a copy of the time sheet(s) to the DPM or DLE within 15 days of their receipt.

(f) Comply with the Railroad's required safety training prior to entering.

6.3.3.3 METHOD OF PAYING RAILROAD RIGHT-OF-ENTRY COSTS

(a) DPM, DLE or DGCE shall make provisions for paying costs based on Department guidelines.

(1) DPM, DLE or DGCE shall inform appropriate District personnel for in-house projects.

(2) The scope of work shall detail the method of payment for design consultant projects. If the method of payment is not included in the original Consultant Agreement, it shall be processed as a supplement.

(b) The party responsible for processing the application for the permit shall advise the Railroad of the procedure for submitting billings.

6.3.4 Preparation and Administration of Subsurface Boring, Sampling and Testing Contract

6.3.4.1 PREPARATION OF PROPOSAL/CONTRACT

A proposal/contract for undertaking the necessary field work shall be prepared using the current Subsurface Boring, Sampling and Testing Contract (Publication 222M) after the draft copy has been accepted and when directed.

Contracts let by the Department shall be advertised in the Pennsylvania Bulletin in addition to the following procedure:

(a) Send notice to all affected property owners as detailed in Publication 222, Subchapter 5D, Section 103.11.

(b) Consider entry onto Railroad right-of-way (see PP6.3.3).

(c) Prepare the Subsurface Boring, Sampling and Testing Contract documents (Publication 222M) in accordance with the conclusions of the Exploration Meeting and Publication 293M. Prepare a plan of the foundation layout indicating the location of the test borings and other pertinent information.

(d) Prepare an Engineer's Estimate of the cost of the work.

(e) Prepare a draft of the letter to be sent to all the contractors on the current Approved Test Boring Contractor's List (see Publication 222. Include in this letter the county, state route and section numbers of the project; anticipated type of borings, samples and field tests with approximate linear footage or amount; location of the holes (water or land); whether or not Maintenance and Protection of Traffic Sketches will be provided or required. Note whether the Railroad requires safety training prior to entry on its right-of-way; anticipated dates for bid opening and Notice to Proceed; and any other pertinent data. Request a reply within ten days advising whether the Contractor is interested in bidding. Attach a map that would allow anyone unfamiliar with the area to find the project.

(f) Prepare a draft of the letter to be used to transmit the Subsurface Boring, Sampling and Testing Contract Documents to the Contractors who respond to Item (e). Include in the letter the county, state route, and section numbers of the project; the place, date and time of the bid opening; and a reminder to follow the instructions in the Instructions to Bidders.

(g) Send three sets of the items outlined in Items (c) through (f) to the District Office for review and concurrence by the District Soils Engineer.

(h) Send the letter requesting interest (Item (e)) upon receipt of concurrence from the District.

(i) Prepare and transmit the contract documents to the contractors expressing interest.

(j) Conduct the pre-bid meeting. At this time all borings should be staked on the project.

(k) Attend the bid opening at the District Office. Check the bid documents for completeness and correctness of quantities and cost tabulations. Tabulate the bid results. Determine the low bidder. Obtain concurrence of the District Office on the acceptability of the low bidder.

(l) Notify all bidders of the results; send each a tabulation of the bids. Return all proposal guarantees, except those of the two lowest bidders. Notify the lowest bidder to complete the forms in the bid package and furnish the required Contract Bond, Additional Bond for Labor and Material, and insurance certificates. Notify the second lowest bidder that his/her proposal guarantee will be held until the lowest bidder completes the necessary paperwork.

(m) Review the lowest Bidder's submissions for completeness. The insurance certificate shall name the Commonwealth of Pennsylvania as a coinsured. When all submissions are complete, return the proposal guarantees and issue a Notice to Proceed. Notice to Proceed should clearly state the day on which the time charges will begin.

(n) Furnish the District Engineer/District Administrator with a copy of all correspondence at the time it is sent out. Only one copy of the advertisement letter is needed. Furnish a copy of all pages in the bidding package in which the Contractor has filled in information, and copies of the bonds and insurance certificate.

6.3.4.2 ADMINISTRATION OF THE CONTRACT

6.3.4.2.1 General

Contracts let by the Department shall be inspected by District personnel or the District's representative. Contracts let by the Consultant shall be inspected by the Consultant's personnel.

The review of all inspections shall be the responsibility of the District Soils Engineer.

One full-time, qualified inspector shall be assigned to each drill rig. Supervision can be provided off-site when all inspectors are Level 2 or onsite when the inspectors are Level 1. A supervisor shall not be permitted to assume the duties of an inspector, except in an emergency.

The District Engineer shall modify the above only in exceptional cases.

It is imperative that every effort be made to ensure that:

(a) Adequate information is obtained to establish all design parameters and to address all subsurface problems that may arise in design or construction.

(b) The District Soils Engineer and the District Bridge Engineer (or Plans Engineer if no bridge work is involved) are promptly informed of any conditions that may result in a change in the contract, or any unusual conditions that are encountered.

(c) The subsurface conditions determined by the operations are fully and accurately described in the logs and the records.

6.3.4.2.2 On-Site Supervisor and Level 2 Inspector

The following minimum qualifications shall be met by onsite supervisors and Level 2 inspectors:

(a) Ability to speak, write, read and understand the English language

(b) Understanding of the general geotechnical design principles involved in the anticipated construction and laboratory testing

(c) Prior experience on three test boring operations

(d) Capability of acting as a direct representative of the Engineer

(e) Knowledge of general drilling practices, the project specifications and Publication 293M, as they apply to the project

(f) Demonstration of the above to the satisfaction of the District Soils Engineer at an interview to be held at least two weeks prior to the start of the work. Names and qualifications shall be submitted prior to the interview.

6.3.4.2.3 Level 1 Inspector

Use of Level 1 inspectors requires onsite supervision. The following minimum qualifications shall be met by a Level 1 inspector:

(a) Ability to speak, write, read and understand the English language

(b) Prior experience on three test boring operations

(c) Knowledge of general drilling practices and description of rock and soil samples as presented in Publication 293M

(d) Demonstration of the above to the satisfaction of the District Soils Engineer at an interview to be held at least two weeks prior to the start of the work. Names and qualifications shall be submitted prior to the interview.

6.3.4.2.4 Duties of Inspection Forces

Prior to the start of the work, the following activities shall be performed:

(a) Forward to the District a copy of the Contractor's submission providing information required by Publication 222M, Sections 104.01, 104.03 and 104.09.

(b) Provide information on the property owners to the Contractor (name, address, telephone number, etc.).

(c) Determine the adequacy of the Contractor's drilling equipment and equipment for maintenance and protection of traffic.

(d) Determine if the Contractor has contacted the involved utilities.

(e) Determine if the required safety courses have been taken by the workers (such as for railroad or hazardous waste work).

During the work, the following activities shall be performed:

(a) Identify boring and test pit locations according to the plans or other instructions.

(b) Ensure that the operations are performed according to the terms of the contract.

(c) Log test borings and test pits are to be made in accordance with PP6.3.4.2.5.

(d) Select locations and depths for sampling and in-situ testing.

(e) Maintain a diary to:

  (1) Document daily activities

  (2) Document field instrument installations

  (3) Record any instructions given to the Contractor, including date, time, person to whom given and the Contractor's response, and whether actual work was performed according to the instructions

  (4) Maintain any records required by the contract

  (5) Record, in detail, any events that may later result in litigation or a claim.

(f) Record changes such as minor shifts in location, depth of boreholes, etc.

(g) Review and countersign the daily time sheets connected with work on the Railroad right-of-way.

(h) Promptly report any unexpected findings to the District Soils Engineer or the District Bridge Engineer and to the Inspector's supervisor.

(i) Perform other duties as directed or required.

Any supervisor or inspector who does not demonstrate proficiency or dependability at the project site, or engages in activities contrary to the best interest of the Department as determined by the District Soils Engineer, will be subject to immediate removal. A qualified replacement will be required before work will be permitted to resume.

6.3.4.2.5 Engineer's Field Logs

Boring and test pit logs shall include information identified in PP6.3.4.2.5.1 and PP6.3.4.2.5.2. Soil samples and rock core shall be described in conformance with ASTM D 2488 as presented in Publications 222M and 293M, and shall be recorded on the standard logs.

6.3.4.2.5.1 Boring Log

The following items shall be noted on the Engineer's field boring log:

(a) General information

(1) Project identification, including county, state route and section number

(2) Test boring identification number

(3) Dates on which the boring was begun and completed

(4) Name of driller

(5) Name of drilling inspector (District or Engineer's representative)

(6) Elevation of top of test boring

(7) Location of test boring relative to project reference line (e.g., segment, offset and offset from centerline) or other suitable reference points

(8) Type of drill rig used

(9) Drilling method used to advance the boring in soil

(10) Type, diameter and depth of any casing used

(11) Size of hammer and free fall used to advance casing (optional)

(12) Size of hammer and free fall used to advance split-barrel sampler and size of sampler

(13) Drilling method used to advance boring in rock

(14) Type and size of core barrel used, including Manufacturer and designation of core bit

(15) Basis for low (less than 70% in soil and 90% in rock) sample recovery

(16) Changes in drilling method and reasons for such changes

(b) Specific sample information

(1) Depth, type, number, relative moisture content and recovery of each soil sample

(2) Blows per 300 mm {1 ft.} to advance casing (optional if driven)

(3) Blows per 150 mm {6 in.} (or less) to advance split-barrel sampler

(4) Relative density of cohesionless soils based on Standard Penetration Test

(5) Estimated unconfined compressive strength of fine-grained soils based on pocket penetrometer and/or torvane tests

(6) Length of core recovered, rock quality designation (RQD) and percent recovery for each run of rock core

(7) Description and identification of each soil and rock sample taken, and depth to the top and bottom of each sample

(8) Depth to top and bottom of profile for each distinctive interval, layer, or horizon in soil, rock, or anomalous zones

(9) Depth to groundwater level, with elapsed time after completion of drilling, and method by which observation was made

(10) Depths at which field tests are made and results of tests (if available)

(11) Difficulties in drilling (obstructions, caving, boulders, rising of sand into bottom of boring, etc.) including basis for low (less than 90% in rock and 70% in soil) or unusual recovery

(12) Depth of loss and return of circulating water and increase in usage of drilling water

(13) Any additional information (such as color changes in drill return water, tool drops, drilling advancement rate, etc.) which may be of assistance in defining the presence of strata changes, boulders, voids, or other subsurface conditions

Standard Engineer's field boring log is shown in Publication 222.

6.3.4.2.5.2 Test Pit Log

The following items shall be noted on the Engineer's field test pit log:

(a) Project identification, including county, state route and section number

(b) Test pit identification number

(c) Date of excavation

(d) Name of drilling inspector (District or Engineer's representative)

(e) Elevation of top of test pit

(f) Location of test pit relative to project reference line (e.g., segment, offset and offset from centerline) or other suitable reference point

(g) Type and size of excavation equipment

(h) Ease of using excavation equipment

(i) Caving of sides

(j) Type of shoring (if used)

(k) Seepage amount and elevation

(l) Depth to top and bottom of profile for each soil type

(m) Depth, type and condition of rock (if encountered)

(n) Relative density or consistency of soils

(o) Pocket penetrometer or torvane test results

(p) Moisture condition of soils

(q) Presence of boulders

(r) Reason for stopping excavation (e.g., caving sides, limit of reach of backhoe)

(s) Photographs of each test pit are recommended

A standard Engineer's field test pit log is shown in Publication 222.

6.3.4.2.6 Disposition of Soil and Rock Samples

Upon completion of drilling operations (or periodically for larger projects), arrange for the contractor to deliver all samples and core boxes to the location designated in Publication 222M (Instruction to Bidders, Article H). If the designated location is not the District's storage facility, arrange for delivery to the District. Any delivery to the District requires 72 hours notice.
Any soil or rock sample taken from a core box for testing shall have a sample jar containing a description of the sample and purpose of removal placed in its core box location.

6.3.4.2.7 Work Orders and Disputed Work

The District Soils and/or Liaison Engineer shall be consulted prior to submitting a work order and if there is a dispute about the type of work.

Work orders shall be submitted to the District Engineer/District Administrator, in letter form, explaining the reason for the work and the quantity, type, unit prices and monetary value for each item of work. Include any sketch that may be necessary to explain or locate the work. Also submit a letter from the Contractor agreeing to the quantity, type, unit prices and monetary value for each item, include any other data that may be needed to justify the price. Both the work order and the Contractor's letter should mention any agreed-to time extension.

The Contractor shall not be informed to proceed with the work until authorization is received from the District Engineer/District Administrator.

The District will supply the forms and instructions to record disputed work.

6.3.4.2.8 Submissions from Exploration Program

The following submissions shall be made to the District:

(a) Copy of the Driller's log and the original Engineer's field log within 48 hours of obtaining the last water-level reading

(b) Typewritten Engineer's field logs with log tracings prepared by the Geotechnical/Foundation Engineer within 14 days of completing the project

(c) Copies of any correspondence, records, notes, tabular forms, etc., developed during the contract, that the Department considers necessary for its records

(d) Geotechnical information

In addition to the items listed in PP1.9.4.3, the following shall be included in the Foundation Report (Subsurface Conditions at Structure Locations):

(1) Brief description of each site including history, surface features, geological formation and items identified during the final exploration plan meeting

(2) Typed logs, plan showing test boring layout and log tracings prepared by the Geotechnical/Foundation Engineer

(3) Soil and rock parameters to be used in design, with calculations and references

(4) Results of the professional engineer certified laboratory tests

(5) Discussions of the applicability of various foundation types and cost comparisons; recommendations for foundation substrata improvements

(6) Other geotechnical information deemed necessary to help justify the foundation-type selected

The foundation submissions shall be made as detailed in PP1.9.4.

6.3.5 Prequalification for Drilling Contractors

The prequalification criteria, submittals and approval procedures shall be as presented in Prequalification Procedures for Drilling Contractors.

6.3.5.1 PREQUALIFICATION APPROVAL

The Soils and Geological Engineering Section of the Materials and Testing Division is responsible for maintaining and periodically distributing a list of prequalified contractors. A Prequalification Request Approval Committee, composed of at least three District Soils Engineers and chaired by a designated individual from the BOCM, shall be responsible for prequalifying drilling contractors. The membership of the District Soils Engineers shall be on a rotational basis.

6.3.5.2 PERFORMANCE EVALUATION

Within three months after each drilling contract is completed, the District Soils Engineer shall prepare, review with the driller and send a completed performance evaluation report to the Soils and Geological Engineering Section, Materials and Testing Division, BOCM.

6.4 SERVICE PURCHASE TEST BORING CONTRACTS

The standard specifications for subsurface boring, sampling and testing as shown in the test boring contract document shall apply to all subsurface investigation work performed for the Department, including work performed under a service purchase contract. If changes are required for a specific job, they shall be handled in a similar manner to special provisions and included in Attachment II of the test boring contract. This attachment is referred to as Modifications and Additions to Standard Specifications for Subsurface Boring and Sampling. It is in this section that modifications to the specifications can be made (which might include changes of method of payment, deletion of sections, modification of work, etc.) to suit various situations which may arise.

REFERENCES

American Society for Testing and Materials, Annual Book of ASTM Standards, Volume 04.08, Soil and Rock Building Stones, 1987

PART 4

VOLUME 1
PART A: POLICIES AND PROCEDURES

CHAPTER 7 - DRAINAGE STRUCTURES, SCOUR AND CULVERTS

CHAPTER 7 - TABLE OF CONTENTS

7.1 HYDRAULICS

7.1.1 Hydraulic Design

For hydraulic design of drainage structures, including bridge waterways, see Design Manual, Part 2, Chapter 10, and the applicable directives.

7.1.2 Hydraulic Data

The following hydraulic data for structures draining an area 1.3 km^2 {0.5 mi^2} or greater shall be shown on the drawings:

(a) Drainage area (in square kilometers) {in square miles}

(b) 100-year and other floods as required and specified in Design Manual, Part 2, Chapter 10, including:

(1) Magnitude (m^3/s) {cfs}

(2) Frequency

(3) Pertinent water surface elevation

(4) Pertinent velocity at the structure entrance

7.2 SCOUR

7.2.1 Scour Investigation

Scour investigations shall be completed for all drainage structures and retaining walls (both cast-in-place and mechanically stabilized (MSE)) along streams. This investigation shall be included with the foundation submission and H&H Report. The investigation shall contain scour calculations where appropriate (see PP7.2.2), including calculations for all piers and abutments without properly designed protective measures. The investigation shall also include site inspections, including inspection of nearby structures. The reference for scour investigation is the FHWA Hydraulic Engineering Circular No. 18 (HEC-18), "Evaluating Scour at Bridges", Fourth Edition, May 2001.

Spread footings on erodible material shall be considered only if scour calculations are completed and can be corroborated by a site inspection by the performance of spread footings in nearby structures which have survived major floods, or if properly designed protective measures are provided. Otherwise, the bridge foundation should be extended to sound bedrock or supported on piles. If a foundation is supported on piles, the pile design must account for the estimated depth of scour and include a check of column strength for the unsupported length.

For spread footings set below scour depth, backfill the excavation with durable rock. Where history of the bridge site indicates that the channel becomes restricted due to accumulation of debris or ice, consider the constricted opening in the scour investigation. Where the maximum highwater elevation at the drainage structure site is due to a backwater condition resulting from the stage of a downstream waterbody, the scour investigation shall consider the calculations based on a 100-year flood resulting from the watershed upstream from the drainage structure site assuming no backwater from a downstream confluence.

Where dams exist upstream from the drainage structure site, the design flood for the dam and its spillway shall be considered in the scour investigation.

No scour analysis for pipe or box culvert is required. Refer to BD-632M for scour protection details for box culverts. Refer to RC-30M for scour protection for pipes

7.2.2 Scour Design Flood and Scour Computation

Total scour depth is comprised of:

(1) aggradation or degradation,

(2) contraction scour, and

(3) local scour.

Footings are to be designed based on the total scour depth obtained from a design flood which is defined as a 100-year flood, the flood of record (if available) or the overtopping flood (if less than the 100 year flood), whichever results in the worst-case scour condition.

The scour due to lateral movement or shifting of the stream should also be evaluated for bridges on floodplains with a history of lateral movements of the stream from one side of the floodplain to the other through geologic time. For multi-span bridges, a scour prism plot (Chapter 8, HEC-18), which illustrates the total scour depth at any location in the bridge opening, and a site evaluation shall be included with the scour analysis in the H&H and Foundation Reports. The scour prism plot should use the scour depths that have considered both engineering judgment and also reduction of scour depth from riprap placement. Pier and abutment foundations shall be designed for the maximum total scour to account for channel and thalweg shifting.

Local scour depths for piers and unprotected abutments shall be calculated using equations that apply to the sites and design conditions. Engineering judgment shall be used to select a depth of foundation based on the computed scour depths from applicable equations and site history. If depth of foundation cannot be established based on engineering judgment, the highest or the lowest scour depth may be used, provided the designer can justify it based on the proven performance of an existing structure at the same location or nearby location on the same stream. The total scour depth for piers shall be the sum of the general contraction scour and the local scour, which includes both the pier scour and the vertical pressure flow pier scour depth. Local pier scour shall be calculated using the CSU equation (Chapter 6, HEC-18) for live-bed and clear-water conditions when the pier footing is not exposed to the flow. The pier width in the CSU equation shall be the pier width perpendicular to the flow direction for the frequency event being considered. When using a value less than 1 for the armoring correction factor (K_4) in the CSU equation, the streambed gradation size requirements in HEC-18 must be satisfied and a bucket sample analysis of the streambed material must be provided. The complex pier foundation equations (Chapter 6, HEC-18) shall be used to calculate local pier scour when the pier footing is exposed to the flow. Arneson’s equation for scour should be used to calculate vertical contraction scour for pressure flow conditions at a structure and added to the local pier scour depth.

Properly designed rock riprap and/or guide banks for abutment protection may negate the need to compute local scour depth for abutments. For piers, a scour depth reduction factors of up to 50% may be used for local scour when multi-layered riprap scour protection are provided as per Figure PP7.2.4-1, PP7.2.4-2 and PP7.2.4-3, and appropriate inspection frequency is provided. Note that the local scour will include the sum of the pier scour and the vertical pressure flow pier scour, if applicable. This provision is not in accordance with HEC-18, and was negotiated with the FHWA based on our past experience, and conservative design practices. Multi-layered riprap scour protection means more than one layer of rock with a minimum placement depth, D, comprised of one full layer (based on the nominal placement thickness indicated in Pub 408, Section 850) on top of the footing, with additional rock rip rap placed to the bottom of footing as shown in the Figures in PP7.2.4.

7.2.3 Superflood

Footings shall be checked for a superflood (Q500) which is defined as a 500-year flood. Scour calculations shall be performed for the 500-year event to determine if the Q500 scour depth is below the bottom of the footing elevation. Stability of all foundations, including unsupported length of piles must be checked for the superflood (Q500) with a factor of safety. 1.0.

7.2.4 Footing Location Guidelines

The following guidelines are provided as a minimum criteria for foundations in river environment:

(a) Spread footings

If scour calculations indicate scour depths terminating above rock, the foundation location for scour may be treated as if it is on soils as per PP7.2.4(a)3.

1. On non-erodible sound bedrock:

The sound bedrock may be defined as rock mass with discontinuities that are tight or open less than 3 mm {1/8 in.} (the rock mass is at least of the quality with RQD 70% and recovery 90%) such as non-erodible limestone or granite.

If necessary, conduct additional testings or re-evaluate the core boring data to confirm the soundness of the bedrock.

The bottom of the footing may be placed flush with the surface of the bedrock or keyed 150 mm to 300 mm {6 in. to 1 ft.} below the bedrock (neat cut around edge of the footing). The foundation report shall describe the determination of the footing placement. If the footing is keyed, sliding can be ignored. Also, clearly indicate on the plans or in the specification that blasting is not permitted for rock excavation for the footing. Concrete of the footing shall be placed against the vertical cut surface.

Place the desirable rock around the footing at least 300 mm {1 ft.} beyond the stream-side boundaries of the footing. The top elevation of the rock protection shall not be higher than the adjoining streambed by 900 mm {3 ft.}, subject to hydraulic requirements. See Figure 1 for rock placement for piers. For abutments and wingwalls, provide rock protection similarly, but in front of the wall only.

2. On erodible rock:

   Place the bottom of footing 900 mm {3 ft.} or more into the erodible rock even though the scour calculations indicate deeper scour for the design flood and superflood as specified in PP7.2.2 and PP7.2.3, respectively. If the calculated scour depth due to superflood is less than 900 mm {3 ft.} into the erodible rock, the bottom of the footing shall be located at the scour depth, but not less than 450 mm {1'-6"} into the rock or 1800 mm {6 ft.} below the adjacent streambed (ground) elevation. The above bottom of footing elevation shall be confirmed by the engineering geologist who is familiar with the area geology and rock formations. This decision should be made based on an analysis of rock cores, including rock quality designation and local geology, as well as hydraulic data and past performance of nearby existing structures on the same stream, or in the case of a bridge placement, the performance of the structure being replaced.

   Blasting shall not be permitted to excavate rock for the footing. Concrete of the footing shall be placed against the excavated vertical surface. See Figure 2 for rock placement for piers. For abutments and wingwalls, place rock protection similarly, but in front of the wall only.

3. On soils including gravel, cobbles and boulders:

   Place the top of the footing below the total depth for design flood as specified in PP7.2.2 with the bottom of footing not less than 1800 mm {6 ft.} below the adjacent streambed (ground) elevation.

   For piers on spread footings without multi-layered riprap scour protection, scour computations shall consider the width of any footing exposed by scour, including exposure by local scour, even if the contraction scour and long-term degradation is above the top of footing.

   Place durable rock in all stream-side excavated areas, unless the foundation is specifically designed for scour otherwise. Please note that additional rock protection is needed if the total scour depth based on superflood is lower than the bottom of footing elevation based on design flood. See Figure 3.

   The top elevation of the rock protection shall not be higher than the adjoining streambed (ground) elevation by 900 mm {3 ft.}, subject to hydraulic opening requirements. For abutments and wingwalls, place rock protection in front of the wall only.

   For abutments and wingwalls completely above the scour design flood flow (see PP7.2.2) elevation, the bottom of footing shall not be less than 1200 mm {4 ft.} below finished grade.

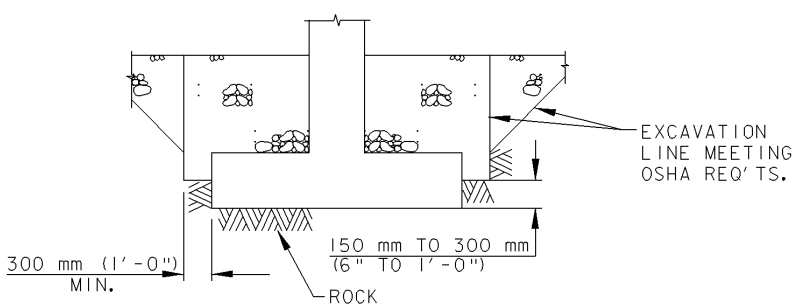


Figure 7.2.4-1 - Footing on Non-Erodible Sound Rocks

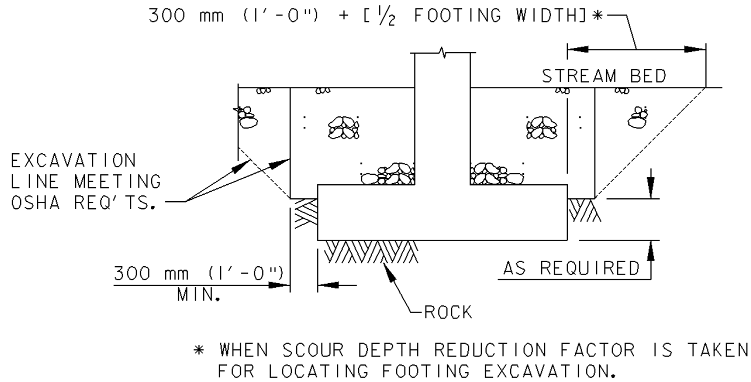


Figure 7.2.4-2 - Footing on Erodible Rocks

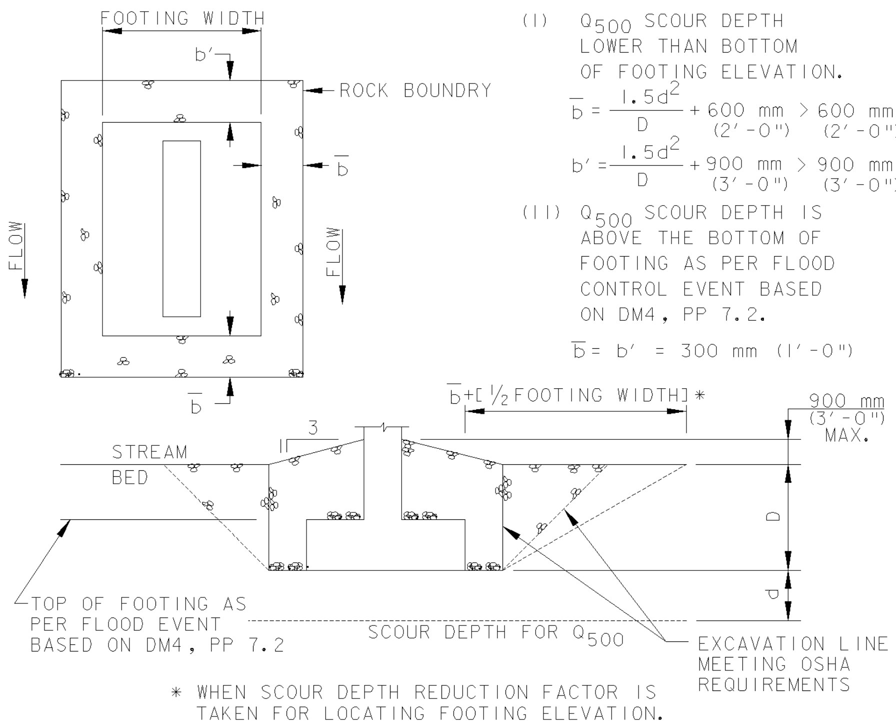


Figure 7.2.4-3 - Footing on Soils

(b) Footings on piles or drilled shafts

Place the top of the footing below streambed a depth equal to the estimated contraction scour (general scour) depth for scour design flood as specified in PP7.2.2. However, the bottom of the footing shall be located at least 1800 mm {6 ft.} below the adjacent streambed (ground) elevation for piers and arch abutments and at least 1200 mm {4 ft.} below the adjacent streambed (ground) elevation for abutments and wingwalls. For friction and end bearing piles, the pile length down to the total scour depth (including local scour) for the superflood flood shall not be considered in load transfer to the soils. For abutments properly protected with rock riprap or other approved means, piles may be assumed to be fully supported for unsupported column bending calculations. For piers, the scour depth reduction (credit) on local scour depths (due to rock protection) shall be accounted for in determining unsupported pile length. Where special conditions indicate that the theoretical friction portion of a friction pile is substantially below the unsupported length due to scour, the top portion of the friction pile may need to be installed using auguring or other methods to preclude pile driving resistance in the unsupported length area during pile installation.

Piles and shafts in total scour zones shall be designed as laterally unsupported columns. Place the rock similar to Figure

2 for rock placement at piers. For abutments and wingwalls, place the rock protection similarly, but in front of the wall only.

If a substructure unit is located beyond the 500-year floodplain limits, the bottom of footing may be placed as if it is not in a river environment.

Special attention should be given to floodplains with a history of lateral movements of the stream from one side of the plain to the other through geologic time. In this situation, the worst scenario shall be used in locating the bottom of the footing.

7.2.5 A Guideline for Riprap Size Selection

Use Figure 1 in determining the rock size. This figure is based on the FHWA formula $D_{50} = 692V^2/(S\text{-}1)(2g)$ {U.S. Customary Units: $D_{50} = 0.692\ V^2/(S\text{-}1)(2g)$} for riprap sizing at piers, but may be used for all substructures by varying V based on the substructure location. For piers, V is taken as 1.5 times the average upstream velocity of the flow in the bridge section just upstream of the piers. For vertical abutments, V shall be taken as 1.8 times the flow velocity. The chart is prepared for determination of D_{50} for values of V up to 5.3 m/s {17.5 fps}. For value of V higher than 5.3 m/s {17.5 fps}, the above formula should be used. Use minimum rock size of R-6.

The quality and gradation of the selected rock should be as per Publication 408, Section 850. The D_{1550} size listed in Section 850 shall be used as D_{50} for design purpose. For uniformity, use the following rock size:

V(m/s)	V(fps)	Rock Size
up to 3.7	up to 12	R-6 or larger
3.8 to 4.7	13 to 15	R-7 or larger
4.8 to 5.3	16 to 17.5	R-8

where V is 1.5 x velocity of flow for piers, or 1.8 x velocity of flow for abutments and wingwalls.

The velocity of flow should be based on the design flood specified in PP7.2.2. However, if the scour depth, based on Q500, is lower than bottom of the footing elevation, then the velocity of the flow should be based on Q500 and not the design flood.

For values of V higher than 5.3 m/s {17.5 fps}, special provisions should be developed to cover the size of the rock by the above- mentioned formula.

In some special cases, the riprap size could be extremely large and impractical from Equation $D_{50} = 692V^2/(S\text{-}1)(2g)$ {U.S. Customary Units: $D_{50} = 0.692\ V^2/(S\text{-}1)(2g)$}. In these cases, design guidance for proper substructure protection may be found in the Hydraulic Engineering Circular (HEC) series.

The following guidelines are provided for riprap design:

1. For practical design purposes, the maximum riprap size should be R-8. A larger rock size may be used by the designer if available and required by design.

2. If a design dictates rock size larger than R-8 and it is not available to the District, the designer should consider other alternate designs such as:

    a. A larger bridge opening which will reduce the riprap size due to the reduced velocity.

    b. A pile supported or a caisson supported footing.

    c. Guide banks or channel improvements.

    d. Normal rock with durable (estimated life of 50 years) woven metal fabric (e.g., gabions).

    e. A cast concrete block (a manufactured concrete rock not a concrete streambed paving) if no other method is suitable.

Provide a V-notch (3/4" minimum) in the stem of each substructure unit to locate the top of rock protection. This is to help the bridge inspector in the field determine rock protection loss. Consider choking R-8 rock with R-4 on the surface to provide better footing for the inspector. In environmentally sensitive areas, consider placement of natural soils above riprap. In these cases, footings may need to be lowered to ensure multiple layers of riprap protecting footing.

For riprap at piers, the lateral extent of riprap <u>on each side</u> of the pier face need not exceed twice the width of the pier. Studies have shown that the local scour effects will not extend beyond a riprap mat of a total width of five times the pier width.

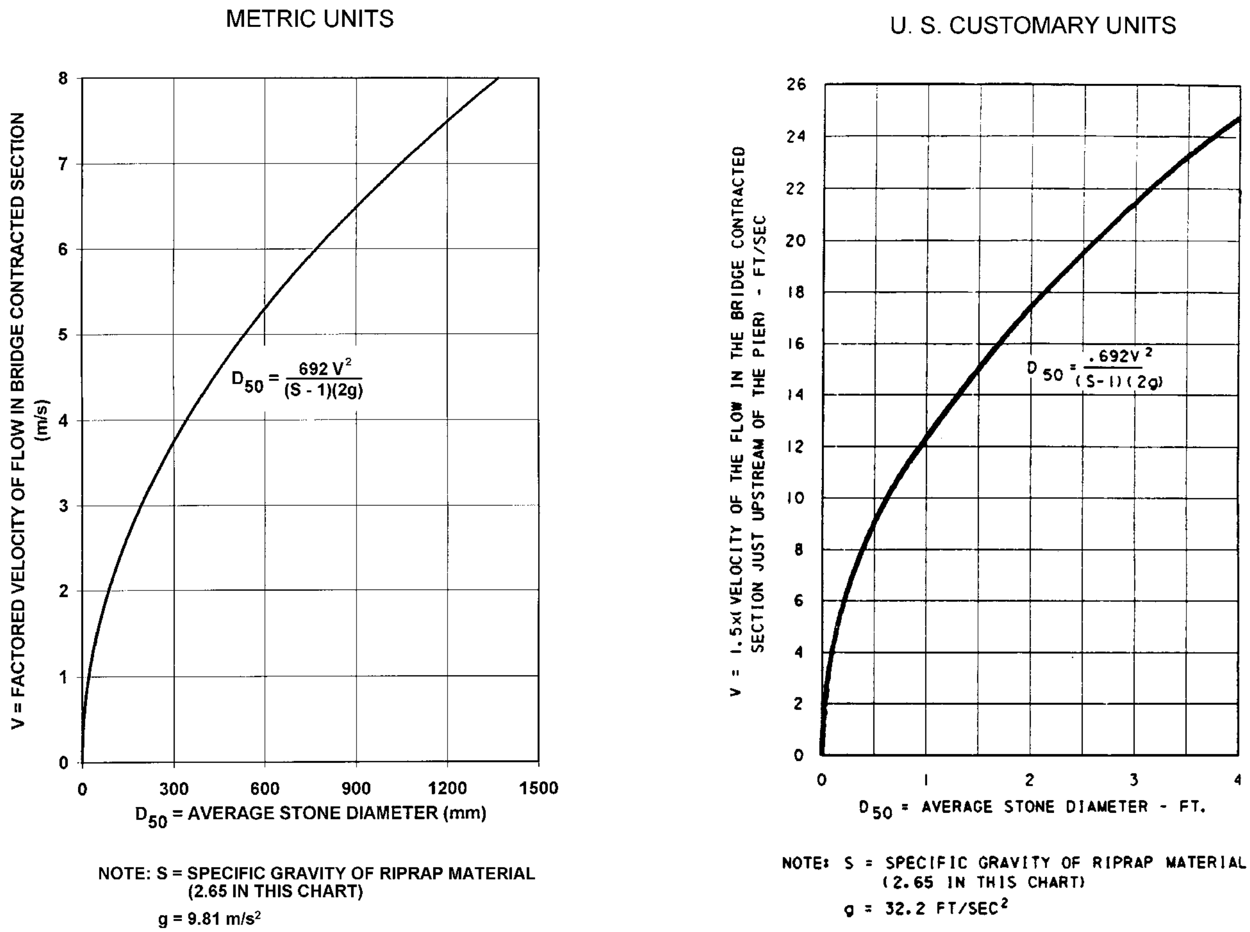


Figure 7.2.5-1 - D_{50} Rock Size

7.2.6 Scour Investigation References

Additional references for scour investigation are as follows:

(a) Emmett M. Laursen and Arthur Tock, Scour Around Bridge Piers and Abutments, Bulletin No. 4, State University of Iowa, May 1956.

(b) Scour at Bridge Waterways, NCHRP, Synthesis No. 5, Highway Research Board, 1970.

(c) Emmett M. Laursen, Scour at Bridge Crossings, Iowa Highway Research Board, Bulletin No. 8, State University of Iowa, August 1958.

(d) Countermeasures for Hydraulic Problems at Bridges, Vol. 1, "Analysis and Assessment", Report No. FHWARD78162, Federal Highway Administration, September 1978.

(e) Countermeasures for Hydraulic Problems at Bridges, Vol. 2, "Case Histories for Sites 1283", Report No. FHWARD78163, Federal Highway Administration, September 1978.

(f) Hydraulic Analyses for the Location and Design of Bridges, Vol. 7, "Highway Drainage Guidelines", AASHTO, etc.

(g) NHI Course No. 13010, Highway in the River Environment, FHWA Publication No. FHWA - HI - 90 - 016, February 1990.

(h) Stream Stability at Highway Structures, Second Edition, Hydraulic Engineering Circular No. 20, FHWA Publication No. FHWA - IP - 90 - 014, November 1995.

(i) Scour at Bridges, FHWA Technical Advisory (TA) T.5140.20, September 1988.

7.3 CULVERTS

7.3.1 Design Specifications

See A12 and D12.

7.3.2 Concrete Box Culvert Alternatives

The procedure regarding concrete box culvert alternatives shall be as follows:

(a) A precast concrete box culvert shall be prepared as a primary design, with a cast-in-place (CIP) concrete box culvert as a contractor-designed alternate, unless it can be documented that a precast concrete culvert is not suitable for a specific site, e.g., non-uniform bearing strata, cost ineffectiveness. If hydraulically and structurally feasible, both precast and CIP concrete box culverts shall be allowed as alternates for a CIP concrete arch and other small steel or concrete bridges, unless documented environmental restrictions for natural streambed preservation control.

(b) The entire culvert, wingwalls, apron and incidental excavation and backfill for the box, wings and aprons shall be included in the lump sum culvert item, except for reinforcement bars which are "AND Items". List culvert length in meters {feet}, excavation and backfill in cubic meters {cubic feet}, and concrete quantities for wings and aprons in terms of cubic meters {cubic feet} in the quantity table as information items and on the Required Item Schedule (pink sheets) in the Proposal.

(c) Provide riprap protection beyond culvert apron as per BD-632M.

7.3.3 Corrugated Metal Buried Structures

Corrugated metal buried structures may be used on local and collector roads with ADT of less than 750. This applies to both Federal and State-funded projects. Structural plate pipe and arches covered by BD-635M may be used for all highways, regardless of ADT, except where ADTT > 500. Chief Bridge Engineer approval is required for corrugated metal buried structures exceeding these limitations.

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 1
PART B: DESIGN SPECIFICATIONS

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 1
PART B: DESIGN SPECIFICATIONS

PREFACE

In Part B, the AASHTO LRFD article numbering system is followed. Where a new article has been added, the suffix P, to designate "Pennsylvania Article", appears at the end of the new article number.

All references to AASHTO LRFD sections, articles, equations, figures or tables carry the prefix A.

References to AASHTO LRFD commentary carry the prefix AC.

References to Design Manual, Part 4, Volume 1, Part A, "Policies and Procedures", carry the prefix PP.

References to Design Manual, Part 4, Volume 1, Part B, "Design Specifications", carry the prefix D.

References to commentary to Design Manual, Part 4 carry the prefix DC.

SECTION 1 - INTRODUCTION

SECTION 1 - TABLE OF CONTENTS

SPECIFICATIONS

1.1 SCOPE OF THE SPECIFICATIONS

The following shall supplement A1.1.

The design of any structure, or portion thereof, which is not covered by the DM-4, Part B, or the AASHTO LRFD Specifications shall be designed by other appropriate PennDOT or AASHTO documents as specified in PP1.1 or applicable Specifications and Standards approved by the Chief Bridge Engineer.

1.2 DEFINITIONS

The following shall supplement the definition of Design Life.

Bridge life is assumed to be 100 years for the main load carrying members of the superstructure and substructure. The design life of decks, barriers and expansion devices are considerably less.

1.3 DESIGN PHILOSOPHY

1.3.1 General

The following shall supplement A1.3.1.

An elastic analysis shall be used to determine the force effects. The resistance of components and connections may be determined using inelastic behavior in the Extreme Event Limit State.

1.3.2 Limit States

1.3.2.1 GENERAL

The following shall supplement A1.3.2.1.

In no case shall the value of η be less than 1.0 nor shall it be greater than 1.0.

When the Load Combination Extreme Event III and IV is investigated, the value of η shall be taken as 1.0.

COMMENTARY

C1.3.2.1

The following shall supplement AC1.3.2.1.

In Pennsylvania, the application of load modifiers other than 1.0 has been found to have a detrimental affect on economical bridge design without a corresponding increase in comfort level.

1.3.3 Ductility

The following shall replace the fourth paragraph of A1.3.3.

For all components for all limit states on all bridges, the ductility load modifier, η_D shall be taken as 1.00.

C1.3.3

The following shall replace the last sentence of the second paragraph of AC1.3.3.

Such ductile performance shall be verified by testing and approved by the Chief Bridge Engineer.

The following shall replace the first sentence of the seventh paragraph of AC1.3.3.

The ductility capacity of structural components or connections may either be established by full or large scale testing, or with analytical models which are based on documented material behavior, but either method must be approved by the Chief Bridge Engineer.

SPECIFICATIONS

COMMENTARY

1.3.4 Redundancy

C1.3.4

The following shall replace the fourth paragraph of A1.3.4.

For all bridges, the redundancy load modifier, η_R shall be taken as 1.00.

Global redundancy for truss bridges shall be provided by any of the following or combination thereof:

- provision of a third line of trusses where possible,
- use of stitched built-up components which are designed to support the entire component load with any one element assumed to be broken, and for which joints and splices have been designed to transmit component loads with any one element of the component assumed to be broken or,
- demonstration through 3-D analysis that failure of any tension component, or other components designated by the Department, of a two-truss system will not cause the collapse of the entire structure.

When the above provision for a stitched built-up component is invoked, the investigation shall be based on Load Combination Extreme Event III as specified in D3.4.1. When the above provision for 3-D analysis is invoked, the investigation shall be based on Load Combination Extreme Event IV as specified in D3.4.1. All live load models, specified in D3.6.1.2, shall be investigated. Approval by the Chief Bridge Engineer shall be required for all two truss systems.

The tie component of tied arched bridges shall meet the requirements of the above provision for a stitched built-up component.

Tied arched bridges will be considered globally redundant by meeting this requirement.

Global redundancy for multiple element hangers composed of multiple bridge strands or multiple bridge rope shall be provided by demonstration through 3-D analysis that failure of any tension component, or other components designated by the Department, will not cause the collapse of the entire structure. This analysis shall use Load Combination Extreme Event IV as specified in D3.4.1. All live load models, specified in D3.6.1.2, shall be investigated.

A two-girder bridge shall be used, only if (1) fracture-critical members are eliminated by developing alternative load paths and (2) approval is obtained from the Chief Bridge Engineer. The designer should evaluate the ability of secondary members to transfer loads and prevent collapse through the use of suitable computer analysis.

A two-girder bridge has the potential for fracture-critical members, i.e., a failure of one girder appears to cause collapse of the entire structure. There are certain situations, however, in which use of two girders may provide a significant economy.

For both two- and three-girder systems, computer-aided designs have been completed where secondary members are designed to transfer load around a failed portion of a girder, thereby preventing a collapse of the bridge.

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1.3.5 Operational Importance

The following shall replace the third paragraph of A1.3.5.

For all bridges, the operational importance load modifier, η_I, shall be taken as 1.0.

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2.3 LOCATION FEATURES

2.3.2 Bridge Site Arrangement

2.3.2.2 TRAFFIC SAFETY

2.3.2.2.1 Protection of Structures

The following shall supplement A2.3.2.2.1.

The minimum lateral clearance for locating substructure units for bridges over highways or railroads shall be as follows:

(a) For bridges over a highway, refer to Design Manual Part 2, Chapter 4.4.C for the minimum horizontal clearance for locating substructure units.

(b) For bridges over railroads, see D2.3.3.4.

2.3.2.2.2 Protection of Users

The following shall replace the last paragraph of A2.3.2.2.2.

Sidewalks on bridges shall be protected by barriers unless approved by the Department.

COMMENTARY

C2.3.2.2.2

The following shall replace the last paragraph of AC2.3.2.2.2.

An example where the Department may waive the sidewalk barrier requirement would be in an urban environment where a curbed approach walkways exists and the posted vehicular speed is less than or equal to 50 km/hr {30 mph}.

2.3.2.2.3 Geometric Standards

The following shall replace the last sentence of A2.3.2.2.3.

For roadway geometry refer to Design Manual, Part 2. For bridge-mounted barriers, structure-mounted guide rail and other protective devices refer to appropriate Standard Drawings.

2.3.3 Clearances

2.3.3.2 HIGHWAY VERTICAL

The following shall replace the first sentence of the first paragraph of A2.3.3.2.

Refer to Design Manual, Part 2, for minimum vertical clearance for overhead bridges.

The following shall supplement A2.3.3.2.

Vertical clearance over the width of the roadway, including shoulders, shall be provided in accordance with Design Manual, Part 2, Chapter 2, Section 2.21, and as follows:

- For bridges over railroads, see D2.3.3.4.
- Minimum required vertical clearance shall preferably be

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maintained within the recovery area. In calculating actual vertical clearance under a beam splice, an allowance of 20 mm {3/4 in.}, plus the thickness of the outside flange splice plate, shall be made.

- Vertical clearance is to be measured at any high point on the outer edge shoulder. 30 mm {1 in.} will be deducted from the actual measurement for posting purposes to account for minor variations as proper location of measurement and jumping of traveling vehicles.
- For vertical sag under the bridge, ensure that the vertical clearance is measured from a chord between any two high points along the traveling direction to account for the maximum truck length permitted on the road.

For prestressed concrete beams, do not take credit for beam camber in determining actual vertical clearance, unless the beam is cast or assembled specifically to provide vertical curvature of the bottom of the beam.

2.3.3.3 HIGHWAY HORIZONTAL

The following shall supplement the first paragraph of A2.3.3.3.

Refer to Design Manual, Part 2, for additional details on bridge widths, including criteria for bridges on Very Low Volume Roads.

2.3.3.4 RAILROAD OVERPASS

The following shall supplement A2.3.3.4.

Pennsylvania Public Utility Commission (PUC) has jurisdiction on railroad overpass clearances.

Refer to Design Manual, Part 1A, Chapter 7, Section 12, for additional details where railroad are overpassed by a highway structure.

Structures carrying railroad tracks shall be designed according to AREMA specifications and the modifications adopted by the railroad system involved.

For structures carrying highways over railroad tracks, the minimum horizontal clearance, specified and provided, from the centerline of track shall be in accordance with Publication 371 and/or Railroad Form D-4279 to face of abutment or pier and shall be shown on the drawings. A 5500 mm {18'-0"} lateral clearance from the centerline of track shall be provided for off-track equipment on one side if requested by the railroad. Class 1 (major) railroads may require additional lateral clearance depending upon the need for drainage ditches and the roadway for off-track equipment. If track and abutment or piers are skewed relative to each other, horizontal clearances to the extremities of the structure shall also be shown. If the track is on a curve within 24 400 mm {80 ft.} of the crossing, additional horizontal clearance is required to compensate for the curve (refer to AREMA, Volume 2, Chapter 28). If a railroad requests clearance in excess of the

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above, complete justification of this request shall be provided. The agreement on the lateral and vertical clearances shall be reached with the operating railroad, or the determination from the PUC shall be secured prior to submitting for TS&L approval.

The minimum vertical clearance over the top of rail shall be in accordance with Publication 371 and/or Railroad Form D-4279 and shall be shown for each track on the drawings. If track and abutments or piers are skewed relative to each other, vertical clearances to the extremities of the structure should also be shown. Approval for any exception to the above minimum clearance over railroad tracks shall be secured from the operating railroad company or the PUC prior to submitting for TS&L approval.

To provide for a drainage ditch parallel to track, the elevation of the top of footings adjacent to track shall be at least 1100 mm {3'-6"} below the elevation of the top of rail, unless rock is encountered.

The edge of footing shall be at least 2200 mm {7 ft.} from the centerline of adjacent track.

If pier bents are used between 5480 mm {18 ft.} and 7620 {25 ft.} from the centerline of tracks, columns shall be protected by crash walls at least 760 mm {2'-6"} thick, which shall extend 3048 mm {10 ft.} above the top of rail and 1828 mm {6 ft.} for single column or 762 mm {2'-6"} for multi-column bents beyond the outside face of outside columns. The crash wall shall rest upon the column footings, extend 152 mm {6 in.} from the face of columns adjacent to traffic, and shall connect all columns in a pier bent. Solid piers with a minimum thickness of 760 mm {2'-6"} and a length of 6100 mm {20 ft.}, single column piers of minimum 1220 mm x 3800 mm {4'-0" x 12'-6"} dimensions or any solid pier sections with equivalent cross sections and minimum 760 mm {2'-6"} thickness negate the need to provide crash walls. Reinforcement to be designed in accordance with A3.6.5.2, but not less than horizontal bars of No. 19 at 300 mm {No. 6 at 12 in.} each face, and vertical stirrups of No. 13 at 300 mm {No. 4 at 12 in.}. Crash walls meeting the same dimension and reinforcement requirements as above shall also be provided in front of prefabricated walls.

Bridge scuppers shall not drain onto railroad tracks. Provision shall be made to direct surface water from the bridge area into an adequate drainage facility along the railroad track, in which case drainage approval by the railroad company is required prior to submission of final plans.

Safety provisions required during excavation in the vicinity of railroad tracks and substructures shall be in accordance with the Special Provision "Maintenance and Protection of Railroad Traffic".

Sheet piling used during excavation for protection of railroad tracks and substructure shall be designed according to AREMA specifications and shall be subject to approval by the railroad company. The use of caisson footings shall be evaluated in lieu of sheet piling and deep foundation.

Complete details of temporary track(s) or a temporary railroad bridge to be constructed by the Department's

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contractor shall be shown on the design drawings. Applicable railroad design standards or design drawings shall be referred to or duplicated on the design drawings.

For highway structures with sidewalks, protective fencing shall be provided on all structures crossing over railroads. The protective fence shall extend at least 2440 mm {8 ft.} from top of sidewalk or driving surface adjacent to the barrier wall. The fence may be placed on top of the barrier wall.

All railroad clearances shall be based on the railroad's current design criteria.

For electrified railroad tracks, these additional requirements apply:

- If a railroad is electrified, it shall be so noted on the preliminary plans submitted for type, size and location approval.

- Protective barrier shall be provided on spans or on part of spans for structures over electrified railroads, as directed by the railroad company. Generally, the protective barrier shall extend at least 3100 mm {10 ft.} beyond the point at which any electrified railroad wire passes under the bridge. However, in no case shall the end of the protective barrier be less than 3100 mm {10 ft.} from the wire measured in a horizontal plane and normal to the wire outside of the limit of the bridge, and less than 1900 mm {6 ft.} from the wire within the limit of the bridge.

- Details of protective barriers are shown on Standard Drawing BC-711M. If conditions warrant or if directed by the railroad company, details shall be modified. Such modifications shall be shown on the design drawings.

- All open or expansion joints in the concrete portion of barriers, divisors, sidewalks, and curbs within the limits of the barrier shall be covered or closed with joint materials. Details of such joints shall be shown on the design drawings.

- In the case of bridges crossing electrified railroad tracks, the details of catenary attachments and their locations, if attached or pertinent to the structure, shall be shown on the plans. Consideration shall be given to realign the catenary by installing support columns on each side of the bridge to avoid catenary attachments to the bridge. Normally, ground cable attachments, cables, miscellaneous materials, etc. are supplied by the contractor and are installed by the railroad. A separate block identifying materials required, description of materials, railroad reference number for materials, and party responsible for providing or installing materials shall be shown on the plans. Approval of grounding plans shall be obtained from the railroad concurrently with approval of the structure drawings.

Where the PUC has jurisdiction over the structure

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involved, PUC Docket Number, either A.____ or C.____ (A stands for Application and C stands for complaint) shall be shown on the first sheet of the design drawings (S-drawings) above the title block, after the PUC has approved the plans.

The responsible designer shall add the PUC Docket Number on all plans, and the District shall add the PUC Docket Number in BMS where the PUC has jurisdiction over the structure involved.

Where applicable, the USDOT/AAR number of the existing structure shall be shown on the first sheet of the design drawings.

2.5 DESIGN OBJECTIVES

2.5.2 Serviceability

2.5.2.2 INSPECTABILITY

D2.5.2.2.1P, D2.5.2.2.2P, D2.5.2.2.3P and D2.5.2.2.4P shall replace A2.5.2.2.

2.5.2.2.1P General

Inspection and maintenance instructions and requirements for critical details shall be stipulated on the plans. The plans shall also include reference to the method of access for inspection of the subject details.

It is necessary to have adequate means of access for bridge safety inspection. Review bridge designs for inspectability at TS&L, final design and construction stages. PP3.6.6 provides information for checking a structures inspectability with PennDOT's underbridge crane.

For special bridge conditions, the inspectability shall be as determined by the Chief Bridge Engineer.

2.5.2.2.2P Inspection Walks

2.5.2.2.2aP General

Unless approved otherwise, inspection walks shall be provided for long bridges (over 300 000 mm {1,000 ft.}) which cannot be readily inspected using inspection crane or which are otherwise inaccessible from underneath. Generally, inspection walks are required under the following conditions:

- Bridge width over 18 000 mm {60 ft.}, inaccessible from underneath
- Superstructure depth over 3500 mm {11.5 ft.}, including beams, barrier, railing or fencing, and noise walls, inaccessible from underneath
- High bridge underclearance (in excess of 9000 mm {30 ft.}), particularly for bridges over large rivers

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2.5.2.2.2bP Design Live Load

A minimum design live load of 3.8 kN/m^2 {80 psf} shall be used.

2.5.2.2.2cP Geometry

The minimum width shall be 1200 mm {4 ft.}. The minimum overhead clearance shall be 1800 mm {6 ft.}. Toe guard protection and railing shall be provided if the walk is not protected by the girders. The walk shall be secured against vandalism and shall not provide entrance to the general public. The entrances shall be locked or secured against access. Provision shall be made to cross from one bay to the next, generally at pier locations for at least one person, 1800 mm {6 ft.} in height, carrying tools and equipment.

2.5.2.2.2dP Connection to the Main Members

Generally, a bolted or threaded insert type of connection shall be provided to secure the walks in position. Lock washers or another positive connection device shall be specified to protect the connection from being loosened due to bridge vibration.

2.5.2.2.3P Inspectability for Enclosed Section

Vent holes and large size (600 mm by 900 mm {2 ft. by 3 ft.} minimum and 900 mm by 1200 mm {3 ft. by 4 ft.} desirable opening) inspection hatches shall be provided for large-span box structures; provision for lighting, cross ventilation, and steps shall be made where required. Large box sections shall have a coat of white paint on the interior.

2.5.2.2.4P Girder Handrail

Where other inspection facilities are not provided, handrails shall be attached to the web of steel girders greater than 1800 mm {72 in.} in depth.

2.5.2.2.5P Sound Barriers

Sound barriers shall be designed and detailed to maintain bridge inspectability. For special conditions, the inspectability shall be determined by the Chief Bridge Engineer.

2.5.2.6 DEFORMATIONS

2.5.2.6.1 General

The following shall replace A2.5.2.6.1.

Bridges shall be designed to avoid undesirable structural or psychological effects due to their deformations.

COMMENTARY

C2.5.2.6.1

The following shall replace AC2.5.2.6.1.

Service load deformations may cause deterioration of wearing surfaces and local cracking in concrete slabs and in metal bridges which could impair serviceability and durability, even if self limiting and not a potential source of collapse.

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2.5.2.6.2 Criteria for Deflection

The following shall replace A2.5.2.6.2.

In applying criteria for deflection, the vehicular load shall include the dynamic load allowance.

To control deflections of structures, the following principles shall apply:

- when investigating the maximum absolute deflection, all design lanes should be loaded, and all supporting components should be assumed to deflect equally,

- for composite design, the design cross-section should include the entire width of the roadway, neglecting any stiffness contribution by barrier, railing or other secondary members of the bridge,

- when investigating maximum relative displacements, the number and position of loaded lanes should be selected to provide the worst differential effect,

- the live load portion of Load Combination Service I of Table A3.4.1-1 should be used, including the dynamic load allowance, IM

- the live load shall be taken from D3.6.1.3.2,

- the provisions of A3.6.1.1.2 should apply,

- for skewed bridges, a normal cross-section may be used; for curved and curved skewed bridges a radial cross-section may be used.

The following deflection limits shall be used for steel, aluminum and/or concrete construction:

- vehicular load, general..................................... Span/800

- vehicular and/or pedestrian loads.................. Span/1000

- vehicular load on cantilever arms Span/300

- vehicular and/or pedestrian loads on cantilever arms... Span/375

For steel I-shaped beams and girders, the provisions of A6.10.4 and A6.11.4, respectively, regarding the control of permanent deflections through flange stress controls, shall apply.

The following deflection limits shall be used for wood construction:

- vehicular load, general..................................... Span/425

- vehicular and/or pedestrian loads..................... Span/800

COMMENTARY

C2.5.2.6.2

The following shall replace AC2.5.2.6.2.

For a multi-beam bridge, this is equivalent to saying that the distribution factor for deflection is equal to the number of lanes divided by the number of beams.

The weight of barrier, railing or other secondary members shall be included for deflection and design. Only the stiffness of these items should be neglected.

From a structural viewpoint, large deflections in wood components cause fasteners to loosen and brittle materials, such as asphalt pavement, to crack and break. In addition, members that sag below a level plane present a poor appearance and can give the public a perception of structural inadequacy. Deflections from moving vehicle loads also produce vertical movement and vibrations that annoy motorists

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- vehicular load on wood planks and panels: extreme relative deflection between adjacent edges.......................................2.5 mm {1/8 in.}

The following provisions shall apply to orthotropic plate decks:

- vehicular load on deck plate Span/300
- vehicular load on ribs of orthotropic metal decks.. Span/1000
- vehicular load on ribs of orthotropic metal decks: extreme relative deflection between adjacent ribs........................... 2.5 mm {1/8 in.}

2.5.2.6.3 Criteria for Span-to-Depth Ratios

The following shall replace the first paragraph of A2.5.2.6.3.

Structures or components of structures shall satisfy the span-to-depth ratios given in Table A2.5.2.6.3-1

where:

S = slab span length (mm) {ft}

L = span length (mm) {ft}

Concrete decks on multi-girder-type bridges shall satisfy the span-to-depth ratios in Table A1 with the heading "Slabs".

2.5.2.7 CONSIDERATION OF FUTURE WIDENING

2.5.2.7.1 Exterior Beams on Multi-Beam Bridges

The following shall replace A2.5.2.7.1.

The load carrying capacity of exterior beams shall not be less than the load carrying capacity of an interior beam, unless specifically approved by the Chief Bridge Engineer.

2.5.3 Constructibility

The following shall supplement A2.5.3.

An acceptable slab placement sequence shall be shown in the contract plans. Figure 1 illustrates the format to be used. The actual sequence shall be determined from an erection analysis (see D6.10.3.2.4P) for the specific structure in question. For steel girder structures and prestressed beams made continuous for live load, see D6.10.3.2.4P and D5.14.1.2.7fP, respectively, for additional requirements concerning slab placement sequence. The required curing strength (if any) of the concrete of a previous placement segment shall be designated as appropriate. Instead of specifying a curing strength of the concrete, a time delay (if

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and alarm pedestrians, Ritter (1990).

Excessive deformation can cause premature deterioration of the wearing surface and affect the performance of fasteners, but limits on the latter have not yet been established.

The intent of the relative deflection criterion is to protect the wearing surface from debonding and fracturing due to excessive flexing of the deck. The restriction on relative rib displacement may be revised or removed when more data is available to formulate appropriate requirements as function of thickness and physical properties of the wearing surface employed.

C2.5.2.7.1

The following shall supplement A2.5.2.7.1.

The stiffness of the interior and exterior beams should be relatively equal.

C2.5.3

The two-curing day waiting period between pours in adjacent continuous spans is for crack control. Studies have shown that longer waiting periods have no significant effect on cracking, primarily because shrinkage is the dominating factor in cracking. The two-curing day period between adjacent pours in the same span will provide enough strength gain to introduce composite action and will increase the stability of the girder over the length of the previous pour.

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any) between placements may be designated, as appropriate. A minimum waiting period of two curing days between positive moment region placements in immediately adjacent continuous spans and between adjacent positive moment placements in the same span shall be specified in the contract plans. A minimum waiting period between other placements need not be specified if analysis suggests that it is unnecessary.

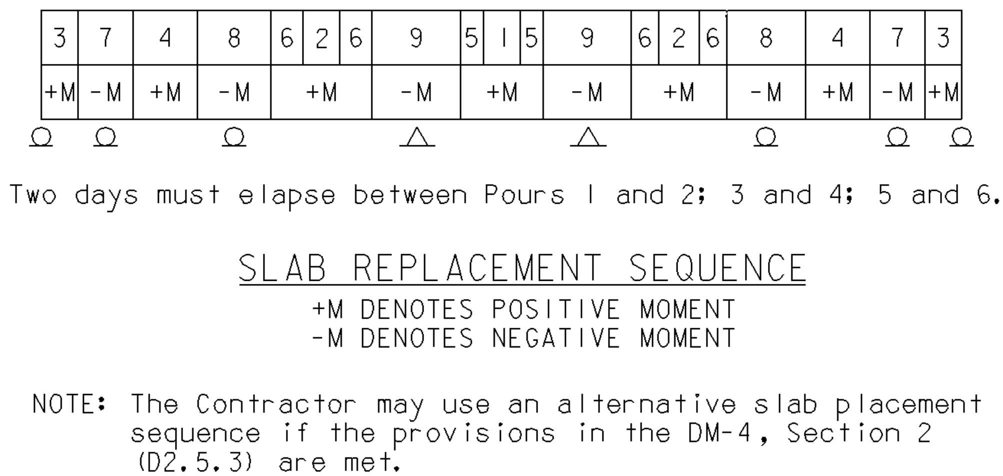

Figure 2.5.3-1 - Example of Slab Placement Sequence for Contract Plans

The Contractor may use an alternate slab placement sequence if the following provisions are met:

- The Contractor shall submit to the Department a revised slab placement sequence with support calculations and computer stress analysis. The revised slab placement sequence shall meet the requirements of which the original slab placement sequence were based on.
- The Department will review and approve calculations.
- The Contractor shall receive written approval prior to the use of the revised slab placement sequence and/or camber values.
- All costs for the development and approval of the revised slab placement sequence and camber values shall be borne by the Contractor.
- The Department will be the sole judge of the acceptability of the revised slab placement sequence and camber values.

2.5.3.1P FALSEWORK

Composite beams shall be designed with no intermediate falsework during placing and curing of the concrete deck.

When falsework is used on a project, it shall be designed

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C2.5.3.1P

For overhang bracket requirements see D6.10.3.2.4.2P.

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for the following items, but not limited to:

- vertical loads,
- horizontal loads,
- differential settlement forces,
- unbalanced temporary loadings (e.g., staged construction), and
- errant highway vehicles.

The following guidelines should be used for the approval of the falsework:

- Every bridge on a project should receive a separate falsework design analysis.
- In the event falsework is moved from one bridge to another, it should be thoroughly inspected for structural damage and plumbness to ensure that all members are in place and properly aligned and corrected.
- Ensure that the requirement of Publication 408, Section 105.02(c), "all drawings for load bearing falsework submissions are to be signed and sealed by a Professional Engineer, registered in the Commonwealth of Pennsylvania", is fully enforced.
- During the falsework review, make sure that it is designed to handle vertical and horizontal loading and to contain enough redundancy to prevent a failure in the entire system. Vertical loading and differential settlement forces, and horizontal lateral and longitudinal forces should be taken into account. Unbalanced temporary loading caused by the placement sequence, should also be considered.
- If an unfortunate event occurs due to the failure of the falsework, preserve and document the in-place failure and assign investigation responsibilities to qualified impartial parties.
- If service load design is used, designers may increase the allowable basic unit stress by 25% for temporary falsework.

2.6 HYDROLOGY AND HYDRAULIC

2.6.6 Roadway Drainage

2.6.6.1 GENERAL

The following shall supplement A2.6.6.1.

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Refer to Appendix P for guidance on jacking and supporting the superstructure.

For purposes of these guidelines, temporary falsework is defined as falsework constructed for no more than one construction season.

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Dimensions for the deck cross slopes shall be shown in the same manner as indicated on the roadway plans (e.g., 2%). The water table cross slope on bridge decks which are not superelevated shall be sloped toward the curb or median. The rate of slope shall be 4% for water table widths of 1800 mm {6 ft.} or less, and 3% for water table widths over 1800 mm {6 ft.}. On superelevated decks, the water table on the high side shall be as specified in Design Manual, Part 2. On the low side, the water table shall slope in the same direction and magnitude as the adjacent lane, but not less than 4% for water table widths of 1800 mm {6 ft.} or less, nor less than 3% for water table widths over 1800 mm {6 ft.}. On a superelevated bridge with a paved median, adjustment of the grades of adjacent roadways may be required to equalize the height of the divisor or median barrier.

2.7P DESIGN DRAWINGS

2.7.1P Moment and Shear Envelope Diagrams

For simple span bridges, the contract plans shall contain a table of following items:

(1) Maximum non-composite dead load moment

(2) Maximum composite dead load moment (including future wearing surface)

(3) Maximum live load plus impact moment for PHL-93 and P-82 loading conditions

(4) Maximum non-composite dead load shear

(5) Maximum composite dead load shear (including future wearing surface)

(6) Maximum live load plus impact shear for PHL-93 and P-82 loading conditions

(7) Composite and non-composite section properties at resisting sections

For multiple span continuous bridges, as a minimum, the contract plans shall contain a diagrammatic presentation of the following on a per-girder basis:

(1) Non-composite dead load moment diagram

(2) Composite dead load moment diagram (including future wearing surface)

(3) Separate positive and negative live load plus impact moment envelope for PHL-93 and P-82 loading conditions

(4) Summation of (1) and (2)

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(5) Non-composite dead load shear diagram

(6) Composite dead load shear diagram (including future wearing surface)

(7) Separate positive and negative live load plus impact shear envelope for PHL-93 and P-82 loading conditions

(8) Summation of (5) and (6)

(9) Composite and non-composite section properties at the resisting sections

The data to construct this presentation shall not have load factors applied.

Also, a table of reactions shall be provided for total dead load, and positive and negative live load, plus impact without load factors applied.

2.7.2P Major, Complex and Unusual Bridges

2.7.2.1P LOAD DATA SHEET

For the new design of major, complex and unusual bridges, additional information shall be included on the design drawings for typical common components such as bearings, floorbeams, and stringers. For these items as a minimum, the contractor plans shall contain a tabular presentation of the following:

- Bearings

  (a) Vertical Force

  1. Total Dead Load
  2. Live Load plus Impact
  3. Summation of 1. and 2.

  (b) Transverse Force Wind Load

  (c) Longitudinal Force

  1. Wind Load
  2. Traction Load
  3. Friction Load

- Truss Members

  (a) Axial Force

  1. Total Dead Load
  2. Live Load plus Impact
  3. Summation of 1. and 2.

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(b) Bending Moment

1. Total Dead Load
2. Live Load plus Impact
3. Summation of 1. and 2.

(c) Section Properties

1. Gross Area
2. Net Area
3. Section Modulus

- Floorbeam

  Provide the same type of information as required in D2.7.1P for girders.

- Stringers

  Provide the same type of information as required in D2.7.1P for girders.

The data for this tabular presentation shall not have load factors applied.

2.8P BRIDGE SECURITY

2.8.1P General

For the purpose of this section bridges deemed important shall include (1) new singular bridges of total replacement value exceeding $100 million or (2) new or existing bridges identified as critical by the Department's Emergency Transportation Operations Section.

A risk management approach shall be utilized to assess structural vulnerability and countermeasures. A threat based component level analysis shall be conducted considering a full range of threats. FHWA's workshop methodology or other BQAD approved methodology shall be used.

The results of such processes and included design/mitigation criteria and features shall be considered sensitive project information that is protected by restricted access. Contract documents shall not include reference to any security standard, design capability, or other information that might provide knowledge of bridge resistance.

For all bridges, restrict access to doors/hatches by using locks or by making inaccessible accept by special mobile equipment, such as snooper, man lift, etc. In cellular structures, avoid vent hole diameters larger than 2 inches when holes are in easy reach. Avoid nooks and areas that allow for concealed access and create a confined pressure effect. If these details are unavoidable consideration should be given to barring access.

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SECTION 3 – LOADS AND LOAD FACTORS

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C3.1 SCOPE

The following shall supplement the second paragraph of AC3.1.

Before any test results are used in the design of a structure, the tests and the test results must be approved by the Chief Bridge Engineer

3.3 NOTATION

3.3.1 General

The following shall supplement A3.3.1

B = Vertical element width (mm) {ft} (3.11.5.6)
C = Pressure coefficient for loads applied on a subgrade (dim) (3.6.1.5.2P)
D = Depth of embedment of discrete and continuous vertical wall elements (mm) {ft} (3.11.5.6); width of uniformly loaded area (mm) {ft} (3.6.1.5.2P)
F = Fictitious force applied at bottom of embedded continuous vertical wall element to provide horizontal force equilibrium for simplified earth pressure distributions (N/mm) {kip/ft} (3.11.5.6)
I_f = Impact factor (dim) (3.6.1.5.2P)
k_p = Passive coefficient of lateral earth pressure (dim) (3.11.5.6)
ℓ = Spacing between vertical wall elements (c/c) (mm) {ft} (3.11.5.6)
M = Length of uniformly loaded area (mm) {ft} (3.6.1.5.2P)
M = constant used in calculating active earth pressure coefficient in certain conditions (dim) (3.11.5.6.2bP)
P_a = Active resistance per vertical wall element (N) {kips}
P_a = Active earth pressure per unit length of wall (N/mm) {kip/ft} (3.11.5.6)
P_o = Intensity of the distributed load at the bottom of the railroad ties (MPa) {ksi} (3.6.1.5.2P)
P_p = Passive resistance per vertical wall element (N) {kips}
P_p = Passive earth pressure per unit length of wall (N/mm) {kip/ft} (3.11.5.6)
S_m = Shear strength of rock mass (MPa) {ksf} (3.11.5.6)
S_u = Undrained shear strength of cohesive soil (MPa) {ksf} (3.11.5.6)
W_l = Live load on structure from railroad loading (N/mm) {kip/ft} (3.6.1.5.2P)
B = Ground surface slope behind wall {+ for slope up from wall; - for slope down from wall} (DEG) (3.11.5.6)
β' = Ground surface slope in front of wall {+ for slope up from wall; - for slope down from wall} (DEG) (3.11.5.6)

3.4 LOAD FACTORS AND COMBINATIONS

3.4.1 Load Factors and Load Combinations

The following shall replace Strength II description in A3.4.1.

- Strength II - Load combination relating to the Design Permit Load (P-82) use of the bridge. This load combination only applies to the superstructure, except for pier caps which support a superstructure with a span length greater than 20 000 mm {65 ft.}. Bearings, (including uplift check), substructure and foundation need not be designed for this load combination. For design, the distribution factors given in

C3.4.1

The following shall replace Strength II commentary in AC3.4.1.

In design, the use of distribution factors in D4.6.2.2 and A4.6.2.2 represents that the P-82 is in all design lanes.

The method for rating takes into account that the P-82 is in one lane and the other lanes are occupied by the vehicular live load.

In AC3.4.1, the commentary for Strength II states that "For bridges longer than the permit vehicle, the presence of the design lane load, preceding and following the permit load in its lane, should be considered." A study done for the Department showed that the P-82 with the interrupted lane

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D4.6.2.2 and A4.6.2.2 shall be used.

For the rating of existing bridges with Strength II criteria, the following equation may be used to determine Strength II live load moments and shear:

$$FR_T = FR_{P-82}\left(\frac{g_1}{1.2}\right) + FR_{PHL-93}\left(g - \frac{g_1}{1.2}\right)$$

where:

FR_T = total force response, moment or shear

$FR_{P\text{-}82}$ = P-82 force response, moment or shear

$FR_{PHL\text{-}93}$ = PHL-93 force response, moment or shear

g_1 = single lane distribution factor, moment or shear

g = multi-lane distribution factor, moment or shear

FR_T need not be taken greater than $FR_{P\text{-}82(g)}$.

The following shall supplement A3.4.1.

- STRENGTH IP - Load combination relating to the pedestrian load and a reduced vehicular live load.
- STRENGTH VI - Load combination relating to the design of piers which includes ice and wind load acting together.
- EXTREME EVENT III - Load combination relating to the failure of one element of a component without the failure of the component.
- EXTREME EVENT IV - Load combination relating to the failure of one component without the collapse of the structure.

The conditions for which Extreme Event III and IV are to be investigated are given in D1.3.4.

COMMENTARY

load only controls for moments in a small range of spans and is only maximum of 2% above the PHL-93 loading. For shear, the maximum difference between the PHL-93 and P-82 with lane load was 7.5% with P-82 and lane load being greater than PHL-93. The Department concluded that this difference was acceptable because the study considered all the lanes loaded with the P-82. Therefore, the P-82 loading need not be considered with a partial lane load.

The following shall supplement AC3.4.1.

Extreme Events III and IV are uncalibrated load combinations. They are intended to force consideration of the safety of damaged structures.

For this extreme event, a 3-D analysis is required. The objective of this analysis is survival of the bridge (i.e., the bridge may have large permanent deflections, but it has not collapsed).

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Table 3.4.1P-1 - Additional PennDOT Load Combinations and Load Factors

Load Combination / Limit State	DC DD DW EH EV ES	LL IM CE BR PL LS	WA	WS	WL	FR	TU CR SH	TG	SE	Use One of These at a Time			
										EQ	IC	CT	CV
STRENGTH IP	γ_p	*	-	-	-	-	-	-	-	-	-	-	-
STRENGTH VI	γ_p	-	1.25	1.25	-	-	-	-	-	-	1.25	-	-
EXTREME EVENT III	γ_p	γ'_{LL}	-	-	-	-	-	-	-	-	-	-	-
EXTREME EVENT IV	γ'_p	γ'_{LL}	-	-	-	-	-	-	-	-	-	-	-
*γ_{LL} = 1.35, γ_{PL} = 1.75													

Table 3.4.1P-2 - Load Factor for Live Load for Extreme III and IV, γ'_{LL}

Case	III	IV
	γ'_{LL}	γ'_{LL}
PHL-93 Loading – all applicable lanes	1.30	1.15
Permit load in governing lane with PHL-93 in other applicable lanes	1.10	1.05

Table 3.4.1P-3 - Load Factors for Permanent Loads for Extreme Event IV, γ'_p

Type of Load	Load Factor	
	Maximum	Minimum
DC: Component and Attachments	1.05	0.95
DW: Wearing Surfaces and Utilities	1.05	0.90

Unless otherwise specified, interaction of force effects shall be accounted for by selecting load factors which maximize and minimize each of the force effects one at a time with the same load factors used to compute the associated force effect.

As an example for a design which involves the interaction of moment and axial force, the following four design cases would be investigated:

- select the load factors which maximize moment and use these load factors in determining axial force
- select the load factors which minimize moment and use these load factors in determining axial force
- select the load factors which maximize axial force and use these load factors in determining moment
- select the load factors which minimize axial force and

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use these load factors in determining moment.

Due to the nature of force interaction, the absolute worst case may not necessarily be that for which one of the force effects is maximized, but an intermediate case. However, the difference between the absolute worst case and the design cases presented here are believed to be within the tolerance of the design process. Therefore, as a reasonable interpretation of the specification, maximum and minimum force effects taken in conjunction with associated force effects for interaction are to be considered. If the Engineer believes that an intermediate case will govern to an appreciable degree, the Engineer shall notify the Chief Bridge Engineer. Then, the Chief Bridge Engineer will determine if intermediate cases shall be investigated.

For MSE wall designs, D11.10.5.2 and D11.10.6.2 state when to apply maximum and minimum EH and EV.

The following shall supplement the sixth paragraph of A3.4.1 relating to TU, CR and SH.

The larger load factor shall be used to determine the final length of the member. The smaller load factor shall be used in determining force effects, such as creep and shrinkage effects in pier caps and columns.

The following shall replace the ninth paragraph of A3.4.1 relating to γ_{TG} and γ_{SE}.

For the application of temperature gradient see D3.12.3. The load factor for settlement γ_{SE} shall be determined on a project-specific basis.

The following shall replace the eleventh paragraph of A3.4.1 relating to γ_{EQ}.

The load factor γ_{EQ} for live load for the Extreme Event-I limit state shall be taken as 0.0.

The Department is currently using γ_{EQ} = 0.0 in accordance with numerous past years of AASHTO practice. We will continue to use γ_{EQ} = 0.0 until further work is completed justifying a different value.

The following shall supplement A3.4.1 for the design of box culverts.

Lateral earth pressures for box culverts shall be computed using the equivalent fluid method given in A3.11.5.5 and D3.11.5.5, and appropriate load factors, EH, as given in Table 3.4.1-2, for horizontal earth pressures.

To maximize the load effect, the maximum at-rest load factor shall be used with the maximum equivalent fluid weight from Table D3.11.5.5-2, and the minimum at-rest load factor shall be used with the minimum equivalent fluid weight. In addition, a 50% reduction in both maximum and minimum unfactored lateral earth pressures, EH and ES, shall be considered for determining the maximum positive moment in the top slab of the culvert, as specified in A3.11.7.

In Table A3.4.1-2, the first bulleted item under EV: Vertical Earth Pressure, “Retaining Structures”, shall be changed to “Retaining Walls and Abutments”.

The following shall supplement AC3.4.1 for the design of box culverts.

Rigid frame action of box culvert structures is assumed to result in relatively small movements as compared to that of a retaining wall or abutment-type structure, thus, an at-rest condition is assumed.

SPECIFICATIONS

3.4.1.1P LOAD FACTORS AND COMBINATIONS FOR TYPICAL PENNDOT BRIDGE COMPONENTS

Tables 1 through 6 provide the load factors with the corresponding limit state condition for the following typical PennDOT bridge components:

- steel girders (Table 1)
- prestressed girders (Table 2)
- abutment/retaining walls (Table 3)
- box culverts (Table 4)
- steel floorbeams (Table 5)
- steel trusses (Table 6)

Tables 1, 2, 4, 5 and 6 also include information for rating these components. (Rating are not typically done for abutment/retaining walls.)

COMMENTARY

C3.4.1.1P

The design live load vehicle in the fatigue load combination designated as HS20-9.0 refers to an HS20 truck with a fixed 9 meter {30 ft.} rear axle spacing.

Table D3.4.1.1P-1 - Load Factors and Live Load Vehicles for Steel Girders

	Load Combination												
	STR I	STR IP[8]	STR IA[6]	STR II	STR III	STR IV[1]	STR V	SERV II	SERV IIA[6]	SERV IIB	FATIGUE[2]	DEFL[2]	CONST/ UNCURED SLAB[9]
γ_{DC}[3]	1.25, 0.90	1.25, 0.90	1.25, 0.90	1.25, 0.90	1.25, 0.90	1.5	1.25, 0.90	1.00	1.00	1.00	---	---	1.25
γ_{DW}[4]	1.50, 0.65	1.50, 0.65	1.50, 0.65	1.50, 0.65	1.50, 0.65	1.50, 0.65	1.50, 0.65	1.00	1.00	1.00	---	---	1.50, 0.65
γ_{LL}	1.75	1.35	1.10	1.35	---	---	1.35	1.30	1.00	1.00	0.75	1.00	1.50
γ_{PL}	---	1.75	---	---	---	---	---	---	---	---	---	---	---
γ_{WS}	---	---	---	---	1.40	---	0.40	---	---	---	---	---	1.25
Design LL Veh.[7]	PHL-93	PHL-93	PHL-93	Permit (P-82)	---	---	PHL-93	PHL-93	PHL-93	Permit (P-82)	HS20-9.0	PennDOT Defl. Trk.	User Def.
Rating Veh.	Rating Applicability: I = Inventory, O = Operating												
PHL-93	I	I	O	---	---	---	---	I	O	---	---	---	---
P-82	---	---	---	O	---	---	---	---	---	O	---	---	---
ML-80	I	I	---	O	---	---	---	I	O	---	---	---	---
HS20	I	I	---	O	---	---	---	I	O	---	---	---	---
H20	I	I	---	O	---	---	---	I	O	---	---	---	---
Spec. Veh.	I	I	---	O	---	---	---	I	O	---	---	---	—

Table D3.4.1.1P-1 - Load Factors and Live Load Vehicles for Steel Girders (Continued)

Notes:

[1]Applicable when DL/LL ratio exceeds 7.0
[2]A load factor of unity is applied to permanent loads for the fatigue and deflection limit state only when specified
[3]DC load factor also used for barrier loads
[4]DW load factor also used for utility loads
[5]All loads applied to non-composite section for non-composite girders (Live loads are applied to the n section for steel)
[6]Load combination for rating only
[7]This row lists the typical design vehicle to be used for each load combination
[8]The reduced load factor for LL with PL (see D3.4.1)
[9]Design live load N/A for uncured slab check

Permanent Loads for Girder Programs			Section Properties[5]
Load	Steel		Steel
DC1	γ_{GR} γ_{SLAB} γ_{SLAB}	Girder Slab Haunch	Nc nc nc
DC2	γ_{DC2}	Barrier	3n
DW	γ_{FWS}	FWS	3n

Table D3.4.1.1P-2 - Load Factors and Live Load Vehicles for Prestressed Concrete Girders

	Load Combination											
	STR I	STR IP[8]	STR IA[6]	STR II	SERV I (P/S compr. chk.)		SERV III (P/S tension chk.		SERV IIIA (M_r @ 0.9 f_y chk.)	SERV IIIB (PennDOT-cracking chk.)	FATIGUE[1]	DEFL[1]
					w/o PL	with PL	w/o PL	with PL				
γ_{DC}[2]	1.25, 0.90	1.25, 0.90	1.25, 0.90	1.25, 0.90	1.00	1.00	1.00	1.00	1.00	1.00	---	---
γ_{DW}[3]	1.50, 0.65	1.50, 0.65	1.50, 0.65	1.50, 0.65	1.00	1.00	1.00	1.00	1.00	1.00	---	---
γ_{LL}	1.75	1.35	1.10	1.35	1.00	0.80	0.80[4]	0.65[9]	1.00	1.00	0.75	1.00
γ_{PL}	—	1.75	---	---	---	1.00	---	1.00	---	---	---	---
$\gamma_{CR,SH}$	0.5	0.5	0.5	0.5	1.00	1.00	1.00	1.00	---	---	---	---
Design LL Veh.[7]	PHL-93	PHL-93	PHL-93	Permit (P-82)	PHL-93		PHL-93		Controlling PHL-93 or P-82	Controlling PHL-93 or P-82	HS20-9.0	PennDOT Defl. Trk.
Rating Vehicle	Rating Applicability: I = Inventory, O = Operating											
PHL-93	I	I	O	---	I		I		O	---	---	---
P-82	---	---	---	O	---		---		O	---	---	---
ML-80	I	I	---	O	I		I		O	---	---	---
HS20	I	I	---	O	I		I		O	---	---	---
H20	I	I	---	O	I		I		O	---	---	---
Spec. Veh.	I	I	---	O	I		I		O	---	---	—

Table D3.4.1.1P-2 - Load Factors and Live Load Vehicles for Prestressed Concrete Girders (Continued)

Notes:

[1]A load factor of unity is applied to permanent loads for the fatigue and deflection limit state only when specified
[2]DC load factor also used for barrier loads
[3]DW load factor also used for utility loads
[4]For rating vehicles, the live load for Service III is to be taken as 1.0 (γ = 0.80 for PHL-93 only)
[5]All loads applied to non-composite section for non-composite girders
(Live loads are applied to the n section for P/S composite girders. For P/S, live load stresses can be based on transformed strands)
[6]Load combination for rating only
[7]This row lists the typical design vehicle to be used for each load combination
[8]The reduced load factor for LL with PL (see D3.4.1)
[9]For rating vehicles (other than PHL-93), the live load factor for Service III is to be taken as 0.80 for the pedestrian load case

Permanent Loads for Girder Programs			Section Properties[5]
Load	P/S		P/S
DC1	γ_{GIR}	Girder	nc
	γ_{SLAB}	Slab	nc
	γ_{SLAB}	Haunch	nc
	γ_{ID}	Int. Dia.	nc
	γ_{ED}	Ext. Dia.	nc
DC2	γ_{DC2}	Barrier	n
DW	γ_{FWS}	FWS	n

Table D3.4.1.1P-3 - Load Factors and Live Load Vehicles for Abutment/Retaining Walls

	Load Combination									
	SERV I	STR I	STR IP	STR II	STR III	STR V	EXTREME I[2]	EXTREME II[3]	Min. γ for Const. Case (Strength)[5]	γ for consolidation/secondary settlement
γ_{DC}	1.00	1.25, 0.90	1.25, 0.90	1.25, 0.90	1.25, 0.90	1.25, 0.90	1.25, 0.90	1.25, 0.90	1.25, 0.90	1.00
γ_{DW}	1.00	1.50, 0.65	1.50, 0.65	1.50, 0.65	1.50, 0.65	1.50, 0.65	1.50, 0.65	0.00	---	1.00
γ_{EV}	1.00	γ_{EV}	γ_{EV}	γ_{EV}	γ_{EV}	γ_{EV}	γ_{EV}	γ_{EV}	γ_{EV}	1.00
γ_{EH}	1.00	γ_{EH}	γ_{EH}	γ_{EH}	γ_{EH}	γ_{EH}	0.00	γ_{EH}	γ_{EH}	1.00
γ_{ES}[4]	1.00	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.00
γ_{LS}[4]	1.00	1.75	1.35	1.35	0.00	1.35	γ_{EQ}	0.50	1.50	0.00
γ_{LLIM}[1]	1.00, 0	1.75, 0	1.35, 0	1.35, 0	0.00	1.35, 0	γ_{EQ}, 0	0.00	---	0.00
γ_{PL}	0.00	0.00	1.75, 0.00	0.00	0.00	0.00	0.00	0.00	---	0.00
γ_{WS}	0.3	0.00	0.00	0.00	1.40	0.40	0.00	0.00	---	0.00
γ_{WL}	1.0	0.00	0.00	0.00	0.00	1.0	0.00	0.00	---	0.00
γ_{WA}	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
γ_{BR}	1.0	1.75	1.35	1.35	0.00	1.35	γ_{EQ}	0.00	---	0.00
γ_{CE}	1.0	1.75	1.35	1.35	0.00	1.35	γ_{EQ}	0.00	---	0.00
γ_{FR}	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	---	0.00
γ_{TU}	1.0	0.5	0.5	0.5	0.5	0.5	0.00	0.00	---	0.00
γ_{EQ}	0.00	0.00	0.00	0.00	0.00	0.00	1.0	0.00	---	0.00
γ_{CT}	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.0	---	0.00
Design LL Vehicle	PHL-93	PHL-93	PHL-93	P-82	---	---	PHL-93	---	---	—

Table D3.4.1.1P-3 - Load Factors and Live Load Vehicles for Abutment/Retaining Walls (Continued)

Notes:

[1]For a negative reaction on an abutment (uplift), use the maximum load factor
[2]For the seismic load case, EH loads (normal lateral earth pressure) replaced by EQ soil loads. γ_{EQ} for live loads = 0.0.
[3]Parapet collision force, CT.
[4]All lateral loads and their vertical components are maximized.
[5]For evaluation of the temporary construction stages using the Strength Limit states (see D11.6.1.2), use the greater of the γ noted under Construction Case column or under the given Strength Limit State column.

	Abutment/Retaining Wall Earth Load Factors	
	Maximum	Minimum
γ_{EV}	1.35	1.00
γ_{EH}	1.50	(4)

Table D3.4.1.1P-4 - Load Factors and Live Load Vehicles for Box Culverts

	Load Combination					
	SERV I	STR I	STR IA	STR II	FATIGUE[1]	Min. γ for Const. Case (Strength)[4]
γ_{DC}	1.00	1.25, 0.90	1.25, 0.90	1.25, 0.90	---	1.25, 0.90
γ_{DW}	1.00	1.50, 0.65	1.50, 0.65	1.50, 0.65	---	---
γ_{EV}	1.00	γ_{EV}	γ_{EV}	γ_{EV}	—	γ_{EV}
γ_{EH}	1.00	γ_{EH}	γ_{EH}	γ_{EH}	—	γ_{EH}
γ_{ES}[3]	1.00	1.50, 0.75	1.50, 0.75	1.50, 0.75	---	1.50,0.75
γ_{LS}	1.00, 0	1.75, 0	1.10, 0	1.35, 0	---	1.50, 0
γ_{LLIM}	1.00, 0	1.75, 0	1.10, 0	1.35, 0	0.75	---
Design LL Vehicle	PHL-93	PHL-93	PHL-93	P-82	HS20-9.0	---
Rating Vehicle[2]	Rating Applicability: I = Inventory, O = Operating					
PHL-93	---	I	O	---	---	---
P-82	---	---	---	O	---	---
ML-80	---	I	---	O	---	---
HS20	---	I	---	O	---	---
H20	---	I	---	O	---	---
Spec. Veh.	—	I	---	O	---	---

Notes:

[1]Fatigue load factor should be factored by PTF. A load factor of unity is applied to permanent loads for the fatigue limit state only when specified.
[2]Rating applicable for box culverts only
[3]Minimum ES of 0.50 applies for top slabs of box culverts
[4]See A3.4.2, Load Factors for Construction Loads

	Box Culvert Earth Load Factors	
	Maximum	Minimum
γ_{EV}	1.30	0.90
γ_{EH}	1.35	0.90[*]
[*]Use 0.50 minimum for culvert top slab		

Table D3.4.1.1P-5 - Load Factors and Live Load Vehicles for Steel Floorbeams

	Load Combination											
	STR I	STR IP[7]	STR IA[5]	STR II	STR III	STR V	SERV II	SERV IIA[5]	SERV IIB	FATIGUE[1]	DEFL[1]	CONST/ UNCURED SLAB[8]
γ_{DC}[2]	1.25, 0.90	1.25, 0.90	1.25, 0.90	1.25, 0.90	1.25, 0.90	1.25, 0.90	1.00	1.00	1.00	---	---	1.25
γ_{DW}[3]	1.50, 0.65	1.50, 0.65	1.50, 0.65	1.50, 0.65	1.50, 0.65	1.50, 0.65	1.00	1.00	1.00	---	---	1.50, 0.65
γ_{LL}	1.75	1.35	1.10	1.35	---	1.35	1.30	1.00	1.00	0.75	1.00	1.50
γ_{PL}	—	1.75	---	---	---	---	---	---	---	---	---	---
γ_{WS}	—	---	—	—	1.40	0.40	---	---	---	---	---	1.25
Design LL Veh.[6]	PHL-93	PHL-93	PHL-93	Permit (P-82)	---	PHL-93	PHL-93	PHL-93	Permit (P-82)	HS20-9.0	PennDOT Defl. Trk.	User Def.
Rating Veh.	Rating Applicability: I = Inventory, O = Operating											
PHL-93	I	I	O	---	---	---	I	O	---	---	---	---
P-82	---	---	---	O	---	---	---	---	O	---	---	---
ML-80	I	I	---	O	---	---	I	O	---	---	---	---
HS20	I	I	---	O	---	---	I	O	---	---	---	---
H20	I	I	---	O	---	---	I	O	---	---	---	---
Spec. Veh.	I	I	---	O	---	---	I	O	---	---	---	—

Table D3.4.1.1P-5 - Load Factors and Live Load Vehicles for Steel Floorbeams (Continued)

Notes:

[1]A load factor of unity is applied to permanent loads for the fatigue and deflection limit state only when specified
[2]DC load factor also used for barrier loads, sidewalk, median barrier, railings, etc.
[3]DW load factor also used for utility loads
[4]All loads applied to non-composite section for non-composite floorbeams (Live loads are applied to the n section for steel)
[5]Load combination for rating only
[6]This row lists the typical design vehicle to be used for each load combination
[7]The reduced load factor for LL with PL (see D3.4.1)
[8]Live load N/A for uncured slab check

Permanent Loads for Floorbeam Programs			Section Properties[4]
Load	Steel		Steel
DC1	γ_{FLBM} γ_{SLAB} γ_{SLAB}	Floorbeam Slab Haunch	Nc nc nc
DC2	γ_{DC2}	Barrier, Sidewalk, Median Barrier, Railings, etc.	3n
DW	γ_{FWS}	FWS	3n

Table D3.4.1.1P-6 - Load Factors and Live Load Vehicles for Steel Trusses

	Load Combination														
	STR I	STR IP[8]	STR IA[5]	STR II	STR III	STR IV[1]	STR V	EXT. EVENT III	EXT. EVENT IV	SERV II	SERV IIA[5]	SERV IIB	FATIGUE[2]	DEFL[2]	CONST
γ_{DC}[3]	1.25, 0.90	1.25, 0.90	1.25, 0.90	1.25, 0.90	1.25, 0.90	1.5	1.25, 0.90	1.25, 0.90	1.05, 0.95	1.00	1.00	1.00	---	---	1.25
γ_{DW}[4]	1.50, 0.65	1.50, 0.65	1.50, 0.65	1.50, 0.65	1.50, 0.65	1.50, 0.65	1.50, 0.65	1.50, 0.65	1.05, 0.90	1.00	1.00	1.00	---	---	1.50, 0.65
γ_{LL}	1.75	1.35	1.10	1.35	---	---	1.35	1.30	1.15	1.30	1.00	1.00	0.75	1.00	1.50
γ_{PL}	---	1.75	---	---	---	---	---	1.10	1.05	---	---	---	---	---	---
γ_{WS}	---	—	---	—	1.40	---	0.40	---	---	---	---	---	---	---	1.25
Design LL Veh.[6]	PHL-93	PHL-93	PHL-93	Permit (P-82)	---	---	PHL-93	PHL-93 P-82	PHL-93, P-82	PHL-93	PHL-93	Permit (P-82)	HS20-9.0	PennDOT Defl. Trk.	User Def.
Rating Veh.	Rating Applicability: I = Inventory, O = Operating														
PHL-93	I	I	O	---	---	---	---	---	---	I	O	---	---	---	---
P-82	---	---	---	O	---	---	---	---	---	---	---	O	---	---	---
ML-80	I	I	---	O	---	---	---	---	---	I	O	---	---	---	---
HS20	I	I	---	O	---	---	---	---	---	I	O	---	---	---	---
H20	I	I	---	O	---	---	---	---	---	I	O	---	---	---	---
Spec. Veh.	I	I	---	O	---	---	---	---	---	I	O	---	---	---	—

Table D3.4.1.1P-6 - Load Factors and Live Load Vehicles for Steel Trusses (Continued)

Notes:

[1]Applicable when DL/LL ratio exceeds 7.0
[2]A load factor of unity is applied to permanent loads for the fatigue and deflection limit state only when specified
[3]DC load factor also used for barrier loads, sidewalk, median barrier, railings, deck, stringers, truss floorbeams, wind and lateral bracing, etc.
[4]DW load factor also used for utility loads
[5]Load combination for rating only
[6]This row lists the typical design vehicle to be used for each load combination
[7]The reduced load factor for LL with PL (see D3.4.1)

SPECIFICATION

3.5 PERMANENT LOADS

3.5.1 Dead Loads: DC, DW and EV

The following shall supplement A3.5.1.

The acceleration due to gravity (g) shall be taken as 9.81 m/s^2 for use in determining force effects from mass.

In addition to the weight of the deck slab, the design dead load shall include provisions for a future wearing surface with a surface area density of 150 kg/m^2 {0.030 ksf} on the deck slab between the curbs. This load shall be considered for all deck slabs, including decks with a bituminous wearing surface, but not for structures under fill. For decks formed using permanent metal deck forms, an additional dead load shall be included based on a surface density of 75 kg/m^2 {0.015 ksf} which takes into account the weight of the form, plus the weight of the concrete in the valleys of the forms.

In Table A3.5.1-1, replace the low density concrete value of 1775 kg/m^3 {0.110 kcf} with 1840 kg/m ^{3} {0.115 kcf}. Also in Table A3.5.1-1, delete the "sand-low-density" concrete value. For use of low density concrete with densities different than 1840 kg/m^3 {0.115 kcf}, see D5.4.2.1 and DC5.4.2.1.

COMMENTARY

C3.5.1

The following shall supplement AC3.5.1.

The normal density concrete and low density concrete with densities of 2400 kg/m^3 {0.150 kcf} and 1840 kg/m^3 {0.115 kcf} respectively include an allowance for reinforcement in the calculation of the density.

For concrete deck slabs, provisions must be made in the design for the addition of a bituminous wearing surface at some future time. Even in cases where the initial design includes a bituminous surface, provision must be made for an additional future wearing surface since the original bituminous material is not always stripped off before the new surface is added.

For structures under fill, the additional dead load associated with a future wearing surface is insignificant when compared with other contributions to the dead load. Therefore, in this case, no allowance for future wearing surface is necessary.

It is recognized that permanent metal deck forms are available for which the surface density is less than 75 kg/m^2 {0.015 ksf}; however, the minimum design load should be retained at this level. Lightweight forms may be advantageous in certain situations, such as rehabilitation, and should be evaluated on a case-by-case basis.

3.5.1.1P APPLICATION OF DEAD LOAD ON GIRDER AND BOX BEAM STRUCTURES

The provisions in this article apply to superstructure types, a, b, c, f, g, h, k and l given in Table A4.6.2.2.1-1.

For composite adjacent and spread beams, the barrier (single barrier) load shall be equally distributed to the nearest three and two beams, respectively, when the barriers are placed after slab has hardened.

The dead load of items, such as fencing and sound barriers, if placed after the slab has hardened, shall be distributed to girder as described above.

Sidewalk dead load shall be distributed to a girder using the lever rule.

For noncomposite girders, the barrier load shall be distributed solely to the fascia girder.

The future wearing surface shall be distributed equally among all girders.

3.6 LIVE LOADS

3.6.1 Gravity Loads: LL, PL

3.6.1.1 VEHICULAR LIVE LOAD

3.6.1.1.2 Multiple Presence of Live Load

Delete the last sentence of the second paragraph of A3.6.1.1.2.

C3.6.1.1.2

Delete the third through seventh paragraphs of AC3.6.1.1.2.

SPECIFICATION

3.6.1.2 DESIGN VEHICULAR LIVE LOAD

3.6.1.2.1 General

The following shall replace the first paragraph of A3.6.1.2.1.

The vehicular live loading on the roadways of bridges or incidental structures, designated PHL-93, shall consist of a combination of the:

- design truck or design tandem, and
- design lane load,

as given in A3.6.1.2 and D3.6.1.2.

3.6.1.2.3 Design Tandem

Modify A3.6.1.2.3 so that weight of each axle is increased from 110 kN to 140 kN {25 kips to 31.25 kips}.

COMMENTARY

C3.6.1.2.1

The following shall supplement AC3.6.1.2.1.

At this time, the Department makes no exceptions to the requirements for application of PHL-93 vehicular live load for bridges on low volume roads.

C3.6.1.2.5

The following shall supplement AC3.6.1.2.5.

The area load applies only to the design truck and tandem. For other design vehicles, the tire contact area should be determined by the engineer.

As a guideline for other truck loads, the tire area in mm^2 $\{in^2\}$ may be calculated from the following dimensions:

Metric Units:

Tire width = P/0.142

Tire length = 165γ(1+IM/100)

U.S. Customary Units:

Tire width = P/0.8

Tire length = 6.4γ(1+IM/100)

where:

γ = load factor, as given in A3.4.1 and D3.4.1, except for buried structures where the load factor shall be 1.35

IM = dynamic load allowance percent

P = wheel load
= 72.5 kN {16 kips} for the design truck, 70 kN {15.625 kips} for the design tandem and 60 kN {13.5 kips} for the P-82

A constant value of γ was chosen for buried structures

SPECIFICATION

COMMENTARY

to simplify the analysis at strength and service limit states. PennDOT has conducted a study to ensure that the use of a constant load factor has a negligible effect.

3.6.1.2.6 Distribution of Wheel Loads through Earth Fills

The following shall replace the first paragraph of A3.6.1.2.6.

Where the depth of fill is less than 600 mm {2 ft.}, live loads shall be distributed to the top slabs of culverts as specified in D4.6.2.12P.

The following shall replace the second paragraph of A3.6.1.2.6.

In lieu of a more precise analysis, or the use of other acceptable approximate methods of load distribution permitted in Section 12, where the depth of fill is 600 mm {2 ft.} or greater, wheel loads may be considered to be uniformly distributed over a rectangular area with sides equal to the dimension of the tire contact area, as specified in A3.6.1.2.5, and increased by either 1.15 times the depth of the fill in select granular backfill, or the depth of the fill in all other cases. The provisions of A3.6.1.1.2 and A3.6.1.3 shall apply.

The following shall replace the last paragraph of A3.6.1.2.6.

Where live load and impact moment in concrete slabs, based on the distribution of wheel load through earth fills, exceeds the live load and impact moment calculated according to A4.6.2.1 and D4.6.3.2, the latter moment shall be used.

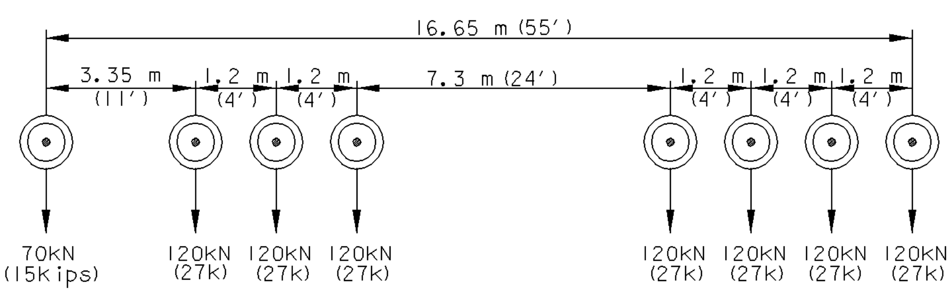
C3.6.1.2.6

The following shall supplement AC3.6.1.2.6.

Traditionally, the effect of fills less than 600 mm {2 ft.} deep on live load has been ignored. Research (McGrath, et al. 2004) has shown that in design of box sections allowing distribution of live load through fill in the direction parallel to the span provides a more accurate design model to predict moment, thrust, and shear forces. Provisions in D4.6.2.12P provide a means to address the effect of shallow fills.

3.6.1.2.7P Design Permit Load (P-82)

The weights and spacings of axles and wheels for the Permit Load (P-82) shall be as specified in Figure 1. A dynamic load allowance shall be considered as specified in D3.6.2 and A3.6.2.



NOTE: P-82 width is the same as the Design Truck.
Transverse wheel location is the same as Design Truck.

Figure 3.6.1.2.7P-1 - Pennsylvania Permit Load (P-82) 910 kN {102 tons}, 8 Axle

Axles which do not contribute to the extreme force effect under consideration shall be neglected.

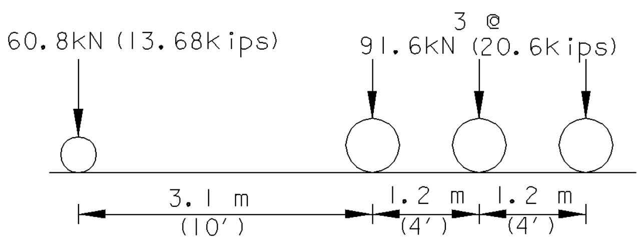
SPECIFICATION

For multi-girder superstructures design, the permit load shall be in one lane or in multiple lanes whichever is the controlling case.

For superstructure with girder-floorbeam-stringer systems and substructure components designs, the permit load shall be in one lane or in one lane with PHL-93 loading in adjacent lanes, whichever is the controlling case.

3.6.1.2.8P Maximum Legal Load (ML-80)

The weights and spacings of axles and wheels for the Maximum Legal Load (ML-80) shall be as specified in Figure 1. The ML-80 truck is used for rating.



NOTE: ML-80 width is the same as the design truck. Transverse wheel location is the same as design truck.

Figure 3.6.1.2.8P-1 - Pennsylvania Maximum Legal Load (ML-80) 335.7 kN {37.74 tons}, 4 Axle

3.6.1.3 APPLICATION OF DESIGN VEHICULAR LIVE LOADS

3.6.1.3.1 General

The following shall replace A3.6.1.3.1.

Unless otherwise specified, the extreme force effect shall be taken as the larger of the following:

- the effect of the design tandem combined with the effect of the design lane load, or
- the effect of one design truck with the variable axle spacing specified in A3.6.1.2.2, combined with the effect of the design lane load, and
- for the negative moment between points of dead load contraflexure, the effect of two design trucks spaced a minimum of 15 000 mm {50 ft.} between the lead axle of one truck and the rear axle of the other truck, combined with the effect of the design lane load; the distance between the 145 kN {32 kips} axles of each truck shall be taken as 4300 mm {14 ft.}.

For the reaction at interior piers only, 90% of the effect of two design trucks spaced a minimum of 15 000 mm {50 ft.} between the lead axle of one truck and the rear

COMMENTARY

C3.6.1.2.8P

The ML-80 is not considered a notional load. Therefore, all of the axles shall be considered when determining force effects.

C3.6.1.3.1

Delete the second and third sentences of the third paragraph of AC3.6.1.3.1.

The following shall supplement AC3.6.1.3.1.

The BXLRFD program does not consider the effect of two design trucks, since the minimum distance between the two design trucks is 15 000 mm {50 ft.} which is at the upper limit of a twin cell culvert. The effects of two tandems are considered for a twin cell box culvert in the BXLRFD program.

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axle of the other truck, combined with 90% of the effect of the design lane load. The distance between the 145 kN {32 kips} axles of each truck shall be taken as 4300 mm {14 ft.}.

- For the negative moment between points of dead load contraflexure, the effect of two tandems with axle weights of 110 kN {25 kips} spaced from 8000 mm to 12 000 mm {26 ft. to 40 ft.} apart, combined with the effect of the design lane load.

  For the reaction at interior piers only, 100% of the effect of two tandems with axle weights of 110 kN {25 kips} spaced from 8000 mm to 12 000 mm {26 ft. to 40 ft.} apart combined with the effect of the design lane load.

Axles which do not contribute to the extreme force effect under consideration shall be neglected.

Both the design lanes and the position of the 3000 mm {10 ft.} loaded width in each lane shall be positioned to produce extreme force effects. The design truck or tandem shall be positioned transversely such that the center of any wheel load is not closer than:

- for the design of the deck overhang - 300 mm {1 ft.} from the face of the curb or railing, and
- for the design of all other components - 600 mm {2 ft.} from the edge of the design lane.

Unless otherwise specified, the lengths of design lanes, or parts thereof, which contribute to the extreme force effect under consideration, shall be loaded with the design lane load.

3.6.1.3.2 Loading for Live Load Deflection Evaluation

The following shall replace A3.6.1.3.2.

The deflection should be taken as 125% of the larger of:

- that resulting from the effect of one design truck with the variable axle spacing specified in A3.6.1.2.2,
- that resulting from the effect of 25% of one design truck with the variable axle spacing specified in A3.6.1.2.2, combined with the effect of the design lane.

COMMENTARY

C3.6.1.3.2

The following shall replace AC3.6.1.3.2.

The LRFD live load deflection criteria was developed such that deflections would be roughly equivalent to those produced by a HS20 vehicle. A 25% increase is specified to be consistent with the Department's past use of the HS25 vehicle for computing deflections.

SPECIFICATION

3.6.1.3.3 Design Loads for Decks, Deck Systems, and the Top Slab of Box Culverts

Replace the three bullets of the second paragraph of A3.6.1.3.3 with the following.

- Where the slab spans primarily in the transverse direction, only the axles of the design truck of A3.6.1.2.2 or design tandem of D3.6.1.2.3 shall be applied to the deck slab or top slab of box culverts.

COMMENTARY

C3.6.1.3.3

Add the following after the second paragraph of AC3.6.1.3.3.

The design truck and tandem without the lane load and with a multiple presence factor of 1.2 results in factored force effects that are similar to the factored force effects using the Standard Specification for typical span ranges of box culverts.

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- Where the slab spans primarily in the longitudinal direction:
  - For top slabs of box culverts of all spans and for all other cases, including slab-type bridges where the span does not exceed 4600 mm {15.0 ft.}, only the axle loads of the design truck or design tandem of A3.6.1.2.2 and D3.6.1.2.3, respectively, shall be applied.
  - For all other cases, including slab-type bridges (excluding top slabs of box culverts) where the span exceeds 4600 mm {15.0 ft.}, all of the load shall be applied.

Replace the third paragraph of A3.6.1.3.3 with the following:

Where the refined methods are used to analyze decks, force effects shall be determined on the following basis:

- Where the slab spans primarily in the transverse direction, only the axles of the design truck of A3.6.1.2.2 or design tandem of D3.6.1.2.3 shall be applied to the deck slab.
- Where the slab spans primarily in the longitudinal direction (including slab-type bridges), all of the loads specified in D3.6.1.2 shall be applied.

3.6.1.3.4 Deck Overhang Load

The following shall replace A3.6.1.3.4.

The deck overhang load shall be as given in D3.6.1.3.1.

Also, the ultimate strength of the deck section shall be greater than the ultimate strength of the barrier, see Section A13 and its Appendix. Horizontal loads on the overhang resulting from vehicle collision with barriers shall be in accordance with the provisions of Section A13 and its Appendix.

3.6.1.4 FATIGUE LOAD

3.6.1.4.2 Frequency

The following shall replace Table A3.6.1.4.2-1.

Table 3.6.1.4.2-1 – Fraction of Truck Traffic in a Single Lane, p

Number of Lanes Available to Trucks	p
1	1.00
2 or more	0.85

COMMENTARY

C3.6.1.3.4

The following shall replace AC3.6.1.3.4.

The deck overhang slab provided in BD-601M has been designed for the vertical design loads (D3.6.1.3.1) or a strength greater than the applied forces transmitted to the overhang when the barrier is subjected to the maximum collision force it can resist (Section A13) whichever is greater. The ultimate strength of the barrier used in the design of the overhang was based on the Department’s Typical Barrier (see Standard Drawing BD-601M) which placed greater demand on the deck overhang than the other Department barriers.

SPECIFICATION COMMENTARY

3.6.1.5 RAIL TRANSIT LOAD

3.6.1.5.1P General

Live loads for rail traffic shall use a combination of axle loads and axle spacings represented by the Cooper E80 loading, as shown in Figure 1.

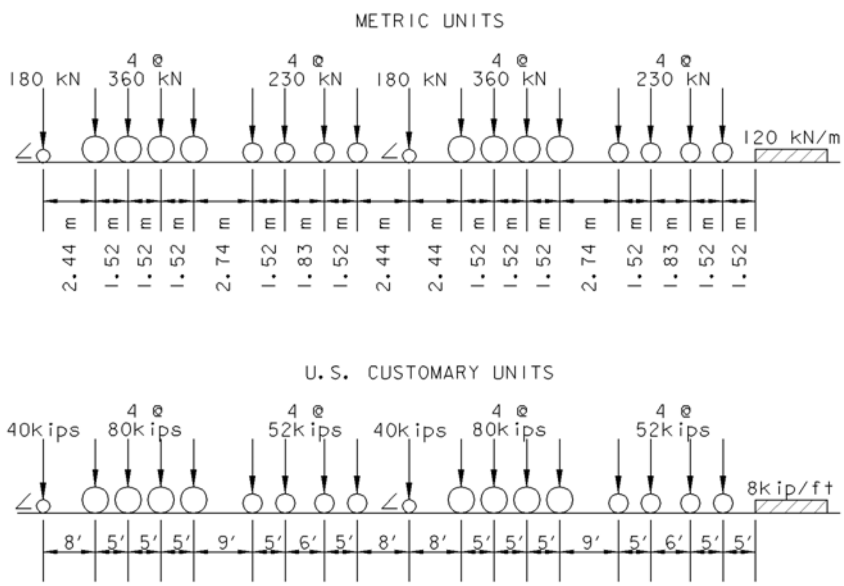


Figure 3.6.1.5.1P-1 - Wheel Spacing for Cooper E80 Design Loading (Load/Axle)

3.6.1.5.2P Distribution of Rail Transit Loads Through Earth Fill

The load intensity, W_l, on a buried structure due to rail transit loading shall be determined using the following relationship:

$$W_l = C\ P_o\ B_c\ (1 + I_f) \qquad (3.6.1.5.2P\text{-}1)$$

Refer to Table 1 for values of C. The series of axle loads and spacing shall be converted into a uniform load at the bottom of the railroad ties. The loading, P_o, at the base of the ties shall be represented by a ground pressure of 97 MPa {2025 ksf}, which represents the locomotive drive-wheel (four at 360 kN {80 kips}) loading distributed over an area 2400 mm by 6100 mm {8 ft. by 20 ft.} and a track structure loading of 3 kN/m {0.2 kip/ft}. The impact factor, I_f, shall range from 40% at zero cover to 0% at 3000 mm {10 ft.} of cover.

The live load and the dead load, including the impact factor, for a Cooper E80 loading can be determined from Figure 1. To obtain the live load per linear meter, multiply the unit load from Figure 1 by the outside horizontal span of the pipe, B_c.

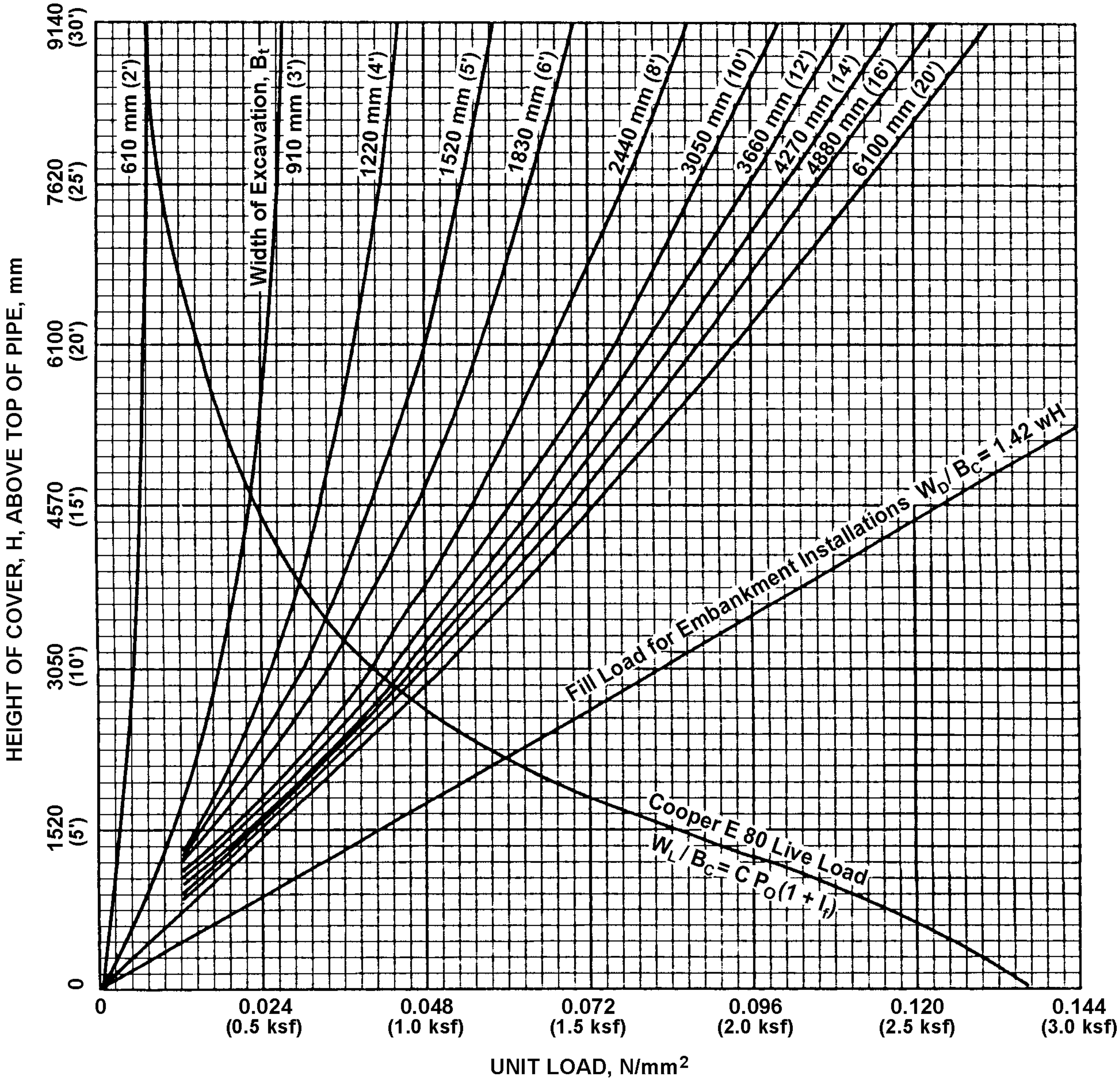
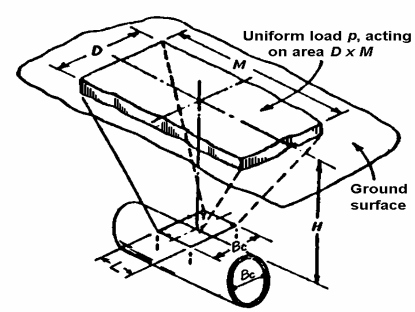


Figure 3.6.1.5.2P-1 - Live and Dead Loads on Pipe Installed Under Railroads (ACPA, 1981)

Table 3.6.1.5.2P-1 - Values of Load Coefficient (C) for Concentrated and Distributed Superimposed Loads Vertically Centered Over Culvert (ASCE, 1969)

$\frac{D}{2H}$ or $\frac{B_c}{2H}$	$\frac{M}{2H}$ or $\frac{L}{2H}$													
	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.5	2.0	5.0
0.1	0.019	0.037	0.053	0.067	0.079	0.089	0.097	0.103	0.108	0.112	0.117	0.121	0.121	0.128
0.2	0.037	0.072	0.103	0.131	0.155	0.174	0.189	0.202	0.211	0.219	0.229	0.238	0.211	0.218
0.3	0.053	0.103	0.149	0.190	0.221	0.252	0.274	0.292	0.306	0.318	0.333	0.345	0.355	0.360
0.4	0.067	0.131	0.190	0.241	0.281	0.320	0.349	0.373	0.391	0.405	0.425	0.440	0.454	0.460
0.5	0.079	0.155	0.224	0.284	0.336	0.379	0.414	0.441	0.463	0.484	0.505	0.525	0.540	0.548
0.6	0.089	0.171	0.252	0.320	0.379	0.428	0.467	0.499	0.524	0.544	0.572	0.596	0.613	0.624
0.7	0.097	0.189	0.274	0.349	0.414	0.467	0.511	0.516	0.584	0.597	0.628	0.650	0.674	0.688
0.8	0.103	0.202	0.292	0.373	0.441	0.499	0.546	0.581	0.615	0.639	0.674	0.703	0.725	0.740
0.9	0.108	0.211	0.306	0.391	0.463	0.524	0.574	0.615	0.647	0.673	0.711	0.742	0.766	0.784
1.0	0.112	0.219	0.318	0.405	0.481	0.544	0.597	0.639	0.673	0.701	0.740	0.774	0.800	0.816
1.2	0.117	0.229	0.333	0.425	0.505	0.572	0.628	0.674	0.711	0.740	0.783	0.820	0.819	0.868
1.5	0.121	0.238	0.345	0.440	0.525	0.596	0.650	0.703	0.742	0.774	0.820	0.861	0.891	0.916
2.0	0.121	0.211	0.355	0.454	0.540	0.613	0.674	0.725	0.766	0.800	0.819	0.894	0.930	0.956

*Influence coefficients for solution of Holl's and Newmark's integration of the Boussinesq equation for vertical stress



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3.6.1.6 PEDESTRIAN LOADS

The following shall supplement A3.6.1.6.

The pedestrian load is distributed using the lever rule.

When pedestrian loads are to be considered, two loading conditions shall be considered. The first loading condition assumes that the sidewalk is not present (i.e., an extended roadway surface and barrier would replace the sidewalk area) and the bridge is used for vehicular live load only. Under the second loading condition, the pedestrian load is present and the vehicular live load is factored at a reduced level. The Strength IP load combination was developed for the second loading condition.

3.6.2 Dynamic Load Allowance: IM

3.6.2.1 GENERAL

The following shall supplement A3.6.2.1.

For permit loads, the static effect of the P-82 shall be increased by a percentage not to exceed IM = 20%.

IM for deck design = 50%

The second to last paragraph in A3.6.2.1 which begins "Dynamic load allowance need not..." shall be deleted.

3.6.2.1.1P Components for which IM is Applicable

The following components shall have the IM factor included in the design:

- all superstructure components including deck and deck joints
- pier caps and shafts
- backwalls and pedestals of abutments
- bearings, except for plain and reinforced elastomeric bearings

For buried components covered in D12 and A12, see D3.6.2.2. For wood component, see D3.6.2.3.

3.6.2.1.2P Components for which IM is Not Applicable

The following components shall not have the IM factor included in the design:

- retaining walls not subject to vertical reactions from the superstructure, including MSE walls
- foundation components which are entirely below ground level, including footings (except for frame and

COMMENTARY

C3.6.2.1

Irregularities in decks such as potholes can result in large localized impact effects. As a result, PennDOT requires that the impact for decks be increased from 33% to 50% for decks. Other elements of the bridge structure should not be greatly affected by high localized impact due to dampening. The combination of 50% impact, the design truck (former HS20 truck) and LRFD deck design criteria will produce deck designs comparable to 30% impact, HS25 and past AASHTO deck design criteria.

C3.6.2.1.2P

The PAPIER program carries the live loads from the pier cap through to the footing without the removal of the effect of the dynamic load allowance (IM) input by the user. This provides a consistent mathematical model throughout the structure, where the moments, shears, and axial forces at the bottom of the column are equal to those at the top of the footing of the pier.

SPECIFICATIONS

box culverts where IM is applicable as per A3.6.2.2), piles, caissons and pedestals

- abutment stems
- plain and reinforced elastomeric bearings
- buried components with 2400 mm {8 ft.} or greater fill above them (see A3.6.2.2)

The pedestrian load shall not have the IM factor applied.

3.6.2.2 BURIED COMPONENTS

The following shall replace A3.6.2.2.

The dynamic load allowance for culverts and other buried structures covered by Section 12, in percent, shall be taken as:

Metric Units:

$$IM = 40\ (1.0 - 4.1 \times 10^{-4}\ D_E) \exists 0\% \qquad (3.6.2.2\text{-}1)$$

U.S. Customary Units:

$$IM = 40\ (1.0 - 0.125\ D_E) \exists 0\%$$

where:

D_E = the minimum depth of earth cover above the structure (mm) {ft.}

Dynamic load allowance shall not be applied to foundation pressures.

3.6.2.3 WOOD COMPONENTS

The following shall replace A3.6.2.3.

For wood bridges and wood components of bridges, the dynamic load allowance specified in A3.6.2.1 may be reduced to 50 percent of the values specified for IM in Table A3.6.2.1-1.

COMMENTARY

C3.6.2.3

Delete the second sentence of CA3.6.2.3.

3.6.4 Braking Force: BR

The following shall supplement A3.6.4.

Dynamic load allowance is not applied to the braking force.

C3.6.4

The following shall supplement CA3.6.4.

LRFD analysis of the capacity of existing substructure units on shorter span bridges may become problematic. Use of the original design braking force, requiring approval of the Chief Bridge Engineer, may be warranted for analysis of these older structures.

SPECIFICATIONS

COMMENTARY

Braking Force Factor Tables previously included in DM-4 have been deleted since AASHTO 3rd edition added lane loading.

The AASHTO 2nd edition used only axle loads, thus Braking Force Factor was developed.

3.6.5 Vehicular Collision Force CT

3.6.5.3 Vehicular Collision with barriers

The following shall supplement A3.6.5.3.

For transverse vehicular collision loading transferred to the substructure for u-wings and retaining walls, use a load of 45 kN {10 kip} acting over 1.5 m {5 ft} length applied at a distance equal to the height of the concrete barrier above the top of the wall.

C3.6.5.3

The following shall supplement AC3.6.5.3.

The transverse vehicular collision loading of 45 kN {10 kip} acting over 1.5 m {5 ft} may be distributed down to the footing at a 1:1 slope. Adjacent to open joints, this load may only be distributed in one direction which will usually be the controlling condition. Distributing the load in one direction is conservative for footing designs, since the footings are continuous at open joints.

3.8 WIND LOAD: WL AND WS

3.8.1 Horizontal Wind Pressure

3.8.1.2 WIND PRESSURE ON STRUCTURES: WS

3.8.1.2.1 General

C3.8.1.2.1

The following shall replace the last paragraph of

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COMMENTARY

AC3.8.1.2.1.

If approved by the Chief Bridge Engineer, wind tunnel tests may be used to provide more precise estimates of wind pressures. Such testing should be considered where wind is a major design load.

C3.8.1.2.2

The following shall supplement AC3.8.1.2.2.

The columns referenced in A3.8.1.2.2 are columns in arch bridges not columns in substructure units.

3.8.3 Aeroelastic Instability

3.8.3.4 WIND TUNNEL TESTS

The following shall replace A3.8.3.4.

If approved by the Chief Bridge Engineer, representative wind tunnel tests may be used to satisfy the requirements of A3.8.3.2 and A3.8.3.3.

3.9 ICE LOADS: IC

3.9.1 General

The following shall supplement A3.9.1.

The forces due to ice shall be applied at the average elevation of the highest expected water elevation and the normal water elevation.

C3.9.1

The following shall supplement AC3.9.1.

The PAPIER program uses a default ice thickness of 150 mm {6 in.} and a default ice crushing strength of 2.75 MPa {58 ksf}.

3.9.5 Vertical Forces due to Ice Adhesion

The following shall replace A3.9.5.

The vertical force on a bridge pier due to rapid water level fluctuation shall be taken as:

Metric Units:

- for a circular pier, in N:
  $0.3\ t^{2} + 0.0169\ R\ t^{1.25}$

- for an oblong pier, in N:
  $0.3\ t^{2} + 0.0169\ R\ t^{1.25} + 2.3*10^{-3}\ L\ t^{1.25}$

U.S. Customary Units:

- for a circular pier, in kips:
  $6.27\ t^{2} + 1.48\ R\ t^{1.25}$

- for an oblong pier, in kips
  $6.27\ t^{2} + 1.48\ R\ t^{1.25} + 0.2\ L\ t^{1.25}$

where:

t = ice thickness (mm) {ft.}

C3.9.5

Delete AC3.9.5.

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R = radius of circular pier (mm) {ft.} or approximated end of oblong pier

L = perimeter of pier excluding half circles at ends of oblong pier (mm) {ft.}

3.10 EARTHQUAKE EFFECTS: EQ

COMMENTARY

C3.10.1 General

The following shall supplement AC3.10.1.

Minimize bridge skew as much as and whenever possible. It is well known that skewed structures perform poorly in seismic events when compared to the performance of normal or non-skewed structures.

These specifications present seismic design and construction requirements applicable to the majority of highway bridges to be constructed in the United States. Bridges not covered by these provisions probably constitute 5 to 15 percent of the total number of bridges designed.

The Project Engineering Panel (PEP) of the Applied Technology Council (ATC) has decided that special seismic design provisions are not required for buried structures. It was recognized by the PEP, however, that this decision may need to be reconsidered as more research data on the seismic performance of this type of structure become available.

These specifications specify minimum requirements. More sophisticated design or analysis techniques may be utilized if deemed appropriate by the Design Engineer.

For bridge types not covered by these specifications, the following factors should be considered.

(a) The recommended elastic design force levels of the specifications should be applicable because force levels are largely independent of the type of bridge structure. It should be noted that the elastic design force levels of the specifications are part of a design philosophy described in Chapter 1 of FHWA Research Report FHWA-IP-87-6. The appropriateness of both the design force levels and the design philosophy must be assessed before they are used for bridges that are not covered by these specifications.

(b) A multi-mode dynamic analysis should be considered, as specified in A4.7.4.3.3

(c) Design displacements are as important as design forces; when possible, the design methodology should consider displacements arising from the effects discussed in DC4.7.4.4.

(d) If a design methodology similar to that used in these specifications is deemed desirable, the design requirements contained herein should be used to ensure compliance with the design philosophy.

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COMMENTARY

Use caution when referencing the flowcharts contained in the Appendix to A3. They reference AASHTO LRFD Specifications which may be modified by the Design Manual, Part 4.

3.10.2 Acceleration Coefficient

The following shall replace A3.10.2.

The acceleration coefficient, A, to be used in the application of these provisions shall be determined from the County designations given in Figure 1.

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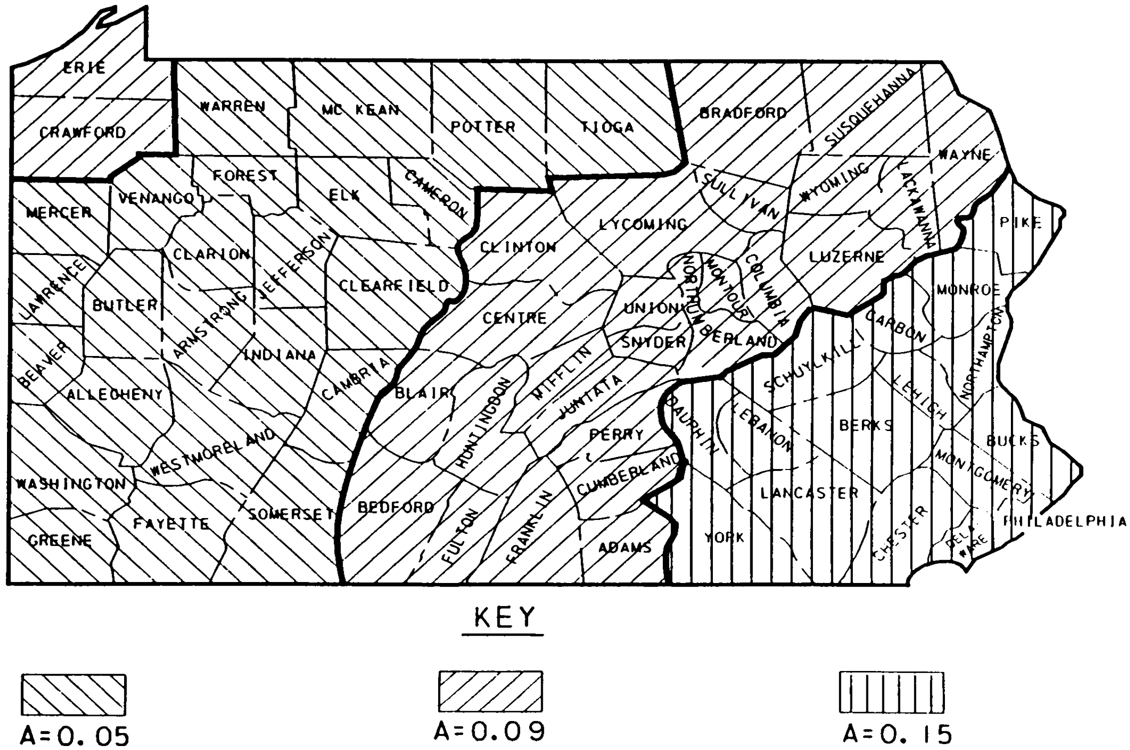

Figure 3.10.2-1 - Acceleration Coefficients for Pennsylvania Counties

3.10.7 Response Modification Factors

3.10.7.1 GENERAL

The following shall replace the first paragraph of A3.10.7.1.

Do not apply the response modification factors to single span structures or to structures in Seismic Zone 1.

The use of R-factors requires the reinforcing details meet the requirements of this document and the LRFD Specifications for consideration of structural ductility. All R-factors from Table A3.10.7.1-1 must be reduced to 1.0 if the details do not meet the specification requirements, unless tests have been done to indicate otherwise.

3.10.9 Calculation of Design Forces

3.10.9.2 SEISMIC ZONE 1

C3.10.9.2

The following shall supplement AC3.10.9.2.

Prior to the redesign phase of the PEP project, the PEP thought that the design of connections for wind forces would be satisfactory for anticipated seismic forces for bridges in Seismic Zone 1. However, when the magnitude of the wind and seismic forces were compared for six bridges, it was found in almost all cases that, for an acceleration coefficient of 0.10, seismic forces were greater

SPECIFICATIONS COMMENTARY

than wind forces. In some cases the difference was significant. Hence it was deemed necessary to include the requirement of this section for the design of the connections. The requirement is simple and somewhat conservative, especially for more flexible bridges, since the forces are based on the maximum elastic response coefficient. If the design forces are difficult to accommodate, it is recommended that Seismic Zone 2 analysis and design procedures be used.

3.10.9.3 SEISMIC ZONE 2

C3.10.9.3

The following shall supplement AC3.10.9.3.

The seismic design forces specified for bridges in Seismic Zone 2 are intended to be relatively simple, but consistent with the overall design concepts and methodology. Inherent in any simplification of a design procedure, however, is a degree of conservatism; for Seismic Zone 2 this occurs in the determination of the design forces for the foundations and connections to columns. If these forces appear to be excessive, more refined methods may be used. An acceptable approach would be to use the methods suggested for Seismic Zones 3 and 4 in the LRFD Specifications. For such a refinement, the foundations and connections to columns are designed for the maximum forces that a column can transmit to these components. In some cases, these may be considerably less than the design forces specified in this section.

This section specifies the design forces for the structural components of the bridge. In the first step, the elastic forces of Load Cases 1 and 2 of A3.10.8 are divided by the appropriate R-factors of A3.10.7. These forces are combined with those from other loads in load combination, Extreme Event I. Each component shall be designed to resist two seismic load combinations, one including Load Case 1 and the other including Load Case 2. Each load case incorporates different proportions of bi-directional seismic loading. This may be important for some components (e.g., bi-axial design of columns) and unimportant for others. In the design loads for each component, the sign of the seismic forces and moments obtained from A3.10.8 can be taken as either positive or negative. The sign of the seismic force or moment that gives the maximum magnitude for the design force (either positive or negative) shall be used.

This section also specifies the design forces for foundations which include the footings, pile caps, and piles. The design forces are essentially twice the seismic design forces of the columns. This will generally be conservative and was adopted to simplify the design procedure for bridges in Seismic Zone 2. However, if seismic forces do not govern the design of columns and piers, there is a possibility that during an earthquake the foundations will be subjected to forces larger than the design forces. This will occur if the columns remain elastic throughout the duration of the seismic ground motion. Thus, for essential bridges in Seismic Zone 2, consideration should be given to the use of

SPECIFICATIONS

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more refined methods in the design of the foundation. An acceptable approach would be to use the methods suggested for Seismic Zones 3 and 4 in the LRFD Specifications. It should be noted that ultimate soil and pile strengths are to be used with the specified foundation seismic design forces.

3.10.9.5 LONGITUDINAL RESTRAINERS

The following shall supplement A3.10.9.5.

Restrainers may only be used with the prior approval of the Chief Bridge Engineer.

3.11 EARTH PRESSURE: EH, ES, LS AND DD

3.11.1 General

The following shall supplement A3.11.1.

Both the vertical and horizontal components of an inclined lateral earth pressure shall be considered for application of load and load factors.

3.11.3 Presence of Water

The following shall supplement A3.11.3.

Walls along a stream or river shall be designed for a minimum differential water pressure due to a 1000 mm {3'-0"} head of water in the backfill soil above the weephole inverts.

C3.11.3

The following shall supplement AC3.11.3.

Evaluation of water pressures and seepage forces is critical in the design of retaining walls because water pressures and seepage forces are the most common causes of retaining wall failure. Seepage forces and water pressures affect the stability of retaining walls by:

- Increasing the weight of soil behind the wall through saturation, thereby increasing the driving soil pressure
- Decreasing the effective weight of soil in front of the wall through upward seepage forces, thereby reducing the resisting soil pressure
- Decreasing the effective stress (normal force) on the wall foundation due to wall weight through uplift, thereby reducing sliding resistance and resistance to overturning.

3.11.5 Earth Pressure: EH

C3.11.5.2 AT-REST PRESSURE COEFFICIENT, k_o

The following shall supplement AC3.11.5.2.

At-rest earth pressures are usually limited to bridge abutments to which superstructures are fixed prior to backfilling (e.g., framed bridges) or to cantilevered walls where the heel is restrained and the base/stem connection prevents rotation of the stem.

C3.11.5.3 ACTIVE PRESSURE COEFFICIENT, k_a

The following shall supplement AC3.11.5.3.

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The differences between the Coulomb Theory currently specified, and the Rankine Theory specified in the past is illustrated in Figure C1. The Rankine theory is the basis of the equivalent fluid method of A3.11.5.5 and the design procedures for mechanically stabilized earth walls.

Gravity and semi-gravity walls usually deflect a sufficient amount during backfilling to develop an active state of stress in the retained soil. This also is true of cantilevered and counterfort walls unless the heel is tied down or otherwise restrained and the base/stem connection prevents sufficient rotation of the stem to develop an active state of stress in the soil.

Wall movements cause the development of friction between the wall and the soil in contact with the wall. This resulting frictional force has the effect of inclining the earth pressure resultant on the wall, whereas the resultant would be normal to the wall in the case of no friction. The angle of inclination of the earth pressure resultant with respect to a line normal to the wall is called the angle of wall friction (δ).

3.11.5.4 PASSIVE PRESSURE COEFFICIENT, k_p

The following shall replace Figures A3.11.5.4-1 and A3.11.5.4-2.

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REDUCTION FACTOR (R) OF k_p FOR VARIOUS RATIOS OF $-\delta/\phi'$

ϕ' \ δ/ϕ'	−0.7	−0.6	−0.5	−0.4	−0.3	−0.2	−0.1	0.0
10	.978	.962	.946	.929	.912	.898	.881	.864
15	.961	.934	.907	.881	.854	.830	.803	.775
20	.939	.901	.862	.824	.787	.752	.716	.678
25	.912	.860	.808	.759	.711	.666	.620	.574
30	.878	.811	.746	.686	.627	.574	.520	.467
35	.836	.752	.674	.603	.536	.475	.417	.362
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	.600	.500	.414	.339	.339	.221	.174

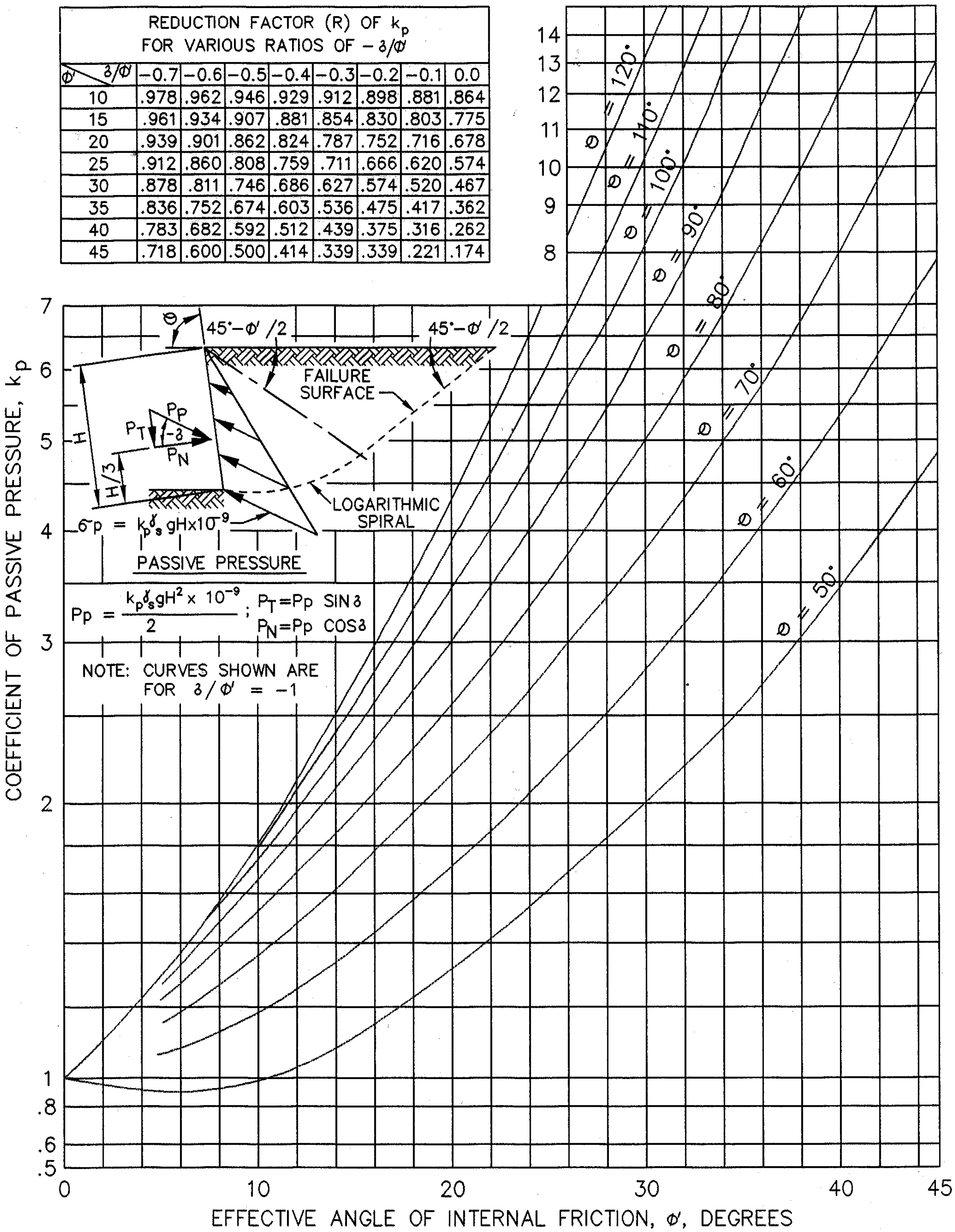

Figure 3.11.5.4-1 - Computational Procedures for Passive Earth Pressure for Sloping Wall with Horizontal Backfill

SPECIFICATIONS COMMENTARY

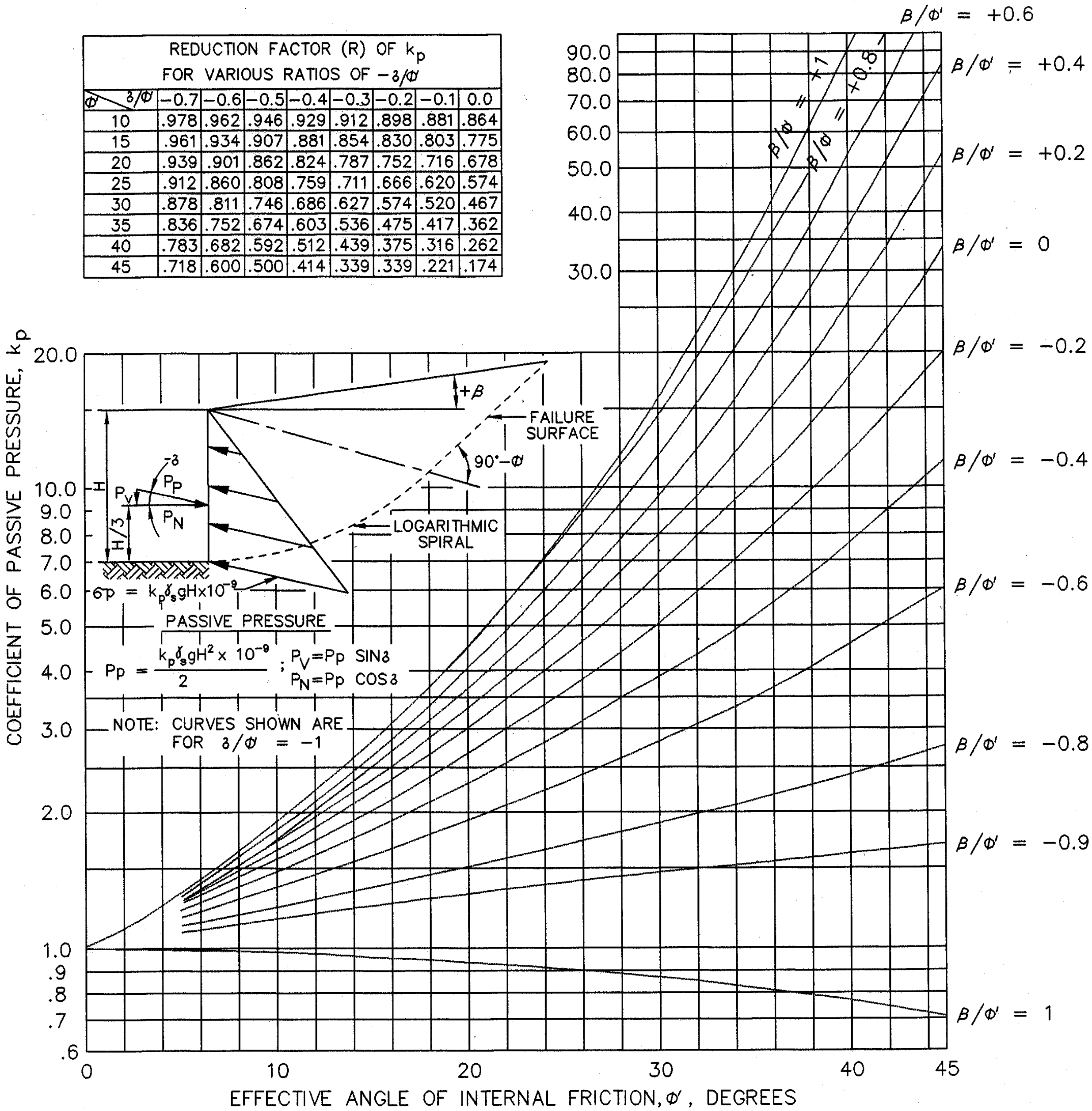

Figure 3.11.5.4-2 - Computational Procedures for Passive Earth Pressure for Vertical Wall With Sloping Backfill

3.11.5.5 EQUIVALENT-FLUID METHOD OF ESTIMATING EARTH PRESSURES

The following shall supplement A3.11.5.5.

Cohesionless soils with a maximum fines content of 5% by weight shall be used for backfill. This criteria can be met by backfilling with AASHTO No. 57 or the Department's open graded subbase (OGS) in conformance with Publication 408, Section 703.

For yielding walls backfilled with these materials, the design earth pressure at any depth shall be defined as increasing at a rate of 5.5×10^{-6} MPa/mm {0.035 ksf/ft},

C3.11.5.5

In the fifth paragraph of AC3.11.5.5, remove the reference to Figure AC3.11.5.3-1.

The following shall supplement AC3.11.5.5.

Soils with more than 5% fines shall be avoided as backfill because of their low permeability and potential frost susceptibility.

For design, the Department's open graded subbase (OGS) shall have the following assumed properties:

- moist density = 1920 kg/m^3 {0.120 kcf}

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plus the live load surcharge from A3.11.6.2 and D3.11.6.2.

For unyielding walls, restrained abutments (e.g., backfilled after superstructure erection), at-rest earth pressure, increasing at 8.0 x 10^{-6} MPa/mm {0.05 ksf/ft}, plus the live load surcharge from A3.11.6.2 and D3.11.6.2, shall be used.

The following shall supplement A3.11.5.5 for the design of box culverts.

For box culverts, equivalent fluid density shall be taken as specified in Table 2.

Table 3.11.5.5-2 - Equivalent Fluid Densities for Box Culverts

	Metric Units		U.S. Customary Units	
	Level Backfill (kg/m^3)	Backfill with β=25º (kg/m^3)	Level Backfill (kcf)	Backfill with β=25º (kcf)
Minimum	720	880	0.045	0.055
Maximum	1120	1280	0.070	0.080

These equivalent fluid densities along with the appropriate maximum and minimum load factors shall be selected to produce the extreme force effects.

3.11.5.6 EARTH PRESSURE FOR NONGRAVITY CANTILEVER WALLS

For permanent walls, the simplified earth pressure distributions shown in Figures 1 and 2, or other suitable earth pressure distributions, may be used. If walls will support or are supported by cohesive soils for temporary applications, walls may be designed based on total stress methods of analysis and undrained shear strength parameters. For this latter case, the simplified earth pressure distributions shown in Figures 3 and 4, or other approved earth pressure distributions, may be used with the following restrictions:

- The ratio of overburden pressure to undrained shear strength (i.e., stability number N = $10^{-9}\gamma gH/c$ {U.S. Customary Units: N = $\gamma H/c$}) must be < 3.

- The active earth pressure shall not be less than 0.25 times the effective overburden pressure at any depth, or 5.5 x 10^{-6} MPa/mm {0.035 ksf/ft} of wall height, whichever is greater.

For temporary walls with discrete vertical elements embedded in granular soil or rock, Figure 1 may be used to determine passive resistance and Figure 3 may be used to

COMMENTARY

- saturated density = 2160 kg/m^3 {0.135 kcf}

- angle of internal friction = 30°

The following shall supplement AC3.11.5.5 for the design of box culverts.

Two soil types were selected for design to reflect potential lateral at-rest earth pressures for box culverts, considering construction practice and soil variability in Pennsylvania. The engineered backfill required for a distance of only 300 mm {1 ft.} from the face of the culvert wall is not sufficient to reduce lateral earth pressures to levels that would be expected for abutments and retaining walls for which more detailed backfill requirements are specified. Lateral earth pressures resulting from the factored load combinations, specified in this article, A3.4.1 and D3.4.1 compare closely with past DM-4 practice.

Although the equivalent fluid weights given in Table 1 correspond to those for "Dense Sand or Gravel" and "Compacted Lean Clay", backfill material shall be in conformance with the requirements given in Publication 408 and the contract documents. Equivalent fluid weights specified herein are for design only. Values of the equivalent fluid pressure for a sloping backfill are provided for the rare case in which the culvert is parallel to the roadway. In such a case, consideration should be given to sliding as a result of the imbalance of lateral loads.

C3.11.5.6

Nongravity cantilevered walls temporarily supporting or supported by cohesive soils are subject to excessive lateral deformation if the undrained soil shear strength is low compared to the shear stresses. Therefore, use of these walls should be limited to soils of adequate strength as represented by the stability number (N = $10^{-9}\gamma gH/c$) {U.S. Customary Units: N = $\gamma H/c$}.

Base movements in the soil in front of a wall become significant for values of N of about 3 to 4, and a base failure can occur when N exceeds about 5 to 6, Terzaghi and Peck (1967).

SPECIFICATIONS COMMENTARY

determine the active earth pressure due to the retained soil.

SPECIFICATIONS COMMENTARY

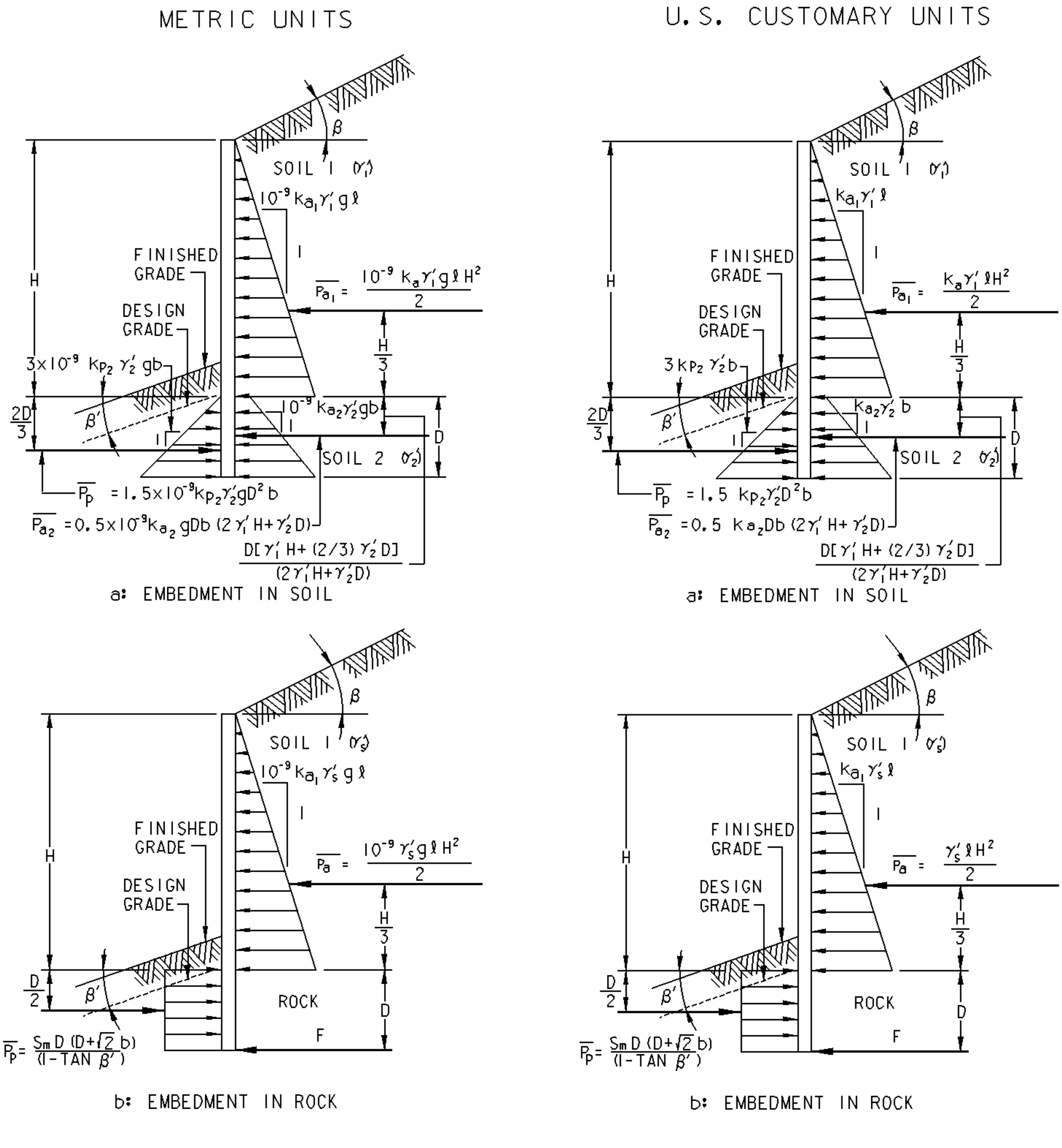

Figure 3.11.5.6-1 - Simplified Earth Pressure Distributions for Permanent Nongravity Cantilevered Walls with Discrete Vertical Wall Elements

SPECIFICATIONS COMMENTARY

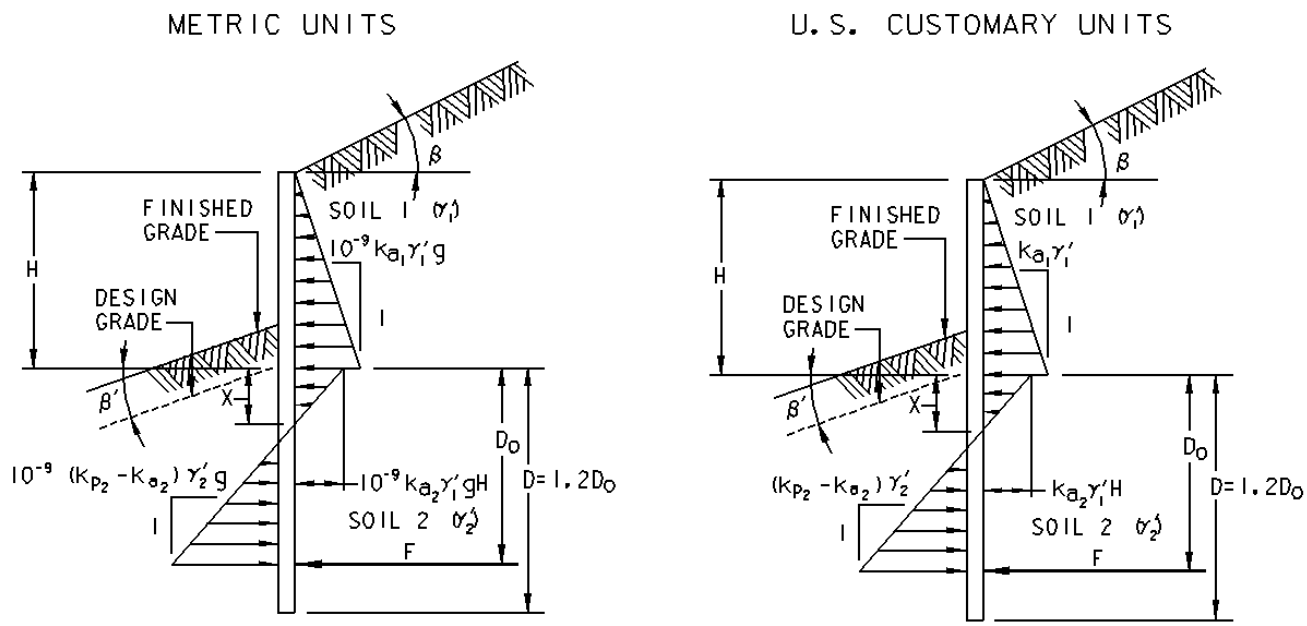

Figure 3.11.5.6-2 - Simplified Earth Pressure Distributions for Permanent Nongravity Cantilevered Walls with Continuous Vertical Wall Elements modified after Teng (1962)

SPECIFICATIONS COMMENTARY

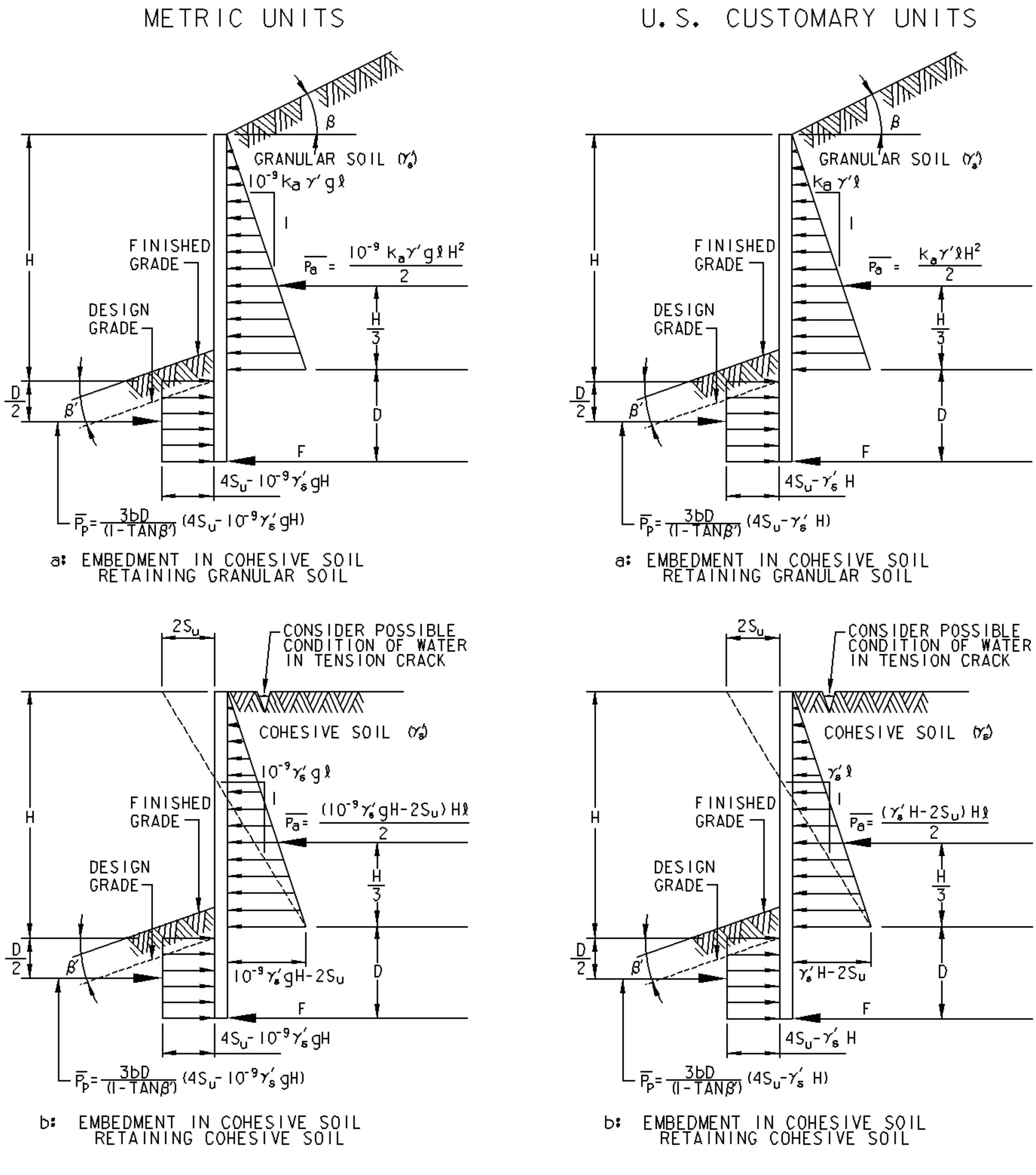

Figure 3.11.5.6-3 - Simplified Earth Pressure Distributions for Temporary Nongravity Cantilevered Walls with Discrete Vertical Wall Elements

SPECIFICATIONS COMMENTARY

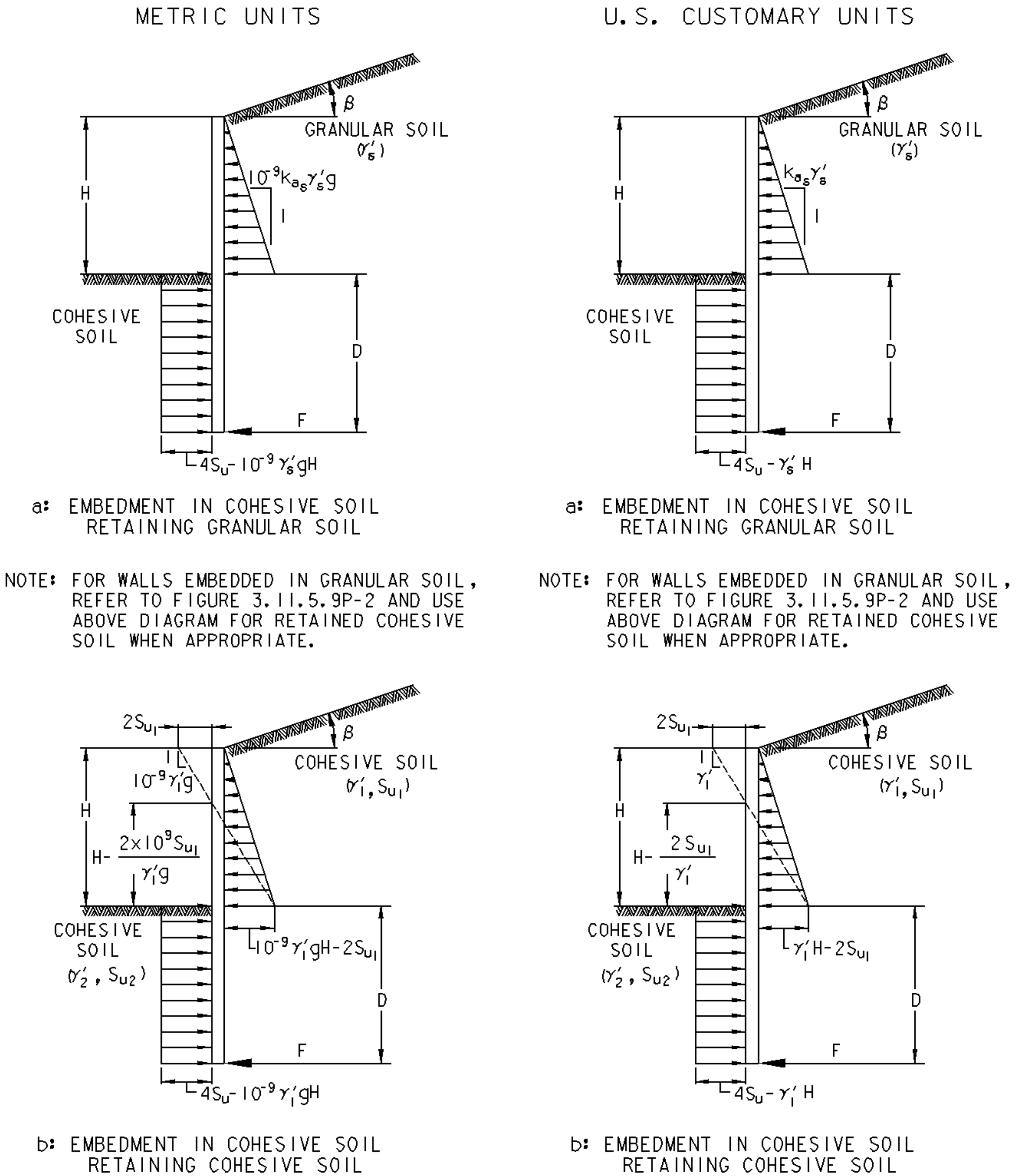

Figure 3.11.5.6-4 - Simplified Earth Pressure Distributions for Temporary Nongravity Cantilevered Walls with Continuous Vertical Wall Elements modified after Teng (1962)

SPECIFICATIONS

Where discrete vertical wall elements are used for support, the width of each vertical element shall be assumed to equal the width of the flange or diameter of the element for driven sections and the diameter of the concrete-filled hole for sections encased in concrete.

The magnitude and location of resultant loads and resisting forces for permanent walls with discrete vertical elements embedded in soil and rock for lateral support may be determined using the earth pressure distributions presented in Figures 1 and 3. The procedure for determining the resultant passive resistance of a vertical element embedded in soil assumes the net passive resistance is mobilized across a maximum of three times the element width or diameter (reduced, if necessary, to account for soft clay, or discontinuities in the embedded depth of soil or rock) and that the active pressure below the facing elements acts only on the actual vertical element width. For the embedded portion of a wall with discrete vertical elements, the net passive resistance shall not be taken greater than that for continuous embedded vertical elements as determined using Figure 2 for permanent walls and Figure 4 for temporary walls.

The magnitude and location of resultant loads and resisting forces for permanent walls with continuous vertical elements may be determined using the earth pressure distributions presented in Figure 2 for permanent walls and Figure 4 for temporary walls.

Some portion of the embedded depth below finished grade, noted as β' in Figures 1-3, (usually 900 mm {3 ft.} for an element in soil, and 300 mm {1 ft.} for an element in rock) is ineffective in providing passive lateral support.

In developing the design lateral pressure, the lateral pressure due to water, live load surcharge, permanent point and line surcharge loads, backfill compaction, or other types of surcharge loads shall be added to the lateral earth pressure.

3.11.5.7 APPARENT EARTH PRESSURES FOR ANCHORED WALLS

COMMENTARY

In Figures 1 and 3, the width of discrete vertical wall elements effective in mobilizing the passive resistance of the soil is based on a method of analysis by Broms (1964a and 1964b) for single vertical piles embedded in cohesive or cohesionless soil and assumes a vertical element. The effective width for passive resistance of three times the element width (b) is due to the arching action in soil and side shear on resisting rock wedges. The maximum width of 3b can be used when material in which the vertical element is embedded is intact. This width shall be reduced if planes or zones of weakness would prevent mobilization of resistance through this entire width. If the element is embedded in soft clay having a stability number less than 3, soil arching will not occur and the actual width shall be used as the effective width for passive resistance. Where a vertical element is embedded in rock (Figure 1b), the passive resistance of the rock is assumed to develop through the shear failure of a rock wedge equal in width to the vertical element (b) and defined by a plane extending upward from the base of the element at an angle of 45°.

The upper 600 to 900 mm {2 to 3 ft.} of the discrete embedded vertical element in soil, or 300 mm {1 ft.} in rock, is typically assumed ineffective in mobilizing passive resistance to account for the effects of freezing and thawing, weathering or other shallow ground disturbance (e.g., utility excavations or pavement replacement in front of the wall).

C3.11.5.7

The following shall supplement the first paragraph of

SPECIFICATIONS

The following shall replace A3.11.5.7:

For anchored walls constructed from the top down, the earth pressure may be estimated in accordance with Articles D3.11.5.7.1 or D3.11.5.7.2.

COMMENTARY

AC3.11.5.7.

The earth pressure diagrams used herein are primarily intended for use in homogeneous soils. They should not be used indiscriminately in stratified or relatively non-homogeneous soil layers; engineering judgment must be used in these cases.

When anchors, especially those near the top of the wall, are tensioned to loads in excess of those estimated using the apparent pressure diagrams, it is possible that the wall could be displaced back into the soil mass, resulting in undesirable deflections or a passive failure of the retained soil. It is important to remember that anchored walls are flexible and that they derive their satisfactory performance from a match between the soil pressure and the wall-anchor loads.

3.11.5.7.1 Cohesionless Soils

The following shall supplement A3.11.5.7.1:

The apparent earth pressure distribution for temporary and permanent anchored walls constructed from the top down and supporting cohesionless soil may be determined using Figures 1(a) and 1(b). Water pressures and surcharge pressures, if applicable, should be added explicitly to the diagrams to evaluate the total lateral load acting on the wall. Determine geostatic water pressure on the wall using the maximum expected water table differential between excavation interior and exterior, based on borings or other information.

In both Figures 1(a) and 1(b), calculate the maximum pressure ordinate p as indicated in the figures.

The earth pressure total load P_a per unit length of wall may then be calculated from the area of the apparent earth pressure distribution diagram as:

Metric Units:

$$P_a = 0.65 k_a \gamma'_S \, g H^2 (10^{-9}) \qquad (3.11.5.7.1\text{-}3)$$

U.S. Customary Units:

$$P_a = 0.65 k_a \gamma'_S \, H^2$$

where:

P_a = total earth pressure load per unit length of wall (N/mm of wall) {kip/ft of wall}

k_a = active earth pressure coefficient (dim)
= $\tan^2(45-\phi_f/2)$ for $\beta=0$
use Equation A3.11.5.3-1 for $\beta \neq 0$

γ'_S = effective unit weight of soil (kg/m^3) {kcf}

H = total excavation depth (mm) {ft.}

C3.11.5.7.1P

Anchored walls are typically constructed with free-draining material placed immediately behind the lagging, and therefore geostatic water pressure on the wall would not be of concern. However, there may be conditions of a permanent water table behind the wall where geostatic water pressure needs to be considered.

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H_1 = distance from ground surface to uppermost ground anchor (mm) {ft.}

H_{n+1} = distance from base of excavation to lowermost ground anchor (mm) {ft.}

T_{hi} = horizontal load in anchor I (N/mm of wall) {kip/ft of wall}

R = reaction force to be resisted by subgrade (i.e., below base of excavation) (N/mm of wall) {kip/ft of wall}

SPECIFICATIONS

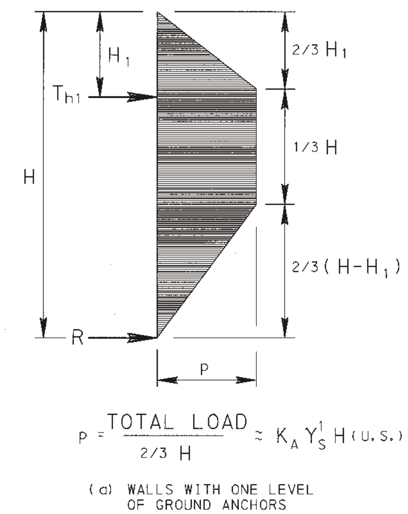

Figure 3.11.5.7.1-1(a) – Apparent Earth Pressure Distribution for Anchored Walls Constructed from the top down in Cohesionless Soils

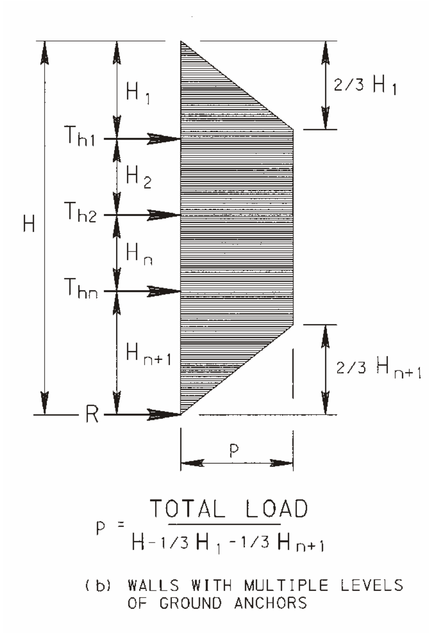

Figure 3.11.5.7.1-1(b) – Apparent Earth Pressure Distribution for Anchored Walls Constructed from the top down in Cohesionless Soils

COMMENTARY

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3.11.5.7.2 Cohesive Soils

The following shall replace A3.11.5.7.2:

The apparent earth pressure distribution for cohesive soils is related to the stability number, N_s, which is defined as:

Metric Units:

$$N_S = \frac{\gamma_S gH(10^{-9})}{S_u} \quad (3.11.5.7.2\text{-}1)$$

U.S. Customary Units:

$$N_s = \frac{\gamma_S H}{S_u}$$

where:

N_S = stability number (dim)

γ_S = total unit weight of soil (kg/m^3) {kcf}

H = total excavation depth (mm) {ft.}

S_u = average undrained shear strength of soil (MPa) {ksf}

Use the undrained shear strength of the soil through which the excavation extends.

3.11.5.7.2a Stiff to Hard, Including Fissured Cohesive Soils

The following shall replace A3.11.5.7.2a:

The apparent earth pressure distribution for temporary anchored walls constructed from the top down and supporting stiff to hard cohesive soils (NS$\leq$ 4) including fissured clays, where temporary conditions are of a controlled short duration and for which there is no available free water, may be determined using Figures 3.11.5.7.1-1(a) and 3.11.5.7.1-1(b). The identified terms remain the same, except that the maximum pressure ordinate of the diagram p shall be calculated as:

Metric Units:

$$p = 0.2\gamma_S gH\left(10^{-9}\right) to\ 0.4\gamma_S gH\left(10^{-9}\right) \quad (3.11.5.7.2a\text{-}1)$$

U.S. Customary Units:

$$p = 0.2\gamma_s H\ to\ 0.4\gamma_s H$$

COMMENTARY

C3.11.5.7.2P

Cohesive soils with a stability number $N_S \leq 4$ are to be considered to be stiff to hard in consistency. Cohesive soils with a stability number $N_S > 4$ are to be considered very soft to medium-stiff in consistency

C3.11.5.7.2a

The following shall supplement AC3.11.5.7.2:

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where:

p = maximum pressure ordinate (MPa) {ksf}

γ_S = total unit weight of soil (kg/m^3) {kcf}

H = total depth of excavation (mm) {ft.}

Surcharge pressures, if applicable, should be added explicitly to the diagrams to evaluate the total lateral load acting on the wall.

For other temporary conditions and for permanent conditions, calculate the earth pressure resultant using the maximum pressure ordinate obtained by either Equation 1 or Equation 3.11.5.7.1-1 with a value of k_a based on the drained friction angle of the clay. For any case, surcharge pressures, if applicable, should be added explicitly to the diagrams to evaluate the total lateral load acting on the wall. For conditions where there is available free water, determine geostatic water pressure on the wall using the maximum expected water table differential between excavation interior and exterior, based on borings or other information.

For permanent walls, the distribution (permanent or temporary) resulting in the maximum total force shall be used for design.

Alternatively, in fissured clays the apparent earth pressure diagram may be based upon previous successful experience with excavations constructed in similar soils. This is because earth pressures in these soils are most influenced by degree of fissuring or jointing in the clay and the potential reduction in strength with time, not necessarily the shear strength of the intact clay.

3.11.5.7.2b Very Soft to Medium-Stiff Cohesive Soils

The following shall replace A3.11.5.7.2b:

The apparent earth pressure distribution for temporary and permanent anchored walls constructed from the top down and supporting soft to medium-stiff cohesive soils may be determined using Figure 1. Soft to medium-stiff cohesive soils are those with a stability number $N_S > 4$.

COMMENTARY

There may be conditions of a permanent water table behind the wall where geostatic water pressure needs to be considered.

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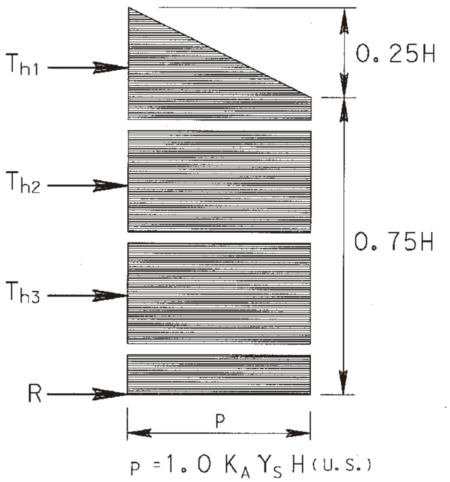

Figure 3.11.5.7.2b-1 - Apparent Earth Pressure Distribution for Anchored Walls Constructed from the top down in Soft to Medium-Stiff Cohesive Soils

Calculate the maximum pressure ordinate of the diagram p as:

Metric Units:

$$p = 1.0 k_a \gamma_S gH (10^{-9}) \qquad (3.11.5.7.2b\text{-}1)$$

U.S. Customary Units:

$$p = 1.0 k_a \gamma_S H$$

where:

k_a = 0.22 for $4 \le N_S < 5.14$

or

Metric Units:

$$k_a = 1 - m \frac{4S_u}{\gamma_S gH(10^{-9})} \qquad (3.11.5.7.2b\text{-}2)$$

U.S. Customary Units:

$$k_a = 1 - m \frac{4S_u}{\gamma_S H}$$

for $N_S \ge 5.14$, and using m = 0.4

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Additionally, if $N_S \geq 6$ and the excavation is underlain by soft clay, calculate k_a by Equation 2 and Equation 3 below, and use the larger of the 2 k_a values in Equation 1 to calculate the maximum pressure ordinate.

Metric Units:

$$k_a = 1 - \frac{4S_u}{\gamma_S gH(10^{-9})} + \frac{2\sqrt{2}d}{H}\left\{1 + \frac{\Delta H}{H}a - \frac{S_{ub}}{\gamma_S gH(10^{-9})}b\right\}$$

U.S. Customary Units:

$$k_a = 1 - \frac{4S_u}{\gamma_S H} + \frac{2\sqrt{2}d}{H}\left\{1 + \frac{\Delta H}{H}a - \frac{S_{ub}}{\gamma_S H}b\right\}$$

where

$$a = 1 + \frac{H + \Delta H}{(2 - x)\sqrt{2}d} \text{ (dim)}$$

$$b = 5.14 + \frac{2S_u \Delta H}{\sqrt{2}S_{ub}d} \text{ (dim)}$$

S_u = undrained shear strength of retained soil (MPa) {ksf}

S_{ub} = undrained shear strength of soil providing bearing resistance (MPa) {ksf}

d = depth of the potential base failure surface below the base of excavation (mm) {ft.}

γ_S = total unit weight of retained soil (kg/m^3) {kcf}

ΔH = depth of unloading at ground surface, if any (mm) {ft.}

X = length of unloading at top of anchored wall excavation, if any (mm) {ft.}

The value of d is taken as the thickness of soft to medium-stiff cohesive soil below the excavation base up to a maximum value of $B_c\sqrt{2}$, where B_c is the excavation width.

In any case, surcharge pressures, if applicable, should be added explicitly to the diagrams to evaluate the total lateral load acting on the wall. For conditions where there is available free water, determine geostatic water pressure on the wall using the maximum expected water table differential between excavation interior and exterior, based

COMMENTARY

There may be conditions of a permanent water table behind the wall where geostatic water pressure needs to be considered.

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on borings or other information.

3.11.5.8 EARTH PRESSURES FOR MECHANICALLY STABILIZED EARTH WALLS

3.11.5.8.1 General

The following shall replace the definition of k_a in A3.11.5.8.1:

k_a = active earth pressure coefficient specified herein

The following shall supplement A3.11.5.8.1:

Lateral earth pressure coefficients for MSE walls may be determined as follows:

- for a horizontal or sloping backfill surface, as shown in Figures A3.11.5.8.1-1 and A3.11.5.8.1-2, active earth pressure coefficient, k_a, in determining safety against soil failure may be taken as:

$$k_a = \cos\beta \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi_f}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi_f}} \qquad (3.11.5.8.1\text{-}2)$$

β = slope of backfill behind wall (DEG)

ϕ_f = internal friction angle of backfill soil (DEG)

- for a broken back backfill surface, the active earth pressure coefficient, k_a, for evaluation of safety against soil failure may be taken as:

$$\mathbf{k}_a = \cos\beta \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi_f}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi_f}} \qquad (3.11.5.8.1\text{-}3)$$

where:

B = notional slope of backfill behind wall as shown in Figure A3.11.5.8.1-3 (DEG)

ϕ_f = internal friction angle (DEG)

- active earth pressure coefficient, k_a, for determining safety against structural failure:

$$k_a = \tan^2\left(45^o - \frac{\phi_f}{2}\right) \qquad (3.11.5.8.1\text{-}4)$$

3.11.6 Surcharge Loads: ES and LS

3.11.6.4 LIVE LOAD SURCHARGE: LS

C3.11.6.4

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The following shall replace Table A3.11.6.4-1.

Table 3.11.6.4-1 - Equivalent Height of Soil forVehicular Loading – Abutment

Metric Units		U.S. Customary Units	
Wall Height (mm)	H_{eq} (mm)	Wall Height (ft)	H_{eg} (ft)
≤ 1500	1200	≤ 5.0	4.0
≥ 3000	900	≥ 10.0	3.0

The following shall supplement A3.11.6.4.

The minimum design surcharge values for abutments in Table 1 are intended to account for normal traffic live loads and do not address the effects of backfill compaction. Refer to A3.11.2 to determine the effects of backfill compaction. For retaining walls, use Table 2.

Table 3.11.6.4-2 - Equivalent Height of Soil (h_{eq}) for Vehicular Loading - Retaining Walls

Metric Units			U.S. Customary Units		
Wall Height (mm)	Distance from back face of wall to the wheel line		Wall Height (ft)	Distance from back face of wall to the wheel line	
	0.0mm	300mm		0.0 ft	1 ft
≤ 1500	1500	900	≤ 5.0	5.0	3.0
3000	1050	900	10.0	3.5	3.0
≥ 4000	900	900	≥ 13.0	3.0	3.0

For box culverts, use 900 mm {3.0 ft} where live load effects are considered.

3.12 FORCE EFFECTS DUE TO SUPERIMPOSED DEFORMATIONS: TU, TG, SH, CR, SE

3.12.2 Uniform Temperature

3.12.2.1.1 TEMPERATURE RANGES

The following shall replace A3.12.2.1.1.

Provision shall be made for forces and movements resulting from variations in temperature. The range of temperature with respect to the normal erection temperature of 20° C {68° F} shall be as given in Table 1.

COMMENTARY

Delete the third paragraph of AC3.11.6.4.

The following shall supplement AC3.11.6.4.

In the development of this specification, the Department had a comparison made between their past abutment and retaining wall service load design method and the LRFD method. With minor modifications contained in this specification, the LRFD method gave similar results to the Department's past design method with one exception. For walls less than 1500 mm {5 ft.} in height on poor soils, the LRFD method may require base width significantly larger than past designs. Since the Department has not experienced problems with short headwalls for pipe culverts, the Standard Drawings may be used for headwalls for pipe culverts.

In Table D.3.11.6.4-2, the distance from back face of wall to edge of traveled way of 0.0 mm {0 ft} corresponds to placement of a point wheel load 600 mm {2 ft} from the back face of the wall. For the case of the uniformly distributed lane load, the 0.0 mm {0 ft} distance corresponds to the edge of the 3000 mm {10 ft} wide traffic lane.

C3.12.2.1

The following shall supplement AC3.12.2.1.1.

The inclusion of an additional temperature fall of 32° C is based on a Departmental study conducted in District 3-0. It was determined that the fixity at the connections of continuous spans produces a frame-type action that induces additional forces.

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Table 3.12.2.1.1-1 – Procedure A Temperature Ranges

Material	Metric Units		U.S. Customary Units	
	Temperature Rise	Temperature Fall	Temperature Rise	Temperature Fall
Steel or Aluminum Structures	23° C	43° C	42° F	78° F
Concrete Structures	18° C	32° C	32° F	58° F
Wood Structures	4° C	8° C	7° F	14° F
	Neoprene	Other	Neoprene	Other
Bearings (Prestressed Concrete Structures) Temp. Range	45° C	64° C	80° F	116° F
Bearings (Steel or Aluminum Structures) Temp. Range	56° C	86° C	100° F	156° F

For the design of integral abutments, the temperature range given for bearings shall be used.

3.12.3 Temperature Gradient

The following shall supplement A3.12.3.

The load factor for temperature gradient shall be taken as zero for those bridges which can be analyzed by the approximate methods given in A4.6.2 and D4.6.2, and are of Type a, b (only precast P/S concrete box girders), e, f, g, h, j, k and l as given in Table A4.6.2.2.1-1.

For Pennsylvania bridges other than those listed above, the Zone 3 data shall be used as given in Table A3.12.3-1.

C3.12.3

The following shall supplement AC3.12.3.

Pennsylvania has not experienced any temperature gradient-related problems in their typical multi-girder bridges. Therefore, as suggested in AC3.12.3, the Department's experience with typical multi-girder bridges has led them to exclude the temperature gradient load condition for these types of bridges.

3.12.7P Minimum Temperature Force for Fixed Substructures

When neoprene bearings are used, the fixed substructure unit(s) shall consider a thermal force equal to the largest thermal force from the largest expansion bearing substructure unit or utilize the results of an equilibrium analysis, whichever is larger.

C3.12.7P

This provision insures that fixed substructures are designed for a minimal thermal force even if an equilibrium analysis indicates no thermal forces are present. This is similar to the forces applied to steel bearings considering frozen bearings.

3.13 FRICTION FORCES: FR

The following shall supplement A3.13.

Friction force acts parallel to the direction of movement and is assumed to act at the bearing elevation at each expansion bearing, with due consideration given to the reactions that must develop at the fixed bearings to satisfy equilibrium. See A14.6.3.1 for horizontal forces.

Consideration of frozen expansion bearings and variation of friction is provided assuming the largest pier or abutment DL reaction times the applicable friction coefficient acts at the fixed pier or utilize the results of an equilibrium analysis, whichever is larger.

C3.13

The following shall replace AC3.13.

Low and high friction coefficients may be obtained from standard textbooks. If so warranted and approved by the Chief Bridge Engineer, the values may be determined by physical tests, especially if the surfaces are expected to be roughened in service.

When a force is transmitted from the superstructure to the substructure through a sliding bearing, the force applied to the substructure is considered a frictional force. However, forces transmitted, via a non-sliding bearing such as an elastomeric bearing, are factored by the appropriate load factor for the driving effect, such as TU.

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3.14 VESSEL COLLISION: CV

3.14.1 General

The following shall supplement A3.14.1.

The vessel collision provisions provided in A3.14 and D3.14 shall only be used in the substructure design of bridges which cross a navigable waterway. The Department defines a navigable waterway as those waterways which:

- presently support commercial barge and/or ship traffic, and
- have supported commercial barge and/or ship traffic within the past 20 years.
- There is some reason to believe that the waterway will support commercial barge and/or ship traffic in the future.

3.14.2 Owner's Responsibility

The following shall replace A3.14.2.

When the vessel collision provisions are applicable according to D3.14.1, the designer must submit at the Type, Size and Location stage the following information for review by the Department:

- vessel traffic density in the waterway
- design velocity of vessels for the bridge
- suggested degree of damage that the bridge components, including protective systems are allowed to sustain

3.15P FORCE TRANSFER TO SUBSTRUCTURE

3.15.1P Longitudinal Force

3.15.1.2P FORCE TRANSFER TO SUBSTRUCTURE

Longitudinal forces, except friction (see D3.13), shall be carried only by fixed bearings.

3.15.1.3P EFFECTIVE LENGTH FOR SUPERSTRUCTURE FORCES

COMMENTARY

C3.14.1

The following shall supplement AC3.14.1.

For the vast majority of bridges over waterways in Pennsylvania, the vessel collision provisions will not be applicable.

The vessel collision provisions will most likely be applicable for bridges over the following waterways:

- lower portions of Delaware River
- lower portions of Schuylkill River
- lower portions of Allegheny River
- lower portions of Monongahela River
- Ohio River

C3.14.15 Protection of Substructures

The following shall supplement AC3.14.15.

Any testing for protection systems of substructures must be approved by the Chief Bridge Engineer.

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Longitudinal forces transmitted to the substructure from the superstructure shall be calculated using the center-to-center bearing length of superstructure restrained by fixed bearings. In the case of consecutively fixed piers, forces to the substructure shall be determined with due consideration to the relative stiffness of the piers.

3.15.1.4P FORCE RESOLUTION TO SUBSTRUCTURE

Longitudinal forces from the superstructure shall be directly applied at the bearings and shall be resolved in the directions perpendicular and parallel to the substructure, as shown in Figure 1. For frame analysis of the substructure, an equivalent parallel component shall be used, as shown in Figure 2.

For structures on a sloping grade with an inclined bearing plate, the reaction component parallel to the grade (longitudinal force) shall be considered.

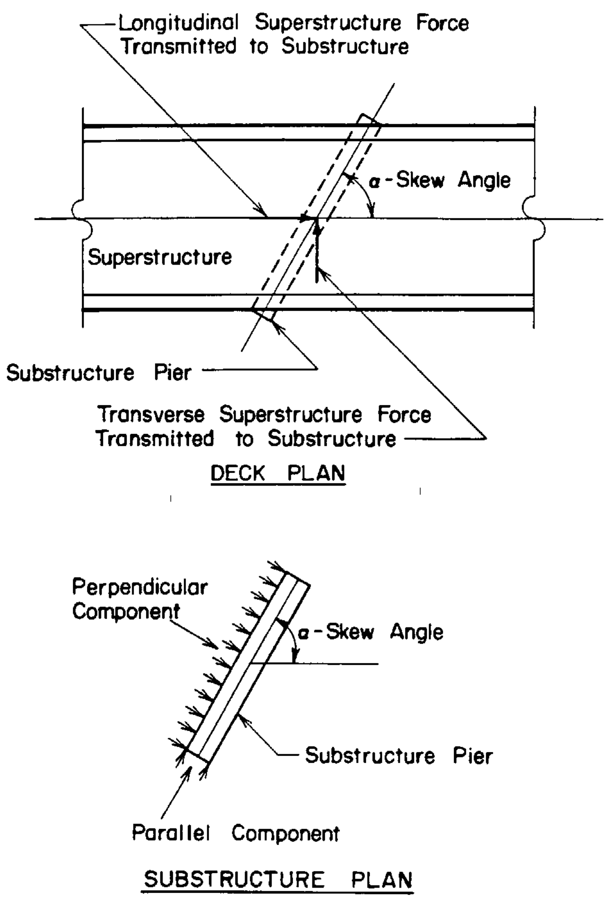

Figure 3.15.1.4P-1 - Force Resolution to Substructure

SPECIFICATIONS

COMMENTARY

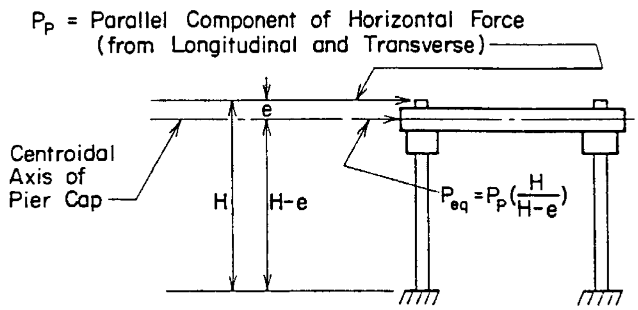

Figure 3.15.1.4P-2 - Equivalent Force for Frame Analysis of Substructure

3.15.2P Transverse Force

3.15.2.1P FORCE TRANSFER TO SUBSTRUCTURE

The transverse forces applied to the superstructure must be resisted by the bearings.

3.15.2.2P EFFECTIVE LENGTHS FOR SUPERSTRUCTURE FORCES

Unless a more rational method of analysis is used, transverse forces acting on a superstructure shall be transmitted to the bearings using the following span lengths:

Continuous Spans	Piers	Average of the two adjacent spans
	Abutments	One-half of the end span
Simple Spans		One-half of the span

3.15.2.3P FORCE RESOLUTION TO SUBSTRUCTURE

Transverse forces from the superstructure shall be resolved in the directions perpendicular and parallel to the substructure, as shown in Figure D3.15.1.4P-1.

3.15.2.4P DETERMINATION OF BEARING REACTIONS

The effect of the transverse force applied at the elevation specified for that force shall be taken into account in determining the vertical reactions at the bearings (see Figure 1).

SPECIFICATIONS

COMMENTARY

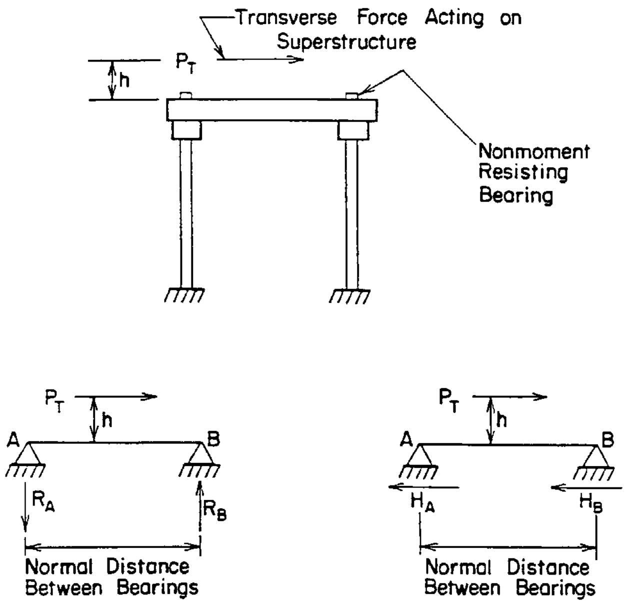

Figure 3.15.2.4P-1 - Bearing Reactions

REFERENCES

American Concrete Pipe Association (ACPA), Concrete Pipe Handbook, Vienna, VA, 435 p., 1981

American Society of Civil Engineers (ASCE), Design and Construction of Sanitary and Storm Sewer, prepared by a joint committee of the ASCE and the Water Pollution Control Federation (WPCF), ASCE - Manuals and Reports of Engineering Practice - No. 37 (WPCF Manual of Practice No. 9), 1969, 350 p.

Broms, B. B., "Lateral Resistance of Piles in Cohesive Soils", Journal of Soil Mechanics and Foundation Engineering Division, ASCE, Vol. 90, No. SM2, pp. 27-64, 1964a.

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Teng, W. C., "Foundation Design", Prentice-Hall, Englewood Cliffs, New Jersey, 466 pp., 1962

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 1
PART B: DESIGN SPECIFICATIONS

SECTION 4 - STRUCTURAL ANALYSIS AND EVALUATION

SECTION 4 - TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

SPECIFICATIONS

4.1 SCOPE

The following shall replace the last paragraph of A4.1.

Bridge structures shall be analyzed elastically, except as noted herein. An inelastic analysis of bridge structures may only be used for Extreme Event Limit States with the approval of the Chief Bridge Engineer.

COMMENTARY

C4.1

Delete the last sentence of the last paragraph of AC4.1.

4.2 DEFINITIONS

The following shall supplement A4.2.

Automatic Mesh Generator - Program or subprogram which creates the layout (arrangement of nodes and elements) of a model for the user if certain basic information is provided.

Influence Surface - Curved surface on which the ordinate is the value of the function (shear, moment, reaction, etc.) when a unit load is placed at the ordinate for a member location (centerline of a girder, support, etc.).

Influence Surface Loader - Computer program or portion of a computer program which calculates and maximizes the value of a function (shear, moment, reaction, deflection, etc.) by using the influence surface for that function.

Line Girder Analysis - Analysis of a bridge in which each girder is removed and analyzed as a single non-interacting element.

Loading Algorithm - Methodology used by the influence surface loader to calculate the moments, shear, etc.

PEP - Project Engineering Panel of Applied Technology Council

Refined Analysis - Analysis according to A4.6.3 and D4.6.3.

Shear Lag - Nonuniform stress pattern due to ineffective transmission of shear.

Skew Angle - Angular measurement between the base line of the bridge and centerline of the pier; a 90° skew angle defining a right bridge.

St. Venant Torsion - Uniform torsion resulting in no deformation of the cross-section.

Three-Dimensional Finite Element Analysis - Analysis in which a three-dimensional continuum is modeled as an assemblage of discrete elements in three-dimensional space.

Warping Torsion - Nonuniform torsion resulting in warping of the cross-section.

4.4 ACCEPTABLE METHODS OF STRUCTURAL ANALYSIS

The following shall supplement A4.4.

The designer shall also follow the requirements in PP1.4 in regards to computer programs.

Any computer program for the "3D or refined" analysis of girder bridges which has not been reviewed by the Department shall be submitted to, and approved by, the Chief Bridge Engineer prior to its use. A sample bridge(s) selected by the Department is to be modeled with the program so that the Department can make comparisons between its reviewed programs and the proposed program.

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COMMENTARY

Computer programs for the analysis of girder bridges approved for use (on LFD design projects) are listed in Appendix J. Only the version of a program listed in Appendix J has been tested and approved. If any changes and/or modifications have been made to a program since its approval date, then re-approval of the program is required. The approval of these programs is subject to the following conditions and limitations:

1. While certain software packages provide design optimization and/or code compliance checks, these aspects were not included in the review and approval process. Acceptance has been based solely upon the review of generalized design forces (moments, shears, reactions, etc.), as calculated by the software.

2. Acceptance of a software package by the Department does not affect the responsibility of the user for the proper application of the software and interpretation of its results. The acceptance of a software package does not constitute an endorsement nor does it relieve the vendor and the designer from their responsibility for accurate, technically correct and sound engineering results and services to the Department.

3. The Department's acceptance does not constitute any form of implied warranty, including warranty of merchantability and fitness for a particular purpose. The Commonwealth makes no warranty or representation, either expressed or implied, with respect to this software or accompanying documentation, including their quality performance, merchantability, or fitness for a particular purpose. In addition, the Commonwealth will not be liable for any direct, indirect, special, incidental, or consequential damages arising out of the use, inability to use, or any defect in the software or any accompanying documentation.

4.5 MATHEMATICAL MODELING

4.5.1 General

The following shall replace the second paragraph of A4.5.1.

Barriers shall not be considered in the calculation of the structural stiffness nor structural resistance of a structure.

The following shall supplement A4.5.1.

Centerline distances shall be used in the analysis of continuous frames, such as boxes, arches and pier bents.

4.5.2 Structural Material Behavior

4.5.2.2 ELASTIC BEHAVIOR

The following shall supplement A4.5.2.2.

For simple and continuous spans, composite stiffness

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shall be used if a concrete deck is used.

4.5.2.3 INELASTIC BEHAVIOR

The following shall replace the first sentence of the second paragraph of A4.5.2.3.

The inelastic model shall be based either upon the results of physical tests or upon a representation of load deformation behavior which is validated by tests, but either method must be approved by the Chief Bridge Engineer.

4.6 STATIC ANALYSIS

4.6.1 Influence of Plan Geometry

4.6.1.2 STRUCTURES CURVED IN PLAN

4.6.1.2.1 General

The following shall supplement A4.6.1.2.1.

Bridges which have kinked girders shall use the provisions of A4.6.1.2.1 to determine if they are to be considered curved.

For the design of horizontally curved steel girder highway bridges, a load and resistance factor design is required. A load factor design may also be permitted if requested at TS&L stage. For a load factor design, use the AASHTO, Guide Specification for Horizontally Curved Highway Bridges, AASHTO, Standard Specifications for Highway Bridges, and the 1993 Design Manual, Part 4. The force effects (i.e., moments, shear, reacting, etc.) for the curved bridge shall be determined using a refined method of analysis. (Please note that the above-referenced LFD documents are in U. S. Customary Units.)

4.6.2 Approximate Methods of Analysis

4.6.2.1 DECKS

4.6.2.1.3 Width of Equivalent Interior Strips

In Table A4.6.2.1.3-1 replace the entry for wood planks as follows:

The width of a primary wood plank strip spanning parallel to traffic shall be taken as 510 mm {20 in.}. The width of a primary wood plank strip spanning perpendicular to traffic shall be taken as the plank width, but not less than 250 mm {10 in.}.

4.6.2.1.8 Live Load Distribution on Fully Filled and Partially Filled Grids

The following shall replace A4.6.2.1.8.

Design in accordance with Tables per BD-604M.

COMMENTARY

C4.6.2.1.6 Calculation of Force Effects

Delete the first paragraph of AC4.6.2.1.6.

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The stiffness ratio, D, shall be taken as:

- for fully filled grids with at least 40 mm {1 1/2 in.} monolithic overfill 2.0
- for all other fully filled grids 2.5
- for partially filled grids with at least 40 mm {1 1/2 in.} monolithic overfill 8.0
- for all other partially filled grid 10.0

When approved by the Chief Bridge Engineer, the stiffness ratio, D, determined from test results may be used.

4.6.2.1.9 Inelastic Analysis

The following shall replace A4.6.2.1.9

The inelastic finite element analysis or yield line analysis are not permitted unless specifically approved by the Chief Bridge Engineer. If approved, this type of analysis is to be only used for Extreme Event Limit State.

4.6.2.2 BEAM-SLAB BRIDGES

4.6.2.2.1 Application

The following shall replace the third paragraph of A4.6.2.2.1.

For any variables exceeding the range of applicability, as specified in A4.6.2.2 and D4.6.2.2, the Chief Bridge Engineer must approve the method for determining the distribution factors.

The following shall supplement A4.6.2.2.1.

The articles in this section which provide approximate distribution factors are not applicable for bridges which are considered curved as defined in A4.6.1.2.1 and D4.6.1.2.1. For curved bridges, a refined method of analysis, as defined in A4.6.3 and D4.6.3, is required.

Additional requirements for skewed structures must be considered as follows:

- Apply the skew adjustment factors as given in A4.6.2.2.3c and D4.6.2.2.3c on all skewed structures as a minimum.
- Steel structures with a skew angle less than 70° require an additional check against uplift at the acute corners.
- Concrete structures with a skew angle less than 45° require an additional check against uplift at the acute corners.
- The design of bearings for bridges with skew angles less than 70° require consideration of out-of-plane rotations.

COMMENTARY

C4.6.2.2.1

The following shall replace the fifth sentence of the twelth paragraph of AC4.6.2.2.1.

The use of transverse mild steel rods secured by nuts, or similar unstressed dowels should not be considered sufficient to achieve full transverse flexural continuity unless demonstrated by test or experience and approved by the Chief Bridge Engineer.

The following shall supplement AC4.6.2.2.1.

AASHTO provides consideration of skew angle by way of moment and shear correction factors. PennDOT agrees with the application of the shear correction factors. PennDOT has decided not to take advantage of the reduction in load distribution factors for moment. However, these factors do not adequately address problems due to out-of-plane rotations, uplift, or cross-frame forces. The provisions in this section are meant to be applied to account for these items. Note that uplift on concrete structures is not considered as critical as that on steel structures.

During routine bridge inspections, the Department has found many occurrences of buckled cross-frame members and poor bearing performance on skewed structures. The Department has found from refined analyses that cross-frames in skewed structures are potentially subjected to higher force levels than cross-frames in normal (90°) structures. This does not mandate a 3-D analysis, but does

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- Steel structures with skew angles less than 70° require a special cross-frame design and the cross-frame members must be considered as main load carrying members.

Table 1 describes how the term L (length) shall be determined for use in the live load distribution factor equations given in A4.6.2.2.2, A4.6.2.2.3, D4.6.2.2.2 and D4.6.2.2.3.

COMMENTARY

mean a special analysis of the cross-frame must be provided in order to account for the differential deflections which occur across a cross-frame.

A crude or approximate uplift check can be made using the second term of the adjustment factor as an estimate of negative live load reaction potential. Comparing the negative reaction with the dead load reaction will provide an estimate of the potential for uplift. The engineer must use engineering judgement regarding the applicability of this method.

An acute corner is defined as the corner of the structure where the angle formed by intersection of edge of the deck and the centerline of bearings is less than 90°.

Proper consideration of out-of-plane rotations during the bearing design is also required. Normally out of plane rotations will require multi-rotational bearings.

This method incorporated in this manual for determining L seems to be appropriate for the level of sophistication of the distribution factors. As additional knowledge is gained on this subject, this method for determining L may be modified.

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Table 4.6.2.2.1-1 - L for Use in Live Load Distribution Factor Equations

CONDITION	FORCE EFFECT	L (mm) {ft.}
A	Positive Moment	The length of the span for which moment is being calculated.
B	Negative Moment - End spans of continuous spans, from end to point of dead load contraflexure	The length of the span for which moment is being calculated.
C	Negative Moment - Near interior supports of continuous spans, from point of dead load contraflexure to point of dead load contraflexure	The average length of the two adjacent spans.
D	Negative Moment - Interior spans of continuous spans, from point of dead load contraflexure to point of dead load contraflexure	The length of the span for which moment is being calculated.
E	Shear	The length of the span for which shear is being calculated.
F	Exterior Reaction	The length of the exterior span.
G	Interior Reaction of Continuous Span	The average length of the two adjacent spans.

Figure 1 provides a graphical representation of the information given in Table 1.

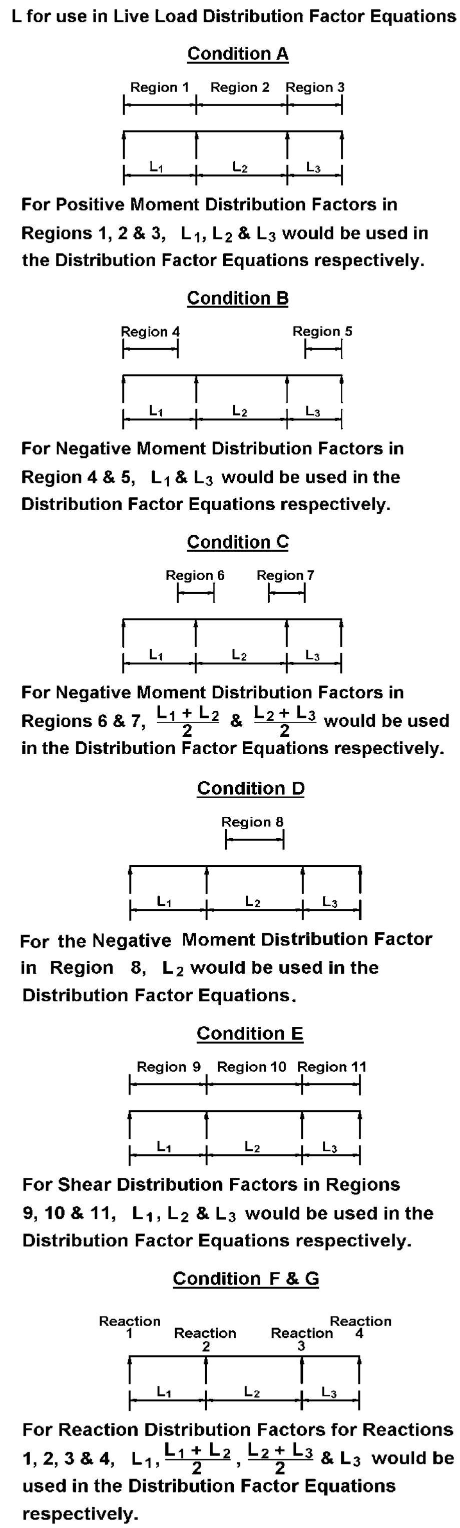

Figure 4.6.2.2.1-1 - L for use in Live Load Distribution Factor Equations

SPECIFICATIONS

In the rare occasion when the continuous span arrangement is such that an interior span does not have any positive dead load moment (i.e., no dead load points of contraflexure), the region of negative moment near the interior supports would be increased to the centerline of the span, and the L used in determining the live load distribution factors would be the average of the two adjacent spans.

4.6.2.2.2 Distribution Factor Method for Moment and Shear

4.6.2.2.2a Interior Beams with Wood Decks

The following shall supplement A4.6.2.2.2a.

The distribution factors given in Table A4.6.2.2.2a-1 for Glued Laminated Panels on Glued Laminated Stringers are applicable for panels with a 150 mm {6 in.} minimum nominal thickness.

4.6.2.2.2b Interior Beams with Concrete Decks

The following shall replace the second paragraph of A4.6.2.2.2b.

For preliminary design, the terms $K_g/(Lt^3)$ $\{K_g/(12Lt^3)\}$ in Table 1 shall be taken as 1.0 for non-composite beams.

The following shall replace Table A4.6.2.2.2b-1.

COMMENTARY

C4.6.2.2.2b

The following shall supplement AC4.6.2.2.2b.

In Table A4.6.2.2.2b-1, in the Category "Concrete Beams used in Multi-Beam Decks", the cross-section, Type g (from Table A4.6.2.2.1-1), with option "if sufficiently connected to act as a unit" has been removed from Table D4.6.2.2.2b-1. This option has been removed because it has been difficult to provide enough post-tensioning for the composite box beams to act as a unit.

Table 4.6.2.2.2b-1 – Distribution of Live Loads Per Lane for Moment in Interior Beams

Metric Units			
Type of Beams	**Applicable Cross-Section from Table A4.6.2.2.1-1**	**Distribution Factors**	**Range of Applicability**
Wood Deck on Wood or Steel Beams	a, 1	See Table A4.6.2.2.2a-1	
Concrete Deck on Wood Beams	1	One Design Lane Loaded: S/3700 Two or More Design Lanes Loaded: S/3000	$S \le 1800$
Concrete Deck, Filled Grid, or Partially Filled Grid on Steel or Concrete Beams: Concrete T-Beams, T- and Double T-Sections	a, e, k	One Design Lane Loaded: $0.06 + \left(\frac{S}{2930}\right)^{1.0}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{Lt_S^3}\right)^{0.1}$	$1100 \le S \le 4900$ $110 \le t_s \le 300$ $6000 \le L \le 152\,000$ If $L > 73\,000$, use $L = 73\,000$ $4x10^9 \le K_g \le 3.2 \times 10^{12}$ $N_b \ge 4$
	i, j if sufficiently connected t act as a unit	Two or More Design Lanes Loaded: $0.075 + \left(\frac{S}{3350}\right)^{1.0}\left(\frac{S}{L}\right)^{0.08}\left(\frac{K_g}{Lt_S^3}\right)^{0.1}$	
		Use lesser of the above or Lever Rule	$N_b = 3$
Cast-in-Place Concrete Multicell Box	d	One Design Lane Loaded: $\left(1.75+\frac{S}{1100}\right)\left(\frac{300}{L}\right)^{0.35}\left(\frac{1}{N_c}\right)^{0.45} \ge \frac{S}{6400}$ Two or More Design Lanes Loaded: $\left(\frac{13}{N_c}\right)^{0.3}\left(\frac{S}{430}\right)\left(\frac{1}{L}\right)^{0.25}$	$2100 \le S \le 4000$ $18\,000 \le L \le 152\,000$ If $L > 73\,000$, use $L = 73\,000$ $N_c \ge 3$ If $N_c > 8$ use $N_c = 8$ For two or more design lanes loaded If $L > 427\,000/N_c$, use $L = 427\,000/N_c$
Concrete Deck on Concrete Spread Box Beams	b, c	One Design Lane Loaded $\left(\frac{S}{910}\right)^{0.35}\left(\frac{Sd}{L^2}\right)^{0.25}$ Two or More Design Lanes Loaded: $\left(\frac{S}{1900}\right)^{0.6}\left(\frac{Sd}{L^2}\right)^{0.125}$	$1800 \le S \le 5500$ $6000 \le L \le 152\,000$ If $L > 43\,000$, use $L = 43\,000$ $430 \le d \le 1700$ $N_b \ge 3$
		Use Lever Rule	$S > 5500$

Table 4.6.2.2.2b-1 – Distribution of Live Loads Per Lane for Moment in Interior Beams (Continued)

Metric Units			
Type of Beams	**Applicable Cross-Section From Table A4.6.2.2.1-1**	**Distribution Factors**	**Range of Applicability**
Concrete Beams used in Multi-Beam Decks	f	One Design Lane Loaded: $k\left(\frac{b}{2.8L}\right)^{0.5}\left(\frac{I}{J}\right)^{0.25}$ where : $k = 2.5(N_b)^{-0.2} \geq 1.5$	$900 \leq b \leq 1500$ $6000 \leq L \leq 152\,000$ If $L > 37\,000$, use $L = 37\,000$ $5 \leq N_b \leq 20$
	g if sufficiently connected to act as a unit	Two or More Design Lanes Loaded: $k\left(\frac{b}{7600}\right)^{0.6}\left(\frac{b}{L}\right)^{0.2}\left(\frac{I}{J}\right)^{0.06}\left(\frac{5.66}{L^{0.15}}\right)^{\frac{N_b}{15}}$	$900 \leq b \leq 1500$ $6000 \leq L \leq 152\,000$ $5 \leq N_b \leq 20$ If $N_b > 12$, use $N_b = 12$
Steel Grids on Steel Beams	h	Regardless of Number of Loaded Lanes: S/D where: C=K(W/L)	
	g, i, j if connected only enough to prevent relative vertical displacement at the interface	$D=300[11.5 - N_L + 1.4N_L (1 - 0.2C)^2]$ when $C \leq 5$ $D=300 \{11.5 - N_L\}$ when $C > 5$ $K=\sqrt{\frac{(1+\mu)I}{J}}$ For preliminary design, the following values of K may be used: **Beam Type** — **K** Non-voided rectangular beams 0.7 Rectangular beams with circular voids 0.8 Box section beams 1.0 Channel beams 2.2 T-beam 2.0 Double T-beam 2.0	
Steel Grids on Steel Beams	a	One Design Lane Loaded: S/2300 If $t_g < 100$ mm S/3050 If $t_g \geq 100$ mm Two or More Design Lanes Loaded: S/2400 If $t_g < 100$ mm S/3050 If $t_g \geq 100$ mm	$S \leq$ 1800 mm $S \leq$ 1800 mm
Concrete deck on Multiple Steel Box Girders	b, c	Regardless of Number of Loaded Lanes: $0.05 + 0.85\frac{N_L}{N_b} + \frac{0.425}{N_L}$	$0.5 \leq \frac{N_L}{N_b} \leq 1.5$

Table 4.6.2.2.2b-1 – Distribution of Live Loads Per Lane for Moment in Interior Beams (Continued)

U.S. Customary Units			
Type of Beams	**Applicable Cross-Section from Table A4.6.2.2.1-1**	**Distribution Factors**	**Range of Applicability**
Wood Deck on Wood or Steel Beams	a, l	See Table A4.6.2.2.2a-1	
Concrete Deck on Wood Beams	l	One Design Lane Loaded: S/12 Two or More Design Lanes Loaded: S/10	$S \leq 6.0'$
Concrete Deck, Filled Grid, or Partially Filled Grid on Steel or Concrete Beams; Concrete T-Beams, T- and Double T-Sections	a, e, k i, j If sufficiently connected to act as a unit	One Design Lane Loaded: $0.06 + \left(\frac{S}{9.6}\right)^{1.0}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12Lt_S^3}\right)^{0.1}$ Two or More Design Lanes Loaded: $0.075 + \left(\frac{S}{11}\right)^{1.0}\left(\frac{S}{L}\right)^{0.08}\left(\frac{K_g}{12Lt_S^3}\right)^{0.1}$	$3.5' \leq S \leq 16.0'$ $4\ 1/2'' \leq t_s \leq 12''$ $20' \leq L \leq 500'$ If $L > 240'$, use $L = 240'$ $10{,}000\ in^4 \leq K_g \leq 7{,}600{,}000\ in^4$ $N_b \geq 4$
		Use lesser of the above or Lever Rule	$N_b = 3$
Cast-in-Place Concrete Multicell Box	d	One Design Lane Loaded: $\left(1.75+\frac{S}{3.6}\right)\left(\frac{1}{L}\right)^{0.35}\left(\frac{1}{N_C}\right)^{0.45} \geq \frac{S}{21}$ Two or More Design Lanes Loaded: $\left(\frac{13}{N_C}\right)^{0.3}\left(\frac{S}{5.8}\right)\left(\frac{1}{L}\right)^{0.25}$	$7.0' \leq S \leq 13.0'$ $60' \leq L \leq 500'$ If $L > 240'$, use $L = 240'$ $N_c \geq 3$ If $N_c > 8$ use $N_c = 8$ For two or more design lanes loaded If $L > 1400/N_c$, use $L = 1400/N_c$
Concrete Deck on Concrete Spread Box Beams	b, c	One Design Lane Loaded: $\left(\frac{S}{3.0}\right)^{0.35}\left(\frac{Sd}{12L^2}\right)^{0.25}$ Two or More Design Lanes Loaded: $\left(\frac{S}{6.3}\right)^{0.6}\left(\frac{Sd}{12L^2}\right)^{0.125}$	$6.0' \leq S \leq 18.0'$ $20' \leq L \leq 500'$ If $L > 140'$, use $L = 140'$ $17'' \leq d \leq 66''$ $N_b \geq 3$
		Use Lever Rule	$S \geq 18.0'$

Table 4.6.2.2.2b-1 – Distribution of Live Loads Per Lane for Moment in Interior Beams (Continued)

U.S. Customary Units			
Type of Beams	**Applicable Cross-Section from Table A4.6.2.2.1-1**	**Distribution Factors**	**Range of Applicability**
Concrete Beams used in Multi-Beam Decks	f	One Design Lane Loaded: $k\left(\frac{b}{33.3L}\right)^{0.5}\left(\frac{I}{J}\right)^{0.25}$ where: $k = 2.5(N_b)^{-0.2} \geq 1.5$	$35'' \leq b \leq 60''$ $20' \leq L \leq 500'$ If $L > 120'$, use $L = 120'$ $5 \leq N_b \leq 20$
	g if sufficiently connected to act as a unit	Two or More Design Lanes Loaded: $k\left(\frac{b}{305}\right)^{0.6}\left(\frac{b}{12L}\right)^{0.2}\left(\frac{I}{J}\right)^{0.06}\left(\frac{2.4}{L^{0.15}}\right)^{\frac{N_b}{15}}$	$35'' \leq b \leq 60''$ $20' \leq L \leq 500'$ $5 \leq N_b \leq 20$ If $N_b > 12$, use $N_b = 12$
	h g, i, j if connected only enough to prevent relative vertical displacement at the interface	Regardless of Number of Loaded Lanes: S/D where: C=K(W/L) $D = 11.5 - N_L + 1.4N_L(1 - 0.2C)^2$ When $C \leq 5$ $D = 11.5 - N_L$ when $C > 5$ $K = \sqrt{\frac{(1+\mu)l}{J}}$ for preliminary design, the following values of K may be used: <u>Beam Type</u> <u>K</u> Non-voided rectangular beams 0.7 Rectangular beams with circular voids 0.8 Box section beams 1.0 Channel beams 2.2 T-beam 2.0 Double T-beam 2.0	
Steel Grids on Steel Beams	a	One Design Lane Loaded: S/7.5' If $t_g < 4''$ S/10.0' If $t_g \geq 4''$	$S \leq 6.0'$
		Two or More Design Lanes Loaded: S/8.0' If $t_g < 4''$ S/10.0' If $t_g \geq 4''$	$S \leq 6.0'$
Concrete deck on Multiple Steel Box Girders	b, c	Regardless of Number of Loaded Lanes: $0.05 + 0.85\frac{N_L}{N_b} + \frac{0.425}{N_L}$	$0.5 \leq \frac{N_L}{N_b} \leq 1.5$

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4.6.2.2.2d Exterior Beams

4.6.2.2.2e Skewed Bridges

Delete A4.6.2.2.2e.

4.6.2.2.3 Distribution Factor Method for Shear

4.6.2.2.3a Interior Beams

The following shall replace the second sentence of Paragraph 1 in A4.6.2.2.3a.

For interior beams not listed in Table 1, lateral distribution of axle load shall be determined by lever rule.

The following shall replace Table A4.6.2.2.3a-1.

COMMENTARY

C4.6.2.2.2d

The following shall supplement AC4.6.2.2.2d.

The value of d_e is to be computed using the midpoint of the exterior web.

C4.6.2.2.2e

The following shall replace AC4.6.2.2.2e.

PennDOT has decided not to take advantage of the reduction in load distribution factors for moment in longitudinal beams on skewed supports.

Table 4.6.2.2.3a-1 – Distribution of Live Loads Per Lane for Shear in Interior Beams

Metric Units				
Type of Superstructure	**Applicable Cross-Section from Table A4.6.2.2.1-1**	**One Design Lane Loaded**	**Two or More Design Lanes Loaded**	**Range of Applicability**
Wood Deck on Wood or Steel Beams	See Table A4.6.2.2.2a-1			
Concrete Deck on Wood Beams	l	Lever Rule	Lever Rule	N/A
Concrete Deck, Filled Grid, or Partially Filled Grid on Steel or Concrete Beams; Concrete T-Beams, T- and Double T-Sections	a, e, k I, j if sufficiently connected to act as a unit	$0.36+\frac{S}{7600}$	$0.2+\frac{S}{3600}-\left(\frac{S}{10\,700}\right)^{2}$	$1100 \le S \le 4900$ $6000 \le L \le 152\,000$ If $L > 73\,000$, use $L = 73\,000$ $110 \le t_s \le 300$ $N_b \ge 4$
		Lever Rule	Lever Rule	$N_b = 3$
Cast-in-Place Concrete Multicell Box	d	$\left(\frac{S}{2900}\right)^{0.6}\left(\frac{d}{L}\right)^{0.1}$	$\left(\frac{S}{2200}\right)^{0.9}\left(\frac{d}{L}\right)^{0.1}$	$1800 \le S \le 4000$ $6000 \le L \le 152\,000$ $890 \le d \le 2800$ $N_c \ge 3$
Concrete Deck on Concrete Spread Box Beams	b, c	$\left(\frac{S}{3050}\right)^{0.6}\left(\frac{d}{L}\right)^{0.1}$	$\left(\frac{S}{2250}\right)^{0.8}\left(\frac{d}{L}\right)^{0.1}$	$1800 \le S \le 5500$ $6000 \le L \le 152\,000$ If $L > 43\,000$, use $L = 43\,000$ $430 \le d \le 1700$ $N_b \ge 3$
		Lever Rule	Lever Rule	$S > 5500$
Concrete Box Beams Used in Multi-Beam Decks	f, g	$0.70\left(\frac{b}{L}\right)^{0.15}\left(\frac{I}{J}\right)^{0.05}$	$\left(\frac{b}{4000}\right)^{0.4}\left(\frac{b}{L}\right)^{0.1}\left(\frac{l}{J}\right)^{0.05}$	$900 \le b \le 1500$ $6000 \le L \le 152\,000$ If $L > 37\,000$, use $L = 37\,000$ $5 \le N_b \le 20$ $6.8\times10^{9} \le J \le 3.75\times10^{11}$ $2.1\times10^{9} \le I \le 3.75\times10^{11}$
Concrete Beams Other Than Box Beams Used in Multi-Beam Decks	H	Lever Rule	Lever Rule	N/A
	i, j if connected only enough to prevent relative vertical displacement at the interface			
Steel Grid Deck on Steel Beams	a	Lever Rule	Lever Rule	N/A
Concrete Deck on Multiple Steel Box Beams	b, c	As specified in Table D4.6.2.2.2b-1		

Table 4.6.2.2.3a-1 – Distribution of Live Loads Per Lane for Shear in Interior Beams (Continued)

U.S. Customary Units				
Type of Superstructure	**Applicable Cross-Section from Table A4.6.2.2.1-1**	**One Design Lane Loaded**	**Two or More Design Lanes Loaded**	**Range of Applicability**
Wood Deck on Wood or Steel Beams	See Table A4.6.2.2.2a-1			
Concrete Deck on Wood Beams	l	Lever Rule	Lever Rule	N/A
Concrete Deck, Filled Grid, or Partially Filled Grid on Steel or Concrete Beams; Concrete T-Beams, T- and Double T-Sections	a, e, k i, j If sufficiently connected to act as a unit	$0.36+\frac{S}{25}$	$0.2+\frac{S}{12}-\left(\frac{S}{35}\right)^2$	3.5' ≤ S ≤ 16.0' 20' ≤ L ≤ 500' If L > 240', use L = 240' 4 1/2" ≤ t_s ≤ 12" N_b ≥ 4
Cast-in-Place Concrete Multicell Box	d	$\left(\frac{S}{9.5}\right)^{0.6}\left(\frac{d}{12L}\right)^{0.1}$	$\left(\frac{S}{7.3}\right)^{0.9}\left(\frac{d}{12L}\right)^{0.1}$	6.0' ≤ S ≤ 13.0' 20' ≤ L ≤ 500' 35" ≤ d ≤ 110" N_c ≥ 3
Concrete Deck on Concrete Spread Box Beams	b, c	$\left(\frac{S}{10}\right)^{0.6}\left(\frac{d}{12L}\right)^{0.1}$	$\left(\frac{S}{7.4}\right)^{0.8}\left(\frac{d}{12L}\right)^{0.1}$	6.0' ≤ S ≤ 18.0' 20' ≤ L ≤ 500' If L > 140', use L = 140' 17" ≤ d ≤ 66" N_b ≥ 3
		Lever Rule	Lever Rule	S > 18.0’
Concrete Box Beams Used in Multi-Beam Decks	f, g	$\left(\frac{b}{130L}\right)^{0.15}\left(\frac{I}{J}\right)^{0.05}$	$\left(\frac{b}{156}\right)^{0.4}\left(\frac{b}{12L}\right)^{0.1}\left(\frac{I}{J}\right)^{0.05}$	35" ≤ b ≤ 60" 20' ≤ L ≤ 500' If L > 120', use L = 120' 5 ≤ N_b ≤ 20 16,500 in^4 ≤ J ≤ 900,000 in^4 5,000 in^4 ≤ I ≤ 900,000 in^4
Concrete Beams Other Than Box Beams Used in Multi-Beam Decks	h i, j if connected only enough to prevent relative vertical displacement at the interface	Lever Rule	Lever Rule	N/A
Steel Grid Deck on Steel Beams	a	Lever Rule	Lever Rule	N/A
Concrete Deck on Multiple Steel Box Beams	b, c	As specified in Table D4.6.2.2.2b-1		

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4.6.2.2.3c Skewed Bridges

The following shall replace the second paragraph of A4.6.2.2.3c.

In determining end shear for beams other than prestressed concrete adjacent box beams, the shear skew adjustment factor shall be applied to the shear distribution factor of exterior beams at the obtuse corners for a distance of one-half the span length (see Figure 1).

In determining end shear for prestressed concrete adjacent box beams, the shear skew adjustment factor shall be applied to the shear distribution factors for all the beams which are on a skew (see Figure 1).

COMMENTARY

C4.6.2.2.3c.

The following shall supplement AC4.6.2.2.3c.

When structures have multiple skew angles, the smallest applicable skew angle associated with that girder should be used in calculation of shear correction factors.

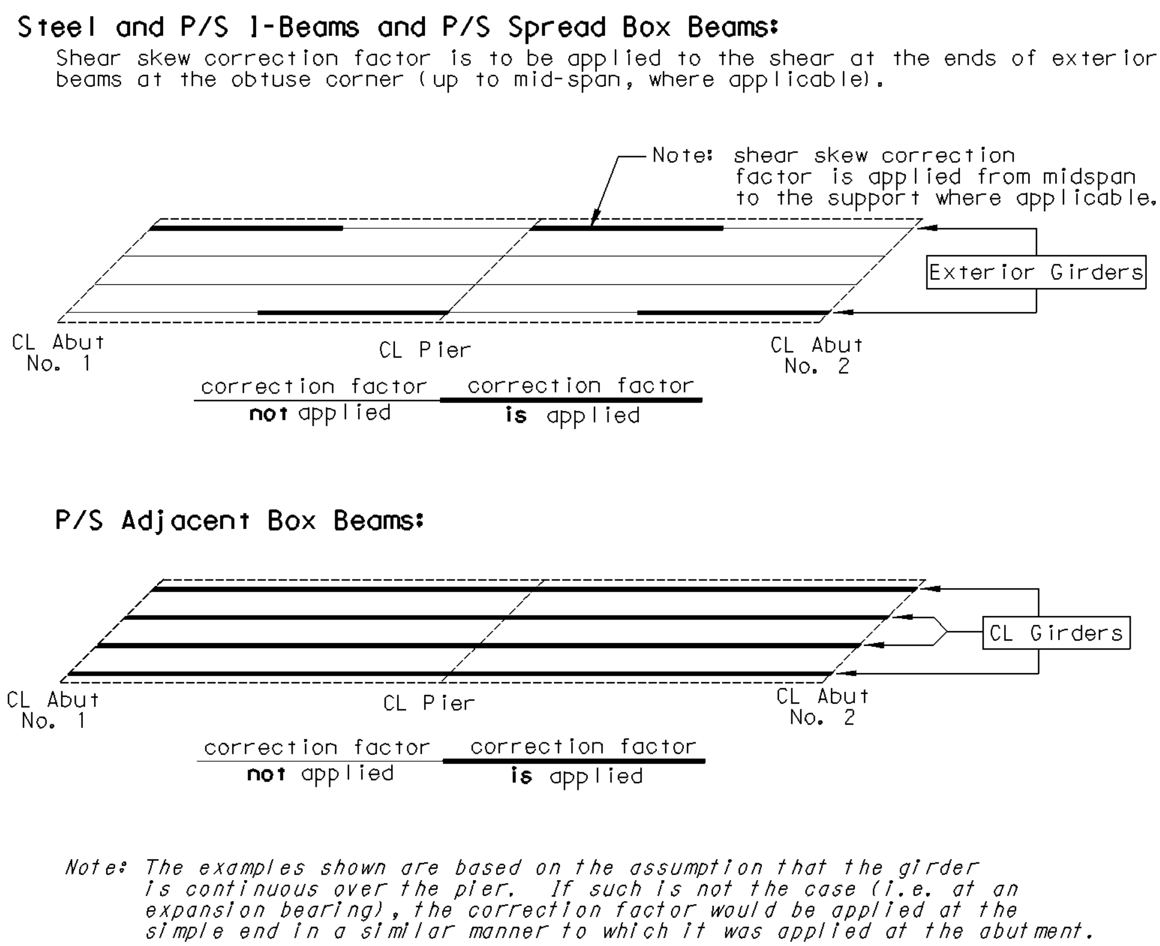

Figure 4.6.2.2.3c-1 - Application of Shear Correction Factor for End Shear

In determining end reactions of continuous beams (such as reactions at abutments) for beams other than prestressed concrete adjacent box beams, the shear skew adjustment factor shall be applied to the shear distribution factor of exterior beams at the obtuse corners (see Figure 2).

In determining end reactions of continuous beams (such as reactions at abutments) for prestressed concrete adjacent box beams, the shear skew adjustment factor shall be applied to the shear distribution factors for all the beams which are on a skew (see Figure 2).

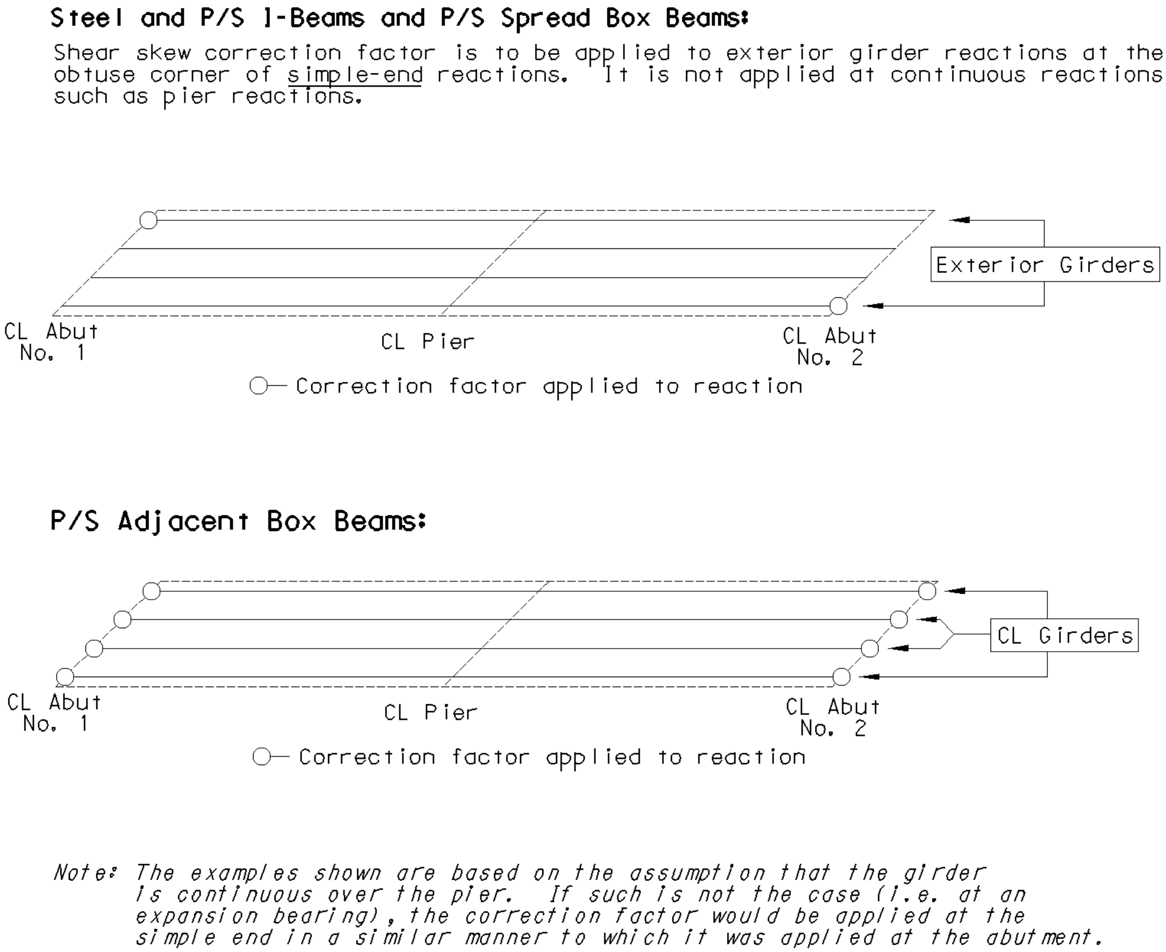

Figure 4.6.2.2.3c-2 - Application of Shear Skew Correction Factor for End Reactions

The following shall replace Table A4.6.2.2.3c-1.

Table 4.6.2.2.3c-1 - Correction Factors for Load Distribution Factors for Support Shear of the Obtuse Corner

Metric Units			
Type of Superstructure	**Applicable Cross-Section from Table 4.6.2.2.1-1**	**Correction Factor**	**Range of Applicability**
Concrete Deck, Filled Grid, or Partially Filled Grid on Steel or Concrete Beams; Concrete T-Beams, T- and Double T-Section	a, e, k and also i, j if sufficiently connected to act as a unit	$1.0+0.20\left(\frac{L\,t_s^{\,3}}{K_g}\right)^{0.3}\tan(90-\theta)$	30º ≤ θ ≤ 90º 1100 ≤ S ≤ 4900 6000 ≤ L ≤ 152 000 If L > 73 000, use L = 73 000 N_b ≥ 4
Cast-in-Place Concrete Multicell Box	d	$1.0+\left[0.25+\frac{L}{70d}\right]\tan(90-\theta)$	30º ≤ θ ≤ 90º 1800 < S ≤ 4000 6000 ≤ L ≤ 152 000 If L > 73 000, use L = 73 000 900 ≤ d ≤ 2700 N_b ≥ 3
Concrete Deck on Spread Concrete Box Beams	b, c	$1.0+\frac{\sqrt{Ld}}{6S}\tan(90-\theta)$	30º ≤ θ ≤ 90º 1800 ≤ S ≤ 3500 If S > 3500, use S = 3500 6000 ≤ L ≤ 152 000 If L > 43 000, use L = 43 000 430 ≤ d ≤ 1700 N_b ≥ 3
Concrete Box Beams Used in Multibeam Decks	f, g	$1.0+\frac{L\sqrt{\tan\theta}}{90d}$	30º ≤ θ ≤ 90º 6000 ≤ L ≤ 152 000 If L > 37 000, use L = 37 000 305 ≤ d ≤ 1700 900 ≤ b ≤ 1500 5 ≤ N_b ≤ 20

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Table 4.6.2.2.3c-1 - Correction Factors for Load Distribution Factors for Support Shear of the Obtuse Corner (Continued)

U.S. Customary Units			
Type of Superstructure	**Applicable Cross-Section from Table 4.6.2.2.1-1**	**Correction Factor**	**Range of Applicability**
Concrete Deck, Filled Grid, or Partially Filled Grid on Steel or Concrete Beams; Concrete T-Beams, T- and Double T-Section	a, e, k and also i, j if sufficiently connected to act as a unit	$1.0+0.20\left(\frac{12\,L\,t_s^{\,3}}{K_g}\right)^{0.3}\tan(90-\theta)$	30º ≤θ ≤90º 3.5' ≤S ≤16.0' 20' ≤ L ≤ 500' If L > 240', use L = 240' $N_b \geq 4$
Cast-in-Place Concrete Multicell Box	d	$1.0+\left[0.25+\frac{12\,L}{70\,d}\right]\tan(90-\theta)$	30º ≤ θ ≤ 90º 6.0' < S ≤ 13.0' 20' ≤ L ≤ 500' If L > 240', use L = 240' 35" ≤ d ≤ 110" $N_b \geq 3$
Concrete Deck on Spread Concrete Box Beams	b, c	$1.0+\frac{\sqrt{\frac{Ld}{12.0}}}{6S}\tan(90-\theta)$	30º ≤ θ ≤ 90º 6.0' ≤ S ≤11.5' If S > 11.5', use S = 11.5' 20' ≤ L ≤ 500' If L > 140', use L = 140' 17" ≤ d ≤ 66" $N_b \geq 3$
Concrete Box Beams Used in Multibeam Decks	f, g	$1.0+\frac{12\,L}{90d}\sqrt{\tan(90-\theta)}$	30º ≤ θ ≤ 90º 20' ≤ L ≤ 500' If L > 120', use L = 120' 17" ≤ d ≤ 66" 35" ≤ b ≤ 60" $5 \leq N_b \leq 20$

4.6.2.3 EQUIVALENT STRIP WIDTHS FOR SLAB-TYPE BRIDGES

The following shall replace the first paragraph of A4.6.2.3.

This article shall be applied to the types of cross-sections which are shown schematically in Table 1. For the purpose of this article, cast-in-place voided slab bridges may be considered as slab bridges.

4.6.2.5 EFFECTIVE LENGTH FACTOR, K

The following shall supplement A4.6.2.5.

For Extreme Event I, Seismic Loading, the effective length factor, K, in the plane of bending may be assumed to be unity in the calculation of λ.

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4.6.2.6 EFFECTIVE FLANGE WIDTH

4.6.2.6.1 General

The following shall replace the last two sentences of the first paragraph.

For the calculation of live load deflections, where required, the provisions of D2.5.2.6.2 shall apply.

COMMENTARY

C4.6.2.6.1

The following shall supplement AC4.6.2.6.1.

For typical continuous span bridges, the effective span length may be taken as the distance between points of dead load contraflexure, and can be approximated as shown in Table 1 below:

Table C4.6.2.6.1-1 - Approximate Effective Span Lengths for Computing Effective Slab Width of Continuous Span Bridges

Positive Flexure Regions		Negative Flexure Regions		
Exterior Span	Interior Span	Support Not Adjacent to Exterior Span	For Two-Span Girder	Support Adjacent to Exterior Span
0.7 L_E	0.5 L_I	0.25 L_{I1} + 0.25 L_{I2}	0.3 L_{E1} + 0.3 L_{E2}	0.30 L_E + 0.25 L_I

Where L_I and L_E represent interior and exterior span lengths, respectively. This approximation is only applicable for the purpose of computing effective span lengths to be used in the determination of the effective slab width. The approximation is based on assumed points of dead load contraflexure for balanced span lengths. In cases where this simplification is not valid, i.e., the spans are not balanced, a more rigorous estimation of the effective span length should be made by determining the actual points of dead load contraflexure. Because of the effective slab width affects the section properties to be used in the girder dead load analysis, an iterative procedure may be required to determine the appropriate values. In such cases, a 5% tolerance is allowed on the effective slab width, b_s.

Where the cross-section of the web or top flange changes over a flexural region, the minimum top flange width or web thickness within that region may be used in the calculation of the effective slab width.

Following the determination of the actual points of dead load contraflexure by the continuous beam analysis, the designer should compute the effective slab width in accordance with A4.6.2.6.1 based on the true effective span length. If the effective span length controls the calculation of b_s, the calculated value should be compared to the previously computed value to determine whether it is within the allowed tolerance. The effective span length will typically control the calculation of b_s only for short-spans having large girder spacings.

For steel girders, an acceptable approximation of the permanent load inflection points is the non-composite dead load contraflexure points based on the non-composite

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section properties.

For prestressed concrete girders, an acceptable approximation of the permanent load inflection points is the composite dead load contraflexure points based on the composite (3n) section properties.

4.6.2.10P GIRDER - FLOORBEAM - STRINGER BRIDGES

4.6.2.10.1P Girder Live Load Distribution Factors

Girder live load distribution factors shall be calculated on the assumption that deck acts as a simple span between the girders or deck acts as beam with overhang for exterior girders (i.e., this assumes the stringers and floorbeams are not present).

4.6.2.10.2P Stringer Live Load Distribution Factors

Stringer live load distribution factors shall be based on D4.6.2.2 and A4.6.2.2.

4.6.2.10.3P Floorbeam Live Load Distribution Factors

4.6.2.10.3aP Floorbeams with the Top Flange not Directly Supporting the Deck

For floorbeams with the top flange not directly supporting the deck, the longitudinal reaction of design live load is determined and then these loads are moved transversely along the floorbeam to produce the maximum force effect assuming the stringers are not present.

4.6.2.10.3bP Floorbeams with the Top Flange Directly Supporting the Deck

For floorbeams with the top flange directly supporting the deck, the floorbeam distribution shall be calculated as given in A4.6.2.2.

4.6.2.11P DISTRIBUTION OF LOAD FROM THE SUPERSTRUCTURE TO THE SUBSTRUCTURE

In order to determine girder reactions (which are used as loads for the substructure design), the deck is assumed to act as a simple beam between interior girders and as a cantilever beam for the exterior girder and the first interior girder.

In the calculation of live load girder reactions, the design vehicle shall be assumed to spread uniformly over 3000 mm {10 ft.} (i.e., not two separate wheel loads).

C4.6.2.11P

For abutments designed on a per meter basis, an acceptable alternate method of distribution would be to divide the sum total of all the loads applied to the abutment (for each limit state) by the abutment front face width. When using the above approach, the live load contribution may be obtained by determining the live load effect for one lane of loading and multiplying that effect by the number of design lanes.

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4.6.2.12P EQUIVALENT STRIP WIDTHS FOR BOX CULVERTS

4.6.2.12.1P General

This article shall be applied to box culverts with depths of fill less than 600 mm {2.0 ft}.

4.6.2.12.2P Case 1: Traffic Travels Parallel to Span

When traffic travels primarily parallel to the span, culverts shall be analyzed for a single loaded lane with the single lane multiple presence factor.

The axle load shall be distributed to the top slab for determining moment, thrust, and shear as follows:

- Perpendicular to the span:

Metric Units:

$$E = 2440 + 0.12\,S \qquad (4.6.2.12.2P\text{-}1)$$

U.S. Customary Units:

$$E = 96 + 1.44\,S$$

- Parallel to the span:

$$E_{span} = LT + LLDF\,(H) \qquad (4.6.2.12.2P\text{-}2)$$

where:

E = equivalent distribution width perpendicular to span (mm){in.}

S = clear span (mm) {ft.}

Espan = equivalent distribution length parallel to span (mm) {in.}

LT = length of tire contact area parallel to span, as specified in D3.6.1.2.5 (mm) {in.}

LLDF = factor for distribution of live load with depth of fill, 1.00, as specified in D3.6.1.2.6

H = depth of fill from top of culvert to top of pavement (mm) {in.}

4.6.2.12.3P Case 2: Traffic Travels Perpendicular to Span

When traffic travels perpendicular to the span, distribute live load to the top slab using the equations in

COMMENTARY

C4.6.2.12.1P

Design for depths of fill of 600 mm {2.0 ft}or greater are covered in D3.6.1.2.6

C4.6.2.12.2P

Culverts are designed under the provisions of Section D12. Box culverts are normally analyzed as two-dimensional frames. Equivalent strip widths are used to simplify the analysis of the three-dimensional response to live loads. Equations 1 and 2 are based on research (McGrath et al., 2004) that investigated the forces in box culverts with spans up to 7300 mm {24 ft}.

The distribution widths are based on distribution of shear forces. Distributions widths for positive and negative moments are wider; however, using the narrower width in combination with a single lane multiple presence factor provides designs adequate for multiple loaded lanes for all force effects.

Although past practice has been to ignore the distribution of live load with depth of fill, consideration of this effect, as presented in Equation 2, produces a more accurate model of the changes in design forces with increasing depth of fill. The increased load length parallel to the span, as allowed by Equation 2, may be conservatively neglected in design. For the BXLRFD computer program, Equation 2 has been ignored.

C4.6.2.12.3P

Culverts with traffic traveling perpendicular to the span can have two or more trucks on the same design strip at the

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A4.6.2.1 for concrete decks with primary strips perpendicular to the direction of traffic.

4.6.2.12.4P Precast Box Culverts

For precast box culverts, the distribution width computed with Equation 4.6.2.12.2P-1 shall not exceed the length between two adjacent joints without a means of shear transfer across the joint. Additionally, if no means of shear transfer is provided, the section ends shall be designed as an edge beam in accordance with the provisions of A4.6.2.1.4b.

Shear transfer may be provided by pavement, soil backfill, or a physical connection between adjacent sections.

COMMENTARY

same time. This must be considered, with appropriate multiple presence factor, in analysis of the culvert structural response.

C4.6.2.12.4P

Precast box culverts manufactured in accordance with AASHTO Materials Specification M273 are often installed with joints that do not provide a means of direct shear transfer across the joints of adjacent sections under service load conditions. This practice is based on research (James, 1984, Frederick, et al., 1988) that showed small deflections and strains under wheel loads with no earth cover, due primarily to the fact that the sections were designed as cracked sections but do not crack under service loading. While there are no known service issues with installation of standard box sections without means of shear transfer across joints, analysis (McGrath et al., 2004) shows that stresses are substantially higher when a box culvert is subjected to a live load at a free edge than when loaded away from a free edge.

Most shallow cover box culvert applications have some fill or a pavement that likely provide sufficient shear transfer to distribute live load to adjacent box sections without shear keys to avoid higher stresses due to edge loading. States and design agencies that utilize grouted shear keys, pavement or systems whose function is to transfer shear across joints may use past performance of these connections and/or materials as a basis for providing adequate shear transfer. Otherwise, for applications with zero depth of cover, and no pavement, soil, or other means of shear transfer such as shear keys, designers should design the culvert section for the specified reduced distribution widths. The use of post-tensioning in accordance with the BC Standard Drawings in conjunction with a cast-in-place slab or bituminous pavement is considered sufficient to provide adequate shear transfer between adjacent culvert sections.

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4.6.3 Refined Methods of Analysis

4.6.3.1 GENERAL

The following shall replace the second paragraph of A4.6.3.1.

Barriers shall not be considered in the calculation of the structural stiffness nor structural resistance of a structure.

The following shall supplement A4.6.3.1.

When a refined method of analysis is performed for beam-slab bridges other than those bridges defined in D4.6.1.2.1 and D4.6.2.2.1 (which must use a refined method analysis), the beams must have capacity not less than if it were designed using the approximate method of analysis given in A4.6.2.2.1 and D4.6.2.2.1.

4.6.3.2 DECKS

4.6.3.2.3 Orthotropic Plate Model

The following shall replace A4.6.3.2.3.

In orthotropic plate modeling, the flexural rigidity of the elements may be uniformly distributed along the cross-section of the deck. Where the torsional stiffness of the deck is not contributed solely by a solid plate of uniform thickness, the torsional rigidity should be established by physical testing, three-dimensional analysis, or generally accepted and verified approximations, and shall be approved by the Chief Bridge Engineer.

4.6.3.3 BEAM-SLAB BRIDGES

The following shall supplement A4.6.3.3.

When a refined method of analysis is performed, the live load force effects carried by each girder may be computed by the techniques listed below in order of decreasing sophistication:

- Three-dimensional finite element method, and
- Two-dimensional grillage analogy

With a two-dimensional grillage analogy or a three-dimensional finite element method, the rating of a bridge is not as straightforward as when the A4.6.2.2 or D4.6.2.2 is used. A refined method of analysis more closely represents the fact that the distribution of live loads on a bridge is not described by a constant distribution factor. When a refined method of analysis is used in the design, a table of live load distribution factors (based on design truck of the PHL-93) for girder maximum positive and negative moments and shear in each span shall be provided on the contract plans to aid in future ratings of the bridge. The live load distribution factor shall be in the form of a ratio of the force effect from the refined method of analysis caused by the design truck of the PHL-93 in that lane, divided by the force effect obtained

COMMENTARY

C4.6.3.1

Delete the second paragraph of AC4.6.3.1.

C4.6.3.3

The following shall supplement the bulleted list of AC4.6.3.3.

- If the program being used allows only for nodal loads, concentrated loads shall be distributed to adjacent nodes by simple statics. If the spacing of girders significantly exceeds 2400 mm {8 ft.}, it is preferable to place intermediate nodes on the transverse members to model load distribution more accurately.
- The framing of members at bearings is very important. Nodes at bearings should not be artificially restrained through the enforcement of fixed support conditions for other than vertical transitional support at all bearings, and longitudinal and transverse transitional support where the detailing dictates. This provision is critical to proper modeling of bridges with significant skew.
- The warping stiffness of girders is difficult to model, unless a warping (not St. Venant torsional) degree of freedom is explicitly provided by the computer program. As an approximation, the dead load moment diagram and live load moment envelope may be converted to a uniform flange bending (i.e., bimoment)

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from application of one design truck of the PHL-93 acting on a single, isolated girder. The commentary provides an example table of live load distribution factors.

Two other areas which must be considered for future ratings of bridges designed with refined method analysis are cross-frame forces and uplift at reactions. Therefore, at Type, Size and Location submission, the designer must include (for approval by the Chief Bridge Engineer) a proposed simplified method for rating a special vehicle for the controlling cross-frame member and uplift reaction condition.

When a grillage analogy or finite element model is implemented, the preferred procedure is the generation and subsequent loading of an influence surface to produce maximum effect. The influence surface shall be loaded to maximize positive and negative design values (moments, shear, diaphragm forces, etc.) for all critical points along the bridge. This process is analogous to the classical use of influence lines. The provisions of A3.6.1.1.2 shall apply to the loading method described above. It is also acceptable, though less rigorous and economical, to apply distribution factors from D4.6.2.2.2 to each girder in a grid system and allow the grid or finite element analysis to more accurately determine the load distribution to interior and exterior beams, and to account for the influences of skew and curvature, if present. The latter procedure is less preferred than the former procedure since uneconomical designs may result.

It is expressly *not* acceptable to load a grid or finite element model with the number of design lanes of live load given in A3.6.1.1.1, without simultaneously positioning the loads longitudinally and transversely to maximize and minimize each design moment, shear, diaphragm load (curved and/or skewed bridges), and reaction. Except in the most obvious cases (e.g., bridges that are essentially right and straight), the transverse and longitudinal position of live load which produces the critical design condition should be determined by the use of influence surfaces.

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loading by the approximation below:

$$W = \frac{\sum M}{Rd}$$

where:

ΣM = total primary moment (N·mm) per mm {k in/ft}

R = radius (mm) {ft.}

d = girder depth (mm) {in.}

This load may be applied to the flange and assumed to be reacted by the diaphragms and slab, as appropriate, analogous to a continuous beam on simple supports. If a warping degree of freedom or some approximate method of calculating warping stress is not included in the program, it will be necessary to combine the loads calculated according to the above method with the loads produced from diaphragms.

- If grid or finite element methods are used to determine dead load forces, then the structure, including diaphragms, must be cambered in accordance with the computer analysis. If some cambering other than that corresponding to the opposite of the natural dead load deflection is anticipated (e.g., cambering for more efficient dead load distribution), it must be reflected in the computer modeling.

The following shall supplement AC4.6.3.3.

Special care shall be exercised in modeling, analysis and interpretation of the results of two- and three-dimensional procedures. AC4.6.3.3 and this commentary provide guidelines. The designer is responsible for the correct application of advanced analysis methods and is advised that various commercial and generic computer programs can report significantly different results for various combinations of skew and/or curvature.

Figure C1 provides an example of a table for live load distribution factors which shall be included on the contract drawings. Since the figure provides only example tables, the table used on the contract drawings shall be developed for the specific structure in question.

When a refined method of analysis is used to design a bridge, the table of live load distribution factors shall be the basis for future ratings of the bridge. In order to provide a realistic rating in the absence of such a table, the rating of the bridge would require modeling the bridge by means of an analysis method similar to the one used in the original design. The live load distribution factors given in D4.6.2.2.2 should not be used for the rating, since they would generally provide an unduly conservative rating or, in a few cases, an unconservative rating.

Girder 1-Moment Distribution Factors*						
	Positive Moment Distribution Factors			Negative Moment Distribution Factors		
Location	1 Lane	2 Lanes	N Lanes	1 Lane	2 Lanes	N Lanes
Span 1				–	–	–
Interior Support 1	–	–	–			
Span 2				–	–	–
Interior Support 2	–	–	–			
Span N				–	–	–
Interior Support N	–	–	–			

Girder 1 – Shear Distribution Factors*			
Location	1 Lane	2 Lanes	N Lanes

*These tables shall be repeated for each girder

Figure C4.6.3.3-1 - Example of Live Load Distribution Factor Tables

A live load distribution factor developed for the design truck may be used for permit and rating vehicles. When the bridge or the vehicle or both are unusual, an analysis should be made to justify the use of an design truck distribution factor for other types of vehicles.

One possible way to develop a simplified method for rating for cross-frame members would be to develop a table of ratings for PHL-93, P-82, ML-80, HS20 and H20 for 1 up to N lanes loaded at the time of original design. When a future rating of a special vehicle is required, the engineer could develop an approximate rating by interpolating among these vehicles used to develop the table.

A similar approach could be used for the uplift reaction condition. A table of uplift reactions for PHL-93, P-82, ML-80, HS20 and H20 for 1 up to N lanes loaded could be developed at the time of original design. When a future rating of a special vehicle is required, the engineer could develop an approximate uplift reaction by interpolating among these vehicles used to develop the table. Next, this uplift reaction could be in computing a rating.

Some programs use an influence surface loading technique that evaluates numbers of lanes of loading with

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appropriate multi-presence reduction factors. Other procedures apply the distribution factors given in D4.6.2.2.2 to each girder in a grid of girders and diaphragms and allow a grid analysis to evaluate the effects of skew, curvature, and the tendency for load to gravitate toward the exterior stringers, but they do not necessarily take advantage of the fact that the true distribution factor is often less than those given in D4.6.2.2.2 and typically is not constant for all effects (moment, shear, deflection, etc.) or locations along the bridge.

In 1986, the Department conducted a parametric study of steel girder bridges to compare the results obtained from these programs for a variety of combinations of skew and curvature. This parametric study could not cover all cases that might be encountered by designers, but did reveal some trends and comparative data which form the basis of a report titled "Review of Computer Programs for the Analysis of Girder Bridges", January 1989. Since 1986, many of the programs used in this study have been changed and/or updated. Therefore, some of the report's comparisons and observations may have lost their relevancy. For a copy of this report, contact the Chief Bridge Engineer's office.

The modeling of diaphragms and boundary conditions at supports and bearings is vital to obtaining the proper results from some of these sophisticated programs. The burden of correctly handling these factors rests squarely upon the shoulders of the designer. Consider the following example, which shows how a very small modeling error produces very erroneous results.

The framing plan shown in Figure C2 represents a real bridge that was designed using a grid-type approach. The designer had a good model for this structure, except that the rotational degree of freedom corresponding to the global x-axis at all of the bearings was fixed. This did not allow the diaphragms at the piers and abutments to respond correctly to the imposed loadings and deformations, and also had the effect, by virtue of vector resolution between global and local systems, of producing artificially stiff ends on the girders.

This incorrect boundary assumption altered both the dead load and live load forces. The effect of this on the dead load reactions obtained at the abutments and piers was dramatic. A modest uplift was reported at the near abutment dead load reaction for Girder 1, and a very substantial uplift was reported at the far abutment dead load reaction for Girder 5. This is shown in the table in Figure C2, as is a moment diagram for noncomposite dead loads which reflects the incorrect dead load reactions. Also shown are the correct dead load reactions which were determined when the structure was modeled using the generic STRESS computer program, with proper boundary conditions at the supports. In this case, a positive dead load reaction was found at all bearings, and a significantly different moment diagram for noncomposite dead load also resulted. Finally, the reaction table shows a set of incorrect dead load reactions obtained with STRESS when same error in

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boundary conditions was made to confirm the significance of proper modeling. The correct and incorrect dead load moment diagrams also are shown in Figure C2.

The differences in the degrees of freedom at the lines of support on this structure were also investigated, utilizing a relatively complete three-dimensional finite element analysis and the SAPIV computer program. The model is illustrated in Figure C3, which shows how the deck slab, girders and cross-frames were modeled in their proper relative positions in cross-section by means of rigid linking members. Also shown in this figure is a comparison of the dead load reactions obtained from STRESS and from SAPIV by applying all the noncomposite dead loads in a single loading. The agreement between these reactions is excellent.

In order to verify that the order of pouring the deck slab units would not contribute to an uplift situation, the pouring sequence was replicated in a three-dimensional SAPIV analysis. The results of the analysis of the three stages of the pouring sequence are shown in Figure C3, as well as the total accumulated load at the end of the pour. A comparison of the sequential loading with the application of a single loading of noncomposite dead load also showed relatively good agreement.

There are cases in which dead load uplift at the reactions due to skew and/or curvature is possible. The simple span bridge featured in the November 1, 1984, issue of Engineering News-Record was correctly analyzed and indicated high corner dead load uplift.

The important point shown in the example in Figures C2 and C3 is that seemingly small errors in modeling the structure can result in very substantial changes in the reactions, shears and moments. The designer must be aware of this potential when using two- or three-dimensional analysis techniques.

Sometimes modeling problems occur because user's manuals are not clear or because a "bug" exists, of which the author/vendor is unaware. Such a case is illustrated for a simply supported, partially curved and skewed bridge in Figure C4. Initially, this bridge was modeled with extra joints at locations other than diaphragms in an effort to improve live load determination. As a result, the number of points along each girder was not equal, but there was no indication of a potential problem in the descriptive literature. The resulting live load moment envelopes for the middle and two exterior girders, which are shown in Figure C4a, are obviously unusual in shape and possibly also in their order of maximum moment, i.e., No. 4, No. 5 and No. 3.

After these results were brought to the attention of the authors/vendors, it was decided that the live load processor was not responding properly to the unequal number of nodes per girder, that nodes should be essentially "radial," and that it was uncertain whether the nodes to which diaphragms were not connected were legitimate. The revised model, shown in Figure C4b, produced clearly better results, as is

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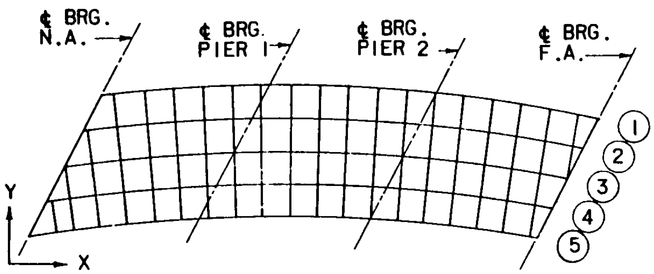
shown in the indicated live load moment envelopes. This example illustrates the need to review the output carefully and to communicate with the authors/vendors about the correct use of their programs.

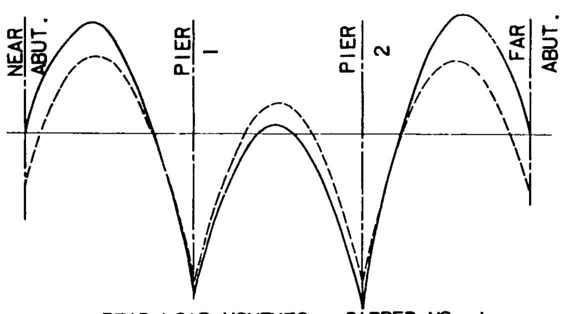
In summary, two- and three-dimensional methods can provide better understanding of load distribution in girder bridges, but must be used with great care.

METRIC UNITS

GIRDER	RADIUS (m)	SPAN LENGTHS 1	2	3
1	297.00	29.31	29.64	29.84
2	294.04	29.77	29.95	30.05
3	291.05	30.28	30.28	30.28
4	288.10	30.81	30.61	30.51
5	285.11	31.39	30.99	30.73



CONTROLS FOR CURVED GIRDER ANALYSIS



DEAD LOAD MOMENTS - GIRDER NO. 1

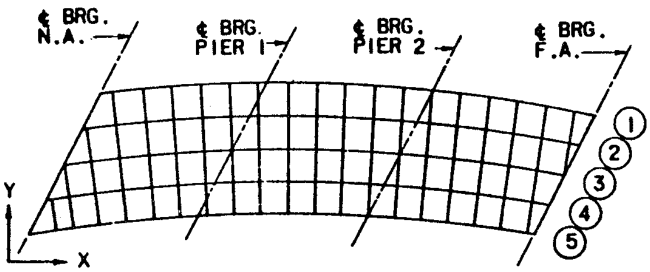
----- DESIGNER'S INCORRECT VALUES
——— 'STRESS' CORRECT VALUES

	GIRD No.	SUPPORT REACTIONS: 'STRESS' CORRECT SUPPORT CONDITION (VERT - kN)	DESIGNER'S INCORRECT REACTIONS (VERT - kN)	'STRESS' INCORRECT SUPPORT CONDITIONS (VERT - kN)	'STRESS' INCORRECT SUPPORT CONDITIONS (MOM - kN-m)
NEAR ABUT.	1	288.81	-80.38	-79.09	-630.72
	2	300.11	401.74	401.30	-1,008.94
	3	289.56	394.76	394.63	-1,260.77
	4	288.59	261.72	261.36	-1,087.05
	5	335.07	750.96	748.73	-702.19
PIER 1	1	923.98	849.12	850.64	0.00
	2	860.38	822.84	819.37	0.00
	3	828.26	795.04	795.88	0.00
	4	927.05	882.31	882.79	0.00
	5	747.93	732.90	730.32	0.00
PIER 2	1	839.52	801.66	802.11	0.00
	2	899.56	851.48	847.61	0.00
	3	886.26	818.57	819.94	0.00
	4	895.74	823.59	823.99	0.00
	5	820.08	761.54	759.05	0.00
FAR ABUT.	1	296.59	994.66	993.02	1,099.80
	2	286.27	209.37	209.68	1,669.49
	3	288.81	383.82	380.26	1,667.19
	4	296.33	524.64	524.64	1,678.44
	5	308.96	-363.09	-362.56	1,102.78
TOTAL		11,607.99	11,617.24	11,608.08	

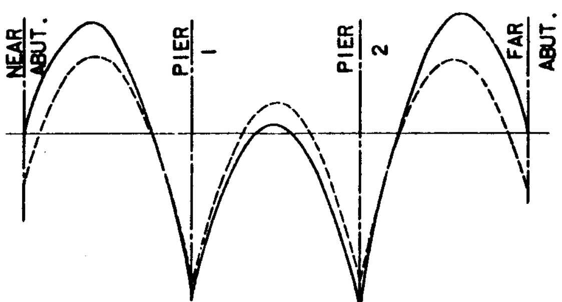
Figure C4.6.3.3-2 - Framing Plan, Comparative Dead Load Reactions and Moment Diagram Showing Effect of Proper and Improper Rotational Boundary Condition as Reflected in a Grid Analysis

U. S. CUSTOMARY UNITS

GIRDER	RADIUS (FT.)	SPAN LENGTHS 1	2	3
1	974.4	96'-2"	97'-3"	97'-11"
2	964.7	97'-8"	98'-3"	98'-7"
3	954.9	99'-4"	99'-4"	99'-4"
4	945.2	101'-1"	100'-5"	100'-1"
5	935.4	103'-0"	101'-8"	100'-10"



CONTROLS FOR CURVED GIRDER ANALYSIS



DEAD LOAD MOMENTS - GIRDER NO. 1

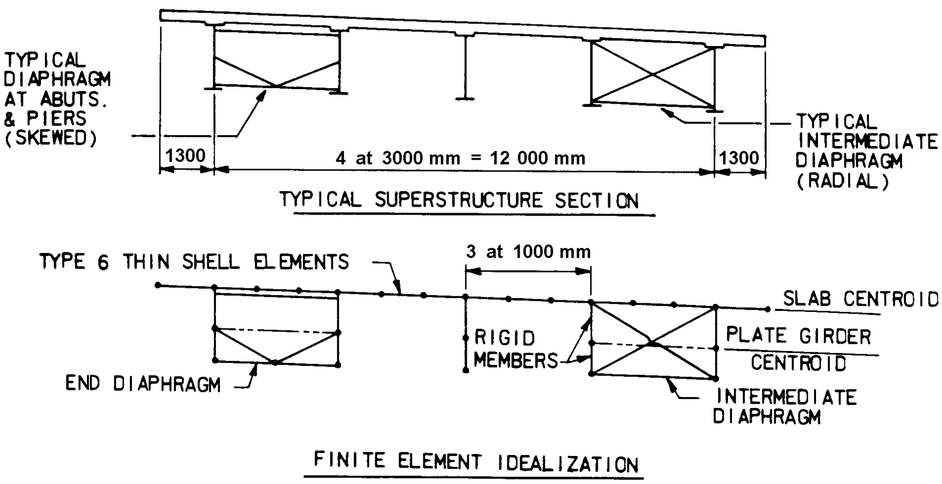
----- DESIGNER'S INCORRECT VALUES

——— 'STRESS' CORRECT VALUES

	GIRD. NO.	DEAD LOAD SUPPORT REACTIONS: 'STRESS' CORRECT SUPPORT CONDITIONS (VERT - K)	DESIGNER'S INCORRECT REACTIONS (VERT - K)	'STRESS' INCORRECT SUPPORT CONDITIONS (VERT - K)	'STRESS' INCORRECT SUPPORT CONDITIONS (MOM X K-FT)
NEAR ABUT.	1	64.93	-18.07	-17.78	-465.1
	2	67.47	90.32	90.22	-744.0
	3	65.10	88.75	88.72	-929.7
	4	64.88	58.84	58.76	-801.6
	5	75.33	168.83	168.33	-517.8
PIER 1	1	207.73	190.90	191.24	0
	2	193.43	184.99	184.21	0
	3	186.21	178.74	178.93	0
	4	208.42	198.36	198.47	0
	5	168.15	164.77	164.19	0
PIER 2	1	188.74	180.23	180.33	0
	2	202.24	191.43	190.56	0
	3	199.25	184.03	184.34	0
	4	201.38	185.16	185.25	0
	5	184.37	171.21	170.65	0
FAR ABUT.	1	66.68	223.62	223.25	811.0
	2	64.36	47.07	47.14	1231.1
	3	64.96	86.29	86.49	1229.4
	4	66.62	117.95	117.95	1237.7
	5	69.46	-81.63	-81.51	813.2
TOTAL		2609.71	2611.79	2609.73	

Figure C4.6.3.3-2 - Framing Plan, Comparative Dead Load Reactions and Moment Diagram Showing Effect of Proper and Improper Rotational Boundary Condition as Reflected in a Grid Analysis (Continued)

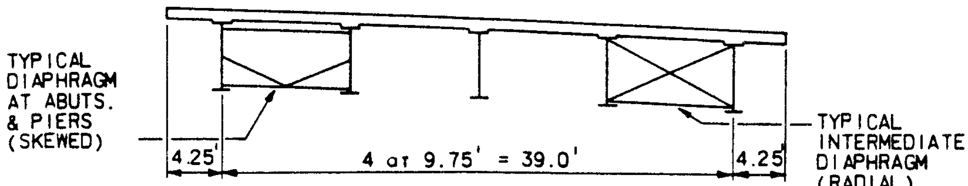
METRIC UNITS



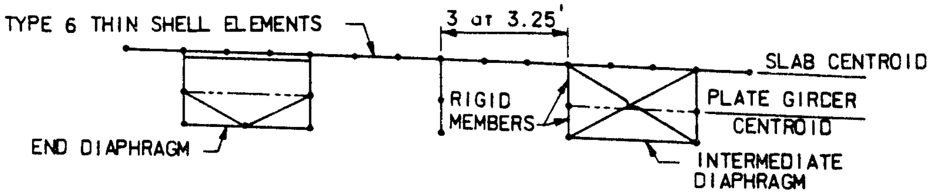
	BEAM NO	SAP NODE	SAP FULL DEAD LOAD ON NON-COMPOSITE SECTION (kN)	STRESS FULL DEAD LOAD ON NON-COMPOSITE SECTION (kN)	SAP SLAB POURS 1 & 2 (kN)	SAP SLAB POUR 3 (kN)	SAP SLAB POUR 4 (kN)	SAP TOTAL OF COLUMNS 3,4,& 5 (kN)
NEAR ABUT.	5	8	238.8	251.3	256.6	-18.7	-1.7	236.1
	4	29	271.7	259.7	291.7	-6.6	-0.9	286.0
	3	64	249.0	246.8	265.9	-20.0	-2.2	243.7
	2	109	255.3	245.0	275.3	-10.2	-1.3	263.7
	1	167	290.0	290.9	326.4	-57.8	-12.9	256.6
PIER 1	5	282	579.1	602.2	419.8	100.0	268.6	788.6
	4	324	729.9	752.1	298.4	148.1	276.6	723.2
	3	363	763.7	725.4	332.2	146.7	298.0	777.0
	2	402	773.9	815.7	392.3	125.4	272.2	789.9
	1	444	666.7	673.4	285.1	130.7	270.8	686.7
PIER 2	5	576	761.9	742.3	330.4	162.8	295.3	788.6
	4	617	765.9	792.1	374.0	117.8	270.4	762.3
	3	656	785.9	773.9	373.6	135.2	288.6	797.5
	2	695	753.9	785.9	333.1	125.8	275.3	734.3
	1	736	737.0	734.3	326.4	114.3	282.8	723.6
FAR ABUT.	5	901	256.2	258.8	282.8	-50.3	-10.6	221.9
	4	935	245.9	243.3	261.1	-8.4	0.8	253.5
	3	959	248.2	246.4	265.5	-18.2	0.4	247.7
	2	979	259.7	255.7	285.5	-16.0	0.4	269.9
	1	991	261.9	268.2	287.7	-17.7	-2.6	267.3
			1011.9	1018.5				1011.9

Figure C4.6.3.3-3 - Finite Element Idealization and Reactions Obtained for Structure Shown in Figure C4.6.3.3-1

U. S. CUSTOMARY UNITS



TYPICAL SUPERSTRUCTURE SECTION



FINITE ELEMENT IDEALIZATION

	BEAM NO	SAP NODE	SAP FULL DEAD LOAD ON NON-COMPOSITE SECTION	STRESS FULL DEAD LOAD ON NON-COMPOSITE SECTION	SAP SLAB POURS 1 & 2	SAP SLAB POUR 3	SAP SLAB POUR 4	SAP TOTAL OF COLUMNS 3, 4 & 5
NEAR ABUT.	5	8	53.7	56.5	57.7	-4.2	-0.4	53.1
	4	29	61.1	58.4	65.6	-1.5	0.2	64.3
	3	64	56.0	55.5	59.8	-4.5	-0.5	54.8
	2	109	57.4	55.1	61.9	-2.3	-0.3	59.3
	1	167	65.2	65.4	73.4	-13.0	-2.9	57.5
PIER 1	5	282	180.2	185.4	94.4	22.5	60.4	177.3
	4	324	164.1	169.1	67.1	33.3	62.2	162.6
	3	363	171.7	163.1	74.7	33.0	67.0	174.7
	2	402	174.0	183.4	88.2	28.2	61.2	177.6
	1	444	149.9	151.4	64.1	29.4	60.9	154.4
PIER 2	5	576	171.3	166.9	74.3	36.6	66.4	177.3
	4	617	172.2	178.1	84.1	26.5	60.8	171.4
	3	656	176.7	174.0	84.0	30.4	64.9	179.3
	2	695	169.5	176.7	74.9	28.3	61.9	165.1
	1	736	165.7	165.1	73.4	25.7	63.6	162.7
FAR ABUT.	5	901	57.6	58.2	63.6	-11.3	-2.4	49.9
	4	935	55.3	54.7	58.7	-1.9	0.2	57.0
	3	959	55.8	55.4	59.7	-4.1	0.1	55.7
	2	979	58.4	57.5	64.2	-3.6	0.1	60.7
	1	991	58.9	60.3	64.7	-4.0	-0.6	60.1
			2275	2290				2275

Figure C4.6.3.3-3 - Finite Element Idealization and Reactions Obtained for Structure Shown in Figure C4.6.3.3-1 (Continued)

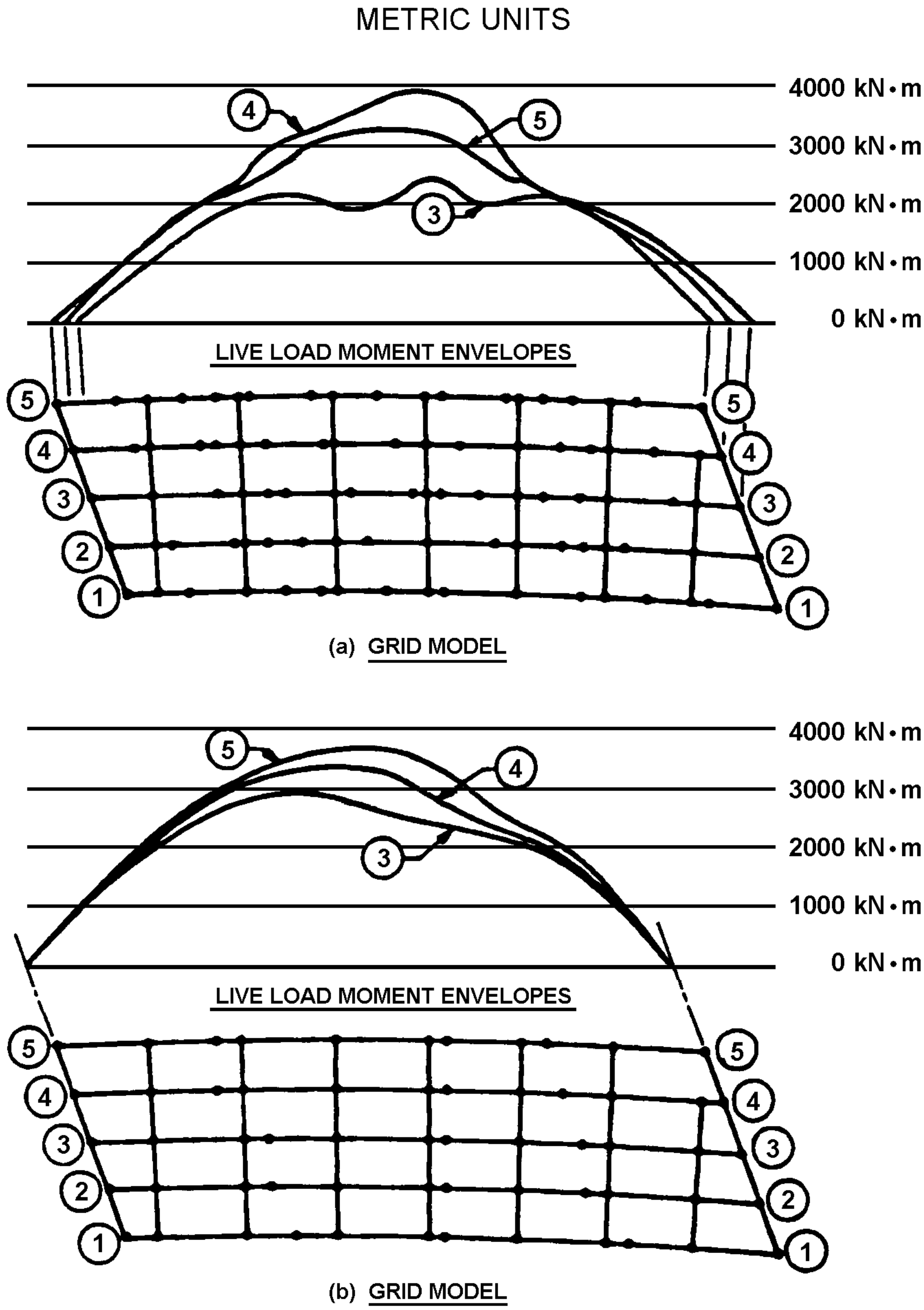


Figure C4.6.3.3-4 - Comparative Live Load Moment Envelopes for the Middle and Two Outside Girders of Curved Skewed Showing the Results of an Apparent "Bug" in Algorithm for Applying Live Load

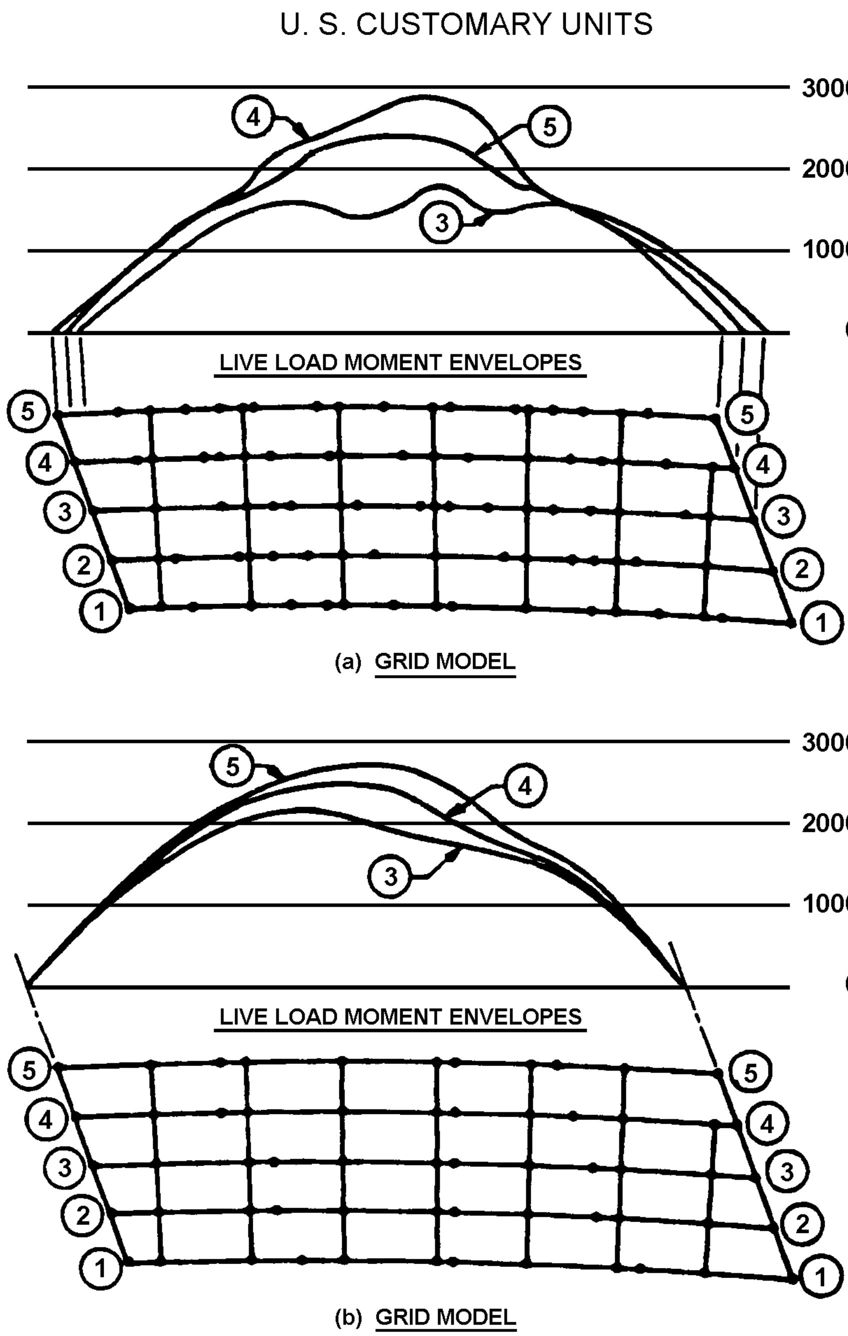


Figure C4.6.3.3-4 - Comparative Live Load Moment Envelopes for the Middle and Two Outside Girders of Curved Skewed Showing the Results of an Apparent "Bug" in Algorithm for Applying Live Load (Continued)

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4.6.4 Redistribution of Negative Moments in Continuous Beam Bridges

4.6.4.1 GENERAL

The following shall replace A4.6.4.1.

The redistribution of force effects in multi-span, multi-beam or girder superstructures is not permitted in the design of Pennsylvania bridges, except as described in D4.6.4.3.

4.6.4.2 REFINED METHOD

Delete A4.6.4.2

4.6.4.3 APPROXIMATE PROCEDURE

The following shall replace A4.6.4.3

A simplified redistribution procedure for compact steel beams given in A6.10.2.2 and D6.10.2.2 may be used.

4.7 DYNAMIC ANALYSIS

4.7.1 Basic Requirements of Structural Dynamics

C4.7.1.4 DAMPING

The following shall supplement AC4.7.1.4.

Damping values obtained from field measurement or tests shall be approved by the Chief Bridge Engineer.

4.7.2 Elastic Dynamic Responses

4.7.2.2 WIND-INDUCED VIBRATION

4.7.2.2.1 Wind Velocities

The following shall supplement A4.7.2.2.1.

The Chief Bridge Engineer will decide if wind tunnel tests are warranted for a structure.

4.7.4 Analysis for Earthquake Loads

4.7.4.3 MULTI-SPAN BRIDGES

4.7.4.3.1 Selection of Method

The following shall supplement A4.7.4.3.1.

A single-mode or a multi-mode analysis is acceptable for bridges whose skew angle is greater than 70º with a total span length less than 152 000 mm {500 ft.}. For major and unusual bridges, and bridges whose skew angle is less than 70º, use the multi-mode spectral analysis. Details of the single-mode spectral method of analysis procedure are given in A4.7.4.3.2.

Use the SEISAB computer program for seismic analysis. In addition, structural analysis programs (such as

C4.7.4.3.1

The following shall replace AC4.7.4.3.1.

The selection of the method of analysis depends on seismic zone, regularity, and importance of the bridge.

Regularity is a function of the number of spans and the distribution of weight and stiffness. Regular bridges have less than seven spans, no abrupt or unusual changes in weight, stiffness, or geometry; and no large changes in these parameters from span-to-span or support-to-support, abutments excluded. A more rigorous analysis procedure may be used in lieu of the recommended minimum.

As a clarification of seismic analysis procedures, the Department will accept a multi-mode analysis in all cases.

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STAAD-III) may be used in lieu of SEISAB for unusual structures, for structures having substructure units with multi-column pier bents, and for structures with multiple simple spans.

Detailed seismic analysis is not required for single span bridges or for bridges in Seismic Zone 1. A seismic analysis may be performed for multi-span bridges in Seismic Zone 1 if designers feel such an analysis will more accurately reflect the connection forces and produce a more economical design.

The following shall replace Table A4.7.4.3.1-1.

Table 4.7.4.3.1-1 – Analysis Procedure

Seismic Zone	Bridges with Two or More Spans
1	None Required
2	A multi-mode analysis for all major and unusual bridges, and bridges whose skew angle is less than 70º. A single mode (or a multi-mode spectral) for bridges whose skew angle is greater than 70º, with total span length less than 152 000 mm {500 ft.}.

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However, a multi-mode analysis is required when the bridge skew angle is less than 70º or the bridge is a major (total of span length greater than 152 000 mm {500 ft.}) or unusual structure. For other cases, a single-mode analysis will suffice.

SEISAB should be used for the seismic analysis. If SEISAB is not suitable for a structure, other structural analysis programs capable of seismic modeling (such as STAAD-III) may be used in lieu of SEISAB. The Designer must stipulate the reasons SEISAB is not suitable and obtain the Department's approval.

It has been found that, in most cases, seismic forces do not control the design. Therefore, when modeling foundations, it is acceptable for designers to perform a number of analyses (computer runs), based on assumptions which sufficiently bound the potential foundation stiffness and then use the worst resulting forces and displacements to design the structure. Use of this modeling procedure is generally acceptable when seismic forces do not control the design.

When seismic forces do control the design, an analysis which more closely models the superstructure and substructure stiffness is required.

4.7.4.3.5P Determination of Elastic Forces and Displacements

For multiple span bridges in Seismic Zone 2, the elastic forces and displacements shall be determined independently along two perpendicular axes by use of a single-mode or multi-mode spectral method of analysis. The resulting forces shall then be combined as specified in A3.10.8. Typically the perpendicular axes are the longitudinal and transverse axes of the bridge, but the choice is open to the designer. The longitudinal axis of a curved bridge may be a chord connecting the two abutments. For single span bridges and bridges in Seismic Zone 1, the elastic forces and displacements shall also be combined as specified in A3.10.8.

C4.7.4.3.5P

When designing a component, it is important to recognize its geometric restraints and/or releases, the direction of the forces and displacements it must accommodate, as well as the skew effects.

4.7.4.4 MINIMUM DISPLACEMENT REQUIREMENTS

The following shall replace the definition of N in Equation A4.7.4.4-1.

N = minimum support length measured perpendicular to abutment or pier face from the end of the beam at the centerline of the bottom flange.

The following shall supplement A4.7.4.4.

The N calculated in Equation A1 shall not be taken less than 300 mm {12 in.}.

C4.7.4.4

The following shall supplement AC4.7.4.4.

For bridges in Seismic Zone 2, the design displacements are specified as the maximum of those determined from the elastic analysis of A4.7.4.3 and D4.7.4.3 or the minimum specified support lengths given by Equation A4.7.4.4-1. This either/or specification was introduced to account for larger displacements that may occur from the analysis of more flexible bridges. It was the opinion of the PEP that displacement obtained from the elastic analysis of bridges should provide a reasonable

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NOTE: S in Equation A1 is based on AASHTO's definition of skew angle, see PP3.2.2 for PennDOT's and AASHTO's definition of skew angle.

The following shall replace Table A4.7.4.4-1.

Table 4.7.4.4-1 – Percentage N by Zone and Acceleration Coefficient

ZONE	ACCELERATION COEFFICIENT	SOIL TYPE	%N
1	<0.025	I or II	100
1	<0.025	III or IV	100
1	≥0.025	All	100
2	All Applicable	All	100
3	All Applicable	All	100
4	All Applicable	All	100

4.7.4.5P BASE ISOLATION DESIGN

4.7.4.5.1P General

This article includes the fundamental requirements for base isolation design, which greatly reduces the earthquake forces that a bridge must resist.

The same acceleration coefficient, A, is prescribed for base isolation design as for cases without base isolation. This coefficient is given on a county-by-county basis in Figure D3.10.2-1. However, a minimum value of 0.1 shall be used.

Seismic zone is not a delineator concerning method of analysis; however, minimum design requirements are governed by the seismic zone. Refer to Table A3.10.4-1 for the appropriate seismic zone designation.

The site coefficient for base isolation design, S_i, which accounts for the effects of the site condition on the elastic response coefficient, is given in Table 1. The soil profile types are the same as those described in A3.10.5.

The R-factors presented in Table A3.10.7.1-1 and A3.10.7.1-2 are used for base isolation design with the exception that the R-factor for all substructures shall not be

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estimate of the displacements resulting from the inelastic response of the bridge. However, it must be recognized that displacements are very sensitive to the flexibility of the foundation; if the foundation is not included in the elastic analysis of A4.7.4.3 and D4.7.4.3, consideration should be given to increasing the specified displacements for bridges founded on very soft soils. This increase may be of the order of 50 percent or more, but, as with any generalization, considerable judgment is required. A better method is to determine upper and lower bounds from an elastic analysis which incorporates foundation flexibility. Special care in regard to foundation flexibility is required for bridges with high piers.

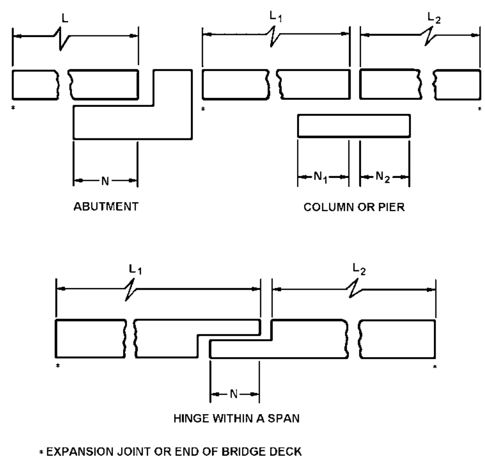

Figure C4.7.4.4-1 - Dimensions for Minimum Support Length Requirements

C4.7.4.5.1P

This article incorporates generic requirements for seismic isolation design. The isolation of structures from the damaging effects of earthquakes is not a new idea. The first patents for base isolation schemes were taken out at the turn of the century, but, until very recently, few structures were built which use these ideas. Early concerns were focused on the fear of uncontrolled displacements at the isolation interface, but these have been largely overcome by the successful development of mechanical energy dissipators. When used in combination with a flexible device, such as an elastomeric bearing or a sliding plate, an energy dissipator can control the response of an isolated structure by limiting both the displacements and the forces. Interest in base isolation, as an effective means of protecting bridges from earthquakes, has, therefore, revived in recent years. To date, there are several hundred bridges in New Zealand, Japan, Italy, and the United States which use base isolation principles and technology in their seismic design.

The intent of seismic isolation is to increase the fundamental period of vibration so that the structure is subjected to lower earthquake forces. However, the

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greater than 2.0.

Each statically stable segment of the structure shall be analyzed for the statically equivalent seismic force given in D4.7.4.5.2P. For Seismic Zone 2, the requirements of D4.7.4.5.6P shall apply.

Table 4.7.4.5.1P-1 – Site Coefficient for Base Isolation (S_i)

Site Coefficient	Soil Profile Type				
	I	II	III	IV	IV
S_i	1.0	1.5	2.0	3.0	3.0

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reduction in force is accompanied by an increase in displacement demand which must be accommodated within the flexible mount. Furthermore, longer period bridges can be lively under service loads.

There are, therefore, three basic elements in those base isolation systems that have been used to date. These are:

(a) A flexible mounting so that the period of vibration of the total system is lengthened sufficiently to reduce the force response.

(b) A damper or energy dissipater so that the relative deflections across the flexible mounting can be limited to a practical design level.

(c) A means of providing rigidity under low (service) load levels such as wind and braking forces.

Flexibility

An elastomeric bearing is not the only means of introducing flexibility into a structure, but it certainly appears to be the most practical and the one with the widest range of application. The idealized force response with increasing period (flexibility) is shown schematically in the force response curve in Figure C1. Reductions in base shear occur as the period of vibration of the structure is lengthened. The extent to which these forces are reduced is primarily dependent on the nature of the earthquake ground motion and the period of the fixed base structure. However, as noted above, the additional flexibility needed to lengthen the period of the structure will give rise to large relative displacements across the flexible mount. Figure C2 shows an idealized displacement response curve from which displacements are seen to increase with increasing period (flexibility).

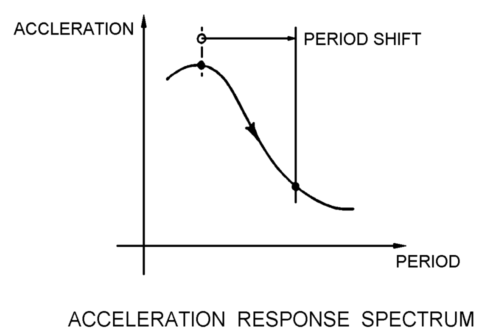

Figure C4.7.4.5.1P-1 - Idealized Force Response Curve

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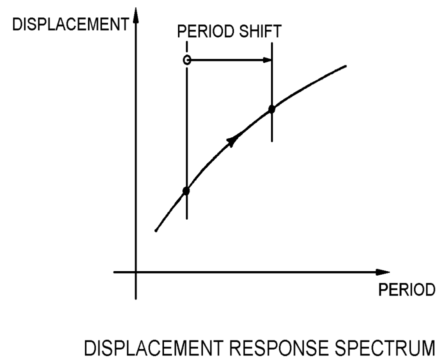

Figure C4.7.4.5.1P-2 - Idealized Displacement Response Curve

Energy Dissipation

Large relative displacements can be controlled if substantial additional damping is introduced into the structure at the isolation level. This is shown schematically in Figure C3. It can also be seen that higher damping removes much of the sensitivity to variations in ground motion characteristics, as is indicated by the smoother force response curves at higher damping.

One of the most effective means of providing a substantial level of damping is through hysteretic energy dissipation. The term "hysteretic" refers to the offset between the loading and unloading curves under cyclic loading. Figure C4 shows an idealized force-displacement loop where the enclosed area is a measure of the energy dissipated during one cycle of motion. Mechanical devices which use the plastic deformation of either mild steel or lead to achieve this behavior have been developed.

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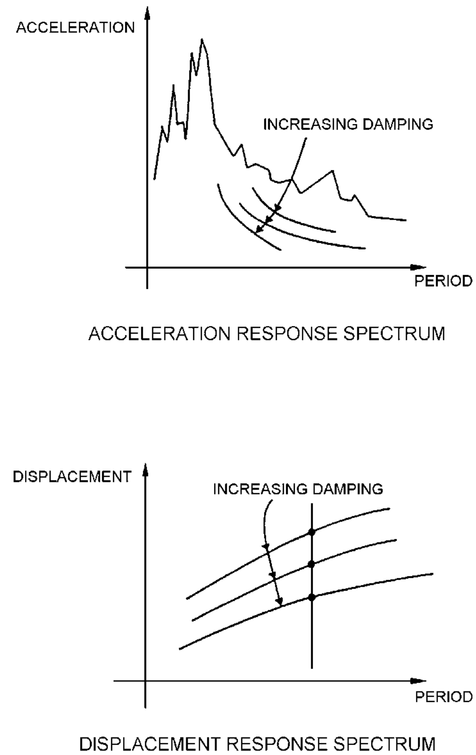

Figure C4.7.4.5.1P-3 - Response Curves for Increasing Damping

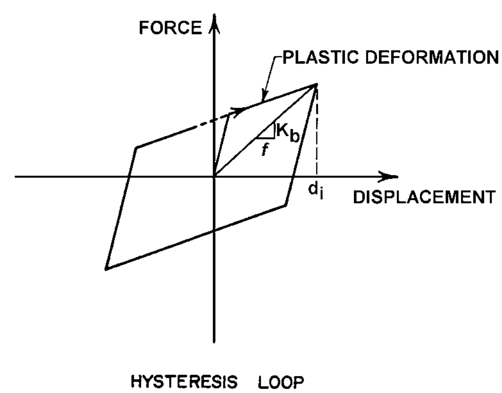

Figure C4.7.4.5.1P-4 - Idealized Hysteresis Loop

<u>Rigidity Under Low Lateral Loads</u>

While lateral flexibility is highly desirable for high seismic loads, it is clearly undesirable to have a structural system that will vibrate perceptibly under frequently occurring loads such as wind loads or braking loads. Mechanical energy dissipators may be used to provide rigidity at these service loads by virtue of their high initial elastic stiffness. Alternately, some base isolation systems require a separate wind restraint device for this purpose - typically a rigid component which is designed to fail at a

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given level of lateral load.

Design Application

The seismic design principles fodr base isolation are best illustrated by Figure C5 which was based on the AASHTO Standard Specification for Highway Bridges.. The same basic concept concerning base isolation also applies to the LRFD Specification. The solid uppermost line is the realistic (elastic) ground response spectrum as recommended in the AASHTO Standard Specification for Highway Bridges for the highest seismic zone. This is the spectrum that is used to determine actual forces and displacements for conventional design. The lowest solid line is the design curve from the AASHTO Standard Specification for Highway Bridges. It is seen to be approximately one-fifth of the realistic forces given by the AASHTO Standard Specification for Highway Bridges. This reduction, to obtain the design forces, is consistent with an R-factor of 5 for a multi-column bent. Also shown is the probable overstrength of a bent so designed. Now if the bridge is isolated, the shear forces in this multi-column bent may be represented by an inelastic spectrum that incorporates the damping of the isolation system, as shown by the small dashed line in Figure C5.

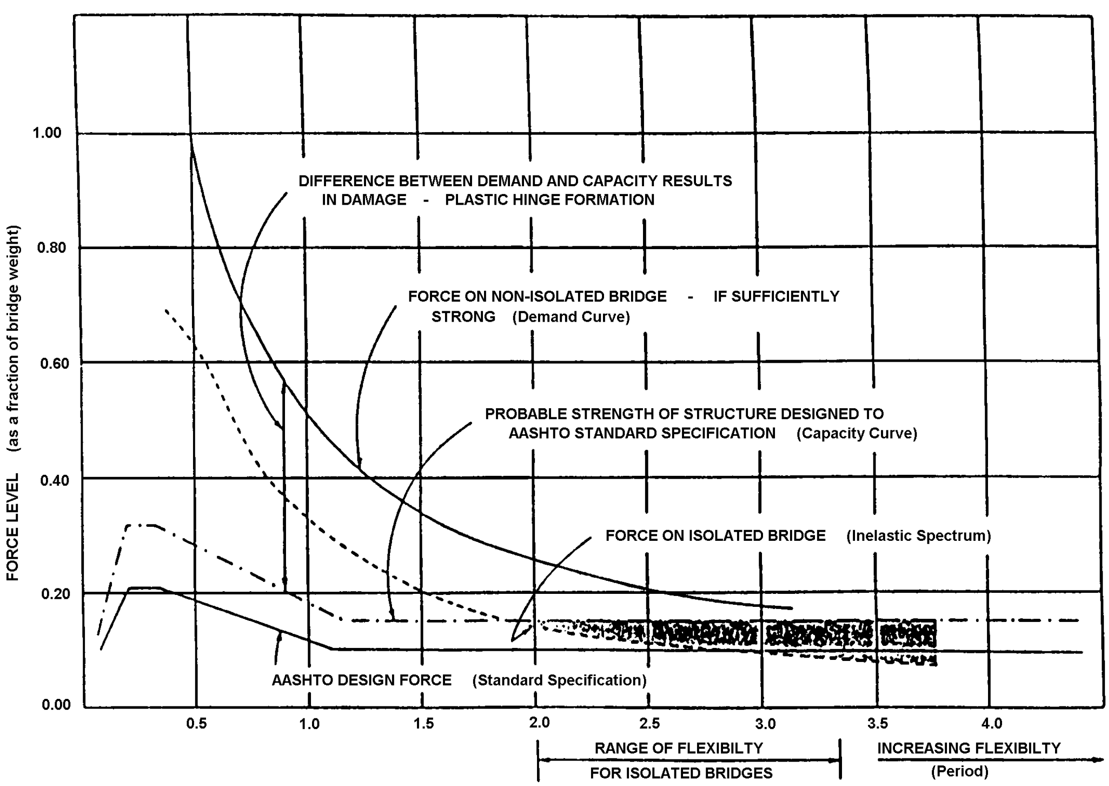

Figure C4.7.4.5.1P-5 – Earthquake Forces

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The period of the bridge will be in the 2.0- to 2.5-second range; in this range the overstrength (actual capacity) of the bent exceeds the realistic forces (demand) for the isolated bridge. This region has been shaded in Figure C5. There is, therefore, no inelastic deformation or ductility required of the bent, and elastic performance (without damage) is ensured. The benefits of seismic isolation for bridges may be summarized as follows:

(a) Reduction in the realistic forces to which a bridge will be subjected by a factor of between 5 and 10.

(b) Elimination of the ductility demand and, hence, damage to the piers.

(c) Control of the distribution of the seismic forces to the substructure elements with appropriate sizing of the elastometric bearings.

(d) Reduction in column design forces by a factor of approximately 2 in comparison with conventional design.

(e) Reduction in foundation design forces by a factor of between 2 and 3 in comparison with conventional design.

The intent of seismic isolation design is to eliminate or significantly reduce damage (inelastic deformation) to the substructure. By limiting the R-factor to a maximum value of 2, the overstrength inherent in the substructures will ensure minimal ductility demand on the substructure for the design earthquake. For essential structures, consideration may be given to reducing the maximum value of R to as low as 1 to ensure complete elastic response of the substructure.

4.7.4.5.2P Statically Equivalent Seismic Force and Coefficient

The statically equivalent force is given by:

$$F = C_s W \qquad (4.7.4.5.2P\text{-}1)$$

where:

C_S = elastic seismic response coefficient

W = weight of the superstructure segment supported by isolation bearings

The elastic seismic response coefficient, C_S, used to determine the equivalent force is given by the dimensionless relation:

C4.7.4.5.2P

For seismic isolation design, the elastic seismic coefficient is again related to the elastic ground response spectra. Here, the form is slightly different from that for non-isolation design (which involved $T^{2/3}$) and, for 5% damping, is given by

$$C_s = \frac{AS_i}{T} \qquad (C4.7.4.5.2P\text{-}1)$$

In this case, A is once again the acceleration coefficient, S_i is the site coefficient for base isolation (Table D4.7.4.5.1P-1), and the 1/T factor accounts for the decrease in the response spectra ordinates as T increases. The specific S_i values reflect the fact that above a period of 1.0 second, there is a 1.0 to 1.5 to 2.0 to 3.0 relationship for the spectral accelerations for soil profile Types I, II, III and IV,

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$$C_s = \frac{K_b d_i}{W} \qquad (4.7.4.5.2P\text{-}2)$$

where:

K_b = equivalent linear stiffness of all bearings supporting the superstructure segment

W = weight of the superstructure segment supported on the isolation bearings

d_i = displacement across the isolation bearings

The displacement d_i (mm) {in.} is given by

Metric Units: $$d_i = \frac{250 A S_i T}{B} \qquad (4.7.4.5.2P\text{-}3)$$

U.S. Customary Units: $$d_i = \frac{10 A S_i T}{B}$$

where:

A = acceleration coefficient as defined in D4.7.4.5.1P

B = damping factor given in Table D4.7.4.5.2P-1

S_i = dimensionless site coefficient for isolation design for the given soil profile as designated in D4.7.4.5.1P

T = period of vibration (seconds) given by

$$T = 2\pi\sqrt{\frac{W}{K_b g}} \qquad (4.7.4.5.2P\text{-}4)$$

where:

g = acceleration of gravity (mm/s^2) {in/sec^2}

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respectively. Once again, C_s should not exceed a value of 2.5A.

If the effects of damping are included, the elastic seismic coefficient is given by

$$C_S = \frac{A S_i}{TB} \qquad (C4.7.4.5.2P\text{-}2)$$

where B is the damping term given in Table D4.7.4.5.2P-1. Note that for 5% damping, B = 1.0.

The quantity C_s is a dimensionless design coefficient which, when multiplied by g, produces the spectral acceleration. This spectral acceleration, S_A, is related to the spectral displacement, S_D, by the relationship

$$S_A = \omega^2 S_D \qquad (C4.7.4.5.2P\text{-}3)$$

Where ω is the circular natural frequency and is given by $2\pi/T$. Therefore, since $S_A \approx C_s g$, then

$$S_A = \left[\frac{A S_i}{TB}\right] g \qquad (C4.7.4.5.2P\text{-}4)$$

and

Metric Units:

$$S_D = \left[\frac{1}{\omega^2}\right]\left[\frac{A S_i}{TB}\right] g \qquad (C4.7.4.5.2P\text{-}5)$$

$$= \left[\frac{T^2}{2\pi}\right]\left[\frac{A S_i}{TB}\right] 9807\,mm/s^2$$

$$= \frac{248.4 A S_i T}{B}\,mm$$

U.S. Customary Units

$$S_D = \left[\frac{1}{\omega^2}\right]\left[\frac{A S_i}{TB}\right]$$

$$= \left[\frac{T^2}{2\pi}\right]\left[\frac{A S_i}{TB}\right] 386.4\,in/\sec^2$$

$$= \frac{9.79 A S_i T}{B}\,in.$$

Denoting S_D as d_i, which is the displacement across the elastomeric bearings, the above is approximated to be

Metric Units: $$d_i = \frac{250 A S_i T}{B}\,mm \qquad (C4.7.4.5.2P\text{-}6)$$

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U.S. Customary Units: $d_i = \frac{10AS_iT}{B} in$

An alternate form for C_s is possible. The quantity C_s is defined by the relationship

$$F = C_S W \qquad \text{(C4.7.4.5.2P-7)}$$

where:

F = earthquake design force

W = weight of the structure

Therefore,

$$C_s = \frac{F}{W} = \frac{K_b d_i}{W}$$

where K_b is the equivalent linear spring of all bearings supporting the superstructure segment (see Figure DC4.7.4.5.1P-4). The equivalence of this form to the previous form is evident by recalling that $K_b = \omega^2 W/g$, from which

Metric Units:

$$C_S = \left[\frac{\omega^2 W}{g}\right]\left[\frac{d_i}{W}\right] = \left[\frac{2\pi^2}{T}\right]\left[\frac{1}{9807}\right]\left[\frac{248.4AS_iT}{B}\right] = \frac{AS_i}{BT}$$

(C4.7.4.5.2P-9)

U.S. Customary Units:

$$C_S = \left[\frac{\omega^2 W}{g}\right]\left[\frac{d_i}{W}\right] = \left[\frac{2\pi^2}{T}\right]\left[\frac{1}{386.4}\right]\left[\frac{9.79AS_iT}{B}\right] = \frac{AS_i}{BT}$$

Table 4.7.4.5.2P-1 – Damping Coefficient B

Damping (% of Critical)*	≤2	5	10	20	30	40	>50
B	0.8	1.0	1.2	1.5	1.7	1.9	2.0

*The percent of critical damping shall be verified by test of the isolation system's characteristics. The damping coefficient shall be based on linear interpolation for damping levels other than those given.

4.7.4.5.3P Requirements for Elastic Force Determination

The statically equivalent force determined according to D4.7.4.5.2P, which is associated with the displacement across the isolation bearings, shall be applied independently along two perpendicular axes and combined as specified in

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A3.10.8.

Typically, the perpendicular axes are the longitudinal and transverse axes of the bridge, but the choice is open to the designer. The longitudinal axis of a curved bridge may be a chord connecting the two abutments.

4.7.4.5.4P Design Displacement for Other Loads

Adequate clearance must be provided to permit shear deflections in the bearings resulting from braking loads, wind loads and centrifugal forces. These deflections are a function of the force-deflection characteristics of the bearings.

4.7.4.5.5P Design Forces for Seismic Zone 1

The design force for the connection between superstructure and substructure at each bearing is given by

$$F_A = K_b d_i \quad (4.7.4.5.5P\text{-}1)$$

where:

K_b = equivalent linear stiffness of the isolation bearing

d_i = displacement of the isolated bridge deck as specified in D4.7.4.5.2P, using a minimum acceleration coefficient, A, of 0.10

4.7.4.5.6P Design Forces for Seismic Zone 2

The requirements of D4.7.4.5.2P shall apply, except that in the determination of seismic design forces an R-factor of 1 shall be used.

4.7.4.5.7P Substructure Design Requirements

4.7.4.5.7.1P Foundations and Abutments

The provisions of this document and the LRFD shall apply to the design of foundations and abutments.

4.7.4.5.7.2P Columns, Footings and Connections

4.7.4.5.7.2aP Structural Steel

The provisions of this document and the LRFD shall apply to the design of structural steel columns and connections.

4.7.4.5.7.2bP Reinforced Concrete

The provision of this document and the LRFD shall apply to the design of reinforced concrete columns, pier footings and connections.

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C4.7.4.5.4P

To protect against fatigue under frequently occurring loads, deflections should be kept as small as practicable. As a guide, the deflection due to braking load should be less than 10% of the deformable rubber thickness, and that for extreme wind loads should be less than 25% of the deformable rubber thickness.

C4.7.4.5.5P

Seismic isolation design provides a significant reduction in the real elastic forces, and the exception of this section permits this force reduction to be utilized in the design of the bearings. However, it should be noted that the acceleration coefficient, which has a maximum value of 0.09 for bridges in Seismic Zone 1, is specified to have a minimum value of 0.10 if seismic isolation is used. This conservatism will ensure, for most areas in Seismic Zone 1, that the isolation bearings are capable of resisting twice the design earthquake. This level of conservatism is greater than that inherent in the requirement for bearings being designed for 0.2 times the dead load.

SPECIFICATIONS

COMMENTARY

4.8 ANALYSIS BY PHYSICAL MODELS

4.8.2 Bridge Testing

The following shall replace A4.8.2.

When approved by the Chief Bridge Engineer, existing bridges may be instrumented and results obtained under various conditions of traffic and/or environmental loads or load tested with special purpose vehicles to establish force effects and/or the load carrying capacity of the bridge.

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5.2 DEFINITIONS

Revise definitions as follows:

Low-density Concrete - Concrete containing low-density aggregate and having an air dry density not exceeding 1840 kg/m^3 {115 pcf}.

Special Anchorage Device - Anchorage device whose adequacy should be proven in a standardized acceptance test and approved by Chief Bridge Engineer prior to its usage. Most multi-plane anchorages and all bond anchorages are Special Anchorage Devices.

5.3 NOTATION

Revise notation of f'_c and f'_{ci} as follows:

f'_c = compressive structural design strength of concrete at 28 days, unless another age is specified (MPa) {ksi} (5.4.2.1)

f'_{ci} = compressive structural design strength of concrete at time of initial loading or transfer (MPa) {ksi} (5.9.4.4P)

5.4 MATERIAL PROPERTIES

5.4.1 General

The following shall replace the first and second paragraphs of A5.4.1.

Designs should be based on the material properties cited herein and on the use of materials which conform to the standards for the grades of construction materials as specified in Publication 408.

When other grades or types of materials are used, their properties, including statistical variability, shall be established prior to design and approved by the Chief Bridge Engineer. The minimum acceptable properties and test procedures for such materials shall be specified in the contract documents.

5.4.2 Normal and Structural Lightweight Concrete

5.4.2.1 COMPRESSIVE STRENGTH

The following shall replace A5.4.2.1.

Minimum mix design compressive strength (MPa) {ksi} shall be in accordance with Section 704.1(b) of Publication 408.

The following classes of cement concrete with corresponding f'_c and n values are to be used for structural designs:

C5.4.2.1

The following shall replace AC5.4.2.1.

The strength requirements in Publication 408 for mix design strength are intended to ensure strengths within a 95% confidence level.

Two different structural design strengths are given for Class AAA concrete. Use 31 MPa {4.5 ksi} structural design strength for analysis of structures that were designed before August 16, 1989, and Class AAA concrete was used in the original design. Use 28 MPa {4.0 ksi} structural design strength for analysis of all structures designed after August 16, 1989 and for the design of all new structures.

SPECIFICATIONS COMMENTARY

Class of Cement Concrete	f'_c Structural Design Strength	Normal Density n Modular Ratio	Low Density n Modular Ratio
AAA	28 MPa {4.0 ksi}	8	12
AA	24 MPa {3.5 ksi}	8	13
A	21 MPa {3.0 ksi}	9	14
C	14 MPa {2.0 ksi}	11	17

The densities for normal density and low density concrete are given in A3.5.1 and D3.5.1, respectively.

The use of different classes of cement concrete shall be as follows:

Class AAA

- deck slab
- precast channel beams
- sidewalks
- top slab of concrete box culverts at grade

Class AA

- curbs
- barriers
- divisors
- concrete diaphragms
- abutment backwalls
- cheek walls
- shear blocks
- U-wings above bridge seat construction joint
- footings if needed to resist high diagonal tension stresses
- noise barriers

Class A

- piers
- abutments below bridge seat
- pedestals
- wingwalls
- retaining walls
- footings
- arch culverts
- spandrel walls
- walls and top and bottom slabs of box culverts under fill
- walls and bottom slab of box culverts at grade
- caissons

SPECIFICATIONS

Class C

- below bottom of footings when specified

Cement concrete for precast prestressed concrete bridge members and other precast components shall comply with the requirements of Publication 408.

Prestressed concrete members shall be designed with a structural design strength (f'_c) between 35 MPa and 55 MPa {5 ksi and 8 ksi}. For use of structural design strength (f'_c) greater than 55 MPa {8 ksi}, approval must be obtained from the Chief Bridge Engineer.

Precast deck panels (when permitted) and precast box culverts shall be designed with a structural design strength (f'_c) of 35 MPa {5 ksi}.

All prestressed components not otherwise specified shall have a structural design strength of 35 MPa {5 ksi} unless approval is obtained from the Chief Bridge Engineer.

Low density concrete shall not be used for prestressed applications, or concrete box culverts.

The use of low density concrete may be considered for deck slabs on rehabilitation projects where reduction of weight is important.

Show the structural design strength (f'_c) of the concrete for each part of the structure on the plans.

5.4.2.2 COEFFICIENT OF THERMAL EXPANSION

The following shall replace A5.4.2.2.

The thermal coefficient of expansion shall be taken as:

- for normal density concrete: $10.8 \times 10^{-6}/°C\{6.0 \times 10^{-6}/°F\}$, and

- for low-density concrete: $9.0 \times 10^{-6}/°C$ $\{5.0 \times 10^{-6}/°F\}$

If a more precise coefficient of thermal expansion is required, laboratory tests shall be performed on the specific mix to be used. The test results must be submitted and approved by the Chief Bridge Engineer prior to usage.

5.4.2.3 SHRINKAGE AND CREEP

5.4.2.3.1 General

The following shall supplement A5.4.2.3.1.

For Figures A5.4.2.3.2-1 and A5.4.2.3.3-2 and Equations AC5.4.2.3.2-1 and AC5.4.2.3.3-1, if the volume to surface (v/s) ratio is greater than 150 mm {6 in.}, use v/s ratio of 150 mm {6 in.}.

For the design of single span prestressed beams, creep and shrinkage effects shall be neglected.

COMMENTARY

It is preferred to use only low density coarse aggregate in the mix for which the density of the concrete will be 1840 kg/m^3 {0.115 kcf}. It is possible to obtain a concrete with a density of only 1680 kg/m^3 {0.105 kcf} by using both coarse and fine low density aggregate, but it is difficult to control such a mix during construction.

C5.4.2.3.1

The following shall supplement AC5.4.2.3.1.

The Department's design procedure for prestressed concrete beams neglects the axial effects induced by creep and shrinkage.

In the past, the Department has neglected creep and shrinkage effects for single span prestressed beams without

SPECIFICATIONS

For the design of prestressed beams, creep and shrinkage effects shall only be applied when they cause adverse effects.

5.4.2.3.2 Creep

The following shall replace the definition of t in the first paragraph of A5.4.2.3.2.

t = maturity of concrete, i.e., time since casting (Day)

The following shall replace the last paragraph of A5.4.2.3.2.

The surface area used in determining the volume to surface area ratio should include only the area that is exposed to atmospheric drying. For prestressed concrete beams, the top surface (beam/slab interface) shall be included in surface area used in determining the volume to surface area ratio (see DC5.4.2.3.2). For prestressed concrete box beams, the surface area of the void shall not be included in determining the volume to surface ratio (see DC5.4.2.3.2).

5.4.2.3.3 Shrinkage

The following shall replace the definition of t in the first paragraph of A5.4.2.3.3.

t = drying time after moist or steam curing (Day)

The following shall supplement A5.4.2.3.3.

In determining k_h, the average ambient relative humidity shall be taken as 70%.

For guidance in calculating the volume-to-surface ratio, see D5.4.2.3.2 and DC5.4.2.3.2.

5.4.2.4 MODULUS OF ELASTICITY

The following shall supplement A5.4.2.4.

:

COMMENTARY

any adverse effects. Therefore, the Department will continue this practice.

C5.4.2.3.2

The following shall supplement AC5.4.2.3.2.

The volume to surface area ratio to be used in the determination of creep and shrinkage effects shall take into consideration the condition of the member during the time period where the creep and shrinkage effects are most significant. For example, the engineer may wish to consider the entire perimeter of a prestressed beam being exposed if the majority of the shrinkage will have taken place prior to the placement of the slab. However, the surface area of the slab itself would not include the portion of the slab of the beam/slab interface.

The Department's experience with prestressed concrete box beams has shown that the void in box beam is not adequately ventilated. Therefore, the surface area of void is not considered in the calculation of volume-to-surface area ratio.

C5.4.2.4

The following shall supplement AC5.4.2.4.

The equation for calculation of E_c is based on the density of concrete without an allowance for steel reinforcement. Therefore, for normal density concrete (2400 kg/m^3) {0.150 kcf} and low density concrete (1840 kg/m^3) {0.115 kcf}, y_c equals 2320 kg/m^3 {0.145 kcf} and 1760 kg/m^3 {0.110 kcf}, respectively, in Equation A5.4.2.4-1.

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Metric Units

f'_c Structural Design Strength (MPa)	Normal Density E_c Modulus of Elasticity (MPa)	Low Density E_c Modulus of Elasticity (MPa)
28	25 000	17 000
24	23 000	16 000
21	22 000	15 000
14	18 000	12 000

U.S. Customary Units:

f'_c Structural Design Strength (ksi)	Normal Density E_c Modulus of Elasticity (ksi)	Low Density E_c Modulus of Elasticity (ksi)
4.0	3,600	2,600
3.5	3,400	2,400
3.0	3,100	2,200
2.0	2,600	1,800

5.4.2.5 POISSON'S RATIO

The following shall replace A5.4.2.5.

Poisson's ratio shall be assumed to be 0.2. If a more precise Poisson's ratio is required, laboratory tests shall be performed on the specific mix to be used. The test results must be submitted and approved by the Chief Bridge Engineer prior to usage.

For components which are expected to be subject to cracking, the effect of Poisson's ratio may be neglected.

5.4.2.6 MODULUS OF RUPTURE

The following shall replace A5.4.2.6.

Metric Units:

The modulus of rupture, f_r in MPa, shall be taken as:

- for normal density concrete .... $0.63\sqrt{f_c^{'}}$
- for "sand-low-density" concrete .... $0.52\sqrt{f_c^{'}}$
- for "all-low-density" concrete........................ $0.45\sqrt{f_c^{'}}$

COMMENTARY

C5.4.2.6

The following shall replace the last sentence of AC5.4.2.6.

The direct tensile strength stress should be used for these cases, see A5.4.2.7.

SPECIFICATIONS

U.S. Customary Units:

The modulus of rupture, f_r in ksi, shall be taken as:

- for normal density concrete .... $0.24\sqrt{f_c^{'}}$
- for "sand-low-density" concrete .... $0.20\sqrt{f_c^{'}}$
- for "all-low-density" concrete........................ $0.17\sqrt{f_c^{'}}$

If a more precise modulus of rupture is required, laboratory tests shall be performed on the specific mix to be used. The test results must be submitted and approved by the Chief Bridge Engineer prior to usage.

5.4.3 Reinforcing Steel

5.4.3.1 GENERAL

The following shall replace A5.4.3.1.

Reinforcing bars, deformed wire, cold-drawn wire, welded plain wire fabric and welded deformed wire fabric shall conform to the materials standards as specified in Publication 408.

Reinforcement bars shall be designed with f_y = 420 MPa {60 ksi}. ASTM-A 615/A 615M, ASTM-A 996/A 996 M, and ASTM-A 706/A 706M, Grade 420 {Grade 60}, reinforcement steel shall be used. Reinforcement bars with f_y = 300 MPa {40 ksi} may only be used with approval of the Chief Bridge Engineer. When approved by the Chief Bridge Engineer, Grade 300 {Grade 40} bars shall comply with ASTM-A 615/A 615M, Grade 300 {Grade 40}.

Welding of reinforcement bars during fabrication or construction will not be allowed unless specified or permitted by the Chief Bridge Engineer. When specified or permitted by the Chief Bridge Engineer, welding of the Grade 420 {Grade 60} bars shall be preceded by preheat according to applicable construction specifications. If specified or approved by the Chief Bridge Engineer, welded splices or other mechanical connections may be used according to the AASHTO Specifications.

Deformed reinforcement shall be used, except for spirals, hoops and wire fabric.

Rail steel (ASTM-A 996/A 996 M), reinforcement bars shall not be used in the design of bridge piers, abutments, beams, footings and piles where ductility is essential for structural performance during seismic activities. Furthermore, this steel shall not be used where bending or welding of the reinforcement is required. The rail steel

(ASTM-A 996/A 996 M) reinforcement bars may be used for deck and diaphragms.

This shall be clearly indicated on both the design

COMMENTARY

C5.4.3.1

The following shall supplement AC5.4.3.1.

As a result of Federally-mandated seismic criteria, the ductility in reinforcement bars used in portions of superstructures and substructures has become an important design consideration. ASTM-A 996/A 996 M is a tough, but brittle material and does not provide sufficient ductility to satisfy seismic design requirements.

The barrier design equations are based on the yield line theory, and ductile failure of reinforcement steel is required.

SPECIFICATIONS

drawings and shop drawings by adding the last sentence of Note 6 in PP1.7.4 to the general note sheet and the individual bar schedule.

5.4.3.3 SPECIAL APPLICATIONS

The following shall supplement the second paragraph of A5.4.3.3.

Requirements for which substructure and superstructure components are required to have epoxy-coated reinforcement is given in D5.4.3.6P.

5.4.3.4P CONTRACT DOCUMENTS

Detailing of bars shall be such that the total length shall not exceed 12 000 mm {40 ft.} for No. 10 and No. 13 bars {No. 3 and No. 4 bars}, and 18 000 mm {60 ft.} for larger bars and 2400 mm {8 ft.} in projections from the primary direction.

Bar marks should be simple and instructive as to number, size and sequence of placement, e.g., 21-F1602 @ 450 {21-F502@18"}. In general, the following key letters are recommended:

- In stem of abutment, wings, and retaining walls ..... W
- In piers..P
- In deck slab...S
- In stems of T-beams .... B
- In curbs and barriers C
- In footings (including dowels)..................................F

Use the prefix E to designate epoxy-coated bars, e.g., 21-ES1602 @ 150 {21-ES502@6"}.

The number of bars and spacing shall be shown only at one place--in plan, section, or elevation, whichever is best. At other locations, show only size and mark.

The bar schedule shall be arranged in tabular form for each part of the structure (Pier 1, Pier 2, deck slab, etc.), indicating the bar mark, size, length and type (straight or bent) of each bar. The bending diagram shall be shown near the bar schedule. Preferably in the contract drawings, bar schedules for substructure and superstructure components shall be segregated to assist in field usage (e.g., abutment bar schedules should follow abutment drawings, pier bar schedule should follow pier drawings, deck bar schedule should follow deck drawings, etc.).

All lap lengths shall be indicated on the drawings.

Embedment length of all dowels shall be shown on the drawings.

Splices of No. 43 and No. 57 bars {No. 14 and No. 18 bars} shall be mechanical splices and shall be shown on the drawings.

COMMENTARY

C5.4.3.4P

The 12 000 mm {40 ft.} and 2400 mm {8 ft.} lengths are limits for the purpose of shipping and handling.

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5.4.3.5P CONSTRUCTIBILITY

To facilitate placement and vibration of concrete in reinforced concrete members over 1200 mm {4 ft.} in depth (pier caps, deep footings, etc.), reinforcement shall be designed and detailed to provide the following clearances:

Stirrup spacing--minimum of 225 mm {9 in.} clear, except in a localized region where a smaller spacing will not interfere with placement or vibration.

Top longitudinal bar spacing--225 mm {9 in.} clear in at least one space and 150 mm {6 in.} minimum at all other spaces.

In cases where anchor bolts must be embedded in heavily reinforced concrete members, the reinforcement shall be designed and detailed to allow the contractor the option of placing the anchor bolts in preformed holes or of drilling when permitted. The bars shall be arranged to clear a circle concentric with, and 50 mm {2 in.} larger than, the anchor bolt.

When specifying a dowel hole diameter, use a minimum diameter 25 mm {1 in.} greater than the diameter of the dowel bar, i.e. = doweled hole for No. 16 bar {No. 5 bar} has a 40 mm {1 5/8 in.} diameter. Set dowels in predrilled holes filled with non-shrink grout. Do not use dowels for concrete members of thickness less than 250 mm {10 in.}.

Bars may be bundled to obtain the required clearance. In no case shall a concrete member be oversized to obtain the required clearance, unless there is no other acceptable alternative.

Except for vertical reinforcement extending from the top of footing into pier columns, projection of vertical reinforcement bars into an adjacent pour shall not exceed 2400 mm {8 ft.}. In such cases, dowels shall be used.

5.4.3.6P EPOXY-COATED REINFORCING BARS

To prevent or minimize the deterioration of structural concrete caused by deicing chemicals, fusion-bonded epoxy-coated rebars shall be provided as follows:

(a) Superstructures

1. All bars in reinforced concrete deck slabs, curbs, barriers and backwalls, including bars that protrude into these elements from some other portion of the structure, shall be epoxy-coated. Diaphragm bars adjoining the expansion dam and protruding into the deck slab shall also be epoxy-coated.

2. As a preventive measure to guard against rusting of steel in prestressed concrete beam ends at joints and subsequent spalling of concrete for all new beams, all mild reinforcing steel for a distance of 2700 mm {9 ft.} from the

COMMENTARY

C5.4.3.6P

The potential for salt spray damage will be greatest at grade separation structures where sprays are generated by traffic below the bridge. In some cases, damaging sprays may be created by wind currents at high-level structures crossing waterways, ravines and other similar natural features.

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beam end shall be fully epoxy-coated, regardless of beam type. This requirement applies only to the beam end adjoining a deck joint regardless of the type of joint dam/seal installed.

3. When adjacent box beams or channels are utilized without a separate concrete deck slab and when it is anticipated that the deck surface will be exposed to deicing chemicals, all bars in the top slab of the box beams or channels shall be epoxy-coated.

(b) Substructures

1. Epoxy-coated rebar and a breathable sealant (as per PP3.4.2) shall be provided in all portions of the substructure above a plane approximately 900 mm {3 ft.} below the finished grade for the following cases:

    - Piers located under expansion joints or exposed to salt spray.
    - Abutments, wingwalls and retaining walls exposed to salt spray.
    - Abutment portions exposed to discharge from troughs located below expansion dams.
    - Abutment stems located below expansion joints.

2. Epoxy-coat main reinforcement bars (J-bars) protruding from abutment, wing and pier footings into the stem or columns.

COMMENTARY

Epoxy-Coated J-Bars:

At least in one instance, significant section losses in the J-bars at the juncture of the abutment stem and the footer have been experienced. In most cases, if not all, this area will accumulate moisture since a crack is always present due to the construction joint. The increased usage of drainable backfill increases the probability of continued moisture in this area. Since this bar is the most critical bar in performance of abutments, it is prudent to require the J-bar to be epoxy-coated for all projects.

5.4.4 Prestressing Steel

5.4.4.1 GENERAL

The following shall replace the first portion of the first sentence of A5.4.4.1.

Uncoated stress-relieved or low-relaxation seven-wire strand, or uncoated plain or deformed, high-strength bars, shall conform to the following material standards:

The following shall supplement A5.4.4.1.

C5.4.3.6

Delete AC5.4.4.1.

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Low relaxation prestressing steel shall be used, unless an alternative is approved by the Chief Bridge Engineer.

In Table A5.4.4.1-1, the maximum diameter for strand shall be changed from 15.24 mm {0.6 in.} to 13.2 mm {0.52 in.}.

The maximum diameter for strand used in prestressed plank beams shall be 9.53 mm {3/8 in.}. The fabricator may use 12.7 mm {1/2 in.} diameter strands, provided the initial force per strand does not exceed the maximum allowable for the 9.53 mm {3/8 in.} strand, i.e., strand-for-strand substitution is permitted.

COMMENTARY

C5.4.4.2 MODULUS OF ELASTICITY

The following shall replace Figure AC5.4.4.2-1.

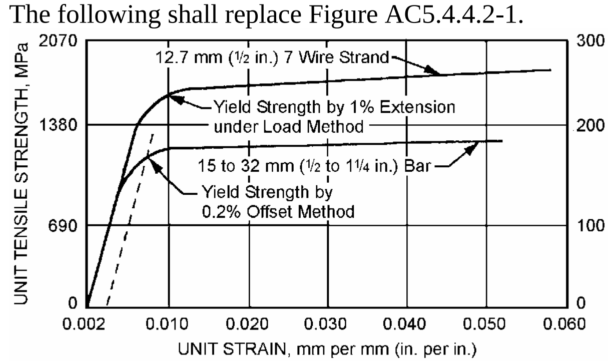

Figure C5.4.4.2-1 - Typical Stress-Strain Curve for Prestressing Steels

5.4.5 Post-Tensioning Anchorages and Couplers

The following shall replace the first paragraph of A5.4.5.

Anchorages and tendon couplers shall conform to the requirements of Publication 408.

C5.4.5

Delete the first paragraph of AC5.4.5.

The following shall replace the second paragraph of AC5.4.5.

Characteristics of anchorages and couplers are summarized below.

The following shall replace the third and last bulleted items of AC5.4.5, respectively.

Couplers are to be used only at locations shown on the contract documents or approved by the Chief Bridge Engineer.

Unless waived by the Chief Bridge Engineer because of suitable previous tests and/or experience, qualification of anchorages and couplers are to be verified by testing. The test results must be submitted and approved by the Chief Bridge Engineer prior to usage.

5.4.6 Ducts

5.4.6.1 GENERAL

The following shall replace A5.4.6.1.

Ducts for tendons shall be rigid or semi-rigid galvanized ferrous metal.

The radius of curvature of tendon ducts shall not be

C5.4.6.1

SPECIFICATIONS

less than 6000 mm {20 ft.}, except in the anchorage areas where 3600 mm {12 ft.} may be permitted.

The effects of grouting pressure on the ducts and the surrounding concrete shall be investigated.

The maximum support interval for the ducts during construction shall be indicated in the contract documents.

Delete AC5.4.6.1.

5.5 LIMIT STATES

5.5.3 Fatigue Limit State

5.5.3.1 GENERAL

Delete the third paragraph of A5.5.3.1.

5.5.3.2 REINFORCING BARS

The following shall supplement A5.5.3.2.

The effective fatigue stress range shall not exceed the permissible fatigue range, f_f, as defined in equation A5.5.3.2-1.

f_{fe} = effective fatigue stress range in reinforcing steel (MPa) {ksi}
= γ (PTF) Δf_s

where:

PTF = Pennsylvania Traffic Factor as given in Table D6.6.1.2.2-1

γ = load factor specified in Table A3.4.1-1 for the fatigue load combination

Δf_s = calculated unfactored fatigue stress range in reinforcing steel (MPa) {ksi}

5.5.3.3 PRESTRESSING TENDONS

The following shall replace A5.5.3.3.

The effective fatigue stress range in prestressing tendons shall not exceed 70 MPa {10 ksi}.

f_{fp} = effective fatigue stress range in prestressing tendons (MPa) {ksi}

COMMENTARY

C5.5.3.1

Delete the third paragraph of AC5.5.3.1.

The following shall supplement AC5.5.3.1.

A5.5.3.1 lists the concrete components that do not have to be checked for the fatigue limit state. Therefore, if the specification does not expressly state that the fatigue limit state does not need to be checked for a certain concrete component, it must be checked for that concrete component. The fatigue limit state check for concrete components applies to both superstructure and substructure units.

C5.5.3.3

Supplement AC5.5.3.3 with the following.

Since Pennsylvania uses allowable tensile stress in a precompressed tensile zone equal to $0.25\sqrt{f'_c}$ {U.S. Customary Units: $0.0948\sqrt{f'_c}$}, it is believed that fatigue is not of concern.

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$= \gamma \text{ (PTF) } \Delta f_p$

where:

PTF = Pennsylvania Traffic Factor as given in Table D6.6.1.2.2-1

γ = load factor specified in Table A3.4.1-1 for the fatigue load combination

Δf_p = calculated fatigue stress range in prestressing tendon (MPa) {ksi}

5.5.4 Strength Limit State

5.5.4.2 RESISTANCE FACTORS

5.5.4.2.1 Conventional Construction

Add the following bullets to the first paragraph:

- for flexure of prestressed voided concrete box beams.. 0.95
- for axial resistance of the concrete portion of concrete filled undamaged steel pipe piles in compression:...φ = 0.55
- for axial resistance of prestressed concrete piles in compression...φ = 0.45
- for axial resistance of precast concrete piles in compression...φ = 0.45
- for combined axial and flexural resistance of undamaged concrete filled steel pipe piles
  axial resistance ....φ = 0.60
  flexural resistance..................................... φ = 0.85

Add the following sentence to the beginning of the third paragraph of A5.5.4.2.1.

Partially prestressed components shall not be used unless written approval is obtained from the Chief Bridge Engineer.

5.5.4.2.2 Segmental Construction

The following shall supplement A5.5.4.2.2.

Unbonded post-tensioning systems shall not be used, except when permitted by the Chief Bridge Engineer for temporary condition, future rehabilitation schemes, or rehabilitation of existing bridges. The last sentence of the second paragraph of A5.5.4.2.2 shall be changed from "...approved by the Engineer" to "...approved by the Chief Bridge Engineer".

COMMENTARY

C5.5.4.2.1

The following shall supplement AC5.5.4.2.1.

This reduction is to account for the possibility of the void forms floating out of place.

The basis for the resistance factors for concrete and concrete filled piles is described in DC5.13.4.1.

5.6 DESIGN CONSIDERATIONS

5.6.1 General

The following shall supplement A5.6.1.

The value of the moment of inertia for the computation of flexural stiffness of slabs, beams, columns, etc. shall be based on gross concrete section with the effect of reinforcement neglected for computation of forces only.

In the computation of flexural stiffness and resistance moments of beams, the haunch shall be taken as zero. However, in the computation of dead load, the haunch shall be taken into account.

For box culverts, include the effect of the haunches for computation of flexural stiffness.

C5.6.1

The following shall supplement AC5.6.1.

The beam haunch depth can vary at the time of construction. Therefore, at the time of design, the Department does not want to count on something that may not be there. However, if the haunch depth of an existing beam can be determined from field measurements, the haunch may be used in the computation of flexural stiffness and resistance moments for the analysis of an existing beam.

5.7 DESIGN FOR FLEXURAL AND AXIAL FORCE EFFECTS

C5.7.1 Assumptions for Service and Fatigue Limit States

PSLRFD rounds the modular ratio for reinforced and prestressed concrete to the nearest integer, except that the ratio due to differences between concrete strengths (such as deck to beam) is rounded to the nearest tenth.

5.7.2 Assumptions for Strength and Extreme Event Limit States

5.7.2.1 GENERAL

The following shall replace the fourth and seventh bulleted items in A5.7.2.1, respectively.

- If a maximum strain exceeding 0.003 is to be utilized, the concrete must be confined, test results must show that it is obtainable and it must be approved by the Chief Bridge Engineer.

- The concrete compressive stress-strain distribution is assumed to be rectangular as given in A5.7.2.2. Other assumed concrete compressive stress-strain distribution shapes may be used if they are substantiated with test results and approved by the Chief Bridge Engineer.

5.7.2.2 RECTANGULAR STRESS DISTRIBUTION

Delete the second paragraph of A5.7.2.2 including equation A5.7.2.2-1 and the definition list following the equation.

The following shall supplement A5.7.2.2.

β_1 shall be based on slab concrete for composite

SPECIFICATIONS

construction and shall be based on beam concrete for non-composite construction.

5.7.3 Flexural Members

5.7.3.1 STRESS IN PRESTRESSING STEEL AT NOMINAL FLEXURAL RESISTANCE

5.7.3.1.1 Components with Bonded Tendons

The following shall replace A5.7.3.1.1.

For rectangular or flanged sections subjected to flexure about one axis where the approximate stress distribution specified in A5.7.2.2 is used and for which f_{pe} is not less than 0.5 f_{pu}, the average stress in prestressing steel, f_{ps}, may be taken as:

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right) \quad (5.7.3.1.1\text{-}1P)$$

for which:

$$k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right) \quad (5.7.3.1.1\text{-}2P)$$

$$c = \frac{A_{ps}\, f_{pu} - 0.85\,\beta_1\, f_{c'}\,[b' h_f + b_s t_s - b_{w'}(h_f + t_s)]}{0.85\,\beta_1\, f_{c'}\, b_{w'} + k\, A_{ps}\dfrac{f_{pu}}{d_p}} \quad (5.7.3.1.1\text{-}3P)$$

$$b' = b\left(\frac{f'_{c\,beam}}{f'_c}\right) \quad (5.7.3.1.1\text{-}4P)$$

$$b_{w'} = b_w\left(\frac{f'_{c\,beam}}{f'_c}\right) \quad (5.7.3.1.1\text{-}5P)$$

where:

A_{ps} = area of prestressing steel (mm^2) {in^2}

f_{pu} = specified tensile strength of prestressing steel (MPa) {ksi}

f_{py} = yield strength of prestressing steel (MPa) {ksi}

A_s = area of mild steel tension reinforcement (mm^2) {in^2}f'_c
= structural design strength of the slab concrete

COMMENTARY

C5.7.3.1.1

The following shall supplement AC5.7.3.1.1. The equation for c has been developed so that it can accommodate the neutral axis in the slab, the flange or the web. The LRFD equations for c only accommodate the neutral axis in the flange or in the web.

For ease of computation, the equation for c was developed with structural design strength of the slab as the basis. Therefore, the width of beam flange and the width of web have to be transformed using the ratio of structural design strengths. Since this is an ultimate condition, the concrete is in the inelastic range and the typical modular ratio based on the modulus of elasticity for the slab and beam concrete does not apply.

Figure D5.7.3.2.6P-1 provides a graphic representation of some of the notations.

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(MPa) {ksi}

f'_{cbeam} = structural design strength of the beam concrete (MPa) {ksi}

f_y = yield strength of tension reinforcement (MPa) {ksi}

b = width of beam top flange (mm) {in.}

b' = transformed width of beam top flange (mm) {in.}

b_s = effective flange width (mm) {in.}

b_w = width of web (mm) {in.}

b'_w = transformed width of web (mm) {in.}

h_f = depth of beam top flange (mm) {in.}

t_s = depth of deck (mm) {in.}

d_p = distance from extreme compression fiber to the centroid of the prestressing tendons (mm) {in.}

c = distance between the neutral axis and the compressive face (mm) {in.}

β_1 = stress block factor specified in A5.7.2.2

For a composite beam with the neutral axis in the deck, $b' = b_s$ and $b'_w = b_s$.

For a composite beam with the neutral axis in the beam top flange, $b'_w = b'$.

For composite beam with the neutral axis in the web, no modifications.

For non-composite beam, f'_c = structural design strength of the beam concrete.

For non-composite beam with the neutral axis in the beam top flange, $t_s = 0$, $b_s = 0$ and $b'_w = b'$.

For non-composite beam with the neutral axis in the web,
$t_s = 0$ and $b_s = 0$.

5.7.3.1.2 Components with Unbonded Tendons

The following shall supplement A5.7.3.1.2.
This article applies to tendons which are unbonded

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full-length and not partial length debonded strands used in prestressed beams. Full-length unbonded tendons are not allowed unless approved by the Chief Bridge Engineer.

5.7.3.2.2 Flanged Sections

The following shall supplement A5.7.3.2.2.

The nominal flexural resistance of prestressed beams shall be determined using D5.7.3.2.6P or D5.7.3.2.7P.

5.7.3.2.5 Strain Compatibility Approach

The following shall supplement A5.7.3.2.5

Strain compatibility approach shall not be used unless approved in writing by the Chief Bridge Engineer.

5.7.3.2.6P Under-Reinforced Prestressed Beams

For under-reinforced prestressed beams subjected to flexure about one axis, where the appropriate stress distribution specified in A5.7.2.2 is used and the tendons are bonded, the general equation for nominal flexural resistance shall be taken as:

$$M_n = T\left(d_p - \frac{\beta_1 t_s}{2}\right) - C_1\ \frac{\beta_1 h_f}{2} - C_2 \beta_1 \left(\frac{c}{2} - \frac{t_s}{2}\right) \tag{5.7.3.2.5P-1}$$

for which:

$C_0 = (b_s\text{-}b')\ t_s\ \beta_1\ 0.85\ f'_c$ (5.7.3.2.6P-2)

$C_1 = (b'\text{-}b'_w)\ (t_s\text{+}h_f)\ \beta_1\ 0.85\ f'_c$ (5.7.3.2.6P-3)

$C_2 = b'_w\ c\beta_1\ 0.85\ f'_c$ (5.7.3.2.6P-4)

$T = A_{ps}\ f_{ps}$ (5.7.3.2.6P-5)

The definitions for the notations in the above equations are given in the list in D5.7.3.1.1.

Equation 1 applies to Figure 1. For a composite beam with the neutral axis in the deck, $b' = b_s$ and $b'_w = b_s$.

For a composite beam with the neutral axis in the beam top flange, $b'_w = b'$.

For composite beam with the neutral axis in the web, no modifications.

For non-composite beam, f'_c = structural design strength of the beam concrete.

COMMENTARY

C5.7.3.2.5P

The location of C_o is where the moment was summed about in the development of Equation 1.

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For non-composite beam with the neutral axis in the beam top flange, $t_s = 0$, $b_s = 0$ and $b'_w = b'$.

For non-composite beam with the neutral axis in the web, $t_s = 0$ and $b_s = 0$.

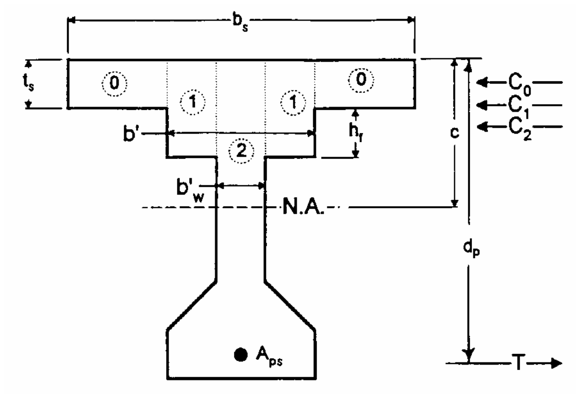

Figure 5.7.3.2.6P-1 - General Beam Cross-Section

5.7.3.2.7P Over-Reinforced Prestressed Beams

For over-reinforced prestressed beams subjected to flexure about one axis, which meet the requirements of A5.7.3.3.1 and D5.7.3.3.1, the general equation for nominal flexural resistance shall be taken as:

$$M_n = \left(0.36\,\beta_1 - 0.08\,\beta_1^2\right)\, f'_c\, b_{w'}\, d_e^2 + 0.85\,\beta_1\, f'_c\,(b' - b_{w'})\,h_f\left(d_e - t_s - \frac{h_f}{2}\right) + 0.85\,\beta_1\, f'_c\,(b_s - b')\,t_s\left(d_e - \frac{t_s}{2}\right) \quad (5.7.3.2.7P\text{-}1)$$

For a composite beam with the neutral axis in the deck, $b' = b_s$ and $b'_w = b_s$

For a composite beam with the neutral axis in the beam top flange, $b'_w = b'$

For composite beam with the neutral axis in the web, no modifications.

For non-composite beam with the neutral axis in the beam top flange $t_s = 0$, $b_s = 0$ and $b'_w = b'$.

For non-composite beam with the neutral axis in the web, $t_s = 0$ and $b_s = 0$

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5.7.3.3 LIMITS FOR REINFORCEMENT

5.7.3.3.1 Maximum Reinforcement

The following shall supplement A5.7.3.3.1.

Over-reinforced prestressed components shall not be used unless approved by the Chief Bridge Engineer.

5.7.3.3.2 Minimum Reinforcement

The following shall replace A5.7.3.3.2.

The minimum reinforcement of any section of a flexural component, reinforced with conventional rebars, prestressing tendons or any combination thereof, is that adequate to develop a factored flexural resistance, M_r, at least equal to the lesser of:

- 1.2 times the cracking moment, M_{cr}, determined on the basis of elastic stress distribution and the modulus of rupture, f_r, of the concrete as specified in D5.4.2.6, or

- 1.33 times the factored moment required by the applicable strength load combinations specified in A3.4.1 and D3.4.1.

For prestressed concrete girders, the resistance provided by the prestressing reinforcement must satisfy the minimum reinforcement provisions for positive flexure at all points along the beam, excluding the ends of the beam in the development zone.

The provisions for shrinkage and temperature reinforcement of A5.10.8 and D5.10.8 must also be considered.

The cracking moments for prestressed beams in the positive moment region shall be as given below:

Non-composite

$$M_{crnc} = M_D + S_b \left[\frac{P}{A_g} + \frac{Pe}{S_b} - \frac{M_D}{S_b} + f_r \right] \quad (5.7.3.3.2\text{-}2P)$$

Composite

$$M_{crc} = M_{DNF} + M_{DCF} + S_{bc}\left[\frac{P}{A_g} + \frac{P\,e}{S_b} - \frac{M_{DNF}}{S_b} - \frac{M_{DCF}}{S_{bc}} + f_r \right] \quad (5.7.3.3.2\text{-}3P)$$

Composite-Transformed

$$M_{crct} = M_{DNF} + M_{DCF} \quad S_{bct}\left[\frac{P}{A_g} + \frac{P\,e}{S_b} - \frac{M_{DNF}}{S_b} - \frac{M_{DCF}}{S_{bc}} + f_r \right] \quad (5.7.3.3.2\text{-}4P)$$

COMMENTARY

C5.7.3.3.2P

In the design of prestressed beams, the creep and shrinkage effects do not need to be taken into account for the cracking moment.

For prestressed beams made continuous, this provision should also be satisfied for both the positive and negative moment connections.

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where:

P = final prestressing force (N) {kips}

e = eccentricity of p/s force, measured from the neutral axis to the centroid of the p/s force (mm) {in.}

f_r = modulus of rupture of the concrete (MPa) {ksi}

M_D = dead load, final moment (N·mm) {k·in}

M_{DNF} = dead load, non-composite, final moment (N·mm) {k·in}

M_{DCF} = dead load, composite, final moment (N·mm) {k·in}

S_b = bottom sections modulus, non-composite, non-transformed (mm^3) {in^3}

S_{bc} = bottom section modulus, composite, transformed for slab (mm^3) {in^3}

S_{bct} = bottom section modulus, composite, transformed for slab and prestressing steel (mm^3) {in^3}

In the calculation of S_{bc}, the slab is transformed by modular ratio of E_{slab}/E_{beam}.

In the calculation of S_{bct}, the slab and the prestressing steel in the beam are transformed by the modular ratio of E_{slab}/E_{beam} and $E_{p/s}/E_{beam}$, respectively.

5.7.3.4 CONTROL OF CRACKING BY DISTRIBUTION OF REINFORCEMENT

Replace the second, third and fourth paragraphs with the following:

The spacing of mild steel reinforcement in the layer closest to the tension face shall satisfy the following:

Metric Units:

$$s \leq \frac{123\,000\gamma_e}{\beta_s f_s} - 2d_c \quad (5.7.3.4\text{-}1)$$

U.S. Customary Units:

$$s \leq \frac{700\gamma_e}{\beta_s f_s} - 2d_c$$

in which:

$$\beta_s = 1 + \frac{d_c}{0.7(h-d_c)} \quad (5.7.3.4\text{-}2)$$

where:

γ_e = exposure factor

C5.7.3.4

Replace the fifth paragraph with the following:

Extensive laboratory work involving deformed reinforcing bars has confirmed that the crack width at the service limit state is proportional to steel stress. However, the significant variables reflecting steel detailing were found to be the thickness of concrete cover and spacing of the reinforcement.

Eq. 1 is expected to provide a distribution of reinforcement that will control flexural cracking. The equation is based on a physical crack model (*Frosh 2001*) rather than the statistically-based model used in previous editions of the specifications. It is written in a form emphasizing reinforcement details, i.e., limiting bar spacing, rather than crack width. Furthermore, the physical crack model has been shown to provide a more realistic estimate of crack widths for larger concrete covers compared to the previous equation (*Destefano 2003*).

Eq.1 with Class 1 exposure condition is based on an assumed crack width of 0.43 mm {0.017in}. Previous research indicates that there appears to be little or no correlation between crack width and corrosion, however, the different classes of exposure conditions have been so defined in order to provide flexibility in the application of

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= 1.00 for Class 1 exposure condition
= 0.75 for Class 2 exposure condition

d_c = thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto (mm) {in.}. The (13 mm) {1/2 in} wearing surface for deck slab, top and bottom slab of box culvert and (25 mm) {1 in} extra cover provided to account for uneven ground level of footing bottom mat reinforcement and bottom slab of box culverts, shall not be included.

f_s = tensile stress in steel reinforcement at the service limit state (MPa) (ksi)

h = overall thickness or depth of the component (mm) {in.}

Class 1 exposure condition applies when cracks can be tolerated due to reduced concerns of appearance and/or corrosion. This applies to all reinforced concrete members except precast and cast-in-place box culverts and segmental construction except for specific conditions as defined under Class 2.

Class 2 exposure condition applies to transverse design of segmental concrete box girders for any loads applied prior to attaining full nominal concrete strength and when there is increased concern of appearance and/or corrosion. Class 2 exposure also applies to precast and cast-in-place box culverts.

In the computation of d_c, the actual concrete cover thickness is to be used except in deck slabs, box culvert slabs and footings as defined in d_c.

When computing the actual stress in the steel reinforcement, axial tension effects shall be considered, while axial compression effects may be considered.

The minimum and maximum spacing of reinforcements shall also comply with the provisions of Articles 5.10.3.1 and 5.10.3.2 respectively.

The effects of bonded prestressing steel may be considered, in which case the value of f_s used in Eq. 1, for the bonded prestressing steel, shall be the stress that develops beyond the decompression state calculated on the basis of a cracked section or strain compatibility analysis.

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these provisions to meet the needs of the Authority having jurisdiction. Class 1 exposure condition could be thought of as an upper bound in regards to crack width for appearance and corrosion. Areas that the Authority having jurisdiction may consider for Class 2 exposure condition would include decks and substructures exposed to water. The crack width is directly proportional to the γ_e exposure factor, therefore, if the individual Authority with jurisdiction desires an alternative crack width, the γ_e factor can be adjusted directly. For example a γ_e factor of 0.5 will result in an approximate crack width of 0.22 mm {0.0085 in}.

Replace the eighth paragraph with the following:

The β_s factor, which is a geometric relationship between the crack width at the tension face versus the crack width at the reinforcement level, has been incorporated into the basic crack control equation in order to provide uniformity of application for flexural member depths ranging from thin slabs in box culverts to deep pier caps and thick footings. The theoretical definition of β_s may be used in lieu of the approximate expression provided.

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5.7.3.6 DEFORMATIONS

5.7.3.6.4P CAMBER OF PRESTRESSED BEAMS

5.7.3.6.4aP Camber Due to Prestressing

Camber due to prestressing shall be calculated by Equations 1, 2 and 3 for beams with straight, draped and debonded strands, respectively.

Straight Strands

$$\Delta_{prestressed} = \frac{P e_s L^2}{8 E_{ci} I} \qquad (5.7.3.6.4aP\text{-}1)$$

Draped Strands

$$\Delta_{prestressed} = \frac{PL^2 [4X^2(e_n - e_s) + 3 e_s]}{24 E_{ci} I} \qquad (5.7.3.6.4aP\text{-}2)$$

Debonded Strands

$$\Delta_{prestressed} = \frac{1}{8E_{ci} I} [P_b e_b L^2 + P_1 e_1 [L^2 - (L_t + 2L_1)^2] + P_2 e_2 [L^2 - (L_t + 2L_2)^2] + \dots P_i e_i [L^2 - (L_t + 2 L_i)^2]] \qquad (5.7.3.6.4aP\text{-}3)$$

for which:

$$P = P_t \left(1 - \frac{\Delta f_s}{100}\right) \qquad (5.7.3.6.4aP\text{-}4)$$

$$P_{b;1,2...i} = P_{t\, b;1,2...i} \left(1 - \frac{\Delta f_s}{100}\right) \qquad (5.7.3.6.4aP\text{-}5)$$

where:

e_s = eccentricity at mid-span (mm) {in.}

e_n = eccentricity at end of beam (mm) {in.}

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C5.7.3.6.2 Deflection and Camber

The following shall supplement AC5.7.3.6.2.

Equation A1 applies to members where it is appropriate to consider them as cracked at the service load levels. Therefore, Equation A1 would apply to reinforced concrete members, but not prestressed concrete members.

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E_{ci} = modulus of elasticity of beam concrete at transfer (MPa) {ksi}

I = moment of inertia of beam (mm^4) {in^4}

L = beam length (mm) {in.}

L_t = transfer length (mm) {in.}

$L_{1,2...i}$ = distance from centerline of bearing to debonding

P = prestressing force at selected time for camber calculations (N) {kips}

P_b = prestressing force at selected time for camber calculations of full-length bonded strands (N) {kips}

P_t = prestressing force at transfer (N) {kips}

P_{tb} = prestressing force at transfer of full-length bonded strands (N) {kips}

$P_{b1,2...i}$ =prestressing force at selected time for camber calc

$P_{tb1,2...i}$ =prestressing force at transfer of debonded group 1

Δf_s = assumed percentage of prestressing loss since transfer for selected time

X = percent of L for drape point

5.7.3.6.4bP Deflection Due to Dead Loads

The maximum downward deflection at mid-span due to the beam weight and internal diaphragms shall be taken as:

$$\Delta_{D1} = \frac{5 M_{D1} L^2}{48\ E_{ci} I} \qquad (5.7.3.6.4bP\text{-}1)$$

where:

M_{D1} = unfactored moment at mid-span due to the beam weight and any internal diaphragms (N·mm) {k·in}

The maximum downward deflection at mid-span due to slab, formwork, external diaphragms and any other dead load which is applied to the beam before the slab has hardened shall be taken as:

$$\Delta_{D2} = \frac{5(M_{D2}) L^2}{48\ E_c I} \qquad (5.7.3.6.4bP\text{-}2)$$

where:

M_{D2} = unfactored moment at mid-span due to dead load

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applied to the beam before the slab has hardened, except the beam weight and internal diaphragms (N·mm) {k·in}

E_e = modulus of elasticity of beam concrete (MPa) {ksi}

For simple span construction, the maximum downward deflection at mid-span due to superimposed dead load shall be taken as:

$$\Delta_{D3} = \frac{5(M_{D3})L^2}{48 E_c I_c} \quad (5.7.3.6.4bP\text{-}3)$$

where:

M_{D3} = unfactored moment at mid-span due to superimposed dead load (MPa) {ksi}

I_c = moment of inertia of composite beam (mm^4) {in^4}

For continuous span construction, the maximum downward deflection at mid-span due to superimposed dead load shall be determined from continuous span analysis.

5.7.3.6.4cP Total Camber at Transfer of Prestressing

The total camber at transfer shall be taken as:

$\Delta_t = \Delta_{prestressed} - \Delta_{D1}$

Δf_s shall be assumed to be zero in determining $\Delta_{prestressed}$.

5.7.3.6.4dP Camber for Bearing Slope

The camber for determining the bearing slope shall be taken as:

$\Delta_b = \Delta_{prestressed} - \Delta_{D1}$

Δf_s shall be assumed to be 10% in determining $\Delta_{prestressed}$.

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5.7.3.6.4eP Total Camber in Beams at Time of Construction

The total camber in the beams at time of construction shall be taken as:

$\Delta_c = (\Delta_{prestressed} - \Delta_{D1})\ C_r$

where:

C_r = 1.6

Δf_s = 10% in determining $\Delta_{prestressed}$

COMMENTARY

C5.7.3.6.4eP

After release, beams may be stored for a period of several days to possibly six months or more. During this time, the camber increases due to creep. The prestressing force, on the other hand, decreases due to shrinkage, creep of the concrete and relaxation of the steel. These are opposing effects.

Assuming the beams are stored from 7 to 80 days, it may be reasonable to estimate that the creep factor, C_r, varies in a range of 1.5 to 2.0 and the prestress loss, Δf_s, varies in a range of 5 to 15% in that time. For design, unless better information is available, C_r = 1.6 and Δf_s = 10% will be used. These are average values from Pennsylvania prestressers. The assumed values used for C_r and Δf_s shall be shown on the design drawings.

5.7.3.6.4fP Final Camber

Negative final camber (sag) shall be limited to L/2000.

5.7.4 Compression Members

5.7.4.2 LIMITS FOR REINFORCEMENT

The following shall replace the last paragraph of A5.7.4.2.

For bridges in Seismic Zones 1 and 2, the minimum area of longitudinal reinforcement may be that required for a component with a reduced effective area of concrete, provided that both the full section and the reduced effective section are capable of resisting the factored loads and that the area of reinforcement is not less than 0.7 percent of the gross area of the column or 1.0 percent of the reduced effective area of the column, whichever is less.

For wall-type piers in Seismic Zones 1 and 2, the minimum reinforcement provisions of 5.10.11.4.2 shall apply for both the weak and strong directions.

C5.7.4.2

Delete the last two paragraphs of AC5.7.4.2.

5.7.4.3 APPROXIMATE EVALUATION OF SLENDERNESS EFFECTS

The following shall supplement A5.7.4.3.

To account for reduced levels of cracking in the reinforced concrete members under service load conditions, use an effective moment of inertia that is 1.43 times the moment of inertia given by Equation A5.7.4.3-1 or Equation A5.7.4.3-2 for all serviceability computations.

C5.7.4.3

The following shall supplement AC5.7.4.3.

The 1.43 multiplier was taken from Section 10.11 of the ACI 318-05 Code.

5.7.4.4 FACTORED AXIAL RESISTANCE

The following shall supplement A5.7.4.4.

For computing factored axial resistance, spiral reinforcement not meeting the requirements of Equation A5.7.4.6-1 shall be considered tie reinforcement.

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5.7.4.6 SPIRALS AND TIES

COMMENTARY

C5.7.4.6P

The actual ratio, r_s, of spiral reinforcement to total volume of concrete may be computed using simple geometry:

$r_s = [A_s (2\pi R)]$/Volume of concrete within spiral $= 2A_s/Rs$

where:

A_s = Spiral bar area (mm^2) {in^2}
R = the radius of the spiral steel (mm) {in}
S = pitch of the spiral (mm) {in}

The volume of concrete is computed as that within the confines of a single spiral loop $= \pi R^2 s$. (mm^3) {in^3}

5.8 SHEAR AND TORSION

5.8.1 Design Procedures

C5.8.1.1 FLEXURAL REGIONS

The following shall supplement AC5.8.1.1.

Nearly all pile-supported footings will meet the criteria to be considered a deep beam.

5.8.2 General Requirements

5.8.2.9 SHEAR STRESS ON CONCRETE

The following shall replace the definition of φ in A5.8.2.9.

φ = resistance factors taken from A5.5.4.2 and D5.5.4.2

C5.8.2.9

The following shall supplement AC5.8.2.9.

The PAPIER program will use the lower limit of 0.72h for d_v since the PAPIER program does not take advantage of the beneficial effect of axial compressive loading in a column when computing the concrete shear strength.

5.8.3 Sectional Design Model

5.8.3.2 SECTION NEAR SUPPORTS

The following shall replace A5.8.3.2.

The provisions of A5.8.1.2 shall be considered.

Where the reaction force in the direction of the applied shear introduces compression into the end region of a member, the location of the critical section for shear shall be taken as the larger of 0.5 $d_v \cot \theta$ or d_v from the internal face of the support, where d_v and θ are measured at the critical section for shear. Otherwise, the design section shall be taken at the internal face of the support. Where the beam-type element extends on both sides of the reaction area, the design section on each side of the reaction shall be determined separately based upon the loads on each side of

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the reaction and whether their respective contribution to the total reaction introduces tension or compression into the end region.

5.8.3.3 NOMINAL SHEAR RESISTANCE

The following shall replace the definition of b_v and d_v in A5.8.3.3.

b_v = effective web width taken as the minimum web width within the depth d_v as determined in D5.8.2.9P (mm) {in.}

d_v = effective shear depth as determined in D5.8.2.9P (mm) {in.}

5.8.3.4 DETERMINATION OF Β AND Θ

5.8.3.4.1 Simplified Procedure for Nonprestressed Sections

C5.8.3.4.1

The following shall supplement AC5.8.3.4.1.

Currently, the computer programs ABLRFD and PAPIER are based on using the simplified procedure.

5.8.3.4.2 General Procedure

C5.8.3.4.2

The following shall supplement AC5.8.3.4.2.

Replacing $(V_u - V_p) \cot \theta$ with $(V_u - V_p)$ is meant to eliminate the need for iteration. This simplified approach is acceptable. However, the PSLRFD computer program is based on the iterative approach included in earlier versions of the AASHTO-LRFD Specifications.

In Pennsylvania, the prestressing steel is not continuous over the supports in the typical method used for making prestressed beams continuous for live load.

For this typical method, the prestressing steel near the supports is on the compression side of the beam. Therefore, for Equations A2 and A3, A_{ps} will be zero and A_s will be the longitudinal reinforcement in the slab for locations near a continuous support.

5.8.3.5 LONGITUDINAL REINFORCEMENT

The following shall replace the definition of φ_f, φ_v and φ_c in A5.8.3.5.

φ_f, φ_v, φ_c, φ = resistance factors taken from Articles A5.5.4.2 and D5.5.4.2 as appropriate for moment, shear and axial resistance

COMMENTARY

C5.8.3.5

The following shall supplement AC5.8.3.5.

Equation 5.8.3.5-1 was developed to check the tensile capacity of the reinforcement due to the combined effects of moment, shear and axial force. Therefore, when using Equation 5.8.3.5-1, the following items should be considered: the absolute value of V_u may be used, M_u is taken as positive if it causes tension in the longitudinal reinforcement, M_u is taken as 0.0 if it causes compression in the tensile reinforcement and N_u is taken as positive if it causes tension in the longitudinal reinforcement, N_u is taken

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as 0.0 if it causes compression in the longitudinal reinforcement.

Equation 5.8.3.5-2 is similar to equation 5.8.3.5-1 except that at simply supported ends the value of both the applied moment and applied axial force is 0.0.

5.8.3.6 SECTIONS SUBJECTED TO COMBINED SHEAR AND TORSION

5.8.3.6.3 Longitudinal Reinforcement

The following shall replace Equation A5.8.3.6.3-1.

$$(A_s f_y + A_{ps} f_{ps}) \geq \frac{M_u}{\phi d_v} + \frac{0.5 N_u}{\phi} + \cot\theta \sqrt{(V_{hcb})^2 + \left(\frac{0.45 p_h T_u}{2 A_o \phi}\right)^2} \qquad (5.8.3.6.3\text{-}1)$$

where:

$$If \ \frac{V_u}{\phi} > 0.5 V_s + V_p$$

$$V_{hcb} = \frac{V_u}{\phi} - 0.5 V_s - V_p$$

$$If \ \frac{V_u}{\phi} \leq 0.5 \ V_s + V_p$$

$$V_{hcb} = 0.0$$

C5.8.3.6.3

The following shall supplement C5.8.3.6.3.

The signs of M_u, V_u and N_u are defined in DC5.8.3.5.

5.8.4 Interface Shear Transfer-Shear Friction

5.8.4.1 GENERAL

The following shall replace the fourth and sixth paragraphs of A5.8.4.1.

Reinforcement for interface shear between concretes of slab and beams or girders may consist of single bars, multiple leg stirrups or the vertical legs of welded wire fabric. The cross-sectional area, A_{vf}, of the reinforcement shall not be less than either that required by Equation A5.8.4.1-1 or 0.485 mm^2 per mm {0.019 in^2 per in.}.

For beams and girders, the longitudinal spacing of the rows of reinforcing bars for interface shear shall not be greater than 530 mm {21 in.}.

C5.8.4.1

The following shall supplement AC5.8.4.1.

The Department has historically used a minimum interface shear reinforcement based on a minimum reinforcement area per unit length with a maximum longitudinal reinforcement spacing. The requirements of D5.8.4.1 are based on this successful past practice.

5.8.4.2 COHESION AND FRICTION

The following shall replace the second bulleted item in A5.8.4.2.

- for concrete placed against clean, hardened concrete with surface intentionally roughened to an

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amplitude of 2 mm {0.08 in.}

$c = 0.70$ MPa {0.100 ksi}

$\mu = 1.0\lambda$

The following shall supplement A5.8.4.2.

"Intentionally roughened to an amplitude of 2 mm {0.08 in.}" finish shall be specified for the top flange of prestressed beams. Therefore, for prestressed beams c = 0.70 MPa {0.100 ksi}.

5.9 PRESTRESSING AND PARTIAL PRESTRESSING

5.9.1 General Design Considerations

5.9.1.1 GENERAL

The following shall supplement A5.9.1.1.

Drape points are usually at a distance 1/3 of span length measured from the centerline of the bearings. The minimum distance between drape points is 6000 mm {20 ft.}. In box beams, internal diaphragms shall be located at the drape points.

C5.9.1.1

The following shall supplement AC5.9.1.1.

The Department has a computer program which offers many options for the analysis or design of prestressed concrete beams.

Common sense should be used in matching beam sizes with prestressing forces, while it may be possible, for instance, to stress a 430 mm {17 in.} deep box beam with a 4500 kN {1,000 kips} force by increasing f'_c to 55 MPa {8 ksi}, the end zone stresses, including secondary stresses, will most likely be excessive and cause distress, evidenced by end zone cracking.

5.9.1.4 SECTION PROPERTIES

The following shall replace A5.9.1.4.

For section properties prior to bonding of post-tensioning tendons, effects of loss of area due to open ducts shall be considered.

For both pretensioned or post-tensioned members after bonding of tendons, section properties shall be based on:

- gross section for permanent loads
- transformed section for live loads

The transformed section for live loads shall include the prestressing steel, but the mild steel reinforcement is to be neglected.

5.9.3 Stress Limitations for Prestressing Tendons

The following shall replace Table A5.9.3-1.

C5.9.3

The following shall supplement AC5.9.3.

Table 5.9.3-1 - Stress Limits for Prestressing Tendons

	Tendon Type	
	Stress Relieved Strand and Plain High-Strength Bars	Low Relaxation Strand
At Jacking: (f_{pj}) - Pretensioning - Post-tensioning	 $0.70\ f_{pu}$ $0.76\ f_{pu}$	 $0.75\ f_{pu}$ $0.80\ f_{pu}$
After Transfer: (f_{pt}) - Pretensioning - Post-tensioning • At anchorages and couplers immediately after anchor set • Post-tensioning - General	 f_{pj}-Δf_{pES}-Δf_{pR1} $0.70\ f_{pu}$ $0.70\ f_{pu}$	 f_{pj}-Δf_{pES}-Δf_{pR1} $0.70\ f_{pu}$ $0.70\ f_{pu}$
At Service Limit State: (f_{pe}) After Losses	 $\leq 0.80\ f_{pj}$	 $\leq 0.80\ f_{pj}$

The following shall supplement A5.9.3.

For the Service IIIA load combination given in Table D3.4.1.1P-2, the stress in the prestressing steel in the row nearest the extreme tension fiber of the member shall not exceed 90% of the yield stress of prestressing strands.

The upper limit for maximum prestressing force for an as-designed structure is 17 795 kN {4,000 kip} and resulting end moment must be less than 13.6 x 10^9 N·mm {120,000 k·in}. For alternate designs, the prestressing force can be greater than this, but the safe capacity of the Fabricator's prestressing bed must not be exceeded.

This 90% of the yield stress provision was maintained from 5.4.6 of the AASHTO Maintenance Inspection of Bridges Manual (1983, 1990 Interim). Typically moment-curvature methods based on stress-strain compatibility are employed to determine the resistance of the section at this level of stress. Therefore, this calculation is ideally suited for a computer program and is included in PennDOT's LRFD Prestressed Concrete Girder Design and Rating Computer Program (PSLRFD).

The end moment is calculated by multiplication of prestressing force by the distance measured from the bottom of the beam to the center of gravity of the strand pattern.

5.9.4 Stress Limits for Concrete

Delete A5.9.4.1, A5.9.4.1.1, A5.9.4.1.2, A5.9.4.2, A5.9.4.2.1, A5.9.4.2.2 and A5.9.4.3.

C5.9.4

Delete AC5.9.4.2.1 and AC5.9.4.2.2.

5.9.4.4P ALLOWABLE CONCRETE STRESSES FOR PRETENSIONED BEAMS

A summary of allowable stresses under design loads for pretensioned beams is given in Table 1.

For the service load combinations, which involves traffic loading, tension stresses in members with bonded prestressing tendons should be investigated using Load Combination Service III in Table A3.4.1-1.

The tension in the precompressed tensile zone shall not exceed $0.25\sqrt{f'_c}$ {U. S. Customary Units: $0.0948\sqrt{f'_c}$} under any condition.

The flexural stresses at the end of the beams are to be

C5.9.4.4P

As a contractor redesign, the flexural stresses at the end

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checked at the centerline of bearing and 300 mm {12 in.} from the centerline of bearing towards mid-span. For flexural stresses, the prestressing strands shall be assumed to develop linearly between the end of the beam and the point 300 mm {12 in.} from the centerline of bearing.

For box beams, solid section properties shall be used to check the stresses at the centerline of bearing and 300 mm {12 in.} from the centerline of bearing towards mid-span.

The allowable range for f'_c, compressive structural design strength of prestressed beam concrete at 28 days, shall be as follows:

$$1.05\, f'_{ci} \le f'_c \le 1.18\, f'_{ci}$$

where:

f'_{ci} = compressive structural design strength of prestressed beam concrete at transfer (MPa) {ksi}

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of a beam may be control by use of mild steel reinforcement or debonding for crack control.

Past experience has shown that in the design of prestressed beams for PennDOT that, if the concrete strength controls the design, it will usually be f'_{ci} instead of f'_c. Therefore, f'_c is based on the f'_{ci} value. As of 1994, the prestressed industry recommends a maximum of 47 MPa {6.8 ksi} for f'_{ci}.

Table 5.9.4.4P-1 - Summary of Allowable Concrete Stresses for Pretensioned Beams

METRIC UNITS									
TYPE OF STRESS			PRETENSIONED BEAMS						
			I-BEAMS		BOX BEAMS BASED ON HOLLOW SECTIONS		PLANK BEAMS		
			C.L. BRG.	All Other Locations	C.L.BRG.	All Other Locations	C.L. BRG.	All Other Locations
STRAIGHT STRANDS DESIGN	INITIAL STRESS AT TRANSFER OF P/S	TENSION	$0.25\sqrt{f'_{ci}}$	$0.25\sqrt{f'_{ci}}$	$0.25\sqrt{f'_{ci}}$	$0.25\sqrt{f'_{ci}}$	$0.25\sqrt{f'_{ci}}$	$0.25\sqrt{f'_{ci}}$
		COMPRESSION	$0.6\ f'_{ci}$	$0.6\ f'_{ci}$	$0.6\ f'_{ci}$	$0.6\ f'_{ci}$	$0.6\ f'_{ci}$	$0.6\ f'_{ci}$
	FINAL STRESS UNDER DESIGN DL+P/S	COMPRESSION	$0.4\ f'_c$	$0.4\ f'_c$	$0.4\ f'_c$	$0.4\ f'_c$	$0.4\ f'_c$	$0.4\ f'_c$
	FINAL STRESS UNDER DESIGN LOADS	TENSION(1)	N/A	$0.25\sqrt{f'_c}$	N/A	$0.25\sqrt{f'_c}$	N/A	$0.25\sqrt{f'_c}$
		COMPRESSION	$0.6\ f'_c$	$0.6\ f'_c$	$0.6\ f'_c$	$0.6\ f'_c$	$0.6\ f'_c$	$0.6\ f'_c$
	FINAL STRESS: COMPRESSION - LIVE LOAD PLUS ONE-HALF OF (DL+P/S)		$0.4\ f'_c$	$0.4\ f'_c$	$0.4\ f'_c$	$0.4\ f'_c$	$0.4\ f'_c$	$0.4\ f'_c$
DEBONDED* OR DRAPED STRANDS DESIGN	INITIAL STRESS AT TRANSFER OF P/S	TENSION	$0.25\sqrt{f'_{ci}}$	$0.25\sqrt{f'_{ci}}$	$0.25\sqrt{f'_{ci}}$	$0.25\sqrt{f'_{ci}}$		
		COMPRESSION	$0.6\ f'_{ci}$	$0.6\ f'_{ci}$	$0.6\ f'_{ci}$	$0.6\ f'_{ci}$		
	FINAL STRESS UNDER DESIGN DL+P/S	COMPRESSION	$0.4\ f'_c$	$0.4\ f'_c$	$0.4\ f'_c$	$0.4\ f'_c$		
	FINAL STRESS UNDER DESIGN LOADS	TENSION**	N/A	$0.25\sqrt{f'_c}$	N/A	$0.25\sqrt{f'_c}$		
		COMPRESSION	$0.6\ f'_c$	$0.6\ f'_c$	$0.6\ f'_c$	$0.6\ f'_c$		
	FINAL STRESS: COMPRESSION - LIVE LOAD PLUS ONE-HALF OF (DL+P/S)		$0.4\ f'_c$	$0.4\ f'_c$	$0.4\ f'_c$	$0.4\ f'_c$		

*For requirements of debonding in lieu of draping and crack control debonding see A5.11.4.2 and D5.11.4.2.

** The allowable stress for final stress under loads, including creep and shrinkage effect for continuous spans, shall be taken as 0.80 modulus of rupture (see D5.4.2.6)

Table 5.9.4.4P-1 - Summary of Allowable Concrete Stresses for Pretensioned Beams (Continued)

U.S. CUSTOMARY UNITS

TYPE OF STRESS			PRETENSIONED BEAMS					
			I-BEAMS		BOX BEAMS BASED ON HOLLOW SECTIONS		PLANK BEAMS	
			C.L. BRG.	All Other Locations	C.L. BRG.	All Other Locations	C.L. BRG.	All Other Locations
STRAIGHT STRANDS DESIGN	INITIAL STRESS AT TRANSFER OF P/S	TENSION	$0.0948\sqrt{f'_{ci}}$	$00948\sqrt{f'_{ci}}$	$0.0948\sqrt{f'_{ci}}$	$0.0948\sqrt{f'_{ci}}$	$0.0948\sqrt{f'_{ci}}$	$0.0948\sqrt{f'_{ci}}$
		COMPRESSION	$0.6\ f'_{ci}$	$0.6\ f'_{ci}$	$0.6\ f'_{ci}$	$0.6\ f'_{ci}$	$0.6\ f'_{ci}$	$0.6\ f'_{ci}$
	FINAL STRESS UNDER DESIGN DL+P/S	COMPRESSION	$0.4\ f'_c$	$0.4\ f'_c$	$0.4\ f'_c$	$0.4\ f'_c$	$0.4\ f'_c$	$0.4\ f'_c$
	FINAL STRESS UNDER DESIGN LOADS	TENSION(1)	N/A	$0.0948\sqrt{f'_c}$	N/A	$0.0948\sqrt{f'_c}$	N/A	$0.0948\sqrt{f'_c}$
		COMPRESSION	$0.6\ f'_c$	$0.6\ f'_c$	$0.6\ f'_c$	$0.6\ f'_c$	$0.6\ f'_c$	$0.6\ f'_c$
	FINAL STRESS: COMPRESSION - LIVE LOAD PLUS ONE-HALF OF (DL+P/S)		$0.4\ f'_c$	$0.4\ f'_c$	$0.4\ f'_c$	$0.4\ f'_c$	$0.4\ f'_c$	$0.4\ f'_c$
DEBONDED* OR DRAPED STRANDS DESIGN	INITIAL STRESS AT TRANSFER OF P/S	TENSION	$0.0948\sqrt{f'_{ci}}$	$0.0948\sqrt{f'_{ci}}$	$0.0948\sqrt{f'_{ci}}$	$0.0948\sqrt{f'_{ci}}$		
		COMPRESSION	$0.6\ f'_{ci}$	$0.6\ f'_{ci}$	$0.6\ f'_{ci}$	$0.6\ f'_{ci}$		
	FINAL STRESS UNDER DESIGN DL+P/S	COMPRESSION	$0.4\ f'_c$	$0.4\ f'_c$	$0.4\ f'_c$	$0.4\ f'_c$		
	FINAL STRESS UNDER DESIGN LOADS	TENSION**	N/A	$0.0948\sqrt{f'_c}$	N/A	$0.0948\sqrt{f'_c}$		
		COMPRESSION	$0.6\ f'_c$	$0.6\ f'_c$	$0.6\ f'_c$	$0.6\ f'_c$		
	FINAL STRESS: COMPRESSION - LIVE LOAD PLUS ONE-HALF OF (DL+P/S)		$0.4\ f'_c$	$0.4\ f'_c$	$0.4\ f'_c$	$0.4\ f'_c$		

*For requirements of debonding in lieu of draping and crack control debonding see A5.11.4.2 and D5.11.4.2.

** The allowable stress for final stress under loads, including creep and shrinkage effect for continuous spans, shall be taken as 0.80 modulus of rupture (see D5.4.2.6)

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5.9.4.5P ALLOWABLE CONCRETE STRESSES FOR SEGMENTALLY CONSTRUCTED BRIDGES

5.9.4.5.1P For Temporary Stresses Before Losses - Fully Prestressed Components

5.9.4.5.1aP Compressive Stresses

The following limits shall apply:

- in pretensioned components $0.60\ f'_{ci}$ (MPa) {ksi}
- in post-tensioned components $0.55\ f'_{ci}$ (MPa) {ksi}

5.9.4.5.1bP Tensile Stresses

The limits in Table 1 shall apply for tensile stresses.

Table 5.9.4.5.1bP-1 - Temporary Tensile Stress Limits for Segmentally Constructed Bridges Before Losses, Fully Prestressed Components

Location	Stress Limit	
	Metric Units	U.S. Customary Units
Longitudinal Stresses Through Joints in the Precompressed Tensile Zone		
• joints with minimum bonded auxiliary reinforcement through the joints, which is sufficient to carry the calculated tensile force at a stress of $0.5\ f_y$; with internal tendons	$0.25\sqrt{f'_{ci}}$ maximum tension (MPa)	$0.0948\sqrt{f'_c}$ maximum tension (ksi)
• joints without the minimum bonded auxiliary reinforcement through the joints; with internal tendons	0.17 MPa minimum compression	0.025 ksi minimum compression
• joints with external tendons	1.38 MPa minimum compression	0.2 ksi minimum compression
Transverse Stresses Through Joints		
• For any type of joint	$0.25\sqrt{f'_{ci}}$ (MPa)	$0.0948\sqrt{f'_c}$ (ksi)
Stresses in Other Areas		
• For areas without bonded nonprestressed reinforcement	0.17 MPa minimum compression	0.025 ksi minimum compression
• For areas with bonded reinforcement sufficient to carry the calculated tensile force in the concrete computed assuming an uncracked section where the reinforcement is proportioned using a stress of $0.5\ f_y$, not to exceed 205 MPa (30 ksi).	$0.50\sqrt{f'_{ci}}$ (MPa)	$0.19\sqrt{f'_c}$ (ksi)

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5.9.4.5.2P For Stresses at Service Limit State After Losses - Fully Prestressed Components

5.9.4.5.2aP Compressive Stresses

Compression shall be investigated using the Service Limit State Load Combination I specified in Table A3.4.1-1, and shall be limited to 0.45 f'_c (MPa) {ksi}.

5.9.4.5.2bP Tensile Stresses

For the service load contributions, which involves traffic loading, tension stresses in members with bonded prestressing tendons should be investigated using Load Combination Service III in Table A3.4.1-1.

The limits in Table 1 shall apply.

Table 5.9.4.5.2bP-1 - Temporary Tensile Stress Limits for Segmentally Constructed Bridges at Service Limit State After Losses, Fully Prestressed Components

Location	Stress Limit	
	Metric Units	U.S. Customary Units
Longitudinal Stresses Through Joints in the Precompressed Tensile Zone		
• Joints with minimum bonded auxiliary reinforcement through the joints, which is sufficient to carry the calculated tensile force at a stress of 0.5 f_y; with internal tendons	$0.25\sqrt{f'_{ci}}$ (MPa)	$0.0948\sqrt{f'_c}$ (ksi)
• Joints without the minimum bonded auxiliary reinforcement through the joints; with internal tendons	0.17 MPa minimum compression	0.025 ksi minimum compression
• Joints with external tendons	1.38 MPa minimum compression	0.2 ksi minimum compression
Transverse Stresses Through Joints		
• Tension in the transverse direction in precompressed tensile zone	$0.25\sqrt{f'_{ci}}$ (MPa)	$0.0948\sqrt{f'_c}$ (ksi)
Stresses in Other Areas		
• For areas without bonded reinforcement	0.17 MPa minimum compression	0.025 ksi minimum compression
• For areas with bonded reinforcement sufficient to carry the calculated tensile force in the concrete computed assuming an uncracked section where the reinforcement is proportioned using a stress of 0.5 f_y, not to exceed 205 MPa (30 ksi).	$0.50\sqrt{f'_{ci}}$ (MPa)	$0.19\sqrt{f'_c}$ (ksi)

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5.9.4.6P ALLOWABLE CONCRETE STRESSES FOR PRESTRESSED PILES

5.9.4.6.1P Compression Stresses

The temporary compressive stress before losses in prestressed piles shall be less than 0.6 f'_{ci} (MPa) {ksi}.

The final compressive stress in prestressed piles shall be less than 0.4 f'_c (MPa) {ksi}.

C5.9.4.6.1P

For use of prestressed piles, see D5.13.4.4.

5.9.4.6.2P Tensile Stresses

The temporary tensile stress before losses in prestressed piles due to handling loads shall be less than $0.415\sqrt{f'_{ci}}$ (MPa) {U.S. Customary Units: $0.158\sqrt{f'_c}$ (ksi)}.

The final tensile stresses in prestressed piles shall be less than $0.5\sqrt{f'_c}$ (MPa) {U.S. Customary Units: $0.19\sqrt{f'_c}$ (ksi)}.

5.9.4.7P PARTIALLY PRESTRESSED COMPONENTS

Compression stresses shall be limited as specified in D5.9.4.4P and D5.9.4.5P for fully prestressed components.

Cracking in the precompressed tensile zone may be permitted. The design of partially prestressed members should be based on a cracked section analysis with various service limit states being satisfied. Tensile stress in reinforcement at the service limit state shall be as specified in A5.7.3.4, in which case f_{sa} shall be interpreted as the change in stress after decompression.

5.9.4.8P TOLERANCES IN PRESTRESSED ALLOWABLE STRESSES

"Tolerance" in this section means an "allowable stress overrun" to avoid unnecessary recycling of computations. The establishment of tolerances does not mean that the allowable design stresses are increased by the tolerance amount. The tolerance is the maximum value for an incidental stress overrun which may occur during the normal design process.

At the temporary stage before all losses (at detensioning), the stress tolerance is 0.15 MPa {0.025 ksi} in tension and 0.35 MPa {0.050 ksi} in compression. At the final stage after losses, the maximum tolerance is 2.5% of the allowable stress.

5.9.5 Loss of Prestress

5.9.5.0P CALCULATION OF LOSS OF PRESTRESS

For PennDOT projects, the calculation of loss of prestress shall be accomplished by using A5.9.5 and D5.9.5 as appropriate. The time-dependent losses shall be

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calculated using refined estimates given in A5.9.5.4 and D5.9.5.4. The approximate lump sum estimate of time-dependent losses given in A5.9.5.3 shall not be used.

5.9.5.1 TOTAL LOSS OF PRESTRESS

The following shall replace A5.9.5.1.

In lieu of more detailed analysis, prestress losses in members constructed and prestressed in a single stage may be taken as:

- In pretensioned members

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR} \quad (5.9.5.1\text{-}1)$$

- In post-tensioned members

$$\Delta f_{pT} = \Delta f_{pF} + \Delta f_{pA} + \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR} \quad (5.9.5.1\text{-}2)$$

where

Δf_{pT} = total loss (MPa) {ksi}

Δf_{pF} = loss due to friction (MPa) {ksi}

Δf_{pA} = loss due to anchorage set (MPa) {ksi}

Δf_{pES} = loss due to elastic shortening (MPa) {ksi}

Δf_{pSR} = loss due to shrinkage (MPa) {ksi}

Δf_{pCR} = loss due to creep of concrete (MPa) {ksi}

Δf_{pR} = loss due to relaxation of steel (MPa) {ksi}

In pretensioned members, the part of the loss due to relaxation which occurs before transfer may be deducted from the total relaxation.

The total prestress loss shall not be less than 20% of the jacking force.

5.9.5.2 INSTANTANEOUS LOSSES

5.9.5.2.3 Elastic Shortening

5.9.5.2.3a Pretensioned Members

The following shall supplement A5.9.5.2.3a.

The term f_{cgp}, used in Equation A5.9.5.2.3a-1, shall be determined at the mid-span of beam as follows:

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C5.9.5.1

The following shall be added to the end of the first sentence of AC5.9.5.1.

...and approved by the Chief Bridge Engineer.

C5.9.5.2.3aP

In determining f_{cgp}, the LRFD Specification allows the use of an approximate value of the stress in the strands immediately after transfer, f_{pi}, of 0.70 f_{pu}. A more rigorous calculation of f_{pi}, however, can be made by taking the jacking stress, f_{pj}, and subtracting the initial losses, Δf_{pES} and Δf_{pR1}. However, f_{pi} is a function of Δf_{pES} which

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$$f_{cgp} = \frac{\frac{A_{ps}}{A_{beam}} f_{s2}\left(1 + \frac{e^2 A_{beam}}{I_g}\right) - \frac{M_{DLb}\, e}{I_g}}{1 + \frac{A_{ps}}{A_{beam}}\left(\frac{E_p}{E_{ci}}\right)\left(1 + \frac{e^2 A_{beam}}{I_g}\right)}$$ (5.9.5.2.3a-2P)

for which:

$f_{s2} = f_{pj} - \Delta f_{pR1}$ (5.9.5.2.3a-3P)

where:

A_{beam} = basic beam cross-sectional area (mm^2) {in^2}

A_{ps} = area of prestressing strand (mm^2) {in^2}

E_{ci} = modulus of elasticity of concrete at transfer (MPa) {ksi}

E_p = modulus of elasticity of prestressing strand (MPa) {ksi}

e = the eccentricity of prestressing strand to the centroid of beam (mm) {in}

f_{pj} = stress in the prestressing steel at jacking (MPa) {ksi}

Δf_{pR1} = loss in prestressing steel stress due to relaxation of steel at transfer (MPa) {ksi}

I_g = moment of inertia of basic beam (mm^4) {in^4}

M_{DLb} = moment due to dead load of beam including interior diaphragms (N·mm) {k·in}

5.9.5.2.4P Prestress Stress at Transfer

The prestress stress immediately following transfer shall be taken as:

$f_{pt} = f_{pj} - \Delta f_{pES} - \Delta f_{pR1}$

where:

fpj = stress in the prestressing steel at jacking (MPa) {ksi}

ΔfpR1 = loss in prestressing steel stress due to relaxation of steel at transfer (MPa) {ksi}

ΔfpES = loss due to elastic shortening (MPa) {ksi}

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contains f_{cgp}. Thus, Equation 2 was developed as a closed form solution for f_{cgp} based on the following three equations.

$$f_{cgp} = \frac{f_{pi} A_{ps}}{A_{beam}} + \frac{f_{pi} A_{ps} e^2}{I_g} - \frac{M_{DLb} e}{I_g}$$ (C5.9.5.2.3a-1P)

where:

$$f_{pi} = f_{pj} - \Delta f_{pR1} - \Delta f_{pES} = f_{s2} - \Delta f_{pES}$$ (C5.9.5.2.3a-2P)

for which:

$$\Delta f_{pES} = \frac{E_p}{E_{ci}} f_{cgp}$$ (C5.9.5.2.3a-3P)

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5.9.5.3 APPROXIMATE LUMP SUM ESTIMATE OF TIME-DEPENDENT LOSSES

Delete A5.9.5.3.

5.9.5.4 REFINED ESTIMATES OF TIME-DEPENDENT LOSSES

5.9.5.4.1 General

The following shall replace the first sentence and bullet list of A5.9.5.4.1.

For non-segmental prestressed members, values of creep-, shrinkage-, and relaxation-related losses may be determined in accordance with the provisions of either A5.4.2.3 or this article for prestressed concrete members with:

- Spans not greater than 75 000 mm {250 ft.},
- Normal density Concrete, and
- Strength in excess of 24 MPa {3.5 ksi} at the time of prestress.

For segmental construction and post-tensioned spliced precast girders, other than during preliminary design, prestress losses shall be determined by the time-step method and the provisions of 5.9.5, including consideration of the time-dependant construction stages and schedule shown in the contract documents. For components with combined pretensioning and post-tensioning, and where post-tensioning is applied in more than one stage, the effects of subsequent prestressing on the creep loss due to previous prestressing shall be considered.

5.9.5.4.3 Creep

The following shall supplement A5.9.5.4.3.

The term Δf_{cdp}, used in Equation A5.9.5.4.3-1, shall be determined at the midspan of beam as follows:

$$\Delta f_{cdp} = \frac{(M_{deck} + M_{ext\ dia})e}{I_g} + \frac{M_{SDL}\ e_c}{I_c} \quad (5.9.5.4.3\text{-}2P)$$

for which:

$$e_c = y_{comp} - y_{bot} + e \quad (5.9.5.4.3\text{-}3P)$$

where:

e = the eccentricity of prestressing strand to the

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centroid of beam (mm) {in}

I_c = moment of inertia of composite beam (mm^4) {in^4}

I_g = gross moment of inertia of the nontransformed basic beam (mm^4) {in^4}

M_{deck} = moment due to deck (N·mm) {k·in}

$M_{ext\ dia}$ = moment due to external diaphragms (N·mm) {k·in}

M_{SDL} = moment due to superimposed dead load (N·mm) {k·in}

y_{comp} = distance from composite neutral axis to the bottom of the beam (mm) {in.}

y_{bot} = distance from noncomposite neutral axis to the bottom of the beam (mm) {in.}

5.9.5.4.4 Relaxation

5.9.5.4.4b At Transfer

The following shall replace A5.9.5.4.4b.

In pretensioned members, the relaxation loss in prestressing steel, initially stressed in excess of 0.50 f_{pu}, may be taken as:

- for stress-relieved strand:

$$\Delta f_{pR1} = 0.0375\ f_{pj} \quad (5.9.5.4.4b\text{-}1)$$

- for low-relaxation strand:

$$\Delta f_{pR1} = 0.025\ f_{pj} \quad (5.9.5.4.4b\text{-}2)$$

C5.9.5.4.4b

Delete AC5.9.5.4.4b

5.9.5.4.4c After Transfer

The following shall replace the last two sentences of A5.9.5.4.4c.

Loss due to relaxation should be based on test data which has been approved by the Chief Bridge Engineer. If test data is not available, the loss shall be assumed to be 21 MPa {3.0 ksi}.

5.10 DETAILS OF REINFORCEMENT

5.10.2 Hooks and Bends

5.10.2.1 STANDARD HOOKS

The following shall supplement A5.10.2.1.

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Reinforcement bars shall not be provided with hooks unless required by design or as detailed in the Standard Drawings; however, dowels anchored into footings of substructures that carry primary stresses shall be provided with hooks.

5.10.2.3 MINIMUM BEND DIAMETERS

The following shall replace A5.10.2.3. Refer to Standard Drawing BC-736M for minimum bend diameter.

5.10.3 Spacing of Reinforcement

5.10.3.1 MINIMUM SPACING OF REINFORCEMENT

5.10.3.1.1 Cast-in-Place Concrete

The following shall replace the third bulleted item in A5.10.3.1.1.

- 60 mm {2 1/2 in.}

5.10.3.1.2 Precast Concrete

The following shall replace the third bulleted item in A5.10.3.1.2.

- 40 mm {1 1/2 in.}

5.10.3.1.3 Multi-Layers

The following shall replace A5.10.3.1.3.

Except in decks, where parallel reinforcing is placed in two or more layers, with clear distance between layers not exceeding 150 mm {6 in.}, the bars in the upper layers shall be placed directly above those in the bottom layer, and the clear distance between layers shall not be less than either 40 mm {1 1/2 in.} or the nominal diameter of the bars.

5.10.3.1.5 Bundled Bars

The following shall supplement A5.10.3.1.5.

Bundled bars shall be tied, wired, or otherwise fastened together to ensure that they remain in their relative position, regardless of their inclination.

COMMENTARY

C5.10.3.1.5

Delete AC5.10.3.1.5.

5.10.3.2 MAXIMUM SPACING OF REINFORCING BARS

The following shall supplement A5.10.3.2.

For box culverts, retaining walls, abutments, wall-type piers and similar structures, the spacing of the reinforcement shall not exceed either 1.5 times the structural thickness of the member or 600 mm {24 in.}.

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5.10.3.3 MINIMUM SPACING OF PRESTRESSING TENDONS AND DUCTS

5.10.3.3.1 Pretensioning Strand

The following shall replace A5.10.3.3.1.

The minimum clear spacing between pretensioning strands shall be the larger of:

- center-to-center spacing of 50.8 mm {2 in.} or
- clear distance of 2 times the maximum size of aggregate.

The clear distance between strands at the end of a member may be decreased, if justified by performance tests of full-scale prototypes of the design and approved by the Chief Bridge Engineer.

The minimum clear distance between groups of bundled strands shall not be less than either 2 times the maximum size of the aggregate or 50.8 mm {2 in.}.

Pretensioning strands may be bundled to touch one another in an essentially vertical plane at, and between, hold-down devices, provided that the spacing, specified herein, is maintained between individual strands near the ends of the beams for a distance not less than the maximum shielded length plus development length.

Groups of eight strands of 13.2 mm {0.52 in.} diameter or smaller may be bundled linearly to touch one another in a vertical plane at and between hold-down devices. The number of strands bundled in any other manner shall not exceed four.

C5.10.3.3.1

The following shall replace AC5.10.3.3.1.

For 13.2 mm {0.52 in.} diameter strands, a clear distance of 37.6 mm {1.48 in.} should be considered sufficient to satisfy this requirement.

When required or permitted by the Chief Bridge Engineer, groups of more than eight strands of 13.2 mm {0.52 in.} diameter or smaller may be bundled to touch one another in a vertical plane.

5.10.3.3.2 Post-Tensioning Ducts Not Curved in the Horizontal Plane

The following shall replace A5.10.3.3.2.

Unless otherwise specified herein, the clear distance between straight post-tensioning ducts shall not be less than 50 mm {2 in.} or 2 times the maximum size of the coarse aggregate.

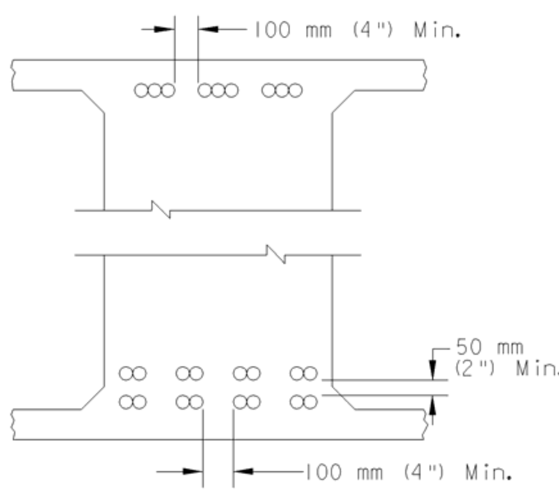
For precast segmental construction when post-tensioning tendons extend through an epoxy joint between components, the clear spacing between post-tensioning ducts shall not be less than the greater of the duct internal diameter or 100 mm (4.0 in.)

Ducts may be bundled together in groups not exceeding three, provided that the spacing, as specified between individual ducts, is maintained between each duct in the zone within 900 mm {3 ft.}of anchorages.

For groups of bundled ducts in construction other than segmental, the minimum clear horizontal distance between adjacent bundles shall not be less than 100 mm {4 in.}. For groups of ducts where the ducts are located in two or more horizontal planes, a bundle shall contain no more than two

C5.10.3.3.2

The following shall replace AC5.10.3.3.2.



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ducts in the same horizontal plane.

The minimum vertical clear distance between bundles shall not be less than 50 mm {2 in.} or 2 times the maximum size of coarse aggregate.

For precast construction, the minimum clear horizontal distance between groups of ducts may be reduced to 75 mm {3 in.}.

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Figure C5.10.3.3.2-1 - Examples of Acceptable Arrangements for Not Curved in the Horizontal Plane Ducts

5.10.6 Transverse Reinforcement for Compression Members

5.10.6.1 GENERAL

The following shall supplement A5.10.6.1.

Nominal spiral reinforcement in place of tie reinforcement of No. 13 bars {No. 4 bars} at approximately 300 mm {12 in.} pitch, but not less than the size and spacing shown on the drawings, may be substituted by the contractor at no additional cost to the Department.

C5.10.6.1

The following shall replace AC5.10.6.1.

A5.10.11.2 applies to Seismic Zone 1, but has no additional requirements for transverse reinforcement for compression members.

5.10.6.2 SPIRALS

The following shall replace the fifth paragraph of 5.10.6.2.

Splices in spiral reinforcement shall be designed as follows:

The total spiral length may be divided into separate segments. Each segment shall be provided with 1 ½ extra turns and 135 degree hook at each end. Each hook shall engage a primary reinforcing bar. The maximum distance between spiral segments is limited to the pitch of the spiral.

C5.10.6.2P

Allowing the spiral to be divided into segments provides easier constructability and allows the use of hooks, which are required for seismic detailing, at the ends of the spirals. Spirals can not be spliced with hooks the conventional way, since the hooks will not allow the spliced segments to be threaded together.

5.10.6.3 TIES

The following shall replace the third paragraph following the bullet list of A5.10.6.3.

Lateral support for longitudinal bars shall be provided by the corner of a tie having an included angle of not more than 135°. Ties shall be arranged so that:

All longitudinal bars at the corners of a column shall be laterally supported by conforming ties and;

The center-to-center distance between any longitudinal bar to the nearest laterally supported bars on either side along the tie shall not exceed 600 mm (24.0 in.)

Where the column design is based on plastic hinging capability, no longitudinal bar shall be farther than 150 mm (6.0 in.) clear on each side along the tie from such a laterally supported bar. Where the bars are located around the periphery of a circle, a complete circular tie may be used if the splices in the ties are staggered.

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5.10.8 Shrinkage and Temperature, and Minimum Reinforcement

5.10.8.1 GENERAL

The following shall supplement A5.10.8.1.

For components which support load, the minimum reinforcement must be the greater of the shrinkage and temperature reinforcement requirements in D5.10.8.2, A5.10.8.2 and D5.10.8.3, or the minimum reinforcement requirements in D5.10.8.4P.

For concrete members which have a sacrificial wearing surface, the structural thickness shall be used to determine whither D5.10.8.2 or D5.10.8.3 applies. However, when using the minimum area equations within A5.10.8.2 and D5.10.8.2, the total thickness of the member shall be used.

5.10.8.2 COMPONENTS LESS THAN OR EQUAL TO 1200 MM {48 IN.} THICK

The following shall supplement A5.10.8.2.

The provisions of A5.10.8.2 shall apply only to components less than 450 mm {18 in.} thick.

The following shall replace the fourth paragraph of A5.10.8.2.

Shrinkage and temperature reinforcement shall not be spaced farther apart than the lesser of 1.5 times the component's structural thickness or 450 mm {18 in.}.

The following shall replace the last paragraph of A5.10.8.2.

For solid structural concrete walls and footings, the area of shrinkage and temperature steel need not exceed:

$$\Sigma A_b = 0.0015\, A_g \qquad (5.10.8.2\text{-}2)$$

5.10.8.3 MASS CONCRETE

The following shall replace A5.10.8.3.

For structural concrete components whose least dimension equals or exceeds 450 mm {18 in.}, reinforcement for shrinkage and temperature stresses shall be provided near exposed surfaces of walls and slabs not otherwise reinforced. The total area of reinforcement provided shall be at least 0.265 mm^2 per mm {0.010 in^2 per in.} in each direction at a maximum spacing of 450 mm {18 in.}.

Where prestressing tendons are used as shrinkage and temperature reinforcement, the relevant provisions of A5.10.8.2 shall apply.

5.10.8.4P MINIMUM REINFORCEMENT

Any member subject to loading or stress shall have minimum steel reinforcement of No. 13 bars at 300 mm {No. 4 bars at 12 in.} or No. 16 bars at 450 mm {No. 5 bars at 18 in.}. For ties in reinforced concrete I-posts for noise

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C5.10.8.2

Delete the first paragraph of AC5.10.8.2.

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walls, the minimum steel reinforcing shall be No. 10 bars at 225 mm {No. 3 bars at 9 in.}. Any exception to these criteria must be approved by the Chief Bridge Engineer.

5.10.9 Post-Tensioned Anchorage Zones

5.10.9.2 GENERAL ZONE AND LOCAL ZONE

5.10.9.2.3 Local Zone

The following shall replace the first paragraph of A5.10.9.2.3.

Design of local zones shall either comply with requirements of A5.10.9.7 and D5.10.9.7 or based on the results of acceptance tests as specified in D5.10.9.7.3 and Publication 408.

5.10.9.2.4 Responsibilities

Delete the last sentence of the first paragraph of A5.10.9.2.4.

The following shall replace the second and third paragraphs of A5.10.9.2.4.

The anchorage device supplier shall be responsible for furnishing anchorage devices which satisfy the requirements of Publication 408. If special anchorage devices are used, the anchorage device supplier shall be responsible for furnishing anchorage devices that also satisfy the acceptance test requirements of D5.10.9.7.3.

5.10.9.3 DESIGN OF THE GENERAL ZONE

5.10.9.3.3 Special Anchorage Devices

The following shall replace A5.10.9.3.3.

The provisions of D5.10.9.7.3 apply.

5.10.9.4 APPLICATION OF THE STRUT-AND-TIE MODEL TO THE DESIGN OF GENERAL ZONE

5.10.9.4.2 Nodes

The following shall replace the first sentence of A5.10.9.4.2.

Local zones which satisfy the requirements of A5.10.9.7 and D5.10.9.7 may be considered as properly detailed and are adequate nodes.

5.10.9.7 DESIGN OF LOCAL ZONES

COMMENTARY

C5.10.9.4.2

The following shall be added to the end of the last sentence of the first paragraph of AC5.10.9.4.2.

...and approved by Chief Bridge Engineer.

C5.10.9.7.2 Bearing Resistance

The following shall replace the first paragraph of C5.10.9.7.2.

These specifications provide bearing pressure limits for anchorage devices, called normal anchorage devices.

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Alternatively, these limits may be exceeded if an anchorage system passes the acceptance test and is approved by the Chief Bridge Engineer.

5.10.9.7.3 Special Anchorage Devices

The following shall replace A5.10.9.7.3.

Special anchorage devices that do not satisfy the requirements, specified in A5.10.9.7.2, may be used, provided that they have been tested by an independent testing agency acceptable to the Chief Bridge Engineer, meet the requirements of Publication 408, Section 1108, and are approved by the Chief Bridge Engineer.

C5.10.9.7.3

The following shall replace the last sentence of the first paragraph in AC5.10.9.7.3.

The results of these tests may be considered acceptable, if the test procedure is generally similar to that specified in AASHTO Guide Specifications for the Design and Construction of Segmental Concrete Bridges, and approved by the Chief Bridge Engineer.

5.10.10 Pretensioned Anchorage Zones

5.10.10.1 FACTORED BURSTING RESISTANCE

The following shall replace A5.10.10.1. The bursting resistance of pretensioned anchorage zones provided by vertical reinforcement in the ends of pretensioned beams at the service limit state shall be taken as:

$P_r = f_s A_s$

where:

f_s = stress in steel not exceeding 165 MPa {24 ksi}

A_s = total area of vertical reinforcement placed near the ends of the beam at maximum spacing of 75 mm {3 in.}

End blocks shall be investigated to help in reducing bursting stresses for prestressed beams or pier caps with forces in excess of 8000 kN {1800 kips}. Closely spaced grids for members with forces in excess of 8000 kN {1800 kips} shall have the grid anchored. The reinforcement for the end blocks shall be shown on the shop drawings and shall be in accordance with recommendations of the anchorage fabricator.

5.10.10.1

The following shall replace AC5.10.10.1

The Department allows crack control debonding as specified in Pub. 408 and had successfully controlled end zone cracking using these provisions.

5.10.10.2 CONFINEMENT REINFORCEMENT

The following shall supplement A5.10.10.2.

For prestressed beams, additional confinement reinforcement shall extend from each end of the beam for 1/3 of the span length.

The additional confinement reinforcement shall not be less than No. 13 {No. 4} deformed bars and match with vertical stirrups with maximum spacing of 530 mm {21 in.}.

C5.10.10.2

The debonding length for beams without debonded design is equal to the maximum crack controlled debonded length as specified Publication 408, Section 1107.01.

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5.10.11 Provisions for Seismic Design

5.10.11.1 GENERAL

The following shall supplement A5.10.11.1.

Slenderness effects (A5.7.4.3) shall be considered in the design of columns in Seismic Zone 2.

See standard drawing BD-629M

C5.10.11.1

The following shall supplement AC5.10.11.1.

Structures utilizing seismic details are under construction and feedback has been received regarding details which have been difficult to construct. Examples of problematic details include:

- Designers must consider that the reinforcement cage for bent caps is constructed on the ground and placed in one piece. It is exceedingly difficult to erect the cage when the bottom longitudinal bars (of the cap) interfere with the column spiral or tie reinforcement. Whenever possible, pier caps should be of sufficient depth that column reinforcement extending into the cap can be fully developed without hooks.

- Some designs are using tie bars with 180º hooks on both ends of the bar. Please note Figure AC5.10.11.4.1d-4 which allows for a lap splice of the ties or a tie bare with a 90º hook on one end.

5.10.11.2 SEISMIC ZONE 1

The following shall supplement A5.10.11.2.

The reinforcement requirements shall be as specified in D5.10.11.4.1f and D5.10.11.4.5P.

The spacing of the transverse reinforcement over the length of the column shall be as per A5.10.6.

5.10.11.3 SEISMIC ZONE 2

The following shall replace A5.10.11.3.

The reinforcement requirements shall be as specified in Sections A5.10.11.4.1d, A.5.10.11.4.1e, D5.10.11.4.1f, A5.10.11.4.2, and D5.10.11.4.5P.

The spacing of the transverse reinforcement over the length of the splice shall not exceed 150 mm {6 in.} or one-quarter of the minimum member dimension.

5.10.11.4 SEISMIC ZONES 3 AND 4

5.10.11.4.1 COLUMN REQUIREMENTS

C5.10.11.4.1c Column Shear and Transverse Reinforcement

The following shall supplement AC5.10.11.4.1c and is commentary regarding the second bulleted item of A5.10.11.4.1c.

A5.10.11.4.1 provides provisions on what is considered a pier and a column in regards to seismic design. In most

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5.10.11.4.1d Transverse Reinforcement for Confinement at Plastic Hinges

The following shall replace the second paragraph of A5.10.11.4.1d

For a circular column, the volumetric ratio of spiral or circular hoop reinforcement, ρ_s, shall not be less than either that required in A5.7.4.6 or:

$$\rho_s = 0.12 \frac{f_{c'}}{f_y} \quad (5.10.11.4.1d\text{-}1)$$

where:

f'_c = specified compressive strength of concrete at 28 days, unless another age is specified (MPa) {(KSI)}

f_y = yield strength of reinforcing bars (MPa) {(KSI)}

The following shall supplement A5.10.11.4.1d.

If spirals are used, a 135 degree hook shall be used at each end of the spiral.

5.10.11.4.1e Spacing of Transverse Reinforcement for Confinement

The following shall replace the last bullet of A5.10.11.4.1e.

- spaced not to exceed one-quarter of the minimum member dimension, 6.0 times the diameter of the longitudinal reinforcement or 150 mm {6 in.} center-to-center.

5.10.11.4.1f Splices

The following shall replace A5.10.11.4.1f.

The provisions of A5.11.5 shall apply for the design of splices.

Lap splices in longitudinal reinforcement for columns and wall piers with shaft height less than or equal to 15m {50 ft.} shall be used only within the center half of the shaft height. For columns and wall piers greater than 15m {50 ft.}, lap splices are not permitted within a distance not less than the smaller of 1/4 of the column shaft height and 6m {20 ft.} at the top and bottom of the column shaft. The splice length shall not be less than 400 mm {16 in.} or 60 bar diameters.

Full-mechanical connection splices conforming to

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cases, wall type piers will qualify as "piers" for which A5.10.11.4.2 will apply.

C5.10.11.4.1d

The following shall supplement AC5.10.11.4.1d.

This specification ensures sufficient ductility be available in order to utilize the response modification factors. If proper detailing is not provided, ductility is considered inadequate and the response modification factors are not applicable.

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A5.11.5 may be used, provided that not more than alternate bars in each layer of longitudinal reinforcement are spliced at a section, and the distance between splices of adjacent bars is greater than 600 mm {24 in.} measured along the longitudinal axis of the column.

For columns and wall piers, bars which extend 7600 mm {25 ft} or less from the top of footing should not be spliced.

Splices shall be staggered such that no more than 50% of the bars are spliced at one location.

5.10.11.4.2 Requirements for Wall-Type Piers

The following shall supplement A5.10.11.4.2.

The requirements of this section may be waived when the shear strength of the pier concrete is able to carry the seismic forces unadjusted by the response modification factors. If so, the horizontal bars and ties in the plastic hinge zone shall be No. 13 bars at 300 mm {No. 4 bars at 12 in.} with a maximum vertical spacing of 150 mm {6 in.}.

If response modification factors are used this section is applicable.

5.10.11.4.5P Footing Requirements

The top and bottom mats of reinforcement shall be tied together by a grid of No. 13 {No. 4} tie bars at a maximum spacing of 1200mm {48 in.} in both directions. Ties shall have a 135 degree hook on one end and a 90 degree hook on the opposite end, alternately placed. See Figure DC5.10.11.4.5P-1.

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C5.10.11.4.5P

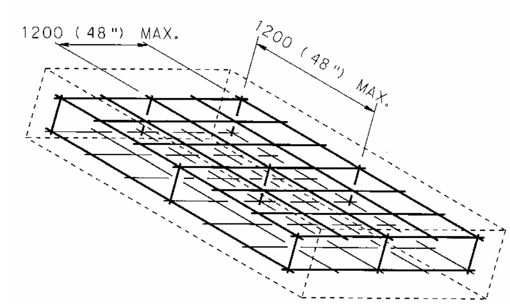

Figure C5.10.11.4.5P-1 Footing Ties

5.11 DEVELOPMENT AND SPLICES OF REINFORCEMENT

5.11.1 General

5.11.1.2 FLEXURAL REINFORCEMENT

5.11.1.2.1 General

The following shall supplement A5.11.1.2.1.

For pier caps of hammerhead piers, the negative moment reinforcement shall be extended for the full-length of the cap.

For pile-supported footings, hooks shall be provided at the ends of the bottom flexural reinforcement mat.

C5.11.1.2.1

The following shall supplement AC5.11.1.2.1

Hooks will provide supplementary anchorage.

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5.11.2 Development of Reinforcement

5.11.2.1 DEFORMED BARS AND DEFORMED WIRES IN TENSION

5.11.2.1.1 Tension Development Length

The following shall supplement A5.11.2.1.1.

Some engineers are misinterpreting A5.11.2.1.3 by assuming that the modification factors of A5.11.2.1.2 cannot be applied if the basic development length is not modified by A5.11.2.1.3.

All applicable modification factors of A5.11.2.1.2 shall be applied to the basic development length, regardless of whether the factors of A5.11.2.1.3 have been applied or not.

5.11.2.4.3 Hooked Bar Tie Requirements

The following shall replace the text in A5.11.2.4.3. (Figure A1 is acceptable and does not need to be replaced.)

For bars being developed by a standard hook at discontinuous ends of members with both side cover and top or bottom cover less than 64 mm {2 1/2 in.}, hooked bar shall be enclosed within ties or stirrups spaced, not greater than 3 d_b, along the full development length, ℓ_{dh}, as shown in Figure A1. In determining the ℓ_{dh} for use in Figure 1, the third bulleted factor in A5.11.2.4.2 shall not apply.

5.11.3 Development by Mechanical Anchorages

The following shall be added to the end of the first paragraph of A5.11.3.

Mechanical anchorages shall be approved by the Chief Bridge Engineer before installation.

5.11.4 Development of Prestressing Strand

5.11.4.1 GENERAL

The following shall replace the second to last paragraph of A5.11.4.1.

The transfer length, ℓ_t, shall be taken as:

Metric Units: $\ell_t = 0.048 f_{pt} d_b$ (5.11.4.1-1)

U.S. Customary Units: $\ell_t = \frac{1}{3} f_{pt} d_b$

where:

d_b = diameter of the strand (mm) {in.}

f_{pt} = stress in prestressing steel immediately after transfer (MPa) {ksi}

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The development length shall be taken as specified in D5.11.4.1.

5.11.4.2 BONDED STRAND

The following shall replace A5.11.4.2.

A pretensioning strand shall be bonded beyond the critical section for no less than the development length taken as:

Metric Units:

$$\ell_d = \ell_t + 0.15\ell_d^1\left(f_{ps} - f_{pe}\right)d_b \qquad (5.11.4.2\text{-}1P)$$

U.S. Customary Units:

$$\ell_d = \ell_t + \ell_d^1\left(f_{ps} - f_{pe}\right)d_b$$

for which:

$$\ell_d^1 = 2.3 - \frac{3.6\omega_p}{\beta_1};\ 1.0 \le \ell_d^1 \le 2.0 \qquad (5.11.4.2\text{-}2P)$$

$$\omega_p = \frac{0.85\beta_1 c}{d_p} \qquad (5.11.4.2\text{-}3P)$$

where:

c = depth of ultimate strength compression block specified in D5.7.3.1.1 (mm) {in.}

β_1 = stress block factor specified in A5.7.2.2

d_b = diameter of strand (mm) {in.}

d_p = distance from extreme compression fiber to the centroid of the prestressing tendons (mm) {in.}

f_{pe} = effective stress in the prestressing steel after losses (MPa) {ksi}

f_{ps} = average stress in prestressing steel at nominal strength (MPa) {ksi}

ℓ_t = transfer length specified in D5.11.4.1 (mm) {in.}

The relationship between stress in a prestressing strand and the distance along the development length shall be taken as shown in Figure 1P.

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C5.11.4.2

Delete AC5.11.4.2.

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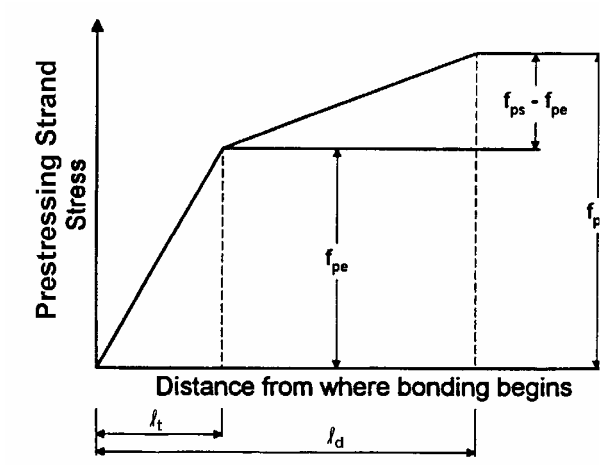

Figure 5.11.4.2-1P - Development of Prestress Strand

5.11.4.3 PARTIALLY DEBONDED STRANDS

The following shall replace A5.11.4.3.

For debonded strands, the development length calculated in D5.11.4.2 shall begin at a point where debonding stops and the bonding begins (i.e., not at the end of strand).

The length of debonding of any strand shall be such that all limit states are satisfied, with consideration of the total development resistance at any section being investigated.

The number of partially debonded strands in lieu of draping shall not be greater than 25% of the total number of strands.

Partially debonded strands in lieu of draping is not allowed in beams with draped strands.

The maximum number of debonded strands in a row shall not exceed 50%. The number of debonded strands may be rounded to the next higher number in case of an odd number of strands in a row.

Debonded strands are permitted in the bottom row. Debonding the exterior strands of any row in the bottom flange shall not be permitted.

When several strands are debonded in lieu of draping, it may be necessary to have more than one cut-off point, but the number of cut-off points shall be limited to a maximum of six. The design shall provide for a 300 mm {12 in.} minimum distance between each cut-off length.

The number of debonded strands at a cut-off section shall be limited to a maximum of six strands.

Select debonded strand pattern uniformly to avoid stress concentrations.

The cutoff pattern shall provide for an increase in eccentricity at each cut-off.

Debonding of adjacent strand in the same row and/or column shall be avoided. In the webs of box beams, debonded strands shall not occur in consecutive rows. In the web of I-beams, do not debond two strands in

C5.11.4.3

The following shall replace AC5.11.4.3.

The limit of six cut-off locations is to accommodate a maximum 25% partially debonded strands. This limit does not include crack control debonding.

SPECIFICATIONS

consecutive rows. Shop drawings must be checked to comply with this requirement.

5.11.4.3.1P Crack Control Debonding

Debonding is also used as a means of controlling detensioning cracks at the ends of beams. This secondary use of debonding is generally referred to as crack control debonding. The actual practice of crack control debonding is given in Publication 408, Section 1107.01.

The percentage of partially debonded strands in lieu of draping, plus the percentage of crack control debonding, shall not exceed 50%.

5.11.5 Splices of Bar Reinforcement

5.11.5.2 GENERAL REQUIREMENTS

COMMENTARY

C5.11.5.2.2 Mechanical Connections

The following shall replace the last sentence of the second paragraph of AC5.11.5.2.2.

Only Department pre-approved mechanical connectors shall be permitted.

5.12 DURABILITY

5.12.3 Concrete Cover

The following shall replace A5.12.3.

The following minimum concrete cover shall be provided for reinforcement:

- Concrete cast against and permanently exposed to earth .... 100 mm {4 in.}
- Concrete exposed to earth........................ 75 mm {3 in.} (Except 50 mm {2 in.} minimum may be used for the stem steel of the safety wings and walls supporting barriers as shown in the bridge standards.)
- Concrete exposed to weather and pier columns .... 75 mm {3 in.}
- Concrete deck slab
  - Top reinforcement .... 60 mm {2 1/2 in.}
  - Bottom reinforcement....................... 25 mm {1 in.}
- Concrete not exposed to weather or in contact ... with ground
  - Primary reinforcement................ 40 mm {1 1/2 in.}
  - Stirrup, tie and spiral .... 25 mm {1 in.}
- Precast concrete pipes................... see A12.10.4.2.4e
- Prestressed concrete
  - Box beams ....see BD-661M
  - I-beams ....see BD-662M
- Reinforced concrete box culverts, cast-in-place
  - Top slab:
    - top bars at grade................. 60 mm {2 1/2 in.}
  - all others .... 50 mm {2 in.}

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Bottom slab:
top bars .... 60 mm {2 1/2 in.}
bottom bars .... 75 mm {3 in.}
Walls.. 50 mm {2 in.}

- Reinforced concrete box culverts, precast
  Top slab:
  top bars at grade................ 60 mm {2 1/2 in.}
  bottom bars........................ 40 mm {1 1/2 in.}
  all others .... 50 mm {2 in.}
  Bottom slab:
  top bars .... 50 mm {2 in.}
  bottom bars........................ 40 mm {1 1/2 in.}
  Walls.. 40 mm {1 1/2 in.}
- Reinforced concrete box culverts, precast with welded deformed wire fabric .... see A12.11.4.4

5.13 SPECIFIC MEMBERS

5.13.2 Diaphragms, Deep Beams, Brackets, Corbels and Beam Ledges

5.13.2.2 DIAPHRAGMS

The following shall replace the fourth paragraph of A5.13.2.2.

For prestressed I-beams and box beams, diaphragm requirements are given on BD-651M.

For segmental box girder bridges, location and design of diaphragm shall be approved by the Chief Bridge Engineer.

5.13.3 Footings

COMMENTARY

C5.13.3.4 MOMENT IN FOOTINGS

Supplement AC 5.13.3.4 as follows:

For the PAPIER program, the design moment will be determined at the 3/4 point of the pressure (for spread footings) or pile load (for pile foundation), to account for the effects of unsymmetrical loading.

C5.13.3.5P DISTRIBUTION OF MOMENT REINFORCEMENT

Figure C1 provides a graphical representation of how the reinforcement in the long and short direction should be applied.

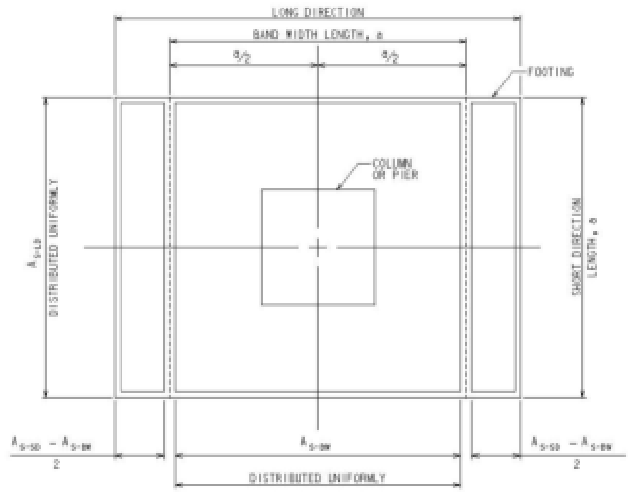


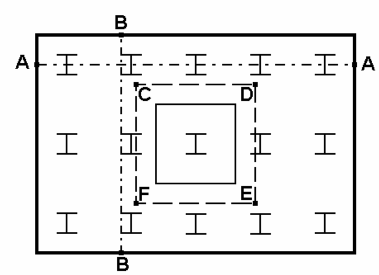
Figure C5.13.3.5P-1 - Example of Distribution of Reinforcement

5.13.3.6 SHEAR IN SLABS AND FOOTINGS

C5.13.3.6.1 Critical Sections for Shear

The following shall supplement AC5.13.3.6.1 and is commentary regarding the last paragraph of A5.13.3.6.1.

Figure C1 provides an example of how to proportion the load from a pile when the pile is intersected by shear section line.



ASSUME ONE FLANGE AREA EQUALS WEB AREA
ASSUME ALL PILES HAVE EQUAL LOAD

SHEAR DESIGN OF THE FOOTING PILE LOADS	
DESIGN SECTION	LOAD
A-A	5 PILES (.5) = 2.5 PILES
B-B	3 PILES
C-D-E-F	12 PILES + 2 ($\frac{5}{6}$) PILES = 13.67 PILES

Figure C5.13.3.6.1-1 - Example of Portioning Pile Loads for Shear Design

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5.13.3.6.3 Two-Way Action

Replace Equation A5.13.3.6.3-3 with:

Metric Units:

$$V_c = 0.166\sqrt{f_c'}\, b_o\, d_v \qquad (5.13.3.6.3\text{-}3)$$

U. S. Customary Units:

$$V_c = 0.0632\sqrt{f_c'}\, b_o\, d_v \qquad (5.13.3.6.3\text{-}3)$$

5.13.3.8 TRANSFER OF FORCE AT BASE OF COLUMN

5.13.3.8.1P Unreinforced Concrete Footings

5.13.3.8.1aP Design Stress

Design stresses in plain concrete footings or pedestals shall be computed assuming a linear stress distribution. For footings and pedestals cast against soil, effective thickness used in computing stresses shall be taken as the overall thickness minus 75 mm {3 in.}. Bending need not be considered, unless projection of footing from face to support member exceeds the footing effective thickness.

5.13.3.8.1bP Pedestals

The ratio of unsupported height to average least lateral dimension of plain concrete pedestals shall not exceed 3.

5.13.4 Concrete Piles

5.13.4.1 GENERAL

The following shall supplement A5.13.4.1.

Piles shall be designed as structural members capable of safely supporting all imposed loads. A pile group composed of both vertical and battered piles which is subjected to lateral load shall be designed assuming that all lateral load is resisted by the horizontal component of the axial capacity of the battered piles. For a pile group composed of only vertical piles which is subjected to lateral load, the pile structural analysis shall include explicit consideration of soil-structure interaction effects using a COM624P (Wang and Reese, 1993) or LPILE 5.0 (ENSOFT, Inc. 2004 for LPILE 5.0) Analysis.

COMMENTARY

C5.13.4.1

The following shall supplement AC5.13.4.1.

Resistance factors, φ, for the Strength Limit State shall be taken as specified in D5.5.4.2.

The resistance factors presented in D5.5.4.2 have been selected in a manner such that, when combined with an average load factor of 1.45, the equivalent factor of safety calculated as the ratio of the appropriate load to resistance factors is comparable to the factor of safety previously used by the Department.

5.13.4.4 PRECAST PRESTRESSED PILES

Precast prestressed piles are not to be used unless approved by the Chief Bridge Engineer.

5.13.4.6 SEISMIC REQUIREMENTS

5.13.4.6.1 Zone 1

The following shall replace A5.13.4.6.1.

The requirements for Zone 1 shall be as specified for

SPECIFICATIONS COMMENTARY

Zone 2, in A5.13.4.6.2.

5.13.4.7P STRUCTURAL RESISTANCE

5.13.4.7.1P Concrete Filled Steel Pipe Piles

The factored axial resistance of undamaged concrete filled steel pipe piles shall be taken as:

$$P_r = \phi_s A_{st} f_y + \phi_c 0.85 A_{gc} f_c^{'} \quad (5.13.4.7.1P\text{-}1)$$

where:

φ_c = resistance factor for concrete as specified in D5.5.4.2

φ_s = resistance factor for steel as specified in D6.5.4.2

A_{st} = area of steel (mm^2) {in^2}

A_{gc} = gross cross-sectional area of concrete (mm^2) {in^2}

5.13.4.7.2P Prestressed Concrete Piles

The factored axial resistance prestressed concrete piles shall be taken as:

$$P_r = \phi\left(f_c^{'} - 0.87\ f_{pe}\right) A_{gc} \quad (5.13.4.7.2P\text{-}1)$$

where:

φ = resistance factor as specified in D5.5.4.2

f_{pe} = stress in member due to prestress (MPa) {ksi}

5.13.4.7.3P Precast Concrete Piles

The resistance factor specified in D5.5.4.2 shall be applied for determination of the stress levels in the gross cross-sectional area of the concrete.

5.13.4.7.4P Buckling

Instability of piles which extend through water or air shall be accounted for as specified in A5.7.4.3 using the resistance factors specified in D.5.5.4.2. Piles which extend through air or water shall be assumed to be fixed at some depth below the ground. Stability shall be determined in accordance with provisions in A5.7.4.3 for compression members using an equivalent length of the pile equal to the laterally unsupported length, plus an embedded depth to fixity. The depth to fixity shall be determined in accordance with D10.7.3.13.4 or use of COM624P or LPILE 5.0.

5.13.4.7.5P Maximum Permissible Driving Stresses

Maximum permissible driving stresses shall be taken as specified in D10.7.8.

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5.14 PROVISIONS FOR STRUCTURE TYPES

5.14.1 Beams and Girders

5.14.1.1 GENERAL

The following shall supplement A5.14.1.1.

Prestressed beam cross-section shall be selected from the Standard Drawing BD-652M. Beam cross-sections and section properties that deviate from the current standards will require specific approval from the Chief Bridge Engineer. In no case shall minimum thickness of beams be less than those given in D5.14.1.2.2.

The allowable skew limitations shown in the Standard Drawing BD-651M shall not be exceeded unless approved by the Chief Bridge Engineer. The limitations shown are maximum limits, tampering with those values may result in severe end zone distress.

5.14.1.2 PRECAST BEAMS

5.14.1.2.2 Extreme Dimensions

The following shall replace A5.14.1.2.2.

The maximum dimensions and weight of precast members manufactured at off-site casting yards shall conform to Department hauling restrictions given in Appendix E.

The thickness of any part of precast concrete beams shall not be less than:

I-BEAMS

Top Flange.. 125 mm {5 in.}
Webs... 200 mm {8 in.}

BULB TEES

Top Flange... 115 mm {4.5 in.}
Webs... 180 mm {7 in.}

BOX BEAMS

Top Flange.. 125 mm {5 in.}
Top Flange of Composite Box Beams.... 75 mm {3 in.}
Webs... 125 mm {5 in.}
Bottom Flange .... 140 mm {5 1/2 in.}

Box beams with internal haunches at the ends may be used in lieu of box beams with draped strands. An internal diaphragm at least 150 mm {6 in.} thick shall be provided at the end of the haunch where the bottom slab thickness changes from 140 mm {5 1/2 in.} to a maximum slab thickness of 200 mm {8 in.} often near the design drape point. A tapered void from the diaphragm to the end block shall be considered which would improve the internal stress flow. The top fiber tensile stresses shall not exceed the values given in Table D5.9.4.4P-1.

COMMENTARY

C5.14.1.1

Dapping at beam ends, where required, is not considered to be a deviation from the standard sections.

C5.14.1.2.2

Delete AC5.14.1.2.2.

SPECIFICATIONS

Field splices in precast members are not permitted, except if approval is obtained from the Chief Bridge Engineer during TS&L stage.

5.14.1.2.4 Detail Design

The following shall supplement A5.14.1.2.4.

For precast concrete beams used in multi-beam decks, the maximum spacing of the welded shear connectors shall not exceed the lesser of 1500 mm {5 ft.} and the width of the flange of the precast member. Welded shear connector anchors shall be located within the middle third of the slab thickness.

5.14.1.2.5 Concrete Strength

Delete A5.14.1.2.5.

5.14.1.2.7 Bridges Composed of Simple Span Precast Girders Made Continuous

5.14.1.2.7a General

The following shall replace A5.14.1.2.7a.

This article applies to bridges consisting of precast concrete girders or cast-in-place concrete slabs made continuous for transient loads by using a cast-in-place closure placement at the piers with tensile reinforcement located in the slab. Bridges made continuous by closure pours (splices) at locations other than at the piers are covered by D5.14.1.4P.

All prestressed concrete bridges shall be designed for all applicable limit states as continuous for live load and superimposed dead load with a continuous deck to eliminate joints in the deck slab. The same number of beams shall be used in adjacent spans, unless special approval is granted by the Chief Bridge Engineer at the TS&L stage. The beam depth for box beams and I-beams shall be within 150 mm {6 in.} and for bulb-tee beams shall be within 205 mm {8 in.} for beams in adjacent spans.

An added requirement for prestressed concrete beam bridges made continuous for superimposed dead load and live load is to design all structure components for the more critical condition of full continuity or the complete loss of continuity at the diaphragms over the interior supports. Positive moment steel in the continuity diaphragm(s) is not required since the structure design is not predicated on ensuring continuity.

A full continuity design option (not including the simple span check) may only be used if there is a clear economic advantage to be gained. For this option, positive moment reinforcement shall be provided in the continuity diaphragm as required by design. Justification for utilizing a full continuity design option must be submitted as part of the TS&L submission.

COMMENTARY

C5.14.1.2.5

Delete AC5.14.1.2.5.

C5.14.1.2.7a

Some of the instances where the full continuity option should be used are as follows:

- When cost analysis shows simple spans design would place the P/S industry at an economic disadvantage (an extra line of girders is required).
- When longer spans are required for a specific project site.
- When underclearance is a limiting factor in beam selection.

Bridges made continuous by closure pours (splices) at locations other than at the piers are covered by D5.14.1.4P.

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5.14.1.2.7b Reinforcement

Delete A5.14.1.2.7b.

5.14.1.2.7c Degree of Continuity at Various Limit States

Delete A5.14.1.2.7c.

5.14.1.2.7dP Precast Beam Splices at Piers

The negative moment splice of the beams at a pier shall be designed for moments from an analysis assuming full continuity for superimposed dead load and live load. Creep and shrinkage effects shall be included when a full continuity option is utilized. When two precast beams of different depth are used in a bridge made continuous for live load, the longitudinal deck mild steel reinforcement for the negative moment splice shall be designed for the smaller section. The longitudinal slab reinforcement steel designed from the above analysis should not be less than that calculated using DE6.10.3.7. The reinforcement shall be extended to a minimum distance of 0.15 times the span length or to the superimposed dead load point of contraflexure, plus development length, whichever is greater. Cut-off points for this reinforcement steel shall be staggered (see A5.11.1.2.1).

For spread and adjacent box beams, the longitudinal slab reinforcement required to make the negative moment splice of the beams shall be taken as the larger calculated based on solid and hollow section properties as follows:

- The solid box beam section shall be considered at the negative moment spike at the center of continuity.
- The hollow box beam section shall be considered at the center of the bearing with the corresponding moment.

If a full continuity design is utilized, the positive moment splice of the beam at a pier shall be designed for moments from an analysis assuming full continuity for superimposed dead load, live load and the effects of creep and shrinkage moments.

At interior piers where the diaphragms contain the closure placement, the design may be based on the strength of the concrete in the precast elements.

5.14.1.2.7eP Precast Beam Design

The beam design shall be based on:

- a simple span analysis for non-composite dead load

and

COMMENTARY

C5.14.1.2.7b

Delete AC5.14.1.2.7b.

C5.14.1.2.7c

Delete AC5.14.1.2.7c.

C5.14.1.2.7dP

If the longitudinal bar size required in the deck over adjacent box beams at a pier to resist negative live load moment is greater than a No. 13 bar {No. 4 bar} size, the slab thickness may need to be increased to more than 125 mm {5 in.} to provide 60 mm {2 1/2 in.} minimum concrete cover over the top bar mat.

C5.14.1.2.7eP

SPECIFICATIONS

- the more critical of either a continuous span analysis assuming full continuity, or a simple span analysis assuming the complete loss of continuity for composite dead load and live load (without creep and shrinkage effects), or a continuous span analysis assuming full continuity for composite dead load and live load with creep and shrinkage effects if a full continuity design option is approved by the Chief Bridge Engineer

The maximum bar size shall be limited to #25 {#8} in top layer and #19 {#6} in bottom layer of the deck slab. Minimum spacing shall be as specified in Articles A5.10.3 and D5.10.3.

The span length for the simple span analysis for noncomposite dead load shall be the length between the centerline of bearings. The span length for the continuous span analysis or a simple span analysis of composite dead load and live load shall be the length between the centerline of piers for interior span and between the centerline of pier and centerline of end bearing for exterior spans.

If the beams are designed using the "simple span check option" in PSLRFD, a separate run with the "continuous analysis" option is not required.

5.14.1.2.7fP Precast Beam and Deck Placement

The creep and shrinkage effects shall be investigated for 30 and 450 days from the day the beams are cast until the deck is poured. The worst effects from either of these two data points shall be used in the positive and negative moment splice designs.

The deck pouring sequence for prestressed concrete bridges made continuous for live load is as follows:

1. Place intermediate diaphragms, shear blocks between beams and end diaphragms at abutments.
2. Place slab in positive moment areas.
3. Place continuity diaphragms at piers.
4. Place slab in negative moment areas. This can be placed two hours after the continuity diaphragms are placed, when the diaphragm depth is over 900 mm {3 ft.}.
5. Place barriers in positive moment region, then negative moment region, unless continuous placement can be maintained.

5.14.1.2.8 Longitudinal Construction Joints

The following shall replace A5.14.1.2.8.

For details of longitudinal construction joints, see standard drawing BC-775M

COMMENTARY

For simple span design option, with prior approval from Chief Bridge Engineer, the bottom layer reinforcement bar size may be increased to a #25 {#8} bar provided a minimum clear distance of 70 mm {2 3/4 in.} between reinforcement bars is maintained at all splice locations and the requirements of A5.7.3.4 and D5.7.3.4 are met.

C5.14.1.2.8

Delete AC5.14.1.2.8.

SPECIFICATIONS

5.14.1.4P SPLICED PRECAST GIRDERS

5.14.1.4.1P General

This article applies to bridges consisting of precast concrete girders fabricated in segments that are joined or spliced longitudinally to form the girders in the final structure by using a cast-in-place closure placement (splice) at locations other than at the piers. Bridges consisting of precast concrete girders or cast-in-place concrete slabs made continuous for transient loads by using a cast-in-place closure placement at the piers with tensile reinforcement located in the slab are covered in D5.14.1.2.7a.

The requirements specified herein shall supplement the requirements of other sections of these Specifications for other than segmentally constructed bridges. Therefore, spliced precast girder bridges shall not be considered as segmental construction for the purposes of design. For special design cases, additional provisions for segmental construction found in 5.14.2 and other articles in these Specifications may be used where appropriate.

The method of construction assumed for the design shall be shown in the contract documents. All supports required prior to the splicing of the girder shall be shown on the contract documents, including elevations and reactions. The stage of construction during which the temporary supports are removed shall also be shown on the contract documents.

The contract documents shall indicate alternative methods of construction permitted and the Contractor's responsibilities if such methods are chosen. Any changes by the Contractor to the construction method or to the design shall comply with the requirements of 5.14.2.5 and the Department’s policies regarding alternative designs.

Stresses due to changes in the statical system, in particular, the effects of the application of load to one structural system and its removal from a different structural system, shall be accounted for. Redistribution of such stresses by creep shall be taken into account and allowance shall be made for possible variations in the creep rate and magnitude.

Spliced girder superstructures which satisfy all service limit state requirements of this article may be designed as fully continuous at all limit states for loads applied after the girder segments are joined.

Prestress losses in spliced precast girder bridges may be estimated using the provisions for other than segmentally constructed bridges in Articles A5.9.5and D5.9.5. The effects of combined pretensioning and post-tensioning and staged post-tensioning shall be considered.

When required, the effects of creep and shrinkage in spliced precast girder bridges may be estimated using the provisions for other than segmentally constructed bridges in Articles A5.4.2.3 and D5.4.2.3.

Precast deck girder bridges, for which some or all of the deck is cast integrally with a girder, may be spliced.

COMMENTARY

C5.14.1.4.1P

Bridges consisting of spliced precast girder segments have been constructed in a variety of locations outside Pennsylvania for many different reasons. An extensive database of spliced girder bridge projects has been compiled and is present in the appendix to Castrodale and White (*2004*).

Splicing of girder segments is generally performed in place, but may be performed at the construction site prior to erection. The final structure may be a simple span or a continuous span unit.

Traditionally, spliced precast girder bridges were considered as a special case of both conventional precast girders and segmental construction. However, it is more appropriate to classify this type of structure as a conventional bridge with additional requirements at the splice locations that are based on provisions developed for segmental construction. The cross-section for bridges utilizing segmented precast girders is typically comprised of several girders with a composite deck.

Spliced precast girder bridges may be distinguished from what is referred to as “segmental construction” elsewhere in these Specifications by several features which typically include:

- The lengths of some or all segments in a bridge are a significant fraction of the span length rather than having a number of segments in each span. In some cases, the segment may be the full span length.
- Design of joints between girder segments at the service limit state does not typically govern the design for the entire length of the bridge for either construction or for the completed structure.
- Cast-in-place closure joints are usually used to join girder segments rather than match-cast joints.
- The bridge cross-section is comprised of several individual girders with a cast-in-place concrete composite deck rather than precasting the full width and depth of the superstructure as one piece. In some cases, the deck may be divided into pieces that are integrally cast with each girder. A bridge of this type is completed by connecting the girders across the longitudinal joints without a cast-in-place composite deck. The latter form of construction requires approval

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Spliced structures of this type, which have longitudinal joints in the deck between each deck girder, shall comply with the additional requirements of Articles A5.14.4.3 and D5.14.4.3.

Spliced precast girders may be made continuous for some permanent loads using details for simple span precast girders made continuous. In such cases, design shall conform to the applicable requirements of Articles A5.14.1.2.7 and D5.14.1.2.7.

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of the Chief Bridge Engineer prior to the start of the design.

- Girder sections are used, such as bulb tee, rather than closed cell boxes with wide monolithic flanges.

Provisional ducts are required for segmental construction (Articles A5.14.2.3.8a and D5.14.2.3.8a) to provide for possible adjustment of prestress force during construction. Similar requirements are not given for spliced precast girder bridges because of the redundancy provided by a greater number of webs and tendons, and typically lower friction losses because of fewer joint locations.

The method of construction and any required temporary support is of paramount importance in the design of spliced precast girder bridges. Such considerations often govern final conditions in the selection of section dimensions and reinforcing and/or prestressing.

Deck girder bridges are often spliced because the significant weight of the cross-section, which is comprised of both a girder and deck, may exceed usual limits for handling and transportation.

5.14.1.4.2P Joints Between Segments

5.14.1.4.2aP General

Joints between girder segments shall be either cast-in-place closure joints or match-cast joints. Match-cast joints shall satisfy the requirements of A5.14.2.4.2.

The sequence of placing concrete for the closure joints and deck shall be specified in the contract documents.

C5.14.1.4.2aP

This Article codifies current best practice, which allows the Designer considerable latitude to formulate new structural systems. The great majority of in-span construction joints have been post-tensioned. Conventionally reinforced joints have been used in a limited number of bridges.

Cast-in-place closure joints are typically used in spliced girder construction. Machined bulkheads have been used successfully to emulate match-cast epoxy joints for spliced girders. Prestress, dead load, and creep effects may cause rotation of the faces of the match-cast epoxy joints prior to splicing. Procedures for splicing the girder segments that overcome this rotation to close the match-cast joint should be shown on the contract plans.

5.14.1.4.2bP Details of Closure Joints

Precast concrete girder segments, with or without a cast-in-place slab, may be made longitudinally continuous for both permanent and transient loads with combinations of post-tensioning and/or reinforcement crossing the closure joints.

The width of a closure joint between precast concrete segments shall allow for the splicing of steel whose continuity is required by design considerations and the accommodation of the splicing of post-tensioning ducts.

C5.14.1.4.2bP

When diaphragms are provided at closure joint locations, designers should consider extending the closure joint at the exterior girder beyond the outside face of the girder. Extending the closure joint beyond the face of the exterior girder also provides improved development of diaphragm reinforcement for bridges subject to extreme events.

The intent of the joint width requirement is to allow proper compaction of concrete in the cast-in-place closure

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The width of a closure joint shall not be less than 300 mm {12.0 in.}, except for joints located within a diaphragm, for which the width shall not be less than 100 mm {4.0 in}.

If the width of the closure joint exceeds 150 mm {6.0 in.}, its compressive chord section shall be reinforced for confinement.

If the joint is located in the span, its web transverse reinforcement per unit length of the girder shall be the larger of that in the adjacent girder segments.

The face of the precast segments at closure joints shall have shear keys in accordance with A5.14.2.4.2.

Shear keys shall be provided at the ends of the girder segments on either side of the closure joints.

joint. Consolidation of concrete in a closure joint is enhanced when the joint is contained within a diaphragm. A wider closure joint may be used to provide more room to accommodate tolerances for potential misalignment of ducts within girder segments and misalignment of girder segments at erection.

The bottom flange near an interior support acts nearly as a column, hence the requirement for confinement steel.

Roughening the ends of the girder segments on either side of closure joints is not considered sufficient to develop the required shear strength

5.14.1.4.2cP Details of Match-Cast Joints

Match-cast joints for spliced precast girder bridges shall be detailed in accordance with A5.14.2.4.2.

C5.14.1.4.2cP

One or more large shear keys may be used with spliced girders rather than the multiple small amplitude shear keys indicated in A5.14.2.4.2. The shear key proportions specified in A5.14.2.4.2 should be used.

5.14.1.3.2d Joint Design

Stress limits for temporary concrete stresses in joints before losses specified in A5.9.4.1 for segmentally constructed bridges shall apply at each stage of prestressing (pretensioning or post-tensioning). The concrete strength at the time the stage of prestressing is applied shall be substituted for f'_{ci} in the stress limits.

Stress limits for concrete stresses in joints at the service limit state after losses specified in 5.9.4.4P for segmentally constructed bridges shall apply. These stress limits shall also apply for intermediate load stages, with the concrete strength at the time of loading substituted for f'_c in the stress limits.

Resistance factors for joints specified in A5.5.4.2.2 for segmental construction shall apply.

The compressive strength of the closure joint concrete at a specified age shall be compatible with design stress limitations.

5.14.1.4.3P Girder Segment Design

Stress limits for temporary concrete stresses in girder segments before losses specified in 5.9.4.4P for other than segmentally constructed bridges shall apply at each stage of prestressing (pretensioning or post-tensioning) with due consideration for all applicable loads during construction. The concrete strength at the time the stage of prestressing is applied shall be substituted for f'_{ci} in the stress limits.

Stress limits for concrete stresses in girder segments at the service limit state after losses specified in 5.9.4.4P for other than segmentally constructed bridges shall apply. These stress limits shall also apply for intermediate load stages, with the concrete strength at the time of loading

C5.14.1.4.3P

Segments of spliced precast girders shall preferably be pretensioned for dead load and all applicable construction loadings to satisfy temporary stress limits in the concrete.

Temporary construction loads must be considered where these loads may contribute to critical stresses in girder segments at an intermediate stage of construction, such as when the deck slab is placed when only a portion of the total prestress has been applied. Temporary construction loads are specified in the *AASHTO Guide Design Specifications for Bridge Temporary Works.*

Because gravity loads induce compression in the bottom flange of girders at support locations, the vertical

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substituted for f'_c in the stress limits.

Where girder segments are precast without prestressed reinforcement, the provisions of Articles A5.7.3.4 and D5.7.3.4 shall apply until post-tensioning is applied.

Where variable depth girder segments are used, the effect of inclined compression shall be considered.

The potential for buckling of tall thin web sections shall be considered.

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force component from inclined flexural stresses in a haunched girder segment generally acts to reduce the applied shear. Its effect can be accounted for in the same manner as the vertical component of the longitudinal prestressing force, V_p. However, the reduction of the vertical shear force from this effect is usually neglected.

5.14.1.4.4P Post-Tensioning

Post-tensioning may be applied either before and/or after placement of deck concrete. Part of the post-tensioning may be applied to provide girder continuity prior to placement of the deck concrete, with the remainder placed after deck concrete placement.

The contract documents shall require that all post-tensioning tendons shall be fully grouted after stressing.

Prior to grouting of post-tensioning ducts, gross cross-section properties shall be reduced by deducting the area of ducts and void areas around tendon couplers.

Post-tensioning shall be shown on the contract documents according to the requirements of A5.14.2.3.9.

Where tendons terminate at the top of a girder segment, the contract documents shall require that duct openings be protected during construction to prevent debris accumulation and that drains be provided at tendon low points.

In the case of multistage post-tensioning, draped ducts for tendons to be tensioned before the slab concrete is placed and attains the minimum specified compressive strength f'_{ci} shall not be located in the slab.

Where some or all post-tensioning tendons are stressed after the deck concrete is placed, provisions shall be shown on the contract plans satisfying the provisions of A2.5.2.3 on maintainability of the deck.

C5.14.1.4.4P

Where some or all post-tensioning is applied after the deck concrete is placed, fewer post-tensioning tendons and a lower concrete strength in the closure joint may be required. However, deck replacement, if necessary, is difficult to accommodate with this construction sequence. Where all of the post-tensioning is applied before the deck concrete is placed, a greater number of post-tensioning tendons and a higher concrete strength in the closure joint may be required. However, in this case, the deck can be replaced if necessary. See Castrodale and White (*2004*).

See A5.10.3.5 for post-tensioning coupler requirements.

Where tendons terminate at the top of the girder, blockouts and pourbacks in the deck slab are required for access to the tendons and anchorages. While this arrangement has been used, it is preferable to anchor all tendons at the ends of girders. Minimizing or eliminating deck slab blockouts by placing anchorages at ends of girders reduces the potential for water seepage and corrosion at the post-tensioning tendon anchors.

This provision is to ensure that ducts as yet unsecured by concrete will not be used for active post-tensioning.

See 5.14.2.3.10e for deck overlay provisions.

5.14.2 Segmental Construction

5.14.2.1 GENERAL

The following shall supplement A5.14.2.1.

The design and construction details of segmental bridges shall be such that:

- deck is replaceable
- only internal bonded post-tensioning is used (except external unbonded post-tensioning may be used for the temporary condition on future strengthening)

C5.14.2.1

The following shall replace the first paragraph of AC5.14.2.1.

For segmental construction, superstructures of single or multiple box sections are generally used. Segmental construction includes construction by free cantilever, span-by-span, or incremental launching methods using either precast or cast-in-place concrete segments which are joined to produce either continuous or simple spans.

Bridges utilizing beam type sections may also be constructed using segmental construction techniques. Such bridges, which are referred to as spliced precast girder bridges, are considered as a special case of conventional bridges. The design of such bridges is covered in 5.14.1.4P.

SPECIFICATIONS COMMENTARY

5.14.2.3 DESIGN

5.14.2.3.3 Construction Load Combinations at the Service Limit State

The following table shall replace Table A1

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Table 5.14.2.3.3-1 - Load Factors and Allowable Tensile Stresses for Construction Load Combinations

Load Combination	LOAD FACTORS																Allowable Tensile Stress				See Note
	Dead Load			Live Load				Wind Load			Other Loads						Metric Units		U.S. Customary Units		
	DC	DIFF	U	CLL	CE	IE	CLE	WS	WUP	WE	CR	SH	TU	TG	EH EV ES	WA	Excluding "Other Loads"	Including "Other Loads"	Excluding "Other Loads"	Including "Other Loads"	
a	1.0	1.0	0.0	1.0	1.0	1.0	0.0	0.0	0.0	0.0	1.0	1.0	1.0	γ_{TG}	1.0	1.0	$0.25\sqrt{f_c^{'}}$	$0.50\sqrt{f_c^{'}}$	$0.095\sqrt{f_c^{'}}$	$0.190\sqrt{f_c^{'}}$	
b	1.0	0.0	1.0	1.0	1.0	1.0	0.0	0.0	0.0	0.0	1.0	1.0	1.0	γ_{TG}	1.0	1.0	$0.25\sqrt{f_c^{'}}$	$0.50\sqrt{f_c^{'}}$	$0.095\sqrt{f_c^{'}}$	$0.190\sqrt{f_c^{'}}$	
c	1.0	1.0	0.0	0.0	0.0	0.0	0.0	0.7	0.7	0.0	1.0	1.0	1.0	γ_{TG}	1.0	1.0	$0.25\sqrt{f_c^{'}}$	$0.50\sqrt{f_c^{'}}$	$0.095\sqrt{f_c^{'}}$	$0.190\sqrt{f_c^{'}}$	
d	1.0	1.0	0.0	1.0	1.0	0.0	0.0	0.7	1.0	0.7	1.0	1.0	1.0	γ_{TG}	1.0	1.0	$0.25\sqrt{f_c^{'}}$	$0.50\sqrt{f_c^{'}}$	$0.095\sqrt{f_c^{'}}$	$0.190\sqrt{f_c^{'}}$	1
e	1.0	0.0	1.0	1.0	1.0	1.0	0.0	0.3	0.0	0.3	1.0	1.0	1.0	γ_{TG}	1.0	1.0	$0.25\sqrt{f_c^{'}}$	$0.50\sqrt{f_c^{'}}$	$0.095\sqrt{f_c^{'}}$	$0.190\sqrt{f_c^{'}}$	2
f	1.0	0.0	0.0	1.0	1.0	1.0	1.0	0.3	0.0	0.3	1.0	1.0	1.0	γ_{TG}	1.0	1.0	$0.25\sqrt{f_c^{'}}$	$0.50\sqrt{f_c^{'}}$	$0.095\sqrt{f_c^{'}}$	$0.190\sqrt{f_c^{'}}$	3

SPECIFICATIONS

5.14.2.3.6 Creep and Shrinkage

The following shall replace the first sentence of the first paragraph of A5.14.2.3.6.

Creep coefficient Ψ (t, t_i) shall be determined either in accordance with A5.4.2.3 and D5.4.2.3 or by comprehensive tests which are approved by the Chief Bridge Engineer.

5.14.2.3.8 Provisional Post-Tensioning Ducts and Anchorages

5.14.2.3.8a General

The following shall supplement A5.14.2.3.8a.

External ducts are not permitted, except for temporary construction or rehabilitation projects.

5.14.2.3.10 Box Girder Cross-Section Dimensions and Details

5.14.2.3.10e Overlays

The following shall replace A5.14.2.3.10e.

Overlays shall be considered for all bridge decks exposed to freeze thaw cycles and application of deicing chemicals. The governing authority should consider providing additional protection against penetration of chlorides. For all types of segmental bridges (precast and cast-in-place), it is recommended that this additional protection be provided by the addition of a minimum of 38 mm {1.5 in.} of concrete cover, added as an overlay. The governing authority may require specific materials and placement techniques stipulated by local practices.

5.14.2.5 USE OF ALTERNATIVE CONSTRUCTION METHODS

The following shall supplement A5.14.2.5.

The use of alternative segmental construction methods requires the approval of the Chief Bridge Engineer.

5.14.4 Slab Superstructures

5.14.4.1 CAST-IN-PLACE SOLID SLAB SUPERSTRUCTURES

The following shall replace the first sentence of the first paragraph of A5.14.4.1.

Cast-in-place, longitudinally reinforced slabs may be either conventionally reinforced or prestressed and may be used as slab-type bridges.

COMMENTARY

C5.14.2.3.7 Prestress Losses

The following shall supplement AC5.14.2.3.7.

In-place friction test results shall be approved by the Chief Bridge Engineer.

SPECIFICATIONS COMMENTARY

The following shall replace the first sentence of the second paragraph of A5.14.4.1.

The distribution of live load may be determined by a refined analysis or as specified in D4.6.2.3.

5.14.4.3 PRECAST DECK BRIDGES

5.14.4.3.1 General

Delete the second paragraph of A5.14.4.3.1.

5.14.4.3.3 Shear-Flexure Transfer Joints

5.14.4.3.3f Structural Overlay

The following shall replace A5.14.4.3.3f.

When a structural overlay is used to qualify for improved load distribution as provided in A4.6.2.2.2 and A4.6.2.2.3, the thickness of structural concrete overlay shall not be less than 125 mm {5 in.}. An isotropic layer of reinforcement shall be provided in accordance with the requirements of A5.10.8. The top surface of the precast components shall be roughened.

5.14.5 Additional Provisions for Culverts

5.14.5.4P SHEAR RESISTANCE PROVIDED BY SINGLE BENT-UP BARS IN BOX CULVERTS

The additional nominal shear resistance provided by a single bar or single group of parallel bars all bent up at the same distance from the support shall be taken as:

Metric Units:

$$V_s = A_v f_y \sin\alpha \le 0.25\sqrt{f_c'}\,bd_v \qquad (5.14.5.4\text{-}1P)$$

U.S. Customary Units:

$$V_s = A_v f_y \sin\alpha \le 0.948\sqrt{f_c'}\,bd_v$$

where:

A_v = cross sectional area of bent-up bar (mm^2) {in^2}

f_y = yield strength of bent-up bar (MPa) {ksi}

α = angle of inclination of transverse reinforcement to longitudinal axis (DEG)

b = width of concrete section (mm) {in.}

d_v = effective shear depth as determined in A5.8.2.7 (mm) {in.}

f'_c = specified compressive design strength of concrete at 28 days, unless other age is specified (MPa) {ksi}

SPECIFICATIONS COMMENTARY

5.14.6P Post-Tensioned Pier Caps

For inverted T-post-tensioned pier caps, a detailed stress analysis and detailed design is required for bracket (corbel) reinforcement, including all forces acting on any pedestal.

This investigation must include, but is not limited to, bearing pressure, bending, flexural shear, diagonal tension, pure shear, punching shear, horizontal forces, vertical forces and torsional forces.

Investigate the combined effect of all vertical and horizontal superstructure forces and any other conditions which are warranted.

To compensate for incidental field adjustments in the location of bearings, all pier's columns, solid piers and abutment's stems shall be designed for a 50 mm {2 in.} longitudinal eccentricity off the theoretical centerline of bearing. The eccentricity need not be considered for footing design.

End faces shall be proportioned to allow proper placement of anchor plates. Outside edges of anchor plates shall not be less than 75 mm {3 in.} from an exposed edge.

In addition to the other strength and service limit state checks, concrete tensile stresses due to Service III Load Combination loads shall not exceed $0.25/fN_c$ {U. S. Customary Units: $0.0948/fN_c$}.

REFERENCES

The following shall supplement the references of A5.

Lutz, LeRoy, A., “Crack Control Factor for Bundled Bars and for Bars of Different Sizes”, ACI Journal, January 1974, pp 9-10.

Frosch, R. J. (2001). "Flexural Crack Control in Reinforced Concrete," Design and Construction Practices to Mitigate Cracking, SP 204, American Concrete Institute, Farmington Hills, Mich., pp. 135-154.

Destefano, R.J., Evans, J., Tadros, M.K., Sun, C., “Flexural Crack Control in Concrete Bridge Structures,” PCI Convention, October 19, 2003.

Castrodale, R. W., and C. D. White. 2004. “Extending Span Ranges of Precast, Prestressed Concrete Girders.” NCHRP Report 517. TRB National Research Council, Washington, D.C.

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 1
PART B: DESIGN SPECIFICATIONS

SECTION 6 – STEEL STRUCTURES

SECTION 6 - TABLE OF CONTENTS

SPECIFICATIONS

6.1 SCOPE

6.1.1P Restrictions of Steel Bridge Types

6.1.1.1P STEEL TIED-ARCH BRIDGES

Steel tied-arch bridges shall be used only after thorough consideration has been given to all factors in design, fabrication and erection, and if the design is approved by the Chief Bridge Engineer. In the preliminary stage, the tied-arch must show a marked economic advantage over alternate designs to warrant further consideration. Tied-arch structures are currently unacceptable to FHWA. Moreover, refer to the FHWA Technical Advisory T-5140.4, dated September 28, 1978, for the problems pertinent to tied-arch structures.

Transverse welds on the tie girders shall be avoided, where possible. Bolted connections shall be used instead of transverse welds.

On Langer-girder tied-arch bridges (those tied arches where the tie girder acts as the major flexural member in addition to providing horizontal reactions to the arch rib) with box girders functioning as tie girders, the internal diaphragms stiffening the box at the floorbeam connections shall be attached to both flanges, as well as the webs. A tie plate should be placed between the tie-girder flange and the floorbeam flange if they lie essentially in the same plane.

Hangers composed of multiple bridge strands shall have either spacers between the strands or dampers, or both.

The dynamic response of the bridge due to traffic shall be investigated by an appropriate three-dimensional, forced-vibration dynamic analysis, especially for tied-arch bridges that do not employ Langer-girders.

6.1.1.2P Steel Box Bridges

Steel box bridges shall be used only after thorough consideration has been given to all factors in design, fabrication, erection and future in-depth inspection, and if the design is approved by the Chief Bridge Engineer. In the preliminary stage, the steel box design must show a marked economic or aesthetic advantage over alternate designs to warrant further consideration.

COMMENTARY

C6.1.1.1P

Steel tied-arch bridges have experienced such problems as lamellar tearing in the hanger connections, detrimental vibration in the main structure and cables, and cracking in fracture-critical members. The design, detailing, and fabrication of the floorbeams are critical for long-term performance. Fatigue cracking has occurred in floorbeams due to out-of-plane distortion in combination with abrupt termination of the flange; proper coping and grinding o fthe cope were not performed.

The designer must use intuitive engineering judgment when selecting the type, location and number of spacers used between the strands of a hanger composed of multiple bridge strands. The need for spacers is not based upon a calculated analysis, but rather on the observation that some bridges without spacers experienced problems and were subsequently retrofitted with spacers.

C6.1.1.2P

Even though steel box girders may provide aesthetically pleasing and sometimes economical structures, the Department has major concerns about steel box girders which are:

- difficult inspection environment,
- inspection complexities,
- future cleaning, painting and/or repair difficulties.
- detailing complexities
- stability during erection

SPECIFICATIONS COMMENTARY

6.2 Definitions

The following shall supplement A6.2.

Controlling Flange—top or bottom flange for the smaller section at a point of splice, whichever flange has the maximum elastic flexural stress at its mid-thickness due to the factored loads.

Non-controlling Flange—the flange at a point of splice opposite the controlling flange.

6.3 NOTATION

The following shall supplement A6.3

A_{bot}	=	area of the bottom flange (mm^2) {in.^{2}} (D6.10.10.1.2)
A_d	=	minimum required cross-sectional area of a diagonal member of top lateral bracing for tub sections (mm^2) {in.^{2}} (DC6.7.5.3)
F_{fat}	=	radial fatigue shear range per unit length, taken as the larger of either F_{fat1} or F_{fat2} (N/mm) {kip/in.} (D6.10.10.1.2)
F_{fat1}	=	radial fatigue shear range per unit length due to the effect of any curvature between brace points (N/mm) {kip/in.} (D6.10.10.1.2)
F_{fat2}	=	radial fatigue shear range per unit length due to torsion caused by effects other than curvature, such as skew (N/mm) {kip/in.} (D6.10.10.1.2)
F_p	=	total radial shear force in the concrete deck at the point of maximum positive live load plus impact moment for the design of the shear connectors at the strength limit state, taken equal to zero for straight spans or segments (N) {kip} (D6.10.10.4.2)
F_{rc}	=	net range of cross-frame force at the top flange (N) {kip} (D6.10.10.1.2)
F_T	=	total radial shear force in the concrete deck between the point of maximum positive live load plus impact moment and the centerline of an adjacent interior support for the design of shear connectors at the strength limit state, taken equal to zero for straight spans or segments (N) {kip} (D6.10.10.4.2)
f_{bu}	=	largest value of the compressive stress throughout the unbraced length in the flange under consideration, calculated without consideration of flange lateral bending (MPa) {ksi} (D6.10.1.6)
L_n	=	arc length between the point of maximum positive live load plus impact moment and the centerline of an adjacent interior support (mm) {ft.} (D6.10.10.4.2)
L_p	=	arc length between an end of the girder and an adjacent point of maximum positive live load plus impact moment (mm) {ft.} (D6.10.10.4.2)
M_u	=	largest value of the major-axis bending moment throughout the unbraced length causing compression in the flange under consideration (N-mm) {k-in.} (D6.10.1.6)
P_T	=	total longitudinal shear force in the concrete deck between the point of maximum positive live load plus impact moment and the centerline of an adjacent interior support for the design of the shear connectors at the strength limit state, taken as the sum of P_p and P_n (N){kip} (D6.10.10.4.2)
R	=	minimum girder radius within a panel (mm) {ft.} (D6.7.4.2)
r_σ	=	desired bending stress ratio in a horizontally curved I-girder, taken equal to $\left| f_\ell / f_{bu} \right|$ (DC6.7.4.2)
V_{fat}	=	longitudinal fatigue shear range per unit length (N/mm) {kip/in.} (D6.10.10.1.2)
w	=	effective length of deck assumed acting radial to the girder (mm) {in.} (D6.10.10.1.2)
Z	=	curvature parameter for determining required longitudinal web stiffener rigidity (D6.10.11.3.3)
β	=	curvature correction factor for longitudinal web stiffener rigidity (D6.10.11.3.3)
σ_{flg}	=	range of longitudinal fatigue stress in the bottom flange without consideration of flange lateral bending (MPa) {ksi} (D6.10.10.1.2)

SPECIFICATIONS

6.4 MATERIALS

6.4.1 Structural Steels

The following shall supplement A6.4.1.

Poisson's ratio for structural steel shall be assumed to be 0.3 in the elastic range.

Unless directed otherwise, all structural steel shall conform to the specifications for structural steel, ASTM A 709/A 709M, Grade 250 {Grade 36}. Other types of steel, such as ASTM A 709/A 709M, Grades 345 and 345W {Grade 50 and 50W}, in combination with ASTM A 709/A 709M, Grade 250 {Grade 36}, or with each other may be considered for economy.

Steel grades 690 or 690W {Grades 100 or 100W} shall not be used unless written approval has been obtained from the Chief Bridge Engineer.

Unpainted ASTM A 709/A 709M, Grade 345W or HPS-485W {Grade 50W or HPS-70W}, steel shall not be specified without written approval of the Chief Bridge Engineer at the TS&L stage. This policy applies to state and local bridges and bridges where State or Federal funding is utilized. Use in contractor-designed alternates must also be approved at the TS&L stage. Use is not permitted in acidic or corrosive environments, in locations subject to salt water spray or fog, in depressed roadway sections (less than 6100 mm {20 ft.} clearance) where salt spray and other pollutants may be trapped, in low underclearance situations where the steel is either less than 1500 mm {5 ft.} from normal water elevation or continuously wet, or where the steel may be buried in soil. The use of Grade 345W or HPS-485W {Grade 50W or HPS-70W} steel is not permitted in bridge types where salt spray and dirt accumulation may be a concern (e.g., trusses or inclined-leg bridges) unless corrosion-susceptible regions are painted.

Do not use Grade 345W or HPS-485W {Grade 50W or HPS-70W} steel for expansion dams, or for stringers or other members under open steel decking.

Where the use of Grade 345W {Grade 50W} or Grade HPS-485W {Grade HPS-70W}unpainted weathering steel is permitted, the following criteria must be met:

(a) The number of expansion joints shall be minimized.

(b) Details to avoid retention of water and debris shall be incorporated in the design.

(c) The steel shall be painted to a length of at least 1.5 times web depth and a minimum of 1500 mm {5 ft.} on each side of the expansion joint.

(d) Drip plates shall be provided.

COMMENTARY

C6.4.1

The following shall supplement AC6.4.1.

For additional information on the economics of steel bridges, see PP4.3.

For additional information, refer to NCHRP Report No. 314, Guidelines for the Use of Weathering Steel in Bridges.

Drip bars attached as indicated on BC-753M.

SPECIFICATIONS

(e) The substructure units shall be protected against staining. Use special drainage details for pier and abutment tops and/or protective coating for reinforced concrete surfaces in accordance with the Publication 408.

(f) Mechanical fasteners made of ASTM A 325 and A 490, Type 3, weathering steels and stainless steels are suitable for weathering steel bridges. Do not use zinc and cadmium galvanized carbon-steel bolts for weathering steel bridges.

(g) Load indicator washers are not recommended.

For existing bridges, where Grade 345W {Grade 50W} unpainted steel is used, clean and paint the beam ends up to 1500 mm {5 ft.} from leaking joints, or to where the weathering steel area is exposed to or subject to salt water spray.

6.4.3 Bolts, Nuts and Washers

6.4.3.1 BOLTS

The following shall replace A6.4.3.1.

Bolts shall conform to one of the following:

- the Standard Specification for Carbon Steel Bolts and Studs, 414 MPa {60 ksi} Tensile Strength, ASTM A 307,

- the Standard Specification for Structural Bolts, Steel, Heat-Treated, 827/724 MPa {120/105 ksi} Minimum Tensile Strength with a required minimum tensile strength of 827 MPa {120 ksi} for diameters 12.7 mm through 25.4 mm {1/2 in. through 1 in.} and 724 MPa {105 ksi} for diameters 28.6 mm through 38.1 mm {1 1/8 in. through 1 1/2 in.}, AASHTO M 164 (ASTM A 325), or

- the Standard Specification for Heat-Treated Steel Structural Bolts, 1034 MPa {150 ksi} Minimum Tensile Strength, AASHTO M 253 (ASTM A 490).

AASHTO M 253 (ASTM A 490) bolts are not allowed unless approved by the Chief Bridge Engineer.

Type 1 bolts should be used with steels other than weathering steel. Type 3 bolts conforming with either ASTM A 325 or ASTM A 490 shall be used with weathering steels. AASHTO M 164 (ASTM A 325), Type 1, bolts may be mechanically galvanized in accordance with AASHTO M 298 (ASTM B 695), Class 50, when approved by the Engineer. Galvanized bolts shall be tension tested

COMMENTARY

Preferably for weathering steel bridges, use mechanical fasteners made of weathering steel. When stainless steel mechanical fasteners are used with weathering steel bridges, there is a possibility of galvanic corrosion of the weathering steel. Due to the small area of the bolt in relation to the material being bolted, the effect is usually negligible.

C6.4.3.1

The following shall supplement AC6.4.3.1.

Although there are metric high-strength bolt standards for ASTM A 325M and A 490M, as of this writing, no installation specification is available for metric high-strength bolts and no domestic high-strength bolt manufacturers are producing the metric bolts. The Department decided to use soft metric conversions of standard "English" high-strength bolts (ASTM A 325 and A 490) until these problems with the metric high-strength bolts are resolved (i.e., standard "English" high-strength bolts will be the same, except they will have a metric name).

A Lehigh University study shows that AASHTO M 253 (ASTM A 490) bolts are more sensitive to the number of threads in the grip than AASHTO M 164 (ASTM A 325) bolts. The decrease in tension in AASHTO M 253 (ASTM A 490) bolts after the maximum tension is reached is much more rapid than the unloading experienced in the AASHTO M 164 (ASTM A 325) bolt assembly. Also, the AASHTO M 253 (ASTM A 490) bolts have reduced ductility compared to the AASHTO M 164 (ASTM A 325) bolt

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after galvanizing, as required by AASHTO M 164 (ASTM A 325).

AASHTO M 253 (ASTM A 490) bolts shall not be galvanized.

Washers, nuts and bolts of any assembly shall be galvanized by the same process. The nuts should be overtapped to the minimum amount required for the fastener assembly, and shall be lubricated with a lubricant containing a visible dye.

COMMENTARY

having the same length of thread in the grip. Hot-dipped galvanized bolts are not permitted due to concerns associated with the quality of the threads.

6.4.3.2 NUTS

The following shall replace A6.4.3.2.

Except as noted below, nuts for AASHTO M 164 (ASTM A 325) bolts shall conform to either the Standard Specification for Carbon and Alloy Steel Nuts, AASHTO M 291 (ASTM A 563), Grades DH, DH3, C, C3, and D, or the Standard Specification for Carbon and Alloy Steel Nuts for Bolts for High-Pressure and High-Temperature Service, AASHTO M 292 (ASTM A 194), Grades 2 and 2H.

Nuts for AASHTO M 253 (ASTM A 490) bolts shall conform to the requirements of AASHTO M 291 (ASTM A 563), Grades DH and DH3 or AASHTO M 292 (ASTM A 194), Grade 2H.

Nuts to be galvanized shall be heat treated, Grade DH . The provisions of D6.4.3.1 shall apply.

Plain nuts shall have a minimum hardness of 89 HRB.

Nuts to be used with AASHTO M 164 (ASTM A 325), Type 3 bolts shall be of Grade C3 or DH3. Nuts to be used with AASHTO M 253 (ASTM A 490), Type 3, bolts shall be of Grade DH3.

C6.4.3.2

The following shall supplement AC6.4.3.2.

Following the same logic as given in DC6.4.3.1, a soft metric conversion of standard "English" nuts (ASTM A 563 and ASTM A 194) will be used.

6.4.3.3 WASHERS

The following shall replace A6.4.3.3.

Washers shall conform to the Standard Specification for Hardened Steel Washers, AASHTO M 293 (ASTM F 436).

The provisions of D6.4.3.1 shall apply to galvanized washers.

6.4.3.4 ALTERNATIVE FASTENERS

The following shall replace the first portion of the first sentence of A6.4.3.4.

Other fasteners or fastener assemblies, not specified heretofore, may not be used unless approved by the Chief Bridge Engineer,

6.4.3.5 LOAD INDICATOR DEVICES

The following shall supplement A6.4.3.5.

For additional requirements concerning load indicator devices, see Publication 408.

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6.4.7 Stainless Steel

The following shall be added to the last sentence of the last paragraph of A6.4.7.

"...and approved by the Chief Bridge Engineer."

6.5 LIMIT STATES

6.5.2 Service Limit State

The following shall replace the second paragraph of A6.5.2

Flexural members shall be investigated at the service limit state as specified in A6.10, D6.10, A6.11, D6.11, DE6.10P and DE6.11P.

6.5.3 Fatigue and Fracture Limit State

The following shall replace the third paragraph of A6.5.3

Flexural members shall be investigated at the fatigue and fracture limit state as specified in A6.10, D6.10, A6.11, D6.11, DE6.10P and DE6.11P.

6.5.4 Strength Limit State

6.5.4.2 RESISTANCE FACTORS

Replace all references to A 325M and A 490M with A 325 and A 490, respectively.

The following shall replace the pile resistance factors in A6.5.4.2.

- for axial resistance of piles in compression and subject to damage due to severe driving conditions where use of a pile tip is necessary................................. $\varphi_c = 0.35$
- for axial resistance of piles in compression under good driving conditions where use of a pile tip is not necessary.. $\varphi_c = 0.45$
- for axial resistance of piles bearing on soluble bedrock $\varphi_c = 0.25$
- for axial resistance of steel portion of concrete filled pipe piles in compression .... $\varphi_c = 0.35$
- for combined axial and flexural resistance of undamaged piles:

COMMENTARY

C6.5.2

The following shall replace AC6.5.2

The intent of the service limit state provisions specified for flexural members in A6.10, D6.10, A6.11, D6.11, DE6.10P and DE6.11P is primarily to prevent objectionable permanent deformations due to localized yielding that would impair rideability under expected severe traffic loadings.

C6.5.4.2

The following shall supplement AC6.5.4.2.

The basis for the resistance factors for driven steel piles is described in DC6.15.2.

SPECIFICATIONS

- axial resistance .... $\varphi_c = 0.60$
- flexural resistance............................ $\varphi_f = 0.85$

6.6 FATIGUE AND FRACTURE CONSIDERATIONS

6.6.1 Fatigue

6.6.1.2 LOAD-INDUCED FATIGUE

6.6.1.2.1 Application

Delete the second sentence of the first paragraph of A6.6.1.2.1

6.6.1.2.2 Design Criteria

The following shall replace A6.6.1.2.2.

For load-induced fatigue considerations, each detail shall satisfy:

$$\gamma\left(PTF\right)\left(\Delta f\right) \le \left(\Delta F\right)_n \qquad (6.6.1.2.2\text{-}1)$$

where:

γ = load factor specified in Table A3.4.1-1 for the fatigue load combination

(Δf) = the force effect, live load stress range due to the passage of the fatigue load as specified in A3.6.1.4 (MPa) {ksi}

$(\Delta F)_n$ = the nominal fatigue resistance as specified in A6.6.1.2.5 and D3.6.1.4 (MPa) {ksi}

= $\frac{1}{2}(\Delta F)_{TH}$ as specified in A6.6.1.2.5 and D3.6.1.4 for Interstate and NHS bridges

PTF = Pennsylvania Traffic Factor as given in Table 1

COMMENTARY

C6.6.1.2.1

Delete the first paragraph of AC6.6.1.2.1.

SPECIFICATIONS

Table 6.6.1.2.2-1 - Pennsylvania Traffic Factor for Cases I, II and III

Case	Type of Road	ADTT	PTF
I	National Highway System	---	1.2
II	Freeways, Expressways, Major Highways and Streets	> 500	1.2
III	Other Highways and Streets not included in Case I or II	---	1.0

6.6.1.2.4 Restricted Use Details

The following shall supplement A6.6.1.2.4.

Details defined as Category D or E in Table A6.6.1.2.3-1 are considered unacceptable for new designs. Such details shall be excluded from new designs, except when approved by the Chief Bridge Engineer.

Girder or floorbeam flanges inserted through a slot cut in the web of an intersecting member and then welded to one or both sides of the web to provide continuity are not acceptable. Moreover, such flanges butted flush against the web of the intersecting member and then welded to it are unacceptable.

Details involving the intersection of the flange of one girder with the web of another girder are unacceptable because a significant embedded crack-like interface may remain between members after the welding. Such a defect can quickly propagate, causing premature failure.

6.6.1.2.5 Fatigue Resistance

Replace all references to A 325M and A 490M with A 325 and A 490, respectively.

The following shall replace Equation A6.6.1.2.5-2.

$$N = (365)\,(100)\,n\,(ADTT)_{SL} \qquad (6.6.1.2.5\text{-}2P)$$

6.6.1.3 DISTORTION-INDUCED FATIGUE

The following shall replace the second paragraph of A6.6.1.3.

To control web buckling and elastic flexing of the web, the provision of A6.10.5.3, D6.10.5.3 and DE6.10.6P shall be satisfied.

COMMENTARY

C6.6.1.2.4P

C6.6.1.2.5

The following shall replace fourth paragraph of AC6.6.1.2.5.

PennDOT's design life is considered to be 100 years. In the overall development of the LRFD Specification, the design life has been considered to be 75 years. This is the reason that the 75 in Equation A6.6.1.2.5-2 is replaced with 100 in Equation D6.6.1.2.5-2P.

C6.6.1.3

The following shall supplement AC6.6.1.3.

The interaction of primary and secondary components of steel bridge structures often results in cracking at unexpected locations in relatively short periods of time. Such cracking was first observed in the webs of girder-type bridges at short gaps between transverse web attachments and the girder flanges. Investigations of this type of crack

SPECIFICATIONS

COMMENTARY

development suggest that the cracking is typical and is caused by out-of-plane displacements which result in large secondary web bending stresses. This is evident in the in-depth case studies presented by Mertz (1984).

Fatigue crack growth resulting from displacement-induced secondary stresses is difficult to anticipate, since it involves the actual behavior of a structure, rather than the assumed behavior. The differences between the actual and the assumed behavior are most critical at very localized regions, such as at the ends of cut-short transverse connection plates. The present design idealization does not account for such localized behavior.

6.6.1.3.4P Distortion-Induced Fatigue: Unacceptable Details and Acceptable Alternative Details

Members and fasteners shall be detailed to reduce the effect of repeated variations or reversals of stress due to out-of-plane deformations or secondary forces. Examples of details which have proven to be unacceptable, based upon this criteria, are shown in Figure 1. Acceptable alternatives to these unacceptable details are shown in Figure 2. These details do not include all possible variations of distortion-sensitive details, but they are considered typical and will provide guidance.

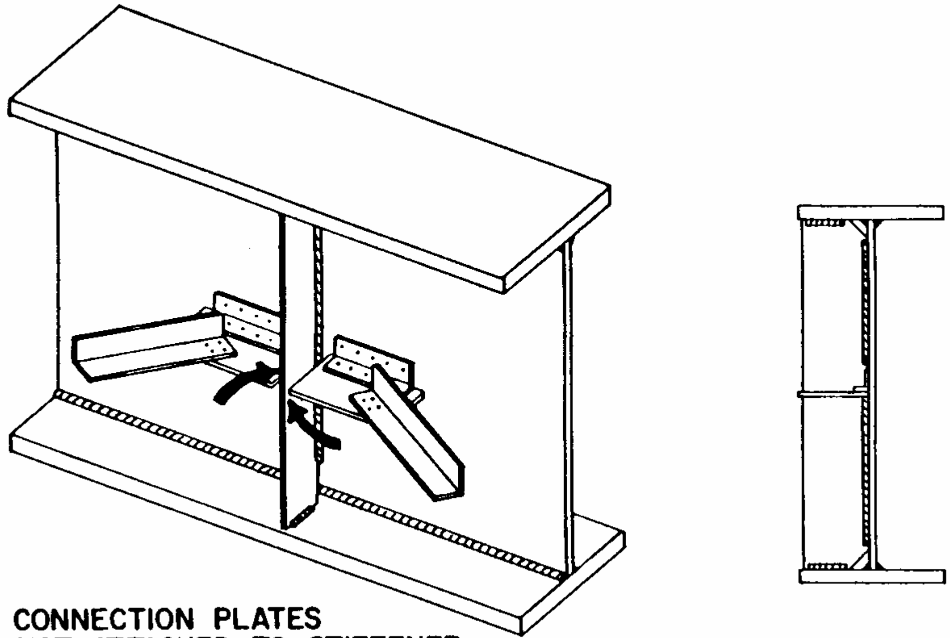
Lateral gusset plates near transverse stiffeners or coped around transverse stiffeners shall be rigidly attached to the transverse stiffener (either bolted or welded). If this rigid attachment is not provided, the potential for localized out-of-plane distortion cracking of the web is created near the juncture of the web and transverse stiffener.

If lateral bracing is required, the preferred approach is to attach the gusset plate to the flange as shown in BD-620M and BC-754M. Welding of the gusset plate to the stiffener must be detailed to prevent intersecting walls.

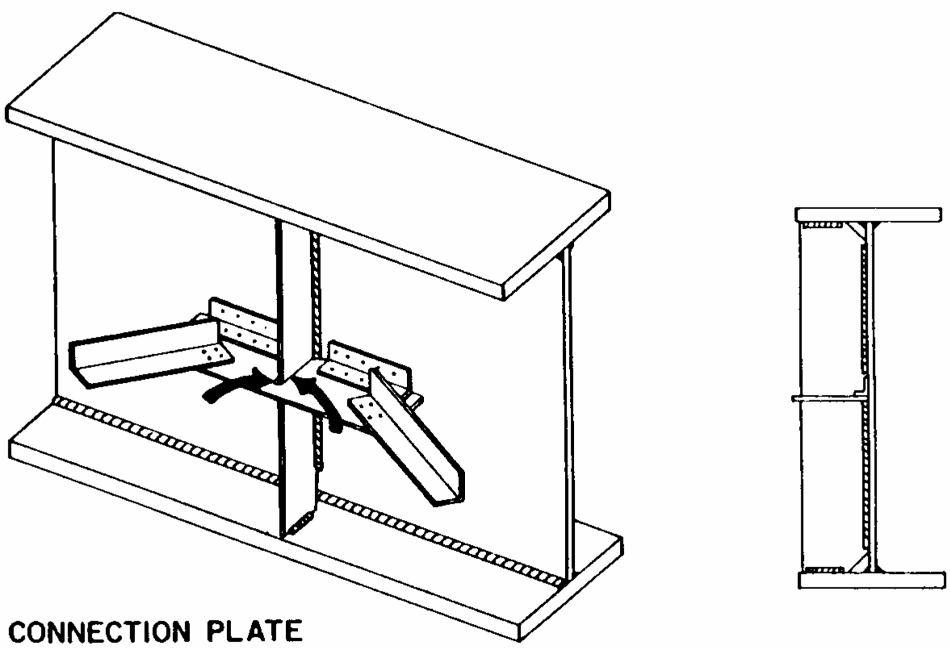
C6.6.1.3.4P

Rather than attempting to quantify the displacement-induced stresses and develop allowable values, it is the Department's philosophy that details susceptible to out-of-plane distortion are not acceptable. Through the design of better details, the inadequacy of the present design idealization in dealing with displacement-induced stresses is minimized.

Connection plates for either diaphragms or floorbeams shall be rigidly attached to both girder flanges (either bolted or welded). Cutting the connection plate short or merely providing a tight fit to the flange is not acceptable, since the potential for localized out-of-plane distortion cracking of the web exists near the juncture of the web and flange.

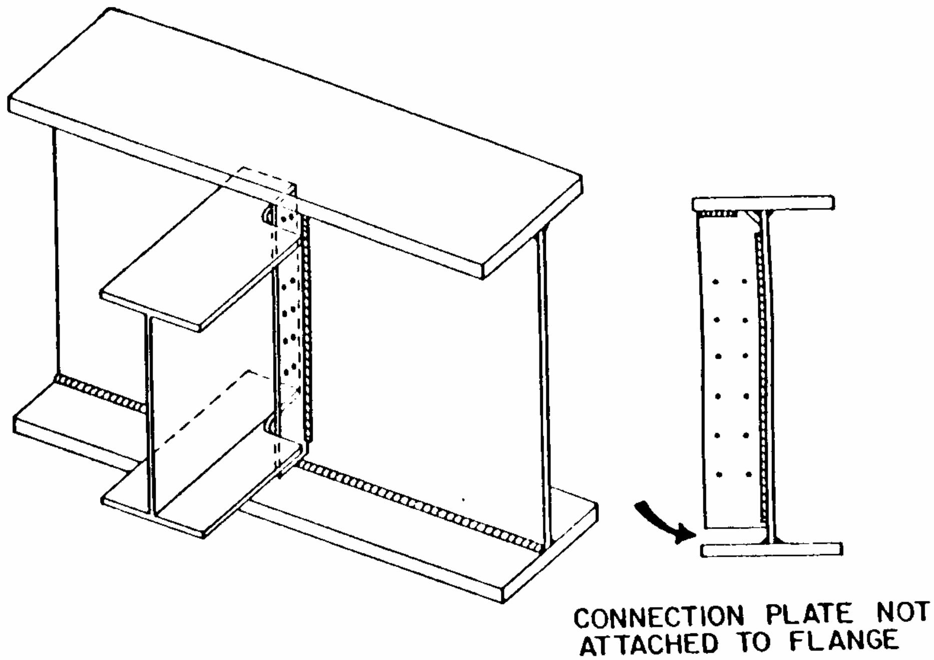


(a.) Lateral Connection Plate at Transverse Stiffener

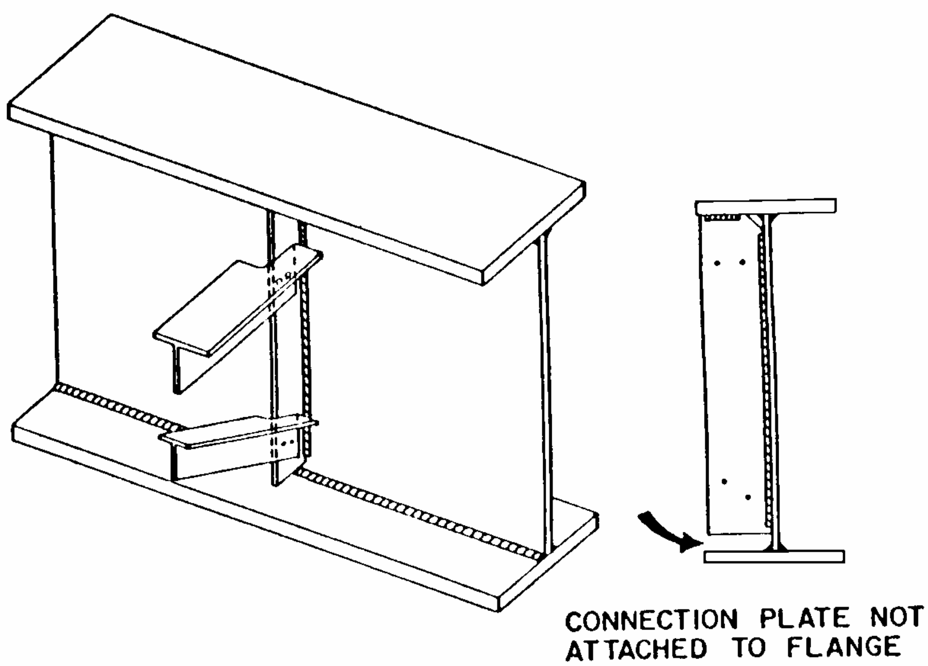


(b.) Lateral Connection Plate at Transverse Stiffener

Figure 6.6.1.3.4P-1 - Unacceptable Details

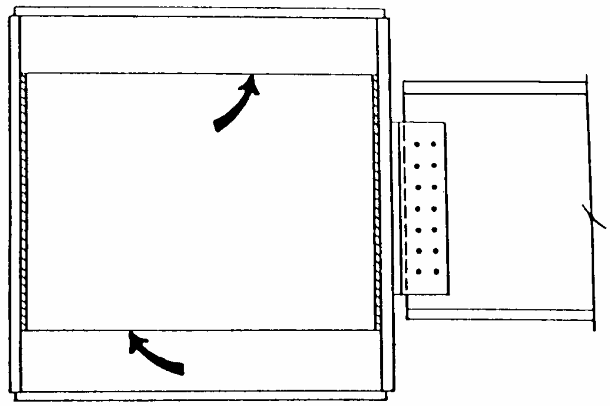


(c.) Girder- floorbeam Connection

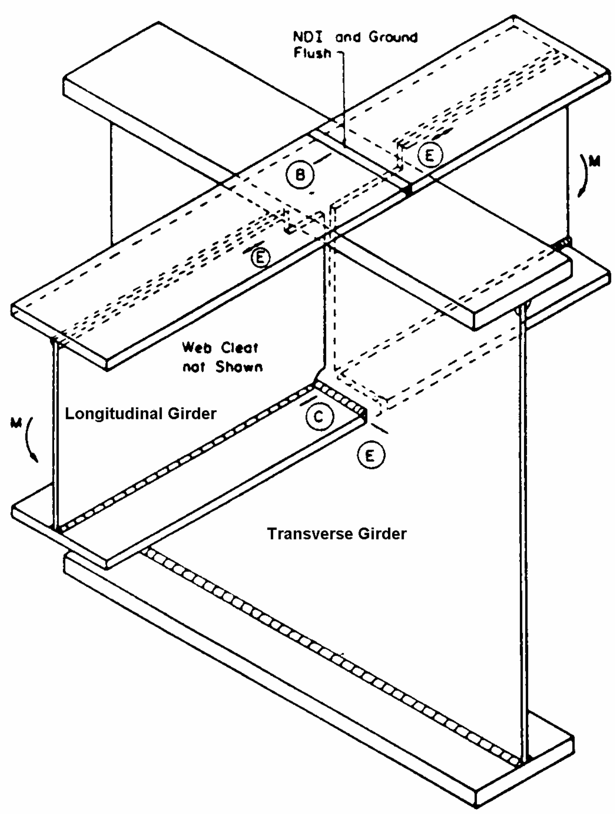


(d.) Girder-Cross - Frame Connection

Figure 6.6.1.3.4P-1 - Unacceptable Details (Continued)

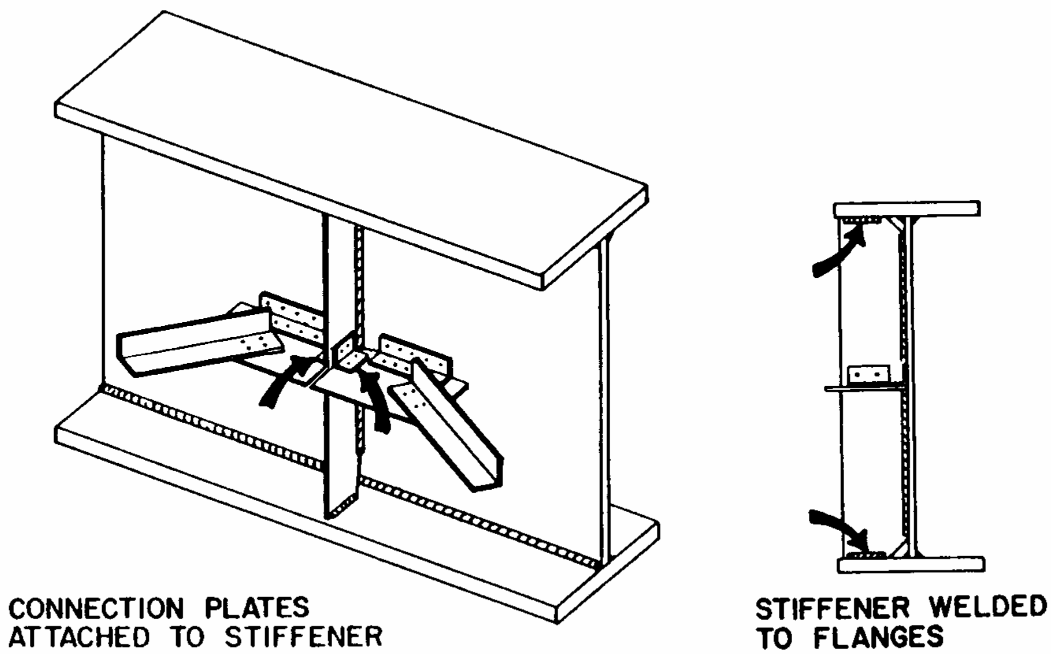


(e.) Internal Box Girder Diaphragm

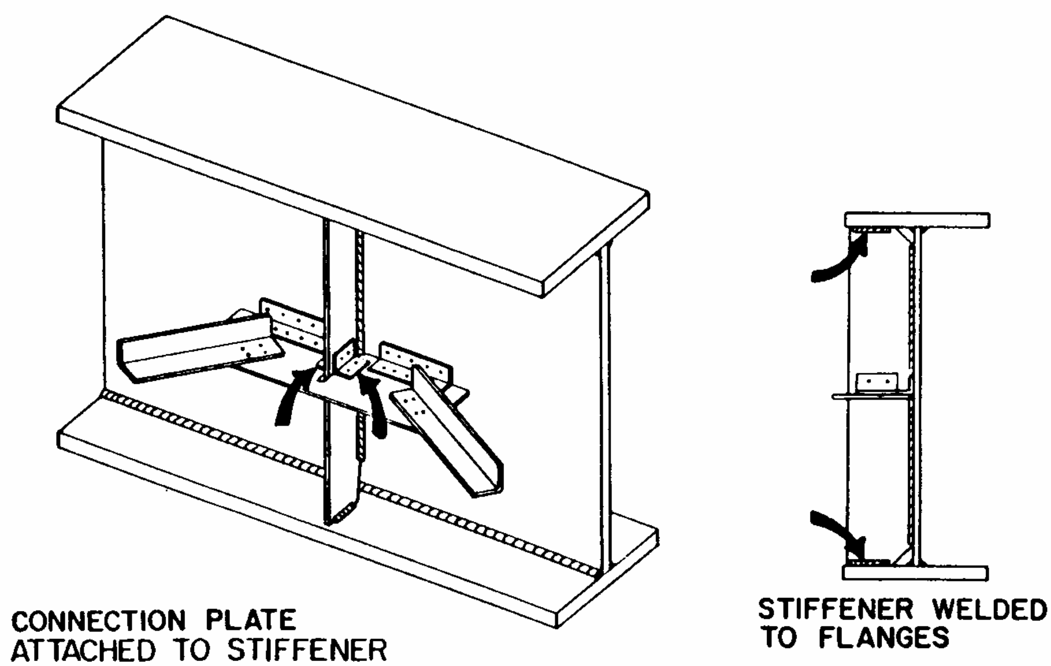


(f.) Longitudinal - to - Transverse Girder Connection

Figure 6.6.1.3.4P-1 - Unacceptable Details (Continued)

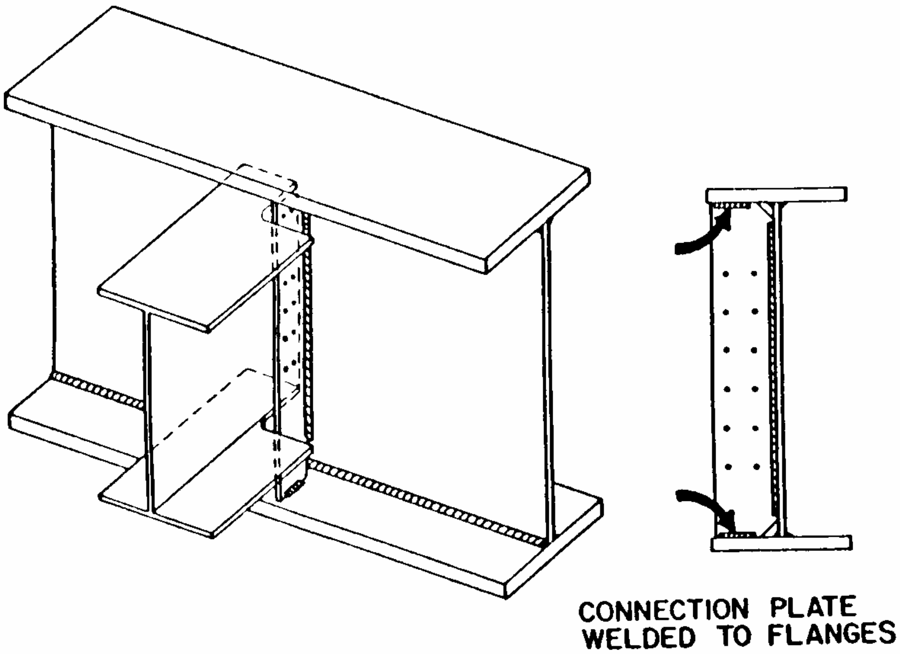


(a.) Lateral Connection Plate at Transverse Stiffener

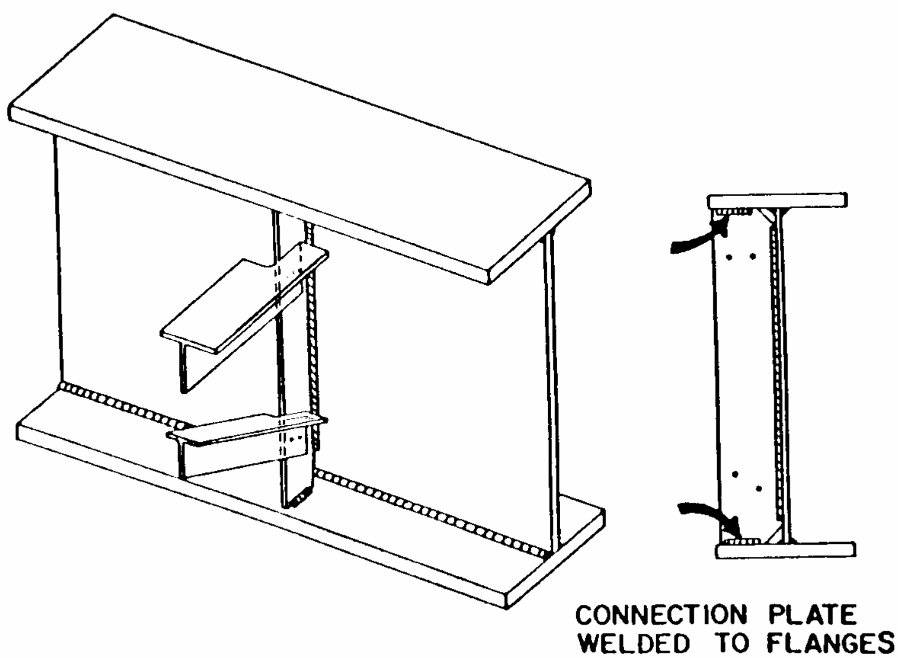


(b.) Lateral Connection Plate at Transverse Stiffener

Figure 6.6.1.3.4P-2 - Acceptable Alternatives Details

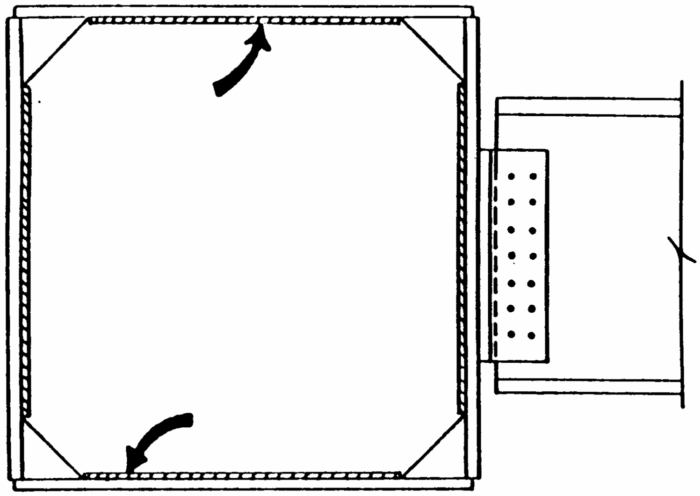


(c.) Girder Floorbeam Connection

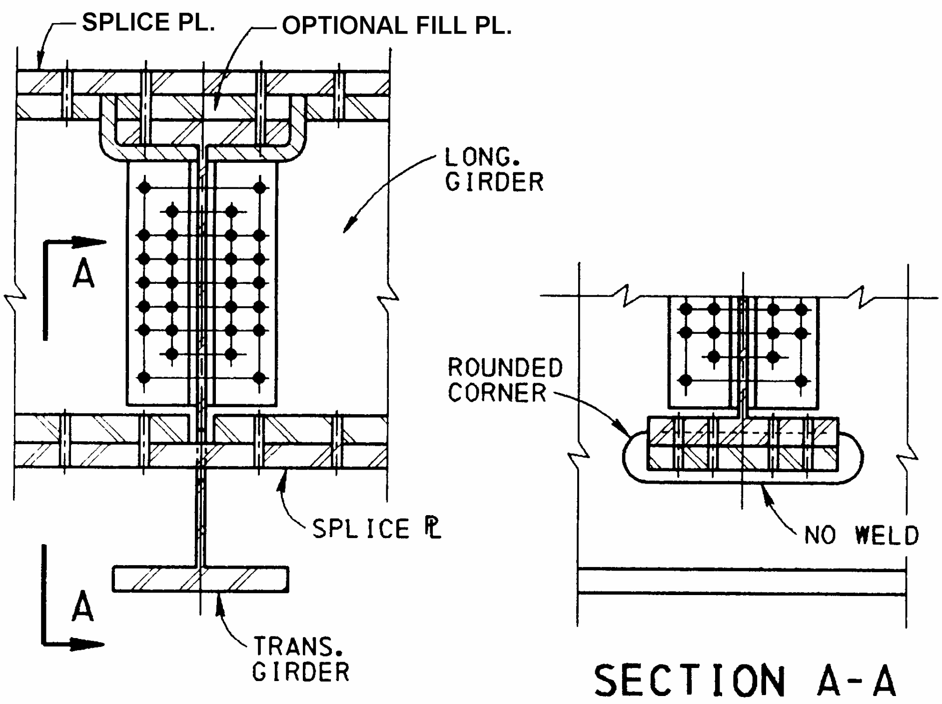


(d.) Girder - Cross-Frame Connection

Figure 6.6.1.3.4P-2 - Acceptable Alternatives Details (Continued)



(e.) Internal box Girder Diaphragm



(f.) Slotted Component Through Which Another Component Passes.

Figure 6.6.1.3.4P-2 - Acceptable Alternatives Details (Continued)

SPECIFICATIONS

6.6.2 Fracture

The following shall supplement A6.6.2.

Charpy V-Notch tests shall be performed as specified as per Publication 408, Section 1105.02(a)4. Diaphragms, cross-frames, bracing and connecting plates for curved girder bridges, straight girder bridges with skew less than 70°, or connections which are entirely welded and without any bolting are to be Charpy V-Notch tested. Typical shop welded, field bolted diaphragms on straight bridges do not require Charpy V-Notch testing (unless bridge skew is less than 70°). Under full dead load, beam ends and all bearing stiffeners, including bearing stiffeners at piers, are to be vertical.

6.7 GENERAL DIMENSION AND DETAIL REQUIREMENTS

6.7.2 Dead Load Camber

The following shall supplement A6.7.2.

Camber is provided for the beams so that after all the dead loads (not including the future wearing surface) are applied, the beam is at the proper elevation. Camber is not used for the control of live load deflections.

For curved girders, the designer shall add the following note in the General Note section of the plans:

"Girder webs shall be plumb under the full dead load existing at the end of construction"

6.7.2.1P CAMBER DUE TO WEIGHT OF DECK SLAB

The camber, due to the weight of deck slab, shall be determined from an analysis in which the weight of the deck slab is applied all at once.

6.7.2.2P CAMBER DETAILS FOR DESIGN DRAWINGS

A diagram and a table of camber ordinates (see Figure 1) shall be shown on the contract plans. Ordinates shall be provided for all beams at one-tenth points and at field splice points (at dead load points of contraflexure if field splices are not provided) to account for the following:

(a) Weight of steel

(b) Weight of deck slab (see D6.7.2.1P)

(c) Superimposed dead load (do not include future wearing surface)

(d) Vertical curve

(e) Superelevation

COMMENTARY

C6.7.2P

Full dead load includes the weight of pavement or overlays included in the initial construction. It does not include the future wearing surface.

SPECIFICATIONS COMMENTARY

(f) 50% of heat curve camber (see D6.7.2.3P)

(g) Total due to above

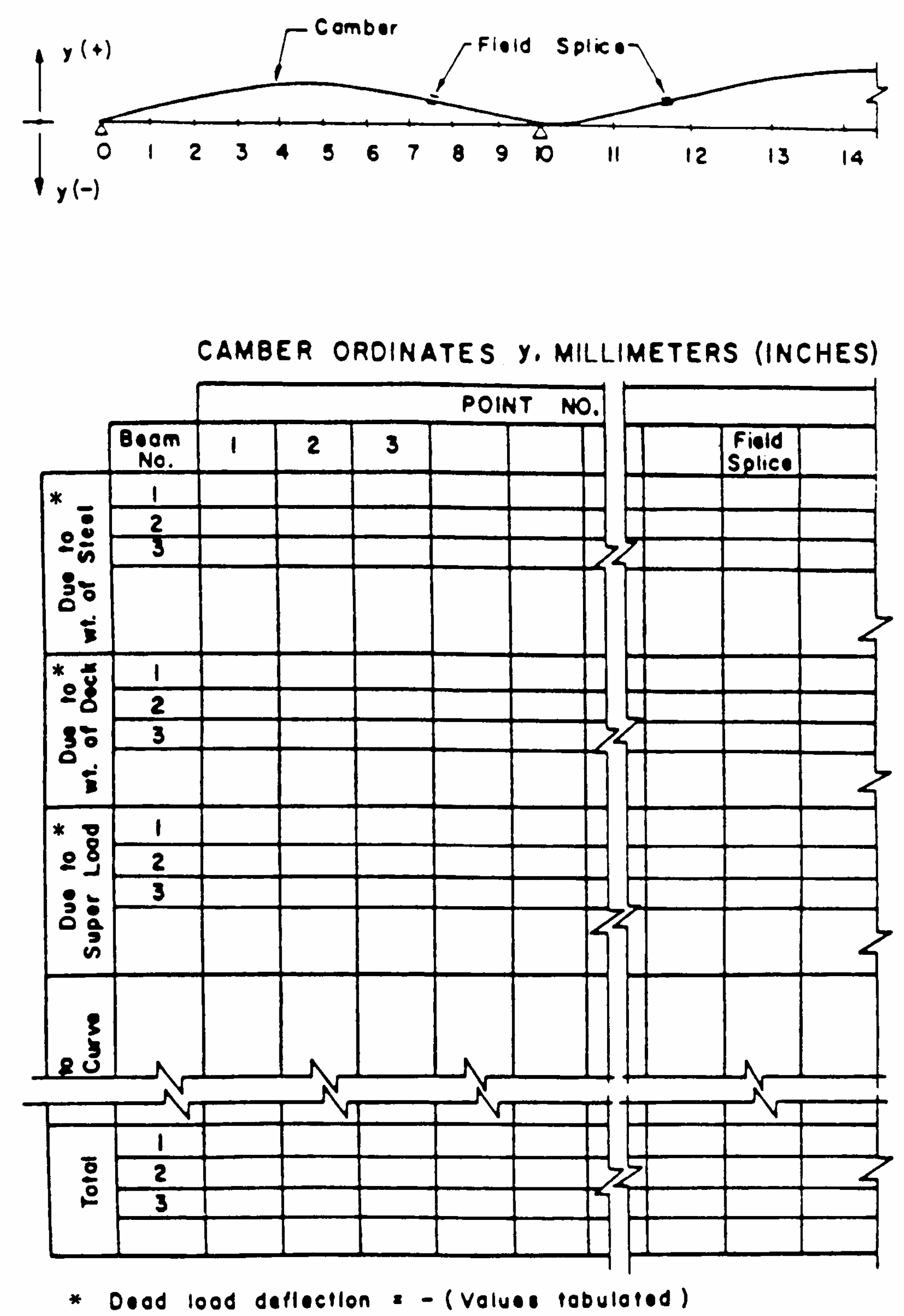


Figure 6.7.2.2P-1 - Camber Details

SPECIFICATIONS

When total camber is less than the minimum that can be maintained in a beam (WF sections), no camber is required, but the following note shall be shown on the contract plans:

> *"Beams shall be placed with any mill camber up; the contractor shall consider and compensate for dead load deflection, due to the weight of the concrete, when forming and constructing the deck slab."*

Designers shall show theoretical dead load deflection data on plans even when no special camber is to be fabricated into the beams (i.e., when using mill camber), since this information is required by the contractor to construct the deck to the correct finished deck elevation.

The requirements for cross-section elevations at 3000 mm {10 ft.} intervals along the length of girder bridges are found in PP1.6.4.10.

6.7.2.3P HEAT-CURVE CAMBER CORRECTIONS

To compensate for possible loss of camber of heat-curved girders in service as residual stresses dissipate, the amount of camber in mm {in.}, Δ, at any section along the length of the girder shall be equal to:

$$\Delta = \frac{\Delta_{DL}}{\Delta_M}\left(\Delta_M + \Delta_R\right) \qquad (6.7.2.3P\text{-}1)$$

where:

For R < 305 000 mm {1,000 ft.}

Metric Units:

$$\Delta_R = \frac{0.02\,L^2 F_y}{E\,Y_o}\left(\frac{305\,000 - R}{260\,000}\right) \qquad (6.7.2.3P\text{-}2)$$

U.S. Customary Units:

$$\Delta_R = \frac{0.02\,L^2 F_y}{E\,Y_o}\left(\frac{1{,}000 - R}{850}\right)$$

For R > 305 000 mm {1,000 ft.}

Δ_R = 0

R = radius of curvature of curved girder (mm) {ft.}

E = modulus of elasticity of the girder flange (MPa) {ksi}

COMMENTARY

C6.7.2.3P

Part of the heat-curve camber loss is attributable to construction loads and will occur during construction of the bridge; total heat-curve camber loss will be complete after several months of in-service loads. Therefore, a portion of the heat-curve camber increase (approximately 50%) should be included in the bridge profile. Camber losses of this nature (but generally smaller in magnitude) are also known to occur in straight beams and girders.

SPECIFICATIONS

F_y = specified minimum yield strength of the girder flange (MPa) {ksi}

Y_o = distance from the neutral axis to the extreme outer fiber (mm) {in.}

L = span length for simple span; the distance between a simple end support and the dead load contraflexure point or the distance between points of dead load contraflexure points for continuous spans (mm) {in.}

Δ_{DL} = camber at any point along L (mm) {in.}

Δ_M = maximum value of Δ_{DL} within L (mm) {in.}

Heat-curve camber loss between dead load contraflexure points adjacent to piers is small and may be neglected.

6.7.3 Minimum Thickness of Steel

The following shall replace the first two paragraphs of A6.7.3.

Structural steel (including bracing, cross-frames, and all types of gusset plates), except for webs of certain rolled shapes, closed ribs in orthotropic decks, fillers and in railings shall not be less than 10 mm {3/8 in.} in thickness. For girders, the minimum flange plate thickness shall be 20 mm {3/4 in.} unless the fabricator can demonstrate the ability to satisfactorily fabricate and erect plate girders with thinner flange plates. For girders with longitudinal stiffeners, the minimum web thickness shall be 12 mm {1/2 in.}. The web thickness of rolled beams or channels shall not be less than 6 mm {0.23 in.}. The thickness of closed ribs in orthotropic decks shall not be less than 5 mm {3/16 in.}.

For girder flanges, bearing stiffeners and splice plates for bridges that are to be metallized, the width of the plates are to be oversized by 3 mm {1/8 in.} to account for edge grinding. The flange, bearing stiffener plates and splice plates to be shown on the plans shall be the oversized plates. For metallized bridges, the estimated quantity of fabricated structural steel shall be based on the oversized plates.

6.7.4 Diaphragms and Cross-Frames

6.7.4.1 GENERAL

The following shall supplement the first paragraph of A6.7.4.1.

The maximum spacing of cross-frames or diaphragms shall be 7600 mm {25 ft.}.

The following shall supplement A6.7.4.1.

Skew effects must be considered when designing

COMMENTARY

C6.7.3P

This requirement of minimum web thickness for girders with longitudinal stiffeners was added to avoid web buckling and oil canning of deep girders. PennDOT has previously used 10 mm {3/8 in.} thickness resulting in web oil canning effect, specifically on I-476 over Conestoga Avenue. The New York Department of Transportation has successfully used the specified criteria.

For metallized bridges, the rolled edges of angles, channels and wide flange beams do not require edge grinding, therefore these components are not to be oversized.

C6.7.4.1

The following shall supplement AC6.7.4.1.

SPECIFICATIONS

diaphragms, especially when the skew angle is less than 70°. Proper consideration of unbraced length and diaphragm loads from non-uniform deflections is mandatory. Design calculations must consider the fact that cross-frames in skewed bridges connect different points of the span of adjacent girders and that these points will not deflect the same amount. Therefore, a check considering these differences must be made, and the resulting design forces must be used in the cross-frame design.

For sharply skewed bridges (typically, skews less than or equal to 60°), a cross-frame or diaphragm normal to the girder shall be located such as to minimize the effects of differential deflections, while satisfying the minimum cross-frame or diaphragm spacing requirement.

For additional analysis criteria for bridges with skew angles less than 70°, see D4.6.2.2.1.

Diaphragm and cross-frame members in horizontally curved bridges shall be considered primary members.

COMMENTARY

In locating intermediate cross-frames or diaphragms in sharply skewed brides, the designer must consider distinct issues associated with each girder connected by the cross-frame or diaphragm. A cross-frame or diaphragm close to the bearing on one girder line may introduce forces into the system (cross-frame or diaphragm and girder flange) due to “nuisance stiffness,” where the deflection of one girder line cannot match the adjacent girder line. In these cases, elimination of a cross-frame or diaphragm is advisable. In addition, the initial cross-frame or diaphragm must be located such that the maximum permitted spacing is not exceeded in the adjacent connected girder. In some cases, the first interior line of cross-frames or diaphragms may not be full width across the superstructure, and the number of bays along a girder length may not be constant for each girder in the superstructure.

Bracing of horizontally curved members is more critical than for straight members. Diaphragm and cross-frame members resist forces that are critical to the proper functioning of curved-girder bridges. Since they transmit the forces necessary to provide equilibrium, they are considered primary members. Therefore, forces in the bracing members must be computed and considered in the design of these members. When the girders have been analyzed neglecting the effects of curvature according to the provisions of A4.6.1.2.4, the diaphragms or cross-frames may be analyzed by the V-load method (United States Steel 1984) or other rational means.

6.7.4.2 I-SECTION MEMBERS

The article heading is changed from: Straight I-Sections.

The following shall supplement the first paragraph of A6.7.4.2

Cross-frame members in horizontally curved bridges should contain diagonals and top and bottom chords.

The following shall replace the last two sentences of second paragraph of A6.7.4.2.

When the supports are skewed less than 70°, intermediate cross-frames shall be normal to the main members. If the supports are skewed, end cross-frames need not be co-linear with the line of bearings, see Standard

C6.7.4.2P

End cross-frames must be parallel to centerline of bearings, but need not coincide with bearing line.

SPECIFICATIONS

Drawing BC-754M.

For additional skewed cross-frame requirements, see D6.7.4.1.

The following shall supplement A6.7.4.2

The spacing, L_b, of intermediate diaphragms or cross-frames in horizontally curved I-girder bridges shall not exceed the following in the erected condition:

$$L_b \leq L_r \leq R/10 \qquad (6.7.4.2\text{-}1)$$

where:

L_r = limiting unbraced length determined from Eq. A6.10.8.2.3-5 (mm) {ft.}

R = minimum girder radius within the panel (mm) {ft.}

In no case shall L_b exceed 7500 mm {25 ft}.

6.7.4.3 BOX SECTION MEMBERS

The article heading is changed from: Straight Box Sections.

The following shall replace the last two paragraphs of A6.7.4.3.

Intermediate internal diaphragms or cross-frames shall be provided. For all single box sections, horizontally curved sections, and multiple box sections in cross-sections of bridges not satisfying the requirements of A6.11.2.3 or with box flanges that are not fully effective according to the provisions of A6.11.1.1 and D6.11.1.1 for curved bridges or DE6.11.1.1P for straight bridges, the internal bracing shall be spaced to control cross-section distortion, with the spacing not to exceed 7500 mm {25 ft}.

For all single box sections, horizontally curved sections, and multiple box sections in bridges not satisfying the requirements of A6.11.2.3 or with box flanges that are not fully effective according to the provisions of A6.11.1.1 and D6.11.1.1 for curved bridges or DE6.11.1.1P for straight bridges, the need for a bottom transverse member within the internal bracing shall be considered. Where provided, the transverse member shall be attached to the box flange unless longitudinal flange stiffeners are used, in which case the transverse member shall be attached to the longitudinal stiffeners by bolting. The cross-sectional area and stiffness of the top and bottom internal bracing members shall also not be less than the area and stiffness of the diagonal members.

COMMENTARY

C6.7.4.3

The following shall replace the first sentence of the fourth paragraph of A6.7.4.3

For all horizontally curved box girder bridges, single box sections, and for box sections in bridges not satisfying the requirements of A6.11.2.3 or with box flanges that are not fully effective, cross-sectional distortion stresses are best controlled by the introduction of internal cross-frames or diaphragms.

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6.7.5 Lateral Bracing

6.7.5.2 I-SECTION MEMBERS

The article heading is changed from: Straight I-Sections.

COMMENTARY

C6.7.5.2

The following shall supplement AC6.7.5.2.

Wherever possible, horizontal lateral bracing in or near the plane of the bottom flange should be eliminated from bridges, and the girders should be designed to carry the wind load between diaphragms according to A4.6.2.7.1. Horizontal lateral bracing is relatively expensive because of the detail associated with it. Furthermore, there are often forces associated with horizontal lateral bracing which can result in distortion-induced fatigue; these forces are also a significant factor on steel bridges. Therefore, horizontal lateral bracing should not be considered for the improvement of redundancy.

When horizontal lateral bracing is required, it should be attached to the bottom flange wherever practical. (BD-620M permits attachment to the top flange.)

For horizontally curved bridges, when the curvature is sharp and temporary supports are not practical, it may be desirable to consider providing both top and bottom lateral bracing to ensure pseudo-box action while the bridge is under construction. Top and bottom lateral bracing provides stability to a pair of I-girders.

6.7.5.3 TUB SECTION MEMBERS

The article title is changed from: Straight Tub Sections.

The following shall supplement the first paragraph of A6.7.5.3.

For horizontally curved girders, a full-length lateral bracing system shall be provided.

C6.7.5.3

The following shall replace the last sentence of the third paragraph of A6.7.5.3.

For both straight and horizontally curved tub sections, a full-length lateral bracing system forms a pseudo-box to help limit distortions brought about by temperature changes occurring prior to concrete deck placement, and to resist the torsion and twist caused by any eccentric loads acting on the steel section during construction. AASHTO (1993) specified that diagonal members of the top lateral bracing for tub sections satisfy the following criterion:

$$A_d \geq 0.76w \qquad \text{(C6.7.5.3-1)}$$

where:

A_d = minimum required cross-sectional area of one diagonal (mm^2) {$in.^2$}

w = center-to-center distance between the top flanges (mm) {in.}

Satisfaction of this criterion was intended to ensure that the top lateral bracing would be sized so that the tub would act as a pseudo-box section with minimal warping torsional displacement and normal stresses due to warping torsion less than or equal to 10 percent of the major-axis bending

SPECIFICATIONS

COMMENTARY

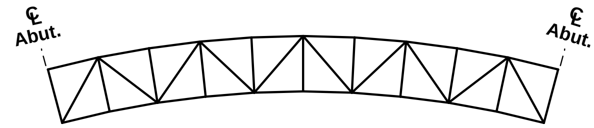

Figure C6.7.5.3-1 Warren-Type Single-Diagonal Top Lateral Bracing System for Tub Section Member: Plan View.

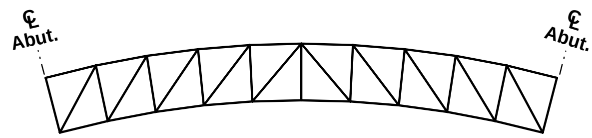

Figure C6.7.5.3-2 Pratt-Type Single-Diagonal Top Lateral Bracing System for Tub Section Member: Plan View.

Where the forces in the bracing members are not available from a refined analysis, the shear flow across the top of the pseudo-box section can be computed from Eq. C6.11.1.1-1 assuming the top lateral bracing acts as an equivalent plate. The resulting shear can then be computed by multiplying the resulting shear flow by the width w, and the shear can then be resolved into the diagonal bracing member(s). Should it become necessary for any reason to compute the St. Venant torsional stiffness of the pseudo-box section according to Eq. AC6.7.4.3-1, formulas are available (Kollbrunner and Basler 1966; Dabrowski 1968) to calculate the thickness of the equivalent plate for different possible configurations of top lateral bracing.

6.8 TENSION MEMBERS

6.8.2 Tensile Resistance

6.8.2.2 REDUCTION FACTOR, U

The following shall replace the first paragraph of A6.8.2.2.

The reduction factors, specified in A6.8.2.2, shall be used to account for shear lag. Reduction factors developed from refined analysis or tests may be used if approved by the Chief Bridge Engineer.

6.8.2.3 COMBINED TENSION AND FLEXURE

The following shall replace the definition of M_{rx} in A6.8.2.3.

M_{rx} = factored flexural resistance about the x-axis taken as ϕ_f times the nominal flexural resistance about

SPECIFICATIONS

the x-axis determined as specified in DE6.10P, DE6.11P or A6.12, as applicable (N-mm) {kip-in.}

6.8.3 Net Area

The following shall replace the first paragraph of A6.8.3.

The net area, A_n, of an element is the product of the thickness of the element and its smallest net width. The net area, A_n, of a member is the sum of the net areas of each element. The width deducted for all holes; standard, oversize,

and slotted; shall be taken as 1.6 mm {1/16 in.} greater than the hole size specified in D6.13.2.4.2. The net width shall be determined for each chain of holes extending across the member or element along any transverse, diagonal, or zigzag line.

6.9 COMPRESSION MEMBERS

6.9.2 Compressive Resistance

6.9.2.2 COMBINED AXIAL COMPRESSION AND FLEXURE

The following shall replace the definition of M_{rx} in A6.8.2.3.

M_{rx} = factored flexural resistance about the x-axis taken equal to ϕ_f times the nominal flexural resistance about the x-axis determined as specified in DE6.10P, DE6.11P or A6.12, as applicable (N-mm) {kip-in.}

6.9.5 Composite Members

6.9.5.1 Nominal Compressive Resistance

The following shall replace the definition of n in A6.9.5.1.

n = modular ratio of the concrete as specified in D5.4.2.1

6.10 I-SECTIONS IN FLEXURE

6.10.0P Applicable Provisions

The provisions of D6.10 apply to steel I-girders curved in plan. The provisions of Appendix E apply to straight girder bridges.

COMMENTARY

C6.8.3

Delete the first paragraph of AC6.8.3.

C6.10

C6.10.0P

The provisions of D6.10 are based on the 2004 Third Edition of AASHTO-LRFD specifications as amended herein to account for the curved girder provisions that were incorporated in AASHTO LRFD by the 2005 interim

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Where Straight girders are referred to in A6.10 and D6.10, this reference should be ignored.

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specifications. The provisions of Appendix E are based on the 1998 Second Edition of the AASHTO-LRFD Design specifications with the 1999 through 2003 Interims. Using the earlier specifications for straight girder bridges will allow continuing using the existing STLRFD computer program, which is based on the Second Edition of AASHTO LRFD with modifications, with relatively few revisions. It is the intent of the Department to update the STLRFD computer program to correspond to the 2004 AASHTO-LRFD, including A6.10 and A6.11, at some point in the future. At which time, both straight and curved girders will be designed using the provisions of the 2004 Third Edition of the Specifications as amended by Design Manual Part 4. Any reference to straight girder bridges in D6.10 is meant to be used in the future when the Design Manual Part 4 and computer programs are fully updated to the third edition of the AASHTO-LRFD specifications.

6.10.1 General

The following shall replace the first sentence in A6.10.1

The provisions of this article apply to flexure of rolled or fabricated kinked (chorded) continuous or horizontally curved steel I-section members symmetrical about the vertical axis in the plane of the web.

C6.10.1

The following shall replace the first paragraph of AC6.10.1.

This article addresses general topics that apply to all types of steel I-sections in horizontally curved bridges, or bridges containing both straight and curved segments. For the application of the provisions of A6.10 and D6.10, bridges containing both straight and curved segments are to be treated as horizontally curved bridges since the effects of curvature on the support reactions and girder deflections, as well as the effects of flange lateral bending, usually extend beyond the curved segments. Note that kinked (chorded) girders exhibit the same actions as curved girders, except that the effect of the noncollinearity of the flanges is concentrated at the kinks. Continuous kinked (chorded) girders should be treated as horizontally curved girders with respect to these Specifications.

The following shall supplement AC6.10.1.

For horizontally curved bridges, in addition to the potential sources of flange lateral bending discussed in the preceding paragraph, flange lateral bending effects due to curvature must always be considered at all limit states and also during construction.

Delete the reference to Appendix B from the first sentence of the second Paragraph of AC6.10.1.

The following shall replace the second sentence of the second paragraph of A6.10.1.

For the majority of straight non-skewed bridges, flange lateral bending effects tend to be most significant during construction and tend to be insignificant in the final constructed condition. Significant flange lateral bending may be caused by wind, by torsion from eccentric concrete deck overhang loads acting on cantilever forming brackets placed along exterior girders, and by the use of staggered cross-frames in conjunction with skews less than 70°.

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The following shall supplement A6.10.1.

Open-framed systems are those which have no horizontal lateral bracing in or near the plane of the bottom flange. Lateral bracing, when used, is provided to resist wind loads, but it is generally not needed since the girders can be designed to carry wind loads between the diaphragms.

If horizontal lateral bracing is included, the open-framed system distribution factors shall be used. If a horizontal lateral bracing system is used, the connections must be detailed to ensure that the fatigue life of the bracing system is at least that of the girder.

Although the lateral wind bracing may not be required for the final constructed condition, the need for lateral wind bracing during construction shall be investigated.

6.10.1.1.1 Stresses

The following shall supplement A6.10.1.1.1

If concrete with expansive characteristics (except shrinkage-compensating concrete, which the Department is studying and may eventually exclude from this provision) is used, composite design shall be used with caution, and provision must be made in the design to accommodate the expansion.

Composite section properties (see D6.10.3.1.1b) shall be assumed in the positive and negative moment regions for the calculation of design moments, shears and deflections.

6.10.1.1.1a Sequence of Loading

The following shall replace last paragraph of A6.10.1.1.1a

For unshored construction, permanent load applied before the concrete deck has attained 75% of its compressive strength shall be assumed carried by the steel section alone; permanent load and live load applied after this stage shall be assumed carried by the composite section. For shored construction, all permanent loads shall be assumed applied after the concrete deck has hardened or has been made composite and the contract documents shall so indicate.

Use of shored systems requires the prior approval of the Chief Bridge Engineer.

For continuous spans, the final dead load moment at each design section shall be taken as the greater of either the dead load moment considering the weight of the concrete deck to be instantaneously applied or a moment based upon

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The application of open-framed system distribution factors for closed-framed systems is generally conservative.

If horizontal lateral bracing system is used, a rational analysis may consider a reduction in lateral live load distribution factor due to the quasi-box action of the closed-frame system.

Any reduction in live load distribution factor must be approved by the Chief Bridge Engineer.

The design procedure for evaluating the need for lateral bracing during construction shall be per BD-620M. As agreed upon by the APC Subcommittee for Steel Bridge Superstructures, the contractor is responsible for stability of the girders during erection, including providing wind bracing during erection as needed. This responsibility includes the analysis, design, material, fabrication and installation (and removal) of wind bracing during erection at no cost to the Department.

C6.10.1.1.1P

If the concrete is expansive, estimate expansion and properly design concrete to flange connection by adding additional shear studs.

C6.10.1.1.1a

Delete the first paragraph of AC6.10.1.1.1

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an incremental analysis of the specified slab placement sequence. Similarly, stresses should be computed based on the more critical of the incremental and instantaneously applied loads.

6.10.1.1.1b Stresses for Sections in Positive Flexure

Delete Equation A6.10.1.1.1b-1

The following shall supplement A6.10.1.1.1b:

For normal and low density concrete, the modular ratio is given in D5.4.2.1.

6.10.1.1.1c Stresses for Sections in Negative Flexure

The following shall replace A6.10.1.1.1c

For calculating flexural stresses in sections subjected to negative flexure, the composite section for both short-term and long-term moments shall consist of the steel section and the longitudinal reinforcement within the effective width of the concrete deck.

Cut-off points for the main reinforcement in cast-in-place decks over interior supports for continuity may be staggered as required by design.

Concrete on the tension side of the neutral axis shall not be considered in calculating resisting moments.

6.10.1.1.1d Concrete Deck Stresses

The following shall replace A6.10.1.1.1d

For calculating longitudinal flexural stresses in the concrete deck due to transient loads, the short-term modular ratio, n, shall be used. For calculating longitudinal flexural stresses in the concrete deck due to permanent loads, the long-term modular ratio, 3n, shall be used.

6.10.1.1.1.fP Lateral Support of Top Flanges Supporting Timber Decks

The compression flanges of girders supporting timber floors shall not be considered to be laterally supported by the flooring, unless the floor and fastenings are specially designed to provide such support. Laminated timber decks shall be provided with steel clips designed to furnish adequate lateral support to the top flange.

6.10.1.2 NONCOMPOSITE SECTIONS

The following shall supplement A6.10.1.2

Whenever technically feasible, all structures shall be made composite.

COMMENTARY

C6.10.1.1.1b

The following shall replace AC6.10.1.1.1b

It is preferable to proportion composite sections in simple spans and the positive moment regions of continuous spans so that the neutral axis lies below the top surface of the steel beam.

C6.10.1.1.1d

Delete AC6.10.1.1.1d

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6.10.1.3 HYBRID SECTIONS

The following shall supplement A6.10.1.3.

The use of girders with web yield strength higher than the flange yield strength requires the prior approval of the Chief Bridge Engineer.

6.10.1.4 VARIABLE WEB DEPTH MEMBERS

The following shall supplement A6.10.1.4.

The use of girders with variable web depth requires the prior approval of the Chief Bridge Engineer.

6.10.1.5 STIFFNESS

The following shall supplement A6.10.1.5

In the computation of flexural stiffness and flexural resistance of beams, the haunch shall be taken as zero. However, in the computation of dead load, the haunch shall be taken into account.

6.10.1.6 FLANGE STRESSES AND MEMBER BENDING MOMENT

The following shall replace the forth paragraph of A6.10.1.6 to the end of the article.

Lateral bending stresses in continuously braced flanges shall be taken equal to zero. Lateral bending stresses in discretely braced flanges shall be determined by structural analysis. All discretely braced flanges shall satisfy:

$$f_\ell \leq 0.6F_{yf} \qquad (6.10.1.6\text{-}1)$$

The flange lateral bending stress, f_ℓ, may be determined directly from first-order elastic analysis in discretely braced compression flanges for which:

$$L_b \leq 1.2L_p\sqrt{\frac{C_bR_b}{f_{bu}/F_{yc}}} \qquad (6.10.1.6\text{-}2)$$

COMMENTARY

C6.10.1.5

The following shall supplement AC6.10.1.5

Field measured haunch depths may be used in the computation for flexural stiffness and resistance when rating existing bridges.

The following shall replace the second paragraph of AC6.10.1.5.

Field tests of composite continuous bridges have shown that there is considerable composite action in negative bending regions (Baldwin et al. 1978; Roeder and Eltvik 1985; Yen et al. 1995). Therefore, the stiffness of the full composite section is to be used over the entire bridge length for the analysis of composite flexural members, but not for stress calculations.

Other stiffness approximations which are based on sound engineering principles may be used if approved by the Chief Bridge Engineer.

C6.10.1.6

The following shall supplement the fourth paragraph of AC6.10.1.6

The determination of flange lateral bending moments due to curvature is addressed in A4.6.1.2.4b.

In all resistance equations, f_{bu}, M_u and f_ℓ are to be taken as positive in sign. However, for service and strength limit state checks at locations where the dead and live load contributions to f_{bu}, M_u or f_ℓ are of opposite sign, the signs of each contribution must be initially taken into account. In such cases, for both dead and live load, the appropriate net sum of the major-axis and lateral bending actions due to the factored loads must be computed, taking the signs into consideration, that will result in the most critical response for the limit state under consideration.

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or equivalently:

$$L_b \leq 1.2 L_p \sqrt{\frac{C_b R_b}{M_u / M_{yc}}} \qquad (6.10.1.6\text{-}3)$$

where:

C_b = moment gradient modifier specified in A6.10.8.2.3, D6.10.8.2.3 or Appendix A Article A6.3.3, as applicable.

f_{bu} = largest value of the compressive stress throughout the unbraced length in the flange under consideration, calculated without consideration of flange lateral bending (MPa) {ksi}

L_b = unbraced length (mm) {in.}

L_p = limiting unbraced length specified in A6.10.8.2.3 and D6.10.8.2.3 (mm) {in.}

M_u = largest value of the major-axis bending moment throughout the unbraced length causing compression in the flange under consideration (N-mm) {kip-in.}

M_{yc}= yield moment with respect to the compression flange determined as specified in AD6.2 (N-mm) {kip-in.}

R_b = web load-shedding factor determined as specified in A6.10.1.10.2

If Eq. 2, or Eq. 3 as applicable, is not satisfied, second-order elastic compression-flange lateral bending stresses shall be determined.

Second-order compression-flange lateral bending stresses may be determined by amplifying first-order values as follows:

$$f_\ell = \left(\frac{0.85}{1 - \frac{f_{bu}}{F_{cr}}} \right) f_{\ell 1} \geq f_{\ell 1} \qquad (6.10.1.6\text{-}4)$$

or equivalently:

$$f_\ell = \left(\frac{0.85}{1 - \frac{M_u}{F_{cr} S_{xc}}} \right) f_{\ell 1} \geq f_{\ell 1} \qquad (6.10.1.6\text{-}5)$$

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where:

f_{bu} = largest value of the compressive stress throughout the unbraced length in the flange under consideration, calculated without consideration of flange lateral bending (MPa) {ksi}

$f_{\ell 1}$ = first-order compression-flange lateral bending stress at the section under consideration, or the maximum first-order lateral bending stress in the compression flange under consideration throughout the unbraced length, as applicable (MPa) {ksi}

Fcr = elastic lateral torsional buckling stress for the flange under consideration determined from Eq. 6.10.8.2.3-8 or Eq. AA6.3.3-8. Eq. AA6.3.3-8 may only be applied for unbraced lengths in straight I-girder bridges in which the web is compact or noncompact.

M_u = largest value of the major-axis bending moment throughout the unbraced length causing compression in the flange under consideration (N-mm) {kip-in.}

S_{xc} = elastic section modulus about the major axis of the section to the compression flange taken as M_{yc}/F_{yc} (mm^3) {$in.^3$}

6.10.1.7 MINIMUM NEGATIVE FLEXURE CONCRETE DECK REINFORCEMENT

The following shall replace the first paragraph of A6.10.1.7

In negative flexure regions of any continuous span, the total cross-sectional area of the longitudinal reinforcement shall not be less than 1 percent of the total cross-sectional area of the slab. The reinforcement used to satisfy this requirement shall have a specified minimum yield strength not less than 420 MPa {60 ksi} and a size not exceeding No. 19 bars {No. 6}.

The required reinforcement shall be placed in two layers uniformly distributed across the slab width, and two-thirds shall be placed in the top layer. The individual bars shall be spaced at intervals not exceeding 300 mm {12 in} within each row.

Shear connectors shall be provided along the entire length of the girder to develop stresses on the plane joining the concrete and steel in accordance with A6.10.10 and D6.10.10.

COMMENTARY

C6.10.1.7

Delete the second paragraph and last paragraph of A6.10.1.7

The following shall replace the last sentence in the paragraph before last in CA6.10.1.7.

The above applies for members that are designed by the provisions of Article A6.10, Article D6.10 or Appendix A.

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6.10.1.11P LATERAL SUPPORT OF TOP FLANGES SUPPORTING TIMBER DECKS

The compression flanges of girders supporting timber floors shall not be considered to be laterally supported by the flooring, unless the floor and fastenings are specially designed to provide such support. Laminated timber decks shall be provided with steel clips designed to furnish adequate lateral support to the top flange.

6.10.3 Constructibility

6.10.3.2 FLEXURE

6.10.3.2.1 Discretely Braced Flanges in Compression

The following shall replace the definition of Fnc following Equation A6.10.3.2.1-3.

F_{nc} = nominal flexural resistance of the flange (MPa). Fnc shall be determined as specified in A6.10.8.2 and D6.10.8.2.. In computing F_{nc} for constructibility, the web load-shedding factor, R_b, shall be taken as 1.0.

6.10.3.2.5P Erection Analysis

Evaluate lateral deflections in accordance with BD-620M.

6.10.3.2.5.1P Slab Placement

An analysis shall be performed to determine an acceptable slab placement sequence. The analysis shall address (but is not limited to) the following items:

(a) Change in the stiffness in the girder as different segments of the slab are placed and as it affects both the temporary stresses and the potential for "locked-in" erection stresses

(b) Bracing (or lack thereof) of the compression flange of girders and its effect on the stability and strength of the girder

(c) Stability and strength of the girder through slab placement

(d) Bracing of overhang deck forms

(e) Uplift at bearings

COMMENTARY

C6.10.3.2.1

The following shall supplement the fourth paragraph of AC6.10.3.2.1.

For horizontally curved bridges, flange lateral bending effects due to curvature must always be considered in discretely braced flanges during construction.

Delete the eighth paragraph of AC6.10.3.2.1.

C6.10.3.2.5.1P

During the mid-1980's, several of the Department's girder bridges experienced problems during placement of the slab. It is believed that bridges with highly unsymmetrical, deep steel girders combined with wide beam spacing and large overhang dimensions are more susceptible to problems during construction than are the typical earlier steel girder bridges which use more nearly symmetrical steel girders combined with closer beam spacing and smaller overhang dimensions. Since significant reduction in the construction cost of a bridge can be achieved by use of highly unsymmetrical, deep steel girders in conjunction with wide beam spacing and large overhang dimensions, an analysis must be performed to ensure that these types of girders provide adequate stability and strength through slab placement.

With skewed, curved, and/or continuous steel girder bridges, temporary uplift conditions at bearings can occur during the deck pour. Designers should evaluate the

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The analysis of slab placement shall be done in an incremental fashion using a concrete modulus of elasticity equal to 70% of the concrete modulus elasticity at 28 days for concrete which is at least 24 hours old, assuming no retarder admixture is permitted. If retarder admixture is specified, it shall be indicated on the contract drawings, and the analysis shall be completed assuming 48 hours before gaining stiffness for lateral resistance. This means the stiffness of the model will change at the many different stages.

In no case shall the final design moment stresses or forces be less than those determined from an analysis in which the weight of the deck slab is applied all at once.

Slab concrete, which is less than 24 hours old (or 48 hours old when retarder is used), cannot be considered to provide lateral support for the embedded top flange of the girder. Conversely, slab concrete which is more than 24 hours old (or 48 hours old when retarder is used) can be considered to provide full lateral support for the embedded top flange of the girder. If the contractor can demonstrate that the concrete will provide lateral support for the embedded top flange in less than 24 hours (or 48 hours old when retarder is used), that limiting time may be used with the approval of the Chief Bridge Engineer.

From the results of the analysis of slab placement and lateral support conditions described above, the bending and shear strength of girder shall be checked.

6.10.3.2.5.2P Deck Slab Overhang Form Support

For the erection condition with the overhang form support system, the strength and stability of the fascia girder shall be ensured by applying the dead load of the overhang concrete and any construction equipment to the girder as follows:

(a) The standard form support system, shown in Figure 1, may be used where:

   (1) Girder web depth is less than 2400 mm {8'-0"}

   (2) Deck slab overhang is less than 1400 mm {4'-9"}

   (3) Slab thickness is equal to or less than 250 mm {10 in.}

   (4) Transverse stiffener spacing does not exceed the depth of the girder

   (5) In regions where γ_w (see A6.10.3.2.1, D6.10.3.2.1 and A6.10.3.2.2) is less than 2.5, the factored dead load shear using a load factor of 4.0 is less than the buckling shear given in A6.10.9.3.

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potential for uplift in bearings as part of the deck pour sequence evaluation. Designers should address temporary uplift conditions as follows:

- Where the temporary uplift is not detrimental to the long-term performance of the bearing, or does not result in adverse stability conditions, temporary uplift is permitted. In this case, the designer should identify in the construction plans the individual bearing locations where uplift is expected and during what stages of the deck pour the uplift will occur. A note stating that the uplift is temporary and permitted as part of construction should also be provided in the construction plans.

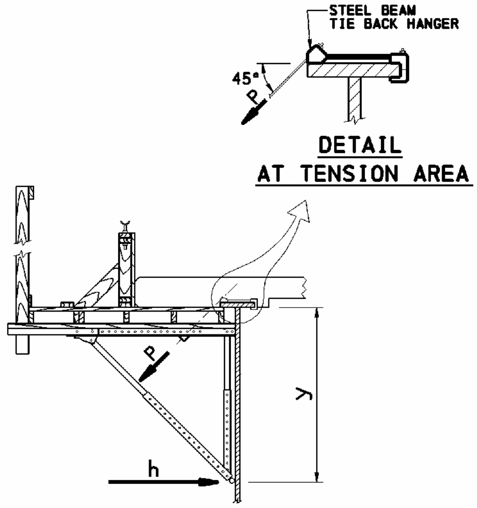
- Where uplift is determined to be unacceptable for individual bearing types or structure stability, the designer should identify in the construction plans the individual bearing locations where uplift is expected. Hold down forces and any other design requirements for restraining devices should be shown in the plans for the contractor's use in designing these components. Forces and design requirements for individual deck pour stages, as applicable, should be provided. The designer should verify the viability of at least one type of restraining device to meet the design requirements and provide schematic details of the device in the construction plans.

C6.10.3.2.5.2P

The requirements of this article can be met by reducing the length of some deck pours, or by increasing the size of the steel girder section, or by a combination of both. For original designs, the designer should obtain input from contractor and fabricators about the economics of those alternatives. Note, also, that only a relatively short length of the critical spans will be affected by the constructibility criterion.

The intent of the required checks is to control the buckling of the flanges and the webs of steel girders. It is felt that there is a potential for fatigue cracking if steel plates are allowed to buckle due to "oil-canning" effects.

The preferred upper limit on the deck slab overhang is 1200mm {4'-0"} considering factors such as deck forming and deck finishing.



TYPICAL OVERHANG FORMING DETAIL
NO SCALE

NOMINAL DEPTH y mm (in)	MAXIMUM PERMISSIBLE HORIZONTAL LOAD h N/mm (kip/ft)
915 (36)	18.25 (1.250)
1420 (56)	10.95 (0.750)
1930 (76)	7.88 (0.540)

MAXIMUM PERMISSIBLE JACK SPACING 1220 mm (4 ft)

The fascia girders are designed for a temporary construction load applied to the web at a maximum 1220mm{4 ft.} interval. This load (see table) approximates the horizontal component of a deck overhang form support bracket and consists of an allowance for the weight of the concrete, forms and incidental loads, plus the deck finishing machine. Where a transverse stiffener spacing, less than that required for the final design shear, is indicated for constructibility, the spacing for the final design shear may be used if the overhang forms are supported from the bottom flange of the fascia girder, or if the girder web is adequately braced to prevent buckling due to loads from web-bearing form support brackets. The contractor has the option to modify the overhang bracket from that described herein provided working drawings including calculations, sealed by a professional engineer licensed in the Commonwealth of Pennsylvania, are submitted for review and acceptance and show the modifications do not cause unacceptable deformations or stresses in the bridge and it is understood the contractor is ultimately responsible for the satisfactory completion of the bridge.

Figure 6.10.3.2.5.2P-1 - Typical Overhang Forming Detail and Note

Where these requirements are satisfied, original designs

For rolled beams spans, this is typically not a

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of fascia girders shall provide transverse stiffeners throughout the span at a maximum spacing of D, including the region where stiffeners are not required for the final design shear or where a spacing larger than D would be satisfactory for the final design shear. This requirement ensures reasonable constructibility. The stiffener spacings required for both constructibility and final design shear shall be shown on the contract drawings (preferably on the girder elevations), and the sketch and note from Figure 1 shall be included on the contract drawings.

(b) For deck slab overhangs which do not meet the requirements of (a), the designer of the original structure shall review the condition with the Chief Bridge Engineer's office as part of the TS&L submission. If it is determined that web-supported overhang form brackets cannot be permitted, the following note shall be included in the general notes:

Support deck slab overhang forms from the bottom flange of the fascia girder, unless the girder web is adequately supported to prevent buckling due to loads from web-bearing form supports.

(c) Contractor-designed alternates shall meet the requirements of this article. The stiffener spacing and a description of the deck overhang form support system, including the loads, shall be shown on the conceptual design drawings submitted for approval.

(d) All DM-4 and appropriate LRFD provisions in regard to flange and web buckling must be checked.

(e) For additional criteria on exterior girder rotation due to large cantilever deck slabs, see D9.7.1.5.1P.

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controlling design consideration with small deck overhangs less than 600 mm {2 ft.} overhangs.

The revision to this note was developed by an APC Subcommittee for Stability of Steel Bridge Superstructures. The note was modified to provide more flexibility to the contractor to use deck overhang form brackets that have nominal depths greater than the typical 915 mm {3'-0"} bracket depth. The maximum permissible horizontal load value was developed based on field measurements of steel bridges constructed in 1999 in District 5-0 with deck overhangs in the range of 1422 mm {4'-8"} and a limited finite element analysis study of lateral web deflections of steel girders subjected to concentrated horizontal forces on the girder web.

Design modifications should consider web stress, overall web deformation, relative web deformation, the resulting deck overhang deflection, and the resulting effects on the finished deck profile. The contractor is responsible for selecting and providing calculations for the overhang forming system as required by Publication 408 Section 1050.3(c)2. Publication 408 Section 105.01 (c) specifies the responsibility of the work remains with the contractor regardless of reviews and/or acceptance of submitted working drawings by the Department.

Unacceptable deformations of the web or top flange results in deflection of the overhang bracket causing problematic deck finish and ride quality.

If an overhang is braced to within 6 inches of the bottom flange, it shall be considered braced to the bottom flange. Deck overhang forms for rolled beams spans, due to there shallow depth, are typically supported in this manner.

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6.10.3.2.4.3P Deck Slab Overhang Rotation

The designer shall consider the effects of out-of-plane girder rotations, common with skewed bridges, on deck elevations.

6.10.3.5 DEAD LOAD DEFLECTION

The following shall replace A6.10.3.5.

The provisions of A6.7.2 and D6.7.2 shall apply, as applicable.

6.10.4 Service Limit State

6.10.4.2 PERMANENT DEFORMATION

6.10.4.2.1 General

Delete the second paragraph of A6.10.4.2.1.

6.10.4.2.2 Flexure

Delete the first paragraph following the first where list in A6.10.4.2.2

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C6.10.3.2.4.3P

Out-of-plane girder rotations will cause the overhang formwork to also rotate. In an increasing magnitude from the web of the fascia girder to the outside edge of the formwork, the formwork will move upward or downward, depending on the direction of rotation, during the deck pour. It may be desirable to pre-rotate the overhang formwork so that the as-designed deck overhang cross slope is obtained after the deck pour is complete. Additionally, it may be desirable to relocate the deck finishing machine support railing from its typical position on the overhang formwork to the fascia girders. This will minimize the upward and downward movements of the finishing machine during the deck pour due to out-of-plane girder rotations. Hand finishing work will be necessary for the deck area beyond the limits of the finishing machine.

The designer may consider approximating the anticipated girder rotation based on girder differential vertical displacements.

C6.10.4.2.1

Delete the second paragraph of A6.10.4.2.1.

C6.10.4.2.2

The following shall supplement AC6.10.4.2.2.

Lateral bending in the bottom flange is only a consideration at the service limit state for all horizontally curved I-girder bridges and for straight I-girder bridges with discontinuous cross-frame or diaphragm lines in conjunction with skews less than 70°. Wind load and deck overhang effects are not considered at the service limit state.

Delete the seventh paragraph of AC6.10.4.2.2.

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6.10.5 Fatigue and Fracture Limit State

6.10.5.1 FATIGUE

The following shall supplement A6.10.5.1

For horizontally curved I-girder bridges, the fatigue stress range due to major-axis bending plus lateral bending shall be investigated.

C6.10.5.1P

In horizontally curved I-girder bridges, the base metal adjacent to butt welds and welded attachments on discretely braced flanges subject to a net applied tensile stress must be checked for the fatigue stress range due to major-axis bending, plus flange lateral bending, at the critical transverse location on the flange. Examples of welded attachments for which this requirement applies include transverse stiffeners and gusset plates receiving lateral bracing members. The base metal adjacent to flange-to-web welds need only be checked for the stress range due to major-axis bending since the welds are located near the center of the flange. Flange lateral bending need not be considered for details attached to continuously braced flanges.

6.10.5.3 SPECIAL FATIGUE REQUIREMENT FOR WEBS

The following shall replace the first paragraph of A6.10.5.3

The live load flexural stress and shear stress resulting from the fatigue load, as specified in A3.6.1.4 and D3.6.1.4, shall be factored by two times the Pennsylvania Traffic Factor (PTF) and the fatigue load factor specified for the fatigue load combination in Table A3.4.1-1. The PTF is specified in D6.6.1.2.2.

For the purposes of this article, the factored fatigue load shall be taken as twice that calculated using the Fatigue load combination specified in Tables A3.4.1-1 and in D3.4.1.1P, with the fatigue live load taken as specified in A3.6.1.4 and D3.6.1.4, multiplied by the Pennsylvania Traffic Factor (PTF). The PTF is specified in D6.6.1.2.2.

6.10.6 Strength Limit State

6.10.6.2 FLEXURE

6.10.6.2.2 Composite Sections in Positive Flexure

The following shall replace A6.10.6.2.2.

Composite sections in kinked (chorded) continuous or horizontally curved steel girder bridges shall be considered as noncompact sections and shall satisfy the requirements of A6.10.7.2.

C6.10.6.2.2

The following shall replace AC6.10.6.2.2.

Composite sections in positive flexure in kinked (chorded) continuous or horizontally curved steel bridges are also to be designed at the strength limit state as noncompact sections as specified in A6.10.7.2. Research has not yet been conducted to support the design of these sections for a nominal flexural resistance exceeding the moment at first yield.

SPECIFICATIONS

6.10.6.2.3 Composite Sections in Negative Flexure and Noncomposite Sections

The following shall replace A6.10.6.2.3.

Sections in kinked (chorded) continuous or horizontally curved steel girder bridges shall be proportioned according to provisions specified in A6.10.8 and D6.10.8.

COMMENTARY

C6.10.6.2.3

The following shall replace AC6.10.6.2.3.

For composite sections in negative flexure and noncomposite sections, the provisions of A6.10.8 and D6.10.8 limit the nominal flexural resistance to be less than or equal to the moment at first yield. As a result, the nominal flexural resistance for these sections is conveniently expressed in terms of the elastically computed flange stress.

For composite sections in negative flexure or noncomposite sections in horizontally curved bridges, the provisions of A6.10.8 and D6.10.8 must be used. Research has not yet been conducted to extend the provisions of Appendix A to sections in kinked (chorded) continuous or horizontally curved steel bridges.

6.10.8 Flexural Resistance - Composite Sections in Negative Flexure and Noncomposite Sections

6.10.8.2 COMPRESSION-FLANGE FLEXURAL RESISTANCE

6.10.8.2.3 Lateral Torsional Buckling Resistance

The following shall replace Equation A6.10.8.2.3-10.

f_1 = stress without consideration of lateral bending at the brace point opposite to the one corresponding to f_2, calculated as the intercept of the most critical assumed linear stress variation passing through f_2 and either f_{mid} or f_0, whichever produces the smaller value of C_b (MPa). f_1 may be determined as follows:

- When the variation in the moment along the entire length between the brace points is concave in shape:

$$f_1 = f_0 \qquad (6.10.8.2.3\text{-}10)$$

- Otherwise:

$$f_1 = 2f_{mid} - f_2 \geq f_0 \qquad (6.10.8.2.3\text{-}11)$$

SPECIFICATIONS

6.10.9 Shear Resistance

6.10.9.1 GENERAL

The following shall replace the first bulleted item of the third paragraph of A6.10.9.1.

- without a longitudinal stiffener and with a transverse stiffener spacing not exceeding 1.5D, or

The following shall replace the fourth paragraph of A6.10.9.1.

Provisions for end panels shall be as specified in A6.10.9.3.3 and D6.10.9.3.3.

The following shall supplement A6.10.9.1.

Transverse stiffener spacing shall also satisfy the requirements of D6.10.3.2.5.2P for deck slab overhang form support.

6.10.9.3.3 End Panels

The following shall replace the last paragraph of A6.10.9.3.3.

The transverse stiffener spacing for end panels without a longitudinal stiffener shall not exceed 0.5D. The transverse stiffener spacing of end panels with a longitudinal stiffener shall not exceed 0.5 times the maximum subpanel depth.

6.10.10 Shear Connectors

6.10.10.1 GENERAL

The following shall replace the third paragraph of A6.10.10.1.

Shear connectors are required along the entire length of the girder when a composite girder analysis has been performed.

COMMENTARY

C6.10.10.1P

Mechanical shear connectors provide for the horizontal shear at the interface between the concrete slab and the steel girder in the positive moment regions and the horizontal shear between the longitudinal reinforcement steel within the effective flange width and the steel girder in the negative moment regions.

SPECIFICATIONS

6.10.10.1.1 Types

The following shall supplement A6.10.10.1.1.

The minimum diameter of studs shall be 19 mm {3/4 in.}.

6.10.10.1.2 Pitch

The following shall replace the part of the article starting immediately after Equation A6.10.10.1.2-1 to the end of A6.10.10.1.2.

in which:

COMMENTARY

C6.10.10.1.2

The following shall supplement A6.10.10.1.2.

At the fatigue limit state, shear connectors are designed for the range of live load shear between the deck and top flange of the girder. In straight girders, the shear range normally is due to only major-axis bending if torsion is

SPECIFICATIONS

V_{sr} = horizontal fatigue shear range per unit length (N/mm) {kip-in.}

$$= \sqrt{\left(V_{fat}\right)^2 + \left(F_{fat}\right)^2} \quad (6.10.10.1.2\text{-}2)$$

V_{fat} = longitudinal fatigue shear range per unit length (N/mm) {kip/in.}

$$= \frac{V_f Q}{I} \quad (6.10.10.1.2\text{-}3)$$

F_{fat} = radial fatigue shear range per unit length (N/mm) {kip/in} taken as the larger of either:

$$F_{fat1} = \frac{A_{bot}\sigma_{flg}\ell}{wR} \quad (6.10.10.1.2\text{-}4)$$

or

$$F_{fat2} = \frac{F_{rc}}{w} \quad (6.10.10.1.2\text{-}5)$$

where:

σ_{flg} = range of longitudinal fatigue stress in the bottom flange without consideration of flange lateral bending (MPa) {ksi}

A_{bot} = area of the bottom flange (mm^2) {in.^{2}}

F_{rc} = net range of cross-frame or diaphragm force at the top flange (N) {kip}

I = moment of inertia of the short-term composite section (mm^4) {in.^{4}}

ℓ = distance between brace points (mm) {ft.}

n = number of shear connectors in a cross-section

p = pitch of shear connectors along the longitudinal axis (mm) {in.}

Q = first moment of the transformed short-term area of the concrete deck about the neutral axis of the short-term composite section (mm^3) {in.^{3}}

R = minimum girder radius within the panel (mm) {ft.}

V_f = vertical shear force range under the fatigue load combination specified in Table A3.4.1-1 and D3.4.1-1 with the fatigue live load taken as specified in A3.6.1.4 and D3.6.1.4 (N) {kip}

COMMENTARY

ignored. Curvature, skew and other conditions may cause torsion, which introduces a radial component of the horizontal shear. These provisions provide for consideration of both of the components of the shear to be added vectorially according to Eq. 2.

The radial shear range, F_{fat}, typically is determined for the fatigue live load positioned to produce the largest positive and negative major-axis bending moments in the span. Therefore, vectorial addition of the longitudinal and radial components of the shear range is conservative because the longitudinal and radial shears are not produced by concurrent loads.

Eq. 4 may be used to determine the radial fatigue shear range resulting from the effect of any curvature between brace points. The shear range is taken as the radial component of the maximum longitudinal range of force in the bottom flange between brace points, which is used as a measure of the major-axis bending moment. The radial shear range is distributed over an effective length of girder flange, w. At end supports, w is halved. Eq. 4 gives the same units as V_{fat}.

Eq. 5 will typically govern the radial fatigue shear range where torsion is caused by effects other than curvature, such as skew. Eq. 5 is most likely to control only in regions adjacent to a skewed support for which the skew angle less than 70° in either a straight or horizontally curved bridge. Eqs. 4 and 5 yield approximately the same value if the span or segment is curved and there are no other sources of torsion in the region under consideration. Note that F_{rc} represents the net range of force transferred to the top flange from all cross-frames at the point under consideration due to the factored fatigue load plus impact. In lieu of a refined analysis, F_{rc} for an exterior girder, which is the critical case, may be taken as 111 200 N {25 kips}, and taken as zero for interior girders. Eq. 5 should only be checked using this value within the regions of a straight or horizontally curved girder in the region of a skewed support with a skew angle less than 70°. Regardless of whether F_{rc} is determined by refined analysis or taken as the above recommended value, it should be multiplied by the factor of 0.75 discussed in Article C6.6.1.2.1 and DC6.6.1.2.1 to account for the probability of two vehicles being in their critical relative position to cause the maximum range of cross-frame force at the top flange.

Eqs. 4 and 5 are provided to ensure that a load path is provided through the shear connectors to satisfy equilibrium at a transverse section through the girders, deck and cross-frame.

SPECIFICATIONS

w = effective length of deck (mm) taken as 1220 mm {48 in.}, except at end supports where w may be taken as 610 mm {24 in.}

Z_r = shear fatigue resistance of an individual shear connector determined as specified in A6.10.10.2 and D6.10.10.2 (N) {kip}

For straight spans or segments, the radial fatigue shear range from Eq. 4 may be taken equal to zero. For straight or horizontally curved bridges with skews not less than 70°, the radial fatigue shear range from Eq. 5 may be taken equal to zero.

The center-to-center pitch of shear connectors shall not exceed 600 mm {24 in.} and shall not be less than six stud diameters.

The center-to-center pitch of channel shear connectors shall not exceed 600 mm {24 in.} and shall not be less than 150 mm {6 in.}.

6.10.10.1.3 Transverse Spacing

The following shall supplement A6.10.10.1.3.

The minimum number of studs in a group shall consist of two in a single transverse row.

6.10.10.1.4 Cover and Penetration

The following shall replace A6.10.10.1.4.

The clear depth of concrete cover over the tops of the shear connectors should not be less than 60 mm {2 1/2 in.}. Shear connectors should penetrate at least 50 mm {2 in.} into the deck.

6.10.10.1.5P SPLICE LOCATIONS

Shear connectors at splice locations shall be arranged to clear fasteners and shall be welded to the splice plate. Up to 20% fewer connectors, than required by design, are acceptable in the splice zone, provided that the deleted connectors are furnished as additional connectors adjacent to the splice.

6.10.10.2 FATIGUE RESISTANCE

The following shall supplement A6.10.10.2.

The fatigue resistance of an individual channel shear connector, Z_r, shall be taken as:

Metric Units:

$$Z_r = B\,w \le 184\,w \qquad (6.10.10.2\text{-}3P)$$

COMMENTARY

C6.10.10.1.4

Delete the second sentence of AC6.10.10.1.4.

For plan presentation, show cover and penetration limits; do not detail stud height (see BC-753M). Stud heights are determined in the field based on actual girder elevations.

C6.10.10.2

The following shall supplement AC6.10.10.2.

Equations 3P and 4P were added because the LRFD specification does not address the fatigue resistance of channel shear connectors. Equations 3P and 4P were converted from Article 10.3.8.5.1.1 of AASHTO Standard Specification for Highway Bridges.

SPECIFICATIONS

U.S. Customary Units:

$$Z_r = B\,w \leq 1.05\,w$$

where:

Metric Units:

$$B = 1525 - 175 \text{ Log } N \qquad (6.10.10.2\text{-}4P)$$

U.S. Customary Units:

$$B = 8.7 - \text{Log } N$$

w = length of channel shear connector measured in a transverse direction on the flange of a girder (mm)

N = number of cycles specified in A6.6.1.2.5 and D6.6.1.2.5

6.10.10.3 SPECIAL REQUIREMENTS FOR POINTS OF PERMANENT LOAD CONTRAFLEXURE

Delete A6.10.10.3.

6.10.10.4 STRENGTH LIMIT STATE

6.10.10.4.2 Nominal Shear Force

The following shall replace A6.10.10.4.2

For simple spans and for continuous spans that are noncomposite for negative flexure in the final condition, the total nominal shear force, P, between the point of maximum positive design live load plus impact moment and each adjacent point of zero moment shall be taken as:

$$P = \sqrt{P_p^{\,2} + F_p^{\,2}} \qquad (6.10.10.4.2\text{-}1)$$

in which:

P_p = total longitudinal shear force in the concrete deck at the point of maximum positive live load plus impact moment (N) {kip} taken as the lesser of either:

$$P_{1p} = 0.85\, f_c' b_s t_s \qquad (6.10.10.4.2\text{-}2)$$

or

COMMENTARY

C6.10.10.3

Delete AC6.10.10.3.

PennDOT requires composite girders to be composite full-length of the bridge with shear connectors full-length of the bridge.

6.10.10.4.2

The following shall supplement AC6.10.10.4.2.

The radial effect of curvature is included in Eqs. 4 and 9. For curved spans or segments, the radial force is required to bring into equilibrium the smallest of the longitudinal forces in either the deck or the girder. When computing the radial component, the longitudinal force is conservatively assumed to be constant over the entire length L_p or L_n, as applicable.

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COMMENTARY

$$P_{2p} = F_{yw}Dt_w + F_{yt}b_{ft}t_{ft} + F_{yc}b_{fc}t_{fc} \quad (6.10.10.4.2\text{-}3)$$

F_p = total radial shear force in the concrete deck at the point of maximum positive live load plus impact moment (N) {kip} taken as:

$$F_p = P_p \frac{L_p}{R} \quad (6.10.10.4.2\text{-}4)$$

where:

b_s = effective width of the concrete deck (mm)

L_p = arc length between an end of the girder and an adjacent point of maximum positive live load plus impact moment (mm) {ft}

R = minimum girder radius over the length, L_p (mm) {ft.}

t_s = thickness of the concrete deck (mm) {in.}

For straight segments, F_p may be taken equal to zero.

For continuous spans that are composite for negative flexure in the final condition, the total nominal shear force, P, between the point of maximum positive design live load plus impact moment and an adjacent end of the member shall be determined from Eq. 1. The total nominal shear force, P, between the point of maximum positive design live load plus impact moment and the centerline of an adjacent interior support shall be taken as:

$$P = \sqrt{P_T^{\,2} + F_T^{\,2}} \quad (6.10.10.4.2\text{-}5)$$

in which:

P_T = total longitudinal shear force in the concrete deck between the point of maximum positive live load plus impact moment and the centerline of an adjacent interior support (N) {ft.} taken as:

$$P_T = P_p + P_n \quad (6.10.10.4.2\text{-}6)$$

P_n = total longitudinal shear force in the concrete deck over an interior support (N) {ft.} taken as the lesser of either:

$$P_{1n} = F_{yw}Dt_w + F_{yt}b_{ft}t_{ft} + F_{yc}b_{fc}t_{fc} \quad (6.10.10.4.2\text{-}7)$$

or

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$$P_{2n} = 0.45 f_c' b_s t_s \qquad (6.10.10.4.2\text{-}8)$$

F_T = total radial shear force in the concrete deck between the point of maximum positive live load plus impact moment and the centerline of an adjacent interior support (N) {kip} taken as:

$$F_T = P_T \frac{L_n}{R} \qquad (6.10.10.4.2\text{-}9)$$

where:

L_n = arc length between the point of maximum positive live load plus impact moment and the centerline of an adjacent interior support (mm) {ft.}

R = minimum girder radius over the length, L_n (mm) {ft.}

For straight segments, F_T may be taken equal to zero.

6.10.11 Stiffeners

6.10.11.1 TRANSVERSE INTERMEDIATE STIFFENERS

6.10.11.1.1 General

The following shall supplement A6.10.11.1.1.

Single-sided stiffeners on horizontally curved girders should be attached to both flanges. When pairs of transverse stiffeners are used on horizontally curved girders, they shall be fitted tightly to both flanges.

Transverse stiffeners shall also satisfy the requirements given in Standard Drawing BC-753M.

The following shall replace the third paragraph of A6.10.11.1.1.

The distance between the end of the web-to-stiffener weld and the near edge of the adjacent web-to-flange or longitudinal stiffener-to-web weld shall not be less than $4t_w$ but not to exceed the lesser of $6t_w$ and 100 mm {4 in.}.

6.10.11.1.2 Projecting Width

following shall supplement A6.10.11.1.2.

For tub sections in regions of negative flexure, b_f shall be taken as the full-width of the widest top flange within the field section under consideration.

COMMENTARY

C6.10.11.1.1P

When single-sided transverse stiffeners are used on horizontally curved girders, they should be attached to both flanges to help to restrain the flanges and to help retain the cross-sectional configuration of the girder when subjected to torsion. The fitting of pairs of transverse stiffeners against the flanges is required for the same reason.

The minimum distance between the end of the web-to-stiffener weld to the adjacent web-to-flange or longitudinal stiffener-to-web weld is set to relieve flexing of the unsupported segment of the web to avoid fatigue-induced cracking of the stiffener-to-web welds, and to avoid inadvertent intersecting welds. The $6t_w$ criterion for maximum distance is set to avoid vertical buckling of the unsupported web. The 100 mm {4 in.} criterion was arbitrarily selected to avoid a very large unsupported length where the web thickness has been selected for reasons other than stability, e.g., webs of bascule girders at trunnions.

SPECIFICATIONS

6.10.11.3 LONGITUDINAL STIFFENERS

6.10.11.3.3 Moment of Inertia and Radius of Gyration

The following shall replace A6.10.11.3.3.

Longitudinal stiffeners shall satisfy:

$$I_\ell \geq Dt_w^3\left[2.4\left(\frac{d_o}{D}\right)^2 - 0.13\right]\beta \qquad (6.10.11.3.3\text{-}1)$$

and

$$r \geq \frac{0.16d_o\sqrt{\dfrac{F_{ys}}{E}}}{\sqrt{1-0.6\dfrac{F_{yc}}{R_hF_{ys}}}} \qquad (6.10.11.3.3\text{-}2)$$

in which:

β = curvature correction factor for longitudinal stiffener rigidity calculated as follows:

- For cases where the longitudinal stiffener is on the side of the web away from the center of curvature:

$$\beta = \frac{Z}{6} + 1 \qquad (6.10.11.3.3\text{-}3)$$

- For cases where the longitudinal stiffener is on the side of the web toward the center of curvature:

$$\beta = \frac{Z}{12} + 1 \qquad (6.10.11.3.3\text{-}4)$$

Z = curvature parameter:

$$= \frac{0.95d_o^2}{Rt_w} \leq 10 \qquad (6.10.11.3.3\text{-}5)$$

where:

d_o = transverse stiffener spacing (mm) {in.}

I_ℓ = moment of inertia of the longitudinal stiffener including an effective width of the web equal to $18t_w$ taken about the neutral axis of the combined section (mm^4) {$in.^4$}. If F_{yw} is smaller than F_{ys}, the strip of the web included in the effective section shall be reduced by the ratio F_{yw}/F_{ys}.

R = minimum girder radius in the panel (mm) {in.}

r = radius of gyration of the longitudinal stiffener

COMMENTARY

*C*6.10.11.3.3

The following shall replace A6.10.11.3.3.

The rigidity required of longitudinal stiffeners on curved webs is greater than the rigidity required on straight webs because of the tendency of curved webs to bow. The factor β in Eq. 1 is a simplification of the requirement in the Hanshin (1988) provisions for longitudinal stiffeners used on curved girders. For longitudinal stiffeners on straight webs, Eq. 5 leads to β = 1.0.

SPECIFICATIONS

COMMENTARY

including an effective width of the web equal to $18t_w$ taken about the neutral axis of the combined section (mm) {in.}. If F_{yw} is smaller than F_{ys}, the strip of the web included in the effective section shall be reduced by the ratio F_{yw}/F_{ys}.

6.10.11.4P STIFFENERS IN RIGID-FRAME KNEES

6.10.11.4.1P Stiffener Spacing

The spacing of stiffeners in rigid-frame knees shall satisfy both of the following equations:

$$f_a \leq F_{yc} - f_{cs} \qquad (6.10.11.4.1P\text{-}1)$$

and

$$f_b \leq F_{yc} \qquad (6.10.11.4.1P\text{-}2)$$

for which:

$$f_a = f_{cs} \left(\frac{3b^2}{Rt}\right)\left(\frac{\beta^2}{4 + 1.14\beta^4}\right) \qquad (6.10.11.4.1P\text{-}3)$$

$$f_b = f_{cs} \left(\frac{3b^2}{Rt}\right)\left(\frac{\beta^3}{3.2 + \beta^3}\right) \qquad (6.10.11.4.1P\text{-}4)$$

$$\beta = \frac{a}{b} \qquad (6.10.11.4.1P\text{-}5)$$

where:

a = spacing of stiffness (mm) {in.}

b = half of flange width (mm) {in.}

F_{yc} = specified minimum yield strength of a compression flange (MPa) {ksi}

f_{cs} = maximum compression Service I load flange stress (MPa) {ksi}

R = radius of flange curvature (mm) {in.}

t = thickness of flange (mm) {in.}

SPECIFICATIONS COMMENTARY

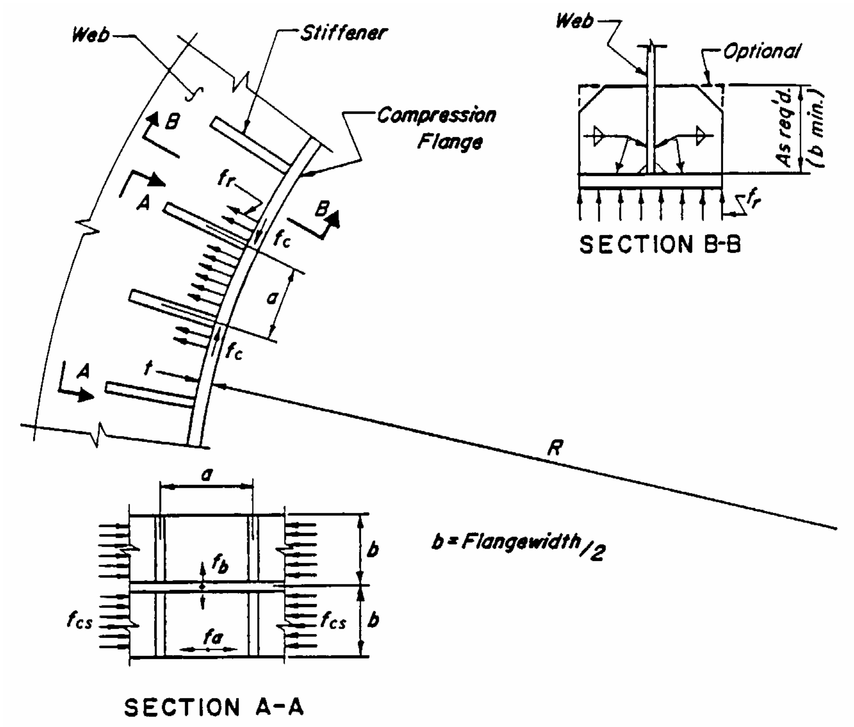

Figure 6.10.11.4.1P-1 - Stiffeners in Rigid-Frame Knees

6.10.11.4.2P Stiffener Design

The factored bearing resistance of stiffeners in rigid-frame knees, taken as specified in A6.10.11.2.3, shall be greater than P_b, taken as:

$$P_b = f_r \, ab \qquad (6.10.11.4.2P\text{-}1)$$

for which:

$$f_r = \frac{f_c t}{R} \qquad (6.10.11.4.2P\text{-}2)$$

where:

a = spacing of stiffeners (mm) {in.}

b = half of flange width (mm) {in.}

f_c = maximum factored compression flange stress (MPa) {ksi}

R = radius of flange curvature (mm) {in.}

t = thickness of flange (mm) {in.}

SPECIFICATIONS

COMMENTARY

6.10.12 Cover Plates

6.10.12.3P COVER PLATE LENGTH AND WIDTH

The length of any welded cover plate added to a rolled beam shall extend the full-length of the rolled beam, including the bearing area, or the full-length of the rolled beam field section in the case of a spliced beam unless otherwise approved by the Chief Bridge Engineer. The use of partial length cover plates is allowed for rehabilitation projects with detailed fatigue analysis. Partial length cover plates must be a bolted connection at the ends.

The width of the plate shall not exceed the width of the flange by 150 mm {6 in.}, or six times the thickness of the cover plate, whichever is less. Bottom flange cover plates preferably shall be wider than the bottom flange. Top flange cover plates shall be of constant width, preferably narrower than the top flange. When a cover plate narrower than the flange is used, the width of the plate shall be at least 50 mm {2 in.} less than the width of the flange. The width of a cover plate connected by fillet welds shall be no greater than 24 times the plate thickness.

C6.10.12.3P

The Department does not allow partial length cover plates for new designs.

6.11 BOX SECTIONS IN FLEXURE

The provisions of A6.11 and D6.11 apply to steel box I-girders curved in plan. The provisions of Appendix E, DE6.11P apply to straight box-girder bridges.

C6.11P

The provisions of D6.11 are based on the 2004 Third Edition of AASHTO-LRFD Specifications as amended herein. The provisions of Appendix E are based on the 1998 Second Edition of the AASHTO-LRFD Design Specifications with the 1999 through 2003 Interims.

6.11.1 General

C6.11.1

The following shall replace the first sentence of the first paragraph of AC6.11.1.

A6.11.1 and D6.11.1address general topics that apply to closed-box and tub sections used as flexural members in horizontally curved bridges, or bridges containing both straight and curved segments. For the application of the provisions of A6.11 and D6.11, bridges containing both straight and curved segments are to be treated as horizontally curved bridges since the effects of curvature on the support reactions and girder deflections, as well as the effects of flange lateral bending and torsional shear, usually extend beyond the curved segments.

The following shall supplement A6.11.1.

For horizontally curved boxes, flange lateral bending effects due to curvature and the effects of torsional shear must always be considered at all limit states and also during construction.

SPECIFICATIONS

6.11.1.1 STRESS DETERMINATION

The following shall replace the third and fourth paragraphs of A6.11.1.1.

The section of an exterior member assumed to resist horizontal factored wind loading within these bridges may be taken as the bottom box flange acting as a web and 12 times the thickness of the web acting as flanges.

The provisions of A4.6.2.2.2b and D4.6.2.2.2b shall not apply to single or multiple box sections in horizontally curved bridges. For these sections, the effects of both flexural and St. Venant torsional shear shall be considered. The St. Venant torsional shear stress in box flanges due to the factored loads at the strength limit state shall not exceed the factored torsional shear resistance of the flange, F_{vr}, taken as:

$$F_{vr} = 0.75\phi_v \frac{F_{yf}}{\sqrt{3}} \qquad (6.11.1.1\text{-}1)$$

COMMENTARY

C6.11.1.1

Delete the seventh paragraph of AC6.11.1.1

6.11.3 Constructibility

6.11.3.2 FLEXURE

C6.11.3.2

The following shall replace the first four sentences of the third paragraph of AC6.11.3.2.

For horizontally curved girders, flange lateral bending effects due to curvature must always be considered during construction.

Delete the fourth paragraph of AC6.11.3.2.

6.11.5 Fatigue and Fracture Limit State

The following shall replace the first sentence of the fourth paragraph of A6.11.5.

Longitudinal warping stresses and transverse bending stresses due to cross-section distortion shall be considered for single and multiple box sections in horizontally curved bridges.

The following shall supplement the last paragraph of A6.11.5.

Unless adequate strength and stability of a damaged structure can be verified by refined analysis, in cross-sections comprised of two box sections, only the bottom flanges in the positive moment regions should be designated as fracture-critical. Where cross-sections contain more than two box girder sections, none of the components of the box sections should be considered fracture-critical.

SPECIFICATIONS

6.11.6 Strength Limit State

6.11.6.2 FLEXURE

6.11.6.2.2 Sections in Positive Flexure

The following shall replace A6.11.6.2.2

Sections in horizontally curved steel girder bridges shall be considered as noncompact sections and shall satisfy the requirements of A6.11.7.2 and A6.10.7.3.

6.11.9 Shear Resistance

The following shall replace the third paragraph of A6.11.9.

For all horizontally curved sections, V_u shall be taken as the sum of the flexural and St. Venant torsional shears.

6.11.10 Shear Connectors

The following shall replace the third paragraph of A6.11.10.

For horizontally curved sections, shear connectors shall be designed for the sum of the flexural and St. Venant torsional shears. The longitudinal fatigue shear range per unit length, V_{fat}, for one top flange of a tub girder shall be computed for the web subjected to additive flexural and torsional shears. The resulting shear connector pitch shall also be used for the other top flange. The radial fatigue shear range due to curvature, F_{fat1}, given by Eq. D6.10.10.1.2-4 may be ignored in the design of box sections in straight or horizontally curved spans or segments.

For checking the resulting number of shear connectors

COMMENTARY

C6.11.6.2.2

The following shall replace AC6.11.6.2.2.

For sections in positive flexure in horizontally curved bridges the nominal flexural resistance is not permitted to exceed the moment at first yield. The nominal flexural resistance in these cases is therefore more appropriately expressed in terms of the elastically computed flange stress.

If the section is part of a horizontally curved bridge, the section must be designed as a noncompact section. The ability of such sections to develop a nominal flexural resistance greater than the moment at first yield in the presence of potentially significant St. Venant torsional shear and cross-sectional distortion stresses has not been demonstrated.

Noncompact sections must also satisfy the ductility requirement specified in 6.10.7.3 to ensure a ductile failure. Satisfaction of this requirement ensures an adequate margin of safety against premature crushing of the concrete deck for sections utilizing 690-MPa {100 ksi} steels and/or for sections utilized in shored construction. This requirement is also a key limit in allowing web bend-buckling to be disregarded in the design of composite sections in positive flexure when the web also satisfies A6.11.2.1.2, as discussed in AC6.10.1.9.1.

6.11.10

Delete the second sentence of the first paragraph of A6.11.10.

The following shall supplement the second paragraph of A6.11.10.

Because of the inherent conservatism of these requirements, the radial fatigue shear range due to curvature need not be included when computing the horizontal fatigue shear range for box sections in either straight or horizontally curved spans or segments.

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to satisfy the strength limit state, the cross-sectional area of the steel box section under consideration and the effective area of the concrete deck associated with that box shall be used in determining P by Eqs. D6.10.10.4.2-2, D6.10.10.4.2-3, D6.10.10.4.2-7, and D6.10.10.4.2-8.

The following shall replace the first sentence of last paragraph of A6.11.10.

For composite box flanges at the fatigue limit state, V_{sr} in Eq. A6.10.10.1.2-1 shall be determined as the vector sum of the longitudinal fatigue shear range given by Eq. D6.10.10.1.2-3 and the torsional fatigue shear range in the concrete deck

6.11.11 Stiffeners

6.11.11.2 LONGITUDINAL COMPRESSION-FLANGE STIFFENERS

6.12 MISCELLANEOUS FLEXURAL MEMBERS

6.12.1 General

6.12.1.2 STRENGTH LIMIT STATE

6.12.1.2.3 Shear

The following shall replace the definition of n in A6.12.1.2.3.

V_n = nominal shear resistance specified in A6.10.9.2 and DE6.10.7.2P, as appropriate, for webs of noncomposite members and 6.12.3 for webs of composite members (N) {kip}

6.12.2 Nominal Flexural Resistance

6.12.2.2 NONCOMPOSITE MEMBERS

6.12.2.2.1 I- and H-Shaped Members

The following shall replace the second paragraph of A6.12.2.2.1.

The provisions of A6.10, D6.10, DE6.10P and DE6.11p shall apply to flexure about an axis perpendicular to the web.

COMMENTARY

C6.11.11.2

The following shall supplement the first paragraph of A6.11.11.2.

For structural tees, b_ℓ should be taken as one-half the width of the flange.

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6.13 CONNECTIONS AND SPLICES

6.13.1 General

The following shall replace the first paragraph of A6.13.1.

Except as specified otherwise, connections and splices for main members (both flanges and webs) shall be designed at the strength limit state for not less than the larger of:

- The average of the flexural moment-induced stress, shear, or axial force due to the factored loadings at the point of splice or connection and the factored flexural, shear, or axial resistance of the member or element at the same point, or
- 75 percent of the factored flexural, shear, or axial resistance of the member or element.

The following shall supplement A6.13.1.

If it is necessary to cope a flange in order to provide clearance at the end connection of a floorbeam or stringer, the bending resistance of the member at the cope location shall not be decreased by more than 50%. No sharp notches shall be introduced as a result of coping. The maximum practical radius shall be maintained at all copes with an absolute minimum radius of 50 mm {2 in.}.

Where diaphragms, cross-frames, lateral bracing, stringers, or floorbeams for straight or horizontally curved flexural members are included in the structural model used to determine force effects, or alternatively, are designed for explicitly calculated force effects from the results of a separate investigation, end connections for these bracing members shall be designed for the calculated factored member force effects. Otherwise, the end connections for these members shall be designed according to the 75 percent resistance provision contained herein.

C6.13.1

The following shall supplement AC6.13.1.

Often stresses on the top flange of a composite girder are low under design loads. This criteria would require a top splice plate to be about 75% of the area of the top flange, and have the appropriate number of bolts.

The exception for bracing members for straight or horizontally curved flexural members, that are included in the structural model used to determine force effects, results from experience with details developed invoking the 75 percent and average load provisions herein. These details tended to become so large as to be unwieldy resulting in large eccentricities and force concentrations. It has been decided that the negatives associated with these connections justifies the exception permitted herein.

6.13.2 Bolted Connections

6.13.2.1 GENERAL

The following shall replace the second paragraph of A6.13.2.1.

High-strength bolted joints shall be designated as slip-critical connections. Bearing-type connections may be used on rehabilitation projects if approved by the Chief Bridge Engineer.

C6.13.2.1P

When detailing bolted connections, tightening clearance between flange and web bolts need to be taken into account. Manual of Steel Construction, American Institute of Steel Construction, provides information on assembling clearances for threaded fasteners which can be used to avoid bolt interference problems.

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6.13.2.3 BOLTS, NUTS AND WASHERS

6.13.2.3.2 Washers

The following shall replace the third and fifth bulleted items of A6.13.2.3.2.

- AASHTO M 253 (ASTM A 490) bolts are to be installed in material having a specified minimum yield strength less than 345 MPa {50 ksi}, irrespective of the tightening method;

- AASHTO M 253 (ASTM A 490) bolts over 25.4 mm {1 in.} in diameter are to be installed in an oversize or short-slotted hole in an outer-ply, in which case a minimum thickness of 7.9 mm {5/16 in.} shall be used under both the head and the nut. Multiple hardened washers shall not be used.

6.13.2.4 HOLES

6.13.2.4.1 Types

6.13.2.4.1b Oversize Holes

The following shall replace A6.13.2.4.1b.

Approval of Chief Bridge Engineer must be obtained before oversize holes can be used in any or all plies of slip-critical connections. Oversize holes are not permitted in diaphragms or cross frames of curved girder bridges. Oversize holes shall not be used in bearing-type connections.

C6.13.2.4.1c Short-Slotted Holes

On skew bridges, short slotted holes for cross-frame connections should be utilized for adjustment for temporary and permanent conditions.

6.13.2.4.2 Size

The following shall replace Table A6.13.2.4.2-1.

Table 6.13.2.4.2-1 - Maximum Hole Sizes

Metric Units				
Bolt Diameter d (mm)	Standard Hole Diameter (mm)	Oversize Hole Diameter (mm)	Short Slot Width x Length (mm x mm)	Long Slot Width x Length (mm x mm)
15.9	17.5	20.7	17.5 x 22	17.5 x 40
19.1	20.7	23.8	20.7 x 25	20.7 x 48
22.2	23.8	27.0	23.8 x 29	23.8 x 56
25.4	27.0	31.8	27.0 x 33	27.0 x 64
≥28.6	d+1.6	d+7.9	d+1.6 x d+9.5	d+1.6 x 2.5d

U.S. Customary Units				
Bolt Diameter d (in.)	Standard Hole Diameter (in.)	Oversize Hole Diameter (in.)	Short Slot Width x Length (in. x in.)	Long Slot Width x Length (in. x in.)
5/8	11/16	13/16	11/16 x 7/8	11/16 x 1-9/16
3/4	13/16	15/16	13/16 x 1	13/16 x 1-7/8
7/8	15/16	1-1/16	15/16 x 1-1/8	15/16 x 2-3/16
1	1-1/16	1-1/4	1-1/16 x 1-5/16	1-1/16 x 2-1/2
≥1-1/8	d+1/16	d+5/16	d+1/16 x d+3/8	d+1/16 x 2.5d

6.13.2.5 SIZE OF BOLTS

The following shall replace A6.13.2.5.

Bolts shall not be less than 15.9 mm {5/8 in.} in diameter. Bolts 15.9 mm {5/8 in.} in diameter shall not be used in primary members, except in 64 mm {2.5 in.} legs of angles and in flanges of sections whose dimensions require 15.9 mm {5/8 in.} fasteners to satisfy other detailing provisions herein. Use of structural shapes which do not allow the use of 15.9 mm {5/8 in.} fasteners shall be limited to handrails.

Angles whose size is not determined by a calculated demand may use:

- 15.9 mm {5/8 in.} diameter bolts in 51 mm {2 in.} legs,
- 19.1 mm {3/4 in.} diameter bolts in 64 mm {2 1/2 in.} legs,
- 22.2 mm {7/8 in.} diameter bolts in 76 mm {3 in.} legs, and
- 25.4 mm {1 in.} diameter bolts in 89 mm {3 1/2 in.} legs.

C6.13.2.5P

Typically, high-strength bolts will be 22.2 mm {7/8 in.} diameter mechanically galvanized AASHTO M 164 (ASTM A 325) bolts. This is the typical high-strength bolt used in the past.

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The diameter of bolts in angles of primary members shall not exceed one-fourth the width of the leg in which they are placed.

Fasteners shall be of the size shown on the contract plans, but generally shall be 22.2 mm {7/8 in.} in diameter.

6.13.2.6 SPACING OF BOLTS

6.13.2.6.6 Edge Distances

The following shall replace Table A6.13.2.6.6-1.

Table 6.13.2.6.6-1 - Minimum Edge Distances

Metric Units			U.S. Customary Units		
Bolt Diameter (mm)	Sheared or Gas Cut Edges (mm)	Rolled Edges of Plates or Shapes (mm)	Bolt Diameter (in.)	Sheared or Gas Cut Edges (in.)	Rolled Edges of Plates or Shapes (in.)
15.9	29	22	5/8	1 1/8	7/8
19.1	32	25	3/4	1 1/4	1
22.2	38	29	7/8	1 1/2	1 1/8
25.4	44	32	1	1 3/4	1 1/4
28.6	51	38	1 1/8	2	1 1/2
31.8	57	41	1 1/4	2 1/4	1 5/8
34.9	60	44	1 3/8	2 3/8	1 3/4

COMMENTARY

C6.13.2.6.1 Minimum Spacing and Clear Distance

The following shall supplement AC6.13.2.6.1.

The preferred distance between centers of bolts in standard holes shall not be less than the values in Table 1:

Table C6.13.2.6.1-1 - Preferred Bolt Spacing

Metric Units		U.S. Customary Units	
Diameter Bolt (mm)	Preferred Distance between Centers of Bolts (mm)	Diameter Bolt (in.)	Preferred Distance between Centers of Bolts (in.)
15.9	60	5/8	2 1/4
19.1	65	3/4	2 1/2
22.2	75	7/8	3
25.4	90	1	3 1/2

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6.13.2.7 SHEAR RESISTANCE

COMMENTARY

C6.13.2.7

The following shall supplement the first paragraph of AC6.13.2.7.

For steel plate girder flange splices greater than 1270 mm {50 in.} in length, the 20% reduction shall be applied to the nominal shear resistance of a bolt, calculated using Equation 1 and 2, because the axial force is parallel to the line of bolts. In such flange splices, the 1270 mm {50 in.} length is to be measured between the extreme bolts on only one side of the connection. The 20 percent reduction should not be applied for web bolts subjected to shear and moment.

6.13.2.8 SLIP RESISTANCE

C6.13.2.8

The following shall replace Table A6.13.2.8-1.

Table 6.13.2.8-1 - Minimum Required Bolt Tension

Metric Units			U.S. Customary Units		
Bolt Diametert (mm)	Required Tension- P_t (kN)		Bolt Diameter (in.)	Required Tension- P_t(kip)	
	M 164 (A 325)	M 253 (A490)		M 164 (A 325)	M 253 (A 490)
15.9	84.5	120	5/8	19	24
19.1	125	178	3/4	28	35
22.2	173	245	7/8	39	49
25.4	227	325	1	51	64
28.6	249	409	1 1/8	56	80
31.8	320	516	1 1/4	71	102
34.9	378	618	1 3/8	85	121
38.1	463	752	1 1/2	103	148

The following shall replace the first bulleted item in the second paragraph (the definition of Class A surface).

- Class A surface: blast cleaned surfaces with Class A coatings

  The following shall supplement A6.13.2.8.

For values of K_s, use Class A surface conditions for design, unless a paint is tested and proven to conform to Class B conditions. If Class B is used, field testing and controls must be specified in the contract drawings or construction specifications.

Delete the fourth paragraph in A6.13.2.8 which starts "The contract document shall specify that joints having..."and replace it with the following.

The following shall supplement AC6.13.2.8.

The revision to the definition of Class A and the requirement to blast clean all faying surfaces is based on results of research conducted jointly by the University of Texas at Austin and the FHWA in the early 1980's on weathering steel connections. An extensive testing program conducted in conjunction with the research showed that weathering steel connections with a mill scale surface had an average slip coefficient, K_s, less than the 0.33 value for Class A. Blast cleaned weathering steel achieved an average slip coefficient above the 0.50 value specified for a Class B contact surface. The testing program incorporated a wide range of variables, including exposure of test specimens to an open environment for periods up to 12 months.

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The following note shall be placed on the contact drawings:

"*Blast clean the faying surfaces of splices and connections of all structural elements in accordance with Publication 408 Section 1060.3(b)3. Reblast unpainted elements that remain unassembled for a period of 12 months or more following the initial cleaning.*"

COMMENTARY

The UT/FHWA research suggests that present LRFD design policy, which allows a mill scale surface for Class A, could result in weathering steel connections that do not meet the slip coefficient value for a Class A contact surface. The revision to the definition of Class A and the requirements to blast clean all faying surfaces will add desired safety into Department projects.

Inherent factors of safety in the design of connections should ensure the serviceability of in-place weathering steel structures where the slip critical condition controlled the design.

Designers are directed to review Publication 35 Bulletin 15 for current paint systems and corresponding slip coefficients.

6.13.2.10 Tensile Resistance

6.13.2.10.3 Fatigue Resistance

C6.13.2.10.3

Replace references to A 325M and A 490M with A 325 and A 490, respectively.

6.13.2.11 COMBINED TENSION AND SHEAR

C6.13.2.11

Replace references to A 325M with A 325.

6.13.3 Welded Connections

6.13.3.1 GENERAL

The following shall supplement A6.13.3.1.

Field welding is generally prohibited. Provisions may be made for attachment of stay-in-place forms, bearing plates and sole plates of pot bearings (but not the pot bearing itself). All areas where field welding is permitted shall be specifically designated on the contract plans. The fatigue provisions of this specification shall apply to the design of all affected members.

The regions of welded structures requiring non-destructive testing (NDT), along with the allowable types of NDT, shall be shown on the contract plans.

C6.13.3.1

The following shall supplement AC6.13.3.1

The AASHTO/AWS D1.5M/D1.5:2002 Bridge Welding Code describes the appropriate application of the types of NDT.

Use the AASHTO/AWS D1.1M/D1.1:2002 Structural Welding Code for the welding of new tubular structures, pipes, piles and existing steel which are not covered by AASHTO/AWS D1.5M/D1.5:2002.

6.13.3.8P INTERSECTING WELDS

Intersecting welds which provide a potential crack path into the web or flange of a girder from an attachment will not be permitted. The termination of the fillet weld to prevent the intersection shall provide a minimum clearance of 40 mm {1 1/2 in.}, unless another clearance is required by other design documents. Transverse groove welds shall not be terminated to prevent the intersection.

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6.13.3.9P INTERMITTENT FILLET WELDS

Intermittent fillet welds are prohibited, unless they are incorporated in the final weld in accordance with AASHTO/AWS Bridge Welding Code.

6.13.3.10P MINIMUM EDGE DISTANCE

A minimum edge distance of 25 mm {1 in.} shall be maintained from a fillet weld termination to the edge of a base metal plate in the direction of the weld.

C6.13.3.10P

An example of minimum edge distance is graphically shown in Figure C1.

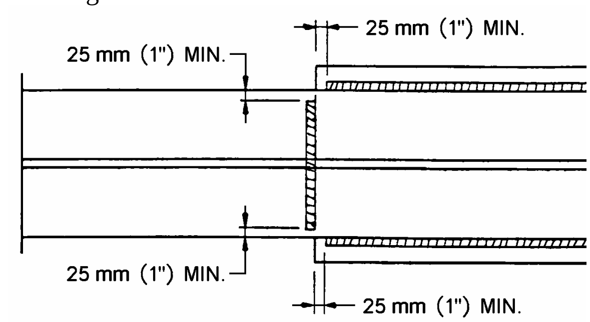

Figure C6.13.3.10P-1 - Minimum Edge Distance

6.13.4 Block Shear Rupture Resistance

The following shall replace the second sentence of the fourth paragraph of A6.13.4.

The net area shall be the gross area, minus the number of holes or fractional holes in the plane, multiplied by the nominal hole diameter specified in Table D6.13.2.4.2-1 plus 1.6 mm {1/16 in.} times the thickness of the component.

6.13.6 Splices

6.13.6.1 BOLTED SPLICES

6.13.6.1.1 General

The following shall replace A6.13.6.1.1.

Bolted splices shall be designed at the strength limit state to satisfy the requirements specified in A6.13.1 and D6.13.1. Where a section changes at a splice, the smaller of the two connected sections shall be used in the design. Develop bolted field splices for steel beams and girders in accordance with BD-616M. Splices shall be designed using the Department’s SPLRFD program.

6.13.6.1.4 Flexural Members

C6.13.6.1.4P

A6.13.6.1.4a, A6.13.6.1.4b and A6.13.6.1.4c are

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revised below to give two sets of provisions. The straight girder provisions correspond to the AASHTO-LRFD Bridge Design Specifications, Second Edition, 1998. The horizontally curved girder provisions correspond to the AASHTO-LRFD Bridge Design Specifications, Third Edition, 2004. Both were revised for PennDOT use.

Using the 1998 AASHTO-LRFD as basis for the design of straight girder splices allows the Department to continue using the existing steel splice design computer program (SPLRFD). It is the intent of the Department to update the SPLRFD computer program in the future to correspond to the provisions of the third edition of the AASHTO-LRFD Specifications.

6.13.6.1.4a General

A6.13.6.1.4a shall be revised as follows:

For straight bridges:
The following shall replace A6.13.6.1.4a.
Splice plates shall be investigated for fatigue of the base metal adjacent to slip-critical connections and specified in Table A6.6.1.2.3-1, using the gross section of the splice plates and member.
Splices subjected to tension shall satisfy the requirements specified in A6.13.5.2.

For horizontally curved bridges:
The following shall replace the fourth paragraph of A6.13.6.1.4a.
The factored flexural resistance of the flanges at the point of splice at the strength limit state shall satisfy the applicable provisions of A6.10.6.2, D6.10.6.2.

6.13.6.1.4b Web Splices

A6.13.6.1.4b shall be revised as follows:

For straight bridges:
The following shall replace A6.13.6.1.4b.
The elastic methods shall be used to determine the shear force in splice bolts. The ultimate strength method is not permitted.
Web splice plates and their connections shall be designed at the strength limit state for:

- The portion of the factored design moment, specified in A6.13.1 and D6.13.1, that is resisted by the web;

- The moment due to eccentricity of a notional shear determined as the shear due to the factored loading multiplied by the design moment specified in A6.13.1 and D6.13.1 and divided by the moment caused by the factored loads and the shear itself; and

C6.13.6.1.4b

A6.13.6.1.4b shall be revised as follows:

For straight bridges:
The following shall replace AC6.13.6.1.4b.
For bolt groups subjected to eccentric shear, a traditional conservative approach is often used in which the bolt group is treated as an elastic cross-section subjected to direct shear and torsion. A vector analysis is performed assuming that there is no friction and that the plates are rigid and the bolts are elastic.

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- The notional shear itself.

At the strength limit state, the flexural stress in the splice plates shall not exceed the specified minimum yield strength of the splice plates.

Web splice bolts shall be designed for the effects of moment due to the eccentric shear.

Web plates shall be spliced symmetrically by plates on each side. The splice plates for shear shall extend the full depth of the girder between flanges. There shall be not less than two rows of bolt on each side of the joint.

For bolted web splices with thickness differences of 2 mm {0.0625 in.} or less, no filler plates are required.

For horizontally curved bridges:

The following shall supplement A6.13.6.1.4b.

The elastic methods shall be used to determine the shear force in splice bolts. The ultimate strength method is not permitted.

The following shall replace the second sentence of the first paragraph of A6.13.6.1.4b.

For all single box sections, and for multiple box sections in bridges not satisfying the requirements of A6.11.2.3, including horizontally curved bridges, or with box flanges that are not fully effective according to the provisions of A6.11.1.1 and D6.11.1.1, the shear shall be taken as the sum of the flexural and St. Venant torsional shears in the web subjected to additive shears.

6.13.6.1.4c Flange Splices

A6.13.6.1.4c shall be revised as follows:

For straight bridges:

The following shall replace A6.13.6.1.4c.

At the strength limit state, the axial stress in the flange splice plate shall satisfy the requirements of A6.13.5.2 if in tension and A6.9.2 and D6.9.2 if compression.

For bolted flexural members, bolted splices in flange parts should not be used between field splices, unless approved by the Engineer. In any one flange, not more than one part should be spliced at the same cross-section. If practicable, splices should be located at points where there is an excess of section.

For horizontally curved bridges:

The following shall replace the first sentence of the paragraph before last of A6.13.6.1.4c.

For all single box sections, and for multiple box sections in bridges not satisfying the requirements of A6.11.2.3, including horizontally curved bridges, or with box flanges that are not fully effective according to the provisions of A6.11.1.1 and D6.11.1.1, longitudinal warping stresses due to cross-section distortion shall be

COMMENTARY

C6.13.6.1.4c

AC6.13.6.1.4c shall be revised as follows:

For straight bridges:

The following shall replace A6.13.6.1.4c.

For compression, use an unbraced length equal to zero, making $P_n = F_y A_s$

For horizontally curved bridges:

The following shall replace the first sentence of the tenth paragraph of AC6.10.6.1.4c.

For the box sections cited in this article, including sections in horizontally curved bridges, longitudinal warping stresses due to cross-section distortion can be significant under construction and service conditions and must therefore be considered when checking the connections of bolted flange splices for slip and for

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considered when checking bolted flange splices for slip and for fatigue.

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fatigue.

The following shall replace the first sentence of the eleventh paragraph of AC6.10.6.1.4c.

In cases for straight girders where flange lateral bending is deemed significant, and for horizontally curved girders, the effects of the lateral bending must be considered in the design of the bolted splices for discretely braced top flanges of tub sections or discretely braced flanges of I-sections.

6.13.6.1.5 Fillers

The following shall replace the third paragraph of A6.13.6.1.5.

Fillers 6.0 mm {1/4 in.} or more in thickness shall consist of not more than two plates, unless approved by the Chief Bridge Engineer.

The following shall replace the fifth paragraph of A6.13.6.1.5.

The specified minimum yield strength of fillers 6.0 mm {1/4 in.}or greater in thickness shall not be less than the larger of 70 percent of the specified minimum yield strength of the connected plate and 250 MPa {36 psi} unless approved by the Chief Bridge Engineer.

6.13.6.2 WELDED SPLICES

The following shall replace the third paragraph of A6.13.6.2.

Welded field splices shall not be used without written approval of the Chief Bridge Engineer.

C6.13.6.2

The following shall supplement AC6.13.6.2.

Use the AASHTO/AWS D1.1M/D1.1:2002 Structural Welding Code for the welding of new tubular structures, pipes, piles, and existing which are not covered by AASHTO/AWS D1.5M/D1.5:2002.

6.15 PILES

6.15.1 General

The following shall replace A 6.15.1.

Piles shall be designed as structural members capable of safely supporting all imposed loads.

For a pile group composed of only vertical piles which is subjected to lateral load, the pile structural analysis shall include explicit consideration of soil-structure interaction effects using a COM624P analysis (Wang and Reese, 1993) or LPILE 5.0 analysis.

Based on the parametric study conducted by the Department, which is described in the commentary, an abutment or retaining wall with a pile group composed of both vertical and battered piles which is subjected to lateral load shall be designed assuming that all lateral load is resisted by the horizontal component of the axial

C6.15.1

The following shall replace AC6.15.1.

To develop the recommended distribution of lateral load among piles supporting a typical bridge abutment, a parametric study (Kelly, et al, 1995) was performed using the program GROUP (Reese, et al, 1994). A second purpose of this parametric study was to determine if the Department's lateral deformation criteria of 12 mm {1/2 in.} for the service limit state and 25 mm {1 in.} for the strength limit state were satisfied. These criteria were met for all analyses representative of the Department's practice. The variables evaluated in the parametric study included:

- HP310X79 {HP12X53} and HP250X62 {HP10X42}

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capacity of the battered piles. The vertical load shall be distributed among piles in the group using a simple elastic procedure. The use of the above design procedure with any of the following conditions requires the approval of the Chief Bridge Engineer:

- Piles with a specified steel yield strength other than 250 MPa {36 ksi}
- Piles with bending stiffness properties less than HP250X62 {HP10X42}
- Very soft clays or very loose sands as defined in Standard Drawing BC-795M
- Piles with bending stiffness properties less than HP310X79 {HP12X53} in soft clays or loose sands as defined in Standard Drawing BC-795M
- Unfactored vertical load to horizontal load ratio less than 3.5 (excluding seismic forces)

COMMENTARY

piles with about four-pile diameters center-to-center spacing;

- End bearing piles on rock driven through a medium stiff-to-stiff clay deposit, and friction piles in deposits of medium dense and loose sand;
- Pile lengths of 3.0, 9.1 and 15.2 m {10, 30, and 50 ft.};
- Vertical and horizontal load levels consistent with common Department designs; and
- Pile-head fixity conditions of fixed, pinned and partially fixed.
- For typical pile groups containing battered piles designed using the simplified procedure, the pile study indicates that:
- Combined stresses in upper portions of the battered and vertical piles due to axial load and bending are generally less than 82 to 95 MPa {11.9 to 13.8 ksi}, which is consistent with previous practice.
- The fraction of the total lateral load resisted by bending of the vertical piles is generally less than about 20 percent.
- The check of structural pile capacity for combined axial load and flexure in the upper portion of the pile using the LRFD Interaction Equations in A6.9.2.2 does not control the pile design.
- Lateral deflections are well below acceptable magnitudes.
- As pile stiffness increases, horizontal deformations and associated bending stresses decrease such that the simplified method remains applicable.

In cases for which pile and soil conditions differ significantly from those conditions examined in the parametric study, a suitable analysis should be performed which incorporates the necessary soil-structure interaction factors. This analysis may comprise finite element analysis, p-y analysis, or other applicable methods.

Lateral deflections and maximum bending stresses for laterally-loaded pile groups generally occur within a depth below the pile cap equal to approximately 10 pile diameters. Therefore, the presence of poor material (very soft clays or very loose sands) within the upper 10 pile diameters invalidates use of the simplified method due to the potential for pile overstressing and excessive deformations under lateral loads. If these conditions exist, the designer may consider the following options:

- Improve in place or remove and replace the poor material. These may be viable options when the thickness of poor material is small and close to the ground surface.

- Perform a more rigorous, problem specific analysis to define pile stress levels and pile group deformations. This type of analysis may be performed using software such as GROUP, Reese, et al (1994).

A thin seam or lens of poor material below the upper 10 pile diameters will not typically affect the applicability of the simplified method.

6.15.2 Structural Resistance

The following shall replace A 6.15.2.

Resistance factors, φ, for the strength limit state shall be taken as specified in D6.5.4.2. The resistance factors for axial resistance of piles in compression which are subject to damage due to driving shall be applied only to that section of the pile likely to experience damage. Therefore, the specified φ factors of 0.35 and 0.45, specified in D6.5.4.2 for piles subject to damage, shall be applied only to the axial capacity of the pile. In addition, the φ factors of 0.60 and 0.85, specified in D6.5.4.2 for axial and flexural resistance, respectively, of undamaged piles, shall be applied to the combined axial and flexural resistance of the pile in the interaction equation for the compression and flexure terms respectively. This design approach is illustrated on Figure 1.

For piles bearing on soluble bedrock, the φ factor of 0.25 shall be applied to the axial capacity of the pile using a pile yield strength Fy = 250 MPa {36 ksi}.

C6.15.2

The following shall replace A 6.15.2.

Due to the nature of pile driving, additional factors must be considered in selection of resistance factors that are not normally accounted for in steel members. The factors considered in development of the specified resistance factors include:

- Unintended eccentricity of applied load about expected point of application,

- Variations in material properties of pile, and

- Pile damage due to driving.

These factors are discussed by Davisson (1983). While the resistance factors specified herein generally conform to the recommendations given by Davisson (1983), they have been modified to reflect the common practice of the Department. Specifically, the resistance factors presented in D6.5.4.2 have been selected in a manner such that, when combined with an average load factor of 1.45, the equivalent factor of safety calculated as the ratio of the appropriate load to resistance factor is comparable to the factor of safety previously used by the Department.

The factored compressive resistance, P_r, includes reduction factors for unintended load eccentricity and material property variations, as well as a reduction for potential damage to piles due to driving, which is most likely to occur near the tip of the pile. The resistance factors for computation of the factored axial pile capacity near the tip of the pile are 0.35 and 0.45 for severe and good driving conditions, respectively. The φ factor of 0.25 for piles bearing on soluble bedrock is intended to safeguard against the potential of the loss of geotechnical capacity in soluble bedrock.

For steel piles, flexure occurs primarily toward the head of the pile. This upper zone of the pile is less likely to experience damage due to driving. Therefore, relative

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to combined axial compression and flexure, the resistance factor for axial resistance (φ_c = 0.60) accounts for both unintended load eccentricity and pile material property variations, whereas the resistance factor for flexural resistance (φ_f = 0.85) accounts only for variations in pile material properties.

Typically, due to the lack of a detailed soil-structure interaction analysis of pile groups containing both vertical and battered piles, evaluation of combined axial and flexural loading will only be applied to pile groups containing no battered piles.

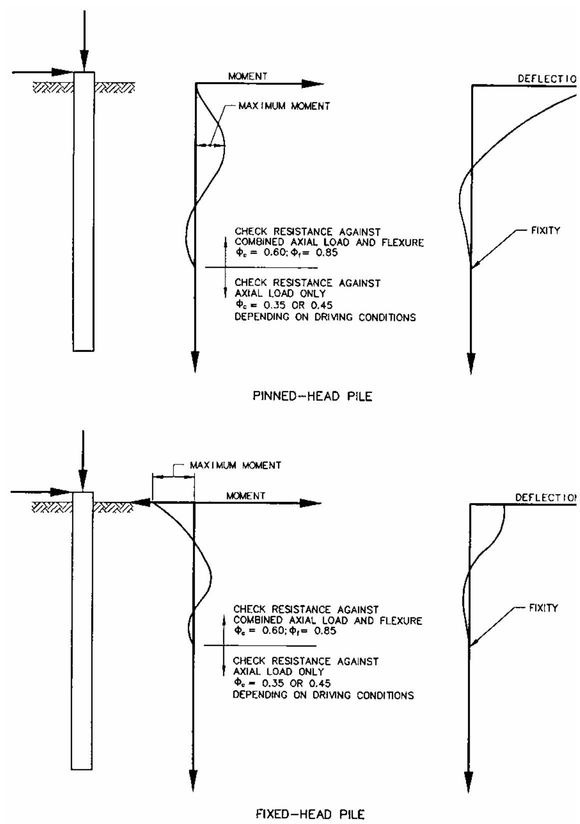

Figure 6.15.2-1 - Distribution of Moment and Deflection in Vertical Piles Subjected to Lateral Load

6.15.3P Compressive Resistance

The design of steel piles shall follow A6.9, except as specified herein.

6.15.3.1 AXIAL COMPRESSION

The following shall replace A 6.15.3.1.

For piles under axial load, the factored resistance of piles in compression, P_r, shall be taken as specified in A6.9.2.1 using the resistance factor, φ_c, specified in D6.5.4.2.

SPECIFICATIONS

6.15.3.2 COMBINED AXIAL COMPRESSION AND FLEXURE

The following shall replace A 6.15.3.2.

Piles subjected to axial load and flexure shall be designed in accordance with A6.9.2.2 using the resistance factors, φ_c and φ_f, specified in D6.5.4.2.

Vertical H-pile foundations designed using COM624P or LPILE 5.0 per D10.7.3.12.2 may use the values given in Tables 1 and 2.

where:

D = Depth of the pile.

Area = Area of the pile.

I_x, I_y = Moment of inertia about their respective axis.

P_{uSERV} = COM624P or LPILE 5.0 pile load equivalent to pile resistance at Service Limit State.

P_{uSTR} = COM624P or LPILE 5.0 pile load equivalent to pile resistance under severe driving conditions as defined in D6.15.2P.

P_r = Factored axial resistance for combined axial and flexural resistance.

M_{rx}, M_{ry} = Factored flexural resistance of the vertical pile in the x-axis and y-axis, respectively.

COMMENTARY

C6.15.3.2

The factored flexural resistance, $M_{rx,}$ is based on either the plastic or elastic moment of pile considering web and compression flange slenderness requirements. The factored flexural resistance, $M_{ry,}$ is based on the plastic moment per AC6.12.2.2.1.

For these tables the piles are considered as braced. If very weak soils, scour or voids are expected, the buckling requirements of DM-4 6.15.3.3 must be considered and the values shown in Tables 1and 2 are not applicable.

P_{uSERV} and P_{uSTR} in Tables 1 and 2 are based on $0.25*f_y*A_s$ and $0.35*f_y*A_{s,}$ respectively. For piles bearing on soluble rock (limestone,etc.) values for P_{uSERV} and P_{uSTR} equal to $0.16*f_y*A_s$ and $0.25*f_y*A_s$, respectively, are to be considered.

The section properties provided are for use in the COM624P or LPILE 5.0 analysis. The combined axial compression and flexural requirements of A6.9.2.2 shall be evaluated considering the results of the COM624P or LPILE 5.0 analysis and the resistances provided in the Tables 1 and 2.

The section properties may also be used in PENNDOT's Integral Abutment Spreadsheet. However; since the capacities in the tables do not consider unbraced length, the structural pile capacity of vertical piles used in an Integral Abutment must be checked in accordance with DM-4 Appendix G using the Integral Abutment Spreadsheet.

Table 6.15.3.2P-1 – Pile Properties, Factored Axial and Flexural Resistances with Full Pile Section.
F_y = 250 MPa {36 ksi}

Metric									
HP	Depth D (mm)	Area (mm^2)	I_x ($10^6 mm^4$)	I_y ($10^6 mm^4$)	P_{uSERV} (kN)	P_{uSTR} (kN)	P_r (kN)	M_{rx} (kN-m)	M_{ry} (kN-m)
360 x 174	361	22200	508	184	1378	1929	3306	679	310
360 x 152	356	19400	439	159	1204	1685	2889	589	270
360 x 132	351	16900	375	135	1049	1468	2517	455	231
360 x 108	346	13800	303	108	856	1199	2055	372	186
310 x 125	312	15900	270	88.2	987	1381	2368	417	180
310 x 110	308	14100	237	77.1	875	1225	2100	368	158
310 x 94	303	11900	196	63.9	738	1034	1772	274	132
310 x 79	299	10000	163	52.6	621	869	1489	232	110
250 x 85	254	10800	123	42.3	670	938	1608	232	104
250 x 62	246	7970	87.5	30.0	495	692	1187	151	75

US Customary Units									
HP	Depth D (in)	Area (in^2)	I_x (in^4)	I_y (in^4)	P_{uSERV} (kips)	P_{uSTR} (kips)	P_r (kips)	M_{rx} (kip-ft)	M_{ry} (kip-ft)
14 x 117	14.21	34.4	1220	443	310	434	743	495	228
14 x 102	14.01	30.0	1050	380	270	378	648	431	197
14 x 89	13.83	26.1	904	326	235	329	564	334	169
14 x 73	13.61	21.4	729	261	193	270	462	273	137
12 x 84	12.28	24.6	650	213	221	310	531	306	132
12 x 74	12.13	21.8	569	186	196	275	471	268	116
12 x 63	11.94	18.4	472	153	166	232	397	202	97
12 x 53	11.78	15.5	393	127	140	195	335	170	81
10 x 57	9.99	16.8	294	101	151	212	363	170	75
10 x 42	9.70	12.4	210	71.7	112	156	268	111	54

Table 6.15.3.2P-2 – Factored Axial and Flexural Resistance with 1.5 mm (1/16") Section Loss. F_y = 250 MPa (36 ksi)

Metric									
HP	Depth D (mm)	Area (mm^2)	I_x ($10^6 mm^4$)	I_y ($10^6 mm^4$)	P_{uSERV} (kN)	P_{uSTR} (kN)	P_r (kN)	M_{rx} (kN-m)	M_{ry} (kN-m)
360 x 174	358	18530	424	152	1150	1610	2760	504	258
360 x 152	353	15730	355	127	976	1367	2343	428	217
360 x 132	348	13240	295	105	821	1150	1971	360	181
360 x 108	343	10210	224	79.7	634	887	1520	278	138
310 x 125	309	12780	217	70.1	793	1110	1903	299	145
310 x 110	305	10980	184	59.5	682	954	1636	257	124
310 x 94	300	8820	145	46.8	547	766	1313	206	98
310 x 79	296	6980	113	36.4	433	606	1039	163	77
250 x 85	251	8290	93.3	31.5	514	720	1,234	158	78
250 x 62	243	5470	59.9	20.2	339	475	814	105	51

US Customary Units									
HP	Depth D (in)	Area (in^2)	I_x (in^4)	I_y (in^4)	P_{uSERV} (kips)	P_{uSTR} (kips)	P_r (kips)	M_{rx} (kip-ft)	M_{ry} (kip-ft)
14 x 117	14.09	28.73	1019	365	259	362	620	369	189
14 x 102	13.89	24.39	853	305	219	307	527	313	159
14 x 89	13.71	20.51	708	253	185	258	443	263	133
14 x 73	13.49	15.83	537	192	142	199	342	203	101
12 x 84	12.16	19.81	521	168	178	250	428	219	106
12 x 74	12.01	17.02	443	143	153	215	368	188	90
12 x 63	11.82	13.66	349	112	123	172	295	151	72
12 x 53	11.66	10.81	272	87.5	97	136	234	119	56
10 x 57	9.87	12.84	224	75.6	116	162	277	116	57
10 x 42	9.58	8.48	143	48.5	76	107	183	77	37

SPECIFICATIONS

6.15.3.3 BUCKLING

The following shall replace the last sentence of A 6.15.3.3.

The depth to fixity shall be determined in accordance with D10.7.3.13.4 for battered piles or COM624P or LPILE 5.0 analyses for vertical piles. Figure D6.15.2-1 illustrates the depth to fixity as determined by COM624P or LPILE 5.0.

6.15.4 Maximum Permissible Driving Stresses

The following shall replace A 6.15.4.

Maximum permissible driving stresses for top driven steel piles shall be taken as specified in D10.7.8.

COMMENTARY

C6.15.3.3

The following shall replace AC 6.15.3.3.

The use of an approximate method in lieu of a P-Δ analysis is allowed if approved by the Chief Bridge Engineer.

SPECIFICATIONS COMMENTARY

APPENDIX A – FLEXURAL RESISTANCE – COMPOSITE SECTIONS IN NEGATIVE FLEXURE AND NONCOMPOSITE SECTIONS WITH COMPACT OR NONCOMPACT WEBS

The following shall be added immediately after the heading of the appendix and before AA6.1.

The provisions of this Appendix are applicable to curved bridge components designed using the provisions of A6.10 and A6.11 as revised by D6.10 and D6.11. They are not applicable to straight bridge components designed using the provisions of Appendix DE.

APPENDIX B – MOMENT REDISTRIBUTION FROM INTERIOR-PIER SECTIONS IN CONTINUOUS-SPAN BRIDGES

Delete Appendix B in its entirety

The provisions of Appendix B correspond to the inelastic design procedures that are not allowed in Pennsylvania.

APPENDIX C – BASIC STEPS FOR STEEL BRIDGE SUPERSTRUCTURES

The following shall be added immediately after the heading of the appendix and before AC6.1.

The provisions of this Appendix are applicable to curved bridge components designed using the provisions of A6.10 and A6.11 as revised by D6.10 and D6.11. They are not applicable to straight bridge components designed using the provisions of Appendix DE.

REFERENCES

The following shall supplement the references of A5.

Albrecht, P., Coburn, S.K., Wattar, F.M., Tinklenberg, G.L. and W.P. Gallagher. Guidelines for the Use of Weathering Steel in Bridges. NCHRP Report 314. TRB, National Research Council, Washington, D.C., June 1989.

Mertz, D. R., "Displacement-Induced Fatigue Cracking in Welded Steel Bridges", Ph.D. dissertation, Lehigh University, 1984

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 1
PART B: DESIGN SPECIFICATIONS

SECTION E6 – STEEL STRUCTURES (APPENDIX E)

SECTION E6 - TABLE OF CONTENTS

SPECIFICATIONS

E6.10.0.1P APPLICABILITY

The provisions of Appendix E shall apply to straight I- and Box girder bridges. The article numbers and article contents correspond to the 1998 Second Edition of the AASHTO LRFD Bridge Design Specifications with the 1999 through 2003 interim specifications. It is the intent of the Department to update the STLRFD computer program to the provisions of the 2004 AASHTO LRFD Bridge Design Specifications in the future. At which point, this Appendix will be eliminated.

COMMENTARY

CE6.10.0.1P

Using the 1998 Second Edition of the AASHTO-LRFD Design specifications with the 1999 through 2003 Interims for straight girder bridges will allow continuing using the existing STLRFD computer program with relatively few revisions. It is the intent of the Department to update the STLRFD computer program to correspond to the 2004 AASHT-LRFD, including A6.10 and A6.11, at some point in the future. At which time, both straight and curved girders will be designed using the provisions of the 2004 Third Edition of the Specifications

E6.10.0.2P NOTATION

A	=	span coefficient (E6.10.4.2.2aP)
A_c	=	area of compression flange; effective combined transformed area of the top flange and concrete slab (mm^2) {$in.^2$} (E6.10.3.1.4bP) (CE6.10.4.3.2aP)
A_e	=	effective flange area (mm^2) {$in.^2$} (E6.10.3.6P)
A_{fb}	=	bottom flange area (mm^2) {$in.^2$} (E6.10.4.3.1bP)
A_{ft}	=	top flange area (mm^2) {$in.^2$}
A_g	=	gross area of a flange (mm^2) {$in.^2$} (E6.10.3.6P)
A_o	=	enclosed area within a box section (mm^2) {$in.^2$} (E6.11.2.1.2aP) (E6.11.1.2.2P)
A_{pn}	=	area of the projecting elements of a stiffener outside of the web-to-flange fillet welds but not beyond the edge of the flange (mm2) {in.2} (E6.10.8.2.3P)
A_r	=	total area of reinforcing steel within the effective flange width (mm^2) {$in.^2$} (E6.10.7.4.3P) (E6.10.3.1.4bP)
A_s	=	area of a transverse intermediate stiffener or total area of both stiffeners for pairs; gross cross-sectional area (mm^2) {in.2} (E6.10.3.1.4bP) (E6.10.8.1.4P)
A_{sc}	=	cross-sectional area of a stud shear connector (mm^2) {$in.^2$} (E6.10.7.4.4cP)
A_t	=	area of the tension flange of a steel section (mm^2) {$in.^2$} (E6.10.3.1.4bP) (E6.10.3.3.2P)
A_{tf}	=	total area of both steel flanges and the longitudinal slab reinforcement within the effective slab width of a composite section (mm2) {in.2} (E6.10.4.3.1cP)
A_w	=	area of the web of a steel section (mm^2) {$in.^2$} (E6.10.3.1.4bP)
a	=	distance from the elastic neutral axis to the centroid of the stress block (mm) {in.}; center-to-center distance between flanges of adjacent boxes in a multiple box section (mm) {in.} (CE6.10.4.3.1cP) (E6.11.1.1.1P)
B	=	constant related to the required area of transverse stiffeners (E6.10.8.1.4P)
B_r	=	bearing resistance (N) {kip} (E6.10.8.2.3P)
b	=	effective slab width; width of a rectangular plate element (E6.10.7.4.4bP) (E6.11.1.2.2P)
b_f	=	width of the compression flange of a steel section (mm) {in.} (E6.10.2.3P) (E6.10.4.1.3P)
b_{fb}	=	bottom flange width (mm) {in.} (E6.10.3.5.1P)
b_ℓ	=	projecting width of a longitudinal stiffener (mm) {in.} (E6.10.8.1.3P) (E6.11.3.2.1P)
b_t	=	width of the tension flange; projecting width of a transverse stiffener (mm) {in.} (E6.10.2.3P) (E6.10.7.4.4bP) (E6.10.8.1.2P)
b_w	=	width of the bottom flange at each edge of a flange assumed to carry wind moments (mm) {in.} (E6.10.3.5.1P)
C	=	ratio of the shear buckling stress to the shear yield strength (E6.10.6.4P)
C_b	=	moment gradient correction factor (E6.10.4.2.5aP)
c	=	coefficient in determination of bending resistance (E6.11.2.1.3aP)
D	=	actual depth of the web plate; web depth (mm) {in.} (E6.10.3.1.4bP) (E6.10.6.3P)
D'	=	depth at which a composite section reaches its theoretical plastic moment capacity when the maximum strain in the concrete slab is at its theoretical crushing strain (mm) {in.} (E6.10.4.2.2aP) (E6.10.4.2.2bP)
D_c	=	depth of web in compression in the elastic range (mm) {in.} (E6.10.2.2P)
D_{cp}	=	depth of web in compression at the plastic moment (mm) {in.} (E6.10.3.1.4bP) (E6.10.3.3.2P)

D_p = distance from the top of the slab to the neutral axis of a composite section at the plastic moment; web depth for webs without longitudinal stiffeners or maximum subpanel depth for webs with longitudinal stiffeners (mm) {in.} (E6.10.4.2.2bP) (E6.10.8.1.3P)
d = depth of steel section; diameter of stud (mm) {in.} (CE6.10.3.1.4aP) (E6.10.7.4.2P)
d_n = distance from outer fiber of bottom flange to neutral axis of transformed short-term composite section (mm) {in.} (E6.10.4.3.1bP)
d_o = spacing of transverse stiffeners (mm) {in.} (E6.10.7.3.2P)
d_s = depth of the steel section (mm) {in.} (E6.10.9.1P)
E_c = modulus of elasticity of concrete (MPa) {ksi} (E6.10.7.4.4cP)
F_n = nominal flexural resistance in terms of stress (MPa) {ksi} (E6.10.4P)
F_r = factored flexural resistance in terms of stress; flexural stress in the compression or tension flange due to the factored loading, whichever flange has the maximum ratio of f_u to F_r in the panel under consideration (MPa) {ksi} (E6.10.3.5.2P) (E6.10.4P) (E6.10.7.3.3bP)
F_u = stress in the bottom flange due to factored loadings other than wind; specified minimum tensile strength of a stud shear connector (MPa) {ksi} (E6.10.3.5.2P) (E6.10.7.4.4cP)
F_w = bending stress at the edges of the flange due to factored wind loading (MPa) {ksi} (E6.10.3.5.2P)
F_{yb} = specified minimum yield strength of a bottom flange (MPa) {ksi} (E6.10.4.3.1bP)
F_{yc} = specified minimum yield strength of a compression flange (MPa) {ksi} (E6.10.3.1.4bP)
F_{yce} = effective yield strength of a compression flange (MPa) {ksi} (E6.10.10.1.2dP)
F_{yf} = specified minimum yield strength of a flange; higher of the specified minimum yield strength of the flanges (MPa) {ksi} (E6.10.8.2.4bP) (E6.10.10.2.3P)
F_{yr} = specified minimum yield strength of longitudinal reinforcing bars (MPa) {ksi} (E6.10.3.1.4bP)
F_{yre} = effective yield strength of longitudinal reinforcement (MPa) {ksi} (E6.10.10.1.2dP)
F_{ys} = specified minimum yield strength of a stiffener (MPa) {ksi} (E6.10.8.1.2P)
F_{yt} = specified minimum yield strength of a tension flange (MPa) {ksi} (E6.10.3.1.4bP)
F_{yte} = effective yield strength of a tension flange (MPa) {ksi} (E6.10.10.1.2dP)
F_{yw} = specified minimum yield strength of the web (MPa) {ksi} (E6.10.6.3P)
F_{ywe} = effective yield strength for a web (MPa) {ksi} (E6.10.10.1.2dP)
f_c = stress in a compression flange due to the factored loading (MPa) {ksi};sum of the various compression flange flexural stresses caused by the different loads, i.e., DC1, DC2, DW, and LL+IM, acting on their respective sections (MPa) {ksi} (E6.10.2.2P) (CE6.10.3.1.4a)
f_{cf} = maximum compressive elastic flexural stress in the compression flange due to the unfactored permanent load and the fatigue loading as specified in A6.10.6.2 (MPa) {ksi} (E6.10.6.3P)
f_f = elastic flange stress caused by the factored loading (MPa) {ksi} (E6.10.10.2.2P)
f_{fl} = lesser of the specified minimum yield strength and the stress due to the factored loading in either flange (MPa) {ksi} (E6.10.4.3.1cP)
f_{fr} = redistribution flange stress (MPa) {ksi} (E6.10.10.2.2P)
f_r = stress in reinforcing steel at M_y (MPa) {ksi} (CE6.10.4.3.1cP)
f_{sr} = bending stress range in the longitudinal reinforcement over the pier (MPa) {ksi} (E6.10.7.4.3P)
f_t = sum of the various tension-flange flexural stresses caused by the different loads (MPa) {ksi} (CE6.10.3.1.4aP)
f_u = flexural stress in the compression or tension flange due to the factored loading, whichever flange has the maximum ratio of f_u to Fr in the panel under consideration (MPa) {ksi} (E6.10.7.3.3bP)
f_v = maximum torsional shear stress in the flange plate of a box girder (MPa) {ksi} (E6.11.2.1.2aP)
h_w = depth of yielded web (mm) {in.} (CE6.10.4.3.1cP)
I = moment of inertia of a short-term composite section in positive bending regions, or moment of inertia of a composite section in negative bending regions (mm^4) {$in.^4$} (E6.10.7.4.1bP)
I_ℓ = moment of inertia of a longitudinal stiffener taken about the edge in contact with the web or flange (mm^4) {$in.^4$} (E6.10.8.1.3P)
I_s = moment of inertia of a longitudinal stiffener about an axis parallel to the bottom flange and at the base of the stiffener (mm4) {in.4} (E6.11.2.1.3aP)
I_t = moment of inertia of a transverse stiffener taken about the edge in contact with the web for single stiffeners, or about the midthickness of the web for stiffener pairs (mm^4) {$in.^4$} (E6.10.8.1.3P)
I_y = moment of inertia of a steel section about the vertical axis in the plane of its web (mm^4) {$in.^4$} (E6.10.2.1P)
I_{yc} = moment of inertia of a compression flange about the vertical axis in the plane of the web (mm^4) {$in.^4$} (E6.10.2.1P)
J = St. Venant torsional inertia (mm^4) {$in.^4$} (E6.10.4.2.6aP) (E6.11.1.2.2P)

k = shear buckling coefficient; elastic bend-buckling coefficient for the web taken equal to $9.0(D/D_c)^2 \geq 7.2$ for webs without longitudinal stiffeners. For webs with longitudinal stiffeners, k shall be determined from either Equation 6.10.4.3.2a-4 or Equation 6.10.4.3.2a-5, as applicable (E6.10.6.3P) (E6.10.7.3.3aP) (E6.11.2.1.3aP)

L_b = unbraced length; distance to the first brace point adjacent to a section required to sustain plastic rotations (mm) {in.} (E6.10.4.1.7P) (E6.10.10.1.1dP)

L_c = length of a channel shear connector (mm) {in.} (E6.10.7.4.4cP)

L_{cp} = length of cover plate (mm) {in.} (E6.10.9.1P)

L_p = lateral bracing limit for flexural capacity governed by plastic bending (mm) {in.} (E6.10.4.2.6aP)

L_r = lateral bracing limit for flexural capacity governed by inelastic lateral torsional buckling (mm) {in.} (E6.10.4.2.6aP)

LE = live load

LFD = load factor design

LRFD = load and resistance factor design

M_{cp} = factored moment at an interior support concurrent with the maximum positive bending at the cross-section under consideration (N-mm) {k-in.} (E6.10.4.2.2aP)

M_h = factored moment at a plastic hinge required to sustain plastic rotations necessary to form a mechanism (N-mm) {k-in.} (E6.10.10.1.1dP) (E6.10.10.1.2bP)

M_l = lower moment due to factored loading at either end of an unbraced length (N-mm) {k-in.} (E6.10.4.1.7P)

M_{max} = maximum flexural resistance (N-mm) {k-in.} (E6.10.10.2.4dP)

M_n = nominal flexural resistance (N-mm) {k-in.} (E6.10.4P) (E6.10.4.2.3P)

M_{np} = nominal flexural resistance at an interior support (N-mm) {k-in.} (E6.10.4.2.2aP)

M_p = plastic moment resistance (N-mm) {k-in.} (E6.10.3.1.3P) (E6.10.3.3.1P)

M_{pe} = effective plastic moment resistance (N-mm) {k-in.} (E6.10.10.1.2bP)

M_r = factored flexural resistance (N-mm) {k-in.} (E6.10.4P) (E6.11.2.1.1P)

M_u = factored flexural moment; maximum panel moment due to factored loads (N-mm) {k-in.} (E6.10.9.2.1P) (E6.10.7.3.3aP)

M_w = maximum lateral moment in the bottom flange due to the factored wind loading (N-mm) {k-in.} (E6.10.3.5.1P)

M_y = yield moment resistance; yield moment when web yielding is disregarded (N-mm) {k-in.} (E6.10.3.1.2P) (E6.10.3.3.1P) (E6.10.4.3.1cP)

M_{yr} = yield moment capacity when web yielding is accounted for (N-mm) {k-in.} (E6.10.4.3.1cP)

NDT = nondestructive testing

n = number of shear connectors in a cross-section or, the number of shear connectors required between the section of maximum positive moment and the adjacent point of 0.0 moment, or between the pier and the adjacent point of 0.0 moment; number of longitudinal stiffeners; number of bolts (E6.10.3.1.1bP) (E6.10.7.4.1bP) (E6.11.2.1.3aP)

n_{AC} = number of additional connectors required in the regions of points of permanent load contraflexure for sections that are noncomposite in negative bending regions (E6.10.7.4.3P)

P_A = absolute value of the compression flange force at the quarter point of the unbraced segment (N) {kip} (CE6.10.4.2.5aP)

P_B = absolute value of the compression flange force at the centerline of the unbraced segment (N) {kip} (CE6.10.4.2.5aP)

P_c = absolute value of the compression flange force at the three-quarter point of the unbraced segment (N) {kip} (CE6.10.4.2.5aP)

P_h = force in a compression flange at the brace point with the higher moment due to factored loading (N) {kip} (E6.10.4.2.5aP)

P_l = force in the compression flange at the brace point with the lower moment due to the factored loading (N) {kip} (E6.10.4.2.5aP)

P_{max} = absolute value of the maximum compression flange force in the unbraced segment (N) {kip} (CE6.10.4.2.5aP)

p = pitch of shear connectors along the longitudinal axis (mm) {in.} (E6.10.7.4.1bP)

Q = first moment of the transformed short-term slab area about the neutral axis of the short-term composite section in positive bending regions, or the first moment of the area of the longitudinal reinforcement about the neutral axis of the composite section in negative bending regions (mm^3) {$in.^3$} (E6.10.7.4.1bP)

Q_{fl} = ratio of the buckling capacity of the flange to the yield strength of the flange (E6.10.4.2.3P)

Q_n = nominal shear strength of one shear connector (N) {kip} (E6.10.7.4.4P)

Q_p = web and compression flange slenderness to reach a flexural resistance of M_r (E6.10.4.2.3P)

Q_r = factored resistances of shear connectors (N) {kip} (E6.10.7.4.4P)

R = plastic rotation (MRADS); shear interaction factor (E6.10.10.2.4dP) (E6.10.7.3.3aP)

R_b* = load-shedding factor for composite section (CE6.10.4.3.2aP)

R_b, R_h = flange stress reduction factors (E6.10.4.3.1P) (E6.10.4.3.2P)
$(R_{sb})_n$ = nominal bearing resistance for the fitted end of bearing stiffeners (N) {kip} (E6.10.8.2.3P)
$(R_{sb})_r$ = factored bearing resistance for the fitted end of bearing stiffeners (N) {kip} (E6.10.8.2.3P)
r_t = for composite sections, radius of gyration of a notional section comprised of the compression flange of the steel section plus one-third of the depth of the web in compression taken about the vertical axis; for noncomposite sections, radius of gyration of the compression flange taken about the vertical axis (mm) {in.} (E6.10.4.1.9P) (E6.10.4.2.6aP)
r_y = minimum radius of gyration of a steel section with respect to the vertical axis in the plane of the web between brace points (mm) {in.} (E6.10.4.1.7P)
S_{xc} = section modulus about the horizontal axis of the section referred to the compression flange (mm^3) {$in.^3$} (E6.10.4.2.6aP)
T = internal torque resulting from the factored loads (N-mm) {k-in.} (E6.11.2.1.2aP)
t_b = compression flange thickness (mm) {in.} (E6.10.7.4.4bP)
t_f = compression flange thickness (mm) {in.}; flange thickness of a channel shear connector; flange thickness of channel shear connector (mm) {in.} (E6.10.4.1.3P) (E6.10.7.4.4cP)
t_{fb} = bottom flange thickness (mm) {in.} (E6.10.3.5.1P)
t_h = thickness of a concrete haunch above the top flange of the steel beam (mm) {in.} (E6.10.4.2.2bP)
t_p = thickness of the projecting element of a stiffener; thickness of a stiffener (mm) {in.} (E6.10.8.1.2P) (E6.11.3.2.1P)
t_s = thickness of a concrete slab; thickness of a stiffener plate (mm) {in.} (E6.10.4.2.2bP) (E6.10.8.3.2P)
t_t = thickness of the tension flange of a steel section (mm) {in.} (E6.10.2.3P) (E6.10.7.4.4bP)
t_w = web or tube thickness (mm) {in.}; web thickness of channel shear connector (mm) {in.} (CE6.10.4.3.1cP) (E6.10.7.4.4cP)
V_h = total horizontal shear force carried by shear connectors (N) {kip} (E6.10.7.4.4bP)
V_n = nominal shear resistance (N) {kip} (E6.10.7.1P)
V_p = plastic shear capacity (N) {kip} (E6.10.7.2P)
V_r = factored shear resistance (N) {kip} (E6.10.7.1P)
V_{sr} = shear force range (N) {kip} (E6.10.7.4.1bP)
V_u = shear due to factored loading (N) {kip} (E6.10.8.1.4P)
V_{ui} = shear due to the factored loadings on one inclined web (N) {kip} (E6.11.2.2.1P)
v_{cf} = live load shear stress (N) {kip}; shear force in web of homogeneous section with transverse stiffeners and with or without longitudinal stiffeners (N) {kip} (E6.10.6.4P)
w = width of a flange between longitudinal stiffeners or distance from the web to the nearest longitudinal stiffener (mm) {in.}; center-to-center distance between flanges of a box section (mm) {in.} (E6.11.3.2.1P) (E6.11.1.1.1P)
Z_r = shear fatigue strength of a shear connector (N) {kip} (E6.10.7.4.3P)
β = factor applied to the gross area of a flange to compute the effective flange area (E6.10.3.6P)
λ_b = coefficient related to b/t ratio (E6.10.4.2.6aP) (E6.10.4.3.2P)
θ = angle of inclination of the web plate to the vertical (DEG) (E6.11.2.2.1P)

SPECIFICATIONS

E6.10.1P General

The provisions of this article shall apply to flexure of rolled or fabricated straight steel I-sections symmetrical about the vertical axis in the plane of the web.

Hybrid sections consisting of a web with a specified minimum yield strength lower than one or both of the flanges may be designed under these specifications. Sections with a higher-strength steel in the web than in the flanges are permitted but shall not be considered hybrid sections.

The provisions of these articles shall apply to compact or noncompact sections and to composite or noncomposite sections.

Flexural members shall be designed for:

- The strength limit state flexural resistance specified in E6.10.4P
- The service limit state control of permanent deflection specified in E6.10.5P
- The fatigue and fracture limit state for details, as specified in A6.5.3, and the fatigue requirements for webs, as specified in E6.10.6P
- The strength limit state shear resistance specified in E6.10.7P and
- Constructibility specified in E6.10.3.2P.

The deflection criteria of D2.5.2.6.2 may be considered.

Open-framed systems are those which have no horizontal lateral bracing in or near the plane of the bottom flange. Lateral bracing, when used, is provided to resist wind loads, but it is generally not needed since the girders can be designed to carry wind loads between the diaphragms.

If horizontal lateral bracing is included, the open-framed system distribution factors shall be used. If a horizontal lateral bracing system is used, the connections must be detailed to ensure that the fatigue life of the bracing system is at least that of the girder.

COMMENTARY

CE6.10.1P

Noncomposite sections are not recommended but are permitted.

The application of open-framed system distribution factors for closed-framed systems is generally conservative.

If horizontal lateral bracing system is used, a rational analysis may consider a reduction in lateral live load distribution factor due to the quasi-box action of the closed-frame system.

Any reduction in live load distribution factor must be approved by the Chief Bridge Engineer.

SPECIFICATIONS

E6.10.1.1P LATERAL SUPPORT OF TOP FLANGE

The compression flanges of girders supporting timber floors shall not be considered to be laterally supported by the flooring, unless the floor and fastenings are specially designed to provide such support. Laminated timber decks shall be provided with steel clips designed to furnish adequate lateral support to the top flange.

E6.10.2P Section Proportion Limits

Based upon flexural considerations, I-section proportions shall satisfy the following inequalities in the final condition.

In E6.10.2.2P, f_c and D_c shall be taken as that appropriate for the factored loading condition under investigation.

E6.10.2.1P GENERAL PROPORTIONS

Flexural components shall be proportioned such that:

$$0.1 \leq \frac{I_{yc}}{I_y} \leq 0.9 \qquad \text{(E6.10.2.1P-1)}$$

where:

I_y = moment of inertia of the steel section about the vertical axis in the plane of the web (mm^4) {$in.^4$}

I_{yc} = moment of inertia of the compression flange of the steel section about the vertical axis in the plane of the web (mm^4) {$in.^4$}

If girders deeper than 3600 mm {12 ft.} are necessary for economical design or esthetic considerations, they shall be designed with a horizontal field splice. An approval from the Chief Bridge Engineer will be required when a horizontal field splice is incorporated. The contract plans shall include a statement to permit the elimination of the horizontal field splice at the Contractor's option.

E6.10.2.2P WEB SLENDERNESS

Webs shall be proportioned such that:

$$\frac{2D_c}{t_w} \leq 6.77\sqrt{\frac{E}{f_c}} \leq 200 \text{ without longitudinal stiffeners} \qquad \text{(E6.10.2.2P-1)}$$

COMMENTARY

CE6.10.2.1P

The ratio of I_{yc}/I_y determines the location of the shear center of a singly symmetric section. Girders with ratios outside of the limits specified are like a "T" section with the shear center located at the intersection of the larger flange and the web. The formulas for lateral torsional buckling used in the Specification are not valid for such sections.

Flanged web splices (i.e., those constructed with angles) are not desirable.

CE6.10.2.2P

The specified web slenderness limit for sections without longitudinal stiffeners corresponds to the upper limit for transversely stiffened webs in AASHTO (1996). This limit defines an upper bound below which fatigue due to excessive lateral web deflections is not a consideration (Yen and Mueller 1966; Mueller and Yen 1968).

The specified web slenderness limit for longitudinally stiffened webs is retained from the Load Factor Design portion of AASHTO (1996). Static tests of large-size plate girders fabricated from A 36 steel with D/t_w ratios greater

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$$\frac{2D_c}{t_w} \leq 13.54 \sqrt{\frac{E}{f_c}} \leq 400 \text{ with longitudinal stiffeners} \quad \text{(E6.10.2.2P-2)}$$

where:

D_c = depth of the web in compression in the elastic range (mm) {in.}

f_c = stress in the compression flange due to the factored loading under investigation (MPa) {ksi}

COMMENTARY

than 400 have demonstrated the effectiveness of longitudinal stiffeners in minimizing lateral web deflections (Cooper 1967). Accordingly, the web slenderness limit given by Equation 2 is used for girders with transverse and longitudinal stiffeners. The specified web slenderness limit is twice that for girders with transverse stiffeners only. Practical upper limits are specified on the limiting web slenderness ratios computed from either Equation 1 or 2. The upper limits are slightly above the web slenderness limit computed from Equation 1 or 2 when fc is taken equal to 250 MPa {36 ksi}.

When the compression flange is at a dead-load stress of f_c considering the deck-placement sequence, the corresponding stress in a web of slenderness $2D_c/t_w$ between the limit specified by Equation 1 and a slenderness of $\lambda_b(E/f_c)^{1/2}$, where λ_b is defined in E6.10.4.2.6aP, will be slightly above the elastic web buckling stress. For this case, the nominal flexural resistance of the steel section must be reduced accordingly by an R_b factor less than 1.0.

E6.10.2.3P FLANGE PROPORTIONS

Compression flanges on fabricated I-sections shall be proportioned such that:

$$b_f \geq 0.3 D_c \quad \text{(E6.10.2.3P-1)}$$

where:

b_f = width of the compression flange (mm) {in.}

D_c = depth of the web of the steel section in compression in the elastic range (mm) {in.}

Tension flanges on fabricated I-sections shall be proportioned such that:

$$\frac{b_t}{2t_t} \leq 12.0 \quad \text{(E6.10.2.3P-2)}$$

where:

b_t = width of tension flange (mm) {in.}

t_t = thickness of tension flange (mm) {in.}

CE6.10.2.3

The minimum compression flange width on fabricated I-sections, given by Equation 1, is specified to ensure that the web is adequately restrained by the flanges to control web bend buckling. Equation 1 specifies an absolute minimum width. In actuality, it would be preferable for b_f to be greater than or equal to $0.4D_c$. In addition, the compression flange thickness, t_f, should preferably be greater than or equal to 1.5 times the web thickness, t_w. These recommended proportions are based on a study (Zureick and Shih 1994) on doubly symmetric tangent I-sections, which clearly showed that the web bend buckling resistance was dramatically reduced when the compression flange buckled prior to the web. Although this study was limited to doubly symmetric I-sections, the recommended minimum flange proportions from this study are deemed to be adequate for reasonably proportioned singly symmetric I-sections by incorporating the depth of the web of the steel section in compression in the elastic range, D_c, in Equation 1. The advent of composite design has led to a significant reduction in the size of compression flanges in regions of positive flexure. These smaller flanges are most likely to be governed by these proportion limits. Providing minimum compression flange widths that satisfy these limits in these regions will help ensure a more stable girder that is easier to handle.

The slenderness of tension flanges on fabricated I-sections is limited to a practical upper limit of 12.0 by Equation 2 to ensure the flanges will not distort excessively when welded to the web. Also, an upper limit on the tension flange slenderness covers the case where the flange may be subject to an unanticipated stress reversal.

SPECIFICATIONS

E6.10.3P Application

E6.10.3.1P COMPOSITE SECTIONS

Composite sections shall be defined as sections consisting of a concrete deck connected to a steel section by shear connectors in accordance with E6.10.7.4P and may be applied to other deck systems that provide proven composite action and lateral support.

If concrete with expansive characteristics (except shrinkage-compensating concrete, which the Department is studying and may eventually exclude from this provision) is used, composite design shall be used with caution, and provision must be made in the design to accommodate the expansion.

Composite section properties (see D6.10.3.1.1b) shall be assumed in the positive and negative moment regions for the calculation of design moments, shears and deflections.

In the computation of flexural stiffness and flexural resistance of beams, the haunch shall be taken as zero. However, in the computation of dead load, the haunch shall be taken into account.

E6.10.3.1.1P Stresses

E6.10.3.1.1aP Sequence of Loading

The elastic stress at any location on the composite section due to the applied loads shall be the sum of the stresses caused by the loads applied separately to the:

- Steel,
- Short-term composite section, and
- Long-term composite section.

Permanent load that is applied before the slab has attained 75 percent of f'_c shall be assumed to be carried by the steel section alone. Permanent and live load that is applied after the slab has attained 75 percent of f'_c shall be assumed to be carried by the composite section. For shored construction, all permanent load shall be assumed to be applied after the slab has attained 75 percent of f'_c, and the contract documents will so indicate.

For continuous spans, the final dead load moment at each design section shall be taken as the greater of either the dead load moment considering the weight of the concrete deck to be instantaneously applied or a moment based upon an incremental analysis of the specified slab placement

COMMENTARY

EC6.10.3.1P

If the concrete is expansive, estimate expansion and properly design concrete to flange connection by adding additional shear studs.

Field measured haunch depths may be used in the computation for flexural stiffness and resistance when rating existing bridges.

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sequence. Similarly, stresses should be computed based on the more critical of the incremental and instantaneously applied loads.

E6.10.3.1.1bP Positive Flexure

For calculating flexural stresses, the composite section shall consist of the steel section and the transformed area of an effective width of the concrete slab.

Positive flexure section properties shall be assumed only when the factored composite loads produce compression in the slab. Otherwise, the negative flexure section properties (see E6.10.3.1.1cP) shall be used.

For transient loads assumed to be applied to the short-term composite section, the slab area shall be transformed by using the short-term modular ratio, n.

For permanent loads assumed to be applied to the long-term composite section, the slab area shall be transformed by using a modular ratio of 3n.

For normal and low density concrete, the modular ratio is given in D5.4.2.1.

E6.10.3.1.1cP Negative Flexure

For calculating stresses from moments, the composite section for both long-term and short-term composite moments shall consist of the steel section and the longitudinal reinforcement within an effective width of the slab**.**

Cut-off points for the main reinforcement in cast-in-place decks over interior supports for continuity may be staggered as required by design.

Concrete on the tension side of the neutral axis shall not be considered in calculating resisting moments.

E6.10.3.1.1dP Effective Width of Slab

In the absence of better information, the provisions of D4.6.2.6 shall apply.

E6.10.3.1.2P Yield Moment

The yield moment, M_y, of a composite section shall be taken as the sum of the moments applied separately to the steel and the short-term and long-term composite sections to cause first yielding in either steel flange when any web yielding in hybrid sections is disregarded.

COMMENTARY

EC6.10.3.1.1bP

It is preferable to proportion composite sections in simple spans and the positive moment regions of continuous spans so that the neutral axis lies below the top surface of the steel beam.

CE6.10.3.1.2P

The yield moment, M_y, of a composite section is needed only for the strength limit state investigation of the following types of composite sections:

- Compact positive bending sections in continuous spans,
- Negative bending sections designed by the Q formula,

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- Hybrid negative bending sections for which the neutral axis is more than 10 percent of the web depth from middepth of the web,

A procedure for calculating the yield moment is presented in Appendix A.

E6.10.3.1.3P Plastic Moment

M_p shall be calculated as the first moment of plastic forces about the plastic neutral axis. Plastic forces in steel portions of a cross-section shall be calculated using the yield strengths of the flanges, the web, and reinforcing steel, as appropriate. Plastic forces in concrete portions of the cross-section that are in compression may be based on the rectangular stress block as specified in A5.7.2.2. Concrete in tension shall be neglected.

The position of the plastic neutral axis shall be determined by the equilibrium condition that there is no net axial force.

CE6.10.3.1.3P

The plastic moment of a composite section in positive flexure can be determined by:

- Calculating the element forces and using them to determine whether the plastic neutral axis is in the web, top flange, or slab,
- Calculating the location of the plastic neutral axis within the element determined in the first step; and
- Calculating M_p. Equations for the five cases most likely to occur in practice are given in Appendix A.

The forces in the longitudinal reinforcement may be conservatively neglected. To do this, set P_{rb} and P_{rt} equal to 0 in the equations in Appendix A.

The plastic moment of a composite section in negative flexure can be calculated by an analogous procedure. Equations for the two cases most likely to occur in practice are also given in Appendix A.

E6.10.3.1.4P Depth of Web in Compression

E6.10.3.1.4aP At Elastic Moment

For sections in positive flexure, the depth of the web in compression, D_c, at the elastic design moment shall be the depth over which the algebraic sum of the stresses in the steel, long-term composite, and short-term composite sections from the dead and live loads, plus impact, is compressive. For sections in negative flexure, D_c may be computed for the section consisting of the steel girder plus the longitudinal reinforcement.

CE6.10.3.1.4aP

For composite sections, D_c is a function of the algebraic sum of the stresses caused by loads acting on the steel, long-term composite, and short-term composite sections. Thus, D_c is a function of the dead-to-live load stress ratio. At sections in positive flexure, D_c of the composite section will increase with increasing span because of the increasing dead-to-live load ratio. As a result, using D_c of the short-term composite section, as has been customary in the past, is unconservative. In lieu of computing D_c at sections in positive flexure from the stress diagrams, the following equation may be used:

$$D_c = \left[\frac{|f_c|}{|f_c| + f_t}\right] d - t_f \qquad \text{(CE6.10.3.1.4aP-1)}$$

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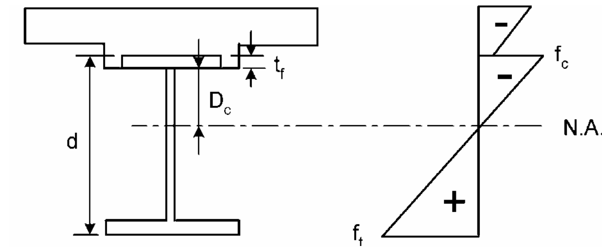

Figure CE6.10.3.1.4aP-1 - Computation of D_c at Sections in Positive Flexure

where:

f_c = sum of the various compression-flange flexural stresses caused by the different loads, i.e., DC1, the component dead load acting on the noncomposite section; DC2, the component dead load acting on the long-term composite section; DW, the wearing surface load; and LL+IM; acting on their respective sections (MPa) {ksi}

f_t = sum of the various tension-flange flexural stresses caused by the different loads (MPa) {ksi}

d = depth of the steel section (mm) {in.}

t_f = the thickness of the compression flange (mm) {in.}

At sections in negative flexure, using D_c of the composite section consisting of the steel section plus the longitudinal reinforcement is conservative.

E6.10.3.1.4bP At Plastic Moment

The depth of web in compression at the plastic moment, D_{cp}, may be determined as:

- For sections in positive flexure, where the plastic neutral axis is in the web, D_{cp} shall be taken as:

$$D_{cp} = \frac{D}{2}\left[\frac{F_{yt}A_t - F_{yc}A_c - 0.85 f_{c'}A_s - F_{yr}A_r}{F_{yw}A_w} + 1\right] \quad \text{(E6.10.3.1.4bP-1)}$$

where:

D_{cp} = depth of the web in compression at the plastic moment (mm) {in.}

CE6.10.3.1.4b

The location of the neutral axis may be determined from the conditions listed in Appendix A.

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D = web depth (mm) {in.}

A_s = area of the slab (mm^2) {$in.^2$}

A_t = area of the tension flange (mm^2) {$in.^2$}

A_c = area of the compression flange (mm^2) {$in.^2$}

A_w = area of the web (mm^2) {$in.^2$}

A_r = area of the longitudinal reinforcement included in the section (mm^2) {$in.^2$}

F_{yc} = specified minimum yield strength of the compression flange (MPa) {ksi}

F_{yr} = specified minimum yield strength of the longitudinal reinforcement included in the section (MPa) {ksi}

F_{yt} = specified minimum yield strength of the tension flange (MPa) {ksi}

F_{yw} = specified minimum yield strength of the web (MPa) {ksi}

f'_c = specified 28-day compressive strength of the concrete (MPa) {ksi}

- For all other sections in positive flexure; D_{cp} shall be taken to be equal to 0, and the compact-section web slenderness requirement in E6.10.4.1.2P shall be considered to be satisfied.

- For sections in negative flexure, where the plastic neutral axis is in the web;

$$D_{cp} = \frac{D}{2A_w F_{yw}} \left(F_{yt} A_t + F_{yw} A_w + \ldots \right.$$
$$\left. F_{yr} A_r - F_{yc} A_c \right) \qquad \text{(E6.10.3.1.4bP-2)}$$

- For all other sections in negative flexure; D_{cp} shall be taken to be equal to D.

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E6.10.3.2P CONSTRUCTIBILITY

E6.10.3.2.1P General

To ensure constructibility, sections that are composite in the final condition but noncomposite during construction shall be investigated as noncomposite sections during the various phases of the deck placement sequence using the appropriate strength load combination in Articles A3.4.1 and D4.4.1. Geometric properties, bracing lengths, and stresses used in calculating nominal flexural resistance in accordance with this article shall be for the steel section only under the factored construction loads. The changes to load, stiffness, and bracing during the stages of the pouring sequence shall be considered.

COMMENTARY

CE6.10.3.2.1P

The entire concrete deck may not be cast in one stage; thus parts of the girders may become composite in sequential stages. If certain deck casting sequences are followed, the temporary moments induced in the girders during the deck staging can be considerably higher than the final noncomposite dead load moments after the sequential casting is complete, and all the concrete has hardened.

Economical composite girders normally have smaller top flanges than bottom flanges in positive bending regions. Thus, more than half of the noncomposite web depth is typically in compression in these regions during deck construction. If the higher moments generated during the deck casting sequence are not considered in the design, these conditions, coupled with narrow top compression flanges, can lead to problems during construction, such as out-of-plane distortions of the girder compression flanges and web. Limiting the length of girder shipping pieces to approximately 85 times the minimum compression-flange width in the shipping piece can help to minimize potential problems.

Sequentially staged concrete placement can also result in significant tensile strains in the previously cast deck in adjacent spans. Temporary dead load deflections during sequential deck casting can also be different from final noncomposite dead load deflections. This should be considered when establishing camber and screed requirements. These constructibility concerns apply to deck replacement construction as well as initial construction.

During construction of steel girder bridges, concrete deck overhang loads are typically supported by cantilever forming brackets placed every 900 or 1200 mm {3 or 4 ft.)along the exterior members. Bracket loads applied eccentrically to the exterior girder centerline create applied torsional moments to the exterior girders at intervals in between the cross-frames, which tend to twist the girder top flanges outward. As a result, two potential problems arise:

- The applied torsional moments cause additional longitudinal stresses in the exterior girder flanges, and

- The horizontal components of the resultant loads in the cantilever-forming brackets are often transmitted directly onto the exterior girder web. The girder web may deflect laterally due to these applied loads.

Consideration should be given to these effects in the design of exterior members. Where practical, forming brackets should be carried to the intersection of the bottom flange and the web.

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6.10.3.2.2 Nominal Flexural Resistance

The following shall supplement A6.10.3.2.2.

The compressive bending stress due to unfactored dead load and construction loads in webs with or without longitudinal stiffeners shall be limited as follows:

$$f_{cw} \leq \frac{5.4\,E}{\gamma_w \left(\frac{D_c}{t_w}\right)^2} \quad \text{(E6.10.3.2.2P-1)}$$

where:

f_{cw} = compressive bending stress in the web due to unfactored dead load and construction loads (MPa) {ksi}

The calculation of f_{cw} shall consider (but is not limited to) the following:

- steel girder weight
- concrete deck weight
- deck form weight
- construction equipment weight
- concrete haunch weight
- pouring sequence effect
- camber effect

E = modulus of elasticity of steel (MPa) {ksi}

D_c = For webs without longitudinal stiffeners, clear distance between the neutral axis of the non-composite section for dead load and the compression flange. For webs with longitudinal stiffeners, clear distance between the neutral axis of the section for dead loads and the longitudinal stiffener (mm) {in.}.

t_w = web thickness (mm) {in.}

γ_w = 1.0 where diaphragms or cross-frames are not staggered; 1.3 where diaphragms or cross-frames are staggered; as required by the Chief Bridge Engineer for other conditions

For sections where the slenderness limit given by Equation 2 are not met, the factored shear resistance, V_r, shall be greater than or equal to four times the shear due to the unfactored dead load. For non-composite sections not

COMMENTARY

C6.10.3.2.2

The following shall supplement AC6.10.3.2.2.

Equation 1 is derived from the basic plate buckling formula for Case 5, shown in Figure C1. Case 5 addresses buckling due to flexural compressive stress in a plate girder. Note that transverse stiffener spacing has little influence on flexural buckling.

For γ_w = 1.0, f_{cw} is the critical buckling stress. Since all webs have some initial out-of-flatness, buckling occurs at a stress smaller than the critical buckling stress. Buckling does not occur suddenly; rather, lateral deflection of the web increases as the moment increases. After buckling occurs, the flexural stress carried by the web is redistributed to the compression flange.

The LRFD Specification may allow flexural buckling to occur to various degrees under construction loading before composite action develops. This buckling, together with initial out-of-flatness, may cause out-of-plane vibrations under live load. Studies have shown that larger initial out-of-flatness produces changing lateral deflection ("oil canning" or "vibrating") under live load, and, therefore, creates a potential for fatigue problems under certain conditions. Also, inspection of older existing bridges has revealed out-of-plane vibrations of webs under live load. Where Category E details (such as lateral bracing connections) are connected to the vibrating webs, stress ranges (determined from strain gages) were observed in one case to be as much as twice that which would occur if the webs were not vibrating, and estimated fatigue life was reduced by 75 to 90 percent of the fatigue life that would be expected without web vibration. The magnitude of oil canning varies from case-to-case.

Limited studies of existing structures suggest that web vibration will not be a serious problem if Category E details do not exist in the areas of web vibration; in that case category, C would determine fatigue life.

Additional studies are needed to confirm this. The potential for fatigue resulting from vibrations of girder webs under live load is a problem that may not have been adequately addressed in the LRFD Specification A6.10.6.3, which was intended to control fatigue due to the lateral deflection of girder webs under live load, may not adequately model in-service performance. This potential problem becomes more important in structures designed using more refined methods of analysis, because the conservative difference between the in-service stress range and the design stress range for conventional design may be significantly reduced by the refined method.

The γ_w factor in Equation 1 is intended to provide for the potential reduction in fatigue life due to web vibrations which may be introduced as a result of web buckling during construction.

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meeting the requirements of Equation 2, V_r shall be greater than or equal to four times the total unfactored shear, including live load.

$$\frac{5.4\,E}{f_{cw}\left(\frac{D_c}{t_w}\right)^2} \geq 2.50 \qquad \text{(E6.10.3.2.2P-2)}$$

COMMENTARY

$$F_{cr} = \frac{k\pi^2 E}{12\,(1-\mu^2)}\left(\frac{t_w}{b}\right)^2 = \text{Critical Buckling Stress}$$

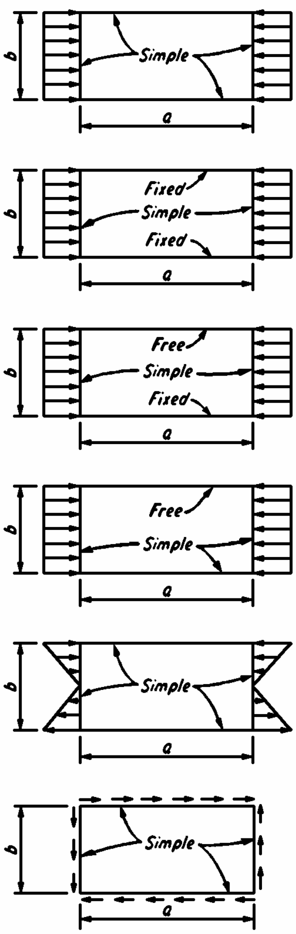
For μ = 0.3 = Poisson's ratio for steel,

k = 23.9 (Case 5), and

b = 2 D_c

$$F_{cr} = \frac{5.4\,E}{\left(\frac{D_c}{t_w}\right)^2} = f_{cw} \text{ when } \gamma_w = 1.0 \text{ in Equation 1}$$

VALUES OF *k* FOR PLATE BUCKLING



Case 1

a/b	0.2	0.3	0.4	0.5	0.6	0.7	0.8
k	27.0	13.2	8.41	6.25	5.14	4.53	4.20
a/b	0.9	1.0	1.1	1.2	1.3	1.4	1.41
k	4.04	4.00	4.04	4.13	4.28	4.47	4.49

Case 2

a/b	0.4	0.5	0.6	0.7	0.8	0.9	1.0
k	9.44	7.69	7.05	7.00	7.29	7.83	7.69

Case 3

a/b	1.0	1.1	1.2	1.3	1.4	1.5	1.6
k	1.70	1.56	1.47	1.41	1.36	1.34	1.33
a/b	1.7	1.8	1.9	2.0	2.2	2.4	
k	1.33	1.34	1.36	1.38	1.45	1.47	

Case 4

a/b	0.5	1.0	1.2	1.4	1.6	1.8	2.0
k	4.40	1.44	1.14	0.952	0.835	0.755	0.698
a/b	2.5	3.0	4.0	5.0	For longer plates		
k	0.610	0.504	0.516	0.506	$k = 0.456 + (a/b)^2$		

Case 5

a/b	0.4	0.5	0.6	0.67	0.75
k	29.1	25.6	24.1	23.9	24.1
a/b	0.8	0.9	1.0	1.33	1.5
k	24.4	25.6	25.6	23.9	24.1

Case 6*

a/b	1.0	1.2	1.4	1.6	1.8	2.0	3.0	5.0	∞
k	9.4	8.0	7.3	7.0	6.8	6.6	6.1	5.6	5.3

* *b* is always the short dimension for Case 6.

Figure CE.6.10.3.2.2P-1 – Formulas for Plate Buckling

SPECIFICATIONS

E6.10.3.2.3P Nominal Shear Resistance

For webs of homogeneous sections with transverse stiffeners, and with or without longitudinal stiffeners, the nominal shear resistance for investigation of the factored permanent load shall be taken as:

$$V_n = CV_p \qquad \text{(E6.10.3.2.3P-1)}$$

where:

C = ratio of the shear buckling stress to the shear yield strength as specified in E6.10.7.3.3aP

V_p = plastic shear capacity specified in E6.10.7.3.3aP (N) {kip}

E6.10.3.2.4P Erection Analysis

Evaluate lateral deflections in accordance with BD-620M.

E6.10.3.2.4.aP Slab Placement

An analysis shall be performed to determine an acceptable slab placement sequence. The analysis shall address (but is not limited to) the following items:

(a) Change in the stiffness in the girder as different segments of the slab are placed and as it affects both the temporary stresses and the potential for "locked-in" erection stresses

(b) Bracing (or lack thereof) of the compression flange of girders and its effect on the stability and strength of the girder

(c) Stability and strength of the girder through slab placement

(d) Bracing of overhang deck forms

(e) Uplift at bearings

The analysis of slab placement shall be done in an incremental fashion using a concrete modulus of elasticity equal to 70% of the concrete modulus elasticity at 28 days for concrete which is at least 24 hours old, assuming no retarder admixture is permitted. If retarder admixture is specified, it shall be indicated on the contract drawings, and the analysis shall be completed assuming 48 hours before gaining stiffness for lateral resistance. This means the stiffness of the model will change at the many different stages.

COMMENTARY

CE6.10.3.2.3P

The web is investigated for the sum of the factored permanent loads acting on both the noncomposite and composite sections during construction because the total shear due to these loads is critical in checking the stability of the web during construction. The nominal shear resistance for this check is limited to the shear buckling or shear yield force. Tension field action is not permitted under factored dead load alone. The shear force in unstiffened webs and in webs of hybrid sections is limited to either the shear yield or shear buckling force at the strength limit state, consequently the requirement in this article need not be investigated for those sections.

CE6.10.3.2.4aP

During the mid-1980's, several of the Department's girder bridges experienced problems during placement of the slab. It is believed that bridges with highly unsymmetrical, deep steel girders combined with wide beam spacing and large overhang dimensions are more susceptible to problems during construction than are the typical earlier steel girder bridges which use more nearly symmetrical steel girders combined with closer beam spacing and smaller overhang dimensions. Since significant reduction in the construction cost of a bridge can be achieved by use of highly unsymmetrical, deep steel girders in conjunction with wide beam spacing and large overhang dimensions, an analysis must be performed to ensure that these types of girders provide adequate stability and strength through slab placement.

With skewed, curved, and/or continuous steel girder bridges, temporary uplift conditions at bearings can occur during the deck pour. Designers should evaluate the potential for uplift in bearings as part of the deck pour sequence evaluation. Designers should address temporary uplift conditions as follows:

- Where the temporary uplift is not detrimental to the long-term performance of the bearing, or does not result in adverse stability conditions, temporary uplift is permitted. In this case, the designer should identify in the construction plans the individual bearing locations where uplift is expected and during what stages of the deck pour the uplift will occur. A note stating that the uplift is temporary and permitted as part of construction

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should also be provided in the construction plans.

Where uplift is determined to be unacceptable for individual bearing types or structure stability, the designer should identify in the construction plans the individual bearing locations where uplift is expected. Hold down forces and any other design requirements for restraining devices should be shown in the plans for the contractor's use in designing these components. Forces and design requirements for individual deck pour stages, as applicable, should be provided. The designer should verify the viability of at least one type of restraining device to meet the design requirements and provide schematic details of the device in the construction plans.

In no case shall the final design moment stresses or forces be less than those determined from an analysis in which the weight of the deck slab is applied all at once.

Slab concrete, which is less than 24 hours old (or 48 hours old when retarder is used), cannot be considered to provide lateral support for the embedded top flange of the girder. Conversely, slab concrete which is more than 24 hours old (or 48 hours old when retarder is used) can be considered to provide full lateral support for the embedded top flange of the girder. If the contractor can demonstrate that the concrete will provide lateral support for the embedded top flange in less than 24 hours (or 48 hours old when retarder is used), that limiting time may be used with the approval of the Chief Bridge Engineer.

From the results of the analysis of slab placement and lateral support conditions described above, the bending and shear strength of girder shall be checked.

E6.10.3.2.4.bP Deck Slab Overhang Form Support

For the erection condition with the overhang form support system, the strength and stability of the fascia girder shall be ensured by applying the dead load of the overhang concrete and any construction equipment to the girder as follows:

(a) The standard form support system, shown in Figure 1, may be used where:

(1) Girder web depth is less than 2400 mm {8'-0"}

(2) Deck slab overhang is less than 1400 mm {4'-9"}

(3) Slab thickness is equal to or less than 250 mm {10 in.}

(4) Transverse stiffener spacing does not exceed the depth of the girder

CE6.10.3.2.4bP

The requirements of this article can be met by reducing the length of some deck pours, or by increasing the size of the steel girder section, or by a combination of both. For original designs, the designer should obtain input from contractor and fabricators about the economics of those alternatives. Note, also, that only a relatively short length of the critical spans will be affected by the constructibility criterion.

The intent of the required checks is to control the buckling of the flanges and the webs of steel girders. It is felt that there is a potential for fatigue cracking if steel plates are allowed to buckle due to "oil-canning" effects.

The preferred upper limit on the deck slab overhang is 1200mm {4'-0"}considering factors such as deck forming and deck finishing.

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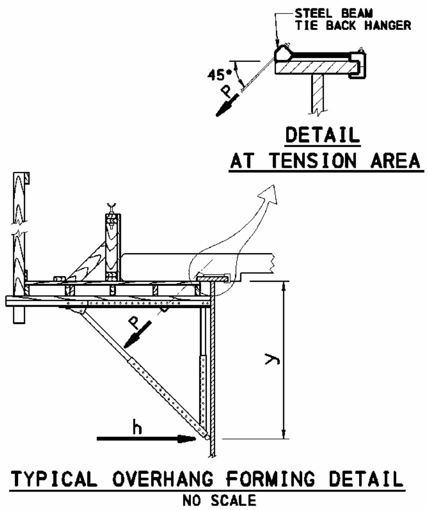
COMMENTARY

(5) In regions where γ_w (see E6.10.3.2.2P) is less than 2.5, the factored dead load shear using a load factor of 4.0 is less than the buckling shear given in E6.10.7.3P.



NOMINAL DEPTH y mm (in)	MAXIMUM PERMISSIBLE HORIZONTAL LOAD h N/mm (kip/ft)
915 (36)	18.25 (1.250)
1420 (56)	10.95 (0.750)
1930 (76)	7.88 (0.540)

MAXIMUM PERMISSIBLE JACK SPACING 1220 mm (4 ft)

The fascia girders are designed for a temporary construction load applied to the web at a maximum 1220 mm{4 ft.} interval. This load (see table) approximates the horizontal component of a deck overhang form support bracket and consists of an allowance for the weight of the concrete, forms and incidental loads, plus the deck finishing machine. Where a transverse stiffener spacing, less than that required for the final design shear, is indicated for constructibility, the spacing for the final design shear may be used if the overhang forms are supported from the bottom flange of the fascia girder, or if the girder web is adequately braced to prevent buckling due to loads from web-bearing form support brackets. The contractor has the option to modify the overhang bracket from that described herein provided working drawings including calculations, sealed by a professional engineer licensed in the Commonwealth of Pennsylvania, are submitted for review and acceptance and show the modifications do not cause unacceptable deformations or stresses in the bridge and it is understood the contractor is ultimately responsible for the satisfactory completion of the bridge.

Figure E6.10.3.2.4.2P-1 - Typical Overhang Forming Detail and Note

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Where these requirements are satisfied, original designs of fascia girders shall provide transverse stiffeners throughout the span at a maximum spacing of D, including the region where stiffeners are not required for the final design shear or where a spacing larger than D would be satisfactory for the final design shear. This requirement ensures reasonable constructibility. The stiffener spacings required for both constructibility and final design shear shall be shown on the contract drawings (preferably on the girder elevations), and the sketch and note from Figure 1 shall be included on the contract drawings.

(b) For deck slab overhangs which do not meet the requirements of (a), the designer of the original structure shall review the condition with the Chief Bridge Engineer's office as part of the TS&L submission. If it is determined that web-supported overhang form brackets cannot be permitted, the following note shall be included in the general notes:

Support deck slab overhang forms from the bottom flange of the fascia girder, unless the girder web is adequately supported to prevent buckling due to loads from web-bearing form supports.

E6.10.3.2.4.3P Deck Slab Overhang Rotation

The designer shall consider the effects of out-of-plane girder rotations, common with skewed bridges, on deck elevations.

COMMENTARY

For rolled beams spans, this is typically not a controlling design consideration with small deck overhangs less than 600 mm (2 ft) overhangs.

The revision to this note was developed by an APC Subcommittee for Stability of Steel Bridge Superstructures. The note was modified to provide more flexibility to the contractor to use deck overhang form brackets that have nominal depths greater than the typical 915 mm (36 inch) bracket depth. The maximum permissible horizontal load value was developed based on field measurements of steel bridges constructed in 1999 in District 5-0 with deck overhangs in the range of 1422 mm (4'-8") and a limited finite element analysis study of lateral web deflections of steel girders subjected to concentrated horizontal forces on the girder web.

Design modifications should consider web stress, overall web deformation, relative web deformation, the resulting deck overhang deflection, and the resulting effects on the finished deck profile. The contractor is responsible for selecting and providing calculations for the overhang forming system as required by Publication 408 Section 1050.3(c)2. Publication 408 Section 105.01 (c) specifies the responsibility of the work remains with the contractor regardless of reviews and/or acceptance of submitted working drawings by the Department.

Unacceptable deformations of the web or top flange results in deflection of the overhang bracket causing problematic deck finish and ride quality.

If an overhang is braced to within 6 inches of the bottom flange, it shall be considered braced to the bottom flange. Deck overhang forms for rolled beams spans, due to there shallow depth, are typically supported in this manner.

C6.10.3.2.4.3P

Out-of-plane girder rotations will cause the overhang formwork to also rotate. In an increasing magnitude from the web of the fascia girder to the outside edge of the formwork, the formwork will move upward or downward, depending on the direction of rotation, during the deck pour. It may be desirable to pre-rotate the overhang formwork so that the as-designed deck overhang cross slope is obtained after the deck pour is complete. Additionally, it may be desirable to relocate the deck finishing machine support

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railing from its typical position on the overhang formwork to the fascia girders. This will minimize the upward and downward movements of the finishing machine during the deck pour due to out-of-plane girder rotations. Hand finishing work will be necessary for the deck area beyond the limits of the finishing machine.

The designer may consider approximating the anticipated girder rotation based on girder differential vertical displacements.

E6.10.3.3P NONCOMPOSITE SECTIONS

Wherever technically feasible, all structures shall be made composite.

Sections where the deck is not connected to the steel section by shear connectors designed in accordance with E6.10.7.4P are considered noncomposite sections.

For non-composite sections in addition to the constructibility check of the compressive bending stress in the webs given in E6.10.3.2.2, Equation E6.10.3.2.2-1 must be satisfied where:

f_{bw} = compressive bending stress in the web due to unfactored dead and live loads (MPa) {ksi}

E6.10.3.3.1P Yield Moment and Plastic Moment

The yield moment, M_y, of a noncomposite section shall be taken as the moment required to cause first yielding in either flange when any web yielding in hybrid sections is disregarded.

CE6.10.3.3.1P

The plastic moment, M_p, of a noncomposite section shall be taken as the resultant moment of the fully plastic stress distribution acting on the section.

The plastic moment of noncomposite sections may be calculated by eliminating the terms pertaining to the concrete slab and longitudinal reinforcement from the equations in Appendix A for composite sections.

E6.10.3.3.2P Depth of Web in Compression at the Plastic Moment

The depth of web in compression at the plastic moment shall be determined as:

If:

$$F_{yw} A_w \geq |F_{yc} A_c - F_{yt} A_t|,$$

Then

$$D_{cp} = \frac{D}{2 A_w F_{yw}} (F_{yt} A_t + F_{yw} A_w - F_{yc} A_c) \quad \text{(E6.10.3.3.2P-1)}$$

Otherwise:

$$D_{cp} = D \quad \text{(E6.10.3.3.2P-2)}$$

CE6.10.3.3.2P

If the inequality is satisfied, the neutral axis is in F_{yw} the web. If it is not, the neutral axis is in the flange, and D_{cp} is equal to the depth of the web.

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where:

D = web depth (mm) {in.}

A_t = area of the tension flange (mm^2) {$in.^2$}

A_c = area of the compression flange (mm^2) {$in.^2$}

A_w = area of the web (mm^2) {$in.^2$}

F_{yc} = specified minimum yield strength of the compression flange (MPa) {ksi}

F_{yt} = specified minimum yield strength of the tension flange (MPa) {ksi}

F_{yw} = specified minimum yield strength of the web (MPa) {ksi}

E6.10.3.4P STIFFNESS

The following stiffness properties shall be used in the analysis of flexural members:

- For loads applied to noncomposite sections: the stiffness properties of the steel section.
- For permanent loads applied to composite sections: the stiffness properties of the long-term composite section, assuming the concrete deck to be fully effective over the entire span length.
- transient loads applied to composite sections: the stiffness properties of the short-term composite section, assuming the concrete deck to be fully effective over the entire span length.

COMMENTARY

CE6.10.3.4P

In line with common practice, it is specified that the stiffness of the steel section alone be used for noncomposite sections, even though numerous field tests have shown that considerable unintended composite action occurs in such sections.

Field tests of composite continuous bridges have shown that there is considerable composite action in negative bending regions (Baldwin et al. 1978; Roeder and Eltvik 1985). Therefore, it is conveniently specified that the stiffness of the full composite section may be used over the entire bridge length, where appropriate.

The Engineer may use other stiffness approximations based on sound engineering principles. One alternative is to use the cracked-section stiffness for a distance on each side of piers equal to 15 percent of each adjacent span length. This approximation is used in Great Britain (Johnson and Buckby 1986).

E6.10.3.5P WIND EFFECTS ON GIRDER FLANGES

E6.10.3.5.1P Compact Sections

The factored moment in the bottom flange due to lateral wind shall be assumed to be carried by a width, b_w, at each edge of the flange, given by:

$$b_w = \frac{b_{fb} - \sqrt{b_{fb}^2 - \dfrac{4M_w}{t_{fb}F_{yb}}}}{2} \leq \frac{b_{fb}}{2} \qquad \text{(E6.10.3.5.1P-1)}$$

where:

b_{fb} = bottom flange width (mm) {in.}

CE6.10.3.5.1P

Compact sections are designed to sustain the plastic moment, which theoretically causes yielding of the entire cross-section. Therefore, the combined effects of wind and other loadings cannot be accounted for by summing the elastic stresses caused by the various loadings. Instead, it is assumed that the lateral wind moment is carried by a pair of fully yielded widths that are discounted from the section assumed to resist the vertical loads. Determination of the wind moment in the flange is covered in A4.6.2.7.

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t_{fb} = bottom flange thickness (mm) {in.}

F_{yb} = specified minimum yield strength of the bottom flange (MPa) {ksi}

M_w = maximum lateral moment in the bottom flange due to the factored wind loading (N-mm) {k-in}

Vertical loads in the load combination being investigated shall be assumed to be carried by an effective composite section obtained by removing a width, b_w, from each edge of the bottom flange. All required resistance determinations shall be based on this effective section.

E6.10.3.5.2P Noncompact Sections

Stresses in the bottom flange of noncompact sections shall be combined as follows:

$$\left(F_u + F_w \right) \leq F_r \qquad \text{(E6.10.3.5.2P-1)}$$

for which:

$$F_w = \frac{6M_w}{t_{fb}\, b_{fb}^2} \qquad \text{(E6.10.3.5.2P-2)}$$

where:

F_w = flexural stress at the edges of the bottom flange due to the factored wind loading (MPa) {ksi}

F_u = flexural stress in bottom flange due to the factored loadings other than wind (MPa) {ksi}

F_r = factored flexural resistance of each flange specified in E6.10.4P (MPa) {ksi}

M_w = maximum lateral moment in the bottom flange due to the factored wind loading (N-mm) {k-in}

No investigation of the top flange shall be required. If the nominal flexural resistance is calculated by the provisions of E6.10.4.2.3P, the effective section, defined in E6.10.3.5.1P, shall be used in these calculations.

COMMENTARY

CE6.10.3.5.2P

For noncompact sections, the combined effects of wind and other loadings are accounted for by summing the elastic stresses caused in the bottom flange by the various loadings. The wind stress in the bottom flange is equal to the wind moment divided by the section modulus of the flange acting in the lateral direction.

The peak wind stresses may be conservatively combined with peak stresses from other loadings, even though they may occur at different locations. This is justified because the wind stresses are usually small and generally do not control the design.

For investigating wind loading on sections designed by the optional Q formula specified in E6.10.4.2.3P, it is necessary to apply the procedures specified in E6.10.3.5.1P for compact sections, even if the actual sections are not compact, because the design using the optional Q formula is performed in terms of moment, rather than stresses.

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E6.10.3.6P EFFECTIVE SECTION

At the strength limit state, the section properties for flexural members with holes in the tension flange shall be computed using an effective flange area. The effective tension flange area shall be taken as:

$$A_e = A_n + \beta A_g \leq A_g \qquad \text{(E6.10.3.6P-1)}$$

in which:

$$\beta = \left(\frac{A_n}{A_g}\right)\left[\left(\frac{\phi_u F_u}{\phi_y F_{yf}}\right) - 1\right] \geq 0.0$$

where:

A_n = net area of the flange calculated as specified in A6.8.3 and D6.8.3 (mm^2) {$in.^2$}

A_g = gross area of the flange (mm^2) {$in.^2$}

φ_u = resistance factor for fracture of tension members as specified in A6.5.4.2 and D6.5.4.2

φ_y = resistance factor for yielding of tension members as specified in A6.5.4.2 and D6.5.4.2

F_u = specified minimum tensile strength of the flange specified in Table A6.4.1-1 (MPa) {ksi}

F_{yf} = specified minimum yield strength of the flange (MPa) {ksi}

The effective compression flange area shall be taken as:

$$A_e = A_g \qquad \text{(E6.10.3.6P-2)}$$

COMMENTARY

7CE6.10.3.6P

Equation 1 defines an effective area for a tension flange with holes to be used to determine the section properties for a flexural member at the strength limit state. The equation replaces the 15 percent rule given in past editions of the Standard Specifications and the First Edition of the LRFD Specifications. If the stress due to the factored loads on the effective area of the tension flange is limited to the yield stress, fracture on the net section of the flange is theoretically prevented and need not be explicitly checked.

The effective area is equal to the net area of the flange plus a factor β times the gross area of the flange. The sum is not to exceed the gross area. For AASHTO M 270M, Grade 690 or 690W steels, with a yield-to-tensile strength ratio of approximately 0.9, the calculated value of the factor β from Equation 1 will be negative. However, since β cannot be less than 0.0 according to Equation 1, β is to be taken as 0.0 for these steels resulting in an effective flange area equal to the net flange area. For all other steels, the factor β depends on the ratio of the tensile strength of the flange to the yield strength of the flange and on the ratio of the net flange area to the gross flange area.

For compression flanges, net section fracture is not a concern and the effective flange area is to be taken as the gross flange area as defined in Equation 2.

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E6.10.3.7P MINIMUM NEGATIVE FLEXURE SLAB REINFORCEMENT

In negative flexural region of any continuous span, the total cross-sectional area of the longitudinal reinforcement shall not be less than 1 percent of the total cross-sectional area of the slab.

The reinforcement used to satisfy this requirement shall have a specified minimum yield strength not less than 420 MPa {60 ksi} and a size not exceeding No. 19 bars { No. 6 }. The required reinforcement shall be placed in two layers uniformly distributed across the slab width, and two-thirds shall be placed in the top layer. The individual bars shall be spaced at intervals not exceeding 150 mm {6

COMMENTARY

CE6.10.3.7

The use of 1 percent reinforcement with a size not exceeding No. 19 bars is intended to provide rebar spacing that will be small enough to control slab cracking. Reinforcement with a yield strength of at least 420 MPa {60 ksi} is expected to remain elastic, even if inelastic redistribution of negative moments occurs. Thus, elastic recovery is expected to occur after the live load is removed, and this should tend to close the slab cracks. Pertinent criteria for concrete crack control are discussed in more detail in AASHTO (1991) and in Haaijer et al. (1987).

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in.} within each row.

Shear connectors shall be provided along the entire length of the girder to develop stresses on the plane joining the concrete and steel in accordance with E6.10.7.4.

E6.10.4P Strength Limit State Flexural Resistance

The factored flexural resistance, in terms of moment and stress, shall be taken as:

$$M_r = \phi_f M_n \quad \text{(E6.10.4P-1)}$$

and

$$F_r = \phi_f F_n \quad \text{(E6.10.4P-2)}$$

where:

φ_f = resistance factor for flexure specified in A6.5.4.2 and D6.5.4.2

M_n = nominal flexural resistance (N-mm) {k-in}

F_n = nominal flexural resistance of each flange (MPa) {ksi}

COMMENTARY

CE6.10.4P

E6.10.4P is written in the form of a flow chart, shown schematically in Figure C1, to facilitate the investigation of the flexural resistance of a particular I-section. Figure C2 shows the expanded flow chart when the optional Q formula of E6.10.4.2.3P is considered.

For compact sections, the calculated moments in simple and continuous spans are compared with the plastic moment capacities of the sections, even though the moments may have been based upon an elastic analysis. Nevertheless, unless an inelastic structural analysis is made, it is customary to call the process an "elastic" one.

The AASHTO Standard Specifications recognize inelastic behavior by:

- Utilizing the plastic moment capacity of compact sections, and
- Permitting an arbitrary 10 percent redistribution of peak negative moments at both overload and maximum load.

The Guide Specifications for Alternate Load Factor Design (ALFD) permit inelastic calculations for compact sections (AASHTO 1991). Most of the provisions of those Guide Specifications are incorporated into E6.10.10P of these Specifications.

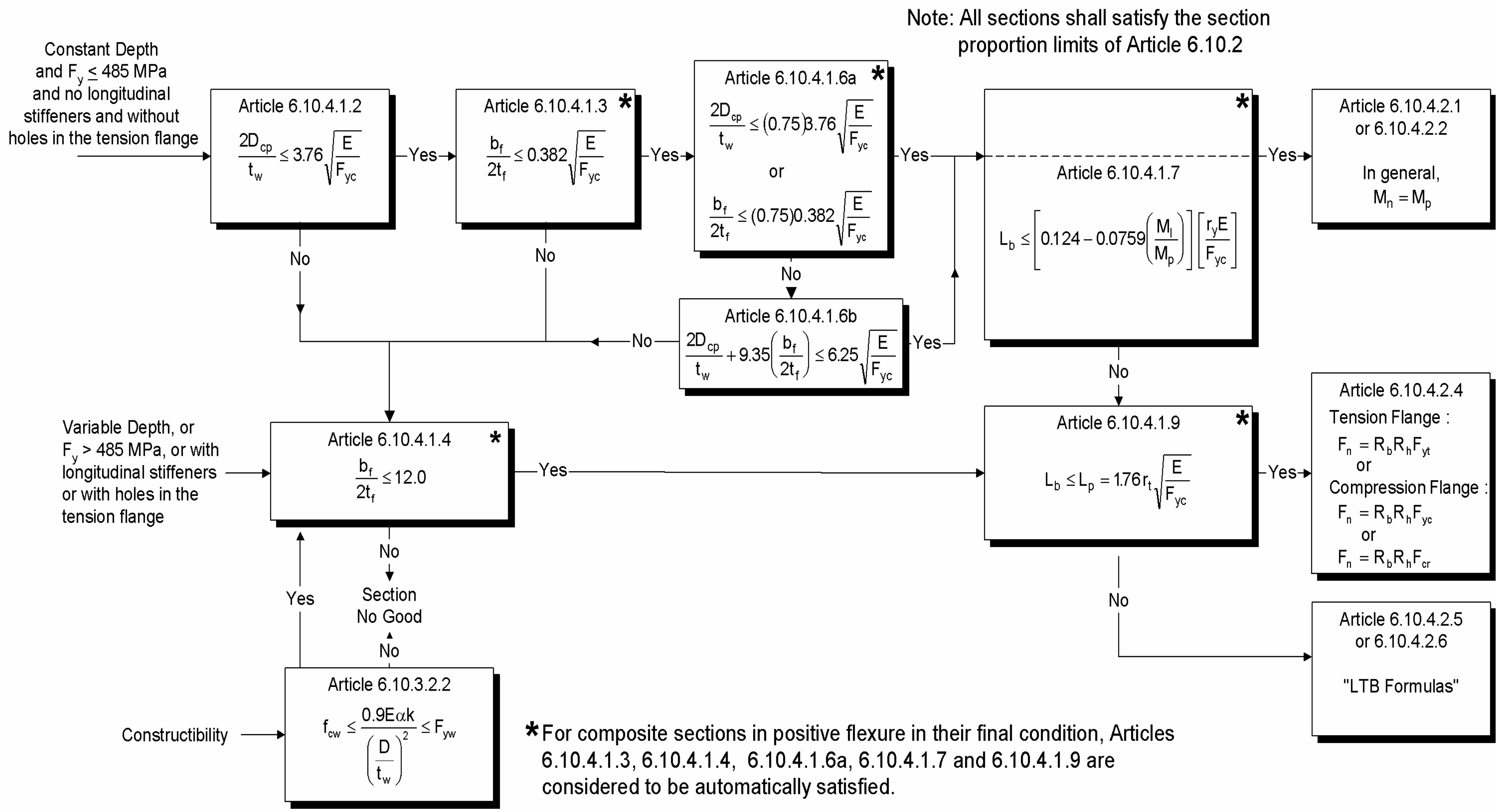

Figure CE6.10.4P-1 - I-Sections in Flexure Flow Chart (see also CE6.10.3.2.2)

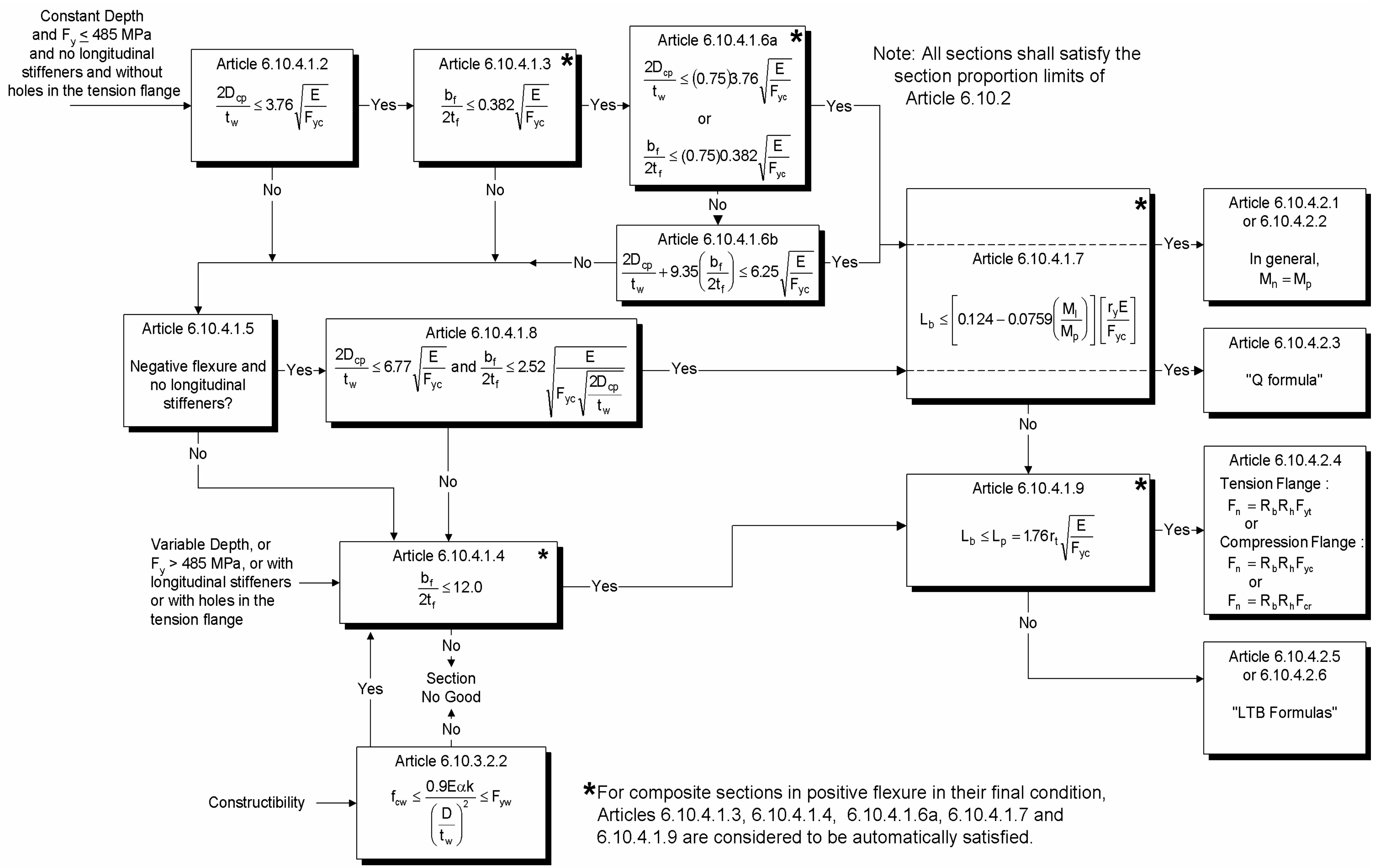

Figure CE6.10.4P-2 - I-Sections in Flexure Flow Chart When the Optional Q Formula is Considered (see also CE6.10.3.2.2P)

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COMMENTARY

E6.10.4.1P CATEGORIZATION OF FLEXURAL RESISTANCE

E6.10.4.1.1P Specified Minimum Yield Strength

The investigation of flexural resistance of I-sections satisfying the section-proportion limits of E6.10.2P shall proceed as follows:

- Where the specified minimum yield strength does not exceed 345 MPa {50 ksi}, and the girder has a constant depth, and does not have either longitudinal stiffeners or holes in the tension flange, then begin with the compact-section web slenderness provisions of E6.10.4.1.2P, or

- Where the specified minimum yield strength does exceed 345 MPa {50 ksi}, or the girder has a variable depth, longitudinal web stiffeners, or holes in the tension flange, then

    - For composite sections in positive flexure, the flexural resistance of each flange shall be as defined by the noncompact section flange flexural resistance provisions of E6.10.4.2.4P, or

    - For all other sections, the investigation shall proceed with the noncompact section compression flange slenderness provisions of E6.10.4.1.4P.

The effective section of flexural members with holes in the tension flange shall be determined as specified in E6.10.3.6.

CE6.10.4.1.1P

Two different entry points for the flow charts are required to characterize the flexural resistance at the strength limit state, in part because the moment-rotation behavior of steels having yield strengths exceeding 345 MPa {50 ksi} has not been sufficiently documented to extend plastic moment capacity to those materials. Similar logic applies to flexural members of variable depth section and with longitudinal stiffeners. At sections of flexural members with holes in the tension flange, it has also not been fully documented that complete plastification of the cross-section can be achieved prior to fracture on the net section of the flange.

In general, compression flange slenderness and bracing requirements need not be investigated and can be considered automatically satisfied at the strength limit state for both compact and noncompact composite sections in positive flexure because the hardened concrete slab prevents local and lateral compression flange buckling. However, when precast decks are used with shear connectors clustered in block-outs spaced several feet apart, consideration should be given to checking the compression flange slenderness requirement at the strength limit state and computing the nominal flexural resistance of the flange according to Equation E6.10.4.2.4aP-2.

E6.10.4.1.2P Compact-Section Web Slenderness

If:

$$\frac{2D_{cp}}{t_w} \le 3.76\sqrt{\frac{E}{F_{vc}}} \qquad \text{(E6.10.4.1.2P-1)}$$

where:

D_{cp} = depth of the web in compression at the plastic moment (mm) {in.}

F_{yc} = specified minimum yield strength of the compression flange (MPa) {ksi}

CE6.10.4.1.2P

The web slenderness requirement of this article is adopted from AISC (1993) and gives approximately the same allowable web slenderness as specified for compact sections in AASHTO (1996). Most composite sections in positive flexure will qualify as compact according to this criterion because the concrete deck causes an upward shift in the neutral axis, which greatly reduces the depth of the web in compression.

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the web is deemed compact, and

- For composite sections in positive flexure, the flexural resistance shall be as defined by the composite compact-section positive flexural resistance provisions of E6.10.4.2.2P, or

- For all other sections, the investigation shall proceed with the compact-section compression-flange slenderness provisions of E6.10.4.1.3P.

Otherwise, the web does not qualify as compact, and:

If the optional Q formula is not to be considered:

- For composite sections in positive flexure, the flexural resistance of each flange shall be as defined by the noncompact section flange flexural resistance provisions of E6.10.4.2.4P, or

- For all other sections, the investigation shall proceed with the noncompact section compression-flange slenderness provisions of E6.10.4.1.4P, or

If the optional Q formula is to be considered, the investigation shall proceed with the Q formula qualification provisions of E6.10.4.1.5P.

E6.10.4.1.3P Compact-Section Compression-Flange Slenderness

If:

$$\frac{b_f}{2t_f} \leq 0.382\sqrt{\frac{E}{F_{yc}}} \qquad \text{(E6.10.4.1.3P-1)}$$

where:

b_f = width of the compression flange (mm) {in.}

t_f = compression-flange thickness (mm) {in.}

the investigation shall proceed with the compact-section web and compression-flange slenderness interaction provisions of E6.10.4.1.6.

Otherwise, the compression flange does not qualify as compact, and:

If the optional Q formula is not to be considered the investigation shall proceed with the noncompact section compression-flange slenderness provisions of E6.10.4.1.4P, or

If the optional Q formula is to be considered, investigation

COMMENTARY

CE6.10.4.1.3P

The compression-flange requirement for compact negative flexural sections is retained from AASHTO (1996).

SPECIFICATIONS

shall proceed with the optional Q formula qualification provisions of E6.10.4.1.5P.

E6.10.4.1.4P Noncompact Section Compression-Flange Slenderness

If:

$$\frac{b_f}{2t_f} \leq 12.0 \qquad \text{(E6.10.4.1.4P-1)}$$

where:

b_f = width of the compression flange (mm) {in.}

t_f = thickness of compression flange (mm) {in.}

the investigation shall proceed with the noncompact section compression-flange bracing provisions of E6.10.4.1.9.

Otherwise the section is not acceptable, and a new section shall be chosen.

The effective section of flexural members with holes in the tension flange shall be determined as specified in E6.10.3.6P.

E6.10.4.1.5P Optional Q Formula Qualification

Where the section under investigation is in negative flexure, the specified minimum yield strength does not exceed 345 MPa {50 ksi}, the girder has a constant depth, no longitudinal web stiffeners, and no holes in the tension flange, then the investigation may proceed with the optional Q formula web and compression flange slenderness provisions of E6.10.4.1.8P.

Otherwise, the investigation shall proceed with the noncompact section compression-flange slenderness provisions of E6.10.4.1.4P.

COMMENTARY

CE6.10.4.1.4P

The slenderness is limited to a practical upper limit of 12.0 in Equation 1 to ensure the flange will not distort excessively when welded to the web. The nominal flexural resistance of the compression flange for noncompact sections, other than for noncompact composite sections in positive flexure in their final condition, that satisfy the bracing requirement of E6.10.4.1.9P depends on the slenderness of the flange according to Equation E6.10.4.2.4aP-2. For sections without longitudinal web stiffeners, the nominal flexural resistance is also a function of the web slenderness. For compression-flange slenderness ratios at or near the limit given by Equation 1, the nominal flexural resistance will typically be below F_{yc} according to Equation E6.10.4.2.4aP-2. To utilize a nominal flexural resistance at or near F_{yc}, a lower compression-flange slenderness ratio will be required.

SPECIFICATIONS

E6.10.4.1.6P Compact-Section Web and Compression-Flange Slenderness Interaction

E6.10.4.1.6aP General

If:

$$\frac{2D_{cp}}{t_w} \le 0.75\,(3.76)\,\sqrt{\frac{E}{F_{yc}}} \qquad \text{(E6.10.4.1.6aP-1)}$$

or

$$\frac{b_f}{2t_f} \le 0.75\,(0.382)\,\sqrt{\frac{E}{F_{yc}}} \qquad \text{(E6.10.4.1.6aP-2)}$$

the investigation shall proceed with the compact-section compression-flange bracing provisions of E6.10.4.1.7P.

Otherwise, the investigation shall proceed with the compact-section web and compression-flange interaction equation of E6.10.4.1.6bP.

E6.10.4.1.6bP Interaction Equation

If:

$$\frac{2D_{cp}}{t_w} + 9.35\left(\frac{b_f}{2t_f}\right) \le 6.25\,\sqrt{\frac{E}{F_{yc}}} \qquad \text{(E6.10.4.1.6bP-1)}$$

the investigation shall proceed with the compression-flange bracing provisions of E6.10.4.1.7P.

Otherwise:

- If the optional Q formula is not to be considered, the investigation shall continue with the noncompact section compression-flange slenderness provisions of E6.10.4.1.4P, or

- If the optional Q formula is to be considered, the investigation may proceed with the optional Q formula qualification provisions of E6.10.4.1.5P.

COMMENTARY

CE6.10.4.1.6a

The slenderness interaction relationship for compact sections is retained from the Standard Specifications. A review of the moment-rotation test data available in the literature suggests that compact sections may not be able to reach the plastic moment when the web and compression-flange slenderness ratios both exceed 75 percent of the limits given in Equations E6.10.4.1.2P-1 and E6.10.4.1.3P-1, respectively. The slenderness interaction relationship given in Equation E6.10.4.1.6bP-1 redefines the allowable limits when this occurs (Grubb and Carskaddan 1980).

SPECIFICATIONS

E6.10.4.1.7P Compact-Section Compression-Flange Bracing

If:

$$L_b \leq \left[0.124 - 0.0759\left(\frac{M_l}{M_p}\right)\right]\left[\frac{r_y E}{F_{yc}}\right] \quad \text{(E6.10.4.1.7P-1)}$$

where:

L_b = the unbraced length (mm) {in.}

r_y = minimum radius of gyration of the steel section, with respect to the vertical axis (mm) {in.}

M_l = the lower moment due to the factored loading at either end of the unbraced length (N-mm) {k-in}

M_p = plastic moment (N-mm) {k-in}

F_{yc} = specified minimum yield strength of the compression flange (MPa) {ksi}

Where (M_l/M_p) shall be taken as negative if the portion of the member within the unbraced length is bent in reverse curvature.

and,

- The compact-section compression-flange slenderness provisions of E6.10.4.1.6aP, or E6.10.4.1.6bP have been satisfied, the compression flange is deemed compact, and the flexural resistance shall be defined by the general compact-section flexural resistance provisions of E6.10.4.2.1P, or

- The compact-section compression-flange slenderness provisions of E6.10.4.1.6aP, or E6.10.4.1.6bP have not been satisfied, the flexural resistance may be defined by the flexural resistance provisions based upon the optional Q formula of E6.10.4.2.3P.

Otherwise, the investigation shall proceed with the noncompact section compression-flange bracing provisions of E6.10.4.1.9P.

The value M_l in Equation 1 may be conservatively calculated using the envelope of factored moments rather than the concurrent values.

COMMENTARY

CE6.10.4.1.7P

This article provides a continuous function relating unbraced length and end moment ratio. There is a substantial increase in the allowable unbraced length if the member is bent in reverse curvature between brace points because yielding is confined to zones close to the brace points. The formula was developed to provide inelastic rotation capacities of at least three times the elastic rotation corresponding to the plastic moment (Yura et al. 1978).

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E6.10.4.1.8P Optional Q Formula Web and Compression-Flange Slenderness

If:

$$\frac{2D_{cp}}{t_w} \le 6.77 \sqrt{\frac{E}{F_{yc}}} \quad \text{(E6.10.4.1.8P-1)}$$

and

$$\frac{b_f}{2t_f} \le 2.52 \sqrt{\frac{E}{F_{yc}\sqrt{\frac{2D_{cp}}{t_w}}}} \quad \text{(E6.10.4.1.8P-2)}$$

the investigation may proceed with the compact-section compression-flange bracing provisions of E6.10.4.1.7P.

Otherwise, the investigation shall proceed with the noncompact section compression-flange slenderness provisions of E6.10.4.1.4P.

E6.10.4.1.9P Noncompact Section Compression-Flange Bracing

If:

$$L_b \le L_p = 1.76\, r_t \sqrt{\frac{E}{F_{yc}}} \quad \text{(E6.10.4.1.9P-1)}$$

where:

r_t = radius of gyration of a notional section comprised of the compression flange of the steel section plus one-third of the depth of the web in compression taken about the vertical axis (mm) {in.}

F_{yc} = specified minimum yield strength of the compression flange (MPa) {ksi}

the flexural resistance of each flange shall be defined by the noncompact section flange flexural resistance of E6.10.4.2.4P.

Otherwise:

- For composite sections in their final condition, the investigation shall proceed with the composite section lateral torsional buckling provisions of E6.10.4.2.5P, or
- For noncomposite sections or the constructibility of composite sections, the investigation shall proceed with

COMMENTARY

CE6.10.4.1.9P

This article defines the maximum unbraced length for which a section can reach the specified minimum yield strength times the applicable flange stress reduction factors, under a uniform moment, before the onset of lateral torsional buckling. Under a moment gradient, sections with larger unbraced lengths can still reach the yield strength. This larger allowable unbraced length may be determined by equating Equation E6.10.4.2.5aP-1 to $R_bR_hF_{yc}$ and solving for L_b resulting in the following equation:

$$L_b \le \left(1.33 - \frac{1.0}{C_b}\right)\left(\frac{r_t}{0.187}\right)\sqrt{\frac{E}{F_{yc}}} \quad \text{(CE6.10.4.1.9P-1)}$$

where:

C_b = moment gradient correction factor specified in E6.10.4.2.5aP

Where C_b is equal to 1.0, Equation C1 reduces to Equation 1..

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the noncomposite section lateral torsional buckling provisions of E6.10.4.2.6.

E6.10.4.2P DEFINITIONS OF FLEXURAL RESISTANCE

Any section anticipated to reach M_p shall be laterally braced.

E6.10.4.2.1P General Compact-Section Flexural Resistance

The nominal flexural resistance shall be taken as:

$$M_n = M_p \quad \text{(E6.10.4.2.1P-1)}$$

where:

M_n = plastic moment as specified in E6.10.3.1.3P (N-mm) {k-in}

E6.10.4.2.2P Composite Compact-Section Positive Flexural Resistance

E6.10.4.2.2aP General

If the section under investigation is in:

- A simple span, or
- A continuous span with compact sections in the negative flexural region over the interior supports,

the nominal flexural resistance of the composite compact section in the positive flexural region shall be taken as:

- If $D_p \leq D'$, then:

$$M_n = M_p \quad \text{(E6.10.4.2.2aP-1)}$$

- If $D' < D_p \leq 5D'$, then:

$$M_n = \frac{5\,M_p - 0.85\,M_y}{4} + \frac{0.85\,M_y - M_p}{4}\left(\frac{D_p}{D'}\right) \quad \text{(E6.10.4.2.2aP-2)}$$

where:

D_p = distance from the top of the slab to the neutral axis

COMMENTARY

CE6.10.4.2.1P

If the limiting values of Articles E6.10.4.1.2P, E6.10.4.1.3P, E6.10.4.1.6P, and E6.10.4.1.7P are satisfied, flexural resistance at the strength limit state is defined as the plastic moment for compact sections.

Staggered diaphragms induce lateral flange bending which shall be accounted for by reducing the flange width when calculating M_p.

CE6.10.4.2.2aP

For simple spans and continuous spans with compact interior support sections, the equation defining the nominal flexural resistance depends on the ratio of D_p, which is the distance from the top of the slab to the neutral axis at the plastic moment to a defined depth D'. D' is specified in E6.10.4.2.2bP and is defined as the depth at which the composite section reaches its theoretical plastic moment capacity, M_p, when the maximum strain in the concrete slab is at its theoretical crushing strain. Sections with a ratio of D_p to D' less than or equal to 1.0 can reach as a minimum M_p of the composite section. Equation 1 limits the nominal flexural resistance to M_p. Sections with a ratio of D_p to D' equal to 5.0 have a specified nominal flexural resistance of 0.85 M_y. For ratios in between 1.0 and 5.0, the linear transition Equation 2 is given to define the nominal flexural resistance. Equations 1 and 2 were derived as a result of a parametric analytical study of more than 400 composite steel sections, including unsymmetrical as well as symmetrical steel sections, as discussed in Wittry (1993). The analyses included the effect of various steel and concrete stress-strain relationships, residual stresses in the steel, and concrete crushing strains. From the analyzes, the ratio of D_p to D' was found to be the controlling variable defining the nominal flexural resistance and ductility of the composite sections. As the ratio of D_p/D' approached a value of 5.0, the analyses indicated that crushing of the slab

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at the plastic moment (mm) {in.}

D' = distance specified in E6.10.4.2.2b (mm) {in.}

M_y = moment capacity at first yield of the short-term composite positive moment section (N-mm) {k-in}

Otherwise, the nominal flexural resistance may be determined by either of the following methods but shall not be taken to be greater than the applicable value of M_n computed from either Equation 1 or Equation 2.

- Approximate Method

$$M_n = 1.3\, R_h M_y \qquad \text{(E6.10.4.2.2aP-3)}$$

- Refined Method

$$M_n = R_h\, M_y + A\, |M_{np} - M_{cp}| \qquad \text{(E6.10.4.2.2aP-4)}$$

where:

R_h = hybrid factor specified in E6.10.4.3.1P

A = for end spans, the distance from the end support to the location of the cross-section in the span divided by the span length; for interior spans, 1.0

M_{cp} = moment due to the factored loadings at the interior support concurrent with the maximum positive flexural moment at the cross-section under consideration (N-mm) {k-in}

M_{np} = nominal flexural resistance at an interior support (N-mm) {k-in}

for which $|M_{np}-M_{cp}|$ for interior spans shall be taken as the smaller value at either end of the span.

Where the refined method is used, the concurrent positive moment shall not exceed R_hM_y for the factored loading that produces the maximum negative moment at the adjacent support.

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would theoretically occur upon the attainment of first yield in the cross-section. Thus, the reduction factor of 0.85 is included in front of M_y in Equation 2 because the strength and ductility of the composite section are controlled by crushing of the concrete slab at higher ratios of D_p/D'. For the section to qualify as compact with adequate ductility at the computed nominal flexural resistance, the ratio of D_p to D' cannot exceed 5.0, as specified. Also, the value of the yield moment M_y to be used in Equation 2 may be computed as the specified minimum yield strength of the beam or girder F_y times the section modulus of the short-term composite section with respect to the tension flange, rather than using the procedure specified in E6.10.3.1.2P.

The inherent conservatism of Equation 2 is a result of the desire to ensure adequate ductility of the composite section. However, in many cases, permanent deflection service limit state criteria will govern the design of compact composite sections. Thus, it is prudent to initially design these sections to satisfy the permanent deflection service limit state and then check the nominal flexural resistance of the section at the strength limit state.

The shape factor (M_p/M_y) for composite sections in positive flexure can be as high as 1.5. Therefore, a considerable amount of yielding is required to reach M_p, and this yielding reduces the effective stiffness of the positive flexural section. In continuous spans, the reduction in stiffness can shift moment from positive flexural regions to negative flexural regions. Therefore, the actual moments in negative flexural regions may be higher than those predicted by an elastic analysis. Negative flexural sections would have to have the capacity to sustain these higher moments, unless some limits are placed on the extent of the yielding of the positive moment section. This latter approach is used in the Specification for continuous spans with noncompact interior-support sections.

The live loading patterns causing the maximum elastic moments in negative flexural sections are different than those causing maximum moments in positive flexural sections. When the loading pattern causing maximum positive flexural moments is applied, the concurrent negative flexural moments are usually below the flexural resistance of the sections in those regions. Therefore, the specifications conservatively allow additional moment above M_y to be applied to positive flexural sections of continuous spans with noncompact interior support sections, not to exceed the nominal flexural resistance given by Equations 1 or 2 to ensure adequate ductility of the composite section. Compact interior support sections have sufficient capacity to sustain the higher moments caused by the reduction in stiffness of the positive flexural region. Thus, the nominal flexural resistance of positive flexural sections in members with compact interior support sections is not limited due to the effect of this moment shifting.

Note that Equation 4 requires the use of the absolute value of the term (M_{np}-M_{cp}).

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COMMENTARY

PennDOT does not allow inelastic analysis.

For composite sections in positive flexure, the hardened concrete deck prevents local and lateral buckling of the compression flange.

E6.10.4.2.2bP Ductility Requirement

The concrete deck shall be protected from premature crushing and spalling when the composite section approaches the plastic moment. In lieu of more exact procedures, the provision herein may be used to satisfy this requirement.

For compact composite cross-sections in positive flexure, if the moment due to the factored loads results in a flange stress that exceeds the yield strength for either flange, times the hybrid factor, R_h, the section shall satisfy:

$$\left(\frac{D_p}{D'}\right) \leq 5 \qquad \text{(E6.10.4.2.2bP-1)}$$

in which:

$$D' = \beta \frac{\left(d + t_s + t_h\right)}{7.5} \qquad \text{(E6.10.4.2.2bP-2)}$$

where:

β = 0.9 for $F_y \leq$ 250 MPa {$F_y \leq$ 36 ksi}

β = 0.7 for 250 MPa < $F_y \leq$ 345 MPa {36 ksi < $F_y \leq$ 50 ksi}

D_p = distance from the top of the slab of a composite section to the neutral axis at the plastic moment (mm) {in.}

d = depth of the steel section (mm) {in.}

t_h = thickness of the concrete haunch above the top flange (mm) {in.}

t_s = thickness of the concrete slab (mm) {in.}

CE6.10.4.2.2bP

The ductility requirement specified in this article is equivalent to the requirement given in AASHTO (1995).

The ratio of D_p to D' is limited to a value of 5.0 to ensure that the tension flange of the steel section reaches strain hardening prior to crushing of the concrete slab. D' is defined as the depth at which the composite section reaches its theoretical plastic moment capacity M_p when the maximum strain in the concrete slab is at its theoretical crushing strain. The term $(d+t_s+t_h)/7.5$ in the definition of D', hereafter referred to as D^*, was derived by assuming that the concrete slab is at the theoretical crushing strain of 0.3 percent and that the tension flange is at the assumed strain-hardening strain of 1.2 percent. The compression depth of the composite section, D_p, was divided by a factor of 1.5 to ensure that the actual neutral axis of the composite section at the plastic moment is always above the neutral axis computed using the assumed strain values (Ansourian 1982). From the results of a parametric analytical study of 400 different composite steel sections, including unsymmetrical as well as symmetrical steel sections, as discussed in Wittry (1993), it was determined that sections utilizing 250 MPa {36 ksi}steel reached M_p at a ratio of D_p/D^* equal to approximately 0.9, and sections utilizing 345 MPa {50 ksi} steel reached M_p at a ratio of D_p/D^* equal to approximately 0.7. Thus, 0.9 and 0.7 are specified as the values to use for the factor, which is multiplied by D^* to compute D' for 250 MPa {36 ksi} and 345 MPa {50 ksi} yield strength steels. Equation 1 need not be checked at sections where the stress in either flange due to the factored loadings does not exceed R_h F_{yf} because there will be insufficient strain in the steel section at or below the yield strength for a potential concrete crushing failure of the deck to occur.

Sections that do not meet the ductility requirement of E6.10.4.2.2bP may be treated as non-compact.

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E6.10.4.2.3P Flexural Resistance Based Upon the Optional Q Formula

The flexural resistance, M_n, may be taken as the lesser of:

- $M_n = M_p$, or (E6.10.4.2.3P-1)

- $$M_n = \left[1 - \left(1 - \frac{0.7}{\left(\frac{M_p}{M_y}\right)}\right)\left(\frac{Q_p - Q_{fl}}{Q_p - 0.7}\right)\right] M_p \qquad \text{(E6.10.4.2.3P-2)}$$

where:

- For unsymmetrical sections,

$$Q_p = 5.47 \left(\frac{M_p}{M_y}\right) - 3.13 \text{ , or} \qquad \text{(E6.10.4.2.3P-3)}$$

- For symmetrical sections,

$$Q_p = 3.0 \qquad \text{(E6.10.4.2.3P-4)}$$

and,

If: $\frac{b_f}{2t_f} \leq 0.382\sqrt{\frac{E}{F_{yc}}}$ (E6.10.4.2.3P-5)

$$Q_{fl} = \frac{30.5}{\sqrt{\frac{2D_{cp}}{t_w}}} \qquad \text{(E6.10.4.2.3P-6)}$$

otherwise,

$$Q_{fl} = \frac{4.45}{\left(\frac{b_f}{2t_f}\right)^2 \sqrt{\frac{2D_{cp}}{t_w}}} \frac{E}{F_{yc}} \qquad \text{(E6.10.4.2.3P-7)}$$

where:

M_p = plastic moment (N-mm) {k-in}

F_{yc} = specified minimum yield strength of the compression flange (MPa) {ksi}

M_y = yield moment specified in E6.10.3.1.2P (N·mm)

COMMENTARY

CE6.10.4.2.3P

Equation 2 defines a transition in the nominal flexural resistance from M_p to approximately 0.7 M_y.

The nominal flexural resistance given by Equation 2 is based on the inelastic buckling strength of the compression flange and results from a fit to available experimental data. The equation considers the interaction of the web and compression-flange slenderness in the determination of the resistance of the section by using a flange buckling coefficient, $k_f = 4.92/(2D_{cp}/t_w)^{1/2}$, in computing the Q_{fl} parameter in Equation 7. Q_{fl} is the ratio of the buckling capacity of the flange to the yield strength of the flange. The buckling coefficient given above was based on the test results reported in Johnson (1985) and data from other available composite and noncomposite steel beam tests. A similar buckling coefficient is given in Section B5.3 of AISC (1993). Equation 6 is specified to compute Q_{fl} if the compression-flange slenderness is less than the value specified in E6.10.4.1.3P to effectively limit the increase in the bending resistance at a given web slenderness with a reduction in the compression-flange slenderness below this value. Equation 6 is obtained by substituting the compression-flange slenderness limit from E6.10.4.1.3P in Equation 7.

Equation 2 represents a linear fit of the experimental data between a flexural resistance of M_p and 0.7 M_y. The Q_p parameter, defined as the web and compression-flange slenderness to reach a flexural resistance of M_p, was derived to ensure the equation yields a linear fit to the experimental data. Equation 2 was derived to determine the maximum flexural resistance and does not necessarily ensure a desired inelastic rotation capacity. Sections in negative flexure that are required to sustain plastic rotations may be designed according to the procedures specified in E6.10.10. If elastic procedures are used and Equation 2 is not used to determine the nominal flexural resistance, the resistance shall be determined according to the procedures specified in E6.10.4.2.4P.

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E6.10.4.2.4P Noncompact Section Flange Flexural Resistance

E6.10.4.2.4aP Compression Flanges

The nominal flexural resistance of the compression flange, in terms of stress, shall be determined as:

- For composite sections in positive flexure in their final condition:

$$F_n = R_b R_h F_{yc} \tag{E6.10.4.2.4aP-1}$$

For all other sections in their final condition and for constructibility:

$$F_n = R_b R_h F_{cr} \tag{E6.10.4.2.4aP-2}$$

where:

$$F_{cr} = \frac{1.904\,E}{\left(\frac{b_f}{2t_f}\right)^2 \sqrt{\frac{2D_c}{t_w}}} \leq F_{yc}$$

without longitudinal web stiffeners

$$F_{cr} = \frac{0.166\,E}{\left(\frac{b_f}{2t_f}\right)^2} \leq F_{yc}$$

with longitudinal web stiffeners

where:

b_f = width of the compression flange (mm) {in.}

D_c = depth of the web in compression in the elastic range (mm) {in.}

R_h = hybrid factor specified in E6.10.4.3.1P

R_b = load-shedding factor specified in E6.10.4.3.2P

F_{yc} = specified minimum yield strength of the compression flange (MPa) {ksi}

COMMENTARY

CE6.10.4.2.4aP

For composite noncompact sections in positive flexure in their final condition, the nominal flexural resistance of the compression flange at the strength limit state is equal to the yield stress of the flange, F_{yc}, reduced by the specified reduction factors. For all other noncompact sections in their final condition and for constructibility, where the limiting value of E6.10.4.1.9P is satisfied, the nominal flexural resistance of the compression flange is equal to F_{cr} times the specified reduction factors. F_{cr} represents a critical compression-flange local buckling stress, which cannot exceed F_{yc}. For sections without longitudinal web stiffeners, F_{cr} depends on the actual compression flange and web slenderness ratios. This equation for F_{cr} was not developed for application to sections with longitudinal web stiffeners. For those sections, the expression for F_{cr} was derived from the compression-flange slenderness limit for braced noncompact sections specified in the Load Factor Design portion of the AASHTO Standard Specifications (2002). By expressing the nominal flexural resistance of the compression flange as a function of F_{cr}, larger compression-flange slenderness ratios may be used at more lightly loaded sections for a given web slenderness. To achieve a value of F_{cr} at or near F_{yc} at more critical sections, a lower compression-flange slenderness ratio will be required.

The nominal flexural resistance of the compression-flange is also modified by the hybrid factor R_h and the load-shedding factor R_b. R_h accounts for the increase in flange stress resulting from web yielding in hybrid girders and is computed according to the provisions of E6.10.4.3.1P. R_h should be taken as 1.0 for constructibility checks because web yielding is limited. R_b accounts for the increase in compression-flange stress resulting from local web bend buckling and is computed according to the provisions of E6.10.4.3.2P. R_b is computed based on the actual stress f_c in the compression flange due to the factored loading under investigation, which should not exceed F_{yc}.

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E6.10.4.2.4bP Tension Flanges

The nominal flexural resistance of the tension flange, in terms of stress, shall be determined as:

$$F_n = R_b R_h F_{yt} \qquad \text{(E6.10.4.2.4bP-1)}$$

where:

F_{yt} = specified minimum yield strength of the tension flange (MPa) {ksi}

E6.10.4.2.5P Composite-Section Flange Flexural Resistance Based Upon Lateral Torsional Buckling

E6.10.4.2.5aP Compression Flanges

The nominal flexural resistance of the compression flange, in terms of stress, shall be determined from Equation E6.10.4.2.4aP-2, but not to exceed the nominal flexural resistance based upon lateral-torsional buckling determined as:

If: $L_b \le L_r = 4.44\, r_t \sqrt{\dfrac{E}{F_{yc}}}$,

$$F_n = C_b R_b R_h F_{yc}\left[1.33 - 0.187\left(\frac{L_b}{r_t}\right)\sqrt{\frac{F_{yc}}{E}}\right] \le R_b R_h F_{yc} \qquad \text{(E6.10.4.2.5aP-1)}$$

Otherwise:

$$F_n = C_b R_b R_h \left[\frac{9.86E}{\left(\dfrac{L_b}{r_t}\right)^2}\right] \le R_b R_h F_{yc} \qquad \text{(E6.10.4.2.5aP-2)}$$

for which:

- For unbraced cantilevers or for members where the moment within a significant portion of the unbraced segment exceeds the larger of the segment end moments

$$C_b = 1.0 \text{, or} \qquad \text{(E6.10.4.2.5aP-3)}$$

- For all other cases,

$$C_b = 1.75 - 1.05\left(\frac{P_l}{P_h}\right) + 0.3\left(\frac{P_l}{P_h}\right)^2 \le K_b \qquad \text{(E6.10.4.2.5aP-4)}$$

COMMENTARY

CE6.10.4.2.5a

The provisions for lateral-torsional buckling in this article differ from those specified in E6.10.4.2.6P because they attempt to handle the complex general problem of lateral-torsional buckling of a constant or variable depth section with stepped flanges constrained against lateral displacement at the top flange by the composite concrete slab. The equations provided in this article are based on the assumption that only the flexural stiffness of the compression flange will prevent the lateral displacement of that element between brace points, which ignores the effect of the restraint offered by the concrete slab (Basler and Thurlimann 1961). As such, the behavior of a compression flange in resisting lateral buckling between brace points is assumed to be analogous to that of a column. These simplified equations, developed based on this assumption, are felt to yield conservative results for composite sections under the various conditions listed above.

The effect of the variation in the compressive force along the length between brace points is accounted for by using the factor C_b. If the cross-section is constant between brace points, M_l/M_h is expressed in terms of P_l/P_h and may be used in calculating C_b. The ratio is taken as positive when the moments cause single curvature within the unbraced length.

C_b has a minimum value of 1.0 when the flange compressive force and corresponding moment are constant over the unbraced length. As the compressive force at one of the brace points is progressively reduced, C_b becomes larger and is taken as 1.75 when this force is 0.0. For the case of single curvature, it is conservative and convenient to use the maximum moments from the moment envelope at both brace points in computing the ratio of M_l/M_h or P_l/P_h, although the actual behavior depends on the concurrent moments at these points.

If the force at the end is then progressively increased in

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where:

C_b = moment gradient correction factor

P_l = force in the compression flange at the brace point with the lower force due to the factored loading (N) {kip}

P_h = force in the compression flange at the brace point with the higher force due to the factored loading (N) {kip}

L_b = the unbraced length (mm) {in.}

r_t = radius of gyration of a notional section comprised of the compression flange of the steel section plus one-third of the depth of the web in compression taken about the vertical axis (mm) {in.}

F_{yc} = specified minimum yield strength of the compression flange (MPa) {ksi}

R_h = hybrid factor specified in E6.10.4.3.1P

R_b = load-shedding factor specified in E6.10.4.3.2P

K_b = 1.75 where maximum moments from the moment envelope are used at both brace points in computing the ratio P_l/P_h

K_b = 2.3 where the concurrent moment is used at the brace point with the lower compression-flange force in computing the ratio P_l/P_h

(P_l/P_h) shall be taken as negative if P_l is a tensile force.

COMMENTARY

tension, which results in reverse curvature, the ratio is taken as negative and C_b continues to increase. However, in this case, using the concurrent moments at the brace points, which are not normally tracked in the analysis, to compute the ratio in Equation 4 gives the lowest value of C_b. Therefore, C_b is conservatively limited to a maximum value of 1.75 if the moment envelope values at both brace points are used to compute the ratio in Equation 4. If the concurrent moment at the brace point with the lower compression-flange force is available from the analysis and is used to compute the ratio, C_b is allowed to exceed 1.75 up to a maximum value of 2.3.

An alternative formulation for C_b is given by the following formula (AISC 1993):

$$C_b = \frac{12.5\,P_{\max}}{2.5\,P_{\max} + 3P_A + 4P_B + 3P_C} \qquad \text{(CE6.10.4.2.5aP-1)}$$

where:

P_{max} = absolute value of the maximum compression-flange force in the unbraced segment (N) {kip}

P_A = absolute value of the compression-flange force at the quarter point of the unbraced segment (N) {kip}

P_B = absolute value of the compression-flange force at the centerline of the unbraced segment (N) {kip}

P_c = absolute value of the compression-flange force at the three-quarter point of the unbraced segment (N) {kip}

This formulation gives improved results for the cases of nonlinear moment gradients and moment reversal.

The effect of a variation in the lateral stiffness properties, r_t, between brace points can be conservatively accounted for by using the minimum value that occurs anywhere between the brace points. Alternatively, a weighted average r_t could be used to provide a reasonable but somewhat less conservative answer.

The use of the moment envelope values at both brace points will be conservative for both single and reverse curvature when this formulation is used.

Other formulations for C_b to handle nontypical cases of compression flange bracing may be found in Galambos (1998).

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E6.10.4.2.6P Noncomposite Section Flexural Resistance Based Upon Lateral Torsional Buckling

E6.10.4.2.6aP Compression Flanges

The nominal flexural resistance of the compression flange shall be determined from Equation E6.10.4.2.4aP-2, but not to exceed the nominal flexural resistance based upon lateral-torsional buckling determined as follows, divided by S_{xc}:

If:

- A longitudinal stiffener is provided, or
- $\frac{2D_c}{t_w} \le \lambda_b \sqrt{\frac{E}{F_{yc}}}$,

$$M_n = 3.14E\, C_b\, R_h \left(\frac{I_{yc}}{L_b}\right)\sqrt{0.772\left(\frac{J}{I_{yc}}\right) + 9.87\left(\frac{d}{L_b}\right)^2} \le R_h M_y \quad \text{(E6.10.4.2.6aP-1)}$$

Otherwise:

If: $L_b \le L_r = 4.44 \sqrt{\frac{I_{yc}\, d}{S_{xc}} \frac{E}{F_{yc}}}$,

$$M_n = C_b\, R_b\, R_h\, M_y \left[1 - 0.5\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le R_b R_h M_y \quad \text{(E6.10.4.2.6aP-2)}$$

Otherwise:

$$M_n = C_b\, R_b\, R_h \frac{M_y}{2}\left(\frac{L_r}{L_b}\right)^2 \le R_b\, R_h\, M_y \quad \text{(E6.10.4.2.6aP-3)}$$

for which:

$$J = \frac{Dt_w{}^3 + b_f t_f{}^3 + b_t t_t^3}{3} \quad \text{(E6.10.4.2.6aP-4)}$$

$$L_p = 1.76\, r_t \sqrt{\frac{E}{F_{yc}}} \quad \text{(E6.10.4.2.6aP-5)}$$

where:

λ_b = 5.76 for sections where D_c is less than or equal to D/2

λ_b = 4.64 for sections where D_c is greater than D/2

COMMENTARY

CE6.10.4.2.6aP

Much of the discussion of the lateral buckling formulas in CE6.10.4.2.5a also applies to this article. The formulas of this article are simplifications of the formulas presented in AISC (1993) and Kitipornchai and Trahair (1980) for the lateral buckling capacity of unsymmetrical girders.

The formulas predict the lateral buckling moment within approximately 10 percent of the more complex Trahair equations for sections satisfying the proportions specified in E6.10.2.1. The formulas treat girders with slender webs differently than girders with stocky webs. For sections with stocky webs with a web slenderness less than or equal to $\lambda_b(E/F_{yc})^{1/2}$, or with longitudinally stiffened webs, bend-buckling of the web is theoretically prevented. For these sections, the St. Venant torsional stiffness and the warping torsional stiffness are included in computing the elastic lateral buckling moment given by Equation 1. For sections with thinner webs or without longitudinal stiffeners, cross-sectional distortion is possible; thus, the St. Venant torsional stiffness is ignored for these sections. Equation 3 is the elastic lateral torsional buckling moment given by Equation 1 with J taken as 0.0.

Equation 2 represents a straight line estimate of the inelastic lateral buckling resistance between R_b R_h M_y and 0.5 R_b R_h M_y. A straight line transition similar to this is not included for sections with stocky webs or longitudinally stiffened webs because the added complexity is not justified.

A discussion of the derivation of the value of λ_b may be found in CE6.10.4.3.2a.

The equation for J herein is a special case of Equation C4.6.2.1-1.

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C_b = moment gradient correction factor specified in E6.10.4.2.5aP

I_{yc} = moment of inertia of the compression flange of the steel section about the vertical axis in the plane of the web (mm^4) {$in.^4$}

S_{xc} = section modulus about the horizontal axis of the section to the compression flange (mm^3) {$in.^3$}

M_y = yield moment for the compression flange defined in E6.10.3.3.1P (N-mm) {k-in}

R_b = load-shedding factor specified in E6.10.4.3.2P

R_h = hybrid factor specified in E6.10.4.3.1P

r_t = minimum radius of gyration of the compression flange taken about the vertical axis (mm) {in.}

t_f = compression-flange thickness (mm) {in.}

F_{yc} = specified minimum yield strength of the compression flange (MPa) {ksi}

D_c = depth of the web in compression in the elastic range (mm) {in.}

d = depth of steel section (mm) {in.}

b_f = compression flange width (mm) {in.}

b_t = tension flange width (mm) {in.}

t_t = tension flange thickness (mm) {in.}

E6.10.4.3P FLANGE-STRESS REDUCTION FACTORS

E6.10.4.3.1P Hybrid Factor, R_h

E6.10.4.3.1aP General

For homogeneous sections, R_h shall be taken as 1.0.

For hybrid sections in which one flange reaches the yield strength under the factored loadings, either E6.10.4.3.1bP or E6.10.4.3.1cP, or both, shall apply, as applicable. The reduction factor should not be applied to compact sections because the effect of the lower-strength material in the web is accounted for in calculating the plastic moment as specified in E6.10.3.1.3P.

COMMENTARY

CE6.10.4.3.1aP

This factor accounts for the nonlinear variation of stresses caused by yielding of the lower strength steel in the web of a hybrid beam. The formulas defining this factor are the same as those given in AASHTO (1996) and are based on experimental and theoretical studies of composite and noncomposite beams and girders (ASCE 1968; Schilling 1968; and Schilling and Frost 1964). The factor applies to noncompact sections in both shored and unshored construction.

Because a vehicle heavier than the design vehicle may result in flange stresses that exceed the yield stress in the web, R_h is to be computed for all hybrid sections, regardless of the factored stress in the flanges.

E6.10.4.3.1bP Positive Flexure of Composite Sections

For composite hybrid sections, the hybrid factor shall be taken as:

$$R_h = 1 - \left[\frac{\beta\psi(1-\rho)^2(3-\psi+\rho\psi)}{6+\beta\psi(3-\psi)} \right] \quad \text{(E6.10.4.3.1bP-1)}$$

where:

ρ = F_{yw}/F_{yb}

β = A_w/A_{fb}

Ψ = d_n/d

d_n = distance from the outer fiber of the bottom flange to the neutral axis of the transformed short-term composite section (mm) {in.}

d = depth of the steel section (mm) {in.}

F_{yb} = specified minimum yield strength of the bottom flange (MPa) {ksi}

F_{yw} = specified minimum yield strength of the web (MPa) {ksi}

A_w = area of the web (mm^2) {in.^{2}}

A_{fb} = bottom flange area (mm^2) {in.^{2}}

E6.10.4.3.1cP Noncomposite Sections and Negative Flexure of Composite Sections

Where the neutral axis of composite hybrid sections, determined as specified in E6.10.3.1.4a, or the neutral axis of noncomposite hybrid sections, is located within 10 percent of the web depth from middepth of the web, the hybrid factor shall be taken as:

$$R_h = \frac{12 + \beta\left(3\rho - \rho^3\right)}{12 + 2\beta} \quad \text{(E6.10.4.3.1cP-1)}$$

where:

ρ = $F_{yw}/f_{fl} \le 1.0$

β = $2A_w/A_{tf}$

A_{tf} = for composite sections, total area of both steel flanges and the longitudinal reinforcement included in the section; for noncomposite sections, area of

CE6.10.4.3.1cP

Equation 1 approximates the reduction in the moment resistance due to yielding for a girder with the neutral axis located at middepth of the web. For girders with the neutral axis located within 10 percent of the depth from the middepth of the web, the change of the value of R_h from that given by Equation 1 is thought to be small enough to ignore. Equation 2 gives a more accurate procedure to determine the reduction in the moment resistance.

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both steel flanges (mm^2) {$in.^2$}

f_{fl} = the specified minimum yield strength in either flange (MPa) {ksi} {ksi}

For other composite or noncomposite hybrid sections, the hybrid factor shall be taken as:

$$R_h = \frac{M_{yr}}{M_y} \qquad \text{(E6.10.4.3.1cP-2)}$$

where:

M_{yr} = yield moment for which web yielding is accounted (N-mm) {k-in}

M_y = yield resistance in terms of moment, when web yielding is disregarded (N-mm) {k-in}

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The following approximate method illustrated in Figure C1 may be used in determining the yield moment resistance, M_{yr}, when web yielding is accounted for. The solid line connecting F_{yf} with f_r represents the distribution of stress at M_y if web yielding is neglected. For unshored construction, this distribution can be obtained by first applying the proper permanent load to the steel section, then applying the proper permanent load and live load to the composite section, and combining the two stress distributions. The dashed lines define a triangular stress block whose moment about the neutral axis is subtracted from M_y to account for the web yielding at a lower stress than the flange. M_y may be determined as specified in E6.10.3.1.2P. Thus,

$$M_{yr} \cong M_y - aP \qquad \text{(CE6.10.4.3.1cP-1)}$$

for which:

$$P \cong \frac{(F_{yf} - F_{yw})}{2} t_w h_w \qquad \text{(CE6.10.4.3.1cP-2)}$$

where:

F_{yf} = specified minimum yield strength of the bottom flange (MPa) {ksi}

F_{yw} = specified minimum yield strength of the web (MPa) {ksi}

t_w = web thickness (mm) {in.}

a = distance from the elastic neutral axis to the centroid of the stress block (mm) {in.}

f_r = stress in reinforcing steel at M_y (MPa) {ksi}

h_w = depth of yielded web (mm) {in.}

Figure 1 is specifically for the case where the elastic neutral axis is above middepth of the web and web yielding occurs only below the neutral axis. However, the same approach can be used if web yielding occurs both above and below the neutral axis or only above the neutral axis. The moment due to each triangular stress block due to web yielding must be subtracted from M_y.

This approach is approximate because web yielding causes a small shift in the location of the neutral axis. The effect of this shift on M_{yr} is almost always small enough to

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be neglected. The exact value of M_{yr} can be calculated from the stress distribution by accounting for yielding (Schilling 1968).

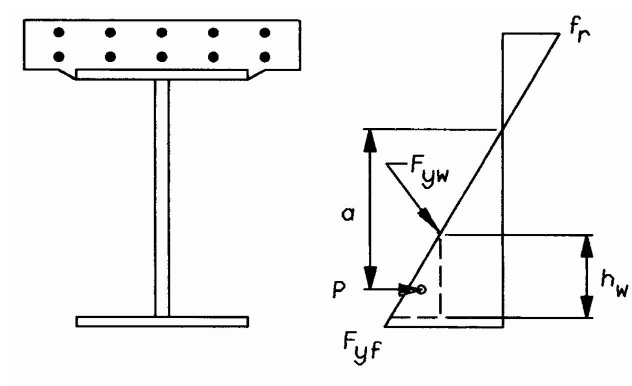

Figure CE6.10.4.3.1cP-1 - Stress Distribution in Hybrid Girder

E6.10.4.3.2P Load-Shedding Factor, R_b

E6.10.4.3.2aP Compression Flanges

If:

$$\frac{2D_c}{t_w} \le \lambda_b \sqrt{\frac{E}{f_c}} \qquad \text{(E6.10.4.3.2aP-1)}$$

Or if one or two longitudinal web stiffeners are provided and:

$$\frac{D}{t_w} \le 1.01 \sqrt{\frac{Ek}{f_c}} \qquad \text{(E6.10.4.3.2aP-2)}$$

Then, Rb shall be taken as 1.0.

Otherwise,

$$R_b = 1 - \left(\frac{a_r}{1200 + 300\, a_r}\right)\left(\frac{2D_c}{t_w} - \lambda_b \sqrt{\frac{E}{f_c}}\right) \qquad \text{(E6.10.4.3.2aP-3)}$$

for which:

If $\frac{d_s}{D_c} \ge 0.4$ then:

$$k = 5.17\left(\frac{D}{d_s}\right)^2 \ge 9.0\left(\frac{D}{D_c}\right)^2 \ge 7.2 \qquad \text{(E6.10.4.3.2aP-4)}$$

CE6.10.4.3.2aP

The R_b factor is a post buckling strength reduction factor that accounts for the nonlinear variation of stresses caused by local buckling of slender webs subjected to flexural stresses. The factor recognizes the reduction in the section resistance caused by the resulting shedding of the compressive stresses in the web to the compression-flange.

For webs without longitudinal stiffeners that satisfy Equation 1 with the compression-flange at a stress f_c, the R_b factor is taken equal to 1.0 since the web is below its theoretical elastic bend-buckling stress. The value of λ_b in Equation 1 reflects different assumptions of support provided to the web by the flanges. The value of 4.64 for sections where D_c is greater than D/2 is based on the theoretical elastic bend-buckling coefficient k of 23.9 for simply supported boundary conditions at the flanges. The value of 5.76 for members where D_c is less than or equal to D/2 is based on a value of k between the value for simply supported boundary conditions and the theoretical k value of 39.6 for fixed boundary conditions at the flanges (Timoshenko and Gere 1961).

For webs with one or two longitudinal stiffeners that satisfy Equation 2 with the compression-flange at a stress f_c, the R_b factor is again taken equal to 1.0 since the web is below its theoretical elastic bend-buckling stress. Two different theoretical elastic bend-buckling coefficients k are specified for webs with one or two longitudinal stiffeners. The value of k to be used depends on the location of the closest longitudinal web stiffener to the compression-flange

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If $\frac{d_s}{D_c} < 0.4$ then:

$$k = 11.64\left(\frac{D}{D_c - d_s}\right)^2 \geq 9.0\left(\frac{D}{D_c}\right)^2 \geq 7.2 \quad \text{(E6.10.4.3.2aP-5)}$$

$$a_r = \frac{2D_c t_w}{A_c} \quad \text{(E6.10.4.3.2aP-6)}$$

where:

λ_b = 5.76 for sections where D_c is less than or equal to D/2

λ_b = 4.64 for sections where D_c is greater than D/2

f_c = stress in the compression flange due to the factored loading under investigation (MPa) {ksi}

A_c = area of the compression flange (mm^2) {in.^{2}}

d_s = distance from the centerline of a plate longitudinal stiffener or the gage line of an angle longitudinal stiffener to the inner surface or leg of the compression-flange element (mm) {in.}

D_c = depth of the web in compression in the elastic range (mm) {in.}

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with respect to its optimum location (Frank and Helwig 1995).

Equations 4 and 5 specify the value of k for a longitudinally stiffened web. The equation to be used depends on the location of the critical longitudinal web stiffener with respect to a theoretical optimum location of $0.4D_c$ (Vincent 1969) from the compression-flange. The specified k values and the associated optimum stiffener location assume simply supported boundary conditions at the flanges. Changes in flange size along the girder cause D_c to vary along the length of the girder. If the longitudinal stiffener is located a fixed distance from the compression-flange, which is normally the case, the stiffener cannot be at its optimum location all along the girder. Also, the position of the longitudinal stiffener relative to D_c in a composite girder changes due to the shift in the location of the neutral axis after the concrete slab hardens. This shift in the neutral axis is particularly evident in regions of positive flexure. Thus, the specification equations for k allow the Engineer to compute the web bend-buckling capacity for any position of the longitudinal stiffener with respect to D_c. When the distance from the longitudinal stiffener to the compression-flange d_s is less than $0.4D_c$, the stiffener is above its optimum location and web bend-buckling occurs in the panel between the stiffener and the tension flange. When d_s is greater than $0.4D_c$, web bend-buckling occurs in the panel between the stiffener and the compression-flange. When d_s is equal to $0.4D_c$, the stiffener is at its optimum location and bend-buckling occurs in both panels. For this case, both equations yield a value of k equal to 129.3 for a symmetrical girder (Dubas 1948).

Since a longitudinally stiffened web must be investigated for the stress conditions at different limit states and at various locations along the girder, it is possible that the stiffener might be located at an inefficient location for a particular condition resulting in a very low bend-buckling coefficient from Equation 4 or 5. Because simply-supported boundary conditions were assumed in the development of Equations 4 and 5, it is conceivable that the computed web bend-buckling resistance for the longitudinally stiffened web may be less than that computed for a web without longitudinal stiffeners where some rotational restraint from the flanges has been assumed. To prevent this anomaly, the specifications state that the k value for a longitudinally stiffened web must equal or exceed a value of $9.0(D/D_c)^2$, which is the k value for a web without longitudinal stiffeners computed assuming partial rotational restraint from the flanges. Also, near points of dead load contraflexure, both edges of the web may be in compression when stresses in the steel and composite sections due to moments of opposite sign are accumulated. In this case, the neutral axis lies outside the web. Thus, the specifications also limit the minimum value of k to 7.2, which is approximately equal to the theoretical bend-buckling coefficient for a web plate under uniform compression assuming fixed boundary

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conditions at the flanges (Timoshenko and Gere 1961).

Equation 3 is based on extensive experimental and theoretical studies (Galambos 1998) and represents the exact formulation for the R_b factor given by Basler (1961). For rare cases where Equation 3 must be used to compute R_b at the strength limit state for composite sections in regions of positive flexure, a separate calculation should be performed to determine a more appropriate value of A_c to be used to calculate a_r in Equation 6. For this particular case, to be consistent with the original derivation of R_b, it is recommended that A_c be calculated as a combined area for the top flange and the transformed concrete slab that gives the calculated value of D_c for the composite section. The following equation may be used to compute such an effective combined value of A_c:

$$A_c = \frac{A_{fb} D + A_w \left(\frac{D}{2} \right)}{D_c} - \left(A_{fb} + A_w \right)$$

(CE6.10.4.3.2aP-1)

where:

A_{fb} = bottom flange area (mm^2) {$in.^2$}

A_w = area of the web (mm^2) {$in.^2$}

In addition, when the top flange is composite, the stresses that are shed from the web to the flange are resisted in proportion to the relative stiffness of the steel flange and concrete slab. The R_b factor is to be applied only to the stresses in the steel flange. Thus, in this case, a modified R_b factor for the top flange, termed R'_b, can be computed as follows:

$$R'_b = 1.0 - \left(1.0 - R_b^*\right)\left(\frac{A_{ft}}{A_c}\right) \qquad \text{(CE6.10.4.3.2aP-2)}$$

where:

R_b^* = load-shedding factor from Equation 1 computed using the effective combined A_c from Equation C1

A_{ft} = top flange area (mm^2) {$in.^2$}

A_c = effective combined transformed area of the top flange and concrete slab from Equation C1 (mm^2) {$in.^2$}

For a composite section with or without a longitudinally stiffened web, D_c must be calculated according to the provisions of E6.10.3.1.4aP.

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E6.10.4.3.2bP Tension Flanges

For tension flanges, R_b shall be taken as 1.0.

R_b is 1.0 for tension flanges because the increase in flange stresses due to web buckling occurs primarily in the compression flange, and the tension flange stress is not significantly increased by the web buckling (Basler 1961).

E6.10.4.4P MOMENT REDISTRIBUTION FOLLOWING ELASTIC ANALYSIS

For continuous span flexural members rolled, or fabricated from steels with a yield strength not greater than 345 MPa {50 ksi}, and with composite or noncomposite negative flexural sections over the interior supports satisfying the requirements of Articles E6.10.4.1.2P and E6.10.4.1.3P, E6.10.4.1.6aP or E6.10.4.1.6bP, and E6.10.4.1.7P, negative moments over piers due to the factored loading, as determined by elastic procedures, may be reduced by a maximum of 10 percent. Such reduction shall be accompanied by an increase in moments throughout adjacent spans statically equivalent and opposite in sign to the decrease of the negative moments at the adjacent supports. Moment redistribution shall not apply to the negative moment of a cantilever.

In order to take advantage of negative flexure moment redistribution of continuous spans of compact sections, the section shall be compact from the support to the first diaphragm on both sides of the support.

Moment redistribution is not allowed for bridges having skews less than 70°.

Moments shall not be redistributed for the fatigue limit state.

E6.10.5P Service Limit State Control of Permanent Deflection

E6.10.5.1P GENERAL

Load Combination Service II in Articles A3.4.1 and D3.4.1 shall apply.

COMMENTARY

CE6.10.4.3.2bP

CE6.10.4.4P

This provision gives partial recognition to the philosophy of plastic design. Figure C1 illustrates the application of this provision in a two-span continuous beam:

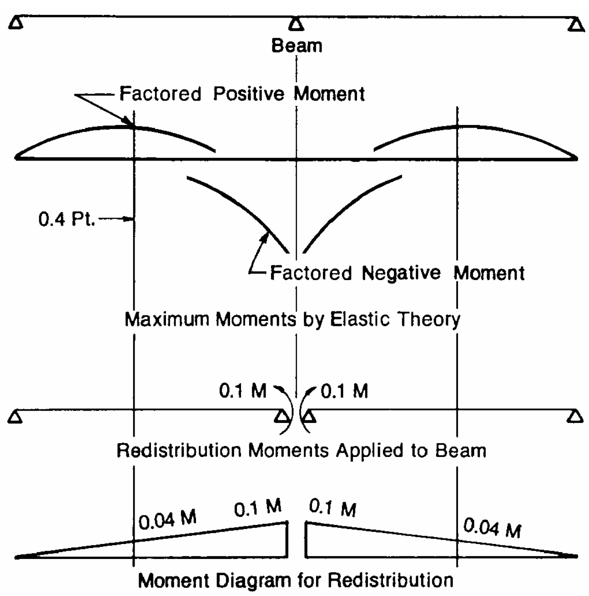

Figure CE6.10.4.4P-1 - Moment Redistribution for Compact Sections at Piers

Bridges with skews less than 70° may have diaphragms which are offset from the support a short distance which may not be adequate to provide lateral bracing in the vicinity of a plastic hinge.

CE6.10.5.1P

The provisions are intended to apply to the design live load specified in Articles A3.6.1.1 and D3.6.1.1. If this

SPECIFICATIONS

The elastic analysis provisions specified in E6.10.4P shall be used in checking both the strength limit state and permanent deflection requirements. The use of inelastic analysis is not permitted.

The web shall satisfy Equation E6.10.3.2.2P-1, using the appropriate value of the depth of the web in compression in the elastic range, D_c.

For members with shear connectors provided throughout their entire length that also satisfy the provisions of E6.10.3.7P, flexural stresses caused by loads applied separately to the composite section may be computed using the short-term or long-term composite section, as appropriate, assuming the concrete slab is fully effective for both positive and negative flexure.

COMMENTARY

criterion were to be applied to a permit load situation, a reduction in the load factor for live load should be considered.

This limit state check is intended to prevent objectionable permanent deflections due to expected severe traffic loadings that would impair rideability. It corresponds to the overload check in the 1996 AASHTO Standard Specifications and is merely an indicator of successful past practice, the development of which is described in Vincent (1969).

Under the load combinations specified in Articles A3.4.1 and D4.4.1, the criterion for control of permanent deflections does not govern for composite noncompact sections; therefore, it need not be checked for those sections. This may not be the case under a different set of load combinations.

Web bend buckling under Load Combination Service II is controlled by limiting the maximum compressive flexural stress in the web to the elastic web bend buckling stress given by Equation E6.10.3.2.2P-1. For composite sections, the appropriate value of the depth of the web in compression in the elastic range, D_c, specified in E6.10.3.1.4aP, is to be used in the equation.

E6.10.3.7P requires that 1 percent longitudinal reinforcement be placed wherever the tensile stress in the slab due to either factored construction loads or due to Load Combination Service II exceeds the factored modulus of rupture of the concrete. By controlling the crack size in regions where adequate shear connection is also provided, the concrete slab can be considered to be effective in tension for computing flexural stresses on the composite section due to Load Combination Service II. If the concrete slab is assumed to be fully effective in negative flexural regions, more than half of the web will typically be in compression increasing the susceptibility of the web to bend buckling.

E6.10.5.2P FLANGE STRESS LIMITATIONS

Flange stresses in positive and negative flexure shall satisfy:

- For both steel flanges of composite sections:

$$f_f \leq 0.95\, R_h F_{yf} \qquad \text{(E6.10.5.2P-1)}$$

- For both flanges of noncomposite sections:

$$f_f \leq 0.80\, R_h F_{yf} \qquad \text{(E6.10.5.2P-2)}$$

where:

f_f = elastic flange stress caused by the factored loading (MPa) {ksi}

CE6.10.5.2P

A resistance factor is not applied because the specified limit is a serviceability criterion for which the resistance factor is 1.0.

SPECIFICATIONS

F_{yf} = specified minimum yield strength of the flange (MPa) {ksi}

For members complying with E6.10.4.4P, the investigation of permanent deflection may be based on moment redistribution.

E6.10.6P Fatigue Requirements for Webs

E6.10.6.1P GENERAL

The provisions of this article shall be used to control out-of-plane flexing of the web due to flexure or shear under repeated live loading.

E6.10.6.2P FATIGUE LOADING

The live load flexural stress and shear stress resulting from the fatigue load, as specified in A3.6.1.4 and D3.6.1.4.2, shall be factored by two times the Pennsylvania Traffic Factor (PTF) and the fatigue load factor specified for the fatigue load combination in Table A3.4.1-1. The PTF is specified in D6.6.1.2.2.

E6.10.6.3P FLEXURE

Webs with or without longitudinal stiffeners shall satisfy the following requirements:

If, $\dfrac{D}{t_w} \le 0.95\sqrt{\dfrac{kE}{F_{yw}}}$, then

$$f_{cf} \le F_{yw} \qquad \text{(E6.10.6.3P-1)}$$

Otherwise,

$$f_{cf} \le 0.9kE\left(\frac{t_w}{D}\right)^2 \qquad \text{(E6.10.6.3P-2)}$$

where:

COMMENTARY

CE6.10.6.1P

If the provisions specified in Articles E6.10.6.3P and E6.10.6.4P are satisfied, significant elastic flexing of the web is not expected to occur, and the member is assumed to be able to sustain an infinite number of smaller loadings without fatigue cracking.

These provisions are included here, rather than in A6.6 and D6.6, because they involve a check of maximum web buckling stresses instead of a check of the stress ranges caused by cyclic loading.

CE6.10.6.3P

The elastic bend-buckling capacity of the web given by Equation 2 is based on an elastic buckling coefficient, k. For webs without longitudinal stiffeners, k is calculated assuming partial rotational restraint at the flanges and simply supported boundary conditions at the transverse stiffeners. The equation for k includes the depth of the web in compression, D_c, in order to address singly symmetric sections. The specifications limit the minimum value of k to 7.2, which is approximately equal to the theoretical bend-buckling coefficient for a web plate under uniform compression assuming fixed boundary conditions at the flanges (Timoshenko and Gere 1961). For webs with longitudinal stiffeners, the value of k depends on the location of the critical longitudinal web stiffener with respect to a theoretical optimum location of $0.4D_c$ (Vincent 1969) from the compression flange and is determined from

SPECIFICATIONS

D = web depth (mm) {in.}

f_{cf} = maximum compressive elastic flexural stress in the compression flange due to the unfactored permanent load and the fatigue loading as specified in E6.10.6.2P (MPa) {ksi}

F_{yw} = specified minimum yield strength of the web (MPa) {ksi}

k = elastic bend-buckling coefficient for the web taken equal to $9.0(D/D_c)^2 \geq 7.2$ for webs without longitudinal stiffeners. For webs with longitudinal stiffeners, k shall be determined from either Equation E6.10.4.3.2aP-4 or Equation E6.10.4.3.2aP-5, as applicable

D_c = depth of web in compression due to the unfactored dead load and the factored fatigue load as specified in E6.10.6.2P (mm) {in.} {in.}(mm) {in.}

t_w = web thickness (mm) {in.}

E6.10.6.4P SHEAR

Webs of homogeneous sections with transverse stiffeners and with or without longitudinal stiffeners shall be proportioned to satisfy:

$$v_{cf} \leq 0.58\, C\, F_{yw} \qquad \text{(E6.10.6.4P-1)}$$

where:

v_{cf} = maximum elastic shear stress in the web due to the unfactored permanent load and the fatigue loading as specified in E6.10.6.2P (MPa) {ksi}

C = ratio of the shear buckling stress to the shear yield strength as specified in E6.10.7.3.3a

F_{yw} = specified minimum yield strength of the web (MPa) {ksi}

E6.10.7P SHEAR RESISTANCE

E6.10.7.1P General

The factored shear resistance of a beam or girder, V_r, shall be taken as:

COMMENTARY

either Equation E6.10.4.3.2aP-4 or Equation E6.10.4.3.2aP-5, as applicable (Frank and Helwig 1995). The k values from these two equations and the associated optimum stiffener location conservatively assume simply supported boundary conditions at the flanges. The specified web slenderness limit of $0.95\sqrt{kE/F_{yw}}$ is the web slenderness at which the compression flange theoretically reaches the yield strength prior to elastic bend-buckling of the web according to Equation 2.

The stress, f_{cf}, is taken as being indicative of the maximum flexural stress in the web.

For the loading and load combination applicable to this limit state, it is assumed that the entire cross-section will remain elastic and, therefore, D_c can be determined as specified in E6.10.3.1.4a.

CE6.10.6.4P

The shear force in unstiffened webs and in webs of hybrid sections is already limited to either the shear yielding or the shear buckling force at the strength limit state by the provisions of E6.10.7.2. Consequently, the requirement in this article need not be checked for those sections.

CE6.10.7.1P

This article applies to:

SPECIFICATIONS

$$V_r = \varphi_v V_n \quad \text{(E6.10.7.1P-1)}$$

where:

V_n = the nominal shear resistance specified in Articles E6.10.7.2P and E6.10.7.3P for unstiffened and stiffened webs, respectively

φ_v = resistance factor for shear specified in Articles A6.5.4.2 and D6.5.4.2

Transverse intermediate stiffeners shall be designed as specified in E6.10.8.1P. Longitudinal stiffeners shall be designed as specified in E6.10.8.3P.

Stiffened web panels of homogeneous sections shall also be investigated for shear under repeated live loading as specified in E6.10.6.4P and for shear and moment interaction as specified in E6.10.7.3.3P.

Interior web panels of homogeneous and hybrid girders:

- Without a longitudinal stiffener and with a transverse stiffener spacing not exceeding 1.5D, or
- With a longitudinal stiffener and a transverse stiffener spacing not exceeding 1.5D

shall be considered stiffened, and the provisions of E6.10.7.3P shall apply. Otherwise, the panel shall be considered unstiffened, and the provisions of E6.10.7.2P shall apply.

Provisions for end panels shall be as specified in E6.10.7.3.3cP and E6.10.7.3.4P.

Transverse stiffener spacing shall also satisfy the requirements of E6.10.3.2.4.2P for deck slab overhang form support.

COMMENTARY

- Sections without stiffeners,
- Sections with transverse stiffeners only, and
- Sections with both transverse and longitudinal stiffeners.

A flow chart for shear capacity of I-sections is shown below.

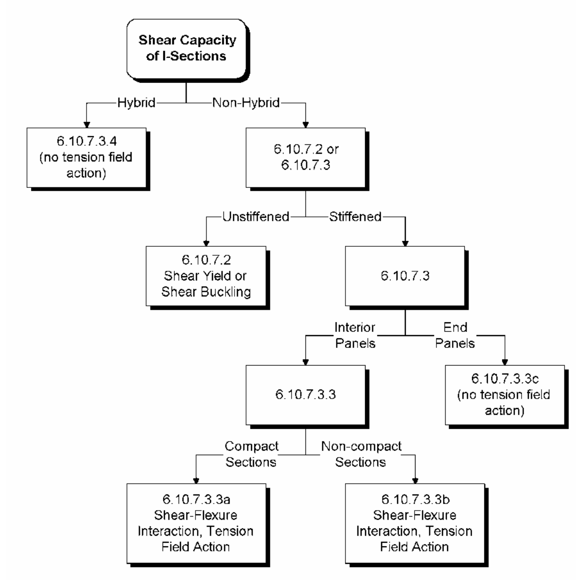

Figure CE6.10.7.1P-1 - Flow Chart for Shear Design of I-Sections

Unstiffened and stiffened interior web panels are defined according to the maximum transverse stiffener spacing requirements specified in this article. The nominal shear resistance of unstiffened web panels in both homogeneous and hybrid sections is defined by either shear yield or shear buckling, depending on the web slenderness ratio, as specified in E6.10.7.2P. The nominal shear resistance of stiffened interior web panels of homogeneous sections is defined by the sum of the shear-yielding or shear-buckling resistance and the post-buckling resistance from tension-field action, modified as necessary by any moment-shear interaction effects, as specified in EE6.10.7.3.3P. For compact sections, this nominal shear resistance is specified by either Equation E6.10.7.3.3aP-1 or Equation E6.10.7.3.3aP-2. For noncompact sections, this nominal shear resistance is specified by either Equation E6.10.7.3.3bP-1 or Equation E6.10.7.3.3bP-2. For homogeneous sections, the nominal shear resistance of end panels in stiffened webs is defined by either shear yielding or shear buckling, as specified in E6.10.7.3.3cP. For hybrid sections, the nominal shear resistance of all stiffened web panels is defined by either shear yielding or shear buckling, as specified in E6.10.7.3.4P.

SPECIFICATIONS

Concurrent moment shall be considered only if tension field action is utilized.

COMMENTARY

Separate interaction equations are given to define the effect of concurrent moment for compact and noncompact sections because compact sections are designed in terms of moments, whereas noncompact sections are designed in terms of stresses. For convenience, it is conservatively specified that the maximum moments and shears from the moment and shear envelopes be used in the interaction equations.

E6.10.7.2P NOMINAL RESISTANCE OF UNSTIFFENED WEBS

The nominal shear resistance of unstiffened webs of hybrid and homogeneous girders shall be taken as:

$$V_n = CV_p \qquad \text{(E6.10.7.2P-1)}$$

for which:

$$V_p = 0.58 F_{yw} D t_w \qquad \text{(E6.10.7.2P-2)}$$

where:

C = ratio of shear buckling stress to the shear yield strength as specified in E6.10.7.3.3aP, with the shear buckling coefficient, k, taken equal to 5.0

V_p = plastic shear force (N) {kip}

F_{yw} = specified minimum yield strength of the web (MPa) {ksi}

D = web depth (mm) {in.}

t_w = thickness of web (mm) {in.}

CE6.10.7.2P

The nominal shear resistance of unstiffened webs of hybrid and homogeneous girders is limited to the elastic shear buckling force given by Equation 1. The consideration of tension-field action (Basler 1961) is not permitted for unstiffened webs. The elastic shear buckling force is calculated as the product of the constant C specified in E6.10.7.3.3aP times the plastic shear force, V_p, given by Equation 2. The plastic shear force is equal to the web area times the assumed shear yield strength of $F_{yw}/(3)^{0.5}$. The shear bucking coefficient, k, to be used in calculating the constant C , calculated using Equation E6.10.7.3.3aP-8 for unstiffened webs modeled as infinitely long strips is 5.0, which is a conservative approximation of the exact value of 5.35 for an infinitely long strip with simply supported edges (Timoshenko and Gere 1961).

E6.10.7.3P NOMINAL RESISTANCE OF STIFFENED WEBS

E6.10.7.3.1P General

The nominal shear resistance of transversely or transversely and longitudinally stiffened interior and end web panels shall be as specified in Articles E6.10.7.3.3P and E6.10.7.3.4P for homogeneous and hybrid sections, respectively. The total web depth, D, shall be used in determining the nominal shear resistance of web panels with longitudinal stiffeners. Transverse stiffeners shall be spaced using the maximum shear in a panel.

Stiffeners shall satisfy the requirements specified in E6.10.8P.

CE6.10.7.3.1P

Longitudinal stiffeners divide a web panel into subpanels. The shear resistance of the entire panel can be taken as the sum of the shear resistance of the subpanels (Cooper 1967). However, the contribution of the longitudinal stiffener at a distance of $2D_c/5$ from the compression flange is relatively small. Thus, it is conservatively recommended that the influence of the longitudinal stiffener be neglected in computing the nominal shear resistance of the web plate.

SPECIFICATIONS

E6.10.7.3.2P Handling Requirement

For web panels without longitudinal stiffeners, transverse stiffeners shall be used if: $\frac{D}{t_w} > 150$

(E6.10.7.3.2P-1)

The spacing of transverse stiffeners, d_o, shall satisfy:

$$d_o \le D\left[\frac{260}{\left(D/t_w\right)}\right]^2 \quad \text{(E6.10.7.3.2P-2)}$$

The spacing of transverse stiffeners, d_o, calculated by Equation 2, shall not exceed 1.5D.

E6.10.7.3.3P Homogeneous Sections

E6.10.7.3.3aP Interior Panels of Compact Sections

The nominal shear resistance of interior web panels of compact sections complying with the provisions of E6.10.7.1P shall be taken as:

If $M_u \le 0.5\ \varphi_f\ M_p$, then:

$$V_n = V_p\left[C + \frac{0.87(1-C)}{\sqrt{1+\left(\frac{d_o}{D}\right)^2}}\right] \quad \text{(E6.10.7.3.3aP-1)}$$

If $M_u > 0.5\ \varphi_f\ M_p$, then:

$$V_n = RV_p\left[C + \frac{0.87(1-C)}{\sqrt{1+\left(\frac{d_o}{D}\right)^2}}\right] \ge CV_p \quad \text{(E6.10.7.3.3aP-2)}$$

for which:

$$R = \left[0.6 + 0.4\left(\frac{M_r - M_u}{M_r - 0.75\,\phi_f\,M_y}\right)\right] \le 1.0 \quad \text{(E6.10.7.3.3aP-3)}$$

$$V_p = 0.58\ F_{yw}\,D\,t_w \quad \text{(E6.10.7.3.3aP-4)}$$

where:

COMMENTARY

CE6.10.7.3.2P

Transverse stiffeners are required on web panels with a slenderness ratio greater than 150 in order to facilitate handling of sections without longitudinal stiffeners during fabrication and erection. The spacing of the transverse stiffeners is arbitrarily limited by Equation 2 (Basler 1961). Substituting a web slenderness of 150 into Equation 2 results in a maximum transverse stiffener spacing of 3D, which corresponds to the maximum spacing requirement in E6.10.7.1P for web panels without longitudinal stiffeners. For higher web slenderness ratios, the maximum allowable spacing is reduced to less than 3D.

The requirement in Equation 2 is not needed for web panels with longitudinal stiffeners because maximum transverse stiffener spacing is already limited to 1.5D.

CE6.10.7.3.3aP

Stiffened interior web panels of homogeneous sections may develop post-buckling shear resistance due to tension-field action (Basler 1961). The action is analogous to that of the tension diagonals of a Pratt truss. The nominal shear resistance of these panels can be computed by summing the contributions of beam action and of the post-buckling tension-field action. The resulting expression is given in Equation 1, where the first term in the bracket relates to either the shear yield or shear buckling force and the second term relates to the post-buckling tension-field force.

The coefficient, C, is equal to the ratio of the elastic shear buckling stress of the panel, computed assuming simply supported boundary conditions, to the shear yield strength assumed to be equal to $F_{yw}/(3)^{0.5}$. Equation 7 is applicable only for C values not exceeding 0.8 (Basler 1961). Above 0.8, C values are given by Equation 6 until a limiting slenderness ratio is reached where the shear buckling stress is equal to the shear yield strength and C = 1.0. Equation 8 for the shear buckling coefficient is a simplification of two exact equations for k that depend on the panel aspect ratio.

When both shear and flexural moment are high in a stiffened interior panel under tension-field action, the web plate must resist the shear and also participate in resisting the moment. Panels whose resistance is limited to the shear buckling or shear yield force are not subject to moment-shear interaction effects. Basler (1961) shows that stiffened web plates in noncompact sections are capable of resisting both moment and shear, as long as the shear force due to the factored loadings is less than $0.6\varphi_v V_n$ or the flexural stress in the compression flange due to the factored loading is less than $0.75\varphi_f F_y$.

For compact sections, flexural resistances are expressed in terms of moments rather than stresses. For convenience, a limiting moment of $0.5\varphi_f M_p$ is defined rather than a

SPECIFICATIONS

M_u = maximum moment in the panel under consideration due to the factored loads (N-mm) {k-in}

V_n = nominal shear resistance (N) {kip}

V_p = plastic shear force (N) {kip}

M_r = factored flexural resistance as specified in E6.10.4P or E6.11.2.1.1P (N-mm) {k-in}

φ_f = resistance factor for flexure specified in A6.5.4.2

M_y = yield moment as specified in Articles E6.10.3.1.2P or E6.10.3.3.1P (N-mm) {k-in}

D = web depth (mm) {in.}

d_o = stiffener spacing (mm) {in.}

C = ratio of the shear buckling stress to the shear yield strength

The ratio, C, shall be determined as specified below:

If $\frac{D}{t_w} < 1.10\sqrt{\frac{Ek}{F_{yw}}}$, then

$$C = 1.0 \tag{E6.10.7.3.3aP-5}$$

If $1.10\sqrt{\frac{Ek}{F_{yw}}} \leq \frac{D}{t_w} \leq 1.38\sqrt{\frac{Ek}{F_{yw}}}$, then:

$$C = \frac{1.10}{\frac{D}{t_w}}\sqrt{\frac{Ek}{F_{yw}}} \tag{E6.10.7.3.3aP-6}$$

If $\frac{D}{t_w} > 1.38\sqrt{\frac{Ek}{F_{yw}}}$, then:

$$C = \frac{1.52}{\left(\frac{D}{t_w}\right)^2}\left(\frac{Ek}{F_{yw}}\right) \tag{E6.10.7.3.3aP-7}$$

for which:

$$k = 5 + \frac{5}{\left(\frac{d_o}{D}\right)^2} \tag{E6.10.7.3.3aP-8}$$

COMMENTARY

limiting moment of $0.75\varphi_f M_y$ in determining when the moment-shear interaction occurs by using an assumed shape factor (M_p/M_y) of 1.5. This eliminates the need to compute the yield moment to simply check whether or not the interaction effect applies. When the moment due to factored loadings exceeds $0.5\varphi_f M_p$, the nominal shear resistance is taken as V_n, given by Equation 2, reduced by the specified interaction factor, R.

Both upper and lower limits are placed on the nominal shear resistance in Equation 2 determined by applying the interaction factor, R. The lower limit is either the shear yield or shear buckling force. Sections with a shape factor below 1.5 could potentially exceed V_n according to the interaction equation at moments due to the factored loadings slightly above the defined limiting value of $0.5\varphi_f M_p$. Thus, for compact sections, an upper limit of 1.0 is placed on R.

To avoid the interaction effect, transverse stiffeners may be spaced so that the shear due to the factored loadings does not exceed the larger of:

- $0.60\ \varphi_v V_n$, where V_n is given by Equation 1 or
- The factored shear buckling or shear yield resistance equal to $\varphi_v C V_p$.

k is known as the shear buckling coefficient.

SPECIFICATIONS

E6.10.7.3.3bP Interior Panels of Noncompact Sections

The nominal shear resistance of interior web panels of noncompact sections complying with the provisions of E6.10.7.1P shall be taken as:

If $f_u \leq 0.75\phi_f F_y$, then: $V_n = V_p \left[C + \dfrac{0.87\,(1 - C)}{\sqrt{1 + \left(\dfrac{d_o}{D}\right)^2}} \right]$

(E6.10.7.3.3bP-1)

If $f_u > 0.75\,\phi_f\,F_y$, then:

$$V_n = RV_p \left[C + \frac{0.87\,(1 - C)}{\sqrt{1 + \left(\dfrac{d_o}{D}\right)^2}} \right] \geq CV_p$$

(E6.10.7.3.3bP-2)

for which:

$$R = \left[0.6 + 0.4 \left(\frac{F_r - f_u}{F_r - 0.75\,\phi_f\,F_y} \right) \right]$$ (E6.10.7.3.3bP-3)

where:

f_u = flexural stress in the compression or tension flange due to the factored loading, whichever flange has the maximum ratio of f_u to F_r in the panel under consideration (MPa) {ksi}

C = ratio of shear buckling stress to the shear yield strength as specified in E6.10.7.3.3aP

F_r = factored flexural resistance of the flange for which f_u was determined as specified in E6.10.4P or E6.11.2.1.1P (MPa) {ksi}

E6.10.7.3.3cP End Panels

The nominal shear resistance of an end panel shall be limited to the shear buckling or shear yield force taken as:

$V_n = CV_p$ (E6.10.7.3.3cP-1)

for which:

$V_n = 0.58 F_{yw} D t_w$ (E6.10.7.3.3cP-2)

where:

COMMENTARY

CE6.10.7.3.3bP

The commentary of E6.10.7.3.3aP applies, except that for noncompact sections, flexural resistances are expressed in terms of stress rather than moment in the interaction equation. The upper limit of 1.0 applied to R in Equation E6.10.7.3.3aP-3 applies to compact sections and need not be applied to Equation E6.10.7.3.3bP-3 for noncompact sections.

CE6.10.7.3.3cP

The shear in end panels is limited to either the shear yield or shear buckling force given by Equation 1 in order to provide an anchor for the tension field in adjacent interior panels.

SPECIFICATIONS

C = ratio of the shear buckling stress to the shear yield strength as specified in E6.10.7.3.3aP

V_p = plastic shear force (N) {kip}

The transverse stiffener spacing for end panels without a longitudinal stiffener shall not exceed 0.5D. The transverse stiffener spacing of end panels with a longitudinal stiffener shall not exceed 0.5 times the maximum subpanel depth.

E6.10.7.3.4P Hybrid Sections

The transverse stiffener spacing requirements of E6.10.7.3.3cP are applicable to hybrid sections.

The nominal shear strength of interior and end web panels is limited to the shear buckling or shear yield force taken as:

$$V_n = CV_p \quad \text{(E6.10.7.3.4P-1)}$$

E6.10.7.4P SHEAR CONNECTORS

E6.10.7.4.1P General

In composite sections, stud or channel shear connectors shall be provided at the interface between the concrete deck slab and the steel section to resist the interface shear.

Simple span composite bridges shall be provided with shear connectors throughout the length of the span.

Shear connectors are required along the entire length of the girder when a composite girder analysis has been performed.

In negative flexure regions, the longitudinal deck reinforcement shall be extended into the positive flexure region as specified in E6.10.3.7P.

E6.10.7.4.1aP Types

Stud and channel shear connectors shall be designed by the provisions of this article.

Shear connectors should be of a type that permits a thorough compaction of the concrete to ensure that their entire surfaces are in contact with the concrete. The connectors shall be capable of resisting both horizontal and vertical movement between the concrete and the steel.

The ratio of the height to the diameter of a stud shear connector shall not be less than 4.0.

The minimum diameter of studs shall be 19 mm {3/4

COMMENTARY

CE6.10.7.3.4P

Tension-field action is not permitted for hybrid sections. Thus, the nominal shear resistance is limited to either the shear yield or the shear buckling force given by Equation 1.

C6.10.7.4.1P

Mechanical shear connectors provide for the horizontal shear at the interface between the concrete slab and the steel girder in the positive moment regions and the horizontal shear between the longitudinal reinforcement steel within the effective flange width and the steel girder in the negative moment regions.

SPECIFICATIONS

in.}.

Channel shear connectors shall have fillet welds not smaller than 5 mm placed along the heel and toe of the channel.

E6.10.7.4.1bP Pitch

The pitch of the shear connectors shall be determined to satisfy the fatigue limit state as specified in E6.10.7.4.2P and E6.10.7.4.3P, as applicable. The resulting number of shear connectors shall not be less than the number required to satisfy the strength limit state as specified in E6.10.7.4.4P.

The pitch, p, of the shear connectors shall satisfy:

$$p \le \frac{nZ_r I}{V_{sr}\, Q} \qquad \text{(E6.10.7.4.1bP-1)}$$

where:

p = pitch of shear connectors along the longitudinal axis (mm) {in.}

n = number of shear connectors in a cross-section

I = moment of inertia of the short-term composite section (mm^4) {in.^{4}}

Q = first moment of the transformed area of the slab about the neutral axis of the short-term composite section (mm^3) {in.^{3}}

V_{sr} = shear force range under LL + I determined for the fatigue limit state (N) {kip}

Z_r = shear fatigue resistance of an individual shear connector determined as specified in E6.10.7.4.2P (N) {kip}

The center-to-center pitch of stud shear connectors shall not exceed 600 mm and shall not be less than six stud diameters.

The center-to-center pitch of channel shear connectors shall not exceed 600 mm {24 in.} and shall not be less than 150 mm {6 in.}.

COMMENTARY

CE6.10.7.4.1bP

The parameters I and Q should be determined using the deck within the effective flange width. However, in negative flexure regions, the parameters I and Q may be determined using the reinforcement within the effective flange width for negative moment, unless the concrete slab is considered to be fully effective for negative moment in computing the longitudinal range of stress, as permitted in Articles A6.6.1.2.1 and D6.6.1.2.1.

The maximum fatigue shear range is produced by placing the fatigue live load immediately to the left and to the right of the point under consideration. For the load in these positions, positive moments are produced over significant portions of the girder length. Thus, the use of the full composite section, including the concrete deck, is reasonable for computing the shear range along the entire span. Also, the horizontal shear force in the deck is most

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often considered to be effective along the entire span in the analysis. To satisfy this assumption, the shear force in the deck must be developed along the entire span. An option is permitted to ignore the concrete deck in computing the shear range in regions of negative flexure, unless the concrete is considered to be fully effective in computing the longitudinal range of stress, in which case the shear force in the deck must be developed. If the concrete is ignored in these regions, the specified maximum pitch must not be exceeded.

E6.10.7.4.1cP Transverse Spacing

Shear connectors shall be placed transversely across the top flange of the steel section and may be spaced at regular or variable intervals.

Stud shear connectors shall not be closer than 4.0 stud diameters center-to-center transverse to the longitudinal axis of the supporting member.

The clear distance between the edge of the top flange and the edge of the nearest shear connector shall not be less than 25 mm {1.0 in.}.The minimum number of studs in a group shall consist of two in a single transverse row.

E6.10.7.4.1dP Cover and Penetration

The clear depth of concrete cover over the tops of the shear connectors should not be less than 60 mm {2 1/2 in.}. Shear connectors should penetrate at least 50 mm {2 in.} into the deck.

CE6.10.7.4.1dP

Stud connectors should penetrate through the haunch between the bottom of the deck and top flange, if present, and into the deck.

E6.10.7.4.1eP Splice Locations

Shear connectors at splice locations shall be arranged to clear fasteners and shall be welded to the splice plate. Up to 20% fewer connectors, than required by design, are acceptable in the splice zone, provided that the deleted connectors are furnished as additional connectors adjacent to the splice.

E6.10.7.4.2P Fatigue Resistance of Shear Connectors in Composite Sections

The fatigue resistance of an individual shear connector, Z_r, shall be taken as:

Metric Units

$$Z_r = \alpha d^2 \geq \frac{38.0\, d^2}{2} \qquad \text{(E6.10.7.4.2P-1)}$$

U.S. Customary units:

CE6.10.7.4.2P

For development of this information, see Slutter and Fisher (1966).

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$$Z_r = \alpha d^2 \geq \frac{5.5\, d^2}{2}$$

for which:

Metric Units:

$$\alpha = 238 - 29.5 \text{ Log N} \qquad \text{(E6.10.7.4.2P-2)}$$

U.S. Customary Units:

$$\alpha = 34.5 - 4.28 \text{ Log N}$$

where:

d = diameter of the stud (mm) {in.}

N = number of cycles specified in Articles A6.6.1.2.5 and D6.6.1.2.5

The pitch shall be determined from Equation E6.10.7.4.1bP-1 using the value of Z_r and the shear force range V_{sr}.

The effect of the shear connector on the fatigue resistance of the flange shall be investigated using the provisions of Articles A6.6.1.2 and D6.6.1.2.

The fatigue resistance of an individual channel shear connector, Z_r, shall be taken as:

Metric Units:

$$Z_r = B\, w \leq 184\, w \qquad \text{(E6.10.7.4.2-3P)}$$

U.S. Customary Units:

$$Z_r = B\, w \leq 1.05\, w$$

where:

Metric Units:

$$B = 1525 - 175 \text{ Log } N \qquad \text{(E6.10.7.4.2-4P)}$$

U.S. Customary Units:

$$B = 8.7 - \text{Log } N$$

w = length of channel shear connector measured in a transverse direction on the flange of a girder (mm) {in.}

N = number of cycles specified in A6.6.1.2.5 and D6.6.1.2.5

COMMENTARY

Equations 3 and 4 were added because the LRFD specification does not address the fatigue resistance of channel shear connectors. Equations 3 and 4 were converted from Article 10.3.8.5.1.1 of AASHTO Standard Specification for Highway Bridges.

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E6.10.7.4.3P Special Requirements for Points of Permanent Load Contraflexure

The provisions of this article are only applicable when analyzing existing bridges where the shear connectors are not provided along the full length of the girder.

Where continuous girders that are composite in the positive flexure regions do not contain shear connectors in the negative flexure region, additional shear connectors shall be provided in the region of points of permanent load contraflexure.

The number of additional connectors, n_{AC}, shall be taken as:

$$n_{AC} = \frac{A_r\, f_{sr}}{Z_r} \qquad \text{(E6.10.7.4.3P-1)}$$

where:

A_r = total area of reinforcement within the effective flange width (mm^2) {in.^{2}}

f_{sr} = stress range in the longitudinal reinforcement specified in Articles A5.5.3.1 and D5.5.3.1 (MPa) {ksi}

Z_r = shear fatigue resistance of an individual shear connector, as specified in E6.10.7.4.2P (N) {kip}

The additional shear connectors shall be placed within a distance equal to one-third of the effective slab width on each side of the point of permanent load contraflexure. Field splices should be placed so that they do not interfere with the shear connectors.

E6.10.7.4.4P Strength Limit State

E6.10.7.4.4aP General

The factored resistance of shear connectors, Q_r, shall be taken as:

$$Q_r = \phi_{sc}\, Q_n \qquad \text{(E6.10.7.4.4aP-1)}$$

where:

Q_n = nominal resistance as specified in E6.10.7.4.4cP

φ_{sc} = resistance factor for shear connectors specified in A6.5.4.2 and D6.5.4.2

The number of shear connectors provided between the

COMMENTARY

CE6.10.7.4.3P

For new designs, PennDOT requires composite girders to have shear connectors along the full length of the girders and the provisions of this article will not be applicable.

The purpose of the additional connectors is to develop the reinforcing bars used as part of the negative flexural section.

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section of maximum positive moment and each adjacent point of 0.0 moment or between each adjacent point of 0.0 moment and centerline of an interior support shall not be less than:

$$n = \frac{V_h}{Q_r} \qquad \text{(E6.10.7.4.4aP-2)}$$

where:

V_h = nominal horizontal shear force as specified in E6.10.7.4.4bP

Q_r = factored shear resistance of one shear connector specified in E6.10.7.4.4aP

E6.10.7.4.4bP Nominal Horizontal Shear Force

The total horizontal shear force, V_h, between the point of maximum positive moment and each adjacent point of 0.0 moment shall be the lesser of either:

$$V_h = 0.85f'_c b t_s \qquad \text{(E6.10.7.4.4bP-1)}$$

or

$$V_h = F_{yw}Dt_w + F_{yt}b_t t_t + F_{yc}b_f t_f \qquad \text{(E6.10.7.4.4bP-2)}$$

where:

f'_c = specified 28-day compressive strength of the concrete (MPa) {ksi}

b = effective width of the slab (mm) {in.}

b_f = width of compression flange (mm) {in.}

b_t = width of tension flange (mm) {in.}

t_s = slab thickness (mm) {in.}

F_{yw} = specified minimum yield strength of the web (MPa) {ksi}

F_{yt} = specified minimum yield strength of the tension flange (MPa) {ksi}

F_{yc} = specified minimum yield strength of the compression flange (MPa) {ksi}

D = web depth (mm) {in.}

COMMENTARY

CE6.10.7.4.4bP

Composite beams in which the longitudinal spacing of shear connectors has been varied according to the intensity of shear and duplicate beams where the number of connectors were uniformly spaced have exhibited essentially the same ultimate strength and the same amount of deflection at service loads. Only a slight deformation in the concrete and the more heavily stressed connectors is needed to redistribute the horizontal shear to other less heavily stressed connectors. The important consideration is that the total number of connectors be sufficient to develop the shear, V_h, on either side of the point of maximum moment.

In negative flexure regions, sufficient shear connectors are required to transfer the ultimate tensile force in the reinforcement from the slab to the steel section.

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t_t = thickness of tension flange (mm) {in.}

t_f = thickness of compression flange (mm) {in.}

t_w = web thickness (mm) {in.}

For continuous span composite sections, the total horizontal shear force between each adjacent point of 0.0 moment and the centerline of an interior support shall be taken as:

$$V_h = A_r F_{yr} \qquad \text{(E6.10.7.4.4bP-3)}$$

where:

A_r = total area of longitudinal reinforcement over the interior support within the effective slab width (mm^2) {$in.^2$}

F_{yr} = specified minimum yield strength of the longitudinal reinforcement (MPa) {ksi}

E6.10.7.4.4cP Nominal Shear Resistance

The nominal shear resistance of one stud shear connector embedded in a concrete slab shall be taken as:

$$Q_n = 0.5\, A_{sc} \sqrt{f_{c'} E_c} \le A_{sc}\; F_u \qquad \text{(E6.10.7.4.4cP-1)}$$

where:

A_{sc} = cross-sectional area of a stud shear connector (mm^2) {$in.^2$}

f'_c = specified 28-day compressive strength of concrete (MPa) {ksi}

E_c = modulus of elasticity of concrete, as specified in Articles A5.4.2.4 and D5.4.2.4 (MPa) {ksi}

F_u = specified minimum tensile strength of a stud shear connector specified in A6.4.4 (MPa) {ksi}

The nominal shear resistance of one channel shear connector embedded in a concrete slab shall be taken as:

$$Q_n = 0.3\left(t_f + 0.5 t_w\right) L_c \sqrt{f_{c'} E_c} \qquad \text{(E6.10.7.4.4cP-2)}$$

where:

t_f = flange thickness of channel shear connector (mm) {in.}

COMMENTARY

CE6.10.7.4.4cP

Studies have defined stud shear connector strength as a function of both the concrete modulus of elasticity and concrete strength (Ollgaard et al. 1971). Note that an upper bound on stud shear strength is the product of the cross-sectional area of the stud times its ultimate tensile strength.

Equation 2 is a modified form of the formula for the resistance of channel shear connectors developed in Slutter and Driscoll (1965), which extended its use to low-density as well as normal density concrete.

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t_w = web thickness of channel shear connector (mm) {in.}

L_c = length of channel shear connector (mm) {in.}

E6.10.8P Stiffeners

E6.10.8.1P TRANSVERSE INTERMEDIATE STIFFENERS

E6.10.8.1.1P General

Transverse stiffeners shall consist of plates or angles welded or bolted to either one or both sides of the web.

Stiffeners not used as connection plates shall be a tight fit at the compression flange, but need not be in bearing with the tension flange.

Stiffeners used as connecting plates for diaphragms or cross-frames shall be connected by welding or bolting to both flanges.

The distance between the end of the web-to-stiffener weld and the near edge of the web-to-flange fillet weld shall not be less than $4t_w$ or more than $6t_w$.

Transverse stiffeners shall also satisfy the requirements given in Standard Drawing BC-753M.

E6.10.8.1.2P Projecting Width

The width, b_t, of each projecting stiffener element shall satisfy:

Metric Units:

$$b_t \geq 50 + \frac{d}{30} \qquad \text{(E6.10.8.1.2P-1)}$$

U.S. Customary Units:

$$b_t \geq 2.0 + \frac{d}{30} \qquad \text{(E6.10.8.1.2P-1)}$$

and

$$16.0\, t_p \geq b_t \geq 0.25\, b_f \qquad \text{(E6.10.8.1.2P-2)}$$

where:

d = steel section depth (mm) {in.}

t_p = thickness of the projecting element (mm) {in.}

F_{ys} = specified minimum yield strength of the stiffener

COMMENTARY

CE6.10.8.1.2P

The requirements in this article are intended to prevent local buckling of the transverse stiffener.

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(MPa) {ksi}

b_f = full width of the wider steel flange at a section (mm) {in.}

E6.10.8.1.3P Moment of Inertia

The moment of inertia of any transverse stiffener shall satisfy:

$$I_t \geq d_O\, t_w\, J \qquad \text{(E6.10.8.1.3P-1)}$$

For which:

$$J = 2.5\left(\frac{D}{d_O}\right) - 2.0 \geq 0.5 \qquad \text{(E6.10.8.1.3P-2)}$$

where:

I_t = moment of inertia of the transverse stiffener taken about the edge in contact with the web for single stiffeners and about the midthickness of the web for stiffener pairs (mm^4) {$in.^4$}

t_w = web thickness (mm) {in.}

d_o = transverse stiffener spacing (mm) {in.}

D = web depth (mm) {in.}

Transverse stiffeners used in conjunction with longitudinal stiffeners shall also satisfy:

$$I_t \geq \left(\frac{b_t}{b_\ell}\right) 2.5\left(\frac{D}{3.0\, d_O}\right) I_\ell \qquad \text{(E6.10.8.1.3-3)}$$

where:

b_t = projecting width of transverse stiffener (mm) {in.}

b_ℓ = projecting width of longitudinal stiffener (mm) {in.}

I_ℓ = moment of inertia of the longitudinal stiffener taken about the edge in contact with the web, based on the effective section as specified in E6.10.8.3.3 (mm^4) {$in.^4$}

D = web depth (mm) {in.}

COMMENTARY

CE6.10.8.1.3

For the web to adequately develop the tension field, the transverse stiffener must have sufficient rigidity to cause a node to form along the line of the stiffener. For ratios of (d_o/D) less than 1.0, much larger values of I_t are required, as discussed in Timoshenko and Gere (1961).

Lateral loads along the length of a longitudinal stiffener are transferred to the adjacent transverse stiffeners as concentrated reactions (Cooper 1967). Equation 3 gives a relationship between the moments of inertia of the longitudinal and transverse stiffeners to ensure that the latter does not fail under the concentrated reactions. Equation 3 is equivalent to Equation 10-111 in AASHTO (1996).

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E6.10.8.1.4P Area

Transverse intermediate stiffeners required to carry the forces imposed by tension-field action of the web as specified in E6.10.7.3P shall satisfy:

$$A_s \geq \left[0.15B\ \frac{D}{t_w}\ (1 - C)\left(\frac{V_u}{V_r}\right) - 18 \right] \frac{F_{yw}}{F_{cr}}\ t_w^2 \qquad \text{(E6.10.8.1.4P-1)}$$

in which:

$$F_{cr} = \frac{0.311E}{\left(\frac{b_t}{t_p}\right)^2} \leq F_{ys}$$

where:

V_r = factored shear resistance as specified in E6.10.7.1P (N) {kip}

V_u = shear due to factored loads at the strength limit state (N) {kip}

A_s = stiffener area; total area of both stiffeners for pairs (mm^2) {$in.^2$}

B = 1.0 for stiffener pairs

B = 1.8 for single angle stiffeners

B = 2.4 for single plate stiffeners

C = ratio of the shear buckling stress to the shear yield strength as specified in E6.10.7.3.3aP

F_{yw} = specified minimum yield strength of the web (MPa) {ksi}

F_{ys} = specified minimum yield strength of the stiffener (MPa) {ksi}

COMMENTARY

CE6.10.8.1.4P

Transverse stiffeners need sufficient area to resist the vertical component of the tension field. The formula for the required stiffener area can give a negative result. In that case, the required area is 0.0. A negative result indicates that the web alone is sufficient to resist the vertical component of the tension field. The stiffener then need only be proportioned for stiffness according to E6.10.8.1.3P and satisfy the projecting width requirements of E6.10.8.1.2P. For web panels not required to develop a tension field, this requirement need not be investigated.

E6.10.8.2P BEARING STIFFENERS

E6.10.8.2.1P General

Bearing reactions and other concentrated loads, either in the final state or during construction, shall be resisted by bearing stiffeners. Bearing stiffeners shall be placed on webs of rolled beams at all bearing locations and other points of concentrated loads unless calculations using the web crippling provisions of AISC LRFD, Third Edition,

CE6.10.8.2.1P

Inadequate provision to resist concentrated loads has resulted in failures, particularly in temporary construction.

If an owner chooses not to utilize bearing stiffeners where specified in this article, the web crippling provisions of AISC (1993) should be used to investigate the adequacy of the component to resist a concentrated load.

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show that bearing stiffeners are not required. The Department's STLRFD program performs this web crippling check.

Bearing stiffeners shall be placed on the webs of plate girders at all bearing locations and at all locations supporting concentrated loads.

Bearing stiffeners shall consist of one or more plates or angles welded or bolted to both sides of the web. The connections to the web shall be designed to transmit the full bearing force due to the factored loads.

The stiffeners shall extend to the full depth of the web and, as closely as practical, to the outer edges of the flanges.

Each stiffener shall be either milled to bear against the flange through which it receives its reaction or attached to that flange by a full penetration groove weld.

E6.10.8.2.2P Projecting Width

The width, b_t, of each projecting stiffener element shall satisfy:

$$b_t \leq 0.48 t_p \sqrt{\frac{E}{F_{ys}}} \qquad \text{(E6.10.8.2.2P-1)}$$

where:

t_p = thickness of projecting element (mm) {in.}

F_{ys} = specified minimum yield strength of the stiffener (MPa) {ksi}

E6.10.8.2.3P Bearing Resistance

The factored bearing resistance for the fitted ends of bearing stiffeners shall be taken as:

$$\left(R_{sb}\right)_r = \phi_b \left(R_{sb}\right)_n \qquad \text{(E6.10.8.2.3P-1)}$$

in which:

$$\left(R_{sb}\right)_n = 1.4 A_{pn} F_{ys} \qquad \text{(E6.10.8.2.3P-2)}$$

where:

F_{ys} = specified minimum yield strength of the stiffener (MPa) {ksi}

A_{pn} = area of the projecting elements of the stiffener outside of the web-to-flange fillet welds but not beyond the edge of the flange (mm^2) {$in.^2$}

φ_b = resistance factor for bearing specified in A6.5.4.2 and D6.5.4.2

COMMENTARY

CE6.10.8.2.2P

The provision specified in this article is intended to prevent local buckling of the bearing stiffener plates.

CE6.10.8.2.3P

To bring bearing stiffener plates tight against the flanges, part of the stiffener must be clipped to clear the web-to-flange fillet weld. Thus, the area of direct bearing is less than the gross area of the stiffener. The bearing resistance is based on this bearing area and the yield strength of the stiffener.

The specified factored bearing resistance is approximately equivalent to the bearing strength given in AISC (1993). The nominal bearing resistance given by Equation 2 is reduced from the nominal bearing resistance of $1.8A_{pn}F_{ys}$ specified in AISC (1993) to reflect the relative difference in the resistance factors for bearing given in the AISC and AASHTO LRFD Specifications.

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E6.10.8.2.4P Axial Resistance of Bearing Stiffeners

E6.10.8.2.4aP General

The factored axial resistance, P_r, shall be determined as specified in A6.9.2.1. The radius of gyration shall be computed about the mid-thickness of the web and the effective length shall be 0.75D, where D is the web depth.

E6.10.8.2.4bP Effective Section

For stiffeners bolted to the web, the effective column section shall consist of the stiffener elements only.

For stiffeners consisting of two plates welded to the web, the effective column section shall consist of the two stiffener elements, plus a centrally located strip of web extending not more than $9t_w$ on each side of the stiffeners. If more than one pair of stiffeners is used, the effective column section shall consist of all stiffener elements, plus a centrally located strip of web extending not more than $9t_w$ on each side of the outer projecting elements of the group.

The strip of web shall not be included in the effective section at interior supports of continuous span hybrid members if:

$$\frac{F_{yw}}{F_{yf}} < 0.70 \qquad \text{(E6.10.8.2.4bP-1)}$$

where:

F_{yw} = specified minimum yield strength of the web (MPa) {ksi}

F_{yf} = higher of the specified minimum yield strengths of the flanges (MPa) {ksi}

E6.10.8.3P LONGITUDINAL STIFFENERS

E6.10.8.3.1P General

Where required, longitudinal stiffeners should consist of either a plate welded longitudinally to one side of the web, or a bolted angle. Longitudinal stiffeners shall be located vertically on the web such that the section satisfies

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CE6.10.8.2.4aP

A portion of the web is assumed to act in combination with the bearing stiffener plates.

The end restraint against column buckling provided by the flanges allows for the use of a reduced effective length.

The web of hybrid girders is not included in the computation of the radius of gyration because the web may be yielding due to longitudinal flexural stress. At end supports where the moment is 0.0, the web may be included.

CE6.10.8.3.1P

For composite sections in regions of positive flexure, the vertical position of a longitudinal web stiffener, most often located a fixed distance from the compression-flange, relative to D_c changes after the concrete slab hardens. Thus,

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both the constructibility criterion specified in Equation E6.10.3.2.2P-1 and the requirements for the strength limit state. The flexural stress in the longitudinal stiffener due to the factored loads shall not exceed $\varphi_f F_{ys}$, where φ_f is the resistance factor for flexure specified in Articles A6.5.4.2 and D6.5.4.2 and F_{ys} is the specified minimum yield strength of the stiffener.

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the computed web bend-buckling resistance is different before and after the slab hardens. As a result, an investigation of several trial locations of the stiffener may be necessary to determine the optimal location of the stiffener to provide both adequate elastic web bend-buckling resistance for constructibility and adequate web post buckling resistance at the strength limit state along the girder. The following equation may be used to determine an initial trial stiffener location for composite sections in regions of positive flexure:

$$\frac{d_s}{D_c} = \frac{1}{1+1.5\sqrt{\dfrac{f_{DC}1+f_{DC}2+f_{DW}+f_{LL+IM}}{f_{DC}1}}} \quad \text{(CE6.10.8.3.1P-1)}$$

where:

d_s = distance from the centerline of a plate longitudinal stiffener, or the gage line of an angle longitudinal stiffener, to the inner surface of leg of the compression-flange element (mm) {in.}

D_c = depth of the web of the steel section in compression in the elastic range (mm) {in.}

f_{xx} = the various compression-flange flexural stresses caused by the different factored loads at the section with the maximum compressive flexural stress, i.e., DC1, the component dead load acting on the noncomposite section; DC2, the component dead load acting on the long-term composite section; DW, the wearing surface load; and LL+IM; acting on their respective sections (MPa) {ksi}

For composite sections in regions of negative flexure and for noncomposite sections, it is suggested that an initial trial stiffener location of $2D_c/5$ from the inner surface of the compression-flange be examined, where D_c is the depth of the web in compression at the section with the maximum flexural compressive stress due to the factored loads. Furthermore, for composite sections in regions of negative flexure, it is suggested that D_c be computed for the section consisting of the steel girder plus the longitudinal reinforcement since the distance between the neutral-axis locations for the steel and composite sections is typically not large in regions of negative flexure. Theoretical and experimental studies on noncomposite girders have indicated that the optimum location of one longitudinal stiffener is $2D_c/5$ for bending and D/2 for shear. Tests have also shown that longitudinal stiffeners located at $2D_c/5$ on these sections can effectively control lateral web deflections under flexure (Cooper 1967). The distance $2D_c/5$ is recommended because shear is always accompanied by

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moment and because a properly proportioned longitudinal stiffener reduces lateral web deflections caused by shear. Also, because D_c may vary along the length of the span, it is recommended that the stiffener be located based on D_c computed at the section with the largest compressive flexural stress. Thus, the stiffener may not be located at its optimum location at other sections with a lower stress and a different D_c. These sections should also be examined to ensure that they satisfy the specified limit states.

In regions where the web undergoes stress reversal, it may be necessary, or desirable, to use two longitudinal stiffeners on the web. Alternately, it may be possible to place one stiffener on the web such that the limit states are adequately satisfied with either edge of the web in compression.

Longitudinal stiffeners placed on the opposite side of the web from transverse intermediate stiffeners are preferred. At bearing stiffeners and connection plates where the longitudinal stiffener and transverse web elements must intersect, the Engineer may discontinue either the longitudinal stiffener or the transverse web element. However, the discontinued element should be fitted and attached to both sides of the continuous element with connections sufficient to develop the flexural and axial resistance of the discontinued element. Preferably, the longitudinal stiffeners should be made continuous. Should the longitudinal stiffener be interrupted and not be attached to the transverse web element, its area should not be included when calculating section properties. All interruptions must be carefully designed with respect to fatigue. For various stiffener end details and their associated fatigue details see (Schilling 1986). Copes should always be provided to avoid intersecting welds.

Longitudinal stiffeners should not be located in yielded portions of the web of hybrid sections. Longitudinal stiffeners are subject to the same flexural stress as the web at their vertical location on the web and must have sufficient rigidity and strength to resist bend buckling of the web. Thus, yielding of the stiffeners should not be permitted on either hybrid or nonhybrid sections.

E6.10.8.3.2P Projecting Width

The projecting width, b_ℓ, of the stiffener shall satisfy:

$$b_\ell \leq 0.48 t_s \sqrt{\frac{E}{F_{ys}}} \qquad \text{(E6.10.8.3.2P-1)}$$

where:

t_s = thickness of stiffener (mm) {in.}

F_{ys} = specified minimum yield strength of the stiffener (MPa) {ksi}

CE6.10.8.3.2P

This requirement is intended to prevent local buckling of the longitudinal stiffener.

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E6.10.8.3.3P Moment of Inertia

The section properties of the stiffener shall be based on an effective section consisting of the stiffener and a centrally located strip of the web not exceeding $18t_w$.

Longitudinal stiffeners shall satisfy:

$$I_\ell \geq D t_w^3 \left[2.4 \left(\frac{d_o}{D} \right)^2 - 0.13 \right] \qquad \text{(E6.10.8.3.3P-1)}$$

$$r \geq 0.234\, d_o \sqrt{\frac{F_{ys}}{E}} \qquad \text{(E6.10.8.3.3P-2)}$$

where:

I_ℓ = moment of inertia of the longitudinal stiffener and the web strip about the edge in contact with the web (mm^4) {$in.^4$}

r = radius of gyration of the longitudinal stiffener and the web strip about the edge in contact with the web (mm) {in.}

D = web depth (mm) {in.}

d_o = transverse stiffener spacing (mm) {in.}

t_w = web thickness (mm) {in.}

F_{ys} = specified minimum yield strength of the stiffener (MPa) {ksi}

E6.10.8.4P STIFFENERS IN RIGID-FRAME KNEES

E6.10.8.4.1P Stiffener Spacing

The spacing of stiffeners in rigid-frame knees shall satisfy both of the following equations:

$$f_a \leq F_{yc} - f_{cs} \qquad \text{(6.10.8.4.1P-1)}$$

and

$$f_b \leq F_{yc} \qquad \text{(6.10.8.4.1P-2)}$$

for which:

$$f_a = f_{cs} \left(\frac{3b^2}{Rt} \right) \left(\frac{\beta^2}{4 + 1.14\beta^4} \right) \qquad \text{(E6.10.8.4.1P-3)}$$

COMMENTARY

CE6.10.8.3.3P

The moment of inertia requirement is to ensure that the stiffener will have adequate rigidity to force a horizontal line of nil deflection in the web panel. The radius of gyration requirement is to ensure that the longitudinal stiffener will be rigid enough to withstand the axial compressive stress without lateral buckling. A partially restrained end condition is assumed for the stiffener acting as a column. It is also assumed in the development of Equation 2 that the eccentricity of the load and initial out-of-straightness cause a 20 percent increase in stress in the stiffener.

A longitudinal stiffener meeting the requirements of Articles E6.10.8.3.2P and E6.10.8.3.3P will have sufficient area to anchor the tension field. Therefore, no additional area requirement is given for longitudinal stiffeners.

SPECIFICATIONS COMMENTARY

$$f_b = f_{cs}\left(\frac{3b^2}{Rt}\right)\left(\frac{\beta^3}{3.2+\beta^3}\right) \qquad \text{(E6.10.8.4.1P-4)}$$

$$\beta = \frac{a}{b} \qquad \text{(E6.10.8.4.1P-5)}$$

where:

a = spacing of stiffness (mm) {in.}

b = half of flange width (mm) {in.}

F_{yc} = specified minimum yield strength of a compression flange (MPa) {ksi}

f_{cs} = maximum compression Service I load flange stress (MPa) {ksi}

R = radius of flange curvature (mm) {in.}

t = thickness of flange (mm) {in.}

SPECIFICATIONS COMMENTARY

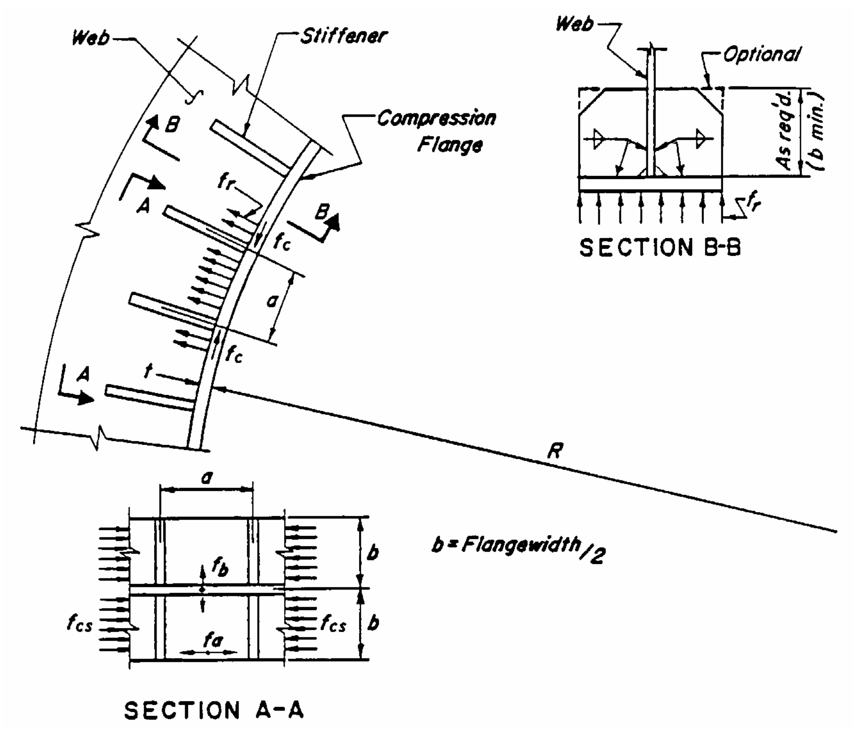

Figure E6.10.8.4.1P-1 - Stiffeners in Rigid-Frame Knees

E6.10.8.4.2P Stiffener Design

The factored bearing resistance of stiffeners in rigid-frame knees, taken as specified in A6.10.8.2.3, shall be greater than P_b, taken as:

$$P_b = f_r ab \qquad \text{(E6.10.8.4.2P-1)}$$

for which:

$$f_r = \frac{f_c t}{R} \qquad \text{(E6.10.8.4.2P-2)}$$

where:

a = spacing of stiffeners (mm) {in.}

b = half of flange width (mm) {in.}

f_c = maximum factored compression flange stress (MPa) {ksi}

R = radius of flange curvature (mm) {in.}

t = thickness of flange (mm) {in.}

SPECIFICATIONS COMMENTARY

E6.10.9P Cover Plates

E6.10.9.1P GENERAL

The length of any cover plate, L_{cp}, in mm {ft.}, added to a member shall satisfy:

Metric Units:

$$L_{cp} \geq 2\,d_s + 900 \qquad \text{(E6.10.9.1P-1)}$$

U.S. Customary Units:

$$L_{cp} \geq 2\,d_s + 3.0$$

where:

d_s = depth of the steel section (mm) {in.}

Partial length welded cover plates shall not be used on flanges more than 20 mm {0.8 in.} thick for nonredundant load path structures subjected to repetitive loadings that produce tension or reversal of stress in the flange.

The maximum thickness of a single cover plate on a flange shall not be greater than two times the thickness of the flange to which the cover plate is attached. Multiple welded cover plates shall not be permitted.

Cover plates may be either wider or narrower than the flange to which they are attached.

E6.10.9.2P END REQUIREMENTS

E6.10.9.2.1P General

The theoretical end of the cover plate shall be taken as the section where the moment, M_u, or flexural stress, F_u, due to the factored loads equals the factored flexural resistance, M_r or F_r. The cover plate shall be extended beyond the theoretical end far enough so that:

- The stress range at the actual end satisfies the appropriate fatigue requirements specified in Articles A6.6.1.2 and D6.6.1.2, and
- The longitudinal force in the cover plate due to the factored loads at the theoretical end can be developed by welds and/or bolts placed between the theoretical and actual ends.

The width at ends of tapered cover plates shall not be less than 75 mm {3.0 in.}.

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E6.10.9.2.2P Welded Ends

The welds connecting the cover plate to the flange between the theoretical and actual ends shall be adequate to develop the computed force in the cover plate at the theoretical end.

Where cover plates are wider than the flange, welds shall not be wrapped around the ends of the cover plate.

E6.10.9.3P COVER PLATE LENGTH AND WIDTH

The length of any welded cover plate added to a rolled beam shall extend the full-length of the rolled beam, including the bearing area, or the full-length of the rolled beam field section in the case of a spliced beam unless otherwise approved by the Chief Bridge Engineer. The use of partial length cover plates is allowed for rehabilitation projects with detailed fatigue analysis. Partial length cover plates must be a bolted connection at the ends.

The width of the plate shall not exceed the width of the flange by 150 mm {6 in.}, or six times the thickness of the cover plate, whichever is less. Bottom flange cover plates preferably shall be wider than the bottom flange. Top flange cover plates shall be of constant width, preferably narrower than the top flange. When a cover plate narrower than the flange is used, the width of the plate shall be at least 50 mm {2 in.} less than the width of the flange. The width of a cover plate connected by fillet welds shall be no greater than 24 times the plate thickness.

COMMENTARY

CE6.10.9.3P

The Department does not allow partial length cover plates for new designs.

E6.10.10P Inelastic Analysis Procedures

Inelastic analysis procedures are not permitted in Pennsylvania.

C6.10.10P

The saving of steel material by using inelastic analysis procedures is minimal when compared to the probability of heavy deck cracking, concerns about long-term performance, localized buckling (plastic type) and potential problems for future bridge redecking.

E6.11P BOX SECTIONS IN FLEXURE

E6.11.1P General

The provisions of these articles may be applied to straight steel multiple or single box sections composite with a concrete deck and symmetrical about the vertical axis in the plane of the web and satisfy the limitations specified in Articles E6.11.1.1.1P and E6.11.1.2.1P.

Box sections shall be designed for:

- Strength limit state according to the provisions

CE6.11.1P

The provisions for box sections are directly applicable to straight bridges, either right or with moderate skew. In the case of bridges with large skew, additional torsional effects may occur in the girders and the lateral distribution of loads may also be affected. In these cases, a more rigorous analysis of stresses is necessary. Box section webs may be vertical or inclined. Inclined webs are advantageous in reducing the width of the bottom flange.

Comprehensive information regarding the design of

SPECIFICATIONS

specified in E6.11.2P,

- Fatigue requirements for webs according to the provisions specified in E6.10.6P,

- Constructibility according to the provisions specified in E6.11.5P,

- Other applicable limit states specified in Articles A6.5 and D6.5,

- Service limit state control of permanent deflections as specified in E6.11.7.

Web plates on box sections may be either perpendicular to the bottom flange or inclined to it. The inclination of web plates shall not exceed 1 to 4.

Doors for exterior access holes should be hinged and provided with locks. All openings in box sections should be screened to exclude animals and pigeons. Air vent holes should be provided in the inside web. The interior of painted box sections should be painted a light color.

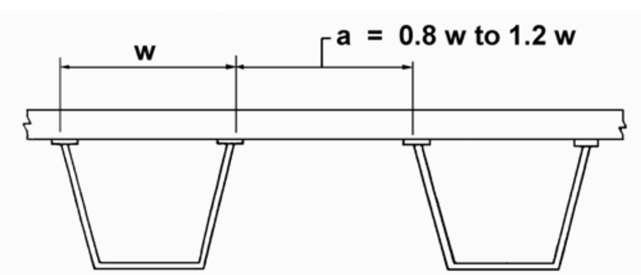
E6.11.1.1P MULTIPLE BOX SECTIONS

E6.11.1.1.1P General

The provisions of E6.11 shall apply to the design of simple and continuous bridges of spans up to 100 000 mm, supported by two or more single-cell composite box sections in a cross-section that complies with the geometric restrictions specified herein.

The distance center-to-center of flanges of adjacent boxes, a, taken at the midspan, shall neither be greater than 120 percent, nor less than 80 percent, of the distance center-to-center of the flanges of each adjacent box, w, as illustrated in Figure 1. In addition to the midspan requirement, where nonparallel box sections are used, the distance center-to-center of adjacent flanges at supports shall neither be greater than 135 percent, nor less than 65 percent of the distance center-to-center of the flanges of each adjacent box. The distance center-to-center of flanges of each individual box shall be the same.

Stiffness for analysis purposes shall be based on uncracked composite properties.



COMMENTARY

steel box girder bridges is contained in FHWA (1980).

For a general overview on box girder bridges, see Wolchuk (1990).

Painting the interior of box sections is primarily done to facilitate inspections. Therefore, the paint quality need not match that normally used for exterior surfaces.

CE6.11.1.1.1P

When box sections are subjected to eccentric loads, their cross-section becomes distorted, giving rise to secondary bending stresses. Loading the opposite side of the bridge produces reversal of stress, and, therefore, possible fatigue effects. The maximum stresses and stress ranges occur in the center girder of those bridges with an odd number of girders.

Limitations specified in this article are necessary because the provisions concerning lateral distribution of loads, secondary distortional bending stresses, and the effectiveness of the bottom flange plate are based on an extensive study of multiple box girder bridges that conform to these limitations. This study utilized uncracked stiffness (Johnston and Mattock 1967). Bridges that do not conform should be investigated using one of the available methods of refined structural analysis.

Some limitations are placed on the variation of distance a with respect to distance w because the studies on which some of the provisions are based were made on bridges in which "w" and "a" were equal. The limitations given for nonparallel box sections will allow some flexibility of layout in design while generally maintaining the validity of the provisions.

Several of the subsequent articles incorporate simplifying assumptions and simplified expressions whose validity has only been demonstrated for the type of bridge defined in E6.11.1.1.1.

Distortional stresses and stress ranges and local plate

SPECIFICATIONS

Figure E6.11.1.1.1P-1 - Center-to-center Flange Distance

The cantilever overhang of the deck slab, including curb and parapet, shall not be greater than either 60 percent of the average distance between the centers of top steel flanges of adjacent box sections, a, or 1800 mm.

The provisions of E6.10.3.7P shall apply, except that shear connectors shall be provided in negative bending regions of box sections. The provisions of E6.10.3.6P shall apply.

E6.11.1.1.2P Live Load Distribution

For multiple box sections, the live load flexural moment for each box shall be determined in accordance with the provisions of D4.6.2.2.2b.

E6.11.1.2P SINGLE BOX GIRDERS

E6.11.1.2.1P General

These Specifications shall not be applied to multiple cell single box sections.

The box section shall be positioned in a central position with respect to the cross-section, and the center of gravity of the dead load shall be as close to the shear center of the box as is practical.

The top of the box may be either open or closed with steel plate. Flange-to-web welds shall comply with the provisions of E6.11.4. For loads applied prior to curing of the deck concrete, the plate in positive moment regions shall be designed according to the provisions for compression flanges specified in E6.11.2.1.3aP. The shear connection between the top plate and concrete deck shall be designed for interface shear resulting from all applicable loads. Buckling of this plate during or prior to placement of the concrete deck shall be considered, and stiffening may be used when required.

For single box sections, structural steel in tension shall be considered fracture-critical, unless analysis shows that the section can support the full dead and live load after sustaining a complete fracture of the tensile steel at any point.

E6.11.1.2.2P Analysis

The spine beam analogy of A4.6.1.2.2 may be used for analysis of single box girders. Both torsional and flexural effects shall be considered. The box may not be considered torsionally rigid unless internal bracing is adequate to maintain the box cross-section. The transverse position of

COMMENTARY

vibration stresses in bridges having proportions corresponding to the specified limitations need not be considered in design.

The requirement that shear connectors be provided in negative moment regions of multiple box girders is necessary to be consistent with the prototype and model bridges that were studied in the original development of the live load distribution provisions for box sections.

CE6.11.1.2.1P

Placing the dead loads near the shear center ensures minimal torsion. Items, such as sound barriers, on one side of the bridge may be critical on single box bridges. Haunched girders with inclined webs are permitted. If the bridge is to be launched, a constant depth box is recommended.

There may be exceptions, such as top flanges in negative moment regions where there is adequate deck reinforcing to act as a top flange, in which case the section need not be considered fracture-critical. In such cases, adequate shear connection must be provided.

CE6.11.1.2.2P

Significant torsional loads may occur during construction and under live load. Live loads at extremes of the deck can cause critical torsional loads without causing critical vertical moments. Live load positioning should be done for flexure and torsion. The position of the bearing

SPECIFICATIONS

the bearings shall be considered in the analysis.

Longitudinal warping stresses shall be considered for fatigue but may be ignored at the strength limit state.

If distortion is adequately prevented, the St. Venant torsional inertia, J, for a box section may be determined as:

$$J = 4 \frac{A_o^2}{\Sigma \frac{b}{t}} \qquad \text{(E6.11.1.2.2P-1)}$$

where:

A_o = area enclosed by the box section (mm^2) {$in.^2$}

b = width of rectangular plate element (mm) {in.}

t = thickness of plate (mm) {in.}

E6.11.1.2.3P Bearings

Bearings for single box sections shall be placed in pairs at supports where practical. Double bearings may be placed either inboard or outboard of the box section webs. If single bearings narrower than the bottom flange are used, they shall be aligned with the shear center of the box, and other supports shall have adequate bearings to ensure against overturning under any load combination. Bearings should be oriented at right angles with respect to the longitudinal axis of the girder. If tie-down bearings are used, the resulting force effects shall be considered in the design.

E6.11.2P Strength Limit State For Box Sections

E6.11.2.1P FLEXURE

E6.11.2.1.1P Factored Flexural Resistance

The factored flexural resistance of box sections in terms of moment and stress shall be taken as:

$$M_r = \phi_f M_n \qquad \text{(E6.11.2.1.1P-1)}$$

COMMENTARY

should be recognized in the analysis in sufficient completeness to permit direct computation of the reactions.

Warping stresses are largest in the corners of the box and should be considered for fatigue (Wright and Abdel-Samad 1968). Tests have indicated that warping stress does not affect the ultimate strength of box girders of typical proportions. The warping constant for a closed box section is approximately equal to 0.0. If the box is extremely wide with respect to the span, a special investigation may be required.

CE6.11.1.2.3P

Placement of bearings is critical on single box sections. Skewed bearings are apt to be difficult to construct. Placing bearings outboard of the box reduces overturning loads on the bearings and may eliminate uplift.

SPECIFICATIONS

$$F_r = \phi_f F_n \qquad \text{(E6.11.2.1.1P-2)}$$

where:

φ_f = resistance factor for flexure specified in Articles A6.5.4.2 and D6.5.4.2

M_n = nominal resistance specified in E6.11.2.1.2aP (N-mm) {k-in}

F_n = nominal flexural resistance specified in E6.11.2.1.2P or E6.11.2.1.3P (MPa) {ksi}

The effective section of flexural members with holes in the tension flange shall be determined as specified in E6.10.3.6P.

E6.11.2.1.2P Positive Flexure

E6.11.2.1.2aP Nominal Flexural Resistance

The nominal flexural resistance for multiple box sections without holes in the tension flange shall be determined as specified in E6.10.4.2.2aP, subject to the requirements specified in Articles E6.10.4.1.2P and E6.10.4.2.2bP. The nominal flexural resistance for multiple box sections with holes in the tension flange shall be determined as specified in E6.10.4.2.4P, subject to the requirements specified in Articles E6.10.2.2P.

The nominal flexural resistance for the top flange of single box sections shall be determined as specified in E6.10.4.2.4P, subject to the requirements specified in E6.10.2.2P. The nominal flexural resistance for the bottom flange of single box sections shall be determined as:

$$F_n = R_b R_h F_{yf} \sqrt{1 - 3\left(\frac{f_v}{F_{yf}}\right)^2} \qquad \text{(E6.11.2.1.2aP-1)}$$

where:

F_{yf} = specified minimum yield strength of the flange (MPa) {ksi}

R_b,
R_h = flange stress reduction factors specified in E6.10.4.3P

f_v = maximum St. Venant torsional shear stress in the flange plate due to the factored loads (MPa) {ksi} determined as:

COMMENTARY

CE6.11.2.1.2aP

The tensile strength of the bottom flange of single box sections is affected by the torsional shear stress. The von Mises yield criterion (Boresi et al. 1978) is used to consider the effect of shear stress. The combined effect of torsional shear and flexure are difficult to determine, but the worst case of either may be added to obtain a conservative estimate.

At sections of flexural members with holes in the tension flange, it has not been fully documented that complete plastification of the cross-section can be achieved prior to fracture on the net section of the flange.

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$$= \frac{T}{2A_o t}$$

T = internal torque resulting from the factored loads (N-mm) {k-in}

A_o = enclosed area within the box section (mm^2) {$in.^2$}

t = plate thickness (mm) {in.}

The shear and flexural stresses on the single box due to the factored loads shall be computed at the same longitudinal location in the box flange.

The compression flange bracing requirements specified in E6.11.2.1.2bP shall also apply to multiple and single box sections.

For simple spans, bottom flanges of multiple and single box sections shall be considered fully effective in resisting flexure if the width of the flange does not exceed one-fifth the span length. If the flange width exceeds one-fifth of the span, only a width equal to the one-fifth of the span shall be considered effective in resisting bending. For continuous spans, this requirement shall be applied to the distance between points of permanent load contraflexure.

Stress analyses of actual box girder bridge designs were carried out to evaluate the effective width using a series of folded plate equations (Goldberg and Leve 1957). Bridges for which the span-to-flange width ratio varied from 5.65 to 35.3 were included in the study. The effective flange width as a ratio of the total flange width covered a range of from 0.89 for the bridge with the smallest span-to-width ratio to 0.99 for the bridge with the largest span-to-width ratio. On this basis, it is reasonable to permit the flange plate to be considered fully effective, provided that its width does not exceed one-fifth of the span of the bridge.

Although the results quoted above were obtained for simply supported bridges, this criterion would apply equally to continuous bridges, using the equivalent span, i.e., the distance between points of permanent load contraflexure over the internal support.

The effective flange width is used to calculate the flexural stress in the flange. The full flange width should be used to calculate the nominal flexural resistance of the flange.

E6.11.2.1.2bP Compression Flange Bracing

The compression flange of multiple or single box sections in positive flexure need not be subject to bracing requirements when investigating the strength limit state. The need for temporary or permanent bracing of the compression flange to maintain the box section geometry throughout all phases of construction, including sequential placement of the concrete deck, shall be considered as specified in E6.11.5.1P.

E6.11.2.1.3P Negative Flexure

The provisions of this article apply to stiffened and unstiffened compression flanges and to tension flanges.

EC6.11.2.1.3P

There are no specific requirements for compression flange bracing at negative bending sections of box sections

SPECIFICATIONS

COMMENTARY

for the strength limit state.

E6.11.2.1.3aP Nominal Flexural Resistance

For multiple and single box sections, the nominal flexural resistance shall be determined as specified herein.

For compression flanges with longitudinal stiffeners, the nominal flexural resistance shall be taken as:

If $\frac{w}{t_f} \leq 0.57\sqrt{\frac{kE}{F_{yc}}}$,

then:

$$F_n = R_b R_h F_{yc} \qquad \text{(E6.11.2.1.3aP-1)}$$

If $0.57\sqrt{\frac{kE}{F_{yc}}} < \frac{w}{t_f} \leq 1.23\sqrt{\frac{kE}{F_{yc}}}$, then:

$$F_n = 0.592\ R_b R_h F_{yc}\left(1 + 0.687 \sin\frac{c\pi}{2}\right) \qquad \text{(E6.11.2.1.3aP-2)}$$

If $\frac{w}{t_f} > 1.23\sqrt{\frac{kE}{F_{yc}}}$,

then:

Metric Units:

$$F_n = 181\,000\ R_b R_h\ k\left(\frac{t_f}{w}\right)^2 \qquad \text{(E6.11.2.1.3aP-3)}$$

U.S. Customary Units:

$$F_n = 26,200\ R_b R_h\ k\left(\frac{t_f}{w}\right)^2 \qquad \text{(E6.11.2.1.3aP-3)}$$

for which:

$$c = \frac{1.23 - \frac{w}{t_f}\sqrt{\frac{F_{yc}}{kE}}}{0.66} \qquad \text{(E6.11.2.1.3aP-4)}$$

K = buckling coefficient specified as:

- If n = 1, then:

CE6.11.2.1.3aP

The provisions for compression flanges with longitudinal stiffeners only are based on the theory of elastic stability (Timoshenko and Gere 1961). The provisions are formulated in such a way that, when more than one longitudinal stiffener is used, the necessary stiffener stiffness can be directly calculated that will result in behavior corresponding to a selected value of the buckling coefficient k. When only one longitudinal stiffener is used, the minimum stiffness specified will result in behavior corresponding to a plate buckling coefficient, k, of 4.

No provisions are included for the design of bottom flange plates for a combination of compression and of shear due to torsion of the girders. This arises from the results obtained in the analytical study of straight bridges of the type covered by these provisions. It was found that when such bridges were loaded so as to produce maximum moment in a particular girder, and hence maximum compression in the flange plate near an intermediate support, the amount of twist in that girder was negligible. It therefore appears reasonable that, for bridges conforming to the limitations set out in these provisions, shear due to torsion need not be considered in the design of the bottom flange plates for maximum compression loads.

For bridges whose proportions do not conform to the limitations of these provisions, further study of the state of stress in the bottom flange should be made (FHWA 1980). A general discussion of the problem of reduction of critical buckling stresses due to the presence of torsional shear may be found in Johnston (1966).

SPECIFICATIONS COMMENTARY

$$k = \left(\frac{8I_s}{wt_f^3}\right)^{\frac{1}{3}} \le 4.0 \quad \text{(E6.11.2.1.3aP-5)}$$

- If n = 2, 3, 4 or 5, then:

$$k = \left(\frac{14.3I_s}{wt_f^3 n^4}\right)^{\frac{1}{3}} \le 4.0 \quad \text{(6.11.2.1.3aP-6)}$$

where:

F_{yc} = specified minimum yield strength of the compression flange (MPa) {ksi}

w = larger of the width of compression flange between longitudinal stiffeners or the distance from a web to the nearest longitudinal stiffener (mm) {in.}

t_f = thickness of the compression flange (mm) {in.}

R_b, R_h = flange stress reduction factors specified in E6.10.4.3P

n = number of equally spaced longitudinal compression flange stiffeners

I_s = moment of inertia of a longitudinal stiffener about an axis parallel to the bottom flange and taken at the base of the stiffener (mm^4) {$in.^4$}

The longitudinal stiffeners shall satisfy the requirements specified in E6.11.3.2.1P.

For compression flanges without longitudinal stiffeners, the nominal bending resistance shall be taken as equal to the nominal bending resistance for the compression flange with longitudinal stiffeners, with the compression flange width between webs, b, substituted for w, and the buckling coefficient k taken as 4.

For tension flanges, nominal flexural resistance shall be taken as:

$$F_n = R_b R_h F_{yt} \quad \text{(E6.11.2.1.3aP-7)}$$

where:

F_{yt} = specified minimum yield strength of the tension flange (MPa) {ksi}

E6.11.2.1.3b Web Slenderness

The provisions of E6.10.2.2P shall apply.

SPECIFICATIONS

E6.11.2.2P SHEAR

E6.11.2.2.1P General

The factored shear resistance of box girders, V_r, shall be taken as:

$$V_r = \varphi_v V_n \qquad \text{(E6.11.2.2.1P-1)}$$

where:

φ_v = resistance factor for shear specified in Articles A6.5.4.2 and D6.5.4.2

V_n = nominal shear resistance for one web specified in E6.10.7P, except as modified herein (N) {kip}

For the case of inclined webs, D in E6.10.7P shall be taken as the depth of the web plate measured along the slope. Each web shall be designed for shear, V_{ui}, due to the factored loads taken as:

$$V_{ui} = \frac{V_u}{\cos\theta} \qquad \text{(E6.11.2.2.1P-2)}$$

where:

V_{ui} = shear due to the factored loads on one inclined web (N) {kip}

θ = the angle of inclination of the web plate to the vertical (DEG)

For single box sections, either the absolute value of the maximum flexural and torsional shears may be added together or extreme shear resulting from concurrent flexure and torsion may be used.

E6.11.2.2.2P Shear Connectors

Shear connectors for straight box sections shall be designed according to the provisions specified in E6.10.7.4P. Shear connectors shall be provided in negative flexure regions.

For single box sections, shear connectors shall be designed for shear caused by flexure and torsion. The entire concrete deck shall be considered effective in computing shear due to bending. The total design shear may be determined as either the vector sum of the shears or the sum of their absolute values.

COMMENTARY

CE6.11.2.2.1P

For multiple box sections, one-half the distribution factor for moment should be used in the calculation of the live load vertical shear in each box section web.

For single box sections, web inclination can be treated the same as for multiple box sections, except that the shears caused by torsion and flexure have to be combined.

CE6.11.2.2.2P

For purpose of calculating interface shear between the deck and girder, the entire deck is considered effective in the composite section to ensure that adequate shear connection is available.

All test specimens in the test program that formed the basis of these provisions had stud connectors throughout the negative flexure region.

SPECIFICATIONS

E6.11.3P Stiffeners

E6.11.3.1P Web Stiffeners

All transverse intermediate web stiffeners for multiple and single box sections shall be designed according to the provisions of E6.10.8.1P.

Longitudinal web stiffeners for box sections shall be designed according to the provisions of E6.10.8.3P.

All bearing stiffeners for multiple and single box sections shall be designed according to the provisions of E6.10.8.2P.

E6.11.3.2P COMPRESSION FLANGE STIFFENERS

E6.11.3.2.1P Longitudinal Stiffeners

Longitudinal compression flange stiffeners for multiple and single box sections shall be equally spaced across the compression flange width. The projecting width, b_ℓ, of the stiffener shall satisfy:

$$b_\ell \le 0.48 t_p \sqrt{\frac{E}{F_{yc}}} \quad \text{(E6.11.3.2.1P-1)}$$

where:

t_p = thickness of stiffener (mm) {in.}

F_{yc} = specified minimum yield strength of the compression flange (MPa) {ksi}

The moment of inertia, I_ℓ, of each stiffener about an axis parallel to the flange and taken at the base of the stiffener shall satisfy:

$$I_\ell \geq \Psi w t_f^3 \quad \text{(6.11.3.2.1P-2)}$$

where:

Ψ = $0.125k^3$ where n = 1
= $0.07k^3n^4$ where n = 2, 3, 4 or 5

n = number of equally spaced longitudinal compression flange stiffeners

w = larger of the width of compression flange between longitudinal stiffeners or the distance from a web to the nearest longitudinal stiffener (mm) {in.}

COMMENTARY

CE6.11.3.2.1P

The equation for the required longitudinal stiffener inertia, I_ℓ, is an approximate expression that within its range of applicability yields values close to those obtained by use of the exact but cumbersome equations of elastic stability (Timoshenko and Gere 1961).The number of longitudinal flange stiffeners, n, should preferably not exceed 2. Equation 2 assumes that the bottom flange plate and the stiffeners are infinitely long and ignores the effect of any transverse bracing or stiffening. Thus, when n exceeds 2, the required moment of inertia from Equation 2 increases dramatically so as to become nearly impractical. For designs where an exceptionally wide box flange is required and n may exceed 2, it is suggested that additional transverse flange stiffeners be provided to reduce the required size of the longitudinal stiffeners to a more practical value. Provisions for the design of box flanges stiffened both longitudinally and transversely, which can be modified for use with Load and Resistance Factor Design, are given in the Allowable Stress Design portion of the AASHTO Standard Specifications (2002). Included are requirements related to the necessary spacing and stiffness of the transverse stiffeners. The bottom strut of the transverse interior bracing in the box can be considered to act as a transverse flange stiffener for this purpose if the strut satisfies the applicable stiffness requirements.

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t_f = compression flange thickness (mm) {in.}

k = buckling coefficient specified in E6.11.2.1.3aP

E6.11.3.2.2P Transverse Stiffeners

If used, transverse stiffeners on flanges should have a size equal to that of a longitudinal stiffener. The projecting width of the stiffener shall satisfy the requirements specified in E6.11.3.2.1P.

COMMENTARY

CE6.11.3.2.2P

When longitudinal compression flange stiffeners are used, it is preferable to have at least one transverse stiffener placed on the compression flange near the point of permanent load contraflexure. If the design is predicated on use of both longitudinal and transverse stiffeners, the state of stress in the bottom flange should be investigated. A comprehensive discussion on box girders is contained in SSRC (1988) and FHWA (1980).

E6.11.4P Flange-to-Web Connections

Except as specified herein, the total effective thickness of the flange-to-web welds shall not be less than the thickness of the web. Where two or more intermediate diaphragms in a span are provided, the weld size shall not be less than the size consistent with the requirements of A6.13.3.4. If fillet welds are used, they shall be placed on both sides of the connecting flange or web plate.

CE6.11.4P

If at least two intermediate diaphragms are not provided in each span, it is essential that the web flange welds be of sufficient size to develop the full web section because of the possibility of secondary flexural stresses developing in box sections as a result of vibrations and/or distortions in the section. In Haaijer (1981), it was demonstrated that the transverse secondary distortional stress range at the web-to-flange welded joint is reduced more than 50 percent when one interior intermediate cross-frame per span is introduced and more than 80 percent when two cross-frames per span are introduced. Thus, if two or more interior intermediate diaphragms or cross-frames are used, the minimum size fillet welds on both sides of the web may be assumed to be adequate. It is essential that welds be deposited on both sides of the connecting flange or web plate whether full penetration or fillet welds are used. This will reduce the bending stresses resulting from the transverse bending moments to a minimum and eliminate the possibility of fatigue failure.

E6.11.5P Constructibility

E6.11.5.1P GENERAL

Box section members shall be investigated for strength and stability during construction, including sequential deck placement. The individual box section geometry shall be maintained throughout all phases of construction, including placement of the concrete deck. The need for temporary or permanent intermediate interior diaphragms or cross-frames, exterior diaphragms or cross-frames, top lateral

CE6.11.5.1P

The Designer should consider possible eccentric loads that may occur during construction. These may include uneven placement of concrete and various equipment. Temporary diaphragms or cross-frames that are not part of the original design should be removed because the structural behavior of the box section, including load distribution, may be significantly affected if they are left in place. Additional information on construction of composite box sections may

SPECIFICATIONS

bracing, or other means shall be investigated to ensure that deformations of the box section are adequately controlled during fabrication, erection, and placement of the concrete deck. Prior to curing of the concrete deck, top flanges of trough-type box sections shall be considered to be braced only at points where internal cross-frames or top lateral bracing are attached. Forces resulting from connections of overhang deck forms to the top flange and/or the web should be considered.

E6.11.5.2P WEBS

The webs of box sections shall satisfy Equation E6.10.3.2.2P-1 when investigating the steel section for deck placement sequence.

E6.11.5.3P COMPRESSION FLANGE

At positive bending sections, the investigation of the compression flange of multiple and single box sections for constructibility shall begin with the noncompact section compression-flange slenderness provisions of E6.10.4.1.4P.

E6.11.5.4P SHEAR

The shear, V_u, due to the factored permanent loads considering the deck placement sequence shall not exceed the factored shear resistance, V_r, taken as:

$$V_r = \varphi_v V_n \qquad (6.11.5.4\text{-}1)$$

where:

V_n = nominal shear resistance specified in E6.10.3.2.3P (N) {kip}

φ_v = resistance factor for shear specified in Articles A6.5.4.2 and D6.5.4.2

E6.11.6P Wind Effects on Exterior Members

The section assumed to resist the horizontal factored wind loading shall consist of the bottom flange acting as a web and 12 times the thickness of the web acting as flanges.

COMMENTARY

be found in Highway Structures Design Handbook (1978) and Steel/Concrete Composite Box-Girder Bridges: A Construction Manual (1978).

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E6.11.7P Service Limit State Control of Permanent Deflections

Load Combination Service II in Articles A3.4.1 and D4.4.1 shall apply. The web shall satisfy Equation E6.10.3.2.2P-1, using the appropriate value of the depth of the web in compression in the elastic range, D_c.

At positive flexural regions of multiple box sections, the flange stresses shall satisfy:

$$f_f \leq 0.95F_{yf} \qquad \text{(E6.11.7P-1)}$$

where:

f_f = elastic flange stress caused by the factored loading (MPa) {ksi}

F_{yf} = yield strength of the flange (MPa) {ksi}

COMMENTARY

CE6.11.7P

This limit state check is intended to prevent objectionable permanent deflections due to expected severe traffic loadings. It affects only serviceability and corresponds to the overload check in the AASHTO Standard Specifications for Highway Bridges, 16th Edition (2002). The development of the overload provisions is described in Vincent (1969). The provision applies only to positive flexural regions of multiple box sections whose nominal bending resistance can exceed the yield strength of the flange at the strength limit state. This check shall not apply to single box sections.

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DESIGN MANUAL

PART 4

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7.4 MATERIALS

7.4.1 General

The following shall supplement A7.4.1.

The use of aluminum as bridge or other structural material is not allowed, except as depicted in Standard Drawings. If aluminum usage is prescribed by the Chief Bridge Engineer, applicable specifications will be determined or established at that time.

Aluminum appurtenances are allowable as per the BC Standard Drawings.

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8.4 MATERIALS

8.4.2 Metal Fasteners and Hardware

8.4.2.2 MINIMUM REQUIREMENTS

8.4.2.2.2 Prestressing Bars

The following shall supplement A8.4.2.2.2.

For additional requirements on prestressing bars see A5.4.4.1 and D5.4.4.1.

8.11 BRACING REQUIREMENTS

8.11.2 Sawn Wood Beams

The following shall replace the third sentence of the first paragraph.

The spacing of intermediate bracing shall be based on lateral stability and load transfer requirements, but shall not exceed 6000 mm {20 ft.}.

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9.2 DEFINITIONS

The following shall supplement A9.2

Overhang Length - The length measured normal to the exterior girder to the edge to the deck.

The following shall replace the skew angle definition in A9.2.

Skew Angle - The angle between the axis of support relative to the longitudinal axis of the bridge, i.e., a 90° skew denotes a rectangular bridge.

9.4 GENERAL DESIGN REQUIREMENTS

9.4.3 Concrete Appurtenances

The following shall replace A9.4.3.

Unless otherwise specified by the Chief Bridge Engineer, concrete curbs, barriers and dividers should be made structurally continuous.

Deflection joints in the concrete portion of the barrier shall be provided by a modified deflection joint detail as per Standard Drawing BC-752M. The spacing of the deflection joints shall be as per Standard Drawing BD-660M. Longitudinal barrier bars shall be continuous through the deflection joints.

COMMENTARY

C9.4.3

The following shall replace AC9.4.3.

Experience indicates that the interruption of concrete appurtenances at locations other than deck joints does not serve the intended purpose of stress relief. Large cracks, only 300 mm {1 ft.} or so away from open joints, have been observed in concrete barriers.

9.5 LIMIT STATES

9.5.1 General

The following shall replace the first paragraph of A9.5.1.

The structural contribution of a concrete appurtenance to the deck will be neglected for all limit states.

C9.5.1

Delete the first paragraph of AC9.5.1.

C9.5.2 Service Limit States

The following shall supplement AC9.5.2.

Before any testing of the deck is begun (in order to determine the limits for excessive deck deformation), the Chief Bridge Engineer must review and approve the testing procedure. The results of this testing must also be approved by the Chief Bridge Engineer.

9.5.5 Extreme Event Limit States

The following shall replace A9.5.5.

Decks shall be designed for force effects transmitted by traffic, barriers and railings using loads, analysis procedures and limit states specified in A13 and D13.

For the barriers shown in the Standard Drawings, the deck and overhang designs given in Standard Drawing BD-601M are designed for the controlling condition.

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9.6 ANALYSIS

9.6.1 Methods of Analysis

The following shall replace A9.6.1.

Approximate elastic methods of analysis, specified in A4.6.2.1, shall be used for the various limit states in A9.5.

If approved by the Chief Bridge Engineer, refined methods, specified in A4.6.3.2, may be used for the various limit states in A9.5.

The empirical design of concrete slabs, specified in A9.7.2, will be used only if prescribed by the Chief Bridge Engineer.

9.7 CONCRETE DECK SLABS

9.7.1 General

9.7.1.1 MINIMUM DEPTH AND COVER

The following shall replace A9.7.1.1.

The minimum depth of concrete deck is 200 mm {8 in.} which includes a 10 mm {1/2 in.} wearing surface. Therefore, the minimum structural depth of concrete deck is 190 mm {7 1/2 in.}.

Minimum cover shall be in accordance with the provisions of D5.12.3.

For composite adjacent box beam superstructures, the deck slab thickness shall be 140 mm {5 1/2 in.} minimum, including a 10 mm {1/2 in.} integral wearing surface with one mat of reinforcement. Slab thickness may need to be increased to provide minimum required cover for bridges made continuous for live load.

When AASHTO Type V or VI beams are placed adjacent, the deck slab thickness shall be 190 mm {7 1/2 in.}, including a 10 mm {1/2 in.} integral wearing surface with two mats of No. 13 {No. 4} reinforcement.

C9.7.1.1

Delete the last paragraph of AC9.7.1.1.

9.7.1.3 SKEWED DECKS

The following shall replace A9.7.1.3.

If the skew angle of the deck is from 90° to 75°, the primary reinforcement shall be placed parallel to the skew. If the skew angle of the deck is less than 75°, the primary reinforcement shall be placed perpendicular to the main supporting components. Standard Drawing BD-660M provides additional information.

C9.7.1.3

The following shall replace AC9.7.1.3.

The intent of this provision is to prevent extensive cracking of the deck, which may be the result of having no appreciable reinforcement acting in the direction of principal flexural stresses due to a heavily skewed reinforcement, as shown in Figure AC9.7.1.3-1.

9.7.1.5 DESIGN OF CANTILEVER SLABS

9.7.1.5.1P Overhang of Deck Slab on Concrete and Steel Girder Bridges

The maximum overhang length shall not exceed either:

C9.7.1.5.1P

The overhang is measured from the centerline of the

SPECIFICATIONS

- the girder depth,
- requirements in A4.6.2.2, or
- requirements given in Table 1 based on the exterior girder deflection.

For overhangs greater than 0.5S, the overhang and girder spacing must be approved by the Chief Bridge Engineer at the type, size and location stage.

Table 9.7.1.5.1P-1 – Exterior Girder Deflection Versus Overhang

Maximum LL+I Deflection Exterior Girder Δ	Maximum Overhang X
$L/800 = \Delta$	0.5 S
$L/1000 < \Delta < L/800$	Equation 1
$\Delta < L/1000$	0.625 S

$$X = S\left\{0.5 + 0.125\left(\frac{0.00125L - \Delta}{0.00025L}\right)\right\} \qquad (9.7.1.5.1P\text{-}1)$$

where:

X = maximum permitted overhang measured normal to the exterior girder from edge of deck to centerline of exterior girder (mm) {ft.}

S = Stringer spacing between the exterior girder centerline and adjacent interior girder centerline. When there is a variable stringer spacing, the stringer spacing is assumed to be spacing between the exterior and interior girders at 1/3 of the span length measured from the narrow end of the stringer spacing (mm) {ft.}

L = span length (mm) {ft.}

Δ = maximum deflection of the exterior girder caused by design live load (excluding permit load), plus impact (mm) {ft.}

Any exception to the above criteria is permitted only with approval of the Chief Bridge Engineer.

The designer shall design and detail one method to stabilize the exterior girder to prevent excessive rotation and/or rollover of the exterior girder. In addition, a note shall be placed on the contract plans alerting contractors that the contractor shall check the need for the temporary

COMMENTARY

girder web for steel I beams. For P/S Box Beams the overhang is measured from the centerline of the web.

Stabilizing the exterior girder of a steel bridge involves bracing the top and bottom flanges of the girder. The stabilization consists of a tension tie connecting the top flanges in conjunction with compression struts bracing the bottom flanges of the girders in the cross section. Typically

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bracing between the exterior girder and adjacent interior girder to prevent exterior girder rotation during the deck placement. The contractor will be required to submit for Department approval the drawings and calculations of the temporary bracing scheme.

The deck slab overhanging the girder adjacent to a longitudinal median joint shall have the same overhang and temporary bracing requirements as provided for the exterior girder.

9.7.1.6P REINFORCEMENT REQUIREMENTS

No transverse reinforcement bars larger than No. 19 {No. 6} shall be used in the deck slab.

Reinforcement in the compression face of the deck slab shall not be considered in the design.

Haunches greater than or equal to 75 mm {3 in.} shall be reinforced according to Standard Drawing BD-601M.

For deck protection guidelines, see D5.4.3.6P.

The minimum spacing of transverse bars shall not be less than 140 mm {5 1/2 in.}.

9.7.1.7P DECK CONSTRUCTION JOINTS

Deck construction joints are permitted as shown on the plan. However, no material from any bulkhead type may remain in place.

9.7.2 Empirical Design

NOTE: An empirical design is only to be used if prescribed by the Chief Bridge Engineer.

9.7.2.4 DESIGN CONDITIONS

The following shall supplement A9.7.2.4.

The bulleted items in A9.7.2.4 are the minimum for an empirical design. For an empirical design to be valid in Pennsylvania, a desk must comply with PennDOT standards and these bulleted items.

COMMENTARY

the bracing of the bottom flanges is constructed with timber struts and the restraint of the top flange is constructed with a structural shape such as an angle or even a reinforcing bar. The bracing and tension ties are typically placed at one-third points between cross frames. The flange tension tie can be embedded in the deck or placed below the deck as long as it does not interfere with the stay-in place deck forms. For top flanges in compression, a welded tension tie attachment to the top flange can be used. Tack welding a reinforcing bar to the stems of shear studs to provide a tension tie is not permitted. This note is required if the criteria in Table D9.7.1.5.1P-1 is violated and the Chief Bridge Engineer has given an exception.

C9.7.1.7P

Stayform sheets have also been used at bridge deck construction joints. This is a undesirable detail if the sheets are left in the deck, since it introduces a formed crack which would be a potential maintenance problem. In addition, corrosion will eventually develop in this joint, even though a galvanized metal sheet is provided. However, this detail may be used if the stayform sheet is removed after hardening of the concrete.

C9.7.2.2 APPLICATION

Delete the second paragraph of AC9.7.2.2.

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9.7.2.5 REINFORCEMENT REQUIREMENTS

The following shall replace the last paragraph of A9.7.2.5.

If the skew is less than 75°, the specified reinforcement in both directions shall be doubled in the end zones of the deck. Each end zone shall be taken as a longitudinal distance equal to the effective length of the slab specified in A9.7.2.3.

COMMENTARY

C9.7.2.5

The following shall replace the last paragraph of AC9.7.2.5.

The intent of this provision is crack control. Beam slab bridges with a skew less than 65° have shown a tendency to develop torsional cracks due to differential deflections in the end zone, OHBDC (1983). The extent of cracking is usually limited to a width that approximates the effective length.

9.7.4 Stay-in-Place Formwork

9.7.4.1 GENERAL

The following shall replace the first bulleted item in the second paragraph of A9.7.4.1.

- 65% of the yield strength of steel, or

The following shall supplement A9.7.4.1.

The use of permanent metal deck forms shall be specified in all superstructure designs, unless it is not feasible. The use of removable forms should be called for on the p7lans as an alternative to the permanent metal deck forms.

Deck slab details and beam and diaphragm haunch details shall conform to details shown on the standards. The overall depth of forms and the support details shall not be shown on the design drawings.

There shall be no pay item for the forms.

If permitted by the Chief Bridge Engineer, prestressed concrete planks which form an integral part of the deck slab may be used in lieu of separate deck forms.

C9.7.4.1

The following shall supplement AC9.7.4.1.

It has been observed during construction that concrete placed by bucket or pumping operation may be 50% to 100% thicker than the design slab thickness at a particular location prior to vibrating and striking for finishing.

9.7.4.2 STEEL FORMWORK

The following shall supplement A9.7.4.2.

Details of forms shall not be shown on the contract drawings, but applicable Standard Drawings for permanent metal deck forms shall be referred to on the supplemental drawings table on the contract drawings for details.

The maximum corrugation depth and width shall be such that the total dead load of the form and of the concrete in the form does not exceed 75 kg/m^2 {0.015 ksf} and that the dead load deflection due to the weight of plastic concrete, deck steel reinforcement and form does not exceed the following:

- For design span lengths, S, less than or equal to 3000 mm {10 ft.}, S/180 or 13 mm {1/2 in.}, whichever is less

- For design span lengths, S, greater than 3000 mm {10 ft.}, S/240 or 19 mm {3/4 in.}, whichever is less.

C9.7.4.2

Delete AC9.7.4.2.

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The total of these loads, for design purposes, shall not be less than 880 kg/m^2 {0.180 ksf}. The permissible form camber shall be based on the actual dead load condition. Camber shall not be used to compensate for deflections in excess of the foregoing limits. Some new forms are manufactured in such a way as to eliminate concrete in the valleys.

In using permanent metal deck forms, an additional dead load consisting of the weight of the metal forms and the weight of the concrete in the valley of the forms shall be taken into account in the design.

The quantity of Class AAA cement concrete shall be computed to include concrete in the valleys of the metal forms (this may be approximated by adding an extra 25 mm thickness to the deck), plus compensation for deflection. A note stating "Quantity of Class AAA Cement Concrete includes Concrete in the Valleys of the Metal Forms" shall be shown under the table of "Summary of Quantities", where applicable.

9.7.4.3 CONCRETE FORMWORK

9.7.4.3.2 Reinforcement

The following shall supplement A9.7.4.3.2.

For prestressed concrete planks, prestressing tendons shall protrude from the plank a sufficient distance to develop anchorage in the cast-in-place portion of the deck slab between the ends of the planks.

The prestressed planks shall be in accordance with the applicable Standard Drawings.

9.7.4.3.5P Additional Requirements

Precast prestressed deck planks used as permanent forms spanning between stringers shall be designed compositely with the cast-in-place portion of the slabs to support additional dead loads and live loads.

The planks shall be analyzed assuming they support their self-weight, any construction loads, and the weight of the cast-in-place concrete, and shall be analyzed assuming they act compositely with the cast-in-place concrete to support moments due to additional dead loads and live loads.

Live load moments shall be computed in accordance with A4.6.2.1.

In calculating stresses in the deck planks due to negative moment near the stringer, no compression due to prestressing shall be assumed to exist.

Deck planks shall be prestressed with pretensioned strands. The strands shall be in a direction transverse to the stringers when the planks are on the supporting stringers.

Reinforcing bars, or equivalent mesh, shall be placed in

COMMENTARY

C9.7.4.3.2

The following shall supplement AC9.7.4.3.2.

Before any testing of epoxy-coated strands is begun (in order to determine transfer and development lengths), the Chief Bridge Engineer must review and approve the testing procedure. The results of this testing must also be approved by the Chief Bridge Engineer.

Research on epoxy-coated tendons for prestressed concrete planks is currently being conducted in another state.

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the plank transverse to the strands to provide at least 0.23 mm^2 per mm {0.11 in^2 per foot} of the plank.

9.7.5 Precast Deck Slabs on Girders

9.7.5.1 GENERAL

The following shall replace A9.7.5.1.

Both reinforced and prestressed precast concrete slab panels may be used. The minimum depth of the slab shall be as specified in D9.7.1.1.

All reinforcing steel and accessories (prestressed anchorage, couplers, etc.) shall be epoxy-coated for the full-depth deck panels in accordance with AASHTO M 284/M 284M. Special provisions concerning precast deck units shall be furnished by the designer, including the deck unit fastening systems, prestressing system corrosion protection, post-tension requirements at the deck panel joints, fabrication and erection requirements, and detailed information on deck unit joints, deck unit adjustment, and, if required, deck unit beam composite interaction. Note that an unbonded post-tensioning system will not be permitted.

9.7.5.2 TRANSVERSELY JOINED PRECAST DECKS

The following shall replace A9.7.5.2.

Transversely joined precast decks shall have sufficient bonded post-tensioning to cause the precast deck panels to behave as a continuous unit across the joints.

9.7.5.3 LONGITUDINALLY POST-TENSIONED PRECAST DECKS

The following shall replace the first paragraph of A9.7.5.3.

Longitudinally post-tensioned precast decks shall have sufficient bonded post-tensioning to cause the precast deck panels to behave as a continuous unit across the joints.

9.7.6 Deck Slabs in Segmental Construction

9.7.6.2 JOINTS IN DECKS

The following shall replace A9.7.6.2.

Joints in the decks of precast segmental bridges may be epoxied match-cast surfaces, or cast-in-place concrete.

The strength of cast-in-place concrete joints shall not be less than that of the precast concrete. The width of the concrete joint shall permit the development of reinforcement in the joint or coupling of ducts, if used, but in no case shall it be less than 300 mm {12 in.}.

Dry joints are not permitted in Pennsylvania.

COMMENTARY

C9.7.5.1P

The requirements given in D9.7.5.2 and D9.7.5.3 for post-tensioning of the joints is to eliminate joint deterioration.

Precast deck may not be practical for:

- short spans or small bridges where the quantity of precast deck is very small,
- continuous bridges, or
- bridges on a curve.

C9.7.5.2

The following shall supplement AC9.7.5.2.

Shear key construction without bond post-tensioned is not allowed in Pennsylvania.

SPECIFICATIONS

COMMENTARY

9.8 METAL DECKS

C9.8.1P General

The use of steel beam flooring as currently manufactured, unless specifically designed for fatigue, should be avoided for projects whenever the anticipated total accumulative truck traffic will exceed 300,000 trucks, with a mass exceeding 2.7 metric tonne {6.0 kips} per lane during the expected life of the flooring, unless the flooring is filled with concrete, either full-depth or half depth, as soon as possible after installation. Open steel beam bridge flooring has generally been a problem to maintain when subjected to heavy truck traffic, mainly as a result of the breaking of the welds and fatigue cracks (with ultimate fracture) of the cross bars, which are the most critical components of the grid floor. The flat-bar types appear to perform better relative to fatigue than the I-bar types.

Concrete-filled steel beam flooring performs satisfactorily relative to fatigue if properly connected to the stringers and girders. However, problems have been encountered with some filled steel grids growing in length and with welds breaking due to corrosion. Also, in some cases, apparently associated with the use of filled steel grid, a significant number of rivets and bolts have failed in stringer-to-floorbeam connections.

The first preference for a bridge deck should always be a conventional reinforced concrete slab.

9.8.2 Metal Grid Decks

9.8.2.1 GENERAL

The following shall replace the third paragraph of A9.8.2.1.

Compute composite section properties for filled or partially filled decks as described in Standard Drawing BD-604M. Where filled or partially filled grid decks are modeled for analysis as either an orthotropic plate, or an equivalent grillage, flexural and torsional rigidities may be obtained by other accepted and verified approximate methods or by physical testing which must be approved by the Chief Bridge Engineer.

9.8.2.2 OPEN GRID FLOORS

Delete A9.8.2.2.

C9.8.2.2

Delete AC9.8.2.2

PennDOT does not allow open grid floors.

9.8.2.3 FILLED AND PARTIALLY FILLED GRID DECKS

9.8.2.3.1 General

The following shall replace the third paragraph of A9.8.2.3.1.

SPECIFICATIONS

A 40 mm {1 1/2 in.} thick structural overfill shall be provided.

The following shall supplement A9.8.2.3.1.

In the negative moment region, a welded connection of the grid deck to the beam or girder is not permitted.

Do not provide any deck joints, except as permitted at specific substructure locations permitted by the Chief Bridge Engineer.

9.8.2.3.3 Fatigue and Fracture Limit State

The following shall supplement A9.8.2.3.3.

The welded internal connection among those elements of the steel grid which are not within the connection fill shall be considered as Category "E" details.

9.8.2.4 UNFILLED GRID DECKS COMPOSITE WITH REINFORCED CONCRETE SLABS

Delete all the articles under A9.8.2.4.

9.8.3 Orthotropic Steel Decks

9.8.3.3 WEARING SURFACE

The following shall supplement A9.8.3.3.

Before any testing of the orthotropic steel deck is begun (in order to determine long-term composite action between deck plate and wearing surface), the Chief Bridge Engineer must review and approve the testing procedure. The results of this testing must also be approved by the Chief Bridge Engineer.

9.8.5 Corrugated Metal Decks

Delete all the articles under A9.8.5.

9.9 WOOD DECKS AND DECK SYSTEMS

9.9.3 Design Requirements

9.9.3.6 SKEWED DECKS

The following shall replace the first sentence of A9.9.3.6.

Where the skew of the deck is greater than 65°, transverse laminations may be placed on the skew angle.

9.9.4 Glued Laminated Decks

9.9.4.1 GENERAL

The following shall supplement A9.9.4.1.

COMMENTARY

C9.8.2.4P

Pennsylvania requires further research on this type of deck. This type of deck will not be used until adequate research is presented and approved by the Chief Bridge Engineer.

C9.8.5P

Pennsylvania does not permit this type of deck.

SPECIFICATIONS

COMMENTARY

The minimum nominal deck thickness for glued laminated decks is 150 mm {6 in.}.

9.9.5 Stress Laminated Decks

9.9.5.1 GENERAL

The following shall replace the second paragraph of A9.9.5.1.

Stress laminated decks shall not be used where the skew is less than 45°.

The following shall supplement A9.9.5.1.

The deck thickness shall not be less than 200 mm {8 in.} nominal and the deck supports shall be continuous at abutments and piers.

9.9.5.6 STRESSING

9.9.5.6.1 Prestressing System

The following shall replace the second paragraph of A9.9.5.6.1.

In stress laminated decks, with skew angles greater than 65°, stressing bars may be parallel to the skew. For skew angles between 45° and 65°, the bars should be place perpendicular to the laminations and, in the end zones, the transverse prestressing bars should either be fanned in plan as shown in Figure A9.9.5.6.1-1, or be in a stepped arrangement as shown in Figure A9.9.5.6.1-2.

The following shall supplement A9.9.5.6.1.

Type C prestressing configuration, given in Figure A9.9.5.6.1-3, is not permitted for soft wood laminations.

9.9.5.6.3 Design Requirements

The following shall supplement A9.9.5.6.3.

The shortest lamination must have at least two prestressing rods passing through. The spacing of the first rod from the lamination end should be generally equal to one-half the center-to-center spacing of prestressing rods and not less than the lamination thickness.

The camber for these decks shall be three times the dead load deflection.

9.9.5.6.4 Corrosion Protection

The following shall supplement A9.9.5.6.4.

Provide triple protection system for prestressing rods, i.e., plastic sleeves, grease and waterproof membrane.

SPECIFICATIONS COMMENTARY

9.10P DECK PROTECTION SYSTEMS

9.10.1P General

This section outlines provisions for deck systems in the design and construction of new bridge decks for Federal, State and locally sponsored projects.

Polypropylene fibers or steel fibers may be specified in Class AAA and HPC concrete decks to control cracking. If used, the contract must include special provisions.

A Single Deck Protection System is the minimum acceptable level of protection for all decks. Provide one deck protection system listed below:

- Filled and partially-fill metal grid deck.
- Epoxy coated reinforcement.
- Galvanized reinforcement.

9.10.2P New Structures or Bridge Replacements

A Dual Deck Protection System or stainless steel reinforcement may be used for decks of new bridges on Federal Aid projects if approved by FHWA on a project-by-project basis.

The dual deck protection system must be a combination of deck protection systems as listed below:

A. The use of epoxy-coated or galvanized reinforcement and the use of 1½ inches of latex modified or micro silica modified concrete overlay over an 8” minimum thickness Class AAA or HPC concrete deck. Provide a 2 inch minimum clear cover over the top mat of reinforcement to the top of Class AAA or HPC Cement Concrete instead of 2.5 inches of clear cover per PennDOT Standard Drawings BD-601M.

B. Filled or partially-fill galvanized metal grid deck system, overfilled 1” during initial placement (overfilled monolithically) and the use of 1½ inches of latex modified or micro silica modified concrete overlay. Provide 1” overfill instead of 1½” per PennDOT Standard Drawings BD-604M.

The following may be used in lieu of a dual deck protection system:

C. Stainless steel (solid stainless steel) top mat reinforcement and bottom mat reinforcement used with an 8” minimum thickness Class AAA or HPC concrete deck. All ties, chairs and hardware in contact with reinforcement must be stainless steel.

SPECIFICATIONS

COMMENTARY

9.10.3P Rehabilitation and/or Deck Replacements

Dual deck protection should only be considered on rehabilitation projects and/or deck replacement projects at the discretion of the District Bridge Engineer, based on the cost benefit of such protection and expected life of the remaining structure, and if approved by FHWA on a project-by-project basis. Reference Section 9.10.2 for dual protection system combinations.

If the expected life of the remaining structure is less than the expected life of a proposed new deck, a Single Deck Protection System is acceptable (reference section 9.10.1 for allowable single deck protection systems).

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 1
PART B: DESIGN SPECIFICATIONS

SECTION 10 - FOUNDATIONS

SECTION 10 - TABLE OF CONTENTS

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SPECIFICATIONS

10.0P INTRODUCTION

This section entirely replaces Section 10 from AASHTO LRFD Bridge Design Specification, 3rd Edition, 2004.

This section contains elements of the AASHTO 2006 Interim Revisions to the 3rd Edition, and reflects the updated article numbering system. As all articles are therefore new, the suffix P is not shown.

10.1 SCOPE

Provisions of this Section shall apply for the design of spread footings, driven piles, and drilled shaft foundations. Design Specifications for micropiles are presented in Appendix B.

The probabilistic LRFD basis of these Specifications, which produces an interrelated combination of load, load factor resistance, resistance factor, and statistical reliability, shall be considered when selecting procedures for calculating resistance other than that specified herein. Other methods, especially when locally recognized and considered suitable for regional conditions, may be used if resistance factors are developed in a manner that is consistent with the development of the resistance factors for the method(s) provided in these Specifications, and are approved by the Owner.

COMMENTARY

C10.1

The development of the resistance factors provided in this Section are summarized in Allen (*2005*), with additional details provided in Appendix A of Barker et al. (*1991*), in Paikowsky et al. (*2004*), and in Allen (*2005*).

The specification of methods of analysis and calculation of resistance for foundations herein is not intended to imply that field verification and/or reaction to conditions actually encountered in the field are no longer needed. These traditional features of foundation design and construction are still practical considerations when designing in accordance with these Specifications.

10.2 DEFINITIONS

Battered Pile—A pile driven at an angle inclined to the vertical to provide higher resistance to lateral loads.

Bearing Pile—A pile whose purpose is to carry axial load through friction, end or point bearing.

Bent—A type of pier comprised of multiple columns or piles supporting a single cap and in some cases connected with bracing.

Bent Cap—A flexural substructure element supported by columns or piles that receives loads from the superstructure.

Column Bent—A type of bent that uses two or more columns to support a cap. Columns may be drilled shafts or other independent units supported by individual footings or a combined footing; and may employ bracing or struts for lateral support above ground level.

Combination Point Bearing and Friction Pile—Pile that derives its capacity from contributions of both point bearing developed at the pile tip and resistance mobilized along the embedded shaft.

Combined Footing—A footing that supports more than one column.

CPT—Cone Penetration Test.

Geomechanics Rock Mass Rating System—Rating system developed to characterize the engineering behavior of rock masses (Bieniawski, 1984).

CU—Consolidated Undrained.

Deep Foundation—A foundation that derives its support by transferring loads to soil or rock at some depth below the structure by point bearing, end bearing, adhesion or friction, or a combination.

DMT—Flat Plate Dilatometer Test.

Drilled Shaft—A deep foundation unit, wholly or partly embedded in the ground, constructed by placing fresh concrete in a drilled hole with or without steel reinforcement. Drilled shafts derive their capacity from the surrounding soil and/or from the soil or rock strata below its tip. Drilled shafts are also commonly referred to as caissons, drilled caissons, bored piles, or drilled piers.

Driveability—A measure of the ability of a hammer/pile system to economically provide an undamaged installed pile with the required capacity.

Effective Stress—The net stress across points of contact of soil particles, generally considered as equivalent to the total stress minus the pore water pressure.

End-Bearing Pile—A pile transferring a majority of its load to the soil and/or rock through friction or adhesion forces along the lower 1/3 of the pile shaft and bearing on the pile tip. (Same as combination point bearing and friction pile.)

ER—Hammer efficiency expressed as percent of theoretical free fall energy delivered by the hammer system actually used in a Standard Penetration Test.

Freeze—A natural phenomenon resulting in a time-dependent increase in pile strength occurring after driving. Freeze results from an increase in soil shear strength due to the dissipation of excess porewater pressures which develop in some cohesive soils during pile driving.

Friction Pile—A pile transferring a major portion of its load to the soil through friction or adhesion forces along the shaft. Only a small portion of the nominal strength is generated by bearing on the pile tip.

IGM—Intermediate Geomaterial, a material that is transitional between soil and rock in terms of strength and compressibility, such as residual soils, glacial tills, or very weak rock.

Isolated Footing—Individual support for the various parts of a substructure unit; the foundation is called a footing foundation.

Karst—A set of physical conditions, landforms and bedrock attributes that may be present in areas that are underlain by bedrock that is soluble in water.

Length of Foundation—Maximum plan dimension of a foundation element.

OCR—Over Consolidation Ratio, the ratio of the preconsolidation pressure to the current vertical effective stress.

Pile—A slender deep foundation unit, wholly or partly embedded in the ground, that is installed by driving, drilling, auguring, jetting, or otherwise and that derives its capacity from the surrounding soil and/or from the soil or rock strata below its tip.

Pile Bent—A type of bent using pile units, driven or placed, as the column members supporting a cap.

Pile Cap—A flexural substructure element located above or below the finished ground line that receives loads from substructure columns and is supported by shafts or piles.

Pile Load Test—A test to determine pile capacity by the application of a static load.

Pile Shoe—A metal piece fixed to the penetration end of a pile to protect it from damage during driving and to facilitate penetration through very dense material.

Piping—Progressive erosion of soil by seeping water that produces an open pipe through the soil through which water flows in an uncontrolled and dangerous manner.

Plunging—A mode of behavior observed in some pile load tests, wherein the settlement of the pile continues to increase with no increase in load.

PMT—Pressuremeter Test.

Point-Bearing Pile—A pile transferring a major portion of its load to the soil and/or rock through bearing on the pile tip. Only a minor portion of the load is transferred by friction or adhesion forces along the pile shaft.

Predetermined Pile Tip Elevation—The tip elevation, determined by the Engineer, from the pile load tests or test piles. The predetermined tip elevation locates the intended bearing strata for the point bearing or end bearing piles, or to verify or adjust the estimated tip elevations indicated.

Redriving—The act of driving a previously driven pile at some time after initial installation was completed. Redriving (also known as "restriking") is usually performed to evaluate the occurrence of freeze or relaxation. Evaluation may be made by comparison of driving resistances between the end of initial driving and the beginning of redriving, and/or the comparison of capacity estimates from dynamic monitoring at those times.

Relaxation—The time-dependent decrease in pile capacity occurring after driving, which typically occurs in relatively fine-grained soils and soft shales, claystones, or siltstones. Relaxation results from a decrease in soil or rock shear strength due to the dissipation of negative porewater pressures which develop in some very stiff cohesive soils and rocks (e.g., shales and claystones) during driving.

RMR—Rock Mass Rating.

RQD—Rock Quality Designation.

Shallow Foundation—A foundation that derives its support by transferring load directly to the soil or rock at shallow depth.

Slickensides—Polished and grooved surfaces in clayey soils or rocks resulting from shearing displacements along planes.

SPT—Standard Penetration Test.

Test Load Pile—A pile in a pile load test, also referred to as a load test pile.

Test Pile—A pile driven to verify the pile hammer's capability and to determine driving characteristics prior to driving test load and bearing piles.

Total Stress—Total pressure exerted in any direction by both soil and water.

UU—Unconsolidated Undrained.

VST—Vane Shear Test (performed in the field).

Width of Foundation—Minimum plan dimension of a foundation element.

10.3 NOTATION

A = pile cross-sectional area (mm^2){$ft.^2$} (10.7.3.8.2)
A' = effective footing area for determination of elastic settlement of footing subjected to eccentric loads {$ft.^2$} (10.6.1.3)
A_p = area of pile tip or base of drilled shaft (mm^2) {$ft.^2$} (10.7.3.8.6a)
A_s = surface area of pile shaft (mm^2) {$ft.^2$} (10.7.3.8.6a)
A_u = uplift area of a belled drilled shaft (mm^2) {$ft.^2$} (10.8.3.7.2)
a_{si} = pile perimeter at the point considered (mm){ft.} (10.7.3.8.6g)
B = footing width; pile group width; pile diameter (mm){ft.} (10.6.1.3) (10.7.2.3.2) (10.7.2.4)
B' = effective footing width (mm){ft.} (10.6.1.3)
C_α = secondary compression index, void ratio definition (dim.) (10.4.6.3)
$C_{\alpha\varepsilon}$ = secondary compression index, strain definition (dim.) (10.6.2.4.3)
C_c = compression index, void ratio definition (dim.) (10.4.6.3)
$C_{c\varepsilon}$ = compression index, strain definition (dim.) (10.6.2.4.3)
C_F = correction factor for K_δ when δ is not equal to ϕ_f (dim.) (10.7.3.8.6f)
C_N = overburden stress correction factor for N (dim.) (10.4.6.2.4)
C_o = Uniaxial compressive strength of intact rock (MPa) {ksf} (10.6.3.2.2)
C_r = recompression index, void ratio definition (dim.) (10.4.6.3)
$C_{r\varepsilon}$ = recompression index, strain definition (dim.) (10.6.2.4.3)
C_{wq}, $C_{w\gamma}$ = correction factors for groundwater effect (dim.) (10.6.3.1.3)
C' = bearing capacity index (dim.) (10.6.2.4.2)
c = cohesion of soil taken as undrained shear strength (MPa){ksf} (10.6.3.1.2a)
c_v = coefficient of consolidation (mm^2/yr.) (10.4.6.3)
c_1 = undrained shear strength of the top layer of soil as depicted in Figure 10.6.3.1.2e-1 (MPa){ksf} (10.6.3.1.2e)
c_2 = undrained shear strength of the lower layer of soil as depicted in Figure 10.6.3.1.2e-1 (MPa){ksf} (10.6.3.1.2e)
c'_1 = drained shear strength of the top layer of soil (MPa){ksf} (10.6.3.1.2f)
c^* = reduced effective stress soil cohesion for punching shear (MPa){ksf} (10.6.3.1.2b)
c' = effective stress cohesion intercept (MPa){ksf} (10.4.6.2.3)
c'_i = instantaneous cohesion at a discrete value of normal stress (MPa){ksf} (C10.4.6.4)
D = depth of pile embedment; pile width or diameter; diameter of drilled shaft (mm){ft.} (10.6.3.1.2g) (10.7.2.3) (10.7.3.8.6g) (10.8.3.5.1c)
DD = downdrag load per pile (N){kips} (C10.7.3.7)
D' = effective depth of pile group (mm){ft.} (10.7.2.3.2)
D_b = depth of embedment of pile into a bearing stratum (mm){ft.} (10.7.2.3.2)
D_{est} = estimated pile length needed to obtain desired nominal resistance per pile (mm){ft.} (C10.7.3.7)
D_f = foundation embedment depth taken from ground surface to bottom of footing (mm){ft.} (10.6.3.1.2a)
D_i = pile width or diameter at the point considered (mm){ft.} (10.7.3.8.6g)
D_p = diameter of the bell on a belled drilled shaft (mm){ft.} (10.8.3.7.2)
D_s = socket diameter (mm) {ft.} (10.8.1.3)
D_w = depth to water surface taken from the ground surface (mm){ft.} (10.6.3.1.3)
E = modulus of elasticity of pile material (MPa){ksi} (10.7.3.8.2)
E_i = modulus of elasticity of intact rock (MPa){ksi} (10.4.6.5)
E_m = rock mass modulus (MPa){ksi} (10.4.6.5)
E_p = modulus of elasticity of pile (MPa){ksi} (10.7.3.13.4)
ER = hammer efficiency expressed as percent of theoretical free fall energy delivered by the hammer system actually used (dim.) (10.4.6.2.4)
E_s = soil (Young's) modulus (MPa){ksi} (C10.4.6.3)
e = void ratio (dim.) (10.6.2.4.3)
e_B = eccentricity of load parallel to the width of the footing (mm){ft.} (10.6.1.3)
e_L = eccentricity of load parallel to the length of the footing (mm){ft.} (10.6.1.3)
e_o = void ratio at initial vertical effective stress (dim.) (10.6.2.4.3)
f'_c = 28-day compressive strength of concrete (MPa){ksi} (10.4.6.4)
f_{pe} = effective stress in the prestressing steel after losses (MPa){ksi} (10.7.8)
f_s = approximate constant sleeve friction resistance measured from a *CPT* at depths below 8*D* (MPa){ksf} (C10.7.3.8.6g)
f_{si} = unit local sleeve friction resistance from *CPT* at the point considered (MPa){ksf} (10.7.3.8.6g)

f_y = yield strength of steel (MPa){ksi} (10.7.8)
H = horizontal component of inclined loads (N){kips} (10.6.3.1.2a)
H_c = height of compressible soil layer (mm){ft.} (10.6.2.4.2)
H_{crit} = minimum distance below a spread footing to a second separate layer of soil with different properties that will affect shear strength of the foundation (mm){ft.} (10.6.3.1.2d)
H_d = length of longest drainage path in compressible soil layer (mm){ft.} (10.6.2.4.3)
H_s = height of sloping ground mass (mm){ft.} (10.6.3.1.2c)
H_{s2} = distance from bottom of footing to top of the second soil layer (mm){ft.} (10.6.3.1.2e)
h_i = length interval at the point considered (mm){ft.} (10.7.3.8.6g)
I = influence factor of the effective group embedment (dim.) (10.7.2.3.2)
I_p = influence coefficient to account for rigidity and dimensions of footing (dim.) (10.6.2.4.4)
I_w = weak axis moment of inertia for a pile (mm^4){$ft.^4$} (10.7.3.13.4)
i_c, i_q, i_γ = load inclination factors (dim.) (10.6.3.1.2a)
J = Bearing capacity correction factor for footings on rock with widely spaced discontinuities (dim) (10.6.3.2.3b)
K_c = correction factor for side friction in clay (dim.) (10.7.3.8.6g)
K_s = correction factor for side friction in sand (dim.) (10.7.3.8.6g)
K_δ = coefficient of lateral earth pressure at midpoint of soil layer under consideration (dim.) (10.7.3.8.6f)
k = Lateral earth pressure coefficient (dim) (C10.7.3.3.2b)
L = length of foundation; pile length (mm){ft.} (10.6.1.3) (10.7.3.8.2)
L' = effective footing length (mm){ft.} (10.6.1.3)
L_i = depth to middle of length interval at the point considered (mm){ft.} (10.7.3.8.6g)
LL = liquid limit of soil (percent) (10.4.6.3)
ℓ = span between adjacent units (mm) {ft.) (10.5.2.2)
N = uncorrected Standard Penetration Test (*SPT*) blow count (blows/300 mm){blows/ft.} (10.4.6.2.4)
$\overline{N}1_{60}$ = average corrected *SPT* blow count along pile side (blows/300 mm){blows/ft.} (10.6.3.1.3)
$N1$ = *SPT* blow count corrected for overburden pressure σ'_v (blows/300 mm) {blows/ft.} (10.4.6.2.4)
$N1_{60}$ = *SPT* blow count corrected for both overburden and hammer efficiency effects (blows/300 mm) {blows/ft.} (10.4.6.2.4) (10.7.2.3.2)
N_c = cohesion term (undrained loading) bearing capacity factor (dim.) (10.6.3.1.2a)
N_{cq}, $N_{\gamma q}$ = Modified bearing capacity factors for effects of footing on or adjacent sloping ground (dim) (10.6.3.1.2c)
N_q = surcharge (embedment) term (drained or undrained loading) bearing capacity factor (dim.) (10.6.3.1.2a)
N' = alternate notation for *N1* (blows/300 mm) (10.6.2.4.2)
N'_q = pile bearing capacity factor from Figure 10.7.3.8.6f-8 (dim.) (10.7.3.8.6f)
N_γ = unit weight (footing width) term (drained loading) bearing capacity factor (dim.) (10.6.3.1.2a)
N_{cm}, N_{qm},
$N_{\gamma m}$ = modified bearing capacity factors (dim.) (10.6.3.1.2a)
N_m = modified bearing capacity factor (dim.) (10.6.3.1.2e)
N_{ms} = Coefficient factor to estimate q_{ult} for rock (dim) (10.6.3.2.2)
N_s = slope stability factor (dim.) (10.6.3.1.2c)
N_u = uplift adhesion factor for bell (dim.) (10.8.3.7.2)
N_1 = number of intervals between the ground surface and a point 8*D* below the ground surface (dim.) (10.7.3.8.6g)
N_2 = number of intervals between 8*D* below the ground surface and the tip of the pile (dim.) (10.7.3.8.6g)
N_{60} = *SPT* blow count corrected for hammer efficiency (blows/300 mm){blows/ft.} (10.4.6.2.4)
n = Exponential factor relating B/L or L/B ratios for inclined loading (dim) (10.6.3.1.2a)
n = porosity (dim.); number of soil layers within zone of stress influence of the footing (dim.) (10.4.6.2.4) (10.6.2.4.2)
n_h = rate of increase of soil modulus with depth (MPa/mm){ksi/ft.} (10.4.6.3)
P_f = probability of failure (dim.) (C10.5.5.2.1)
PL = plastic limit of soil (percent) (10.4.6.3)
p_a = atmospheric pressure (MPa){ksf} (Sea level value equivalent to 1 atm or 0.101 MPa {2.12 ksf}) (10.8.3.5.1b)
Q = load applied to top of footing or shaft (N){kips}; load test load (N){kips} (C10.6.3.1.2b) (10.7.3.8.2)
Q_p = factored load per pile, excluding downdrag load (N) {kips} (C10.7.3.7)
Q_{T1} = total load acting at the head of the drilled shaft (N) {kips} (C10.8.3.5.4d)
q = net foundation pressure applied at $2D_b/3$; this pressure is equal to applied load at top of the group divided by the area of the equivalent footing and does not include the weight of the piles or the soil between the piles (MPa){ksf} (10.7.2.3.2)

q_c = static cone tip resistance (MPa){ksf} (C10.4.6.3) (10.7.2.3.2)
$\overline{q_c}$ = average static cone tip resistance over a depth *B* below the equivalent footing (MPa){ksf} (10.6.3.1.3)
q_{c1} = average q_c over a distance of *yD* below the pile tip (path a-b-c) (MPa){ksf} (10.7.3.8.6g)
q_{c2} = average q_c over a distance of 8*D* above the pile tip (path c-e) (MPa){ksf} (10.7.3.8.6g)
q_L = limiting unit tip resistance of a single pile from Figure 10.7.3.8.6f-9 (MPa){ksf} (10.7.3.8.6f)
q_ℓ = limiting tip resistance of a single pile (MPa){ksf} (10.7.3.8.6g)
q_{max} = Maximum magnitude of footing contact pressure (MPa) {ksf} (10.6.3.1.2eP)
q_{min} = Minimum magnitude of footing contact pressure (MPa) {ksf} (10.6.3.1.2eP)
q_n = nominal bearing resistance (MPa){ksf} (10.6.3.1.1)
q_o = applied vertical stress at base of loaded area (MPa){ksf} (10.6.2.4.2)
q_p = nominal unit tip resistance of pile (MPa){ksf} (10.7.3.8.6a)
q_R = factored bearing resistance (MPa){ksf} (10.6.3.1.1)
q_s = unit shear resistance; unit side resistance of pile (MPa){ksf} (10.6.3.4) (10.7.3.8.6a)
q_{sbell} = nominal unit uplift resistance of a belled drilled shaft (MPa){ksf} (10.8.3.7.2)
q_u = uniaxial compression strength of rock (MPa){ksf} (10.4.6.4)
q_{ult} = nominal bearing resistance (MPa){ksf} (10.6.3.1.2a)
q_1 = nominal bearing resistance of footing supported in the upper layer of a two-layer system, assuming the upper layer is infinitely thick (MPa){ksf} (10.6.3.1.2d)
q_2 = nominal bearing resistance of a fictitious footing of the same size and shape as the actual footing but supported on surface of the second (lower) layer of a two-layer system (MPa){ksf} (10.6.3.1.2d)
R = Resultant of pressure on base of footing (N) {tons} (10.6.3.1.2eP)
RQD = Rock Quality Designation (dim) (10.6.3.2.2)
R_{ep} = nominal passive resistance of soil available throughout the design life of the structure (N) {kips} (10.6.3.4)
R_n = nominal resistance of footing, pile or shaft (N){kips} (10.6.3.4)
R_{ndr} = nominal pile driving resistance including downdrag (N) {kips} (C10.7.3.3)
R_{nstat} = nominal resistance of pile from static analysis method (N) {kips} (C10.7.3.3)
R_p = pile tip resistance (N) {kips} (10.7.3.8.6a)
R_R = factored nominal resistance of a footing, pile or shaft (N) {kips} (10.6.3.4)
R_s = pile side resistance (N); nominal uplift resistance due to side resistance (N){kips} (10.7.3.8.6a) (10.7.3.10)
R_{sbell} = nominal uplift resistance of a belled drilled shaft (N){kips} (10.8.3.7.2)
R_{sdd} = skin friction which must be overcome during driving (N) {kips} (C10.7.3.7)
R_τ = nominal sliding resistance between the footing and the soil (N) {kips} (10.6.3.4)
R_{ug} = nominal uplift resistance of a pile group (N) {kips} (10.7.3.11)
r = radius of circular footing or *B*/2 for square footing (mm){ft.} (10.6.2.4.4)
S_c = primary consolidation settlement (mm){ft.} (10.6.2.4.1)
$S_{c(1\text{-}D)}$ = single dimensional consolidation settlement (mm){ft.} (10.6.2.4.3)
$S_{c(3\text{-}D)}$ = Three-dimensional consolidation settlement (mm) {ft.} (10.6.2.4.3)
S_e = elastic settlement (mm){ft.} (10.6.2.4.1)
S_s = secondary settlement (mm){ft.} (10.6.2.4.1)
S_t = total settlement (mm){ft.} (10.6.2.4.1)
s_f = pile top movement during load test (mm){ft.} (10.7.3.8.2)
S_u = undrained shear strength (MPa){ksf} (10.4.6.2.2)
$\overline{S}_u$ = average undrained shear strength along pile side (MPa){ksf} (10.7.3.9)
s, m = fractured rock mass parameters (10.4.6.4)
s_c, s_q, s_γ = shape factors (dim.) (10.6.3.1.2a)
T = time factor (dim.) (10.6.2.4.3)
t = time for a given percentage of one-dimensional consolidation settlement to occur (yr.) (10.6.2.4.3)
t_1, t_2 = arbitrary time intervals for determination of secondary settlement, S_s (yr.) (10.6.2.4.3)
U = percentage of consolidation (10.6.2.4.3)
V = total vertical force applied by a footing (N){kips}; pile displacement volume (mm^3/mm) {$ft.^3$/ft.}(10.6.3.1.2a) (10.7.3.8.6f)
W_g = weight of block of soil, piles and pile cap (N){kips} (10.7.3.11)
W_{T1} = vertical movement at the head of the drilled shaft (mm){ft.} (C10.8.3.5.4d)
X = width or smallest dimension of pile group (mm){ft.} (10.7.3.9)
Y = length of pile group (mm){ft.} (10.7.3.9)
Z = total embedded pile length; penetration of shaft (mm){ft.} (C10.7.3.8.6g)

z = depth below ground surface (mm){ft.} (C10.4.6.3)
z_w = Depth from footing base down to the highest anticipated groundwater level (mm) {ft.} (10.6.3.1.2g)
α = adhesion factor applied to s_u (dim.) (10.7.3.8.6b)
α_E = reduction factor to account for jointing in rock (dim.) (10.8.3.5.4b)
α_t = coefficient from Figure 10.7.3.8.6f-7 (dim.) (10.7.3.8.6f)
β = reliability index; coefficient relating the vertical effective stress and the unit skin friction of a pile or drilled shaft (dim.) (C10.5.5.2.1) (10.7.3.8.6c)
β_m = punching index (dim.) (10.6.3.1.2e)
β_z = factor to account for footing shape and rigidity (dim.) (10.6.2.4.2)
γ = unit density of soil (kg/m^3) {kcf}(10.6.3.1.2a)
γ_m = Moist unit weight of soil (kg/m^3) {kcf} (10.6.3.1.2g)
γ_p = load factor for downdrag (C10.7.3.7)
ΔH_i = elastic settlement of layer i (mm){ft.} (10.6.2.4.2)
δ = elastic deformation of pile (mm){ft.}; friction angle between foundation and soil (°) (C10.7.3.8.2) (10.7.3.8.6f)
δ' = Differential settlement between adjacent footings (mm) {ft.} (A10.5.2.2)
ε_v = vertical strain of over consolidated soil (mm/mm) (10.6.2.4.3)
η = shaft efficiency reduction factor for axial resistance of a drilled shaft group (dim.) (10.7.3.9)
λ = empirical coefficient relating the passive lateral earth pressure and the unit skin friction of a pile (dim.) (10.7.3.8.6d)
μ_c = reduction factor for consolidation settlements to account for three-dimensional effects (dim.) (10.6.2.4.3)
ϕ_f = angle of internal friction of drained soil (°) (10.4.6.2.4)
ϕ_{fm} = Angle of friction of rock mass (°) (10.6.3.2.3a)
ϕ'_f = drained (long term) effective angle of internal friction of clays (°) (10.4.6.2.3)
ϕ'_i = instantaneous friction angle of the rock mass (°) (10.4.6.4)
ϕ'_1 = effective stress angle of internal friction of the top layer of soil (°) (10.6.3.1.2f)
ϕ^* = reduced effective stress soil friction angle for punching shear (°) (10.6.3.1.2b)
φ = resistance factor (dim.) (10.5.5.2.3)
φ_b = resistance factor for bearing of shallow foundations (dim.) (10.5.5.2.2)
$\varphi_{b\ell}$ = resistance factor for driven piles or shafts, block failure in clay (dim.) (10.5.5.2.3)
φ_{da} = resistance factor for driven piles, drivability analysis (dim.) (10.5.5.2.3)
φ_{dyn} = resistance factor for driven piles, dynamic analysis and static load test methods (dim.) (10.5.5.2.3)
φ_{ep} = resistance factor for passive soil resistance (dim.) (10.5.5.2.2)
φ_{load} = resistance factor for shafts, static load test (dim.) (10.5.5.2.4)
φ_{qp} = resistance factor for tip resistance (dim.) (10.8.3.5)
φ_{qs} = resistance factor for shaft side resistance (dim.) (10.8.3.5)
φ_τ = resistance factor for sliding resistance between soil and footing (dim.) (10.5.5.2.2)
φ_{stat} = resistance factor for driven piles or shafts, static analysis methods (dim.) (10.5.5.2.3)
φ_{ug} = resistance factor for group uplift (dim.) (10.5.5.2.3)
φ_{up} = resistance factor for uplift resistance of a single pile or drilled shaft (dim.) (10.5.5.2.3)
φ_{upload} = resistance factor for shafts, static uplift load test (dim.) (10.5.5.2.4)
κ = shear strength ratio (c_2/c_1) for two layered cohesive soil system below footing (dim) (10.6.3.1.2kP)
π = 3.14
σ'_v = the effective vertical stress at midpoint of soil layer under consolidation (MPa) {ksf} (C10.7.3.8.6d)

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10.4 SOIL AND ROCK PROPERTIES

10.4.1 Informational Needs

The expected project requirements shall be analyzed to determine the type and quantity of information to be developed during the geotechnical exploration. This analysis should consist of the following:

- Identify design and constructability requirements, e.g., provide grade separation, support loads from bridge superstructure, provide for dry excavation, and their effect on the geotechnical information needed.
- Identify performance criteria, e.g., limiting settlements, right of way restrictions, proximity of adjacent structures, and schedule constraints.
- Identify areas of geologic concern on the site and potential variability of local geology.
- Identify areas of hydrologic concern on the site, e.g., potential erosion or scour locations.
- Develop likely sequence and phases of construction and their effect on the geotechnical information needed.
- Identify engineering analyses to be performed, e.g., bearing capacity, settlement, global stability.
- Identify engineering properties and parameters required for these analyses.
- Determine methods to obtain parameters and assess the validity of such methods for the material type and construction methods.
- Determine the number of tests/samples needed and appropriate locations for them.

C10.4.1

The first phase of an exploration and testing program requires that the Engineer understand the project requirements and the site conditions and/or restrictions. The ultimate goal of this phase is to identify geotechnical data needs for the project and potential methods available to assess these needs.

Geotechnical Engineering Circular #5—Evaluation of Soil and Rock Properties (*Sabatini et al., 2002*) provides a summary of information needs and testing considerations for various geotechnical applications.

10.4.2 Subsurface Exploration

Subsurface explorations shall be performed to provide the information needed for the design and construction of foundations. The extent of exploration shall be based on variability in the subsurface conditions, structure type, and any project requirements that may affect the foundation design or construction. The exploration program should be extensive enough to reveal the nature and types of soil deposits and/or rock formations encountered, the engineering properties of the soils and/or rocks, the

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In addition to the normal site investigation report, the Engineer may require the submission of a report which shall include the results of an investigation to determine potential hazards and seismic design requirements related to (1) slope instability, (2) liquefaction, (3) fill settlement and (4) increases in lateral earth pressure, all as a result of earthquake motions. Seismically-induced slope instability in approach fills or cuts may displace abutments and lead to significant differential settlement and structural damage. Fill settlement and abutment displacements due to lateral

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potential for liquefaction, and the ground water conditions. The exploration program should be sufficient to identify and delineate problematic subsurface conditions such as karstic formations, mined out areas, swelling/collapsing soils, existing fill or waste areas, etc.

Borings should be sufficient in number and depth to establish a reliable longitudinal and transverse substrata profile at areas of concern such as at structure foundation locations and adjacent earthwork locations, and to investigate any adjacent geologic hazards that could affect the structure performance.

As a minimum, the subsurface exploration and testing program shall obtain information adequate to analyze foundation stability and settlement with respect to:

- Geological formation(s) present,
- Location and thickness of soil and rock units,
- Engineering properties of soil and rock units, such as unit weight, shear strength and compressibility,
- Ground water conditions,
- Ground surface topography, and
- Local considerations, e.g., liquefiable, expansive or dispersive soil deposits, underground voids from solution weathering or mining activity, or slope instability potential.

Perform subsurface explorations in accordance with Policies and Procedures, Chapter 6; and Publication 293, Geotechnical Engineering Manual, Part 2.

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pressure increases may have led to many bridge access problems and structural damage. Liquefaction of saturated cohesionless fills or foundation soils may contribute to slope and abutment instability and lead to a loss of foundation bearing capacity and lateral pile support. Liquefaction failures of the above types have led to many bridge failures during the past earthquakes.

Areas of karst geology present special challenges because of the unique weathering, erosion, drainage and subsidence features that may be found there. Karst refers to a set of physical conditions, landforms, and bedrock attributes that may be present in areas that are underlain by bedrock that is soluble in water. In Pennsylvania, karst conditions are associated with carbonate rocks such as limestone and dolomite. The PA Geologic Survey (DCNR, 2000) has mapped the distribution of limestone and dolomite in Pennsylvania. Characteristics of karst areas include irregular and pinnacled bedrock surfaces; soil overburden mixed with rock fragments; and open cavities, conduits or caverns within the bedrock.

Sinkhole development is often associated with karst areas. Overburden soils are typically residual material resulting from weathering of the underlying bedrock. This residual soil can migrate or be washed into openings in the rock. As the soil continues to migrate into the openings in the rock a very soft zone or void develops in the overburden soil. When insufficient material remains to support the overlying soils, the roof collapses and a sinkhole develops. Sinkhole development is often associated with the movement of water. Sinkholes can be naturally occurring due to the percolation of surface water from natural drainage patterns. Sinkhole development can also be triggered by changes in drainage patterns due to development, construction activity that removes a portion of the overburden leaving insufficient material to bridge underlying voids, or dewatering and the associated drop in groundwater level. *For further description of sinkholes, sinkhole remediation and sinkhole prevention see the Ground Subsidence Management Policy (Currently in draft form, BOCM 2007)*

Prior to planning a subsurface investigation in karst areas a thorough review of published and unpublished information should be performed in accordance with PP6.2.1. Carefully review historic aerial photographs to identify sinkhole scars, closed depressions and possible zones of bedrock fracturing. Stereoscopic aerial photographs are more effective than individual photos in identifying these features. Review available mapping of karst features and sinkholes (Kochanov).

During the review of published and unpublished information and the visual site inspection (PP6.2.2) particular attention should be paid to the presence of karst landforms such as sinkholes, closed depressions, resurgent springs and bedrock outcrops. Areas that farmers avoid could be pinnacles or sinkholes. Changes in vegetation

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sometimes indicate sinkhole activity. Forested areas in fully farmed lands may also indicate areas of shallow rock. Old sinkholes may have been used as dumps, and can be found below debris piles. Any trends in karst features should be noted (e.g. sinkhole alignment). Information on past sinkhole activity may be gained from interviews with local residents, municipal officials or local contractors.

Publication 293 provides guidance regarding selection of number and depth of borings, drilling techniques, sampling methods, in-situ testing and geophysical testing. Test borings with Standard Penetration Test (SPT) sampling in soil and rock coring are recommended for any subsurface investigation in karst areas. It is important that the boring logs include observations of such conditions as soft or wet zones in the soil overburden, drill water return (or the lack of), reaction of rock core samples to dilute HCL solution, and voids or soil-filled seams in bedrock. It is desirable to obtain SPT samples of soil seams in bedrock. In order to do this it will be necessary to advance casing below the top of rock or to use NX drilling tools. NX core barrels are large enough to accommodate a 2-inch split-barrel sampler for SPT sampling.

Test borings with SPT sampling and rock coring are relatively expensive and time consuming and provide data at the boring location only. Because subsurface conditions in karst areas are highly variable supplemental techniques such as pneumatic-powered, track-mounted percussion drilling (air-track), electronic cone penetrometer and geophysical methods can be valuable in obtaining a more complete understanding of subsurface conditions at the site. In all cases it is desirable to obtain data with the supplementary methods close to SPT/rock core borings so the results can be correlated.

Air-track drilling has the advantages of mobility, speed of drilling and relative economy. It is effective at penetrating boulders and ledges. An experienced operator can qualitatively detect voids, zones of broken rock and the soil/rock interface. However, when drilling rock, it is difficult to distinguish between raveling decomposed rock zones and zones of residual soil or between zones of soft soil and open cavities. Air-track drilling can be problematic through thick overburden or in intensely weathered bedrock with numerous clay-filled cavities and steeply sloping rock surface.

Use of an electric cone penetrometer can provide data on soft soils or cavities within the soil overburden, geotechnical data, and inferred top of rock at a relatively reasonable cost. Disadvantages are the inability to penetrate cobbles, boulders or ledges.

Since karst areas are characterized by highly variable subsurface conditions, geophysical investigations can be helpful to supplement and refine intrusive subsurface investigation programs, such as SPT, CPT, etc. See D10.4.5 for a general discussion of geophysical tests and references for detailed guidelines.

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In karst areas, physical properties of interest include top of rock profile, voids or soil-filled seams in bedrock, voids in the overburden soils and soft zones in the overburden soils. Several geophysical methods can be used to obtain information on these properties.

Table C10.4.2 presents a summary of geophysical methods that are used in karst areas. The methods presented are not all inclusive, but represent the most commonly used methods. Table C10.4.2 is specific to the application of geophysical methods to karst areas.

Table C10.4.2 - Geophysical Methods Used In Karst Areas

GEOPHYSICAL METHOD	INFORMATION OBTAINED	LIMITATIONS
Seismic Refraction	- Depth to bedrock - Depth to water table - Cave detection and mapping - Fault and fracture delineation	- Sensitive to acoustic noise and vibrations - Not effective if stiffness decreases with depth or if soft layer underlies stiff layer (including a frozen surface layer) - Works best when sharp stiffness discontinuity is present - Provides limited information below top of rock surface
Electrical Resistivity	- Depth to bedrock - Depth to water table - Dissimilar strata - Fault and fracture delineation - Clay seam detection and mapping	- Resolution decreases with increasing depth - Susceptible to interference from nearby metal pipes, cables, or fences. - Heavy surface vegetation complicates data collection.
Electromagnetics (EM)	- Delineate areas of shallow/deep bedrock - Voids in soil	- Qualitative estimate of depths; extra effort required to characterize depth of target - Resolution decreases with increasing depth - Susceptible to interference from nearby metal pipes, fences, vehicles, noise from power lines and atmospheric storms.
Ground Penetrating Radar (GPR)	- Depth to bedrock - Depth to water table - Void detection	- Not effective below the water table or in clay. - Depth of penetration about 10 m {33 feet} - Susceptible to interference from metal reinforcement, guardrails, and power lines.
Gravity	- Voids in soil and rock - Fault and fracture delineation	- Susceptible to interference from vibrations - Results are non-unique (i.e. more than one subsurface condition can give the same result) - Primarily large scale reconnaissance tool. - For voids in rock another geophysical method may be needed to delineate the bedrock surface to aid in data interpretation. - Void detectability decreases rapidly with depth.
Multichannel Analysis of Surface Wave (MSAW)	- Depth to bedrock - Depth to water table - Cave detection and mapping - Clay seam detection and mapping - Fault and fracture delineation	- Method relatively new, therefore less industry experience with results

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In addition to identifying methods that address physical properties of interest several additional considerations should be addressed when selecting a geophysical method, or combination of methods for use (Anderson, et al., 2003).

- What methods provide the required spatial resolution and target definition.
- What methods will perform well under the physical site constraints?
- What methods are cost effective?
- What methods provide complementary data?
- What non-geophysical control is required to refine the interpretation of the acquired geophysical data?
- Is the overall program, including intrusive subsurface investigations, cost effective?

While published guidelines referenced in D10.4.5 are helpful, the geophysical industry continues to develop and evolve. During the planning of any geophysical program it is recommended to consult an experienced geophysicist. Geophysical testing data should always be correlated with information from direct methods of exploration, preferably SPT/rock core borings. In addition, existing subsurface data should be shared with the geophysicist so that appropriate methods are selected and accurate interpretations are rendered.

10.4.3 Laboratory Tests

Soil and rock properties for use in the service limit state and strength limit state evaluation of foundations shall be based on the results of the field and/or laboratory testing. For additional information on field and laboratory testing, see Publication 293.

10.4.3.1 SOIL TESTS

Laboratory testing should be conducted to provide the basic data with which to classify soils and to measure their engineering properties.

When performed, laboratory tests shall be conducted in accordance with the AASHTO, ASTM, or Department procedures applicable to the design properties needed.

C10.4.3.1

Laboratory tests of soils may be grouped broadly into two general classes:

- Classification or index tests. These may be performed on either disturbed or undisturbed samples.
- Quantitative or performance tests for permeability, compressibility and shear strength. These tests are generally performed on undisturbed samples, except for materials to be placed as controlled fill or materials that do not have a stable soil-structure, e.g., cohesionless materials. In these cases, tests should be performed on

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specimens prepared in the laboratory.

Detailed information regarding the types of tests needed for foundation design is provided in Geotechnical Engineering Circular #5—Evaluation of Soil and Rock Properties (*Sabatini et al., 2002*).

10.4.3.2 ROCK TESTS

If laboratory strength tests are conducted on intact rock samples for classification purposes, they should be considered as upper bound values. If laboratory compressibility tests are conducted, they should be considered as lower bound values. Additionally, laboratory tests should be used in conjunction with field tests and field characterization of the rock mass to give estimates of rock mass behavioral characteristics. When performed, laboratory tests shall be conducted in accordance with the ASTM or owner-supplied procedures applicable to the design properties needed.

C10.4.3.2

Rock samples small enough to be tested in the laboratory are usually not representative of the entire rock mass. Laboratory testing of rock is used primarily for classification of intact rock samples, and, if performed properly, serves a useful function in this regard.

Detailed information regarding the types of tests needed and their use for foundation design is provided in Geotechnical Engineering Circular #5—Evaluation of Soil and Rock Properties, April 2002 (*Sabatini et al., 2002*).

10.4.4 In-Situ Tests

In-situ tests may be performed to obtain deformation and strength parameters of foundation soils or rock for the purposes of design and/or analysis. In-situ tests should be conducted in soils that do not lend themselves to undisturbed sampling as a means to estimate soil design parameters. When performed, in-situ tests shall be conducted in accordance with the appropriate ASTM or AASHTO standards.

Where in-situ test results are used to estimate design properties through correlations, such correlations should be well established through long-term widespread use or through detailed measurements that illustrate the accuracy of the correlation.

For additional information regarding the applicability of various in-situ soil tests, refer to Publication 293.

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Detailed information on in-situ testing of soils and rock and their application to geotechnical design can be found in Sabatini et al. (*2002*) and Wyllie (*1999*).

Correlations are in some cases specific to a geological formation. While this fact does not preclude the correlation from being useful in other geologic formations, the applicability of the correlation to those other formations should be evaluated.

For further discussion, see D10.4.6.

10.4.5 Geophysical Tests

Geophysical testing should be used only in combination with information from direct methods of exploration, such as *SPT*, *CPT*, etc. to establish stratification of the subsurface materials, the profile of the top of bedrock and bedrock quality, depth to groundwater, limits of types of soil deposits, the presence of voids, anomalous deposits, buried pipes, and depths of existing foundations. Geophysical tests shall be selected and conducted in accordance with available ASTM standards. For those cases where ASTM standards are not available, other widely accepted detailed guidelines, such as Sabatini et al. (*2002*), AASHTO Manual on Subsurface Investigations (*1988*), Arman et al. (*1997*) and Campanella (*1994*), should be used.

For additional information on geophysical methods,

C10.4.5

Geophysical testing offers some notable advantages and some disadvantages that should be considered before the technique is recommended for a specific application. The advantages are summarized as follows:

- Many geophysical tests are noninvasive and thus, offer significant benefits in cases where conventional drilling, testing and sampling are difficult, e.g., deposits of gravel, talus deposits, or where potentially contaminated subsurface soils may occur.

- In general, geophysical testing covers a relatively large area, thus providing the opportunity to generally characterize large areas in order to optimize the

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refer to Publication 293.

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locations and types of in-situ testing and sampling. Geophysical methods are particularly well suited to projects that have large longitudinal extent compared to lateral extent, e.g., new highway construction.

- Geophysical measurement assesses the characteristics of soil and rock at very small strains, typically on the order of 0.001 percent, thus providing information on truly elastic properties, which are used to evaluate service limit states.

- For the purpose of obtaining subsurface information, geophysical methods are relatively inexpensive when considering cost relative to the large areas over which information can be obtained.

Some of the disadvantages of geophysical methods include:

- Most methods work best for situations in which there is a large difference in stiffness or conductivity between adjacent subsurface units.

- It is difficult to develop good stratigraphic profiling if the general stratigraphy consists of hard material over soft material or resistive material over conductive material.

- Results are generally interpreted qualitatively and, therefore, only an experienced Engineer or geologist familiar with the particular testing method can obtain useful results.

- Specialized equipment is required (compared to more conventional subsurface exploration tools).

- Since evaluation is performed at very low strains, or no strain at all, information regarding ultimate strength for evaluation of strength limit states is only obtained by correlation.

There are a number of different geophysical in-situ tests that can be used for stratigraphic information and determination of engineering properties. These methods can be combined with each other and/or combined with the in-situ tests presented in D10.4.4 to provide additional resolution and accuracy. ASTM D 6429, Standard Guide for Selecting Surface Geophysical Methods, provides additional guidance on selection of suitable methods.

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10.4.6 Selection of Design Properties

10.4.6.1 GENERAL

Soil and rock properties for use in the service limit state and strength limit state evaluation of foundations shall be based on the results of the field and/or laboratory testing methods described in Publication 293.

Subsurface soil or rock properties shall be determined using one or more of the following methods:

- In-situ testing during the field exploration program, including consideration of any geophysical testing conducted,
- Laboratory testing, and
- Back analysis of design parameters based on site performance data.

Local experience, local geologic formation specific property correlations, and knowledge of local geology, in addition to broader based experience and relevant published data, should also be considered in the final selection of design parameters. If published correlations are used in combination with one of the methods listed above, the applicability of the correlation to the specific geologic formation shall be considered through the use of local experience, local test results, and/or long-term experience.

The focus of geotechnical design property assessment and final selection shall be on the individual geologic strata identified at the project site.

The design values selected for the parameters should be appropriate to the particular limit state and its correspondent calculation model under consideration.

The determination of design parameters for rock shall take into consideration that rock mass properties are generally controlled by the discontinuities within the rock mass and not the properties of the intact material. Therefore, engineering properties for rock should account for the properties of the intact pieces and for the properties of the

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Soil and rock properties used in the design of foundations must represent the soil and rock mass as it will behave when subjected to loading by the actual foundation (i.e., use of γ, c and ϕ_f for undrained loading, and γ', c' and ϕ'_f for drained loading in equations for geotechnical resistance.). The depth of soil below a foundation which is within the zone of loading influence typically ranges from two to four times the minimum plan dimension of the foundation element. For soil or rock that is generally homogeneous within the zone of influence, properties associated with the intact material are appropriate. This condition, however, is not typical. Usually, soil or rock within the zone of influence varies highly in consistency and contains bedding planes or other depositional features, soft seams, joints, fractures, or other discontinuities which will govern the behavior of the ground mass under load. In such cases, it is the strength and deformation characteristics of these critical features which should be evaluated.

A geologic stratum is characterized as having the same geologic depositional history and stress history, and generally has similarities throughout the stratum in terms of density, source material, stress history, and hydrogeology. The properties of a given geologic stratum at a project site are likely to vary significantly from point to point within the stratum. In some cases, a measured property value may be closer in magnitude to the measured property value in an adjacent geologic stratum than to the measured properties at another point within the same stratum. However, soil and rock properties for design should not be averaged across multiple strata.

It should also be recognized that some properties, e.g., undrained shear strength in normally consolidated clays, may vary as a predictable function of a stratum dimension, e.g., depth below the top of the stratum. Where the property within the stratum varies in this manner, the design parameters should be developed taking this variation into account, which may result in multiple values of the property within the stratum as a function of a stratum dimension such as depth.

The observational method, or use of back analysis, to determine engineering properties of soil or rock is often used with slope failures, embankment settlement or excessive settlement of existing structures. With landslides or slope failures, the process generally starts with determining the geometry of the failure and then determining the soil/rock parameters or subsurface conditions that result from a combination of load and resistance factors that approach 1.0. Often the determination of the properties is aided by correlations with index tests or experience on other projects. For embankment settlement, a range of soil properties is generally determined based on

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rock mass as a whole, specifically considering the discontinuities within the rock mass. A combination of laboratory testing of small samples, empirical analysis, and field observations should be employed to determine the engineering properties of rock masses, with greater emphasis placed on visual observations and quantitative descriptions of the rock mass.

Refer to Publication 293 for guidance on behavior and engineering properties of soil and rock in the Commonwealth.

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laboratory performance testing on undisturbed samples. Monitoring of fill settlement and pore pressure in the soil during construction allows the soil properties and prediction of the rate of future settlement to be refined. For structures such as bridges that experience unacceptable settlement or retaining walls that have excessive deflection, the engineering properties of the soils can sometimes be determined if the magnitudes of the loads are known. As with slope stability analysis, the subsurface stratigraphy must be adequately known, including the history of the groundwater level at the site.

Local geologic formation-specific correlations may be used if well established by data comparing the prediction from the correlation to measured high quality laboratory performance data, or back-analysis from full scale performance of geotechnical elements affected by the geologic formation in question.

The Engineer should assess the variability of relevant data to determine if the observed variability is a result of inherent variability of subsurface materials and testing methods or if the variability is a result of significant variations across the site. Methods to compare soil parameter variability for a particular project to published values of variability based on database information of common soil parameters are presented in Sabatini (*2002*) and Duncan (*2000*). Where the variability is deemed to exceed the inherent variability of the material and testing methods, or where sufficient relevant data is not available to determine an average value and variability, the Engineer may perform a sensitivity analysis using average parameters and a parameter reduced by one standard deviation, i.e., “mean minus 1 sigma,” or a lower bound value. By conducting analyses at these two potential values, an assessment is made of the sensitivity of the analysis results to a range of potential design values. If these analyses indicate that acceptable results are provided and that the analyses are not particularly sensitive to the selected parameters, the Engineer may be comfortable with concluding the analyses. If, on the other hand, the Engineer determines that the calculation results are marginal or that the results are sensitive to the selected parameter, additional data collection/review and parameter selection are warranted.

When evaluating service limit states, it is often appropriate to determine both upper and lower bound values from the relevant data, since the difference in displacement of substructure units is often more critical to overall performance than the actual value of the displacement for the individual substructure unit.

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10.4.6.2 SOIL STRENGTH

10.4.6.2.1 General

The selection of soil shear strength for design should consider, at a minimum, the following:

- the rate of construction loading relative to the hydraulic conductivity of the soil, i.e., drained or undrained strengths;
- the effect of applied load direction on the measured shear strengths during testing;
- the effect of expected levels of deformation for the geotechnical structure; and
- the effect of the construction sequence.

C10.4.6.2.1

Refer to Sabatini et al. (*2002*) for additional guidance on determining which soil strength parameters are appropriate for evaluating a particular soil type and loading condition. In general, where loading is rapid enough and/or the hydraulic conductivity of the soil is low enough such that excess pore pressure induced by the loading does not dissipate, undrained (total) stress parameters should be used. Where loading is slow enough and/or the hydraulic conductivity of the soil is great enough such that excess pore pressures induced by the applied load dissipate as the load is applied, drained (effective) soil parameters should be used. Drained (effective) soil parameters should also be used to evaluate long term conditions where excess pore pressures have been allowed to dissipate or where the designer has explicit knowledge of the expected magnitude and distribution of the excess pore pressure.

10.4.6.2.2 Undrained Strength of Cohesive Soils

Where possible, laboratory consolidated undrained (CU) and unconsolidated undrained (UU) testing should be used to estimate the undrained shear strength, S_u, supplemented as needed with values determined from in-situ testing. Where collection of undisturbed samples for laboratory testing is difficult, values obtained from in-situ testing methods may be used. For relatively thick deposits of cohesive soil, profiles of S_u as a function of depth should be obtained so that the deposit stress history and properties can be ascertained.

C10.4.6.2.2

For design analyses of short-term conditions in normally to lightly overconsolidated cohesive soils, the undrained shear strength, S_u, is commonly evaluated. Since undrained strength is not a unique property, profiles of undrained strength developed using different testing methods will vary. Typical transportation project practice entails determination of S_u based on laboratory CU and UU testing and, for cases where undisturbed sampling is very difficult, field vane testing. Other in-situ methods can also be used to estimate the value of S_u.

Specific issues that should be considered when estimating the undrained shear strength are described below:

- Strength measurements from hand torvanes, pocket penetrometers, or unconfined compression tests should not be solely used to evaluate undrained shear strength for design analyses for bridges and walls with an exposed height greater than 3000 mm {10 ft.}. Consolidated undrained (CU) triaxial tests and in-situ tests should be used.
- For relatively deep deposits of cohesive soil, e.g., approximately 6000 mm {20 ft.} depth or more, all available undrained strength data should be plotted with depth. The type of test used to evaluate each undrained strength value should be clearly identified. Known soil layering should be used so that trends in undrained strength data can be developed for each soil layer.
- Review data summaries for each laboratory strength test method. Moisture contents of specimens for strength testing should be compared to moisture

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contents of other samples at similar depths. Significant changes in moisture content will affect measured undrained strengths. Review boring logs, Atterberg limits, grain size, and unit weight measurements to confirm soil layering.

- CU tests on normally to slightly over consolidated samples that exhibit disturbance should contain at least one specimen consolidated to at least $4\sigma_p'$ to permit extrapolation of the undrained shear strength at σ_p'.

- Undrained strengths from CU tests correspond to the effective consolidation pressure used in the test. This effective stress needs to be converted to the equivalent depth in the ground.

- A profile of σ_p’ (or OCR) should be developed and used in evaluating undrained shear strength.

- Correlations for S_u based on in-situ test measurements should not be used for final design for bridges and walls with an exposed height greater than 3000 mm {10 ft.} unless they have been calibrated to the specific soil profile under consideration. Correlations for S_u based on *SPT* tests should be avoided.

10.4.6.2.3 Drained Strength of Cohesive Soils

Long-term effective stress strength parameters, c' and ϕ'_f, of clays should be evaluated by slow consolidated drained direct shear box tests, consolidated drained (CD) triaxial tests, or consolidated undrained (CU) triaxial tests with pore pressure measurements. In laboratory tests, the rate of shearing should be sufficiently slow to ensure substantially complete dissipation of excess pore pressure in the drained tests or, in undrained tests, complete equalization of pore pressure throughout the specimen.

C10.4.6.2.3

The selection of peak, fully softened, or residual strength for design analyses should be based on a review of the expected or tolerable displacements of the soil mass.

The use of a nonzero cohesion intercept (c') for long-term analyses in natural materials must be carefully assessed. With continuing displacements, it is likely that the cohesion intercept value will decrease to zero for long-term conditions, especially for highly plastic clays.

Correlations for long-term effective strength parameters should not be used for bridges or retaining walls with an exposed height greater than 3000 mm {10 ft.}.

10.4.6.2.4 Drained Strength of Granular Soils

The drained friction angle of granular deposits should be evaluated by correlation to the results of *SPT* testing, *CPT* testing, or other relevant in-situ tests. Laboratory shear strength tests on undisturbed samples, if feasible to obtain, or reconstituted disturbed samples, may also be used to determine the shear strength of granular soils.

If *SPT N* values are used, unless otherwise specified for the design method or correlation being used, they shall be corrected for the effects of overburden pressure determined as:

$$N1 = C_N N \qquad (10.4.6.2.4\text{-}1)$$

$N1$ = *SPT* blow count corrected for overburden pressure,

C10.4.6.2.4

Because obtaining undisturbed samples of granular deposits for laboratory testing is extremely difficult, the results of in-situ tests are commonly used to develop estimates of the drained friction angle, ϕ_f. If reconstituted disturbed soil samples and laboratory tests are used to estimate the drained friction angle, the reconstituted samples should be compacted to the same relative density estimated from the available in-situ data. The test specimen should be large enough to allow the full grain size range of the soil to be included in the specimen. This may not always be possible, and if not possible, it should be recognized that the shear strength measured would likely be conservative.

A method using the results of *SPT* testing is presented. Other in-situ tests such as *CPT* and *DMT* may be used. For

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σ'_v (blows/300 mm){blows/ft.}

Metric Units:

C_N = [0.77 $\log_{10}$ (1.92/σ'_v)], and C_N < 2.0

U.S. Customary Units:

C_N =[0.77 $\log_{10}$ (40/σ'_v)], and C_N < 2.0

σ'_v = vertical effective stress (MPa){ksf}

N = uncorrected *SPT* blow count (blows/300 mm){blows/ft.}

SPT N values should also be corrected for hammer efficiency, if applicable to the design method or correlation being used, determined as:

$$N_{60} = (ER / 60\%)N \quad (10.4.6.2.4\text{-}2)$$

where:

N_{60} = *SPT* blow count corrected for hammer efficiency (blows/300 mm){blows/ft.}

ER = hammer efficiency expressed as percent of theoretical free fall energy delivered by the hammer system actually used

N = uncorrected *SPT* blow count (blows/300 mm){blows/ft.}

When *SPT* blow counts have been corrected for both overburden effects and hammer efficiency effects, the resulting corrected blow count shall be denoted as $N1_{60}$, determined as:

$$N1_{60} = C_N N_{60} \quad (10.4.6.2.4\text{-}3)$$

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details on determination of ϕ_f from these tests, refer to Sabatini et al. (*2002*).

The use of automatic trip hammers is increasing. In order to use correlations based on standard rope and cathead hammers, the *SPT N* values must be corrected to reflect the greater energy delivered to the sampler by these systems.

Hammer efficiency (ER) for specific hammer systems used in local practice may be used in lieu of the values provided. If used, specific hammer system efficiencies shall be developed in general accordance with ASTM D 4945 for dynamic analysis of driven piles or other accepted procedure.

The following values for ER may be assumed if hammer specific data are not available, e.g., from older boring logs:

ER = 60 percent for conventional drop hammer using rope and cathead

ER = 80 percent for automatic trip hammer

Corrections for rod length, hole size, and use of a liner may also be made if appropriate. In general, these are only significant in unusual cases or where there is significant variation from standard procedures. These corrections may be significant for evaluation of liquefaction. Information on these additional corrections may be found in *Youd and Idriss, 1997*.

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The drained friction angle of granular deposits should be determined based on the following correlation.

Table 10.4.6.2.4-1 Correlation of *SPT* $N1_{60}$ values to drained friction angle of granular soils (modified after *Bowles, 1977*)

$N1_{60}$	ϕ_f
<4	25–30
4-10	27–32
10-30	30–35
30-50	35–40
> 50	38–43

For gravels and rock fill materials where *SPT* testing is not reliable, Figure 1 should be used to estimate the drained friction angle.

Metric Units:

Rock Fill Grade	Particle Unconfined Compressive Strength (MPa)
A	>220
B	165 to 220
C	125 to 165
D	80 to 125
E	≤80

U.S. Customary Units:

Rock Fill Grade	Particle Unconfined Compressive Strength (ksf)
A	>4610
B	3460– 4610
C	2590– 3460
D	1730– 2590
E	≤1730

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The $N1_{60}$-ϕ_f correlation used is modified after Bowles (*1977*). The correlation of Peck, Hanson and Thornburn (*1974*) falls within the ranges specified. Experience should be used to select specific values within the ranges. In general, finer materials or materials with significant silt-sized material will fall in the lower portion of the range. Coarser materials with less than 5 percent fines will fall in the upper portion of the ranges. The geologic history and angularity of the particles may also need to be considered when selecting a value for ϕ_f.

Care should be exercised when using other correlations of *SPT* results to soil parameters. Some published correlations are based on corrected values ($N1_{60}$) and some are based on uncorrected values (N). The designer should ascertain the basis of the correlation and use either $N1_{60}$ or N as appropriate.

Care should also be exercised when using *SPT* blow counts to estimate soil shear strength if in soils with coarse gravel, cobbles, or boulders. Large gravels, cobbles, or boulders could cause the *SPT* blow counts to be unrealistically high.

The secant friction angle derived from the procedure to estimate the drained friction angle of gravels and rock fill materials depicted in Figure 1 is based on a straight line from the origin of a Mohr diagram to the intersection with the strength envelope at the effective normal stress. Thus the angle derived is applicable only to analysis of field conditions subject to similar normal stresses. See Terzaghi, Peck, and Mesri (*1996*) for additional details regarding this procedure.

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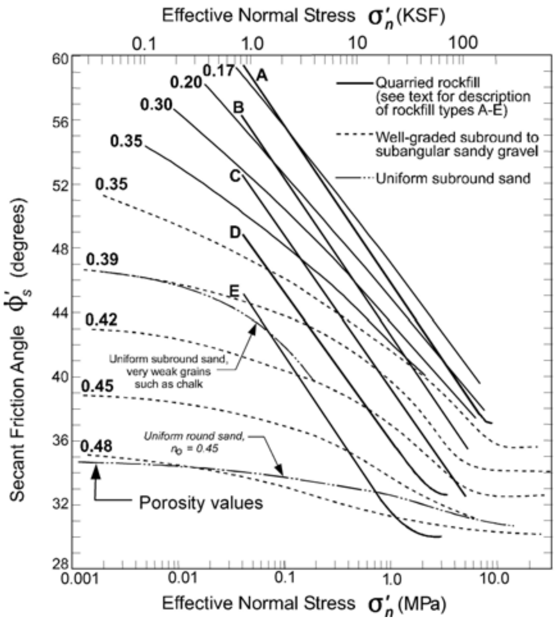

Figure 10.4.6.2.4-1 Estimation of drained friction angle of gravels and rock fills (modified after *Terzaghi, Peck, and Mesri, 1996*).

10.4.6.3 SOIL DEFORMATION

C10.4.6.3

Consolidation parameters C_c, C_r, C_α should be determined from the results of one-dimensional consolidation tests. To assess the potential variability in the settlement estimate, the average, upper and lower bound values obtained from testing should be considered.

It is important to understand whether the values obtained are computed based on a void ratio definition or a strain definition. Computational methods vary for each definition.

For preliminary analyses or where accurate prediction of settlement is not critical, values obtained from correlations to index properties may be used. Refer to Sabatini et al. (*2002*) for discussion of the various correlations available. If correlations for prediction of settlement are used, their applicability to the specific geologic formation under consideration should be evaluated.

Preconsolidation stress may be determined from one-dimensional consolidation tests and in-situ tests. Knowledge of the stress history of the soil should be used to supplement data from laboratory and/or in-situ tests, if available.

A profile of σ_p', or OCR = σ_p'/σ_o', with depth should be developed for the site for design applications where the stress history could have a significant impact on the design properties selected and the performance of the foundation. As with consolidation properties, an upper and lower bound profile should be developed based on laboratory tests and plotted with a profile based on particular in-situ test(s), if used. It is particularly important to accurately compute preconsolidation stress values for relatively shallow depths where in-situ effective stresses are low. An underestimation of the preconsolidation stress at shallow depths will result in overly conservative estimates of settlement for shallow soil layers.

The coefficient of consolidation, c_v, should be determined from the results of one-dimensional consolidation tests. The variability in laboratory determination of c_v results should be considered in the final

Due to the numerous simplifying assumptions associated with conventional consolidation theory, on which the coefficient of consolidation is based, it is unlikely that even the best estimates of c_v from high-quality laboratory

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selection of the value of c_v to be used for design.

Where evaluation of elastic settlement is critical to the design of the foundation or selection of the foundation type, in-situ methods such as PMT or *DMT* for evaluating the modulus of the stratum should be used.

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tests will result in predictions of time rate of settlement in the field that are significantly better than a prediction within one order of magnitude. In general, the in-situ value of c_v is larger than the value measured in the laboratory test. Therefore, a rational approach is to select average, upper, and lower bound values for the appropriate stress range of concern for the design application. These values should be compared to values obtained from previous work performed in the same soil deposit. Under the best-case conditions, these values should be compared to values computed from measurements of excess pore pressures or settlement rates during construction of other structures.

CPTu tests in which the pore pressure dissipation rate is measured may be used to estimate the field coefficient of consolidation.

For preliminary analyses or where accurate prediction of settlement is not critical, values obtained from correlations to index properties presented in Sabatini et al. (*2002*) may be used.

For preliminary design or for final design where the prediction of deformation is not critical to structure performance, i.e., the structure design can tolerate the potential inaccuracies inherent in the correlations. The elastic properties (E_s, ν) of a soil may be estimated from empirical relationships presented in Table C1.

The specific definition of E_s is not always consistent for the various correlations and methods of in-situ measurement. See Sabatini et al. (*2002*) for additional details regarding the definition and determination of E_s.

An alternative method of evaluating the equivalent elastic modulus using measured shear wave velocities is presented in Sabatini et al. (*2002*).

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Table C10.4.6.3-1 Elastic Constants of Various Soils (Modified after U.S. *Department of the Navy, 1982,* and *Bowles, 1988*).

Metric Units:

Soil Type	Typical Range of Young's Modulus Values, E_s (MPa)	Poisson's Ratio, ν (dim.)
Clay:		
Soft sensitive	2.4–15	0.4–0.5 (undrained)
Medium stiff to stiff	15–50	
Very stiff	50–100	
Loess	15–60	0.1–0.3
Silt	2–20	0.3–0.35
Fine Sand:		
Loose	7.5–10	0.25
Medium dense	10–20	
Dense	20–25	
Sand:		
Loose	10–25	0.20–0.36
Medium dense	25–50	
Dense	50–75	0.30–0.40
Gravel:		
Loose	25–75	0.20–0.35
Medium dense	75–100	
Dense	100–200	0.30–0.40

Estimating E_s from *SPT N* Value	
Soil Type	E_s (MPa)
Silts, sandy silts, slightly cohesive mixtures	$0.4\ N1_{60}$
Clean fine to medium sands and slightly silty sands	$0.7\ N1_{60}$
Coarse sands and sands with little gravel	$1.0\ N1_{60}$
Sandy gravel and gravels	$1.1\ N1_{60}$

Estimating E_s from q_c (static cone resistance)	
Sandy soils	$4q_c$

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Table C10.4.6.3-1 Elastic Constants of Various Soils (Modified after U.*S. Department of the Navy, 1982,* and *Bowles, 1988*). (Continued)

U.S. Customary Units:

Soil Type	Typical Range of Young's Modulus Values, E_s (ksi)	Poisson's Ratio, ν (dim.)
Clay: Soft sensitive Medium stiff to stiff Very stiff	 0.347–2.08 2.08–6.94 6.94–13.89	0.4–0.5 (undrained)
Loess Silt	2.08–8.33 0.278–2.78	0.1–0.3 0.3–0.35
Fine Sand: Loose Medium dense Dense	 1.11–1.67 1.67–2.78 2.78–4.17	0.25
Sand: Loose Medium dense Dense	 1.39–4.17 4.17–6.94 6.94–11.11	 0.20–0.36 0.30–0.40
Gravel: Loose Medium dense Dense	 4.17–11.11 11.11–13.89 13.89–27.78	 0.20–0.35 0.30–0.40

Estimating E_s from *SPT N* Value	
Soil Type	E_s (ksi)
Silts, sandy silts, slightly cohesive mixtures	$0.056\ N1_{60}$
Clean fine to medium sands and slightly silty sands	$0.097\ N1_{60}$
Coarse sands and sands with little gravel	$0.139\ N1_{60}$
Sandy gravel and gravels	$0.167\ N1_{60}$
Estimating E_s from q_c (static cone resistance)	
Sandy soils	$0.028q_c$

The modulus of elasticity for normally consolidated granular soils tends to increase with depth. An alternative method of defining the soil modulus for granular soils is to assume that it increases linearly with depth starting at zero at the ground surface in accordance with the following equation:

$$E_s = n_h \times z \tag{C10.4.6.3-1}$$

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where:

E_s = the soil modulus at depth z (MPa){ksi}
n_h = rate of increase of soil modulus with depth as defined in Table C2 (MPa/mm){ksi/ft.}
z = depth below the ground surface (mm){ft.}

Table C10.4.6.3-2 Rate of Increase of Soil Modulus with Depth n_h (MPa/mm) for Sand.

Metric Units:

Consistency	Dry or Moist	Submerged
Loose	9.4×10^{-3}	4.7×10^{-3}
Medium	0.025	0.013
Dense	0.063	0.031

U.S. Customary Units:

Consistency	Dry or Moist	Submerged
Loose	0.417	0.208
Medium	1.11	0.556
Dense	2.78	1.39

The potential for soil swell that may result in uplift on deep foundations or heave of shallow foundations should be evaluated based on Table 1.

The formulation provided in Eq. C1 is used primarily fc analysis of lateral response or buckling of deep foundations.

Table 10.4.6.3-1 Method for Identifying Potentially Expansive Soils (*Reese and O'Neill, 1988*).

Metric Units:

Liquid Limit *LL* (%)	Plastic Limit *PL* (%)	Soil Suction (MPa)	Potential Swell (%)	Potential Swell Classification
>60	>35	>0.38	>1.5	High
50–60	25–35	0.14–0.38	0.5–1.5	Marginal
<50	<25	<0.14	<0.5	Low

U.S. Customary Units:

Liquid Limit *LL* (%)	Plastic Limit *PL* (%)	Soil Suction (ksf)	Potential Swell (%)	Potential Swell Classification
>60	>35	>8	>1.5	High
50–60	25–35	3–8	0.5–1.5	Marginal
<50	<25	<3	<0.5	Low

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10.4.6.4 ROCK MASS STRENGTH

The strength of intact rock material should be determined using the results of unconfined compression tests on intact rock cores, splitting tensile tests on intact rock cores, or point load strength tests on intact specimens of rock.

The rock should be classified using the rock mass rating system (RMR) as described in Table 1. For each of the five parameters in the table, the relative rating based on the ranges of values provided should be evaluated. The rock mass rating (RMR) should be determined as the sum of all five relative ratings. The RMR should be adjusted in accordance with the criteria in Table 2. The rock classification should be determined in accordance with Table 3.

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Because of the importance of the discontinuities in rock, and the fact that most rock is much more discontinuous than soil, emphasis is placed on visual assessment of the rock and the rock mass.

Other methods for assessing rock mass strength, including in-situ tests or other visual systems that have proven to yield accurate results may be used in lieu of the specified method.

Use RMR as presented in Tables 1 and 2. Other versions of RMR are present in the literature should not be substituted.

The rating adjustment for joint orientations presented in Table 2 requires a consideration of the effect of the joint orientation relative to the applied loading direction relative to the proposed construction. For example, for slope stability, an unfavorable orientation would be in a direction parallel to the load direction or inclined and dipping towards the open face. For settlement, an unfavorable orientation of a gouge-filled joint would be at right angles to the load direction resulting in closure of discontinuities and settlement.

Table 10.4.6.4-1 Geomechanics Classification of Rock Masses.

Metric Units:

<table>
<tr><th colspan="3">PARAMETER</th><th colspan="7">RANGES OF VALUES</th></tr>
<tr><td rowspan="3">1</td><td rowspan="2">Strength of intact rock material</td><td>Point load strength index</td><td>>8 MPa</td><td>4–8 MPa</td><td>2–4 MPa</td><td>1–2 MPa</td><td colspan="3">For this low range, uniaxial compressive test is preferred</td></tr>
<tr><td>Uniaxial compressive strength</td><td>>200 MPa</td><td>100–200 MPa</td><td>50–100 MPa</td><td>25–50 MPa</td><td>10–25 MPa</td><td>3.5–10 MPa</td><td>1.0–3.5 MPa</td></tr>
<tr><td colspan="2">Relative Rating</td><td>15</td><td>12</td><td>7</td><td>4</td><td>2</td><td>1</td><td>0</td></tr>
<tr><td rowspan="2">2</td><td colspan="2">Drill core quality RQD</td><td colspan="2">90% to 100%</td><td>75% to 90%</td><td>50% to 75%</td><td colspan="2">25% to 50%</td><td><25%</td></tr>
<tr><td colspan="2">Relative Rating</td><td colspan="2">20</td><td>17</td><td>13</td><td colspan="2">8</td><td>3</td></tr>
<tr><td rowspan="2">3</td><td colspan="2">Spacing of joints</td><td colspan="2">>3000 mm</td><td>900–3000 mm</td><td>300–900 mm</td><td colspan="2">50–300 mm</td><td><50 mm</td></tr>
<tr><td colspan="2">Relative Rating</td><td colspan="2">30</td><td>25</td><td>20</td><td colspan="2">10</td><td>5</td></tr>
<tr><td rowspan="2">4</td><td colspan="2">Condition of joints</td><td colspan="2">• Very rough surfaces
• Not continuous
• No separation
• Hard joint wall rock</td><td>• Slightly rough surfaces
• Separation <1.25 mm
• Hard joint wall rock</td><td>• Slightly rough surfaces
• Separation <1.25 mm
• Soft joint wall rock</td><td colspan="2">• Slicken-sided surfaces
or
• Gouge <5 mm thick
or
• Joints open 1.25–5 mm
• Continuous joints</td><td>• Soft gouge >5 mm thick
or
• Joints open >5 mm
• Continuous joints</td></tr>
<tr><td colspan="2">Relative Rating</td><td colspan="2">25</td><td>20</td><td>12</td><td colspan="2">6</td><td>0</td></tr>
<tr><td rowspan="4">5</td><td rowspan="3">Ground water conditions
(use one of the three evaluation criteria as appropriate to the method of exploration)</td><td>Inflow per 10 000 mm tunnel length</td><td colspan="2">None</td><td colspan="2"><25 L/min.</td><td colspan="2">25–125 L/min.</td><td>>125 L/min.</td></tr>
<tr><td>Ratio = joint water pressure/ major principal stress</td><td colspan="2">0</td><td colspan="2">0.0–0.2</td><td colspan="2">0.2–0.5</td><td>>0.5</td></tr>
<tr><td>General Conditions</td><td colspan="2">Completely Dry</td><td colspan="2">Moist only (interstitial water)</td><td colspan="2">Water under moderate pressure</td><td>Severe water problems</td></tr>
<tr><td colspan="2">Relative Rating</td><td colspan="2">10</td><td colspan="2">7</td><td colspan="2">4</td><td>0</td></tr>
</table>

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U.S. Customary Units:

	Parameter		Ranges of Values						
1	Strength of intact rock material	Point load strength index	>175 ksf	85–175 ksf	45–85 ksf	20–45 ksf	For this low range, uniaxial compressive test is preferred		
		Uniaxial compressive strength	>4320 ksf	2160–4320 ksf	1080–2160 ksf	520–1080 ksf	215–520 ksf	70–215 ksf	20–70 ksf
	Relative Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90% to 100%	75% to 90%	50% to 75%	25% to 50%	<25%		
	Relative Rating		20	17	13	8	3		
3	Spacing of joints		>10 ft.	3–10 ft.	1–3 ft.	2 in.–1 ft.	<2 in.		
	Relative Rating		30	25	20	10	5		
4	Condition of joints		• Very rough surfaces • Not continuous • No separation • Hard joint wall rock	• Slightly rough surfaces • Separation <0.05 in. • Hard joint wall rock	• Slightly rough surfaces • Separation <0.05 in. • Soft joint wall rock	• Slicken-sided surfaces or • Gouge <0.2 in. thick or • Joints open 0.05–0.2 in. • Continuous joints	• Soft gouge >0.2 in. thick or • Joints open >0.2 in. • Continuous joints		
	Relative Rating		25	20	12	6	0		
5	Groundwater conditions (use one of the three evaluation criteria as appropriate to the method of exploration)	Inflow per 30 ft. tunnel length	None	<400 gal./hr.	400–2000 gal./hr.	>2000 gal./hr.			
		Ratio = joint water pressure/ major principal stress	0	0.0–0.2	0.2–0.5	>0.5			
		General Conditions	Completely Dry	Moist only (interstitial water)	Water under moderate pressure	Severe water problems			
	Relative Rating		10	7	4	0			

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Table 10.4.6.4-2 Geomechanics Rating Adjustment for Joint Orientations.

Strike and Dip Orientations of Joints		Very Favorable	Favorable	Fair	Unfavorable	Very Unfavorable
Ratings	Tunnels	0	–2	–5	–10	-12
	Foundations	0	–2	–7	–15	-25
	Slopes	0	–5	–25	–50	-60

Table 10.4.6.4-3 Geomechanics Rock Mass Classes Determined From Total Ratings.

RMR Rating	100–81	80–61	60–41	40–21	<20
Class No.	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

The shear strength of fractured rock masses should be evaluated using the Hoek and Brown criteria, in which the shear strength is represented as a curved envelope that is a function of the uniaxial compressive strength of the intact rock, q_u, and two dimensionless constants m and s. The values of m and s as defined in Table 4 should be used.

The shear strength of the rock mass should be determined as:

$$\tau = \left(\cot\ \phi_i' - \cos\ \phi_i'\right) m\ \frac{q_u}{8} \qquad (10.4.6.4\text{-}1)$$

in which:

$$\phi_i' = \tan^{-1}\left\{4h\ \cos^2\left[30 + 0.33\ \sin^{-1}\left(h^{-\frac{3}{2}}\right)\right] - 1\right\}^{-\frac{1}{2}}$$

$$h = 1 + \frac{16\left(m\sigma_n' + sq_u\right)}{(3m^2 q_u)}$$

where:

τ = the shear strength of the rock mass (MPa){ksf}

ϕ'_i = the instantaneous friction angle of the rock mass (degrees)

q_u = average unconfined compressive strength of intact rock core (MPa){ksf}

σ'_n = effective normal stress (MPa){ksf}

m, s = constants from Table 4 (dim.)

This method was developed by Hoek (*1983*) and Hoek and Brown (*1988, 1997*). Note that the instantaneous cohesion at a discrete value of normal stress can be taken as:

$$c_i = \tau - \sigma_n'\ \tan\phi_i' \qquad (C10.4.6.4\text{-}1)$$

The instantaneous cohesion and instantaneous friction angle define a conventional linear Mohr envelope at the normal stress under consideration. For normal stresses significantly different than that used to compute the instantaneous values, the resulting shear strength will be unconservative. If there is considerable variation in the effective normal stress in the zone of concern, consideration should be given to subdividing the zone into areas where the normal stress is relative constant and assigning separate strength parameters to each zone. Alternatively, the methods of Hoek (*1983*) may be used to compute average values for the range of normal stresses expected.

Use caution in using RMF values when applying Hoek and Brown Criteria to poor to very poor quality rock masses. The methods of Hoek (2000), Chapter 11 utilizing the Geological Strength Index (GSI) can be used for these conditions.

Disturbance from stress relief and method of excavation can also impact the shear strength of rock masses. See Hoek (2002) for a modification of the Hoek and Brown criteria for disturbance.

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Table 10.4.6.4-4 Approximate relationship between rock-mass quality and material constants used in defining nonlinear strength (Hoek and Brown, 1988)

<table>
<tr><td rowspan="2">Rock Quality</td><td rowspan="2">Constants</td><td colspan="5">Rock Type

A = Carbonate rocks with well developed crystal cleavage—dolomite, limestone and marble
B = Lithified argrillaceous rocks—mudstone, siltstone, shale and slate (normal to cleavage)
C = Arenaceous rocks with strong crystals and poorly developed crystal cleavage—sandstone and quartzite
D = Fine grained polyminerallic igneous crystalline rocks—andesite, dolerite, diabase and rhyolite
E = Coarse grained polyminerallic igneous & metamorphic crystalline rocks—amphibolite, gabbro gneiss, granite, norite, quartz-diorite</td></tr>
<tr><td>A</td><td>B</td><td>C</td><td>D</td><td>E</td></tr>
<tr><td>INTACT ROCK SAMPLES
Laboratory size specimens free from discontinuities
CSIR rating: RMR = 100</td><td>m
s</td><td>7.00
1.00</td><td>10.00
1.00</td><td>15.00
1.00</td><td>17.00
1.00</td><td>25.00
1.00</td></tr>
<tr><td>VERY GOOD QUALITY ROCK MASS
Tightly interlocking undisturbed rock with unweathered joints at 900–3000 mm {3-10 ft.}
CSIR rating: RMR = 85</td><td>m
s</td><td>2.40
0.082</td><td>3.43
0.082</td><td>5.14
0.082</td><td>5.82
0.082</td><td>8.567
0.082</td></tr>
<tr><td>GOOD QUALITY ROCK MASS
Fresh to slightly weathered rock, slightly disturbed with joints at 900–3000 mm {3-10 ft.}
CSIR rating: RMR = 65</td><td>m
s</td><td>0.575
0.00293</td><td>0.821
0.00293</td><td>1.231
0.00293</td><td>1.395
0.00293</td><td>2.052
0.00293</td></tr>
<tr><td>FAIR QUALITY ROCK MASS
Several sets of moderately weathered joints spaced at 300—900 mm {1-3 ft.}
CSIR rating: RMR = 44</td><td>m
s</td><td>0.128
0.00009</td><td>0.183
0.00009</td><td>0.275
0.00009</td><td>0.311
0.00009</td><td>0.458
0.00009</td></tr>
<tr><td>POOR QUALITY ROCK MASS
Numerous weathered joints at 50–300 mm {2-12 in.}; some gouge. Clean compacted waste rock.
CSIR rating: RMR = 23</td><td>m
s</td><td>0.029
3×10^{-6}</td><td>0.041
3×10^{-6}</td><td>0.061
3×10^{-6}</td><td>0.069
3×10^{-6}</td><td>0.102
3×10^{-6}</td></tr>
<tr><td>VERY POOR QUALITY ROCK MASS
Numerous heavily weathered joints spaced <50 mm {2 in} with gouge. Waste rock with fines.
CSIR rating: RMR = 3</td><td>m
s</td><td>0.007
1×10^{-7}</td><td>0.010
1×10^{-7}</td><td>0.015
1×10^{-7}</td><td>0.017
1×10^{-7}</td><td>0.025
1×10^{-7}</td></tr>
</table>

Where it is necessary to evaluate the strength of a single discontinuity or set of discontinuities, the strength along the discontinuity should be determined as follows:

- For smooth discontinuities, the shear strength is represented by a friction angle of the parent rock

The range of typical friction angles provided in Table C1 may be used in evaluating measured values of friction angles for smooth joints.

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material. To evaluate the friction angle of this type of discontinuity surface for design, direct shear tests on samples should be performed. Samples should be formed in the laboratory by cutting samples of intact core.

- For rough discontinuities the nonlinear criterion of Barton (*1976*) should be applied.

10.4.6.5 ROCK MASS DEFORMATION

The elastic modulus of a rock mass (E_m) shall be taken as the lesser of the intact modulus of a sample of rock core (E_i) or the modulus estimated by multiplying the intact rock modulus, E_i, obtained from uniaxial compression tests by a reduction factor, α_E, which accounts for the frequency of discontinuities by the rock quality designation (RQD), using the following relationship (*Gardner 1987*):

$$E_m = \alpha_E E_i \qquad (10.4.6.5\text{-}1)$$

in which:

$$\alpha_E = 0.0231(RQD) - 1.32 \geq 0.15 \qquad (10.4.6.5\text{-}2)$$

The magnitude of consolidation and secondary settlements in rock masses containing soft seams or other

COMMENTARY

Table C10.4.6.4-1. Typical ranges of friction angles for smooth joints in a variety of rock types (Modified after Barton, 1976; Jaeger and Cook, 1976)

Rock Class	Friction Angle Range	Typical Rock Types
Low Friction	20–27°	Schists (high mica content), shale, marl
Medium Friction	27–34°	Sandstone, siltstone, chalk, gneiss, slate
High Friction	34–40°	Basalt, granite, limestone, conglomerate

Note: Values assume no infilling and little relative movement between joint faces.

When a major discontinuity with a significant thickness of infilling is to be investigated, the shear strength will be governed by the strength of the infilling material and the past and expected future displacement of the discontinuity. Refer to Sabatini et al. (*2002*) for detailed procedures to evaluate infilled discontinuities.

C10.4.6.5

Preliminary estimates of the elastic modulus of intact rock may be made from Table C1. Note that some of the rock types identified in the table are not present in the U.S.

It is extremely important to use the elastic modulus of the rock mass for computation of displacements of rock materials under applied loads. Use of the intact modulus will result in unrealistic and unconservative estimates.

For preliminary design or when site-specific test data cannot be obtained, guidelines for estimating values of E_i, such as those presented in Table C1, may be used. For preliminary analyses or for final design when in-situ test results are not available, a value of α_E = 0.15 should be used to estimate E_m.

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material with time-dependent settlement characteristics may be estimated by applying procedures specified in D10.6.2.4.3.

Poisson's ratio for rock should be determined from tests on intact rock core.

COMMENTARY

Where tests on rock core are not practical, Poisson's ratio may be estimated from Table C2.

Table 10.4.6.5-1 Summary of Elastic Moduli for Intact Rock (Modified after Kulhawy, 1978).

Metric Units:

Rock Type	No. of Values	No. of Rock Types	Elastic Modulus, E_i (MPa × 10^3)			Standard Deviation (MPa × 10^3)
			Maximum	Minimum	Mean	
Granite	26	26	100.0	6.410	52.70	24.48
Diorite	3	3	112.0	17.100	51.40	42.68
Gabbro	3	3	84.1	67.600	75.80	6.69
Diabase	7	7	104.0	69.000	88.30	12.27
Basalt	12	12	84.1	29.000	56.10	17.93
Quartzite	7	7	88.3	36.500	66.10	16.00
Marble	14	13	73.8	4.000	42.60	17.17
Gneiss	13	13	82.1	28.500	61.10	15.93
Slate	11	2	26.1	2.410	9.58	6.62
Schist	13	12	69.0	5.930	34.30	21.93
Phyllite	3	3	17.3	8.620	11.80	3.93
Sandstone	27	19	39.2	0.620	14.70	8.20
Siltstone	5	5	32.8	2.620	16.50	11.38
Shale	30	14	38.6	0.007	9.79	10.00
Limestone	30	30	89.6	4.480	39.30	25.72
Dolostone	17	16	78.6	5.720	29.10	23.72

U.S. Customary Units:

Rock Type	No. of Values	No. of Rock Types	Elastic Modulus, E_i (ksi $\times 10^3$)			Standard Deviation (ksi $\times 10^3$)
			Maximum	Minimum	Mean	
Granite	26	26	14.5	0.93	7.64	3.55
Diorite	3	3	16.2	2.48	7.45	6.19
Gabbro	3	3	12.2	9.8	11.0	0.97
Diabase	7	7	15.1	10.0	12.8	1.78
Basalt	12	12	12.2	4.20	8.14	2.60
Quartzite	7	7	12.8	5.29	9.59	2.32
Marble	14	13	10.7	0.58	6.18	2.49
Gneiss	13	13	11.9	4.13	8.86	2.31
Slate	11	2	3.79	0.35	1.39	0.96
Schist	13	12	10.0	0.86	4.97	3.18
Phyllite	3	3	2.51	1.25	1.71	0.57
Sandstone	27	19	5.68	0.09	2.13	1.19
Siltstone	5	5	4.76	0.38	2.39	1.65
Shale	30	14	5.60	0.001	1.42	1.45
Limestone	30	30	13.0	0.65	5.7	3.73
Dolostone	17	16	11.4	0.83	4.22	3.44

Table C10.4.6.5-2 Summary of Poisson's Ratio for Intact Rock (Modified after Kulhawy, 1978)

Rock Type	No. of Values	No. of Rock Types	Poisson's Ratio, ν			Standard Deviation
			Maximum	Minimum	Mean	
Granite	22	22	0.39	0.09	0.20	0.08
Gabbro	3	3	0.20	0.16	0.18	0.02
Diabase	6	6	0.38	0.20	0.29	0.06
Basalt	11	11	0.32	0.16	0.23	0.05
Quartzite	6	6	0.22	0.08	0.14	0.05
Marble	5	5	0.40	0.17	0.28	0.08
Gneiss	11	11	0.40	0.09	0.22	0.09
Schist	12	11	0.31	0.02	0.12	0.08
Sandstone	12	9	0.46	0.08	0.20	0.11
Siltstone	3	3	0.23	0.09	0.18	0.06
Shale	3	3	0.18	0.03	0.09	0.06
Limestone	19	19	0.33	0.12	0.23	0.06
Dolostone	5	5	0.35	0.14	0.29	0.08

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10.4.6.6 ERODIBILITY OF ROCK

Consideration should be given to the physical characteristics of the rock and the condition of the rock mass when determining a rock's susceptibility to erosion in the vicinity of bridge foundations. Physical characteristics that should be considered in the assessment of erodibility include cementing agents, mineralogy, joint spacing, and weathering.

COMMENTARY

C10.4.6.6

There is no consensus on how to determine erodibility of rock masses near bridge foundations. Refer to Richardson and Davis (*2001*) "Evaluating Scour at Bridges-Fourth Edition", Mayne et al. (*2001*), Appendix M for guidance on two proposed methods. The first method was proposed in an FHWA memorandum of July 1991 and consists of evaluating various rock index properties. The second method is documented in Smith (*1994*) "Preliminary Procedure to Evaluate Scour in Bedrock" which uses the erodibility index proposed by G. W. Annandale. The Engineer should consider the appropriateness of these two methods when determining the potential for a rock mass to scour.

10.4.7 Special Soil, Rock and Other Problem Conditions

Geologic and environmental conditions can influence the performance of foundations and may require special consideration during design. To the extent possible, the presence and influence of such conditions shall be evaluated

As part of the subsurface exploration program. A representative, but not exclusive, listing of problem conditions requiring special consideration is presented in Table 1 for general guidance.

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Table 10.4.7-1 - Problem Conditions Requiring Special Consideration

PROBLEM TYPE	DESCRIPTION	COMMENTS
Soil	Organic Soil; Highly Plastic Clay	Low strength and high compressibility
	Sensitive Clay	Potentially large strength loss upon large straining
	Micaceous Soil	Potentially high compressibility (often saprolitic)
	Expansive Clay/Silt; Expansive Slag	Potentially large expansion upon wetting
	Liquefiable Soil	Complete strength loss and high deformations due to earthquake loading
	Collapsible Soil	Potentially large deformations upon wetting (caliche; loess)
	Pyritic Soil	Potentially large expansion upon oxidation, potentially corrosive environment
Rock	Laminated Rock	Low strength when loaded parallel to bedding
	Expansive Shale	Potentially large expansion upon wetting; degrades readily upon exposure to air/water
	Pyritic Shale	Expands upon exposure to air/water, degrades readily upon exposure, potentially corrosive environment
	Soluble Rock	Soluble in flowing and standing water (limestone, dolostone, gypsum)
	Weak Claystone (Red Beds)	Low strength and readily degradable upon exposure to air/water
	Gneissic and Schistose Rock	Highly distorted with irregular weathering profiles and steep discontinuities
Condition	Subsidence	Typical in areas of underground mining or high groundwater extraction
	Sinkholes/Solutioning	Karst topography; typical of areas underlain by carbonate rock strata
	Negative Skin Friction/Expansion Loading	Additional compressive/uplift load on deep foundations due to settlement/uplift of soil
	Corrosive Environments	Acid mine drainage; degradation of certain soil/rock types
	Permafrost/Frost	Typical in northern climates
	Capillary Water	Rise of water level in silts and fine sands leading to strength loss

See Publication 293M for conditions typical of the Commonwealth of Pennsylvania for preliminary identification of the potential for special problem conditions at a site.

10.4.7.1 PROBLEM SOILS

10.4.7.1.1 Organic Soils

Organic soils, such as peats and organic silts and clays, are common to most lacustrine, estuarine, and fluvial environments. These soils exhibit low strength and excessive deformability. Excessive deformability can result in large settlements which may place additional downward forces (i.e., negative skin friction) on piles and drilled shafts.

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10.4.7.1.2 Sensitive Clays

Sensitive clays exhibit significant loss of strength due to disturbance. Poor construction practice in sensitive clays can result in additional downward forces (i.e., negative skin friction) on piles and drilled shafts due to excessive settlement of the surrounding soil relative to the pile or shaft, and in reduced axial and lateral load capacity due to soil strength loss.

10.4.7.1.3 Micaceous Soils

Micaceous soils result from the in-place weathering of igneous and metamorphic rocks. The presence of mica in the soil matrix is indicative of lower strength and higher compressibility than similar soils without mica. If micaceous soils are present, laboratory and/or field testing shall be performed to evaluate the soil strength and compressibility.

10.4.7.2 PROBLEM ROCKS

10.4.7.2.1 Shales

Certain shales exhibit little frictional resistance along natural bedding planes and can be unstable if loaded other than normal to the bedding surfaces. Some shales expand markedly when unloaded and exposed to the air due to absorption of water by the clay minerals. Other shale types, pyritic shales for instance, expand during chemical weathering of the rock. In addition, sulfuric acid formed during the oxidation of the pyrite deteriorates concrete because of its deleterious effect on cement. When these conditions are anticipated or encountered, provisions shall be taken to minimize their impact on foundation performance, including extending foundations to greater depths, minimizing the time the foundation excavation remains open during construction, and using sulfate resistance cement.

10.4.7.2.2 Soluble Rocks

Open channels and joints, caverns, sinkholes, discontinuities and irregular top of rock topography are characteristic of soluble carbonate rock types (e.g., limestone and dolostone) where special attention shall be given to the potential for solution features. When soluble rock formations are encountered, consideration shall be given to extending drilled shafts or micropiles below the soluble formations and to the potential for additional lateral and downward loading on the shaft or micropile due to collapsed rock. Publication 408 requires probing a minimum depth of ten times the shaft diameter below the tip of each drilled shaft prior to concreting.

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10.4.7.2.3 Gneissic and Schistose Rocks

Gneissic and schistose rock formations are the consequence of metamorphic geologic processes that result in banded and distorted strata and irregular weathering profiles. In designing drilled shafts in these rock types, consideration shall be given to defining the top of rock, the depth of weathering and the potential for instability where the tip of the shaft is installed in steeply dipping strata.

10.4.7.3 OTHER PROBLEM CONDITIONS

10.4.7.3.1 Mine Voids

Subsidence or caving of mine voids can lead to extreme and unpredictable settlements. Total collapse of a foundation may occur if an underlying mine void goes undetected during the site subsurface exploration. Slumping soil about a drilled shaft can cause additional loading on the shaft similar to the effects of negative skin friction where a shaft is installed through compressible soils.

10.4.7.3.2 Corrosive Environments

The oxidation of pyrite in pyritic shales can create a corrosive environment for concrete by formation of sulfuric acid. Other sources of sulfate species are soils, groundwater, coal and acid mine drainage or industrial runoff. Use of sulfate-resistant cement in the concrete or a high quality watertight concrete shall be considered when deterioration from sulfate is a potential problem.

Large concentrations of chlorides are sometimes present in groundwater, soils and industrial runoff, and may contribute to the corrosion of steel. When sulfur species are present in combination with chlorides, the corrosive process is accelerated. When these conditions are encountered, consideration shall be given to protecting the reinforcing steel with additional concrete cover, using a greater cross-sectional area of reinforcement to compensate for long-term loss, or using high-strength reinforcing which provides greater corrosion resistance than mild steel.

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10.5 LIMIT STATES AND RESISTANCE FACTORS

10.5.1 General

The limit states shall be as specified in A1.3.2 and D1.3.2; foundation-specific provisions are contained in this Section.

Foundations shall be proportioned so that the factored resistance is not less than the effects of the factored loads specified in Section 3.

Foundations shall be designed to support the appropriate load combinations without structural failure of the foundation or a bearing resistance failure of the supporting soil and/or rock (strength limit state), or unacceptable settlements and associated structural deformations (service limit state). The foundation load combinations and tolerable structural deformations are controlled by the superstructure and substructure design. The resistances and movements of the selected foundation type shall be estimated using the procedures prescribed in D10.6 (Spread Footings), D10.7 (Driven Piles) and D10.8 (Drilled Shafts).

10.5.2 Service Limit States

10.5.2.1 GENERAL

Foundation design at the service limit state shall include:

- Settlements
- Horizontal movements
- Overall stability, and
- Scour at the design flood

Consideration of foundation movements shall be based upon structure tolerance to total and differential movements, rideability and economy. Foundation movements shall include all movement from settlement, horizontal movement, and rotation.

C10.5.2.1

In bridges where the superstructure and substructure are not integrated, settlement corrections can be made by jacking and shimming bearings. A2.5.2.3 requires jacking provisions for these bridges.

The cost of limiting foundation movements should be compared with the cost of designing the superstructure so that it can tolerate larger movements or of correcting the consequences of movements through maintenance to determine minimum lifetime cost. The Owner may establish more stringent criteria.

The design flood for scour is defined in A2.6.4.4.2 and PP7.2.2, and is specified in A3.7.5 as applicable at the service limit state.

Estimated allowable bearing pressures were developed for use with working stress design for shallow foundations on rock. These values may be used for preliminary sizing of foundations. See Appendix C.

10.5.2.2 TOLERABLE MOVEMENTS AND MOVEMENT CRITERIA

Foundation movement criteria shall be consistent with the function and type of structure, anticipated service life, and consequences of unacceptable movements on structure performance. Foundation movement shall include vertical, horizontal, and rotational movements. The tolerable movement criteria shall be established by either empirical procedures or structural analyses, or by consideration of

C10.5.2.2

SPECIFICATIONS

both.

Allowable settlement criteria for footings on soil shall be developed by the structural designer consistent with the function and type of structure, the anticipated service life and the consequences of unacceptable settlements on the performance of the structure. Foundation settlement analyses shall be based on the results of in situ and/or laboratory testing to characterize the load deformation behavior of the foundation soils. Settlement analyses shall be performed to determine the relationship between estimated settlement and footing bearing pressure to optimize the footing size with respect to the loads to be supported.

The allowable settlement for shallow footings supporting bridge structures shall be based on the angular distortion (δ'/ℓ) between adjacent support units (i.e., between piers or piers and abutments) where δ' and ℓ are the differential settlement and span between adjacent units, respectively. In addition, the maximum net settlement of a footing shall not exceed 25 mm {1 in.}. The dimensionless ratio δ'/ℓ shall be limited to 0.0025 and 0.0015 for simple and continuous span bridges, respectively. Special treatment shall be given to differential settlement in a rigid frame type of structure (e.g., pier bent, rigid frame bridge, etc.). Rigid frames shall be designed for anticipated settlements. The above allowable differential settlement limits do not apply to rigid frame structures.

Foundation settlement shall be investigated using all applicable loads in the Service I Load Combination specified in Table 3.4.1-1. Transient loads may be omitted from settlement analyses for foundations bearing on or in cohesive soil deposits that are subject to time-dependant consolidation settlements.

10.5.2.3 OVERALL STABILITY

The evaluation of overall stability of earth slopes with or without a foundation unit shall be investigated at the service limit state as specified in D10.6.2.5.

10.5.2.4 ABUTMENT TRANSITIONS

Vertical and horizontal movements caused by embankment loads behind bridge abutments shall be investigated.

COMMENTARY

Differential settlement between substructure units results in stress redistribution in continuous beams. Large total settlements reduce vertical clearance and result in misalignment of approach roadway.

The net settlement of a footing is the settlement that occurs after the supported columns or beams are set and framed.

C10.5.2.4

Settlement of foundation soils induced by embankment loads can result in excessive movements of substructure elements. Both short and long term settlement potential should be considered.

Settlement of improperly placed or compacted backfill behind abutments can cause poor rideability and a possibly dangerous bump at the end of the bridge.

Lateral earth pressure behind and/or lateral squeeze below abutments can also contribute to lateral movement of abutments and should be investigated, if applicable.

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10.5.3 Strength Limit States

10.5.3.1 GENERAL

Design of foundations at strength limit states shall include consideration of the nominal geotechnical and structural resistances of the foundation elements. Design at strength limit states shall not consider the deformations required to mobilize the nominal resistance, unless a definition of failure based on deformation is specified.

The design of all foundations at the strength limit state shall consider:

- structural resistance and
- loss of lateral and vertical support due to scour at the design flood event.

COMMENTARY

C10.5.3.1

For the purpose of design at strength limit states, the nominal resistance is considered synonymous with the ultimate capacity of an element as previously defined under allowable stress design, i.e., AASHTO (*2002*).

For design of foundations such as piles or drilled shafts that may be based directly on static load tests, or correlation to static load tests, the definition of failure may include a deflection-limited criteria.

Structural resistance includes checks for axial, lateral and flexural resistance.

The design event for scour is defined in Section 2 and is specified in A3.7.5 as applicable at the strength limit state. Scour investigations shall be made in accordance with PP7.2.

10.5.3.2 SPREAD FOOTINGS

The design of spread footings at the strength limit state shall also consider:

- nominal bearing resistance,
- overturning or excessive loss of contact,
- sliding at the base of footing, and
- constructability.

C10.5.3.2

The designer should consider whether special construction methods are required to bear a spread footing at the design depth. Consideration should be given to the potential need for shoring, cofferdams, seals, and/or dewatering. Basal stability of excavations should be evaluated, particularly if dewatering or cofferdams are required.

Effort should be made to identify the presence of expansive/collapsible soils in the vicinity of the footing. If present, the structural design of the footing should be modified to accommodate the potential impact to the performance of the structure, or the expansive/collapsible soils should be removed or otherwise remediated. Special conditions such as the presence of karstic formations or mines should also be evaluated, if present.

10.5.3.3 DRIVEN PILES

The design of pile foundations at the strength limit state shall also consider:

- axial compression resistance for single piles,
- pile group compression resistance,
- uplift resistance for single piles,
- uplift resistance for pile groups,
- pile punching failure into a weaker stratum below the bearing stratum,
- single pile and pile group lateral resistance, and

C10.5.3.3

The commentary in D10.5.3.2 is applicable if a pile cap is needed.

For pile foundations, as part of the evaluation for the strength limit states identified herein, the effects of downdrag, soil setup or relaxation, and buoyancy due to groundwater should be evaluated.

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- constructability, including pile drivability.

10.5.3.4 DRILLED SHAFTS

The design of drilled shaft foundations at the strength limit state shall also consider:

- axial compression resistance for single drilled shafts,
- shaft group compression resistance,
- uplift resistance for single shafts,
- uplift resistance for shaft groups,
- single shaft and shaft group lateral resistance,
- shaft punching failure into a weaker stratum below the bearing stratum, and
- constructability, including method(s) of shaft construction.

10.5.4 Extreme Events Limit States

Foundations shall be designed for extreme events as applicable.

10.5.5 Resistance Factors

10.5.5.1 SERVICE LIMIT STATES

Resistance factors for the service limit states shall be taken as 1.0, except as provided for overall stability in D10.6.2.5.

A resistance factor of 1.0 shall be used to assess the ability of the foundation to meet the specified deflection criteria after scour due to the design flood.

10.5.5.2 STRENGTH LIMIT STATES

10.5.5.2.1 General

Resistance factors for different types of foundation systems at the strength limit state shall be taken as specified in D10.5.5.2.2, D10.5.5.2.3, and D10.5.5.2.4.

COMMENTARY

C10.5.3.4

See DC10.5.3.2 and DC10.5.3.3.

The design of drilled shafts for each of these limit states should include the effects of the method of construction, including construction sequencing, whether the shaft will be excavated in the dry or if wet methods must be used, as well as the need for temporary or permanent casing to control caving ground conditions. The design assumptions regarding construction methods must carry through to the contract documents to provide assurance that the geotechnical and structural resistance used for design will be provided by the constructed product.

C10.5.4

Refer to Section 10, Appendix A for guidance regarding seismic analysis and design.

Extreme events include the check flood for scour, vessel and vehicle collision, seismic loading, and other site-specific situations that the Engineer determines should be included.

C10.5.5.2.1

Smaller resistance factors should be used if site or material variability is anticipated to be unusually high or if design assumptions are required that increase design uncertainty that have not been mitigated through conservative selection of design parameters.

Certain resistance factors in D10.5.5.2.2, D10.5.5.2.3 and D10.5.5.2.4 are presented as a function of soil type, e.g., sand or clay. Naturally occurring soils do not fall neatly into

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these two classifications. In general, the terms "sand" and "cohesionless soil" may be connoted to mean drained conditions during loading, while "clay" or "cohesive soil" implies undrained conditions. For other or intermediate soil classifications, such as silts or gravels, the designer should choose, depending on the load case under consideration, whether the resistance provided by the soil will be a drained or undrained strength, and select the method of computing resistance and associated resistance factor accordingly.

In general, resistance factors for bridge and other structure design have been derived to achieve a reliability index, β, of 3.5, an approximate probability of failure, P_f, of 1 in 5,000. However, past geotechnical design practice has resulted in an effective reliability index, β, of 3.0, or an approximate probability of a failure of 1 in 1,000, for foundations in general , and for highly redundant systems, such as pile groups, an approximate reliability index, β, of 2.3, an approximate probability of failure of 1 in 100 (*Zhang et al., 2001*; *Paikowsky et al., 2004*; *Allen, 2005*). If the resistance factors provided in this article are adjusted to account for regional practices using statistical data and calibration, they should be developed using the β values provided above, with consideration given to the redundancy in the foundation system.

For bearing resistance, lateral resistance, and uplift calculations, the focus of the calculation is on the individual foundation element, e.g., a single pile or drilled shaft. Since these foundation elements are usually part of a foundation unit that contains multiple elements, failure of one of these foundation elements usually does not cause the entire foundation unit to reach failure, i.e., due to load sharing and overall redundancy. Therefore, the reliability of the foundation unit is usually more, and in many cases considerably more, than the reliability of the individual foundation element. Hence, a lower reliability can be successfully used for redundant foundations than is typically the case for the superstructure.

Note that not all of the resistance factors provided in this article have been derived using statistical data from which a specific β value can be estimated, since such data were not always available. In those cases, where data were not available, resistance factors were estimated through calibration by fitting to past allowable stress design safety factors, e.g., the AASHTO *Standard Specifications for Highway Bridges* (*2002*).

Additional discussion regarding the basis for the resistance factors for each foundation type and limit state is provided in D10.5.5.2.2, D10.5.5.2.3, and D10.5.5.2.4. Additional, more detailed information on the development of the resistance factors for foundations provided in this article, and a comparison of those resistance factors to previous Allowable Stress Design practice, e.g., AASHTO (*2002*), is provided in Allen (*2005*).

The foundation resistance after scour due to the design flood shall provide adequate foundation resistance using the

Scour design for the design flood must satisfy the requirement that the factored foundation resistance after

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resistance factors given in this article.

10.5.5.2.2 Spread Footings

The resistance factors provided in Table 1 shall be used for strength limit state design of spread footings, with the exception of site specific considerations in DC10.5.5.2.

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scour is greater than the factored load determined with the scoured soil removed. The resistance factors will be those used in the Strength Limit State, without scour.

C10.5.5.2.2

Table 10.5.5.2.2-1 Resistance Factors for Geotechnical Resistance of Shallow Foundations at the Strength Limit State.

METHOD/SOIL/CONDITION			RESISTANCE FACTOR
Bearing Resistance	φ_b	Sand - Semi-empirical procedure using SPT data	0.45
		- Semi-empirical procedure using CPT data	0.55
		- Theoretical Estimation using ϕ_f estimated from SPT data using ϕ_f estimated from CPT data using ϕ_f measured directly in lab or field tests	 0.35 0.45 0.55
		Clay - Semi-empirical procedure using CPT data	0.50
		- Theoretical Estimation using shear resistance measured in lab tests	 0.55
		using shear resistance measured in field vane tests	0.55
		using shear resistance estimated from CPT data	0.50
		Rock - Semi-empirical procedure, Carter and Kulhawy (1988)	0.55
		Plate Load Test	0.55
Sliding	φ_τ	Precast concrete placed on sand using ϕ_f estimated from SPT data using ϕ_f estimated from CPT data using ϕ_f measured directly in lab or field tests	 0.90 0.90 0.90
		Concrete cast-in-place on sand using ϕ_f estimated from SPT data using ϕ_f estimated from CPT data using ϕ_f measured directly in lab or field tests	 0.80 0.80 0.80
		Precast concrete placed on rock using δ from Table A3.11.5.3-1 using δ measured directly in lab or field tests	 1.00 0.90
		Concrete cast-in-place on rock using δ from Table A3.11.5.3-1 using δ measured directly in lab or field tests	 1.00 0.80

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Table 10.5.5.2.2-1 - Resistance Factors for Geotechnical Resistance of Shallow Foundations at the Strength Limit State (Continued)

METHOD/SOIL/CONDITION			RESISTANCE FACTOR
Sliding	φ_τ	Precast or cast-in-place concrete on clay	0.85
		Soil on soil	1.0
	φ_p	Passive earth pressure component of sliding resistance	0.50

The resistance factors in Table 1 were developed using both reliability theory and calibration by fitting to Allowable Stress Design (ASD). In general, ASD safety factors for footing bearing capacity range from 2.5 to 3.0, corresponding to a resistance factor of approximately 0.55 to 0.45, respectively, and for sliding, an ASD safety factor of 1.5, corresponding to a resistance factor of approximately 0.9. Calibration by fitting to ASD controlled the selection of the resistance factor in cases where statistical data were limited in quality or quantity. Regionally specific values have also been incorporated in to bearing resistance factors for theoretical methods in sand.

The resistance factor for sliding of cast-in-place concrete on sand is slightly lower than the other sliding resistance factors based on reliability theory analysis (Barker et al., 1991). The higher interface friction coefficient used for sliding of cast-in-place concrete on sand relative to that used for precast concrete on sand causes the cast-in-place concrete sliding analysis to be less conservative, resulting in the need for the lower resistance factor. A more detailed explanation of the development of the resistance factors provided in Table 1 is provided in Allen (*2005*).

The resistance factors for sliding of shallow footings on sand are higher for precast footings than for cast-in-place footings, indicating that estimation of sliding resistance is more reliable for precast footings. However, as indicated by Equation D10.6.3.4-2, the nominal resistance of a precast footing is only 80% of the nominal resistance of a cast-in-place footing, primarily because the precast footing has a formed base.

The resistance factors for plate load tests and passive resistance were based on engineering judgment and past ASD practice.

10.5.5.2.3 Driven Piles

Resistance factors shall be selected from Table 1 based on the method used for determining the nominal axial pile resistance. If the resistance factors provided in Table 1 are to be applied to nonredundant pile groups, i.e., less than five

C10.5.5.2.3

Where nominal pile axial resistance is determined during pile driving by dynamic analysis, dynamic formulae, or static load test, the uncertainty in the pile axial resistance is strictly due to the reliability of the resistance

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piles in the group, the resistance factor values in the table should be reduced by 20 percent to reflect a higher target β value. Greater reductions than this should be considered when a single pile supports an entire bridge pier, i.e., an additional 20 percent reduction in the resistance factor to achieve a β value of approximately 3.5. If the resistance factor is decreased in this manner, the η_R factor provided in A1.3.4 and D1.3.4 should not be increased to address the lack of foundation redundancy. These reductions do not apply to integral abutments.

If pile resistance is verified in the field using a dynamic method such as dynamic measurements combined with signal matching, the resistance factor for the field verification method should be used to determine the number of piles of a given nominal resistance needed to resist the factored loads in the strength limit state.

Regarding load tests, and dynamic tests with signal matching, the number of tests to be conducted to justify the resistance factors provided in Tables 1, 2, and 3 should be based on the variability in the properties and geologic stratification of the site to which the test results are to be applied. A site shall be defined as a project site, or a portion of it, where the subsurface conditions can be characterized as geologically similar in terms of subsurface stratification, i.e., sequence, thickness, and geologic history of strata, the engineering properties of the strata, and groundwater conditions. Note that a site as defined herein may be only a portion of the area in which the structure (or structures) is located. For sites where conditions are highly variable, a site could even be limited to a single pier.

To be consistent with the calibration conducted to determine the resistance factors in Tables 1, 2, and 3, the signal matching analysis (*Rausche et al., 1972*) of the dynamic test data should be conducted as described in Hannigan et al. (*2005*).

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determination method used in the field during pile installation.

In most cases, the nominal bearing resistance of each pile is field-verified using a dynamic method (see D10.7.3.8.2, D10.7.3.8.3, D10.7.3.8.4, or D10.7.3.8.5). The actual penetration depth where the pile is stopped using the results of the dynamic analysis will likely not be the same as the estimated depth from the static analysis. Hence, the reliability of the pile bearing resistance is dependent on the reliability of the method used to verify the bearing resistance during pile installation (see *Allen, 2005*, for additional discussion on this issue). Once the number of piles with a given nominal resistance needed to resist the factored loads is determined, the estimated depth of pile penetration to obtain the desired resistance is determined using the resistance factor for the static analysis method, equating the factored static analysis resistance to the factored dynamic analysis resistance (see DC10.7.3.3).

Dynamic methods may be unsuitable for field verification of nominal axial resistance of soft silts or clays where a large amount of setup is anticipated and it is not feasible to obtain dynamic measurement of pile restrikes over a sufficient length of time to assess soil setup. Dynamic methods may not be applicable for determination of axial resistance when driving piles to rock (see D10.7.3.2).

The resistance factors in Table 1 were developed using either statistical analysis of pile load tests combined with reliability theory (*Paikowsky, et al., 2004*), fitting to allowable stress design (ASD), or both. Where the two approaches resulted in a significantly different resistance factor, engineering judgment was used to establish the final resistance factor, considering the quality and quantity of the available data used in the calibration. See Allen (*2005*) for a more detailed explanation on the development of the resistance factors for pile foundation design.

For all axial resistance calculation methods, the resistance factors were, in general, developed from load test results obtained on piles with diameters of 600 mm {24 in.} or less. Very little data were available for larger diameter piles. Therefore, these resistance factors should be used with caution for design of significantly larger diameter piles.

Where driving criteria are established based on a static load test, the potential for site variability should be considered. The number of load tests required should be established based on the characterization of site subsurface conditions by the field and laboratory exploration and testing program. One or more static load tests should be performed per site to justify using the resistance factors in Table 2 for piles installed within the site.

Tables 2 and 3 identify resistance factors to be used and numbers of tests needed depending on whether the site variability is classified as low, medium, or high. Site variability may be determined based on judgment, or based on the following suggested criteria (*Paikowsky et al., 2004*):

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Step 1: For each identified significant stratum at each boring location, determine the average property value, e.g., *SPT* value, q_c value, etc., within the stratum for each boring.

Step 2: Determine the mean and coefficient of variation of the average values for each stratum determined in Step 1.

Step 3: Categorize the site variability as low if the COV is less than 25 percent, medium of the COV is 25 percent or more, but less than 40 percent, and high if the COV is 40 percent or more.

See Paikowsky et al. (*2004*) for additional discussion regarding these site variability criteria.

The dynamic testing with signal matching should be evenly distributed within a pier and across the entire structure in order to justify the use of the specified resistance factors. However, within a particular footing an increase in safety is realized where the most heavily loaded piles are tested. The number of production piles tested using dynamic measurements with signal matching should be determined in consideration of the site variability to justify the use of the specified resistance factors.

See D10.7.3.8.2, D10.7.3.8.3, and D10.7.3.8.4 for additional guidance regarding pile load testing, dynamic testing and signal matching, and wave equation analysis, respectively, as they apply to the resistance factors provided in Table 1.

Paikowsky et al. (*2004*) indicate that the resistance factors for static pile resistance analysis methods can vary significantly for different pile types. The resistance factors presented are average values for the method. See Paikowsky et al. (*2004*) and Allen (*2005*) for additional information regarding this issue.

The resistance factor for the Nordlund/Thurman method was derived primarily using the Peck et al. (*1974*) correlation between *SPT* $N1_{60}$ and the soil friction angle, using a maximum design soil friction angle of 36^o, assuming the contributing zone for the end bearing resistance is from the tip to two pile diameters below the tip.

For the clay static pile analysis methods, if the soil cohesion was not measured in the laboratory, the correlation between *SPT N* and S_u by Hara et al. (*1974*) was used for the calibration. Use of other methods to estimate S_u may require the development of resistance factors based on those methods.

For the statistical calibrations using reliability theory, a target reliability index, β, of 2.3 (an approximate probability of failure of 1 in 100) was used. The selection of this target reliability assumes a significant amount of redundancy in the foundation system is present, which is typical for pile groups containing at least five piles in the group (*Paikowsky et al., 2004*). For smaller groups and single piles, less

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redundancy will be present. The issue of redundancy, or the lack of it, is addressed in A1.3.4 and D1.3.4 through the use of η_R. The values for η_R provided in that article have been developed in general for the superstructure, and no specific guidance on the application of η_R to foundations is provided. Paikowsky et al. (*2004*) indicate that a target reliability, β, of 3.0 or more, i.e., an approximate probability of failure of 1 in 1000 or less, is more appropriate for these smaller pile groups that lack redundancy. The η_R factor values recommended in A1.3.4 and D1.3.4 are not adequate to address the difference in redundancy, based on the results provided by Paikowsky et al. (*2004*). Therefore, the resistance factors specified in Table 1 should be reduced to account for reduced redundancy.

The resistance factors provided for uplift of single piles are generally less than the resistance factors for axial skin friction under compressive loading. This is consistent with past practice that recognizes the skin friction in uplift is generally less than the skin friction under compressive loading, and is also consistent with the statistical calibrations performed in Paikowsky et al. (*2004*). Since the reduction in uplift resistance that occurs in tension relative to the skin friction in compression is taken into account through the resistance factor, the calculation of skin friction resistance using a static pile resistance analysis method should not be reduced from what is calculated from the methods provided in D10.7.3.8.6.

If a pile load test(s) is used to determine the uplift resistance of single piles, consideration should be given to how the pile load test results will be applied to all of the production piles. For uplift, the number of pile load tests required to justify a specific resistance factor are the same as that required for determining compression resistance. Therefore, Table 2 should be used to determine the resistance factor that is applicable. Extrapolating the pile load test results to other untested piles as specified in D10.7.3.10 does create some uncertainty, since there is not a way to directly verify that the desired uplift resistance has been obtained for each production pile. This uncertainty has not been quantified. Therefore, it is recommended that a resistance factor of not greater than 0.60 be used if an uplift load test is conducted.

Regarding pile drivability analysis, the only source of load is from the pile driving hammer. Therefore, the load factors provided in Section 3 do not apply. In past practice, e.g., AASHTO (*2002*), no load factors were applied to the stresses imparted to the pile top by the pile hammer. Therefore, a load factor of 1.0 should be used for this type of analysis. Generally, either a wave equation analysis or dynamic testing, or both, are used to determine the stresses in the pile resulting from hammer impact forces. Intuitively, the stresses measured during driving using dynamic testing should be more accurate than the stresses estimated using the wave equation analysis without dynamic testing. However, a statistical analysis and calibration using reliability theory has not been conducted as yet, and a

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recommendation cannot be provided to differentiate between these two methods regarding the load factor to be applied. See D10.7.8 for the specific calculation of the pile structural resistance available for analysis of pile drivability. The structural resistance available during driving determined as specified in D10.7.8 considers the ability of the pile to handle the transient stresses resulting from hammer impact, considering variations in the materials, pile/hammer misalignment, and variations in the pile straightness and uniformity of the pile head impact surface.

Table 10.5.5.2.3-1 Resistance Factors for Driven Piles.

Condition/Resistance Determination Method		Resistance Factor
Nominal Resistance of Single Pile in Axial Compression—Dynamic Analysis and Static Load Test Methods, φ_{dyn}	Driving criteria established by static load test(s); quality control by dynamic testing and/or calibrated wave equation, or minimum driving resistance combined with minimum delivered hammer energy from the load test(s). For the last case, the hammer used for the test pile(s) shall be used for the production piles.	Values in Table 2
	Driving criteria established by dynamic test with signal matching at beginning of redrive conditions only of at least one production pile per pier, but no less than the number of tests per site provided in Table 3. Quality control of remaining piles by calibrated wave equation and/or dynamic testing.	0.65
	Wave equation analysis, without pile dynamic measurements or load test, at end of drive conditions only	0.50
Nominal Resistance of Single Pile in Axial Compression—Static Analysis Methods, φ_{stat}	Skin Friction and End Bearing: Clay and Mixed Soils	
	α-method (*Tomlinson, 1987*; *Skempton, 1951*)	0.70
	β-method (*Esrig & Kirby, 1979*; *Skempton, 1951*)	0.50
	λ-method (*Vijayvergiya & Focht, 1972*; *Skempton, 1951*)	0.55
	Skin Friction and End Bearing: Sand	
	Nordlund/Thurman Method (*Hannigan et al., 2005*)	0.50
	SPT-method (Meyerhof)	0.45
	CPT-method (Schmertmann)	0.55
Block Failure, φ_{bl}	Clay	0.65
Uplift Resistance of Single Piles, φ_{up}	Nordlund Method	0.40
	α-method	0.60
	β-method	0.40
	λ-method	0.45
	SPT-method	0.35
	CPT-method	0.45
	Load test	0.80
Group Uplift Resistance, φ_{ug}	Sand and clay	0.55

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Table 10.5.5.2.3-1 Resistance Factors for Driven Piles. (Continued)

Horizontal Geotechnical Resistance of Single Pile or Pile Group	All soils and rock		1.0
Structural Limit State	Steel piles Concrete piles Timber piles	See the provisions of D6.5.4.2 See the provisions of D5.5.4.2.1 See the provisions of A8.5.2.2 and A8.5.2.3	
Pile Drivability Analysis, φ_{da}	Steel piles Concrete piles Timber piles In all three Articles identified above, use φ identified as "resistance during pile driving"	See the provisions of D6.5.4.2 See the provisions of D5.5.4.2.1 See the provisions of A8.5.2.2	

Table 10.5.5.2.3-2 Relationship between Number of Static Load Tests Conducted per Site and φ (after Paikowsky et al., 2004).

Number of Static Load Tests per Site	Resistance Factor, φ		
	Site Variability[a]		
	Low[a]	Medium[a]	High[a]
1	0.80	0.70	0.55
2	0.90	0.75	0.65
3	0.90	0.85	0.75
≥4	0.90	0.90	0.80

[a] See commentary.

Table 10.5.5.2.3-3 Number of Dynamic Tests with Signal Matching Analysis per Site to Be Conducted During Production Pile Driving (after *Paikowsky et al., 2004*).

Site Variability[a]	Low[a]	Medium[a]	High[a]
Number of Piles Located Within Site	Number of Piles with Dynamic Tests and Signal Matching Analysis Required (*BOR*)		
≤15	3	4	6
16–25	3	5	8
26–50	4	6	9
51–100	4	7	10
101–500	4	7	12
>500	4	7	12

[a] See commentary.

10.5.5.2.4 Drilled Shafts

Resistance factors shall be selected based on the method used for determining the nominal shaft resistance.

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The resistance factors in Table 1 were developed using either statistical analysis of shaft load tests combined with

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When selecting a resistance factor for shafts in clays or other easily disturbed formations, local experience with the geologic formations and with typical shaft construction practices shall be considered.

Where the resistance factors provided in Table 1 are to be applied to a nonredundant foundation such as a single shaft supporting a bridge pier, the resistance factor values in the table should be reduced by 20 percent to reflect a higher target β value of 3.5, an approximate probability of failure of 1 in 5,000, to be consistent with what has been used generally for design of the superstructure. Where the resistance factor is decreased in this manner, the η_R factor provided in A1.3.4 and D1.3.4 shall not be increased to address the lack of foundation redundancy.

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reliability theory (*Paikowsky et al., 2004*), fitting to allowable stress design (ASD), or both. Where the two approaches resulted in a significantly different resistance factor, engineering judgment was used to establish the final resistance factor, considering the quality and quantity of the available data used in the calibration. The available reliability theory calibrations were conducted for the Reese and O'Neill (*1988*) method, with the exception of shafts in intermediate geo-materials (IGMs), in which case the O'Neill and Reese (*1999*) method was used. In D10.8, the O'Neill and Reese (*1999*) method is recommended. See Allen (*2005*) for a more detailed explanation on the development of the resistance factors for shaft foundation design, and the implications of the differences in these two shaft design methods on the selection of resistance factors.

For the statistical calibrations using reliability theory, a target reliability index, β, of 3.0, an approximate probability of failure of 1 in 1,000, was used. The selection of this target reliability assumes a small amount of redundancy in the foundation system is present, which is typical for shaft groups containing at least two to four shafts in the group (*Paikowsky et al., 2004*). For single shafts, less redundancy will be present. The issue of redundancy, or the lack of it, is addressed in A1.3.4 and D1.3.4 through the use of η_R. The values for η_R provided in that article have been developed in general for the superstructure, and no specific guidance on the application of η_R to foundations is provided. The η_R factor values recommended in A1.3.4 and D1.3.4 are not adequate to address the difference in foundation redundancy, based on the results provided by Paikowsky et al. (*2004*) and others (see also *Allen, 2005*). Therefore, the resistance factors specified in Table 1 should be reduced to account for the reduced redundancy.

For shaft groups of five or more, greater redundancy than what has been assumed for the development of the shaft resistance factors provided in Table 1 is present. For these larger shaft groups, the resistance factors provided for shafts in Table 1 may be increased by up to 20 percent to achieve a reliability index of 2.3.

Where installation criteria are established based on a static load test, the potential for site variability should be considered. The number of load tests required should be established based on the characterization of site subsurface conditions by the field and laboratory exploration and testing program. One or more static load tests should be performed per site to justify using the resistance factor in Table D10.5.5.2.3-2 for drilled shafts installed within the site.

Table D10.5.5.2.3-2 identifies resistance factors to be used and numbers of tests needed depending on whether the site variability is classified as low, medium, or high. Site variability may be determined based on judgment, or based on the following suggested criteria (*Paikowsky et al., 2004*):

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Step 1: For each identified significant stratum at each boring location, determine the average property value, e.g., SPT value, q_c value, etc., within the stratum for each boring.

Step 2: Determine the mean and coefficient of variation of the average values for each stratum determined in Step 1.

Step 3: Categorize the site variability as low if the COV is less than 25 percent, medium of the COV is 25 percent or more, but less than 40 percent, and high if the COV is 40 percent or more.

See Paikowsky et al. (*2004*) for additional discussion regarding these site variability criteria.

For the specific case of shafts in clay, the resistance factor recommended by Paikowsky et al. (*2004*) is much lower than the recommendation from Barker et al. (*1991*). Since the shaft design method for clay is nearly the same for both the 1988 and 1999 methods, a resistance factor that represents the average of the two resistance factor recommendations is provided in Table 1. This difference may point to the differences in local geologic formations and local construction practices, pointing to the importance of taking such issues into consideration when selecting resistance factors, especially for shafts in clay.

IGMs are materials that are transitional between soil and rock in terms of their strength and compressibility, such as residual soils, glacial tills, or very weak rock. See DC10.8.2.2.3 for a more detailed definition of an IGM.

Since the mobilization of shaft base resistance is less certain than side resistance due to the greater deformation required to mobilize the base resistance, a lower resistance factor relative to the side resistance is provided for the base resistance in Table 1. O'Neill and Reese (*1999*) make further comment that the recommended resistance factor for tip resistance in sand is applicable for conditions of high quality control on the properties of drilling slurries and base cleanout procedures. If high quality control procedures are not used, the resistance factor for the O'Neill and Reese (*1999*) method for tip resistance in sand should be also be reduced. The amount of reduction should be based on engineering judgment.

Shaft compression load test data should be extrapolated to production shafts that are not load tested as specified in D10.8.3.5.6. There is no way to verify shaft resistance for the untested production shafts, other than through good construction inspection and visual observation of the soil or rock encountered in each shaft. Because of this, extrapolation of the shaft load test results to the untested production shafts may introduce some uncertainty. Hence, a reduction of the resistance factor used for design relative to the values provided in Table D10.5.5.2.3-2 may be warranted. Statistical data are not available to quantify this at this time. A resistance factor somewhere between the

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resistance factors specified for the static analysis method in Table D10.5.5.2.3-1 and the load test resistance factors specified in Table D10.5.5.2.3-2 should be used. Historically, resistance factors higher than 0.70, or their equivalent safety factor in previous practice, have not been used. Therefore, it is recommended that Table D10.5.5.2.3-2 be used, but that the resistance factor not be greater than 0.70.

This issue of uncertainty in how the load tests are applied to shafts not load tested is even more acute for shafts subjected to uplift load tests, as failure in uplift can be more abrupt than failure in compression. Hence, a resistance factor of 0.60 for the use of uplift load test results is recommended.

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Table 10.5.5.2.4-1 Resistance Factors for Geotechnical Resistance of Drilled Shafts.

Method/Soil/Condition			Resistance Factor
Nominal Axial Compressive Resistance of Single-Drilled Shafts, φ_{stat}	Side resistance in clay	α-method (*O'Neill and Reese, 1999*)	0.65
	Tip resistance in clay	Total Stress (*O'Neill and Reese, 1999*)	0.55
	Side resistance in sand	β-method (*O'Neill and Reese, 1999*)	0.65
	Tip resistance in sand	O'Neill and Reese (*1999*)	0.55
	Side resistance in IGMs	O'Neill and Reese (*1999*)	0.65
	Tip resistance in IGMs	O'Neill and Reese (*1999*)	0.55
	Side resistance in rock	Horvath and Kenney (*1979*) O'Neill and Reese (*1999*) Carter and Kulhawy (*1988*)	0.65 0.55 0.55
	Tip resistance in rock	Canadian Geotechnical Society (*1985*) Pressuremeter Method (*Canadian Geotechnical Society, 1985*) O'Neill and Reese (*1999*)	0.50 0.50 0.50
Block Failure, φ_{bl}	Clay		0.65
Uplift Resistance of Single-Drilled Shafts, φ_{up}	Clay	α-method (*O'Neill and Reese, 1999*)	0.55
	Sand	β-method (*O'Neill and Reese, 1999*)	0.55
	Rock	Horvath and Kenney (*1979*) Carter and Kulhawy (*1988*)	0.55 0.45
Group Uplift Resistance, φ_{ug}	Sand and clay		0.55
Horizontal Geotechnical Resistance of Single Shaft or Shaft Group	All materials		1.0
Static Load Test (compression), φ_{load}	All Materials		Values in Table D10.5.5.2.3-2, but no greater than 0.70
Static Load Test (uplift), φ_{upload}	All Materials		0.60

10.5.5.3 EXTREME LIMIT STATES

10.5.5.3.1 General

Design of foundations at extreme limit states shall be consistent with the expectation that structure collapse is

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prevented and that life safety is protected.

10.5.5.3.2 Scour

The foundation shall be designed so that the nominal resistance remaining after the scour resulting from the check flood (see A2.6.4.4.2 and PP7.2.3, Superflood) provides adequate foundation resistance to support the unfactored Strength Limit States loads with a resistance factor of 1.0. For uplift resistance of piles and shafts, the resistance factor shall be taken as 0.80 or less.

The foundation shall resist not only the loads applied from the structure but also any debris loads occurring during the flood event.

10.5.5.3.3 Other Extreme Limit States

Resistance factors for extreme limit state, including the design of foundations to resist earthquake, ice, vehicle or vessel impact loads, shall be taken as 1.0. For uplift resistance of piles and shafts, the resistance factor shall be taken as 0.80 or less.

10.6 SPREAD FOOTINGS

10.6.1 General Considerations

10.6.1.1 GENERAL

Provisions of this article shall apply to design of isolated, continuous strip and combined footings for use in support of columns, walls and other substructure and superstructure elements. Special attention shall be given to footings on fill, to make sure that the quality of the fill placed below the footing is well controlled and of adequate quality in terms of shear strength and compressibility to support the footing loads.

Spread footings shall be proportioned and designed such that the supporting soil or rock provides adequate nominal resistance, considering both the potential for adequate bearing strength and the potential for settlement, under all applicable limit states in accordance with the provisions of this Section.

Spread footings shall be proportioned and located to maintain stability under all applicable limit states, considering the potential for, but not necessarily limited to, overturning (eccentricity), sliding, uplift, overall stability and loss of lateral support.

10.6.1.2 BEARING DEPTH

Where the potential for scour, erosion or undermining exists, spread footings shall be located to bear below the maximum anticipated depth of scour, erosion, or

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C10.5.5.3.2

The axial nominal strength after scour due to the check flood must be greater than the unfactored pile or shaft load for the Strength Limit State loads. The specified resistance factors should be used provided that the method used to compute the nominal resistance does not exhibit bias that is unconservative. See Paikowsky et al. (*2004*) regarding bias values for pile resistance prediction methods.

Design for scour is discussed in Hannigan et al. (*2005*).

C10.5.5.3.3

The difference between compression skin friction and tension skin friction should be taken into account through the resistance factor, to be consistent with how this is done for the strength limit state (see DC10.5.5.2.3).

C10.6.1.1

Problems with insufficient bearing and/or excessive settlements in fill can be significant, particularly if poor, e.g., soft, wet, frozen, or nondurable, material is used, or if the material is not properly compacted.

Spread footings should not be used on soil or rock conditions that are determined to be too soft or weak to support the design loads without excessive movement or loss of stability. Alternatively, the unsuitable material can be removed and replaced with suitable and properly compacted engineered fill material, or improved in place, at reasonable cost as compared to other foundation support alternatives.

Footings should be proportioned so that the stress under the footing is as nearly uniform as practicable at the service limit state. The distribution of soil stress should be consistent with properties of the soil or rock and the structure and with established principles of soil and rock mechanics.

C10.6.1.2

Consideration should be given to the use of either a geotextile or graded granular filter material to reduce the susceptibility of fine grained material piping into rip rap or

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undermining as specified in A2.6.4.4, and PP7.2.

Spread footings shall be located below the depth of frost potential. Depth of frost potential shall be determined on the basis of local or regional frost penetration data.

For footings constructed on slopes, a minimum horizontal bench 1200 mm {4 ft.} wide shall be provided as shown in Figure D11.1.1.1.1P-1. The actual bench width shall consider overall stability of the slope in accordance with D10.6.2.5. In no case shall the minimum embedment depth to the bottom of footings be less than 900 mm {3 ft.} or the expected depth of frost penetration, nor shall the soil cover over the footing be less than 300 mm {1 ft.}.

Footing depths and scour protection must follow the provisions of PP7.2 when scour is a possibility. Scour investigations shall be made in accordance with PP7.2.

10.6.1.3 EFFECTIVE FOOTING DIMENSIONS

For eccentrically loaded footings, a reduced effective area, $B' \times L'$, within the confines of the physical footing shall be used in geotechnical design for settlement or bearing resistance. The point of load application shall be at the centroid of the reduced effective area.

The reduced dimensions for an eccentrically loaded rectangular footing shall be taken as:

$$B' = B - 2e_B \qquad (10.6.1.3\text{-}1)$$

$$L' = L - 2e_L \qquad (10.6.1.3\text{-}2)$$

where:

e_B = eccentricity parallel to dimension B (mm){ft.}

e_L = eccentricity parallel to dimension L (mm){ft.}

The effective footing area shall be determined as follows:

$$A' = B'L' \qquad (10.6.1.3\text{-}3)$$

Refer to Figure C1 for eccentric loading and Figure C10.6.3.1.2a-3 for inclined loading definitions and footing

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open-graded granular foundation material.

Evaluation of seepage forces and hydraulic gradients should be performed as part of the design of foundations that will extend below the groundwater table. Upward seepage forces in the bottom of excavations can result in piping loss of soil and/or heaving and loss of stability in the base of foundation excavations. Dewatering with wells or wellpoints can control these problems. Dewatering can result in settlement of adjacent ground or structures. If adjacent structures may be damaged by settlement induced by dewatering, seepage cut-off methods such as sheet piling or slurry walls may be necessary.

C10.6.1.3

The reduced dimensions for a rectangular footing are shown in Figure C1.

For loads eccentric relative to the centroid of the footing, reduced footing dimensions (B' and L') shall be used to determine bearing capacity factors and modifiers (i.e., slope, footing shape and load inclination factors), and to calculate the nominal bearing resistance of the footing. The reduced footing dimensions shall be determined as described in D10.6.1.3.

Figure C1 is provided primarily for the evaluation of existing footings subjected to eccentric loading about both the longitudinal and transverse axes. Such conditions may result from the evaluation of new foundations for rehabilitated structures where the distribution and direction of load cannot be controlled and/or reconstruction of footings to dimensions to ensure that the resultant of the factored load is within 1/4 of the footing dimensions from the footing centroid is not possible or practical due to surface constraints.

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dimensions. Footings under eccentric loads shall be designed to ensure that the factored bearing resistance is not less than the effects of factored loads at all applicable limit states.

Eccentric load limitations at the Strength Limit State are given in D10.6.3.3 for footings bearing on soil or rock.

For footings that are not rectangular, similar procedures should be used based upon the principles specified above.

10.6.1.4 BEARING STRESS DISTRIBUTIONS

When proportioning footing dimensions to meet settlement and bearing resistance requirements at all applicable limit states, the distribution of bearing stress on the effective area shall be assumed to be:

- uniform for footings on soils, or
- linearly varying, i.e., triangular or trapezoidal as applicable, for footings on rock

The distribution of bearing stress shall be determined as specified in D11.6.3.2.

Bearing stress distributions for structural design of the footing shall be as specified in D10.6.5.

The value of q_r obtained using the reduced footing dimensions represents an equivalent uniform bearing pressure and not the actual contact pressure distribution beneath the footing. This equivalent uniform bearing resistance shall be compared to the factored bearing

COMMENTARY

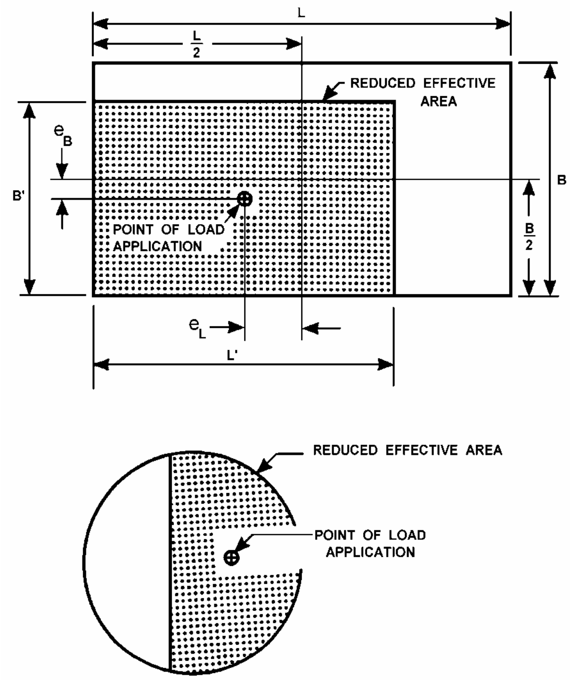

Figure C10.6.1.3-1 Reduced Footing Dimensions.

For footings that are not rectangular, such as the circular footing shown in Figure C1, the reduced effective area is always concentrically loaded and can be estimated by approximation and judgment. Such an approximation could be made, assuming a reduced rectangular footing size having the same area and centroid as the shaded area of the circular footing shown in Figure C1.

Eccentric loads have the effect of reducing the bearing resistance of a footing. "Actual" footing contact pressures for eccentrically loaded footings are typically calculated assuming a rigid footing.

For non-rectangular sections, L may be estimated as

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pressure in accordance with D11.6.3.2. The actual contact pressure distribution shall be used for structural design of the footing as described in D10.6.5.

Transient foundation uplift or rocking involving separation from the subsoil of up to one-half of an end bearing foundation pile group or up to one-half of the contact area of foundation footings is permitted under loading for Extreme Event I (seismic loading), provided that foundation soils are not susceptible to loss of strength under the imposed cyclic loading.

10.6.1.5 ANCHORAGE OF INCLINED FOOTINGS

Inclined footing bases shall not be used without the approval of the Chief Bridge Engineer.

Footings that are founded on inclined smooth solid rock surfaces and that are not restrained by an overburden of resistant material shall be effectively anchored by means of rock anchors, rock bolts, dowels, keys or other suitable means.

10.6.1.6 GROUNDWATER

Spread footings shall be designed in consideration of the highest anticipated groundwater table.

The influences of groundwater table on the bearing resistance of soils or rock and on the settlement of the structure shall be considered. In cases where seepage forces are present, they should also be included in the analyses.

10.6.1.7 UPLIFT

Where spread footings are subjected to uplift forces, they shall be investigated both for resistance to uplift and for structural strength.

10.6.1.8 NEARBY STRUCTURES

Where foundations are placed adjacent to existing structures, the influence of the existing structure on the behavior of the foundation and the effect of the foundation on the existing structures shall be investigated.

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follows:

Abutment L = Bearing-to-bearing + 1200 mm {4 ft.}

Wing L = Construction Joint to End of Wing

C10.6.1.5

Design of anchorages should include consideration of corrosion potential and protection.

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10.6.1.9 REPLACEMENT OF UNSUITABLE MATERIAL

Where unsuitable material exists to a depth of 3000 mm {10 ft.} or less below the footing bearing level, consideration shall be given to the use of spread footings on compacted structure backfill as an alternative to deep foundations. Where unsuitable material extends more than 3000 mm {10 ft.} below the footing bearing level, deep foundations shall be used. Unsuitable material is defined as material which will not provide adequate bearing capacity with acceptable settlements as determined in accordance with D10.6.3 and D10.6.2.

To permit the use of footings, unsuitable material shall be removed and replaced as shown in Figure 1. For footings on clay soil which provide adequate bearing resistance, a minimum overexcavation of 150 mm {6 in.} shall be required. All unsuitable material removed below the footing bearing level shall be replaced with compacted structure backfill, and all unsuitable material above the footing bearing level shall be replaced with compacted embankment material in accordance with the Standard Drawings (see Standard Drawing RC-12M).

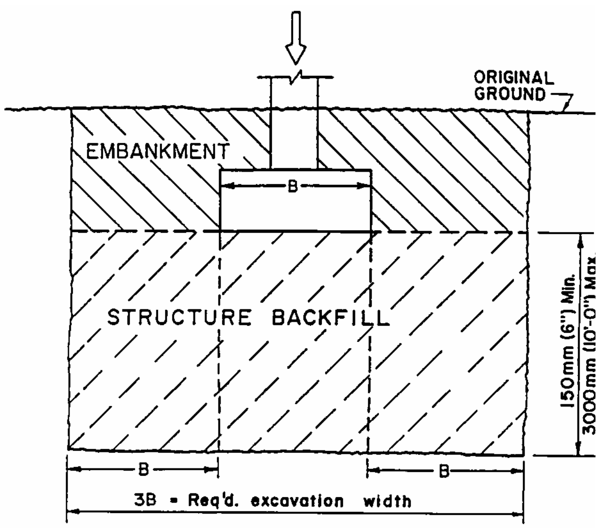

Figure 10.6.1.9-1 - Unsuitable Material Replacement

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OSHA regulations in regard to excavation must be followed in all cases of removal and replacement of unsuitable material.

Instances which require a relatively shallow depth of removal and replacement of unsuitable material may not require excavation width to be as wide as 3B, as shown in Figure D10.6.9-1. A narrower excavation width will be acceptable provided engineering analysis shows that the factored bearing resistance will be adequate and the foundation settlement will be acceptable.

10.6.2 Service Limit State Design

10.6.2.1 GENERAL

Service limit state design of spread footings shall include evaluation of total and differential settlement and overall stability. Overall stability of a footing shall be evaluated where one or more of the following conditions exist:

C10.6.2.1

The design of spread footings is frequently controlled by movement at the service limit state. It is therefore usually advantageous to proportion spread footings at the service limit state and check for adequate design at the strength and extreme limit states.

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- horizontal or inclined loads are present,
- the foundation is placed on embankment,
- the footing is located on, near or within a slope,
- the possibility of loss of foundation support through erosion or scour exists, or
- bearing strata are significantly inclined.

10.6.2.2 TOLERABLE MOVEMENTS

The requirements of D10.5.2.2 shall apply.

10.6.2.3 LOADS

Immediate settlement shall be determined using load combination Service-I, as specified in Table D3.4.1-1. Time-dependent settlements in cohesive soils should be determined using only the permanent loads, i.e., transient loads should not be considered.

10.6.2.4 SETTLEMENT ANALYSES

10.6.2.4.1 General

Foundation settlements should be estimated using computational methods based on the results of laboratory or insitu testing, or both. The soil parameters used in the computations should be chosen to reflect the loading history of the ground, the construction sequence, and the effects of

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C10.6.2.3

The type of load or the load characteristics may have a significant effect on spread footing deformation. The following factors should be considered in the estimation of footing deformation:

- the ratio of sustained load to total load,
- the duration of sustained loads, and
- the time interval over which settlement or lateral displacement occurs.

The consolidation settlements in cohesive soils are time-dependent; consequently, transient loads have negligible effect. However, in cohesionless soils where the permeability is sufficiently high, elastic deformation of the supporting soil due to transient load can take place. Because deformation in cohesionless soils often takes place during construction while the loads are being applied, it can be accommodated by the structure to an extent, depending on the type of structure and construction method.

Deformation in cohesionless, or granular, soils often occurs as soon as loads are applied. As a consequence, settlements due to transient loads may be significant in cohesionless soils, and they should be included in settlement analyses.

C10.6.2.4.1

Elastic, or immediate, settlement is the instantaneous deformation of the soil mass that occurs as the soil is loaded. The magnitude of elastic settlement is estimated as a function of the applied stress beneath a footing or embankment. Elastic settlement is usually small and

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soil layering.

Both total and differential settlements, including time dependant effects, shall be considered.

Total settlement, including elastic, consolidation, and secondary components may be taken as:

$$S_t = S_e + S_c + S_s \qquad (10.6.2.4.1\text{-}1)$$

where:

S_e = elastic settlement (mm){ft.}

S_c = primary consolidation settlement (mm){ft.}

S_s = secondary settlement (mm){ft.}

The effects of the zone of stress influence, or vertical stress distribution, beneath a footing shall be considered in estimating the settlement of the footing.

Consideration shall be given to the settlement of underlying natural soils due to the weight of soil fill.

Spread footings bearing on a layered profile consisting of a combination of cohesive soil, cohesionless soil and/or rock shall be evaluated using an appropriate settlement estimation procedure for each layer within the zone of influence of induced stress beneath the footing.

The distribution of vertical stress increase below circular or square and long rectangular footings, i.e., where $L > 5B$, may be estimated using Figure 1.

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neglected in design, but where settlement is critical, it is the most important deformation consideration in cohesionless soil deposits and for footings bearing on rock. For footings located on over-consolidated clays, the magnitude of elastic settlement is not necessarily small and should be checked.

In a nearly saturated or saturated cohesive soil, the pore water pressure initially carries the applied stress. As pore water is forced from the voids in the soil by the applied load, the load is transferred to the soil skeleton. Consolidation settlement is the gradual compression of the soil skeleton as the pore water is forced from the voids in the soil. Consolidation settlement is the most important deformation consideration in cohesive soil deposits that possess sufficient strength to safely support a spread footing. While consolidation settlement can occur in saturated cohesionless soils, the consolidation occurs quickly and is normally not distinguishable from the elastic settlement.

Secondary settlement, or creep, occurs as a result of the plastic deformation of the soil skeleton under a constant effective stress. Secondary settlement is of principal concern in highly plastic or organic soil deposits. Such deposits are normally so obviously weak and soft as to preclude consideration of bearing a spread footing on such materials.

The principal deformation component for footings on rock is elastic settlement, unless the rock or included discontinuities exhibit noticeable time-dependent behavior.

Methods for determining the distribution of vertical stress below a loaded area are usually based on elastic theory. While most soils are not elastic, the approach is valid, provided the additional stress imposed by foundation loading is below about 75% of the failure stress.

For guidance on vertical stress distribution for complex footing geometries, see Poulos and Davis (*1974*) or Lambe and Whitman (*1969*).

For highway applications, the stress distributions of most value include those presented in Figures C1 and C2. The charts presented in Figure C1 present solutions for the cases of (a) the vertical stress increase below the corner of a uniformly loaded rectangular area and (b) the vertical stress increase at various locations below a uniformly loaded circular area. By applying superposition principles, the solution for the vertical stress below the corner of a uniformly loaded rectangular area may be used to determine the vertical stress increase below the center or point along the edge of a rectangular or square area. The charts presented in Figure C2 provide solutions for the distribution of vertical stress increase below areas having a variable intensity of surface loading typical of an embankment.

Some methods used for estimating settlement of footings on sand include an integral method to account for the effects of vertical stress increase variations. For guidance regarding application of these procedures, see Gifford et al. (*1987*).

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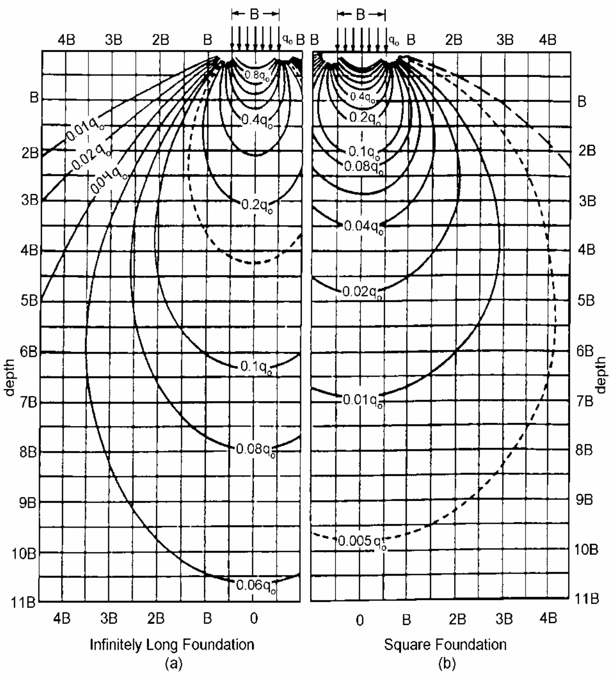
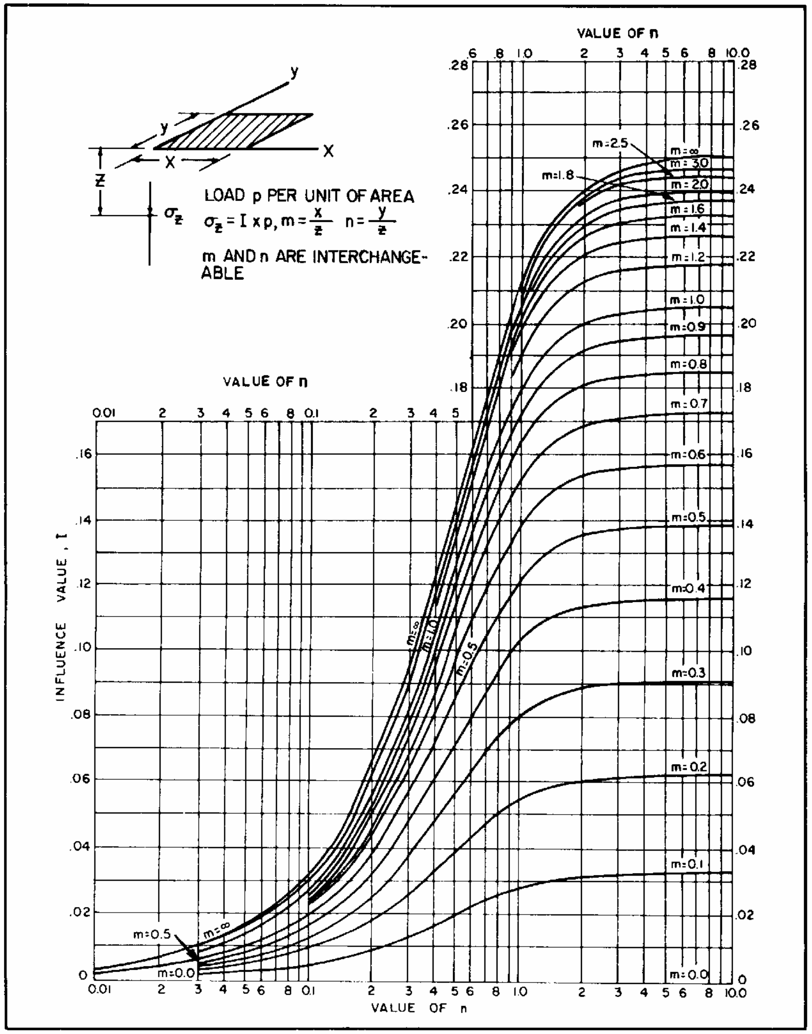


Figure 10.6.2.4.1-1 Boussinesq Vertical Stress Contours for Continuous and Square Footings Modified after Sowers (1979).



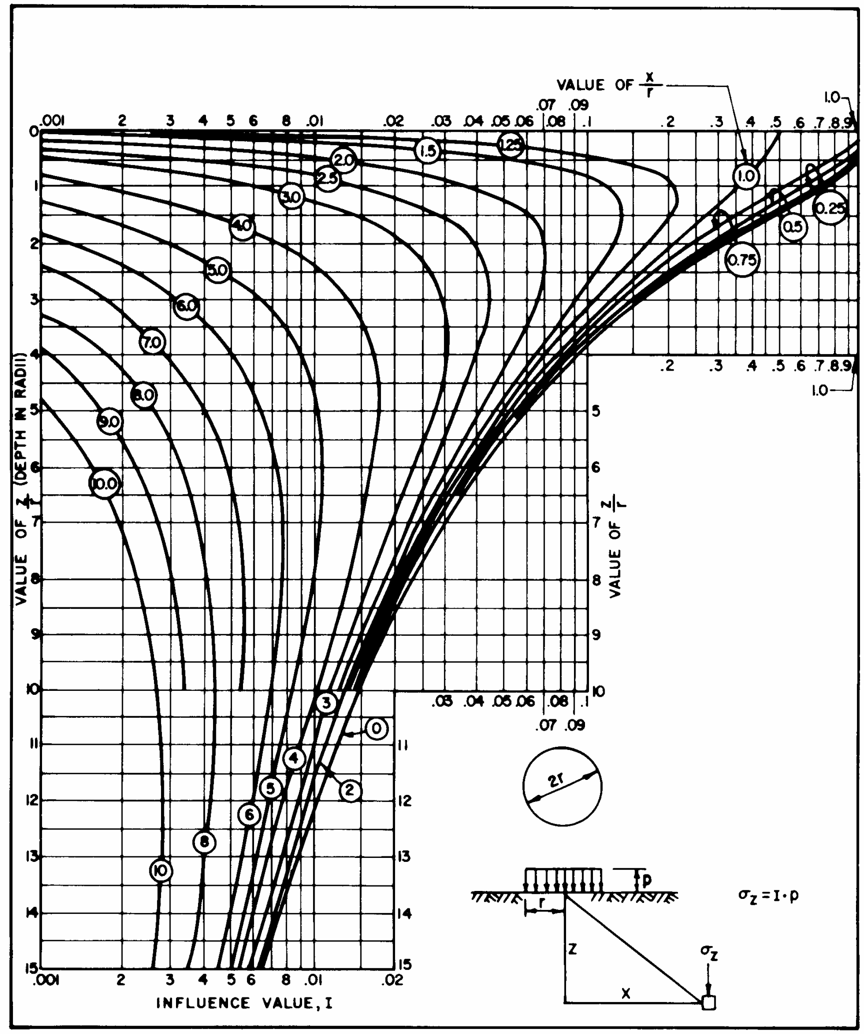
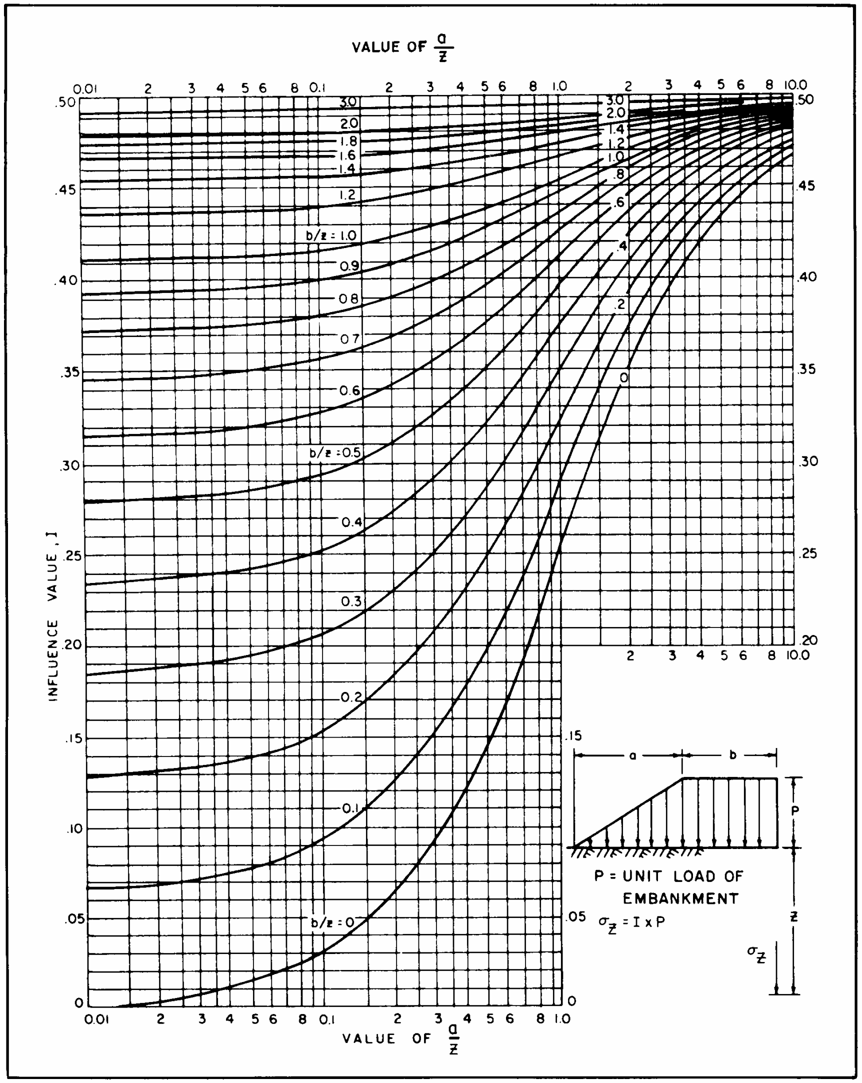


Figure C10.6.2.4.1-1 - Influence Coefficients for Vertical Stress Below Uniformly Loaded Rectangular and Circular Areas (U. S. Department of the Navy, 1986)



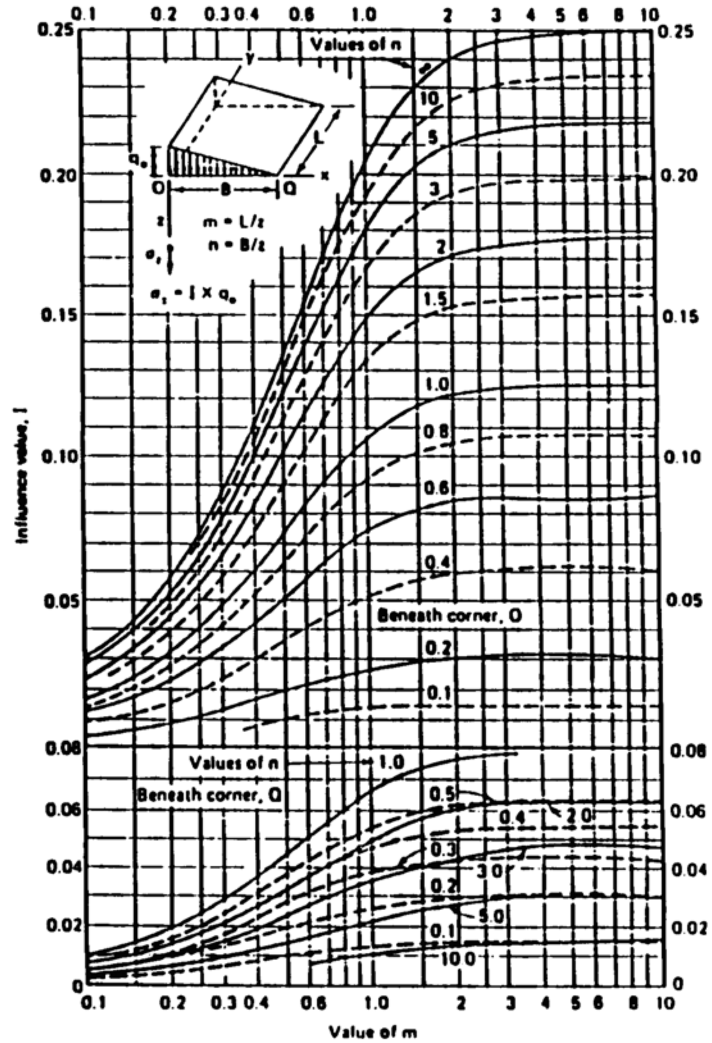


Figure C10.6.2.4.1-2 - Influence Coefficients for Vertical Stress Below Embankment and Triangular Areas (U. S. Department of the Navy, 1986)

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10.6.2.4.2 Settlement of Footings on Cohesionless Soils

The settlement of spread footings bearing on cohesionless soil deposits shall be estimated as a function of effective footing width and shall consider the effects of footing geometry and soil and rock layering with depth.

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C10.6.2.4.2

Elastic settlement is the principal component of the total settlement experienced by footings on granular soils and stiff cohesive soils.

Although methods are recommended for the determination of settlement of cohesionless soils, experience has indicated that settlements can vary considerably in a construction site, and this variation may not be predicted by conventional calculations.

Settlements of cohesionless soils occur rapidly, essentially as soon as the foundation is loaded. Therefore, the total settlement under the service loads may not be as important as the incremental settlement between intermediate load stages. For example, the total and differential settlement due to loads applied by columns and cross beams is generally less important than the total and differential settlements due to girder placement and casting of continuous concrete decks.

Generally conservative settlement estimates may be obtained using the elastic half-space procedure or the empirical method by Hough. The Schmertmann method (Fang, 1991) may be used with approval from the Department. Additional information regarding the accuracy of the methods described herein is provided in Gifford et al. (*1987*) and Kimmerling (*2002*). This information, in combination with local experience and engineering judgment, should be used when determining the estimated settlement for a structure foundation, as there may be cases, such as attempting to build a structure grade high to account for the estimated settlement, when overestimating the settlement magnitude could be problematic.

Details of other procedures can be found in textbooks and engineering manuals, including:

- Terzaghi and Peck, 1967
- Sowers, 1979
- U.S. Department of the Navy, 1982
- D'Appolonia (Gifford *et al., 1987*)—This method includes consideration for over-consolidated sands.
- Tomlinson, 1986
- Gifford et al., 1987
- Barker, et al., 1991

The average elastic settlement of footings on cohesionless soils and stiff cohesive soils may be estimated using the following:

When more than one layer of soil is encountered beneath a footing, the elastic settlement of each layer shall be calculated separately using the layer thickness coefficient for that layer. To determine the effective width B' for use in Figure 1, the stress, q_n, at the top of a given layer is assumed

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Metric Units: $S_e = \mu_0 \, \mu_1 \dfrac{q_o B'}{E_s}$ (10.6.2.4.2-1)

U.S. Customary Units: $S_e = \mu_0 \, \mu_1 \dfrac{q_o B'}{144 E_s}$

where:

μ_o = depth factor taken as 1.0

μ_1 = layer thickness coefficient taken from Figure 1

q_o = vertical stress at base of loaded area (MPa) {ksf}

B' = effective footing width as specified in D10.6.1.3 (mm) {ft.}

E_s = Young's modulus of soil taken as specified in D10.4.6.3 if direct measurements of E_s are not available from the results of in situ or laboratory tests (MPa){ksi}

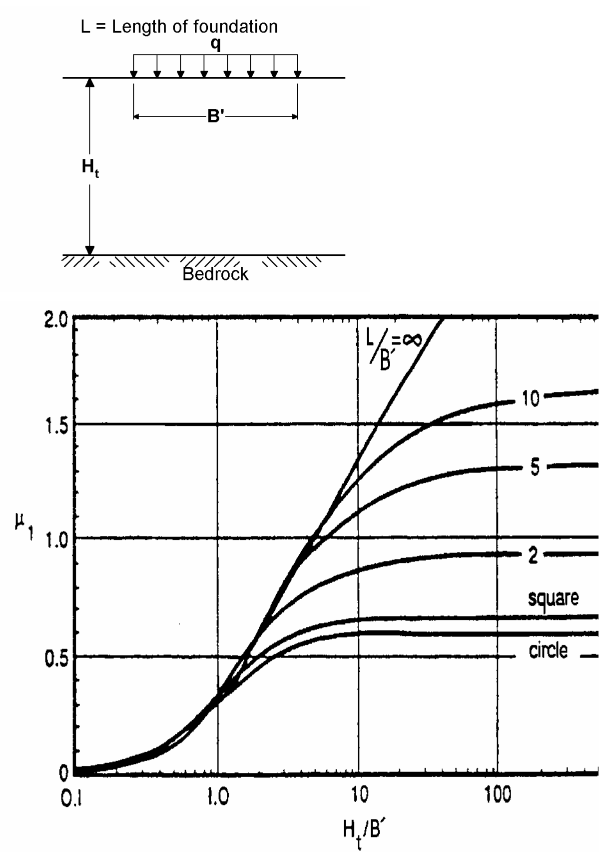

Figure 10.6.2.4.2-1 - Settlement Influence Factor μ_1 after Christian and Carrier (1978)

Unless E_s varies significantly with depth, E_s should be determined at a depth of about 1/2 to 2/3 of B below the footing, where B is the footing width. If the soil modulus

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to be distributed over an effective area which has been increased by the depth to the layer, H, resulting in an effective width of B' + H.

When calculating the settlement for piers on multi-layered soils, the length, L, should also be increased proportionately. The effective footing length L would be L + H and the effective footing width would be B' + H, when selecting the appropriate curve for the soil layer under consideration. The stress used in Equation 1 would be calculated as per Equation C1.

$q_n = q_o[B'/(B' + H)][L/(L + H)]$ (C10.6.2.4.2-1)

For general guidance regarding the estimation of elastic settlement of footings on sand, see Gifford et al. (*1987*) and Kimmerling (*2002*).

The stress distributions used to calculate elastic settlement assume the footing is flexible and supported on a homogeneous soil of infinite depth. The settlement below a flexible footing varies from a maximum near the center to a minimum at the edge equal to about 50 percent and 64 percent of the maximum for rectangular and circular footings, respectively. The settlement profile for rigid footings is assumed to be uniform across the width of the footing.

The accuracy of settlement estimates using elastic theory are strongly affected by the selection of soil modulus and the inherent assumptions of infinite elastic half space. Accurate estimates of soil moduli are difficult to obtain because the analyses are based on only a single value of soil modulus, and Young's modulus varies with depth as a function of overburden stress. Therefore, in selecting an appropriate value for soil modulus, consideration should be given to the influence of soil layering, bedrock at a shallow depth, and adjacent footings.

For footings with eccentric loads, the area, *A*′, should be computed based on reduced footing dimensions as specified in D10.6.1.3.

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varies significantly with depth, a weighted average value of E_s should be used.

Estimation of spread footing settlement on cohesionless soils by the empirical Hough method shall be determined using Equations 2 and 3. *SPT* blow counts shall be corrected as specified in D10.4.6.2.4 for depth, i.e. overburden stress, before correlating the *SPT* blow counts to the bearing capacity index, C'.

$$S_e = \sum_{i=1}^{n} \Delta H_i \qquad (10.6.2.4.2\text{-}2)$$

in which:

$$\Delta H_i = H_c \frac{1}{C'} \log\left(\frac{\sigma'_o + \Delta\sigma_v}{\sigma'_o}\right) \qquad (10.6.2.4.2\text{-}3)$$

where:

n = number of soil layers within zone of stress influence of the footing

ΔH_i = elastic settlement of layer i (mm){ft.}

H_C = initial height of layer i (mm){ft.}

C' = bearing capacity index from Figure 2 (dim.)

In Figure 2, N′ shall be taken as $N1_{60}$, Standard Penetration Resistance, N (blows/300 mm){blows/ft.}, corrected for overburden pressure as specified in D10.4.6.2.4.

σ'_O = initial vertical effective stress at the midpoint of layer i (MPa){ksf}

$\Delta\sigma_V$ = increase in vertical stress at the midpoint of layer i (MPa){ksf}

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The Hough method was developed for normally consolidated cohesionless soils.

The Hough method has several advantages over other methods used to estimate settlement in cohesionless soil deposits, including express consideration of soil layering and the zone of stress influence beneath a footing of finite size.

The subsurface soil profile should be subdivided into layers based on stratigraphy to a depth of about three times the footing width. The maximum layer thickness should be about 3000 mm {10 ft}.

While Cheney and Chassie (*2000*), and Hough (*1959*), did not specifically state that the SPT N values should be corrected for hammer energy in addition to overburden pressure, due to the vintage of the original work, hammers that typically have an efficiency of approximately 60 percent were in general used to develop the empirical correlations contained in the method. If using *SPT* hammers with efficiencies that differ significantly from this 60 percent value, the N values should also be corrected for hammer energy, in effect requiring that $N1_{60}$ be used.

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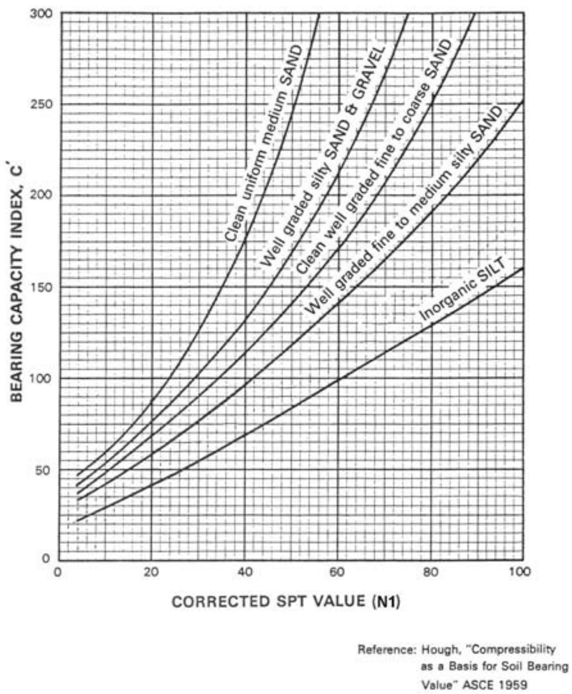

Figure 10.6.2.4.2-2 Bearing Capacity Index versus Corrected SPT (modified from *Cheney and Chassie, 2000*, after *Hough, 1959*).

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The Hough method is applicable to cohesionless soil deposits. The "Inorganic Silt" curve should generally not be applied to soils that exhibit plasticity. The settlement characteristics of cohesive soils that exhibit plasticity should be investigated using undisturbed samples and laboratory consolidation tests as prescribed in D10.6.2.4.3.

10.6.2.4.3 Settlement of Footings on Cohesive Soils

For foundations on stiff cohesive soils, the elastic settlement may be determined using Equation D10.6.2.4.2-1.

Spread footings in which cohesive soils are located within the zone of stress influence shall be investigated for consolidation settlement. Elastic and secondary settlements shall also be investigated in consideration of the timing and sequence of construction loading and the tolerance of the structure to total and differential movements.

Where laboratory test results are expressed in terms of void ratio, e, the consolidation settlement shall be taken as:

- For overconsolidated soils where $\sigma'_p > \sigma'_o$, see Figure 1:

$$S_c = \left[\frac{H_c}{1+e_o}\right]\left[C_r \log\left(\frac{\sigma'_p}{\sigma'_o}\right) + C_c \log\left(\frac{\sigma'_f}{\sigma'_p}\right)\right] \tag{10.6.2.4.3-1}$$

- For normally consolidated soils where $\sigma'_p = \sigma'_o$:

$$S_c = \left[\frac{H_c}{1+e_o}\right]\left[\left(C_c \log \frac{\sigma'_f}{\sigma'_p}\right)\right] \tag{10.6.2.4.3-2}$$

C10.6.2.4.3

In practice, footings on cohesive soils are most likely founded on overconsolidated clays, and settlements can be estimated using elastic theory (*Baguelin et al., 1978*), or the tangent modulus method (*Janbu, 1963, 1967*). Settlements of footings on overconsolidated clay usually occur at approximately one order of magnitude faster than soils without preconsolidation, and it is reasonable to assume that they take place as rapidly as the loads are applied. Infrequently, a layer of cohesive soil may exhibit a preconsolidation stress less than the calculated existing overburden stress. The soil is then said to be underconsolidated because a state of equilibrium has not yet been reached under the applied overburden stress. Such a condition may have been caused by a recent lowering of the groundwater table. In this case, consolidation settlement will occur due to the additional load of the structure and the settlement that is occurring to reach a state of equilibrium. The total consolidation settlement due to these two components can be estimated by Eq. 3 or Eq. 6.

Normally consolidated and underconsolidated soils should be considered unsuitable for direct support of spread footings due to the magnitude of potential settlement, the time required for settlement, for low shear strength concerns, or any combination of these design considerations. Preloading or vertical drains may be considered to mitigate these concerns.

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- For underconsolidated soils where $\sigma'_p < \sigma'_o$:

$$S_c = \left[\frac{H_c}{1+e_o}\right]\left[C_c \log\left(\frac{\sigma'_f}{\sigma'_{pc}}\right)\right] \qquad (10.6.2.4.3\text{-}3)$$

Where laboratory test results are expressed in terms of vertical strain, ε_v, the consolidation settlement shall be taken as:

- For overconsolidated soils where $\sigma'_p > \sigma'_o$, see Figure 2:

$$S_c = H_c\left[C_{r\varepsilon}\log\left(\frac{\sigma'_p}{\sigma'_o}\right) + C_{c\varepsilon}\log\left(\frac{\sigma'_f}{\sigma'_p}\right)\right] \qquad (10.6.2.4.3\text{-}4)$$

- For normally consolidated soils where $\sigma'_p = \sigma'_o$:

$$S_c = H_c C_{c\varepsilon}\log\left(\frac{\sigma'_f}{\sigma'_p}\right) \qquad (10.6.2.4.3\text{-}5)$$

- For underconsolidated soils where $\sigma'_p < \sigma'_o$:

$$S_c = H_c C_{c\varepsilon}\log\left(\frac{\sigma'_f}{\sigma'_{pc}}\right) \qquad (10.6.2.4.3\text{-}6)$$

where:

H_c = initial height of compressible soil layer (mm){ft.}

e_o = void ratio at initial vertical effective stress (dim.)

C_r = recompression index, as specified in Figure 1 (dim.)

C_c = compression index, as specified in Figure 1 (dim.)

$C_{r\varepsilon}$ = recompression ratio, as specified in Figure 2 (dim.)

$C_{c\varepsilon}$ = compression ratio, as specified in Figure 2 (dim.)

σ'_p = maximum past vertical effective stress in soil at midpoint of soil layer under consideration (MPa){ksf}

σ'_o = initial vertical effective stress in soil at midpoint of soil layer under consideration (MPa){ksf}

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To account for the decreasing stress with increased depth below a footing and variations in soil compressibility with depth, the compressible layer should be divided into vertical increments, i.e., typically 1500 to 3000 mm {5.0 to 10.0 ft.} for most normal width footings for highway applications, and the consolidation settlement of each increment analyzed separately. The number and thickness of compressible layers will depend on the total depth of compressible soil below the footing and the desired accuracy of the settlement computations. In general, layers should be thinnest near the footing base to account for rapid changes in vertical stress distribution due to the applied stress, and variations in σ_p' with depth. The total value of S_c is the summation of S_c for each increment.

The magnitude of consolidation settlement depends on the consolidation properties of the soil. These properties include the compression and recompression constants, C_c and C_r, or $C_{c\varepsilon}$, and $C_{r\varepsilon}$; the preconsolidation stress, σ'_p; the current, initial vertical effective stress, σ'_o; and the final vertical effective stress after application of additional loading, σ'_f. An overconsolidated soil has been subjected to larger stresses in the past than at present. This could be a result of preloading by previously overlying strata, desiccation, groundwater lowering, glacial overriding or an engineered preload. If $\sigma'_o = \sigma'_p$, the soil is normally consolidated. Because the recompression constant is typically about an order of magnitude smaller than the compression constant, an accurate determination of the preconsolidation stress, σ'_p, is needed to make reliable estimates of consolidation settlement.

The reliability of consolidation settlement estimates is also affected by the quality of the consolidation test sample and by the accuracy with which changes in σ'_p with depth are known or estimated. As shown in Figure C1, the slope of the e or ε_v versus log σ'_v curve and the location of σ'_p can be strongly affected by the quality of samples used for the laboratory consolidation tests. In general, the use of poor quality samples will result in an overestimate of consolidation settlement. Typically, the value of σ'_p will vary with depth as shown in Figure C2. If the variation of σ'_p with depth is unknown, e.g., only one consolidation test was conducted in the soil profile, actual settlements could be higher or lower than the computed value based on a single value of σ'_p.

The cone penetrometer test may be used to improve understanding of both soil layering and variation of σ'_p with depth by correlation to laboratory tests from discrete locations.

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σ'_f = final vertical effective stress in soil at midpoint of soil layer under consideration (MPa){ksf}

σ'_{pc} = current vertical effective stress in soil, not including the additional stress due to the footing loads, at midpoint of soil layer under consideration (MPa){ksf}

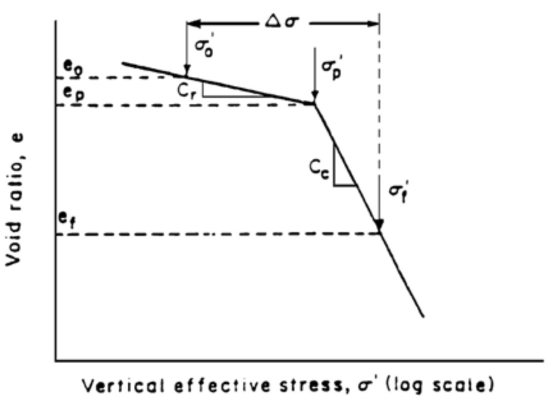

Figure 10.6.2.4.3-1 Typical Consolidation Compression Curve for Overconsolidated Soil: Void Ratio versus Vertical Effective Stress, EPRI (*1983*).

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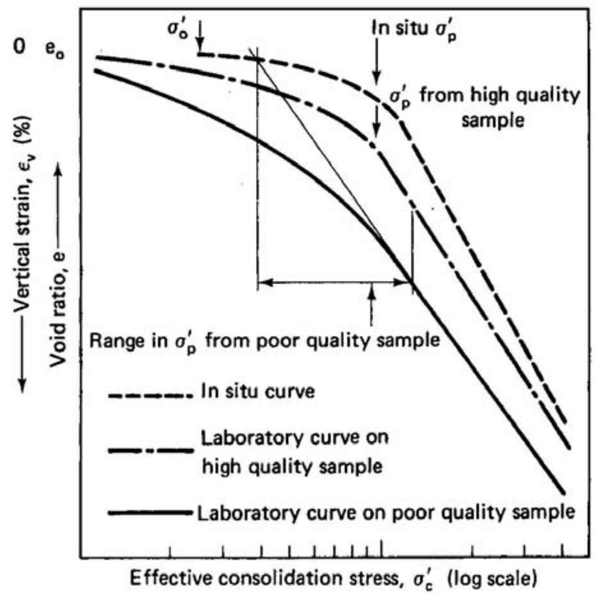

Figure C10.6.2.4.3-1 Effects of Sample Quality on Consolidation Test Results, Holtz and Kovacs (1981).

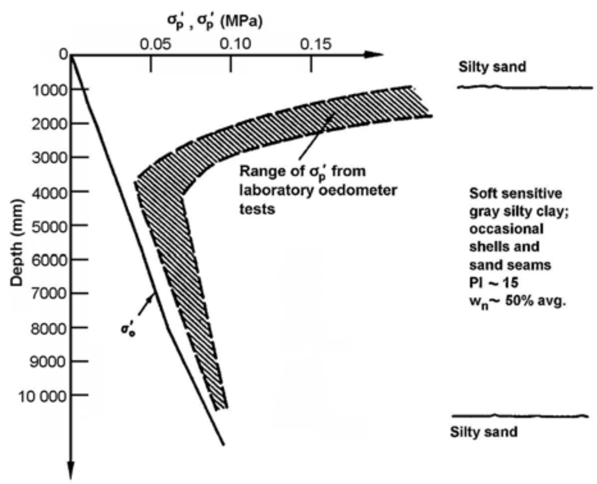

Figure C10.6.2.4.3-2 Typical Variation of Preconsolidation Stress with Depth for Marine Clays, Holtz and Kovacs (1981).

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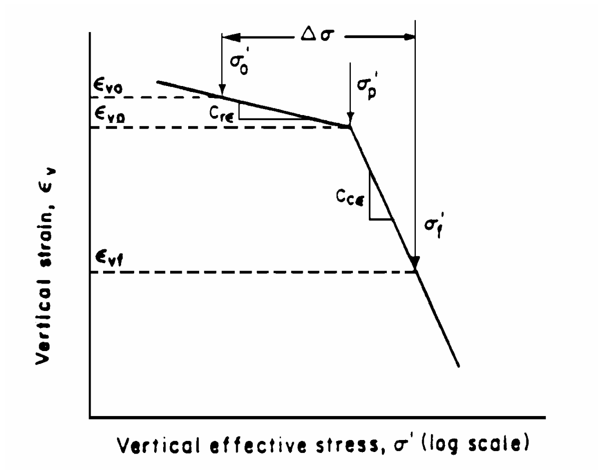

Figure 10.6.2.4.3-2 Typical Consolidation Compression Curve for Overconsolidated Soil: Vertical Strain versus Vertical Effective Stress, EPRI (*1983*).

If the footing width, B, is small relative to the thickness of the compressible soil, H_c, the effect of three-dimensional loading shall be considered and shall be taken as:

$$S_{c(3-D)} = \mu_c S_{c(1-D)} \qquad (10.6.2.4.3\text{-}7)$$

where:

μ_c = reduction factor taken as specified in Figure 3 (dim.)

$S_{c(1\text{-}D)}$ = single dimensional consolidation settlement (mm){ft.}

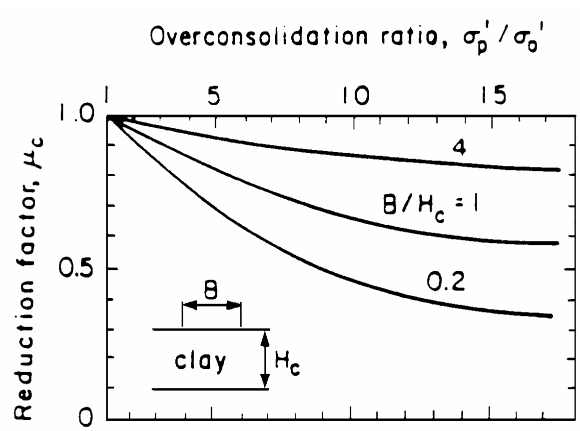

Figure 10.6.2.4.3-3 Reduction Factor to Account for Effects of Three-Dimensional Consolidation Settlement (*EPRI, 1983*).

Due to the difficulty in accurately calculating consolidation settlement, 3-D effects are typically ignored since they reduce the predicted amount of settlement and may lead to underestimation of settlement.

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The time, t, to achieve a given percentage of the total estimated one-dimensional consolidation settlement shall be taken as:

$$t = \frac{TH_d^{\,2}}{c_v} \qquad (10.6.2.4.3\text{-}8)$$

where:

T = time factor taken as specified in Figure 4 for the excess pore pressure distributions shown in the figure (dim.)

H_d = length of longest drainage path in compressible layer under consideration (mm){ft.}

c_V = coefficient of consolidation taken from the results of laboratory or in-situ testing (mm^2/yr.) {ft.^{2}/yr}

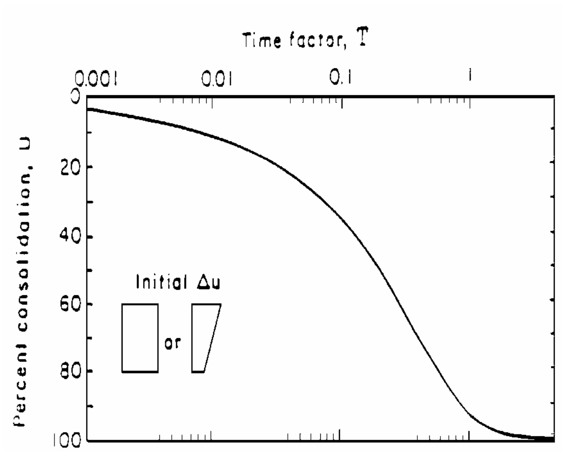

Figure 10.6.2.4.3-4 Percentage of Consolidation as a Function of Time Factor, T (*EPRI, 1983*).

Where laboratory test results are expressed in terms of void ratio, e, the secondary settlement of footings on cohesive soil shall be taken as:

$$S_s = \frac{C_\alpha}{1+e_o} H_c \log\left(\frac{t_2}{t_1}\right) \qquad (10.6.2.4.3\text{-}9)$$

COMMENTARY

Consolidation occurs when a saturated compressible layer of soil is loaded and water is squeezed out of the layer. The time required for the (primary) consolidation process to end will depend on the permeability of the soil. Because the time factor, T, is defined as logarithmic, the consolidation process theoretically never ends. The practical assumption is usually made that the additional consolidation past 90 percent or 95 percent consolidation is negligible, or is taken into consideration as part of the total long term settlement.

Refer to Winterkorn and Fang (*1975*) for values of T for excess pore pressure distributions other than indicated in Figure 4.

The length of the drainage path is the longest distance from any point in a compressible layer to a drainage boundary at the top or bottom of the compressible soil unit. Where a compressible layer is located between two drainage boundaries, H_d equals one-half the actual height of the layer. Where a compressible layer is adjacent to an impermeable boundary (usually below), H_d equals the full height of the layer.

Computations to predict the time rate of consolidation based on the result of laboratory tests generally tend to over-estimate the actual time required for consolidation in the field. This over-estimation is principally due to:

- The presence of thin drainage layers within the compressible layer that are not observed from the subsurface exploration nor considered in the settlement computations,
- The effects of three-dimensional dissipation of pore water pressures in the field, rather than the one-dimensional dissipation that is imposed by laboratory odometer tests and assumed in the computations, and
- The effects of sample disturbance, which tend to reduce the permeability of the laboratory tested samples.

If the total consolidation settlement is within the serviceability limits for the structure, the time rate of consolidation is usually of lesser concern for spread footings. If the total consolidation settlement exceeds the serviceability limitations, superstructure damage will occur unless provisions are made for timing of closure pours as a function of settlement, simple support of spans and/or periodic jacking of bearing supports.

The use of spread footings when total consolidation settlement exceeds serviceability limitations is undesirable and another foundation type should be considered.

The secondary compression component of settlement results from compression of bonds between individual clay particles and domains, as well as other effects on the microscale that are not yet clearly understood (*Holtz and Kovacs, 1981*). Secondary settlement is most important for highly plastic clays and organic and micaceous soils. Accordingly, secondary settlement predictions should be

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Where laboratory test results are expressed in terms of vertical strain, ε_v, the secondary settlement of footings on cohesive soils shall be taken as:

$$S_s = C_{\alpha\varepsilon} H_c \log\left(\frac{t_2}{t_1}\right) \quad (10.6.2.4.3\text{-}10)$$

where:

H_c = initial height of compressible soil layer (mm){ft.}

e_o = void ratio at initial vertical effective stress (dim.)

t_1 = time when secondary settlement begins, i.e., typically at a time equivalent to 90 percent average degree of primary consolidation (yr.)

t_2 = arbitrary time that could represent the service life of the structure (yr.)

C_α = secondary compression index estimated from the results of laboratory consolidation testing of undisturbed soil samples (dim.)

$C_{\alpha\varepsilon}$ = modified secondary compression index estimated from the results of laboratory consolidation testing of undisturbed soil samples (dim.)

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considered as approximate estimates only.

If secondary compression is estimated to exceed serviceability limitations, either deep foundations or ground improvement should be considered to mitigate the effects of secondary compression. Experience indicates preloading and surcharging may not be effective in eliminating secondary compression.

10.6.2.4.4 Settlement of Footings on Rock

For footings bearing on fair to very good rock, according to the Geomechanics Classification system, as defined in D10.4.6.4, and designed in accordance with the provisions of this Section, elastic settlements may generally be assumed to be less than 15 mm {0.5 in}. When elastic settlements of this magnitude are unacceptable or when the rock is not competent, an analysis of settlement based on rock mass characteristics shall be made.

Where rock is broken or jointed (relative rating of 10 or less for *RQD* and joint spacing), the rock joint condition is poor (relative rating of 10 or less) or the criteria for fair to very good rock are not met, a settlement analysis should be conducted, and the influence of rock type, condition of discontinuities, and degree of weathering shall be considered in the settlement analysis.

The elastic settlement of footings on broken or jointed rock, in mm {ft}, should be taken as:

- For circular (or square) footings;

  Metric Units:

C10.6.2.4.4

In most cases, it is sufficient to determine settlement using the average bearing stress under the footing.

Where the foundations are subjected to a very large load or where settlement tolerance may be small, settlements of footings on rock may be estimated using elastic theory. The stiffness of the rock mass should be used in such analyses.

The accuracy with which settlements can be estimated by using elastic theory is dependent on the accuracy of the estimated rock mass modulus, E_m. In some cases, the value of E_m can be estimated through empirical correlation with the value of the modulus of elasticity for the intact rock between joints. For unusual or poor rock mass conditions, it may be necessary to determine the modulus from in-situ tests, such as plate loading and pressuremeter tests.

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$$\rho = q_o\left(1-\nu^2\right)\frac{\gamma I_p}{E_m}\text{ ; and} \qquad (10.6.2.4.4\text{-}1)$$

U.S. Customary Units:

$$\rho = q_o\left(1-\nu^2\right)\frac{\gamma I_p}{144E_m}$$

in which:

$$I_p = \frac{\left(\sqrt{\pi}\right)}{\beta_z} \qquad (10.6.2.4.4\text{-}2)$$

- For rectangular footings;

  Metric Units:

$$\rho = q_o\left(1-\nu^2\right)\frac{BI_p}{E_m} \qquad (10.6.2.4.4\text{-}3)$$

U.S. Customary Units:

$$\rho = q_o\left(1-\nu^2\right)\frac{BI_p}{144E_m}$$

in which:

$$I_p = \frac{\left(L/B\right)^{1/2}}{\beta_z} \qquad (10.6.2.4.4\text{-}4)$$

where:

q_o = applied vertical stress at base of loaded area (MPa){ksf}

ν = Poisson's Ratio (dim.)

r = radius of circular footing or B/2 for square footing (mm){ft.}

I_p = influence coefficient to account for rigidity and dimensions of footing (dim.)

E_m = rock mass modulus (MPa){ksi}

β_z = factor to account for footing shape and rigidity (dim.)

Values of I_p should be computed using the β_z values presented in Table 1 for rigid footings. Where the results of laboratory testing are not available, values of Poisson's

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ratio, ν, for typical rock types may be taken as specified in Table DC10.4.6.5-2. Determination of the rock mass modulus, E_m, should be based on the methods described in D10.4.6.5.

Table 10.6.2.4.4-1 Elastic Shape and Rigidity Factors, EPRI (1983).

L/*B*	Flexible, β_z (average)	β_z Rigid
Circular	1.04	1.13
1	1.06	1.08
2	1.09	1.10
3	1.13	1.15
5	1.22	1.24
10	1.41	1.41

If soil-filled discontinuities are present below a footing, settlement of such layers should be estimated using the procedures for footings on soil.

10.6.2.5 OVERALL STABILITY

The overall stability of footings, slope and foundation soil or rock shall be evaluated using Load Combination Service I Limit State; the provisions of D3.4.1, D10.5.2.3 and D11.6.2.3 and the resistance factors given in Table 1.

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Overall stability refers to deep seated failures.

Until a computer program is developed to analyze overall stability which incorporates the LRFD methodology, the Department will use the factor of safety method where S.F. = 1/φ and the GSTABL computer program (Gregory Geotechnical Software, 2003).

Table 10.6.2.5-1 Minimum Resistance Factors, φ, for Overall Stability at the Service Limit State

GEOMETRY	SPT* AND VISUAL CLASSIFICATION	SPT* AND LAB CLASSIFICATION	SPT* AND LAB STRENGTH TESTING
General Spread Footing	0.55	0.65	0.75
Abutment Supported Above Retaining Wall	Not Acceptable	0.55	0.65

* Standard Penetration Test data from soil borings.

The overall stability shall be evaluated using limiting equilibrium methods of analysis, such as those employed by the GSTABL computer program, (Gregory Geotechnical Software, 2003). Depending on whether the anticipated mode of failure is circular or planar, the Modified Bishop Method (or equivalent) or the Janbu Method (or equivalent), respectively, shall be used to evaluate the overall stability of the slope. Line, point, or area loads due to shallow foundations located within the soil mass defined by the failure surface shall be included in the analysis.

The Modified Bishop Method is appropriate where the failure surface is expected to be circular. The Janbu Method is appropriate where the failure surface is expected to be planar.

If the foundation within the soil mass defined by the failure surface is supported by deep foundations, i.e. piles or drilled shafts, that bear in material below the failure surface, it is not necessary to include loads due to those foundations in the analysis.

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10.6.2.6 BEARING RESISTANCE AT THE SERVICE LIMIT STATE

The settlement of spread footings shall be estimated in accordance with D10.6.2.4 and compared to the movement criteria established in D10.5.2.2.

10.6.3 Strength Limit State Design

10.6.3.1 BEARING RESISTANCE OF SOIL

10.6.3.1.1 General

The Department's preferred method for predicting bearing resistance of soils under footings is the theoretical method given in D10.6.3.1.2.

Bearing resistance of spread footings shall be determined based on the highest anticipated position of groundwater level at the footing location.

The factored bearing resistance, q_R, at the strength limit state shall be taken as:

$$q_R = \varphi_b \; q_n \qquad (10.6.3.1.1\text{-}1)$$

where:

φ_b = resistance factor specified in D10.5.5.2.2

q_n = nominal bearing resistance (MPa){ksf}

Where loads are eccentric, the effective footing dimensions, L' and B', as specified in D10.6.1.3, shall be used instead of the overall dimensions L and B in all equations, tables and figures pertaining to bearing resistance.

10.6.3.1.2 Theoretical Estimation

10.6.3.1.2a Basic Formulation

The nominal bearing resistance shall be estimated using

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C10.6.3.1.1

The bearing resistance of footings on soil should be evaluated using soil shear strength parameters that are representative of the soil shear strength under the loading conditions being analyzed. The bearing resistance of footings supported on granular soils should be evaluated for both permanent dead loading conditions and short-duration live loading conditions using effective stress methods of analysis and drained soil shear strength parameters. The bearing resistance of footings supported on cohesive soils should be evaluated for short-duration live loading conditions using total stress methods of analysis and undrained soil shear strength parameters. In addition, the bearing resistance of footings supported on cohesive soils should be evaluated for permanent dead loading conditions using effective stress methods of analysis and drained soil shear strength parameters.

The position of the groundwater table can significantly influence the bearing resistance of soils through its effect on shear strength and unit weight of the foundation soils. In general, the submergence of soils will reduce the effective shear strength of cohesionless (or granular) materials, as well as the long-term (or drained) shear strength of cohesive (clayey) soils. Moreover, the effective unit weights of submerged soils are about half of those for the same soils under moist conditions. Thus, submergence may lead to a significant reduction in the bearing resistance provided by the foundation soils, and it is essential that the bearing resistance analyses be carried out under the assumption of the highest groundwater table expected within the service life of the structure.

Because the effective dimensions will vary slightly for each limit state under consideration, strict adherence to this provision will require re-computation of the nominal bearing resistance at each limit state.

C10.6.3.1.2a

The three modes of shear failure (general, local, and

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accepted soil mechanics theories and should be based on measured soil parameters. The soil parameters used in the analyses shall be representative of the soil shear strength under the considered loading and subsurface conditions.

The nominal bearing resistance of spread footings on cohesionless soils shall be evaluated using effective stress analyses and drained soil strength parameters.

The nominal bearing resistance of spread footings on cohesive soils shall be evaluated for total stress analyses and undrained soil strength parameters. The bearing resistance of cohesive soils shall also be evaluated for permanent loading conditions using effective stress analyses and drained soil strength parameters.

For spread footings bearing on compacted soils, the nominal bearing resistance shall be evaluated using the more critical of either total or effective stress analyses.

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punching) are shown in Figure C1.

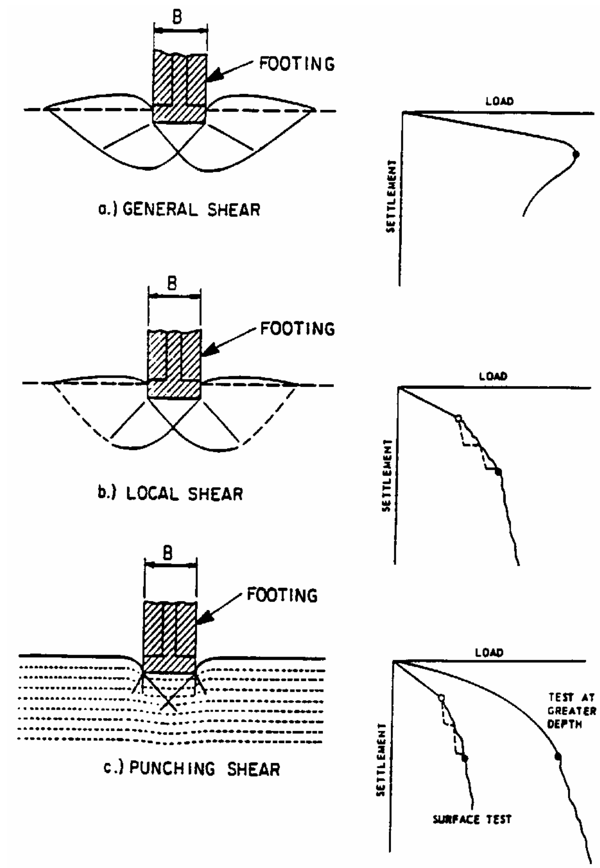

Figure C10.6.3.1.2a-1 Bearing Capacity Failure Modes for Footings on Soil (*Vesic 1963*).

General shear failure is characterized by a well-defined failure surface extending to the ground surface and is accompanied by sudden rotation and tilting of the footing and bulging of soil on both sides of the footing. General shear failure occurs in relatively incompressible soil and in saturated normally consolidated clays in undrained loading.

Local shear and punching shear are described in DC10.6.3.1.2b.

The failure mode for a particular footing depends primarily on the compressibility of the soil and the footing depth. The relationship between footing depth, mode of failure, and relative density for footings in sand is shown in Figure C2.

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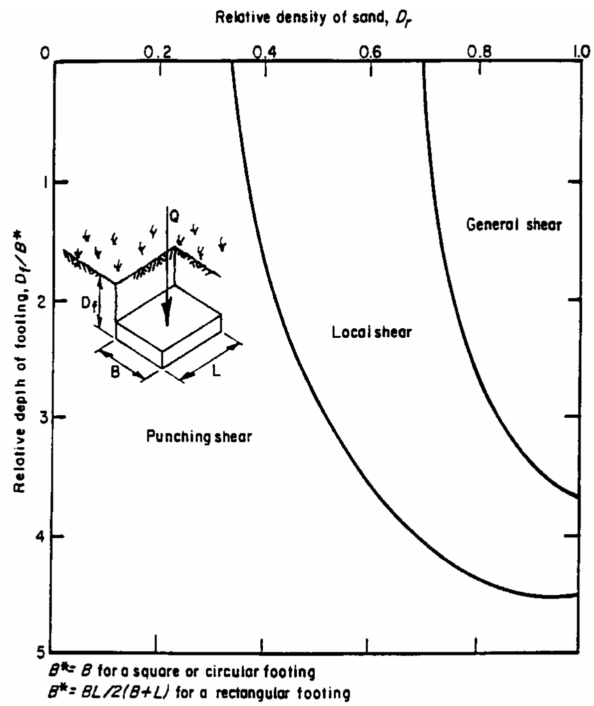

Figure C10.6.3.1.2a-2 Modes of Bearing Capacity Failure for Footings in Sand.

The equation for the ultimate bearing capacity for the case of general shear (Equation C1) includes three separate terms. The first is a function of the soil shear strength (ϕ_f and c), the second is a function of the footing width (B) and the third is a function of the footing depth (D_f) and soil unit weight (γ).

The general equation for theoretical estimation of ultimate bearing capacity (Equation 1) is an approximate, semi-empirical equation which assumes an infinitely long strip footing with a horizontal base carrying a concentric, vertical load, bearing well above the water table in a homogeneous, plastic soil with a horizontal ground surface.

Numerous investigators have developed bearing capacity factors (N_c, N_γ and N_q) for use in the general bearing capacity equation. The indicated values are those developed by Prandtl and Reissner for N_c and N_q, and those developed by Caquot and Kerisel for N_γ, which are generally considered the most reliable and are currently the most widely used by practicing engineers, Hunt (1986). The values shown in Table 1 can also be calculated using the following relationships:

$$N_q = \left(e^{\pi \tan \phi_f}\right) \tan^2\left(45° + \phi_f/2\right) \qquad \text{(C10.6.3.1.2d-1)}$$

$$N_c = (N_q - 1) \cot \phi_f \text{ (for } \phi_f > 0°) \qquad \text{(C10.6.3.1.2d-2)}$$

$$N_c = 2 + \pi \text{ (for } \phi_f = 0°) \qquad \text{(C10.6.3.1.2d-3)}$$

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$N_\gamma = 2(N_q + 1) \tan \phi_f$ (C10.6.3.1.2d-4)

The bearing resistance at the strength limit state (for general shear failure) may be estimated using the following relationship for continuous footings (i.e., L > 5B):

Metric Units:

$$q_n = cN_c + 0.5g\gamma BN_\gamma(10^{-9}) + g\gamma D_f N_q(10^{-9}) \quad (10.6.3.1.2a\text{-}1)$$

U.S. Customary Units:

$$q_n = cN_c + 0.5\gamma BN_\gamma + \gamma D_f N_q$$

A modified form of the general bearing capacity equation may be used to account for the effects of footing shape, ground surface slope and inclined loading as follows:

Metric Units:

$$q_{ult} = cN_c s_c i_c + 0.5g\gamma BN_\gamma s_\gamma i_\gamma\,(10^{-9}) + g\gamma D_f N_q s_q i_q\,(10^{-9}) \quad (10.6.3.1.2a\text{-}2)$$

U.S. Customary Units:

$$q_{ult} = cN_c s_c i_c + 0.5\gamma BN_\gamma s_\gamma i_\gamma + \gamma D_f N_q s_q i_q$$

where:

c = cohesion, taken as undrained shear strength (MPa){ksf}

N_c = cohesion term (undrained loading) bearing capacity factor as specified in Table 1 (dim.)

N_q = surcharge (embedment) term (drained or undrained loading) bearing capacity factor as specified in Table 1 (dim.)

N_γ = unit density (footing width) term (drained loading) bearing capacity factor as specified in Table 1 (dim.)

γ = effective unit weight of soil above or below the bearing depth of the footing (kg/m^3) {kcf}

D_f = footing embedment depth (mm){ft.}

B = footing width (mm){ft.}

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s_c, s_γ, s_q = For footing shapes other than continuous footings (i.e., L < 5B), footing shape correction factors as specified in Table 3 (dim.). For L ≥ 5B, shape factors equal 1.0.

i_c, i_γ, i_q = load inclination factors determined from Eqs. 5 or 6, and 7 and 8 (dim.)

Refer to Figures DC10.6.1.3-1 and DC10.6.3.1.2a-3 for loading definitions and footing dimensions. For cases in which the loading is eccentric, the terms L and B shall be replaced by L' and B', respectively, in the above equations.

For ϕ_f = 0:

$$i_c = 1 - \left(\frac{nH}{cBLN_c}\right) \quad (10.6.3.1.2a\text{-}3)$$

For ϕ_f > 0:

$$i_c = i_q - [(1 - i_q)/N_c \tan \phi_f] \text{(for } \phi_f > 0) \quad (10.6.3.1.2a\text{-}4)$$

in which:

$$i_q = \left[1 - \frac{H}{(V + cBL\cot\phi_f)}\right]^n \quad (10.6.3.1.2a\text{-}5)$$

$$i_\gamma = \left[1 - \frac{H}{V + cBL\cot\phi_f)}\right]^{(n+1)} \quad (10.6.3.1.2a\text{-}6)$$

$$n = \left[(2 + L/B)/(1 + L/B)\right]\cos^2\theta$$
$$... + \left[(2 + B/L)/(1 + B/L)\right]\sin^2\theta \quad (10.6.3.1.2a\text{-}7)$$

where:

B = footing width (mm){ft.}

L = footing length (mm){ft.}

H = unfactored horizontal load (N){kips}

COMMENTARY

The shape factors are semi-empirical factors based on load tests of footings with various shapes.

Depth factors are not included in Equation D10.6.3.1.2a-2, which treats all soil above the footing bearing level as a surcharge load, and neglects the shearing resistance of the overburden along the failure surface. Depth factors to account for overburden shearing resistance have been developed by several investigators and could be applied where the soil above the footing is expected to provide significant shear resistance. However, Vesic notes that there is evidence that the additional shearing resistance is negligible for backfilled footings or footings in compressible overburden and should, therefore, be neglected, Winterkorn and Fang (1975).

The effect of an inclined load is to induce a horizontal component in the foundation reaction, such that failure of the footing may occur by either general shear (bearing resistance) or sliding. Inclined loads have the effect of reducing the bearing capacity of a footing.

The basis of the load inclination factors computed by Eqs. 3 to 6 is a combination of bearing resistance theory and small scale load tests on 25 mm wide plates on London Clay and Ham River Sand (*Meyerhof, 1953*). Therefore, the factors do not take into consideration the effects of depth of embedment. Meyerhof further showed that for footings with a depth of embedment ratio of D_f/B = 1, the effects of load inclination on bearing resistance are relatively small. The theoretical formulation of load inclination factors were further examined by Brinch-Hansen (*1970*), with additional modification by Vesic (*1973*) into the form provided in Eqs. 3 to 6.

Figure C3 shows the convention for determining the θ angle in Eq. 7.

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V = unfactored vertical load (N){kips}

θ = projected direction of load in the plane of the footing, measured from the side of length L (deg.)

Failure by sliding shall be considered by comparing the factored horizontal (tangential) component of force on the footing to the maximum factored shear resistance in accordance with D10.6.3.4.

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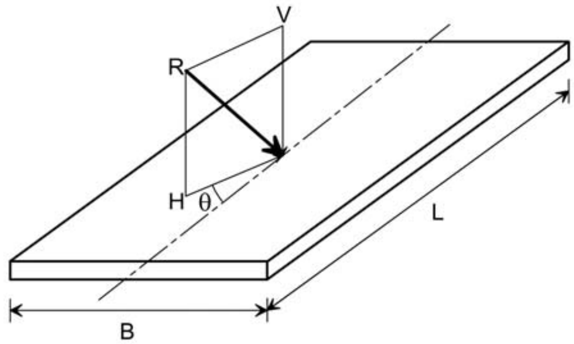

Figure C10.6.3.1.2a-3 Inclined Loading Conventions.

Table 10.6.3.1.2a-1 Bearing Capacity Factors Nc (Prandtl, 1921), Nq (Reissner, 1924), and Nγ (Vesic, 1975).

ϕ_f	N_c	N_q	N_γ	ϕ_f	N_c	N_q	N_γ
0	5.14	1.0	0.0	23	18.1	8.7	8.2
1	5.4	1.1	0.1	24	19.3	9.6	9.4
2	5.6	1.2	0.2	25	20.7	10.7	10.9
3	5.9	1.3	0.2	26	22.3	11.9	12.5
4	6.2	1.4	0.3	27	23.9	13.2	14.5
5	6.5	1.6	0.5	28	25.8	14.7	16.7
6	6.8	1.7	0.6	29	27.9	16.4	19.3
7	7.2	1.9	0.7	30	30.1	18.4	22.4
8	7.5	2.1	0.9	31	32.7	20.6	26.0
9	7.9	2.3	1.0	32	35.5	23.2	30.2
10	8.4	2.5	1.2	33	38.6	26.1	35.2
11	8.8	2.7	1.4	34	42.2	29.4	41.1
12	9.3	3.0	1.7	35	46.1	33.3	48.0
13	9.8	3.3	2.0	36	50.6	37.8	56.3
14	10.4	3.6	2.3	37	55.6	42.9	66.2
15	11.0	3.9	2.7	38	61.4	48.9	78.0
16	11.6	4.3	3.1	39	67.9	56.0	92.3
17	12.3	4.8	3.5	40	75.3	64.2	109.4
18	13.1	5.3	4.1	41	83.9	73.9	130.2
19	13.9	5.8	4.7	42	93.7	85.4	155.6
20	14.8	6.4	5.4	43	105.1	99.0	186.5
21	15.8	7.1	6.2	44	118.4	115.3	224.6
22	16.9	7.8	7.1	45	133.9	134.9	271.8

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Table 10.6.3.1.2a-2 Shape Correction Factors sc, sγ, sq.

Factor	Friction Angle	Cohesion Term (s_c)	Unit Weight Term (s_γ)	Surcharge Term (s_q)
Shape Factors s_c, s_γ, s_q	$\phi_f = 0$	$1+\left(\frac{B}{5L}\right)$	1.0	1.0
	$\phi_f > 0$	$1+\left(\frac{B}{L}\right)\left(\frac{N_q}{N_c}\right)$	$1-0.4\left(\frac{B}{L}\right)$	$1+\left(\frac{B}{L}\tan\phi_f\right)$

For circular footings, B equals L. For cases in which the loading is eccentric, the terms L and B shall be replaced by L' and B', respectively, in the above equations.

10.6.3.1.2b Considerations for Punching Shear

If local or punching shear failure is possible, the nominal bearing resistance shall be estimated using reduced shear strength parameters c^* and ϕ^* in Eqs. 1 and 2. The reduced shear parameters may be taken as:

$$c^* = 0.67c \quad (10.6.3.1.2b\text{-}1)$$

$$\phi^* = \tan^{-1}(0.67\tan\ \phi_f) \quad (10.6.3.1.2b\text{-}2)$$

where:

c^* = reduced effective stress soil cohesion for punching shear (MPa){ksf}

ϕ^* = reduced effective stress soil friction angle for punching shear (deg.)

C10.6.3.1.2b

Local shear failure is characterized by a failure surface that is similar to that of a general shear failure but that does not extend to the ground surface, ending somewhere in the soil below the footing. Local shear failure is accompanied by vertical compression of soil below the footing and visible bulging of soil adjacent to the footing but not by sudden rotation or tilting of the footing. Local shear failure is a transitional condition between general and punching shear failure. Punching shear failure is characterized by vertical shear around the perimeter of the footing and is accompanied by a vertical movement of the footing and compression of the soil immediately below the footing but does not affect the soil outside the loaded area. Punching shear failure occurs in loose or compressible soils, in weak soils under slow (drained) loading, and in dense sands for deep footings subjected to high loads.

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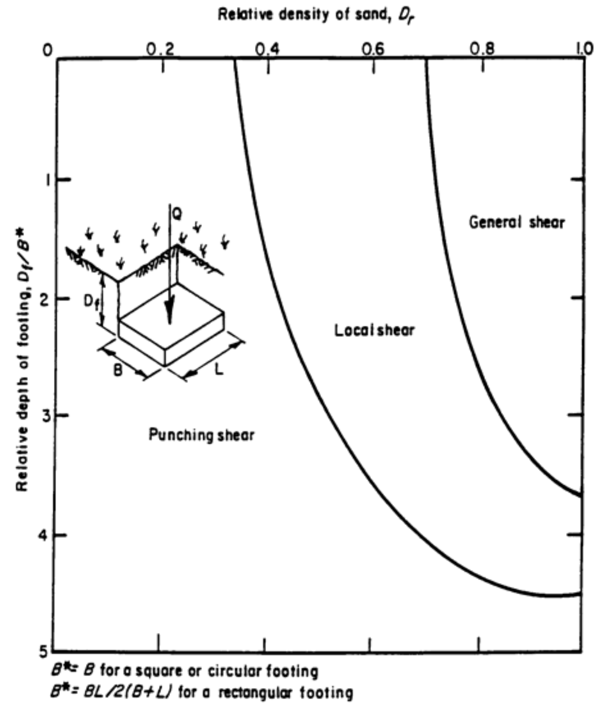

Figure C10.6.3.1.2b-1 Modes of Bearing Capacity Failure for Footings in Sand.

10.6.3.1.2c Considerations for Footings on Slopes

For footings located on slopes or within 3B of a slope crest, q_n may be determined using the following revised version of Equation D10.6.3.1.2a-2.

Metric Units: $q_n = cN_{cq}s_ci_c + 0.5\gamma gBN_{\gamma q}s_\gamma i_\gamma(10^{-9})$ (10.6.3.1.2c-1)

U.S. Customary Units: $q_n = cN_{cq}s_ci_c + 0.5\gamma BN_{\gamma q}s_\gamma i_\gamma$

Refer to Figure 1 for values of N_{cq} and $N_{\gamma q}$ for footings on slopes and Figure 2 for values of N_{cq} and $N_{\gamma q}$ for footings at the top of slopes. For footings in or above cohesive soil slopes, the stability number in the figures, N_s, is defined as follows:

- For $B < H_S$:

$N_s = 0$ (10.6.3.1.2c-2)

- For $B \geq H_S$:

Metric Units: $N_s = \gamma gH_s(10^{-9})/c$ (10.6.3.1.2c-3)

U.S. Customary Units: $N_s = \gamma H_s/c$

where:

C10.6.3.1.2c

The effect of a sloping ground surface in front of a footing or an inclined footing base is to reduce bearing resistance. The reduction in bearing capacity is due to the decreased volume of soil providing passive resistance to failure on the downslope side of the footing. The modified bearing capacity factors (N_{cq} and $N_{\gamma q}$) are based on the stability number (N_s) for cohesive soil and the effective stress angle of friction (ϕ_f) for cohesionless soil or drained analysis of cohesive soil. For cohesive soils with $\phi = 0$, $N_{\gamma q} = 1.0$. For cohesionless soils with c = 0, N_{cq} is not needed. The ground slope factors do not account for pre-existing shearing stresses in the ground which, in combination with footing loads, may cause a slope failure.

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COMMENTARY

B = footing width (mm){ft.}

H_s = height of sloping ground mass (mm){ft.}

Overall stability shall be evaluated for footings on or adjacent sloping ground surfaces as described in D10.6.2.5

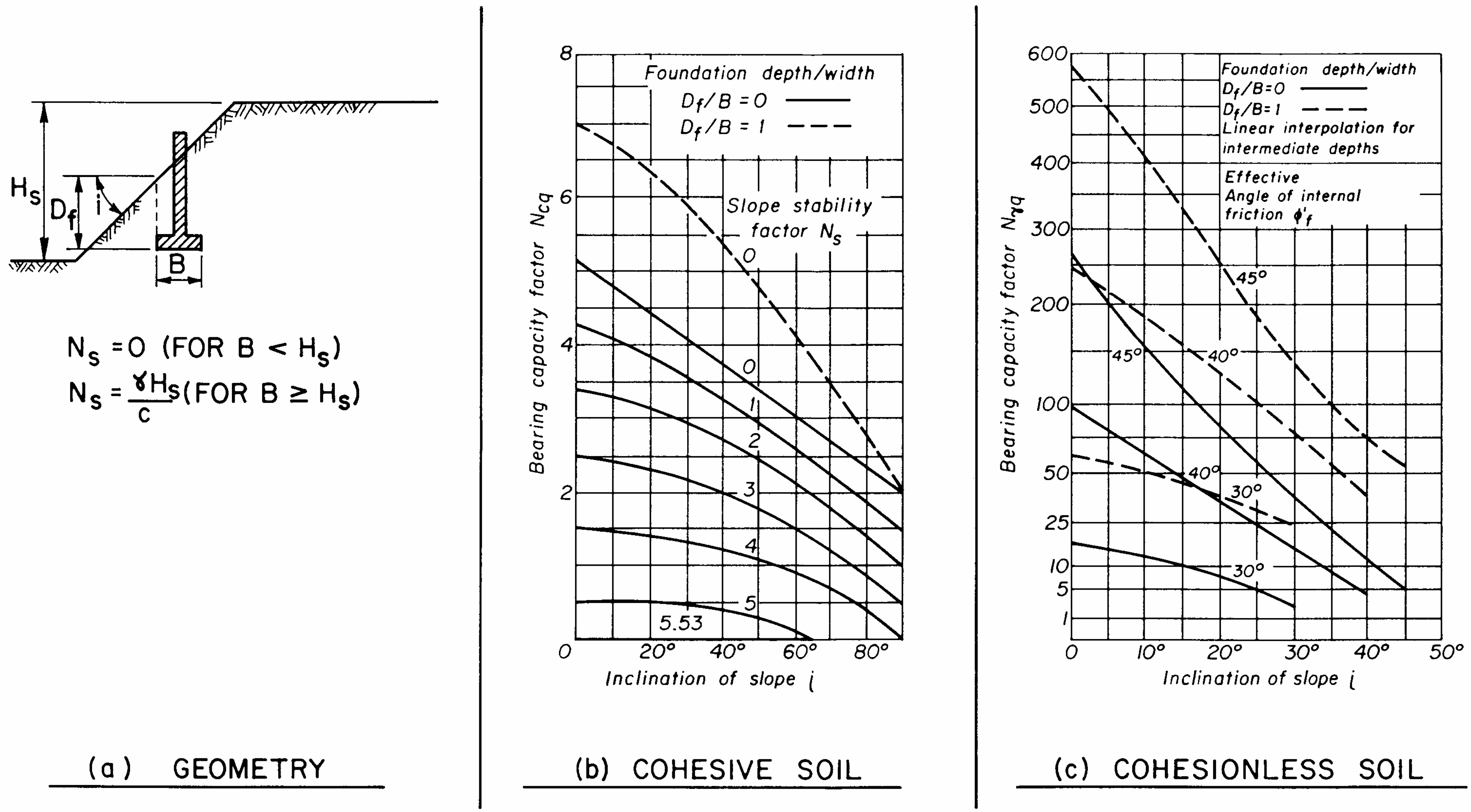

Figure 10.6.3.1.2c-1 - Modified Bearing Capacity Factors for Footing on Sloping Ground, Modified after Meyerhof (1957)

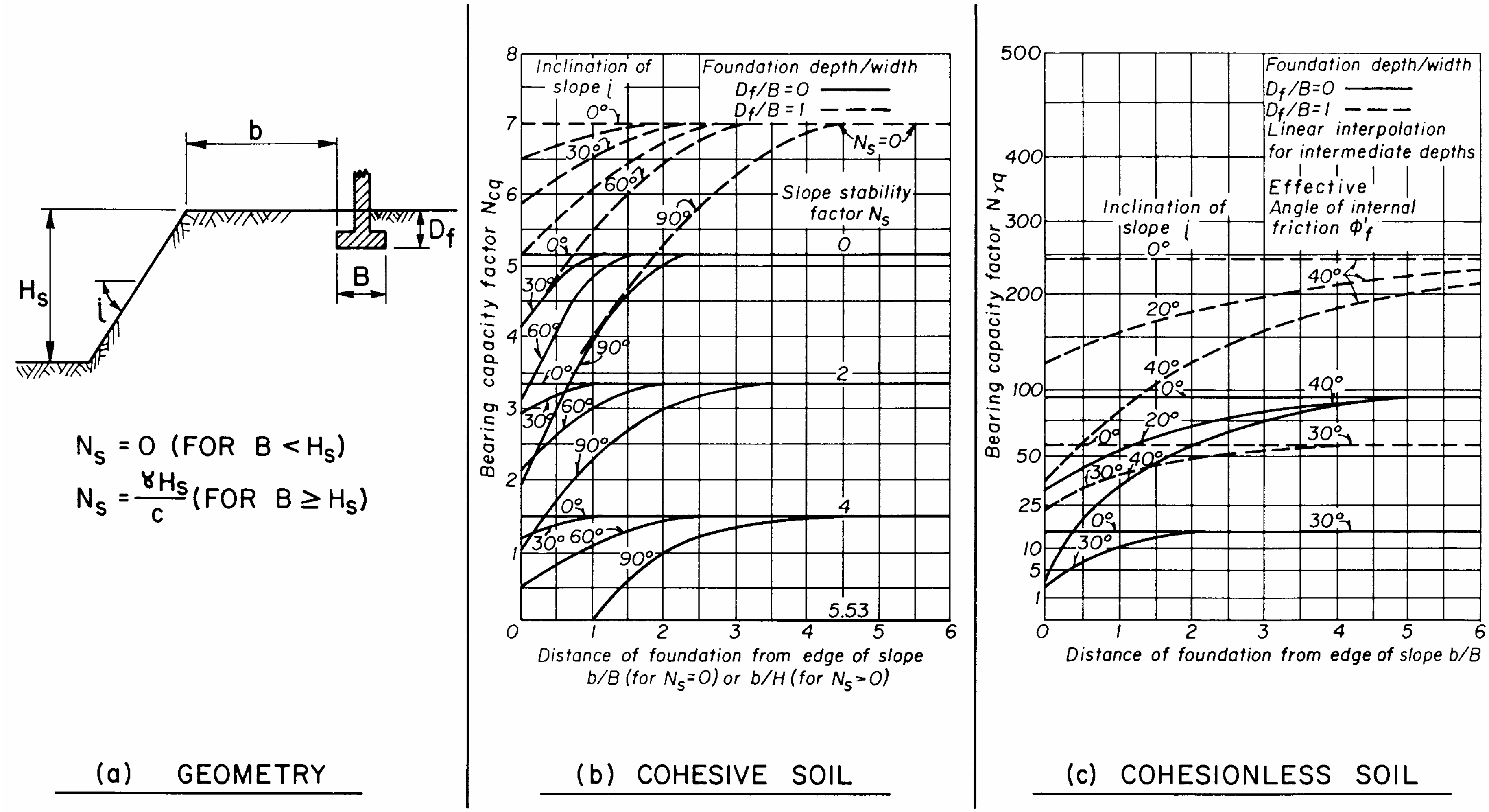

Figure 10.6.3.1.2c-2 - Modified Bearing Capacity Factors for Footing Adjacent Sloping Ground, Modified after Meyerhof (1957)

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10.6.3.1.2d Considerations for Two-Layer Soil Systems

If the soil profile is layered, the general bearing capacity equation shall be modified to account for differences in failure mode between the layered case and the homogeneous soil case assumed in Equation D10.6.3.1.2a-2.

Where the soil profile contains a second layer of soil with different properties affecting shear strength within a distance below the footing less than H_{crit}, the bearing resistance of the layered soil profile shall be determined using the provisions for two-layered soil systems herein. The distance H_{crit}, in mm {ft}, may be taken as:

$$H_{crit} = \frac{(3B)\ln\left(\frac{q_1}{q_2}\right)}{2\left(1+\frac{B}{L}\right)} \qquad (10.6.3.1.2d\text{-}1)$$

where:

q_1 = nominal bearing resistance of footing supported in the upper layer of a two-layer system, assuming the upper layer is infinitely thick (MPa){ksf}

q_2 = nominal bearing resistance of a fictitious footing of the same size and shape as the actual footing but supported on surface of the second (lower) layer of a two-layer system (MPa){ksf}

B = footing width (mm){ft.}

L = footing length (mm){ft.}

10.6.3.1.2e Two-layered Soil System in Undrained Loading

Where a footing is supported on a two-layered soil system subjected to undrained loading, the nominal bearing resistance may be determined using Equation D10.6.3.1.2a-1 with the following modifications:

Metric Units:

$$q_n = c_1 N_m + g\gamma D_f\,(10^{-9}) \qquad (10.6.3.1.2e\text{-}1)$$

U.S. Customary Units:

$$q_n = c_1 N_m + \gamma D_f$$

where:

c_1 = undrained shear strength of the top layer of soil as depicted in Figure 1 (MPa){ksf}

COMMENTARY

Bearing failure of spread footing foundations on layered cohesive soils occurs by two principal modes depending on whether the bearing stratum is underlain by a stiffer or softer soil unit. For cases where the soft layer overlies a stiffer layer, failure partially occurs by lateral flow of the soft soil layer. For cases where the stiff layer overlies a soft layer, failure usually occurs by punching of the stiffer soil into the underlying softer soil.

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N_m = a bearing capacity factor as specified below (dim.)

Where the bearing stratum overlies a stiffer cohesive soil, N_m, may be taken as specified in Figure 2.

Where the bearing stratum overlies a softer cohesive soil, N_m may be taken as:

$$N_m = \left(\frac{1}{\beta_m} + \kappa s_c N_c\right) \le s_c N_c \qquad (10.6.3.1.2e\text{-}2)$$

where:

$$\beta_m = \frac{BL}{2(B+L)H_{s2}} \qquad (10.6.3.1.2e\text{-}3)$$

$$\kappa = \frac{c_2}{c_1} \qquad (10.6.3.1.2e\text{-}4)$$

where:

β_m = the punching index (dim.)

c_1 = undrained shear strength of upper soil layer (MPa){ksf}

c_2 = undrained shear strength of lower soil layer (MPa){ksf}

H_{s2} = distance from bottom of footing to top of the second soil layer (mm){ft.}

s_c = shape correction factor determined from Table D10.6.3.1.2a-3

N_c = as determined by Table D10.6.3.1.2a-1

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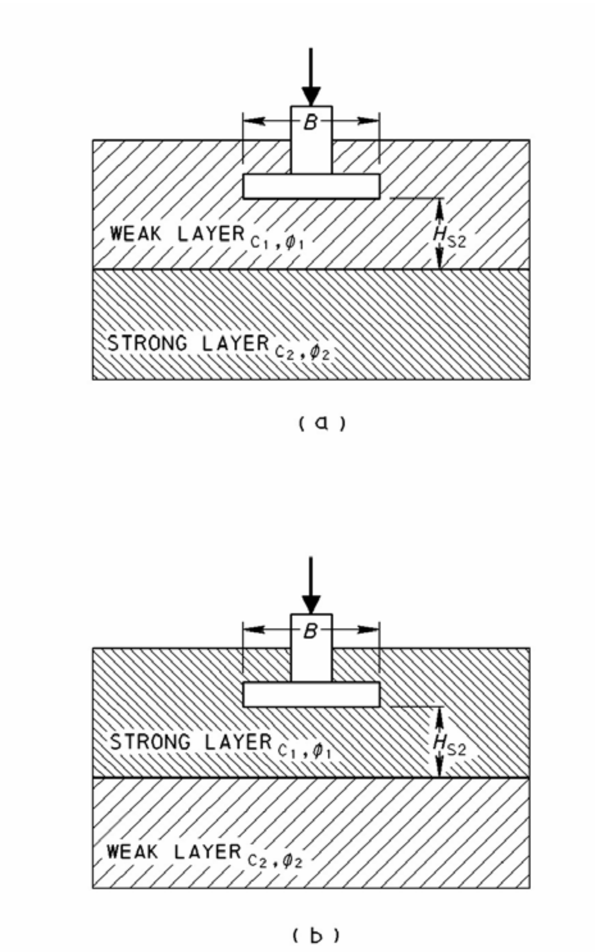

Figure 10.6.3.1.2e-1 Two-Layer Soil Profiles.

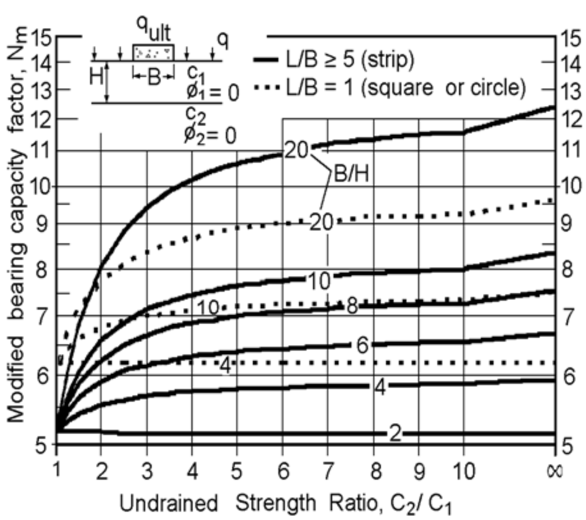

Figure 10.6.3.1.2e-2 Modified Bearing Factor for Two-Layer Cohesive Soil with Weaker Soil Overlying Stronger Soil (*EPRI, 1983*).

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10.6.3.1.2f Two-layered Soil System in Drained Loading

Where a footing supported on a two-layered soil system is subjected to a drained loading, the nominal bearing resistance, in MPa {ksf}, may be taken as:

$$q_n = \left[q_2 + \left(\frac{1}{K}\right)c_1' \cot\phi_1'\right] e^{2\left[1+\left(\frac{B}{L}\right)\right]K\tan\phi_1'\left(\frac{H}{B}\right)} - \left(\frac{1}{K}\right)c_1' \cot\phi_1' \qquad (10.6.3.1.2f\text{-}1)$$

where:

$$K = \frac{1-\sin^2\phi_1'}{1+\sin^2\phi_1'} \qquad (10.6.3.1.2f\text{-}2)$$

where:

c'_1 = drained shear strength of the top layer of soil as depicted in Figure D10.6.3.1.2e-1 (MPa){ksf}

q_2 = nominal bearing resistance of a fictitious footing of the same size and shape as the actual footing but supported on surface of the second (lower) layer of a two-layer system (MPa){ksf}. For a strong layer overlying a weak layer, use reduced shear strength values in accordance with D10.6.3.1.2b.

ϕ'_1 = effective stress angle of internal friction of the top layer of soil as depicted in Figure D10.6.3.1.2e-1(deg.)

10.6.3.1.2g Groundwater

Nominal bearing resistance shall be determined using the highest anticipated groundwater level at the footing location. The effect of ground water level on the ultimate bearing resistance shall be considered by using a weighted average soil unit weight in Equation D10.6.3.1.2a-2. If ϕ_f < 37°, the following equations may be used to determine the weighted average unit weight:

for $z_w \geq B$: use $\gamma = \gamma_m$ (no effect) (10.6.3.1.2g-1)

for $z_w < B$: use $\gamma = \gamma' + (z_w/B)(\gamma_m - \gamma')$ (10.6.3.1.2g-2)

for $z_w \leq 0$: use $\gamma = \gamma'$ (10.6.3.1.2g-3)

If $\phi_f \geq 37°$, the following equations may be used to determine the weighted average unit weight:

for $z_w \geq D$: use $\gamma = \gamma_m$ (no effect) (10.6.3.1.2g-4)

for $z_w < D$: $\gamma = (2D - z_w)(z_w\gamma_m/D^2) + (\gamma'/D^2)(D - z_w)^2$

COMMENTARY

C10.6.3.1.2f

If the upper layer is a cohesionless soil and ϕ_1' equals 25° to 50°, Eq. 1 reduces to:

$$q_n = q_2 e^{0.67\left[1+\left(\frac{B}{L}\right)\right]\frac{H}{B}} \qquad (C10.6.3.1.2f\text{-}1)$$

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(10.6.3.1.2g-5)

D = 0.5Btan (45° + ϕ_f/2) (10.6.3.1.2g-6)

for $z_w \leq 0$: use $\gamma = \gamma'$ (10.6.3.1.2g-7)

Refer to Figure 1 for definition of terms used in these equations.

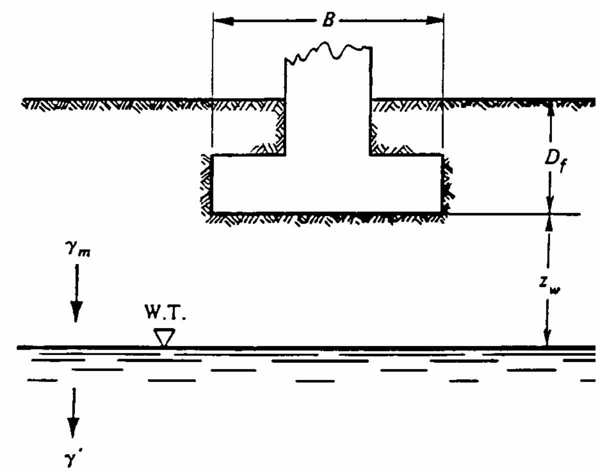

Figure 10.6.3.1.2g-1 - Definition Sketch for Influence of Groundwater Table on Bearing Capacity

10.6.3.1.2h Footing Base Inclination

Inclined footing bases shall not be used without the approval of the Chief Bridge Engineer.

10.6.3.1.3 Semiempirical Procedures

The nominal bearing resistance of foundation soils may be estimated from the results of in-situ tests or by observed resistance of similar soils. The use of a particular in-situ test and the interpretation of test results should take local experience into consideration. The following in-situ tests may be used:

- Standard Penetration Test
- Cone Penetration Test

The nominal bearing resistance in sand, in MPa {ksf}, based on SPT results may be taken as:

Metric Units:

$$q_n = \left(3.2\times10^{-5}\right)\bar{N}1_{60}\,B\left(C_{wq}\frac{D_f}{B}+C_{w\gamma}\right) \qquad (10.6.3.1.3\text{-}1)$$

COMMENTARY

C10.6.3.1.3

In application of these empirical methods, the use of average *SPT* blow counts and *CPT* tip resistances is specified. The resistance factors recommended for bearing resistance included in Table D10.5.5.2.2-1 assume the use of average values for these parameters. The use of lower bound values may result in an overly conservative design. However, depending on the availability of soil property data and the variability of the geologic strata under consideration, it may not be possible to reliably estimate the average value of the properties needed for design. In such cases, the Engineer may have no choice but to use a more conservative selection of design input parameters to mitigate the additional risks created by potential variability or the paucity of relevant data.

The original derivation of Eqs. 1 and 2 did not include inclination factors (*Meyerhof, 1956*).

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COMMENTARY

U.S. Customary Units:

$$q_n = \frac{\overline{N}1_{60}\,B}{5}\left(C_{wq}\frac{D_f}{B} + C_{w\gamma}\right)$$

where:

$\overline{N}1_{60}$ = average *SPT* blow count corrected for both overburden and hammer efficiency effects (blows/300 mm){blows/ft.} as specified in D10.4.6.2.4. Average the blow count over a depth range from the bottom of the footing to 1.5*B* below the bottom of the footing.

B = footing width (mm){ft.}

C_{wq}, $C_{w\gamma}$ = correction factors to account for the location of the ground water table as specified in Table 1 (dim.)

D_f = footing embedment depth taken to the bottom of the footing (mm){ft.}

The nominal bearing resistance, in MPa {ksf}, for footings on cohesionless soils based on *CPT* results may be taken as:

Metric Units:

$$q_n = \left(8.2\times10^{-5}\right)\overline{q_c}\,B\left(C_{wq}\frac{D_f}{B} + C_{w\gamma}\right) \qquad (10.6.3.1.3\text{-}2)$$

U.S. Customary Units:

$$q_n = \frac{\overline{q}_c B}{40}\left(C_{wq}\frac{D_f}{B} + C_{w\gamma}\right)$$

where:

$\overline{q_c}$ = average cone tip resistance within a depth range *B* below the bottom of the footing (MPa){ksf}

B = footing width (mm){ft.}

C_{wq}, $C_{w\gamma}$ = correction factors to account for the location of the ground water table as specified in Table 1 (dim.)

D_f = footing embedment depth taken to the bottom of the footing (mm){ft.}

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Table 10.6.3.1.3-1 Coefficients Cwq and Cwγ for Various Groundwater Depths.

D_w	C_{wq}	$C_{w\gamma}$
0.0	0.5	0.5
D_f	1.0	0.5
>1.5B + D_f	1.0	1.0

10.6.3.1.4 Plate Load Tests

The nominal bearing resistance may be determined by plate load tests, provided that adequate subsurface explorations have been made to determine the soil profile below the foundation. Where plate load tests are conducted, they should be conducted in accordance with AASHTO T 235 and ASTM D 1194.

The nominal bearing resistance determined from a plate load test may be extrapolated to adjacent footings where the subsurface profile is confirmed by subsurface exploration to be similar.

COMMENTARY

C10.6.3.1.4

Plate load tests have a limited depth of influence and furthermore may not disclose the potential for long-term consolidation of foundation soils.

Scale effects should be addressed when extrapolating the results to performance of full scale footings. Extrapolation of the plate load test data to a full scale footing should be based on the design procedures provided herein for settlement (service limit state) and bearing resistance (strength and extreme event limit state), with consideration to the effect of the stratification, i.e., layer thicknesses, depths, and properties. Plate load test results should be applied only within a sub-area of the project site for which the subsurface conditions, i.e., stratification, geologic history, and properties, are relatively uniform.

10.6.3.2 BEARING RESISTANCE OF ROCK

10.6.3.2.1 General

The methods used for design of footings on rock shall consider the presence, orientation, and condition of discontinuities, weathering profiles, and other similar profiles as they apply at a particular site.

The designer shall judge the competency of a rock mass by taking into consideration both the nature of the intact rock and the orientation and condition of discontinuities of the overall rock mass. The competency of the rock mass should be verified using the procedures for *RMR* rating in D10.4.6.4.

Rock masses may also pose special problems that shall be considered in design. Typical examples of special problems include weathering, chemical effects, solutioning and subsidence. See Publication 293M, "Geotechnical Engineering Manual", to identify the presence and extent of these special problem conditions throughout the Commonwealth.

C10.6.3.2.1

The design of spread footings bearing on rock is frequently controlled by either overall stability, i.e., the orientation and conditions of discontinuities, or load eccentricity considerations. The designer should verify adequate overall stability at the service limit state and size the footing based on eccentricity requirements at the strength limit state before checking nominal bearing resistance at both the service and strength limit states.

10.6.3.2.2 Semiempirical Procedures

The factored bearing stress of the foundation shall not be taken to be greater than the factored compressive resistance of the footing concrete.

The design of footings on rock shall account for the

C10.6.3.2.2

The bearing resistance of a footing on jointed or broken rock is dependent on the relationship between the joint spacing, the footing geometry and the condition of the joints.

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condition and spacing of joints and other discontinuities. The ultimate bearing resistance of footings on rock can be estimated using the following relationship:

$$q_n = N_{ms}C_o \qquad (10.6.3.2.2\text{-}1)$$

Refer to Table 1 for values of N_{ms}. Values of C_o should be determined from the results of laboratory testing of rock core obtained within 2B of the base for square and circular footings and within 4B of the base of the footing for footing where L/B ≥ 5. Where rock strata within this interval are variable in strength, the rock with the lowest capacity should be used to determine q_n. As a guide, Table 2 can be used to estimate C_o. For rocks defined by very poor quality, the value of q_n should be determined as the value of q_n for an equivalent soil mass.

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The bearing resistance of jointed or broken rock may be estimated using the semi-empirical procedure developed by Carter and Kulhawy (*1988*).

$$q_n = \left[\sqrt{s} + \left(m\sqrt{s} + s\right)^{\frac{1}{2}}\right]C_o \qquad (C10.6.3.2.2\text{-}1)$$

This procedure is based on the unconfined compressive strength of the intact rock core sample.

The terms s and m are constants from Table D10.4.6.4-4. Values of the term in brackets (designated as N_{ms}) as a function of rock type and quality are presented in Table 1, such that q_n can be determined using Equation 1.

Table 10.6.3.2.2-1 - Values of Coefficient Nms for Estimation of the Nominal Bearing Resistance of Footings on Broken or Jointed Rock, Modified after Hoek (1983)

ROCK MASS QUALITY	GENERAL DESCRIPTION	RMR(1) RATING	RQD(2) (%)	N_{ms}(3)				
				A	B	C	D	E
Excellent	Intact rock with joints spaced >3 m {10 ft.} apart	100	95-100	3.8	4.3	5.0	5.2	6.1
Very Good	Tightly interlocking, undisturbed rock with rough unweathered joints spaced 1 to 3 m {3 to 10 ft.} apart	85	90-95	1.4	1.6	1.9	2.0	2.3
Good	Fresh to slightly weathered rock, slightly disturbed with joints spaced 1 to 3 m {3 to 10 ft.} apart	65	75-90	0.28	0.32	0.38	0.40	0.46
Fair	Rock with several sets of moderately weathered joints spaced 300 mm to 1 m {1 to 3 ft.} apart	44	50-75	0.049	0.056	0.066	0.069	0.081
Poor	Rock with numerous weathered joints spaced 30 to 500 mm {1 to 20 in.} apart with some gouge	23	25-50	0.015	0.016	0.019	0.020	0.024
Very Poor	Rock with numerous highly weathered joints spaced< 50 mm {2 in.} apart	3	<25	Use q_{ult} for an equivalent soil mass				

(1)Geomechanics Rock Mass Rating (RMR) System, in accordance with D10.4.6.4

(2)Range of RQD values provided for general guidance only; actual determination of rock mass quality should be based on RMR.

(3)Value of N_{ms} as a function of rock type; refer to Table 2 for typical range of values of C_o for different rock types in each category

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Table 10.6.3.2.2-2 - Typical Range of Uniaxial Compressive Strength (Co) as a Function of Rock Category and Rock Type

ROCK CATEGORY	GENERAL DESCRIPTION	ROCK TYPE	Co[1] MPa	Co[1] ksf
A	Carbonate rocks with well-developed crystal cleavage	Dolostone	30-310	600-6,400
		Limestone	20-290	600-6,000
		Carbonatite	40-70	800-1,400
		Marble	40-240	800-5,000
		Tactite-Skarn	130-330	2,800-6,800
B	Lithifield argillaceous rock	Argillite	30-150	600-3,000
		Claystone	1-8	30-170
		Marlstone	50-190	1,000-4,000
		Phyllite	20-240	600-5,000
		Siltstone	10-120	200-2,400
		Shale[2]	10-35	150-740
		Slate	140-210	3,000-4,400
C	Arenaceous rocks with strong crystals and poor cleavage	Conglomerate	30-220	600-4,600
		Sandstone	70-170	1,400-3,600
		Quartzite	60-380	1,200-8,000
D	Fine-grained igneous crystalline rock	Andesite	100-180	2,000-3,800
		Diabase	20-575	450-12,000
E	Coarse-grained igneous and metamorphic crystalline rock	Amphibolite	120-280	2,400-5,800
		Gabbro	125-310	2,600-6,400
		Gneiss	20-310	500-6,400
		Granite	10-330	300-6,800
		Quartzdiorite	10-100	200-2,000
		Quartzmonzonite	130-160	2,800-3,400
		Schist	10-140	200-3,000
		Syenite	180-430	3,800-9,000

[1]Range of Uniaxial Compressive Strength values reported by various investigations
[2]Not including oil shale

10.6.3.2.3 Analytic Method

The nominal bearing resistance of foundations on rock shall be determined using established rock mechanics principles based on the rock mass strength parameters determined in accordance with D10.4.6.4. The influence of discontinuities on the failure mode shall also be considered.

C10.6.3.2.3

Depending upon the relative spacing of joints and rock layering, bearing capacity failures for foundations on rock may take several forms. Except for the case of a rock mass with closed joints, the failure modes are different from those in soil. Procedures for estimating bearing resistance for each of the failure modes can be found in Kulhawy and Goodman (*1980*), Kulhawy and Goodman (*1987*), Goodman (*1989*),

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The bearing capacity of a footing on jointed or broken rock is dependent on the relationship between the joint spacing and the footing geometry, and the condition of the joints as described below.

10.6.3.2.3a Footings on Closely-Spaced Joint Sets

Closely-spaced joints are discontinuities with a spacing less than the minimum plan dimension of the footing. Footings on closely spaced joint sets may fail either by uniaxial compression or shear, depending on the condition and orientation of the joints.

- Open Joints

Open joint sets are defined as discontinuities across which load cannot be transferred due to the presence of an open space or soft joint filler. For open joints, failure is likely to occur by uniaxial compression of the rock columns. The nominal bearing resistance of the rock mass may be determined for this case by the following relationship:

$$q_n = 2K_e\, c \tan(45 + \phi_{fm}/2) \qquad (10.6.3.2.3a\text{-}1)$$

In determining qn, strength parameters representing rock mass shall be used. For cases in which the parameters are determined from intact samples rather than the rock mass, the value of c, obtained from intact rock, shall be reduced by the factor Ke, using the following relationship from Gardner (1987):

$$K_e = 0.0231(RQD) - 1.32 \geq 0.15 \qquad (10.6.3.2.3a\text{-}2)$$

Values of the rock mass angle of friction (ϕ_{fm}) are typically 50 to 75% of those of the intact material, Kulhawy and Goodman (1980). Values of c and ϕ_{fm} for the intact rock can be determined from the results of triaxial compression tests on rock core samples. For poor quality rock in which intact samples cannot be obtained, use of the pressuremeter test may be considered for determining the in situ rock properties. When in situ testing or triaxial compression testing is not or cannot be performed, the nominal bearing capacity of the rock mass can be estimated by assuming c equal to approximately 5 to 10% of C_o, obtained from the results of uniaxial compressive strength or point load tests, and assuming ϕ_{fm} equal to zero as presented in Kulhawy and Goodman (1980).

- Closed Joints

Closed joint sets are defined as discontinuities across which load can be transferred because of contact between

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and Sowers (*1979*).

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rock surfaces on either side of the joints. For closed joints, failure is likely to occur by general shear failure of the rock across the joint sets. For this case, the nominal bearing capacity shall be determined using Equation D10.6.3.1.2a-1 and the bearing capacity factors as defined in Figure 1. Where applicable, terms in this equation shall be modified by factors described in 10.6.3.1.2e through 10.6.3.1.2h. Bearing capacity factors shall be developed using a value of ϕ_{fm} adjusted to account for rock mass characteristics.

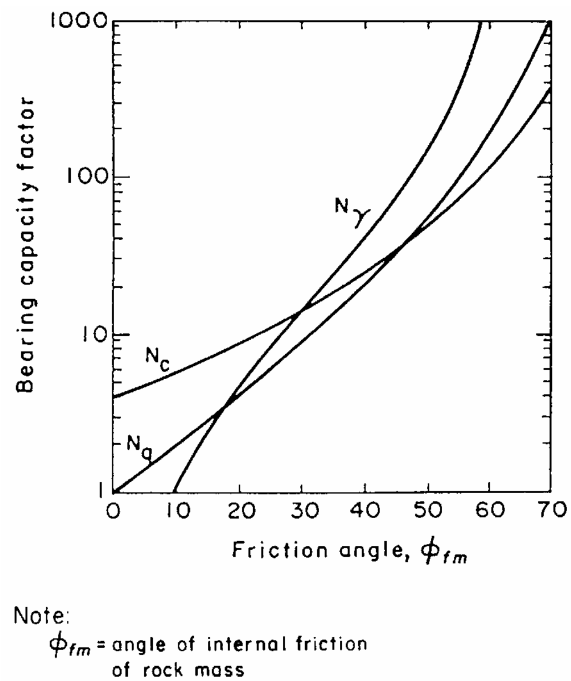

Figure 10.6.3.2.3a-1 - Bearing Capacity Factors for Development of General Wedge Shear Zone for Footings on Rock with Closed Joints

10.6.3.2.3b Footings on Widely-Spaced Joint Sets

Widely-spaced joints are discontinuities with a spacing greater than the minimum plan dimension of the footing. Where wide joint sets are present, failure occurs by splitting of rock below the footing, which ultimately leads to general shear. For square and circular footings, the nominal bearing capacity can be determined by the following relationship, Kulhawy and Goodman (1980):

For circular footings:

$$q_{ult} = J(K_e c)N_{cr} \qquad (10.6.3.2.3b\text{-}1)$$

For square footings:

$$q_{ult} = 0.85J(K_e c)N_{cr} \qquad (10.6.3.2.3b\text{-}2)$$

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See Figure 1 for values of J and N_{cr} to use in the equations. The bearing capacity factor shall be developed using a value of ϕ_{fm} adjusted to account for rock mass characteristics as discussed in 10.6.3.2.3aP. If the shear strength of the rock is not or cannot be determined by in-situ testing or triaxial testing of rock core samples, the bearing capacity can be conservatively estimated by assuming ϕ_{fm} equal to zero and c equal to approximately 5 to 10% of C_o.

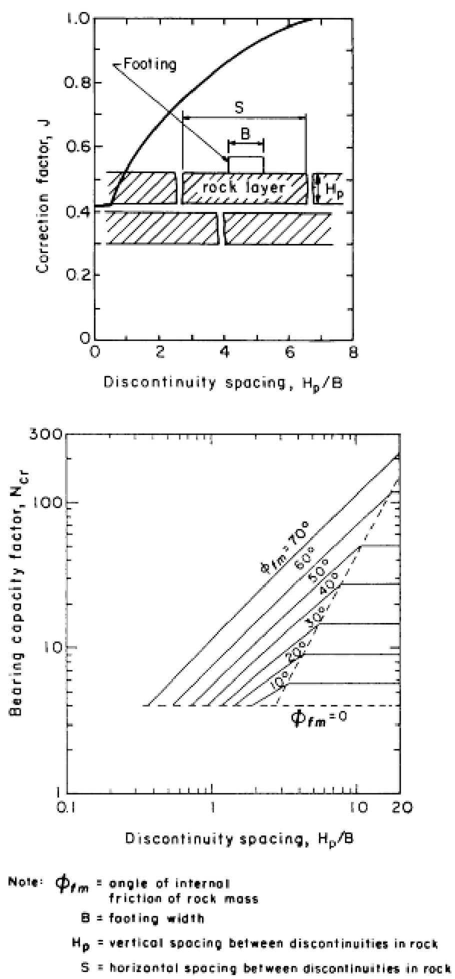

Figure 10.6.3.2.3b-1 - Bearing Capacity Factors for Footings on Rock with Widely-Spaced Joint Sets

10.6.3.2.3c Footings on Rigid Layer Over Weaker Layer

For a thick rigid layer overlying a weaker layer, failure will probably occur by flexure of the upper layer where the flexure strength of unfractured rock is approximately 10 to 20% of q_n. For a thin rigid layer overlying a weaker layer, failure will probably occur by tensile failure of the upper layer where the tensile strength of unfractured rock is approximately 5 to 10% of q_n

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10.6.3.2.4 Load Test

Where appropriate, load tests may be performed to determine the nominal bearing resistance of foundations on rock.

10.6.3.3 ECCENTRIC LOAD LIMITATIONS

The eccentricity of loading at the strength limit state, evaluated based on factored loads shall not exceed:

- One-fourth of the corresponding footing dimension, *B* or *L*, for footings on soils, or
- Three-eighths of the corresponding footing dimensions *B* or *L*, for footings on rock.

10.6.3.4 FAILURE BY SLIDING

Failure by sliding shall be investigated for footings that support horizontal or inclined load and/or are founded on slopes.

Failure by sliding shall be considered by comparing the factored tangential component of force on the footing to the factored shear resistance (R_R).

The factored resistance against failure by sliding, in N, shall be taken as:

$$R_R = \varphi R_n = \varphi_\tau R_\tau \qquad (10.6.3.4\text{-}1)$$

where:

R_n = nominal sliding resistance against failure by sliding (N){kips}

φ_τ = resistance factor for shear resistance between soil and foundation specified in Table D10.5.5.2.2-1

R_τ = nominal sliding resistance between soil and foundation (N){kips}

If the soil beneath the footing is cohesionless or rock, the nominal sliding resistance between soil and foundation shall be taken as:

$$R_\tau = V \tan\delta \qquad (10.6.3.4\text{-}2)$$

for which:

$\tan\delta$ = $\tan \phi_f$ for concrete cast against soil

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C10.6.3.3

A comprehensive parametric study was conducted for cantilevered retaining walls of various heights and soil conditions. The base widths obtained using the LRFD load factors and eccentricity of *B*/4 were comparable to those of ASD with an eccentricity of *B*/6.

C10.6.3.4

Sliding failure occurs if the force effects due to the horizontal component of loads exceed the more critical of either the factored shear resistance of the soils or the factored shear resistance at the interface between the soil and the foundation.

For footings on cohesionless soils, sliding resistance depends on the roughness of the interface between the foundation and the soil.

The magnitudes of active earth load and passive resistance depend on the type of backfill material, the wall movement, and the compactive effort. Their magnitude can be estimated using procedures described in Sections 3 and 11.

The units for R_R, and R_n, are shown in N {kips}. For elements designed on a unit length basis, these quantities will have the units of N {kips} per unit length. Use consistent units in analyses.

Rough footing bases usually occur where footings are cast in-situ. Precast concrete footings may have smooth bases.

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= 0.8 tan ϕ_f for precast concrete footing
= tan δ, using δ from Table A3.11.5.3-1 for concrete on rock
= tan ϕ_{fw} for sliding of one soil over another soil

where:

ϕ_f = internal friction angle of drained soil (deg)

ϕ_{fw} = internal friction angle of weaker soil (deg)

V = total vertical force (N){kips}

The nominal resistance for sliding for foundations on soils exhibiting both frictional and cohesive shear strength components, i.e., "c-ϕ" soils, may be taken as:

$$R_\tau = V \tan\delta + c_a B' \quad (10.6.3.3\text{-}3)$$

where:

V = total vertical force per unit width (N/mm) {kips/ft}

tanδ = tanϕ_f for concrete cast against soil
= 0.8 tan ϕ_f for a precast concrete footing

c_a = adhesion between footing and soil, taken as c(0.21 + 0.0258/c $\leq$ 1.0), unless better data is available (MPa) {ksf}

B' = effective footing width as specified in D10.6.1.3 (mm) {ft.}

Footings shall not bear directly on unsuitable soil or clay. For such cases, unsuitable soil or clay shall be replaced in accordance with D10.6.1.9.

For footings on clay, for which the minimum overexcavation of 150 mm {6 in.} is specified in accordance with D10.6.1.9, the sliding resistance shall be taken as the lesser of:

- the undrained shear strength of the clay, or
- one-half the normal stress on the interface between the footing and soil, as shown in Figure 1.

The following notation shall be taken to apply to Figure 1:

q_s = unit shear resistance, equal to S_u or 0.5 σ'_v, whichever is less

R_τ = nominal sliding resistance between soil and foundation (N) {kips} expressed as the shaded area

COMMENTARY

Engineering judgment should be exercised when making the determination of whether or not to count on the cohesive shear strength of a c-ϕ soil. A c-ϕ analysis should only be performed on soils which exhibit a true effective stress cohesion strength under long-term, drained conditions. The drained, effective stress strength parameters for this condition should be determined using a minimum of three shear strength tests or three undrained tests with pore pressure measurements.

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under the q_s diagram

S_u = undrained shear strength (MPa){ksf}

σ'_v = vertical effective stress (MPa){ksf}

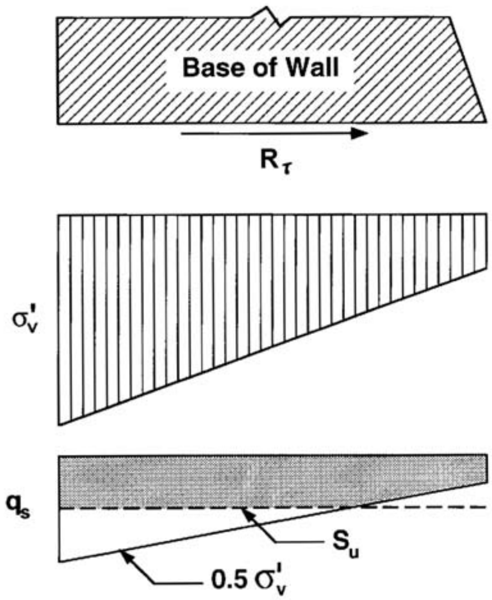

Figure 10.6.3.4-1 Procedure for Estimating Nominal Sliding Resistance for Footings orWalls on Clay.

Passive resistance shall be neglected in evaluation of sliding failure, unless the base of the footing extends below the depth of maximum scour, freeze-thaw or other disturbances. In the latter case only, the embedment below the greater of these depths may be considered effective.

The passive resistance should be neglected if the soil providing passive resistance is soft, loose or disturbed, or if the contact between the soil and footing is not tight.

In cases where the passive pressure is used, the resultant force shall be applied at 0.4 H, where H is the height of the effective soil in front of the footing.

Unacceptable deformations may occur before passive resistance is mobilized.

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10.6.4 Extreme Event Limit State Design

10.6.4.1 GENERAL

Extreme limit state design checks for spread footings shall include, but not necessarily be limited to:

- bearing resistance,
- eccentric load limitations (overturning),
- sliding, and
- overall stability.

Resistance factors shall be as specified in D10.5.5.3.

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10.6.4.2 ECCENTRIC LOAD LIMITATIONS

For footings, whether on soil or on rock, the eccentricity of loading for extreme limit states shall not exceed the limits provided in D11.6.5.

If live loads act to reduce the eccentricity for the Extreme I limit state, γ_{EQ} shall be taken as 0.0.

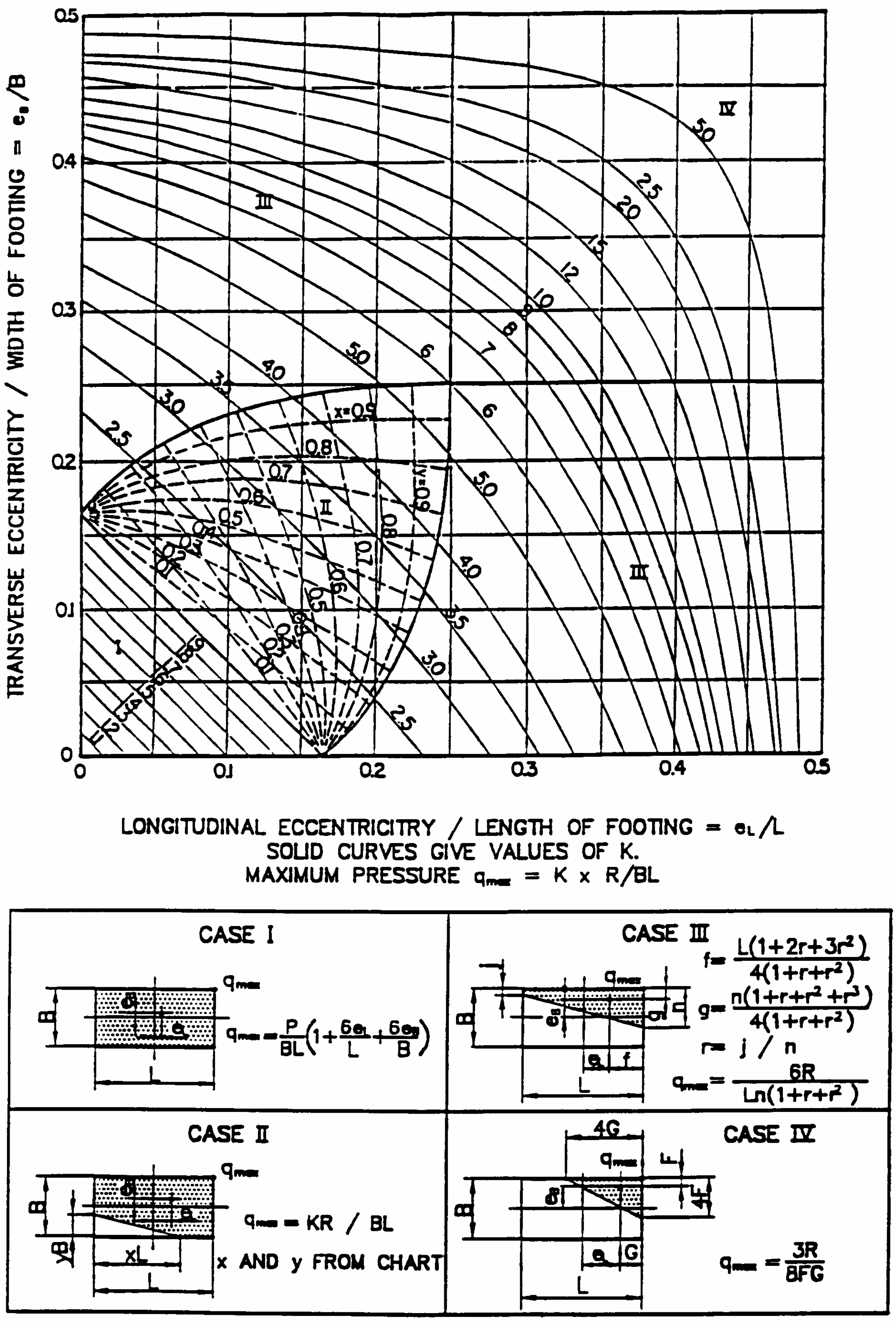

Figure 10.6.4.2-1 - Contact Pressure for Footing Loaded Eccentrically about Two Axes, Modified after AREA (1980)

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10.6.5 Structural Design

The structural design of footings shall comply with the requirements given in Section 5.

For structural design of an eccentrically loaded foundation, a triangular or trapezoidal contact stress distribution based on factored loads shall be used for footings bearing on all soil and rock conditions.

The actual distribution of contact pressure for a rigid footing with eccentric loading about one axis is shown in Figure 1.

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For purposes of structural design, it is usually assumed that the bearing stress varies linearly across the bottom of the footing. This assumption results in the slightly conservative triangular or trapezoidal contact stress distribution. In reality, the contact pressure is nonlinear due to the flexibility of the footing which tends to reduce the value of q_{max} to a value less than the high toe contact pressure associated with a rigid footing.

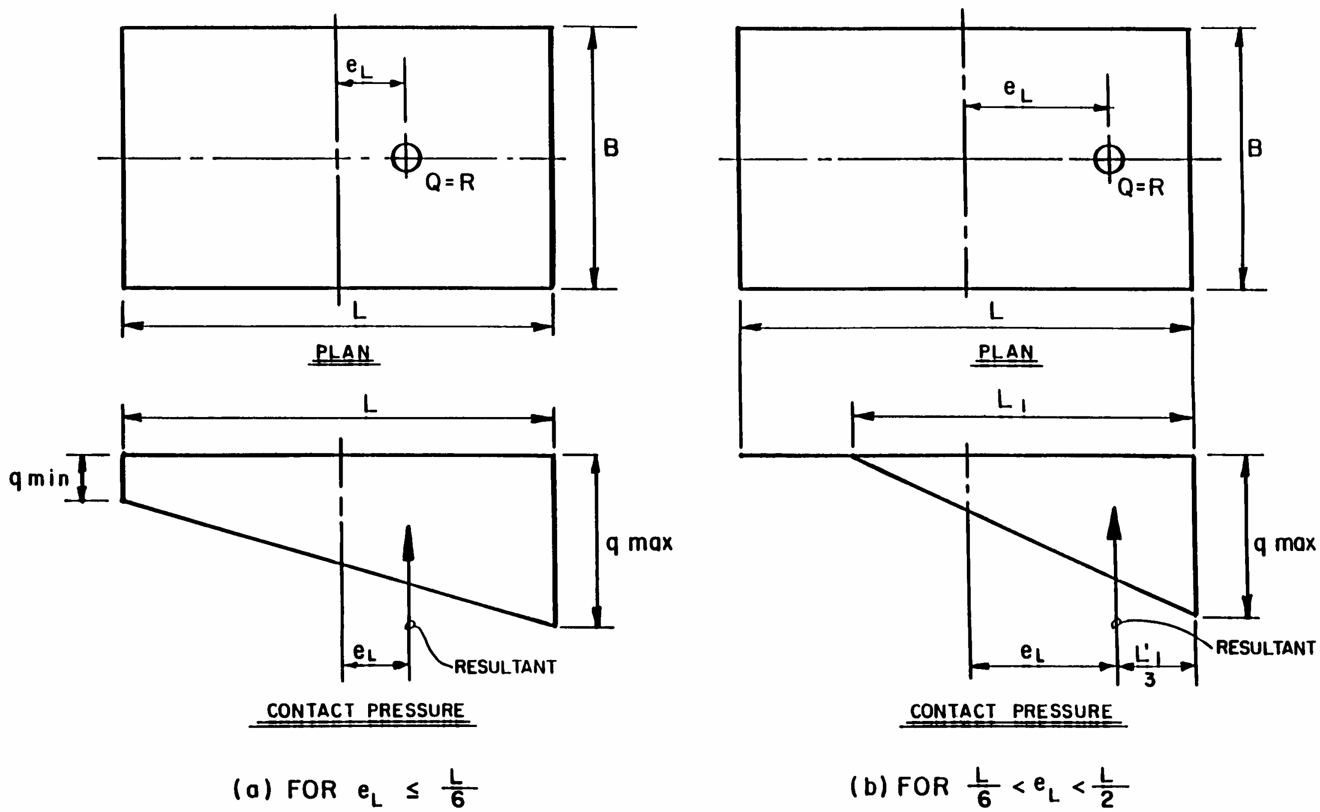

Figure 10.6.5-1 – Contact Pressure for Footing Loaded Eccentrically about One Axis

For an eccentricity (e_L) in the L direction, the actual maximum and minimum contact pressures may be determined as follows:

for e_L < L/6:

$$q_{max} = V\left[1+\left(6e_L/L\right)\right]/BL \quad (10.6.5\text{-}1)$$

$$q_{min} = V\left[1-\left(6e_L/L\right)\right]/BL \quad (10.6.5\text{-}2)$$

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for L/6 < e_L < L/2:

$$q_{max} = 2V \Big/ \left(3B\left[\left(L/2\right) - e_L\right]\right) \quad (10.6.5\text{-}3)$$

$$q_{min} = 0 \quad (10.6.5\text{-}4)$$

$$L_1 = 3\left[\left(L/2\right) - e_L\right] \quad (10.6.5\text{-}5)$$

For an eccentricity (e_B) in the B direction, the maximum and minimum contact pressures may be determined using Equations 1 through 5 by replacing terms labeled L by B, and terms labeled B by L.

The actual distribution of contact pressure for rigid footings with eccentric loading about both axes are shown in Figure D10.6.4.2-1.

10.6.5.1 UNREINFORCED CONCRETE FOOTINGS

Special approval from the Chief Bridge Engineer is needed to permit plain cement concrete footings.

10.6.5.2 FOUNDATION PEDESTALS

The thickness of foundation pedestals shall be a minimum of 600 mm {24 in.}. Sufficient longitudinal reinforcement shall be provided at the top and bottom faces of the footing slab for continuous beam action between pedestals. Additional dowels may be required for integral action between the stem and footing. If required to ensure integral action between footings and pedestals, transverse shear keys and dowels (No. 19 x 1200 mm at approximately 450 mm c/c each face) {No. 6 x 4 ft. at approximately 18 in. c/c each face} shall be provided. See Figure 1 for other design details.

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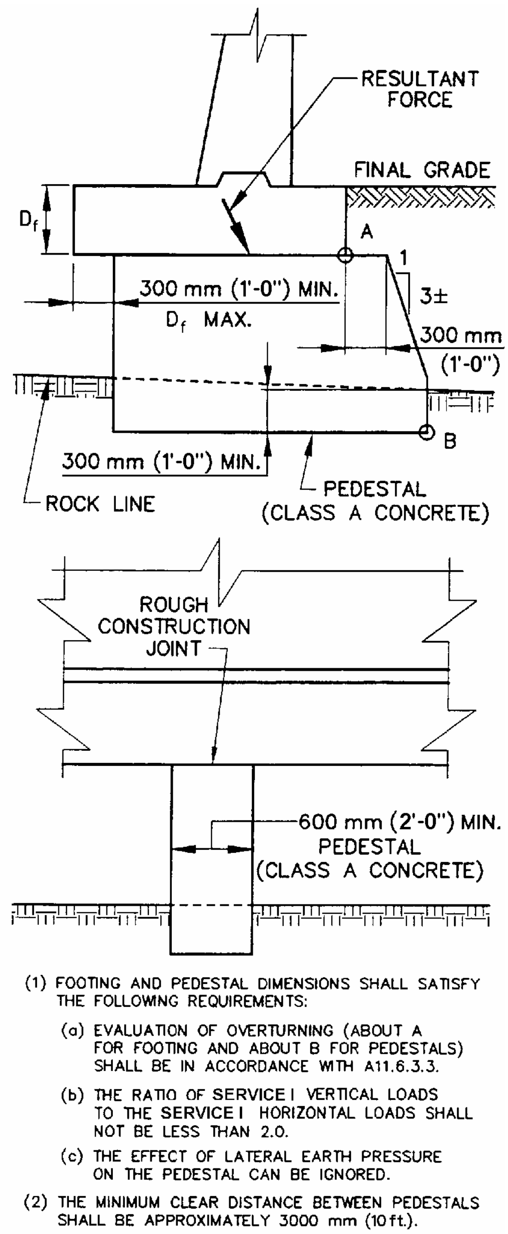

Figure 10.6.5.2-1 - Foundation Pedestal

10.6.6 Foundation Submission

As part of the foundation approval process, a foundation submission letter shall be submitted to the Department for use in foundation review in compliance with PP1.9.4 and D11.4.4P. The letter shall include the basis for foundation capacity determination and the relevance of field conditions and construction procedures to develop the foundation capacity.

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10.7 DRIVEN PILES

This article presents design specifications for driven piles. Refer to Appendix B for design specifications for micropiles.

10.7.1 General

10.7.1.1 APPLICATION

Piling should be considered when spread footings cannot be founded on rock, or on competent soils at a reasonable cost. At locations where soil conditions would normally permit the use of spread footings but the potential exists for scour, liquefaction or lateral spreading, piles bearing on suitable materials below susceptible soils should be considered for use as a protection against these problems. Piles must be sufficiently embedded below theoretical scour depth (refer to PP7.2 to determine the estimated scour depth) to properly support the structure. Where unsuitable material is less than 3000 mm {10 ft.} in depth, consideration shall be given to the economics of spread footings supported on compacted structure backfill material as an alternate to piles. Piles should also be considered where right-of-way or other space limitations would not allow the use spread footings, or where removal of existing soil that is contaminated by hazardous materials for construction of shallow foundations is not desirable.

Piles should also be considered where an unacceptable amount of settlement of spread footings may occur. Preloading of compressible soil by embankment surcharge may be used to reduce settlement to permit use of spread footings, or to reduce negative friction loading on piles. The cost of preloading compared to pile foundations shall be evaluated and submitted with the Foundation Design Report.

10.7.1.2 MINIMUM PILE SPACING, CLEARANCE, AND EMBEDMENT INTO CAP

Footings shall be proportioned so that pile spacing is a minimum of 900 mm {3 ft.} or two and one-half times the diameter for round or octagonal piles or in integral abutments and two times the diagonal dimension for square piles (including H-piles) not in integral abutments. Maximum pile spacing shall be 4500 mm {15 ft.}. The distance from the side of any pile to the nearest edge of the footing shall be a minimum of 225 mm {9 in.}, and not less than 450 mm {1'-6"} from the centerline of pile.

Piles shall be embedded at least 300 mm {1 ft.} into footings, or 450 mm {1'-6"} into stub abutments supported by a single row of piles. For details and reinforcement bar locations see Figure 1. Minimum footing thickness shall be 750 mm {2'-6"}.

Reinforcement bar and pile locations shall be as follows:

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- Middle and/or rear piles aligned with the front row piles for ease of construction

- Reinforcement bars placed evenly between piles and hooked for the reinforcement provided transversely in between piles

- Cutting of flanges or rebars to accommodate reinforcement shall not be allowed (except that holes may be burned through piles within the embedment zone to accommodate steel reinforcement required for seismic loading)

- For main reinforcement bars near piles in the footing, a clearance of 25 mm {1 in.} is recommended. Reinforcement bar spacing near piles should be adjusted accordingly.

If piles are out of position more than 150 mm {6 in.} preventing proper placement of reinforcement bars, the Contractor will have the option of either placing the reinforcement bars on top of the piles and providing additional thickness at no additional cost to the Department, or providing bottom reinforcement bars in such a way (splayed or spliced) to provide adequate structural strength, if approved by the District Bridge Engineer.

Contractors may be permitted to redesign pile footings to provide the bottom reinforcement bars above piles at no additional cost to the Department, provided the following requirements are met:

- Meet the same strength and serviceability requirements appropriate for the original design

- Minimum 900 mm {3 ft.} thick footing

- Minimum earth cover above the top of the footing, as shown on the original plan

- Maintain the original design bottom of footing elevation in a scour environment

- Meet seismic requirements in accordance with A11 of LRFD Appendix A

For Integral abutments, refer to Appendix G and BD-667M.

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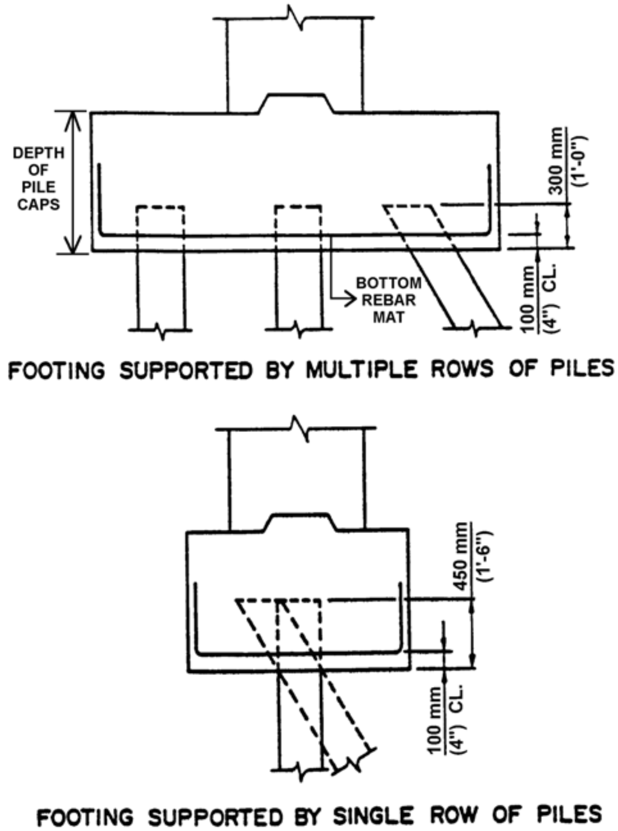

Figure 10.7.1.2-1 - Minimum Projection of Piles into Cap and Clearance to Bottom Reinforcement Bars

10.7.1.3 PILES THROUGH EMBANKMENT FILL

Piles to be driven through embankments should penetrate a minimum of 3000 mm {10 ft} through original ground unless refusal on bedrock or competent bearing strata occurs at a lesser penetration. The minimum penetration length of piles shall be 3000 mm {10 ft.} unless otherwise approved by the Chief Bridge Engineer for a specific project.

Fill used for embankment construction should be a select material, which does not obstruct pile penetration to the required depth. For piles through embankment fill, the plans shall specify that select fill be placed and compacted to the footing elevation before pile driving. The stiffest cost-effective pile with tip reinforcement shall be used to facilitate embankment penetration without auguring or predrilling. Auguring to the original groundline will be permitted, if necessary. Where point-bearing or end bearing piles extend through fill and an underlying compressible soil deposit, the effects of negative friction (downdrag) on the pile shall be considered (see D10.7.3.7).

C10.7.1.3

If refusal occurs at a depth of less than 3000 mm {10 ft}, other foundation types, e.g., footings or shafts, may be more effective.

To minimize the potential for obstruction of the piles, the maximum size of any rock particles in the fill should not exceed 150 mm {6 in.}. Pre-drilling pile locations should be considered in situations where obstructions in the embankment fill cannot be avoided, particularly for displacement piles. Note that predrilling may reduce the pile skin friction and lateral resistance, depending on how the predrilling is conducted. The diameter of the predrilled hole, and the potential for caving of the hole before the pile is installed will need to be considered to assess the effect this will have on skin friction and lateral resistance. See D10.7.1.6.5 for further discussion of predrilling.

For integral abutments 3000 mm {10 ft.} minimum pile penetration must neglect predrilled portion.

If compressible soils are located beneath the embankment, piles should be driven after embankment settlement is complete, if possible, to minimize or eliminate downdrag forces.

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10.7.1.4 BATTER PILES

When the lateral resistance of the soil surrounding the piles is inadequate to counteract the horizontal forces transmitted to the foundation, or when increased rigidity of the entire structure is required, batter piles should be considered for use in the foundation. If batter piles are used in areas of significant seismic loading, the design of the pile foundation shall recognize the increased foundation stiffness that results.

Where ground settlements of greater than 6 mm {1/4 in.} are estimated, batter piles should be avoided or installed after the settlement is substantially complete. Otherwise, the effects of pile bending due to downdrag forces shall be evaluated during design.

Batter piles may be used in pile groups to reduce bending in vertical piles and thereby maximize their axial load capacity. Batter piles are recommended for abutments and retaining walls. The bearing resistance of a pile group containing batter piles may be determined by treating the batter piles as vertical piles.

Layout of batter piles shall be arranged so that batter piles will not interfere with each other below the pile cap, or with piles below adjacent pile caps.

In general, a pile batter of 1 (horizontal) on 4 (vertical) is considered desirable, but in cases where sufficient horizontal resistance is not otherwise attainable, a batter of 1 on 3 may be specified. In no case shall the batter exceed 1 on 3. The Engineer should consider the slope of top of bedrock when setting pile batter.

10.7.1.5 PILE DESIGN REQUIREMENTS

Pile design shall address the following issues as appropriate:

- Nominal axial resistance to be specified in the contract, type of pile, and size of pile group required to provide adequate support, with consideration of how nominal axial pile resistance will be determined in the field.

- Group interaction.

- Pile quantity estimation from estimated pile penetration required to meet nominal axial resistance and other design requirements.

- Minimum pile penetration necessary to satisfy the requirements caused by uplift, scour, downdrag, settlement, liquefaction, lateral loads, and seismic conditions.

- Foundation deflection to meet the established

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C10.7.1.4

From a general viewpoint, batter piles provide a much stiffer resistance to horizontal loads than would be possible with vertical piles. They can be very effective in resisting static horizontal loads.

Batter piles attract lateral load in groups which include vertical piles due to their greater resistance to lateral deformation. Therefore, where a combination of batter and vertical piles is subjected to seismic loading, special consideration should be given to the distribution of load among piles and the design of the pile-cap connection.

Due to increased foundation stiffness, batter piles may not be desirable in resisting horizontal dynamic loads if the structure is located in an area where seismic loads are potentially high.

Settlement induces bending moments in the shafts of batter piles (Tomlinson, 1987). No documented procedure is available to analyze the behavior of batter piles subjected to downdrag forces. A rational method of analysis for bending of batter piles should evaluate the piles as beams on an elastic foundation and that the piles assume deflected shape of the soil mass. The analysis should consider the effects of pile type and geometry, the length of pile within the compressible soil zone, the initial vertical and lateral loading at the pile cap, the pile fixity, and the lateral subgrade reaction between the soil and pile.

It is the Department preference that no more than 50 percent of the back row of piles be battered if it is economical.

C10.7.1.5

The driven pile design process is discussed in detail in Hannigan et al. (*2005*).

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movement and associated structure performance criteria.

- Pile foundation nominal structural resistance.
- Verification of pile drivability to confirm that acceptable driving stresses and blow counts can be achieved with an available driving system to meet all contract acceptance criteria.
- Long-term durability of the pile in service, i.e. corrosion and deterioration.

The load supporting resistance shall be determined using current Department practice according to applicable general notes in PP1.7.5 and Publication 408, Section 1005. The pile type, load resistance and driving method shall be as approved by the Chief Bridge Engineer. Refer to D10.7.3 for methods to determine pile resistance.

10.7.1.6 DETERMINATION OF PILE LOADS

10.7.1.6.1 General

The loads and load factors to be used in pile foundation design shall be as specified in Section 3. Computational assumptions that shall be used in determining individual pile loads are described in Section 4.

10.7.1.6.2 Downdrag

The provisions of A3.11.8 shall apply for determination of load due to negative skin resistance.

Where piles are driven to end bearing on a dense stratum or rock and the design of the pile is structurally controlled, downdrag shall be considered at the strength and extreme limit states.

For friction piles that can experience settlement at the pile tip, downdrag shall be considered at the service, strength and extreme limit states. Determine pile and pile group settlement according to D10.7.2.

The nominal pile resistance available to support structure loads plus downdrag shall be estimated by considering only the positive skin and tip resistance below the lowest layer acting in negative skin resistance,

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C10.7.1.6.1

The specification and determination of top of cap loads is discussed in Section 3. The Engineer should select different levels of analysis, detail and accuracy as appropriate for the structure under consideration. Details are discussed in Section 4.

C10.7.1.6.2

Downdrag occurs when settlement of soils along the side of the piles results in downward movement of the soil relative to the pile. See AC3.11.8.

In the case of friction piles with limited tip resistance, the downdrag load can exceed the geotechnical resistance of the pile, causing the pile to move downward enough to allow service limit state criteria for the structure to be exceeded. Where pile settlement is not limited by pile bearing below the downdrag zone, service limit state tolerances will govern the geotechnical design of piles subjected to downdrag.

This design situation is not desirable and the preferred practice is to mitigate the downdrag induced foundation settlement through a properly designed surcharge and/or preloading program, or by extending the piles deeper for higher resistance.

The static analysis procedures in D10.7.3.8.6 may be used to estimate the available pile resistance to withstand the downdrag plus structure loads.

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computed as specified in A3.11.8.

The effects of negative skin friction can be ignored for point bearing piles if effective measures are taken to reduce effects of negative skin friction. If downdrag is considered, use the minimum value for γ_p as specified in Table A3.4.1-2.

Negative skin friction loads that cause excessive settlement may be reduced by measures such as:

- application of bitumen or other viscous coating to the pile surfaces before installation,
- inducing or permitting consolidation settlement to occur prior to pile installation, or
- isolating pile from surrounding settling soil by installing the pile through casing.

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Although the use of bituminous material coating is permitted, environmental concerns and practical application (bitumen loss during driving) often prevent this option from being feasible.

10.7.1.6.3 Uplift

Piles driven in swelling soils may be subjected to uplift forces in the zone of seasonal moisture change. Uplift loads can be reduced by application of bitumen or other viscous coatings to the pile surface in the swelling zone.

Tension in piles is not permitted at Service Limit states.

At Strength Limit States, the uplift capacity may be taken as 10 percent of the axial structural capacity.

Piles penetrating expansive soil shall extend to a depth into moisture-stable soils sufficient to provide adequate anchorage to resist uplift. Sufficient clearance should be provided between the ground surface and underside of caps or beams connecting piles to preclude the application of uplift loads at the pile/cap connection due to swelling ground conditions.

C10.7.1.6.3

Evaluation of potential uplift loads on piles extending through expansive soils requires evaluation of the swell potential of the soil and the extent of the soil strata that may affect the pile. One reasonably reliable method for identifying swell potential is presented in Table D10.4.6.3-1. Alternatively, ASTM D 4829 may be used to evaluate swell potential. The thickness of the potentially expansive stratum must be identified by:

- examination of soil samples from borings for the presence of jointing, slickensiding, or a blocky structure and for changes in color, and
- laboratory testing for determination of soil moisture content profiles.

Based on Department experience, uplift on piles due to swelling soils is generally not a problem in Pennsylvania.

10.7.1.6.4 Nearby Structures

Where pile foundations are placed adjacent to existing structures, the influence of the existing structure on the behavior of the foundation, and the effect of the new foundation on the existing structures, including vibration effects due to pile installation, shall be investigated.

C10.7.1.6.4

Vibration due to pile driving can cause settlement of existing foundations as well as structural damage to the adjacent facility. The combination of taking measures to mitigate the vibration levels through use of nondisplacement piles, predrilling, etc., and a good vibration monitoring program should be considered.

A pre-construction survey of nearby structures to document conditions prior to driving should also be considered.

10.7.1.6.5 Pre-Drilling for Driven Piles

Pre-drilling for driven piles shall be categorized as Mandatory Pre-drilling or Pre-drilling for Unforeseen

C10.7.1.6.5

Pile driving is a very economical construction method. For limited situations, it will be necessary to pre-drill piles

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Obstructions.

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to reach the predetermined tip elevations. However, many geotechnical reports and construction contracts specify pre-drilling of piles because of anticipated or perceived difficulties in driven piles obtaining predetermined tip elevations. In the past these construction contracts have included substantial predrilling quantities as a contingency. With the utilization of the special provisions in D10.7.1.6.5a and D10.7.1.6.5b in construction contracts, the Department should realize an improvement in bid consistency and resolution of construction issues associated with predrilling.

10.7.1.6.5a Mandatory Pre-drilling

Mandatory Pre-drilling shall be specified in situations that require drilling to obtain the necessary 3m (10-foot) pile length, or to obtain a required pile tip elevation by penetrating through upper inadequate rock layers, or vibration mitigation or other situations that require mandatory pre-drilling to obtain a required tip elevation. Pre-drilled piles shall be indicated on the pile layout plans. The contract documents shall include standard special provision ID 10051A. This special provision is a bid item that includes all operations necessary to perform the work including mobilization, access to the foundations, drilling, maintaining an open hole, casing and backfilling with aggregate.

This special provision shall not be used as a contingency item in the contract.

The special provision uses a granular material to backfill the hole. For situations that require any special backfill material this material shall be specified in the contract documents.

Special backfill material may include slurry to be used in limestone formations.

The special provision indicates the steel casing to be removed, therefore it is temporary casing. If permanent casing is required, the construction plans shall indicate the required wall thickness, the length of the casing and include a bid item Meter (linear foot) for the permanent casing.

Permanent casing may be used to eliminate downdrag loads on piles.

10.7.1.6.5b Pre-drilling for Unforeseen Obstructions

Pre-drilling for Unforeseen Obstructions shall be specified in situations where it is uncertain if piles can be driven to the predetermined tip elevation due to unforeseen obstructions. Potential obstructions include but are not limited to reinforced concrete, timber, boulders, rock pinnacles or existing piles. The contract documents shall include standard special provisions:

- ID 10052A, Predrilling for Unforeseen Obstructions, Pile Extraction and Re-Driving.
- ID 10053A, Mobilization for Pre-drilling.

C10.7.1.6.5b

Various scenarios may exist in the field regarding the payment under this special provision. The following four scenarios provide guidance relative to the use of this special provision:

1.1 A production pile (with tip reinforcement) is driven but hangs up above tip elevation. [Paid in accordance with Publication 408, Section 1005.]

1.2 It is extracted. [Paid for by the item for Pile Extraction and Redrive.]

1.3 The hole is drilled to the predetermined pile tip elevation. [Soil drilling is to be paid at $328/M ($100/lf) and obstruction drilling is to be paid at

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Items for the mobilization and pile extraction and redrive are bid as Predetermined Amounts (PDA). Items for Predrilling for Unforeseen Obstructions include earth drilling and obstruction drilling which shall be bid as predetermined minimum contract unit price. The proposal shall indicate $328/M ($100/LF) for earth drilling and $1640/M ($500/LF) for obstruction drilling. The estimated quantity for earth drilling shall be 10% of the total length of piles at each substructure unit and for obstruction drilling 1% of the total length of piles at each substructure unit.

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$1640/M ($500/lf) per the special provision.]

1.4 Original pile (accepted by engineer) is placed in the hole, backfilled, and driven to refusal at tip elevation. [Driven pile is paid for by the item for Pile Extraction and Redrive.]

2.1 A production pile is driven but hangs up above tip elevation. [Paid in accordance with Publication 408, Section 1005.]
2.2 It is extracted and discarded. [Paid for by the item for Pile Extraction and Redrive. The material cost of the discarded pile is in addition to the pile extraction and redrive operations.]
2.3 The hole is drilled to the predetermined pile tip elevation. [Soil drilling is to be paid at $328/M ($100/lf) and obstruction drilling is to be paid at $1640/M ($500/lf) per the special provision.]
2.4 New pile is placed in the hole, backfilled, and driven to refusal at tip elevation. [Driven pile is paid for by the item for Pile Extraction and Redrive.]

3.1 A production pile is driven but hangs up above tip elevation. [Paid in accordance with Publication 408, Section 1005.]
3.2 It is extracted. [Paid for by the item for Pile Extraction and Redrive.]
3.3 The hole is drilled to an elevation above the predetermined pile tip elevation. [Soil drilling is to be paid at $328/M ($100/lf) and obstruction drilling is to be paid at $1640/M ($500/lf) per the special provision.]
3.4 Original pile is placed in the hole, backfilled, but does not reach drill hole elevation. It is extracted and discarded. [No payment is made for the extraction and discarded pile.]
3.5 The hole is redrilled to the original drill hole elevation. [No payment is made for redrilling.]
3.6 A new pile is placed in the hole, backfilled and driven to refusal at tip elevation. [Redriven pile is paid for by the item for Pile Extraction and Redrive to the drill hole elevation. Driven pile is paid in accordance with Publication 408, Section 1005 for the pile length from bottom of drill hole to pile tip elevation.]

4.1 A production pile is driven but hangs up above the tip elevation. [Paid in accordance with Publication 408, Section 1005.]
4.2 It is extracted. [Paid for by the item for Pile Extraction and Redrive.]
4.3 The hole is drilled to the predetermined pile tip elevation. [Soil drilling is to be paid at $328/M ($100/lf) and obstruction drilling is to be paid at $1640/M ($500/lf) per the special provision.]
4.4 The hole is filled with aggregate (less than 20’) and then the pile is set up and redriven but does not reach predetermined tip elevation. [No payment is made for

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the redrive since the contractor elected to backfill the hole prior to redriving the pile.]

4.5 The pile is extracted. [No payment is made for the extraction since the contractor elected to backfill the hole prior to redriving the pile.]

4.6 The hole is redrilled to the predetermined pile tip elevation. [No payment is made for the redrilling since the contractor elected to backfill the hole prior to redriving the pile.]

4.7 The pile is placed in the hole, then backfilled, and driven to tip elevation. [Driven pile is paid for by the item for Pile Extraction and Redrive.]

A pile placed in a predrilled hole is paid for by the item for Pile Extraction and Redrive to the predrilled hole elevation. A pile driven below the predrilled hole elevation is paid in accordance with Publication 408, Section 1005.

10.7.2 Service Limit State Design

10.7.2.1 GENERAL

Service limit state design of driven pile foundations includes the evaluation of settlement due to static loads, and downdrag loads if present, overall stability, lateral squeeze, and lateral deformation. Overall stability of a pile supported foundation shall be evaluated where:

- The foundation is placed through an embankment,
- The pile foundation is located on, near or within a slope,
- The possibility of loss of foundation support through erosion or scour exists, or
- Bearing strata are significantly inclined.

Unbalanced lateral forces caused by lack of overall stability or lateral squeeze should be mitigated through stabilization measures, if possible.

C10.7.2.1

Lateral analysis of pile foundations is conducted to establish the load distribution between the superstructure and foundations for all limit states, and to estimate the deformation in the foundation that will occur due to those loads. This article only addresses the evaluation of the lateral deformation of the foundation resulting from the distributed loads.

In general, it is not desirable to subject the pile foundation to unbalanced lateral loading caused by lack of overall stability or caused by lateral squeeze.

10.7.2.2 TOLERABLE MOVEMENTS

The provisions of D10.5.2.2 shall apply.

Design horizontal movements shall not exceed 13 mm {1/2 in.} at the Service Limit State or 25 mm {1 in.} at any Strength or Extreme Limit State (refer to D10.7.2.4).

C10.7.2.2

See DC10.5.2.2.

10.7.2.3 SETTLEMENT

10.7.2.3.1 Equivalent Footing Analogy

For purposes of calculating the settlements of pile groups, loads should be assumed to act on an equivalent

C10.7.2.3.1

Pile design should ensure that strength limit state considerations are satisfied before checking service limit

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footing based on the depth of embedment of the piles into the layer that provides support as shown in Figures 1 and 2.

Pile group settlement shall be evaluated for pile foundations in cohesive soils, soils that include cohesive layers, and piles in loose granular soils. The load used in calculating settlement shall be the permanently applied load on the foundation.

In applying the equivalent footing analogy for pile foundation, the reduction to equivalent dimensions *B′* and *L′* as used for spread footing design does not apply.

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state considerations.

For piles tipped adequately into dense granular soils such that the equivalent footing is located on or within the dense granular soil, and furthermore are not subjected to downdrag loads, a detailed assessment of the pile group settlement may be waived.

Methods for calculating settlement are discussed in Hannigan et al., (*2005*).

Settlement predictions for a pile group using an equivalent footing approximation at a depth of 2/3 Db can be very conservative, and do not explicitly account for the effects of length to diameter ratio, L/d, relative pile spacing, s/d, and the number of piles in the group. Elastic solutions have been derived for this problem (Poulos and Davis, 1980; Poulos, 1988) which account for all of these variables. The soil modulus used in these solutions should be chosen, such that it is representative of the general range of the expected load deformation behavior, and may be estimated using applicable correlations with in-situ test results.

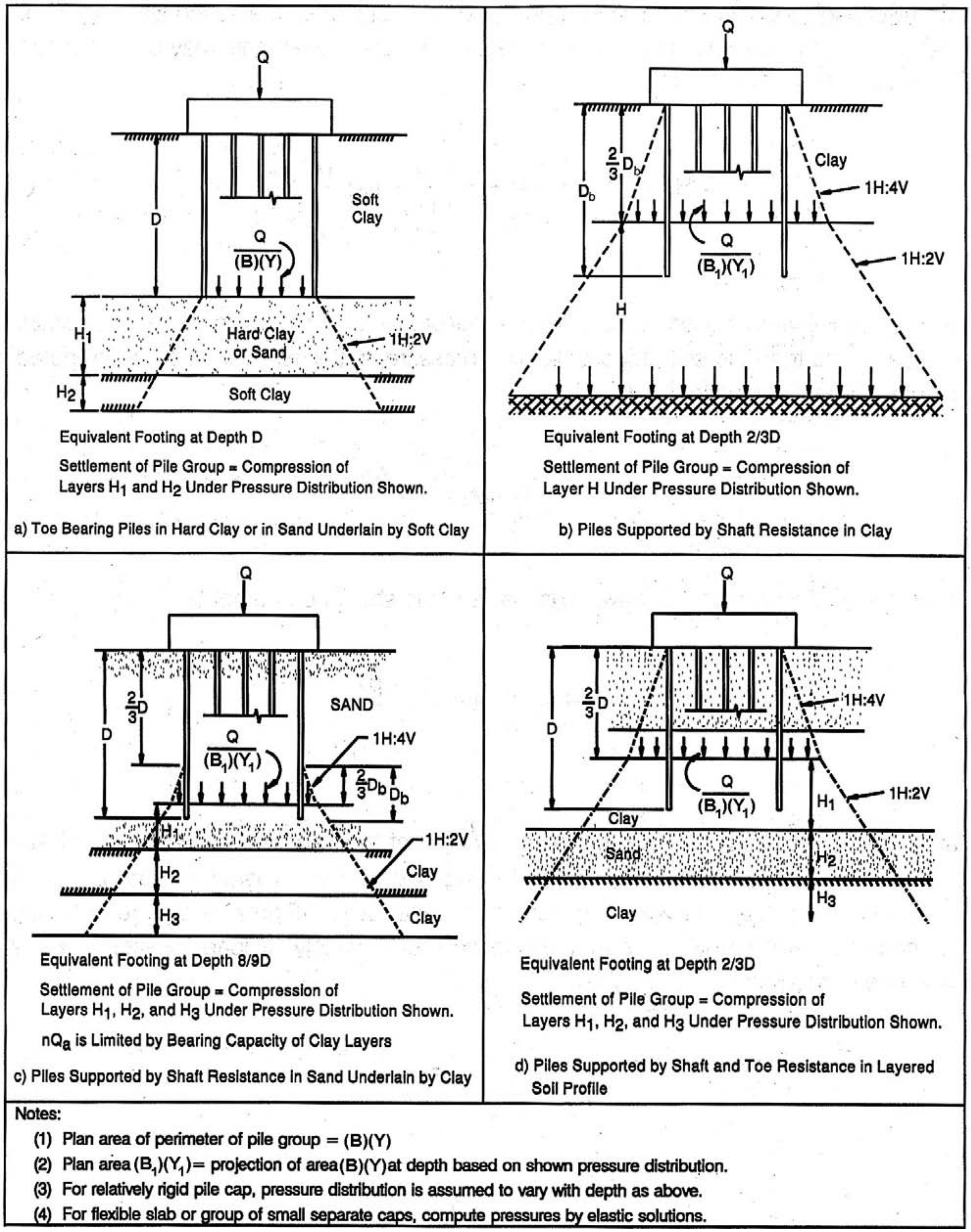

Figure 10.7.2.3.1-1 Stress Distribution Below Equivalent Footing for Pile Group after Hannigan et al. (*2005*).

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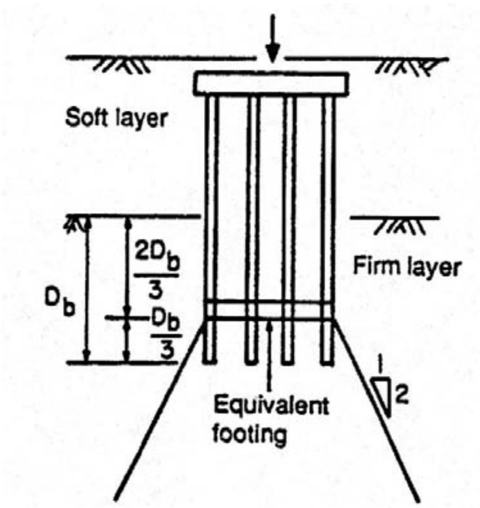

Figure 10.7.2.3.1-2 Location of Equivalent Footing (after Duncan and Buchignani, 1976).

10.7.2.3.2 Pile Groups in Cohesionless Soil

The settlement of pile groups in cohesionless soils may be taken as:

Using *SPT*:

Metric Units: $\rho = \frac{30qI\sqrt{B}}{N1_{60}}$ (10.7.2.3.2-1)

U.S. Customary Units: $\rho = \frac{qI\sqrt{B}}{N1_{60}}$

Using *CPT*:

Metric Units: $\rho = \frac{qBI}{24q_c}$ (10.7.2.3.2-2)

U.S. Customary Units: $\rho = \frac{qBI}{2q_c}$

in which:

$$I = 1 - 0.125\frac{D'}{B} \geq 0.5 \quad (10.7.2.3.2\text{-}3)$$

where:

ρ = settlement of pile group (mm){in.}

q = net foundation pressure applied at $2D_b/3$, as shown in Figure D10.7.2.3.1-1; this pressure is equal to the applied load at the top of the group divided by the area of the equivalent footing and does not include the weight of the piles or the soil between the piles (MPa){ksf}

B = width or smallest dimension of pile group

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C10.7.2.3.2

The provisions are based upon the use of empirical correlations proposed by Meyerhof (*1976*). These are empirical correlations and the units of measure must match those specified for correct computations. This method may tend to over-predict settlements.

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(mm){ft.}

I = influence factor of the effective group embedment (dim.)

D' = effective depth taken as $2D_b/3$ (mm){ft.}

D_b = depth of embedment of piles in layer that provides support, as specified in Figure D10.7.2.3.1-1 or Figure D10.7.2.3.1-2 (mm){ft.}

$N1_{60}$ = *SPT* blow count corrected for both overburden and hammer efficiency effects (blows/300 mm){blows/ft.} as specified in D10.4.6.2.4.

q_c = static cone tip resistance (MPa){ksf}

Alternatively, other methods for computing settlement in cohesionless soil, such as the Hough method as specified in D10.6.2.4.2 may also be used in connection with the equivalent footing approach.

The corrected *SPT* blow count or the static cone tip resistance should be averaged over a depth equal to the pile group width B below the equivalent footing. The *SPT* and *CPT* methods (Eqs. 1 and 2) shall only be considered applicable to the distributions shown in Figure D10.7.2.3.1-1b and Figure D10.7.2.3.1-2.

10.7.2.3.3 Pile Groups in Cohesive Soil

Procedures used for shallow foundations shall be used to estimate the settlement of a pile group using the equivalent footing location specified in Figure D10.7.2.3.1-1 or Figure D10.7.2.3.1-2.

10.7.2.4 HORIZONTAL PILE FOUNDATION MOVEMENT

Horizontal movement induced by lateral loads shall be evaluated. The provisions of D10.5.2.2 and D10.7.2.2 shall apply regarding horizontal movement criteria.

The horizontal movement of pile foundations shall be estimated using procedures that consider soil-structure interaction. The provisions of D10.7.3.12.2 and D10.7.3.13.3 shall apply.

The orientation of nonsymmetrical pile cross-sections shall be considered when computing the pile lateral stiffness.

Lateral resistance of single piles may be determined by static load test. If a static lateral load test is to be performed, it shall follow the procedures specified in ASTM D 3966.

COMMENTARY

C10.7.2.4

Pile foundations are subjected to horizontal loads due to wind, traffic loads, bridge curvature, vessel or traffic impact and earthquake.

If a static load test is used to assess the site specific lateral resistance of a pile, information on the methods of analysis and interpretation of lateral load tests presented in the *Handbook on Design of Piles and Drilled Shafts Under Lateral Load* (*Reese, 1984*) and *Static Testing of Deep Foundations* (*Kyfor et al., 1992*) should be used.

The lateral displacement of pile groups may also be estimated using soil-structure methods of analysis such as Reese, et. al., (1987) and Poulos and Davis (1980). The FHWA computer program FB-Pier may also be used.

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10.7.2.5 SETTLEMENT DUE TO DOWNDRAG

The nominal pile resistance available to support structure loads plus downdrag shall be estimated by considering only the positive skin and tip resistance below the lowest layer contributing to the downdrag. In general, the available factored geotechnical resistance should be greater than the factored loads applied to the pile, including the downdrag, at the service limit state. In the instance where it is not possible to obtain adequate geotechnical resistance below the lowest layer contributing to downdrag, e.g., friction piles, to fully resist the downdrag, the structure should be designed to tolerate the full amount of settlement resulting from the downdrag and the other applied loads.

If adequate geotechnical resistance is available to resist the downdrag plus structure loads in the service limit state, the amount of deformation needed to fully mobilize the geotechnical resistance should be estimated, and the structure designed to tolerate the anticipated movement.

COMMENTARY

C10.7.2.5

The static analysis procedures in D10.7.3.8.6 may be used to estimate the available pile resistance to withstand the downdrag plus structure loads.

Resistance may also be estimated using a dynamic method, e.g., dynamic measurements with signal matching analysis, pile driving formula, etc., per D10.7.3.8, provided the skin friction resistance within the zone contributing to downdrag is subtracted from the resistance determined from the dynamic method during pile installation. The skin friction resistance within the zone contributing to downdrag may be estimated using the static analysis methods specified in D10.7.3.8.6, from signal matching analysis, or from pile load test results. Note that the static analysis methods may have bias that, on average, over or under predicts the skin friction. The bias of the method selected to estimate the skin friction within the downdrag zone should be taken into account as described in D10.7.3.3.

For the establishment of settlement tolerance limits, see D10.5.2.1.

10.7.2.6 LATERAL SQUEEZE

Bridge abutments supported on pile foundations driven through soft soils that are subject to unbalanced embankment fill loading shall be evaluated for lateral squeeze.

C10.7.2.6

Guidance on evaluating the potential for lateral squeeze and potential mitigation methods are included in Hannigan et al., (*2005*).

10.7.3 Strength Limit State Design

10.7.3.1 GENERAL

For strength limit state design, the following shall be determined:

- Loads and performance requirements;
- Pile type, dimensions, and nominal axial pile resistance in compression;
- Size and configuration of the pile group to provide adequate foundation support;
- Estimated pile length to be used in the construction contract documents to provide a basis for bidding;
- A minimum pile penetration, if required, for the particular site conditions and loading, determined based on the maximum (deepest) depth needed to meet all of the applicable requirements identified in D10.7.6;
- The maximum driving resistance expected in order to reach the minimum pile penetration required, if applicable, including any soil/pile skin friction that will

C10.7.3.1

A minimum pile penetration should only be specified if needed to ensure that uplift, lateral stability, depth to resist downdrag, depth to resist scour, and depth for structural lateral resistance are met for the strength limit state, in addition to similar requirements for the service and extreme event limit states. See D10.7.6 for additional details. Assuming dynamic methods, e.g., wave equation calibrated to dynamic measurements with signal matching analysis,

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not contribute to the long-term nominal axial resistance of the pile, e.g., soil contributing to downdrag, or soil that will be scoured away;

- The drivability of the selected pile to achieve the required nominal axial resistance or minimum penetration with acceptable driving stresses at a satisfactory blow count per unit length of penetration; and
- The nominal structural resistance of the pile and/or pile group.

COMMENTARY

pile formulae, etc., are used during pile installation to establish when the bearing resistance has been met, a minimum pile penetration should not be used to ensure that the required nominal pile bearing, i.e., compression, resistance is obtained.

A driving resistance exceeding the nominal bearing, i.e., compression, resistance required by the contract may be needed in order to reach a minimum penetration elevation specified in the contract.

The drivability analysis is performed to establish whether a hammer and driving system will likely install the pile in a satisfactory manner.

10.7.3.2 POINT BEARING PILES ON ROCK

10.7.3.2.1 General

As applied to pile compressive resistance, this article shall be considered applicable to soft rock, hard rock, and very strong soils such as very dense glacial tills that will provide high nominal axial resistance in compression with little penetration.

C10.7.3.2.1

Certain geologic conditions within the state pose driveability, long-term stability and pile length predictability problems. Such geologic conditions are typically associated with the following characteristics:

a. Greatly varying depth to bedrock over small areas

b. Greatly varying quality of bedrock over small areas

c. Presence of voids, soil-filled seams and/or other discontinuities within bedrock

If pile penetration into rock is expected to be minimal, the prediction of the required pile length will usually be based on the depth to rock.

A definition of hard rock that relates to measurable rock characteristics has not been widely accepted. Local or regional experience with driving piles to rock provides the most reliable definition.

In general, it is not practical to drive piles into rock to obtain significant uplift or lateral resistance. If significant lateral or uplift foundation resistance is required, drilled shaft foundations should be considered. If it is still desired to use piles, a pile drivability study should be performed to verify the feasibility of obtaining the desired penetration into rock.

10.7.3.2.2 Piles Driven to Soft Rock

If absolute refusal cannot be achieved for a pile bearing in weak rock, the unit bearing resistance shall be estimated by treating the soft rock as soil in accordance with D10.7.3.8.

If the pile can be drive to Case 1 or Case 2 absolute refusal (see Publication 408, Section 1005), the nominal axial resistance shall be estimated in accordance with D10.7.3.2.3.

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10.7.3.2.3 Piles Driven to Hard Rock

If absolute refusal can be achieved for a pile bearing in rock, the nominal resistance is controlled by the structural limit state. The nominal axial resistance shall not exceed the values obtained from A6.9.4.1 and D6.9.4.1 with the resistance factors specified in D6.5.4.2 and D6.15 for severe driving conditions. A pile-driving acceptance criteria shall be developed that will prevent pile damage. Pile dynamic measurements should be used to monitor for pile damage when nominal axial resistances exceed 2.5×10^6 N {600 kips}.

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C10.7.3.2.3

Care should be exercised in driving piles to hard rock to avoid tip damage.

10.7.3.2.4 Piles Bearing on Soluble Bedrock

Where variations in the elevation, quality and discontinuities are present to an appreciable degree in limestone, or other soluble bedrock, the following measures shall be considered:

a. Use of steel HP 250 × 85 {HP 10 x 57} piles designed with axial resistance factor for piles bearing on soluble bedrock as per D6.5.4.2. The piles shall be driven to Case 1 Absolute Refusal (see Publication 408, Section 1005).

b. Tip protection

c. Allowance for variations in pile location of a minimum of 150 mm {6 in.}

d. Allowance for deviation from plumbness greater than normally specified

e. Design of pile caps to withstand loss of up to 35% of piles in a pile group, considered as a localized loss

f. Use of combined footings where feasible

g. Relative feasibility and cost of using drilled shaft or micropile foundations in lieu of driven piles

C10.7.3.2.4

Although some limestone formations do not exhibit irregularities and discontinuities to such a degree that pile foundations are adversely affected, comprehensive subsurface exploration, as described in DC10.4.2 is required to evaluate pertinent conditions.

10.7.3.2.5 Piles Bearing on Sloping Bedrock

When piles are driven to end or point bearing on sloping bedrock, special pile tips and driving procedures are appropriate to minimize deflection of piles along the bedrock surface.

C10.7.3.2.5

The severity of installation problems increases with increasing hardness of the bedrock surface and decreasing strength of laterally supporting soils above bedrock. These conditions often occur in karst areas. A special driving procedure has been used successfully to promote secure seating.

Piles which are anticipated to bear on sloping bedrock and to experience driving difficulties are to be clearly identified on the bridge plans. A note is to be included on the plans indicating that the specified piles are to be driven

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in accordance with the procedure delineated below. A special provision is to be included to present the special driving procedure.

The driving procedure for piles seated on sloping bedrock is as follows: Stop driving the pile when the pile tip is at or slightly above bedrock. Continue driving with the stroke of the hammer reduced to 150 mm {6 inches} or to the minimum practical value. For Air or Steam hammers reduce the stroke by reducing the pressure. For Diesel hammers, reduce the stroke by shutting off the fuel and operating the hammer as a drop hammer. When the penetration for 10 to 20 blows is zero, double the stroke and continue driving until the pile is properly seated, or until the maximum energy is obtained from air or steam hammers, or until the stroke becomes limited by the trip device for diesel hammers. Drive the pile to absolute refusal, following the specified driving procedure, unless otherwise directed by the Engineer.

The Engineer may order additional piles to be driven if driving records suggest that any of the piles are not properly seated. The number of piles which are to be properly seated on bedrock must conform to the number shown on the drawings. Payment for the seating of test and bearing piles on sloping bedrock will be in accordance with Sections 1005.4(a) and 1005.4(b) of Publication 408 regardless of any additional piles ordered driven so that the specified number of seated piles matches the drawings. Piles driven in conformance with the above requirements, but determined by the Engineer to be inadequately seated, will be considered acceptable for payment.

10.7.3.3 PILE LENGTH ESTIMATES FOR CONTRACT DOCUMENTS

Subsurface geotechnical information combined with static analysis methods (D10.7.3.8.6), preconstruction test pile programs (D10.7.9), and/or pile load tests (D10.7.3.8.2) shall be used to estimate the depth of penetration required to achieve the desired nominal bearing for establishment of contract pile quantities. Local experience shall also be considered when making pile quantity estimates, both to select an estimation method and to assess the potential prediction bias for the method used to account for any tendency to over-predict or under-predict pile compressive resistance. If the depth of penetration required to obtain the desired nominal bearing, i.e., compressive, resistance is less than the depth required to meet the provisions of D10.7.6, the minimum penetration required per D10.7.6 should be used as the basis for estimating contract pile quantities.

C10.7.3.3

The estimated pile length required to support the required nominal resistance is determined using a static analysis; knowledge of the site subsurface conditions, and/or results from a pile load test. The pile length used to estimate quantities for the contract should also consider requirements to satisfy other design considerations, including service and extreme event limit states, as well as minimum pile penetration requirements for lateral stability, uplift, downdrag, scour, group settlement, etc.

One solution to the problem of predicting pile length is the use of a preliminary test program at the site. Such a program can range from a very simple operation of driving a few piles to evaluate drivability, to an extensive program where different pile types are driven and static and dynamic testing is performed.

In lieu of local experience, if a static analysis method is used to estimate the pile length required to achieve the desired nominal bearing for establishment of contract pile quantities, the factored resistance used to determine the size of the pile group required should be equated to the factored resistance estimated using the static analysis method as

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follows:

$$\varphi_{dyn} \text{ x } R_n = \varphi_{stat} \text{ x } R_{nstat} \qquad \text{(C10.7.3.3-1)}$$

where:

φ_{dyn} = the resistance factor for the dynamic method used to verify pile bearing resistance during driving specified in Table D10.5.5.2.3-1

R_n = the nominal pile bearing resistance (N){kips}

φ_{stat} = the resistance factor for the static analysis method used to estimate the pile penetration depth required to achieve the desired bearing resistance specified in Table D10.5.5.2.3-1

R_{nstat} = the predicted nominal resistance from the static analysis method used to estimate the penetration depth required (N){kips}

Using Eq. C1 and solving for R_{nstat}, use the static analysis method to determine the penetration depth required to obtain R_{nstat}.

The resistance factor for the static analysis method inherently accounts for the bias and uncertainty in the static analysis method. However, local experience may dictate that the penetration depth estimated using this approach be adjusted to reflect that experience.

Note that R_n is considered to be nominal bearing resistance of the pile needed to resist the applied loads, and is used as the basis for determining the resistance to be achieved during pile driving, R_{ndr} (see D10.7.6 and D10.7.7). R_{nstat} is only used in the static analysis method to estimate the pile penetration depth required.

10.7.3.4 NOMINAL AXIAL RESISTANCE CHANGE AFTER PILE DRIVING

10.7.3.4.1 General

Consideration should be given to the potential for change in the nominal axial pile resistance after the end of pile driving. The effect of soil relaxation or setup should be considered in the determination of nominal axial pile resistance for soils that are likely to be subject to these phenomena.

C10.7.3.4.1

Relaxation is not a common phenomenon but more serious than setup since it represents a reduction in the reliability of the foundation.

Pile setup is a common phenomenon that can provide the opportunity for using larger pile nominal resistances at no increase in cost. However, it is necessary that the resistance gain be adequately proven. This is usually accomplished by restrike testing with dynamic measurements (*Komurka, et. al, 2003*).

10.7.3.4.2 Relaxation

If relaxation is possible in the soils at the site the pile

C10.7.3.4.2

Relaxation is a reduction in axial pile resistance. While

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shall be tested in re-strike after a sufficient time has elapsed for relaxation to develop.

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relaxation typically occurs at the pile tip, it can also occur along the sides of the pile (*Morgano and White, 2004*). It can occur in dense sands or sandy silts and in some shales. Relaxation in the sands and silts will usually develop fairly quickly after the end of driving, perhaps in only a few minutes, as a result of the return of the reduced pore pressure induced by dilation of the dense sands during driving. In some shales, relaxation occurs during the driving of adjacent piles and that will be immediate. There are other shales where the pile penetrates the shale and relaxation requires perhaps as much as two weeks to develop. In some cases, the amount of relaxation can be large.

10.7.3.4.3 Setup

Setup in the nominal axial resistance may be used to support the applied load. Where increase in resistance due to setup is utilized, the existence of setup shall be verified after a specified length of time by re-striking the pile.

C10.7.3.4.3

Setup is an increase in the nominal axial resistance that develops over time predominantly along the pile shaft. Pore pressures increase during pile driving due to a reduction of the soil volume, reducing the effective stress and the shear strength. Setup may occur rapidly in cohesionless soils and more slowly in finer grained soils as excess pore water pressures dissipate. In some clays, setup may continue to develop over a period of weeks and even months, and in large pile groups it can develop even more slowly.

Setup, sometimes called “pile freeze,” can be used to carry applied load, providing the opportunity for using larger pile nominal axial resistances, if it can be proven. Signal matching analysis of dynamic pile measurements made at the end of driving and later in re-strike can be an effective tool in evaluating and quantifying setup. (*Komurka et al., 2003*; *Bogard and Matlock, 1990*).

10.7.3.5 GROUNDWATER EFFECTS AND BUOYANCY

Nominal axial resistance shall be determined using the groundwater level consistent with that used to calculate the effective stress along the pile sides and tip. The effect of hydrostatic pressure shall be considered in the design.

C10.7.3.5

Unless the pile is bearing on rock, the tip resistance is primarily dependent on the effective surcharge that is directly influenced by the groundwater level. For drained loading conditions, the vertical effective stress is related to the ground water level and thus it affects pile axial resistance. Lateral resistance may also be affected.

Buoyant forces may also act on a hollow pile or unfilled casing if it is sealed so that water does not enter the pile. During pile installation, this may affect the driving resistance observed, especially in very soft soils.

10.7.3.6 SCOUR

The effect of scour shall be considered in selecting the pile penetration. The pile foundation shall be designed so that the pile penetration after the design scour event satisfies the required nominal axial and lateral resistances.

C10.7.3.6

The resistance factors will be those used in the design without scour. The axial resistance of the material lost due to scour should be determined using a static analysis and it should not be factored, but consideration should be given to the bias of the static analysis method used to predict resistance. Method bias is discussed in D10.7.3.3.

The piles will need to be driven to the required nominal

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axial resistance plus the side resistance that will be lost due to scour. The resistance of the remaining soil is determined through field verification. The pile is driven to the required nominal axial resistance plus the magnitude of the skin friction lost as a result of scour, considering the prediction method bias.

Another approach that may be used takes advantage of dynamic measurements. In this case, the static analysis method is used to determine an estimated length. During the driving of test piles, the skin friction component of the axial resistance of pile in the scourable material may be determined by a signal matching analysis of the dynamic measurements obtained when the pile is tipped below the scour elevation. The material below the scour elevation must provide the required nominal resistance after scour occurs.

The pile foundation shall be designed to resist debris loads occurring during the flood event in addition to the loads applied from the structure.

Pile cap depths and scour protection must follow the provisions of PP7.2 when scour is a possibility. Scour investigations must be made in accordance with PP7.2.

In some cases, the flooding stream will carry debris that will induce horizontal loads on the piles.

Additional information regarding pile design for scour is provided in Hannigan et al. (*2005*).

10.7.3.7 DOWNDRAG

The foundation should be designed so that the available factored geotechnical resistance is greater than the factored loads applied to the pile, including the downdrag, at the strength limit state. The nominal pile resistance available to support structure loads plus downdrag shall be estimated by considering only the positive skin and tip resistance below the lowest layer contributing to the downdrag. The pile foundation shall be designed to structurally resist the downdrag plus structure loads.

In the instance where it is not possible to obtain adequate geotechnical resistance below the lowest layer contributing to downdrag, e.g., friction piles, to fully resist the downdrag, or if it is anticipated that significant deformation will be required to mobilize the geotechnical resistance needed to resist the factored loads including the downdrag load, the structure should be designed to tolerate the settlement resulting from the downdrag and the other applied loads as specified in D10.7.2.5.

C10.7.3.7

The static analysis procedures in D10.7.3.8.6 may be used to estimate the available pile resistance to withstand the downdrag plus structure loads.

Resistance may also be estimated using a dynamic method per D10.7.3.8, provided the skin friction resistance within the zone contributing to downdrag is subtracted from the resistance determined from the dynamic method during pile installation. The skin friction resistance within the zone contributing to downdrag may be estimated using the static analysis methods specified in D10.7.3.8.6, from signal matching analysis, or from pile load test results. Note that the static analysis method may have a bias, on average over or under predicting the skin friction. The bias of the method selected to estimate the skin friction should be taken into account as described in DC10.7.3.3.

Pile design for downdrag is illustrated in Figure C1.

where:

R_{Sdd} = skin friction which must be overcome during driving through downdrag zone (N){kips}

$Q_p = \Sigma\gamma_i Q_i$ = factored load per pile, excluding downdrag load (N){kips}

DD = downdrag load per pile (N){kips}

$D_{est.}$ = estimated pile length needed to obtain desired nominal resistance per pile (mm){ft.}

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φ_{dyn} = resistance factor, assuming that a dynamic method is used to estimate pile resistance during installation of the pile (if a static analysis method is used instead, use φ_{stat})

γ_p = load factor for downdrag

The summation of the factored loads ($\Sigma\gamma_i Q_i$) should be less than or equal to the factored resistance ($\varphi_{dyn} R_n$). Therefore, the nominal resistance R_n should be greater than or equal to the sum of the factored loads divided by the resistance factor φ_{dyn}. The nominal bearing resistance (N){kips} of the pile needed to resist the factored loads, including downdrag, is therefore taken as:

$$R_n = \frac{\left(\Sigma\gamma_i Q_i\right)}{\varphi_{dyn}} + \frac{\gamma_p DD}{\varphi_{dyn}} \qquad \text{(C10.7.3.7-1)}$$

The total nominal driving resistance, R_{ndr} (N){kips}, needed to obtain R_n, accounting for the skin friction that must be overcome during pile driving that does not contribute to the design resistance of the pile, is taken as:

$$R_{ndr} = R_{Sdd} + R_n \qquad \text{(C10.7.3.7-2)}$$

where:

R_{ndr} = nominal pile driving resistance required (N){kips}

Note that R_{Sdd} remains unfactored in this analysis to determine R_{ndr}.

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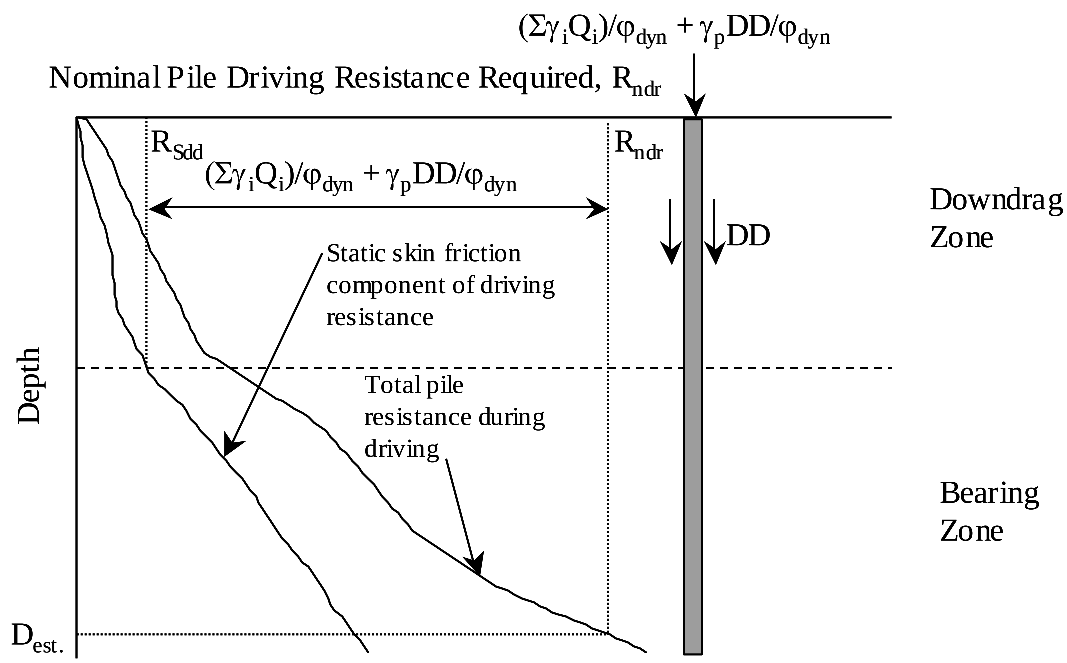

Figure C10.7.3.7-1 Design of Pile Foundations for Downdrag.

10.7.3.8 DETERMINATION OF NOMINAL AXIAL PILE RESISTANCE IN COMPRESSION

10.7.3.8.1 General

For piles bearing on rock, determine the nominal resistance in accordance with D10.7.3.2.

In general, the Department prefers the use of the semi-empirical methods (α-method, β-Method, λ-Method, Nordlund/Thurman Method) in D10.7.3.8.6 to estimate the axial resistance of piles in soil.

The nominal axial resistance of a pile shall be determined in accordance with the following procedure.

1. Performance and evaluation of a subsurface exploration.

2. Static analysis of pile capacity using the procedures in D10.7.3.2.3 and D10.7.3.8.6, and delivery of the Foundation Submission to the Department for review, in accordance with PP1.9.4.
3. Review of the Foundation Submission by the Chief Bridge Engineer. The Foundation Approval shall stipulate requirements for pile type, estimated length, bearing resistance and stratum, test pile requirements and driving criteria as per PP1.7.5.

4. Based on the proposed pile hammer system, the Department shall perform a wave equation analysis and provide a Pile Hammer Approval.

C10.7.3.8.1

In the case of steel H-piles, the structural capacity of the pile will usually control the design for piles bearing on rock.

The bearing resistance of a pile in soil is derived from the tip resistance and/or shaft resistance, i.e., skin friction. Both the tip and shaft resistances develop in response to foundation displacement. The maximum values of each are unlikely to occur at the same displacement. The shaft resistance is typically fully mobilized at displacements of about 2.5 to 10 mm {0.1 to 0.4 in.}. The tip capacity, however, is mobilized after the pile settles about 8 percent of its diameter (*Kulhawy et al. 1983*).

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5. Based on the monitored performance of test piles driven to absolute refusal in accordance with PP1.7.5 and Publication 408, Section 1005, and other testing (i.e., pile load test and/or dynamic monitoring) and analyses (i.e., wave equation analysis), the Department may revise the Foundation Approval and/or the Pile Hammer Approval.

Pile nominal axial resistance should be field verified during pile installation using load tests, dynamic tests**,** or wave equation. The resistance factor selected for design shall be based on the method used to verify pile axial resistance as specified in D10.5.5.2.3. The production piles shall be driven to the minimum blow count determined from the static load test, dynamic test, or wave equation, used unless a deeper penetration is required due to uplift, scour, lateral resistance, or other requirements as specified in D10.7.6. If it is determined that dynamic methods are unsuitable for field verification of nominal axial resistance, and a static analysis method is used without verification of axial resistance during pile driving by static load test, or dynamic test, the piles shall be driven to the tip elevation determined from the static analysis, and to meet other limit states as required in D10.7.6.

10.7.3.8.2 Static Load Test

If a static pile load test is used to determine the pile axial resistance, the test shall not be performed less than 5 days after the test pile was driven unless approved by the Engineer. The load test shall follow the procedures specified in ASTM D 1143, and the loading procedure should follow the Quick Load Test Method, unless detailed longer-term load-settlement data is needed, in which case the standard loading procedure should be used. Unless specified otherwise by the Engineer, the pile axial resistance shall be determined from the test data as:

- for piles 600 mm {24 in.} or less in diameter (length of side for square piles), the Davisson Method;
- for piles larger than 900 mm {36 in.} in diameter (length of side for square piles), at a pile top movement, s_f (mm){in.}, as determined from Eq. 1; and
- for piles greater than 600 mm {24 in.} but less than 900 mm in diameter, a criteria to determine the pile axial resistance that is linearly interpolated between the criteria determined at diameters of 600 and 900 mm {24 and 36 in}.

$$s_f = \frac{QL}{12\,AE} + \frac{B}{2.5} \qquad (10.7.3.8.2\text{-}1)$$

where:

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This article addresses the determination of the nominal bearing (compression) resistance needed to meet strength limit state requirements, using factored loads and factored resistance values. From this design step, the number of piles and pile resistance needed to resist the factored loads applied to the foundation are determined. Both the loads and resistance values are factored as specified in A3.4.1/D3.4.1 and D10.5.5.2.3, respectively, for this determination.

C10.7.3.8.2

The Quick Test Procedure is desirable because it avoids problems that frequently arise when performing a static test that cannot be started and completed within an eight-hour period. Tests that extend over a longer period are difficult to perform due to the limited number of experienced personnel that are usually available. The Quick Test has proven to be easily performed in the field and the results usually are satisfactory. However, if the formation in which the pile is installed may be subject to significant creep settlement, alternative procedures provided in ASTM D1143 should be considered.

The Davisson Method of axial resistance evaluation is performed by constructing a line on the load test curve that is parallel to the elastic compression line of the pile. The elastic compression line is calculated by assuming equal compressive forces are applied to the pile ends. The elastic compression line is offset by a specified amount of displacement. The Davisson Method is illustrated in Figure C1 and described in more detail in Hannigan et al. (*2005*).

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Q = test load (N){kips}

L = pile length (mm){ft.}

A = pile cross-sectional area (mm^2) {ft.^{2}}

E = pile modulus (MPa){ksi}

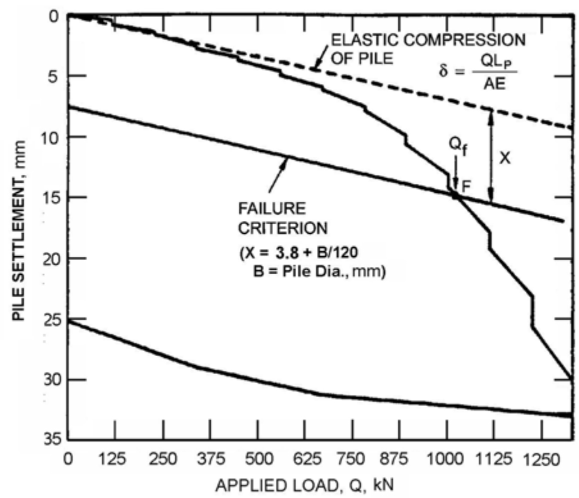
B = pile diameter (length of side for square piles) (mm){ft.}

Driving criteria should be established from the pile load test results using one of the following approaches:

1. Use dynamic measurements with signal matching analysis calibrated to match the pile load test results; a dynamic test shall be performed on the static test pile at the end of driving and again as soon as possible after completion of the static load test by re-strike testing. The signal matching analysis of the re-strike dynamic test should then be used to produce a calibrated signal matching analysis that matches the static load test result. Perform additional production pile dynamic tests with calibrated signal matching analysis (see Table D10.5.5.2.3-3 for the number of tests required) to develop the final driving criteria.
2. If dynamic test results are not available use the pile load test results to calibrate a wave equation analysis, matching the wave equation prediction to the measured pile load test resistance, in consideration of the hammer used to install the load test pile.
3. For the case where the bearing stratum is well defined, relatively uniform in extent, and consistent in its strength, driving criteria may be developed directly from the pile load test result(s), and should include a minimum driving resistance combined with a minimum hammer delivered energy to obtain the required bearing resistance. In this case, the hammer used to drive the pile(s) that are load tested shall be used to drive the production piles.

COMMENTARY

Metric Units:



U.S. Customary Units:

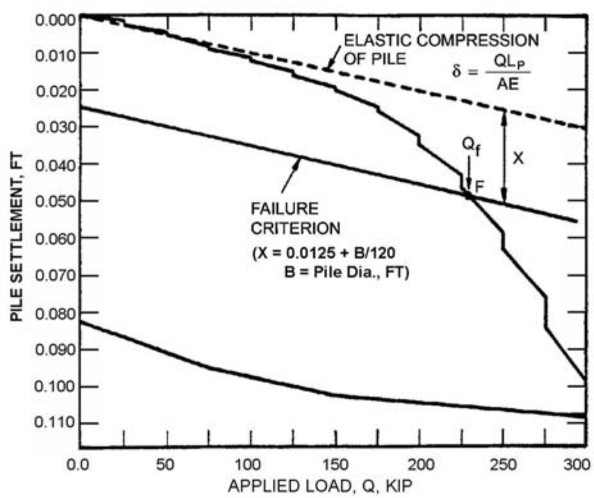


Figure C10.7.3.8.2-1 Alternate Method Load Test Interpretation (*Cheney and Chassie, 2000*, modified after *Davisson, 1972*)

For piles with large cross-sections, i.e., greater than 600 mm {24 in.}, the Davisson Method will under predict the pile nominal axial resistance.

The specific application of the four driving criteria development approaches provided herein may be site specific, and may also depend on the degree of scatter in the pile load test and dynamic test results. If multiple load tests and dynamic tests with signal matching are conducted at a given site as defined in D10.5.5.2.3, the Engineer will need to decide how to "average" the results to establish the final driving criteria for the site, and if local experience is

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4. For the case where driving to a specified tip elevation without field verification using dynamic methods is acceptable and dynamic methods are determined to be unsuitable for field verification of nominal axial resistance as specified in D10.5.5.2.3, the load test results may be used to calibrate a static pile resistance analysis method as specified in D10.7.3.8.6. The calibrated static analysis method should then be used to determine the depth of penetration into the bearing zone needed to obtain the desired nominal pile resistance. In this case, the bearing zone shall be well defined based on subsurface test hole or probe data.

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available, in consideration of that local experience. Furthermore, if one or more of the pile load tests yield significantly higher or lower nominal resistance values than the other load tests at a given project site, the reason for the differences should be thoroughly investigated before simply averaging the results together or treating the result(s) as anomalous.

Regarding the first driving criteria development approach, the combination of the pile load and dynamic test results should be used to calibrate a wave equation analysis to apply the test results to production piles not subjected to dynamic testing, unless all piles are dynamically tested. For piles not dynamically tested, hammer performance should still be assessed to ensure proper application of the driving criteria. Hammer performance assessment should include stroke measurement for hammers that have a variable stroke, bounce chamber pressure measurement for double acting hammers, or ram velocity measurement for hammers that have a fixed stroke. Hammer performance assessment should also be conducted for the second and third driving criteria development approaches.

Regarding the fourth driving criteria development approach, it is very important to have the bearing zone well defined at each specific location within the site where piles are to be driven. Note that a specific resistance factor for this approach to using load test data to establish the driving criteria is not provided. While some improvement in the reliability of the static analysis method calibrated for the site in this manner is likely, no statistical data are currently available from which to fully assess reliability and establish a resistance factor. Therefore, the resistance factor for the static analysis method used should be used for the pile foundation design.

Note that it may not be possible to calibrate the dynamic measurements with signal matching analysis to the pile load test results if the driving resistance at the time the dynamic measurement is taken is too high, i.e., the pile set per hammer blow is too small. In this case, adequate hammer energy is not reaching the pile tip to assess end bearing and produce an accurate match, though in such cases, the prediction will usually be quite conservative. In general, a tip movement (pile set) of 2.4 to 4 mm { 0.10 to 0.15 in} is needed to provide an accurate signal matching analysis.

In cases where a significant amount of soil setup occurs, a more accurate result may be obtained by combining the end bearing determined using the signal matching analysis obtained for the end of driving (*EOD*) with the signal matching analysis for the side friction at the beginning of redrive (*BOR*).

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10.7.3.8.3 Dynamic Testing

10.7.3.8.3a General

For high-capacity major structures, erratic bearing strata, or unusual foundation conditions, dynamic testing shall be conducted in accordance with Publication 408, Section 1005, where economically feasible for any or all of the following:

a. To provide estimates of nominal axial pile resistance under actual field conditions

b. To provide estimates of pile freeze and pile relaxation through redrive testing of previously monitored piles, and/or comparison with static-load test results

c. To provide, in conjunction with CAPWAP analysis, accurate quake and damping factors for use in refined wave equation analyses and/or further dynamic monitoring

d. To measure hammer energy transmitted to the pile, to permit evaluation of driving stresses, pile damage and other driveability factors

Dynamic testing may be performed in conjunction with test piles and static load testing of selected test piles to verify resistance estimates or provide a basis for adjusting wave equation input data. Dynamic testing is particularly applicable for piles deriving an appreciable portion of their resistance from end bearing on rock or for piles driven to blow counts exceeding ten blows per 25 mm {1 in.}.

C10.7.3.8.3

Dynamic testing entails measurement and evaluation of force and acceleration of the pile during driving to estimate pile axial resistance, hammer energy, driving stresses and other related parameters. Manipulation of the measured quantities is performed rapidly in the field using a portable minicomputer, which applies the measured input data to the solution of a dynamic equilibrium analysis of the pile. Detailed discussion of dynamic data acquisition and analysis is presented in the Dynamic Monitoring Manual.

The dynamic testing equipment presently used estimates the mobilized soil resistance and equates it to the nominal pile resistance. When pile sets are equal to or less than the quake value of the bearing strata, dynamic estimates of pile capacity tend to be conservative because full soil resistance is not mobilized. While conservative strength estimates are typical at small sets under normal conditions, dynamic capacity estimates may be unconservative where relaxation or bearing materials with high damping are present. This emphasizes the importance of redrive testing (D10.7.3.8.5) as a means of adjusting initial dynamic strength estimates to allow for unusual conditions.

10.7.3.8.3b Input Factors

Pile property and geologic input factors for dynamic testing shall accurately reflect pile, soil and rock properties. Input factors for pile properties shall be obtained from pile manufacturer literature. Preliminary soil and rock input factors (i.e., damping and quake) can be obtained from Table D10.7.3.8.4d-1. Refined soil and rock input factors resulting from CAPWAP analysis shall be used as they become available.

C10.7.3.8.3b

The greatest source of inaccuracy in dynamic testing input is typically the quake and damping factor input. To provide the most accurate values of quake and damping for use in dynamic testing, CAPWAP analyses should be performed as soon as dynamic data acquisition begins. CAPWAP analyses involve digitalization of the measured force and velocity traces and subsequent modeling of the measured force trace by iterative application of progressively refined values of quake, damping, and load distribution to match the measured velocity trace. Detailed discussion of CAPWAP analyses is presented in the Dynamic Monitoring Manual.

10.7.3.8.3c Correlation with Static Load Tests

When dynamic testing is performed in conjunction with static load testing, the strengths obtained by both wave

C10.7.3.8.3c

A correlation between dynamic capacity estimates and static load test capacities of 10% (where correlation % =

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equation analysis and dynamic testing shall be calibrated with the load test results as described in D10.7.3.8.2. Correlation shall consist of tabulating the difference between static load test strengths, wave equation analysis, and dynamic monitoring strength estimates, and assessing the consistency of overprediction or underprediction of static test nominal strengths by dynamic methods. If redriving is performed to evaluate time-dependent strength changes, the dynamic monitoring strength obtained at the beginning of redriving shall also be correlated with static load test results. The designer shall evaluate the consistency of dynamic monitoring strength estimates with regard to their reliable use for strength determination after initial static load testing has been completed. If no acceptable consistent correlation is determined, continued dynamic monitoring during the remainder of pile installation shall serve the primary purpose of verifying of acceptable hammer performance and driving stresses.

10.7.3.8.3d Redriving

Pile freeze or relaxation can be determined by redrive testing of previously installed piles. A minimum of five days shall elapse between the end of initial driving and redrive testing, unless otherwise approved by the Chief Bridge Engineer. Care shall be taken to minimize hammer energy fluctuations because at small pile sets, differences in hammer energy between the end of initial driving and the beginning of redrive testing will affect dynamic capacity estimates.

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[(Dynamic Test Capacity) - (Static Load Test Capacity) :- Static Load Test Capacity] x 100 is considered accurate). When this correlation is achieved, an abbreviated static load testing program may be applied to those remaining portions of the project involving similar soil and rock conditions.

When pile driving criteria involve final sets of less than 0.1 to 0.3 cm {0.05 to 0.1 in.} per blow (i.e., driving resistances greater than 3 to 8 blows per cm) {10 to 20 blows per in.} load testing should be performed in conjunction with dynamic monitoring.

C10.7.3.8.3d

Generally, the factors most affecting the validity of redriving results (with respect to real pile strength changes which have occurred) relate to differences in hammer performance between the end of initial driving and the start of redriving. Fluctuations in hammer energy between initial driving and redriving complicate the comparison of driving resistances and, in most cases, dynamically monitored strengths. Driving resistances observed at one hammer energy may not easily be compared to driving resistances obtained at other energies. Therefore, it is often difficult to draw meaningful conclusions regarding pile freeze or relaxation when hammer energies vary. In the case of dynamic capacity estimates at relatively small pile sets per blow (generally sets less than the quake value of the soil or rock supporting the pile), the full capacity of the pile is not mobilized and the capacity determined by dynamic monitoring tends to reflect the hammer energy expended on the pile rather than the actual pile capacity. Thus, when piles are driven or redriven at relatively high driving resistances, hammer energy fluctuations may be reflected as pile strength changes which have not actually occurred.

The problems described above illustrate the importance of consistent hammer performance for evaluation of redrive testing. Most hammers (e.g., diesel, air-steam) have certain operating characteristics which tend to introduce some degree of variation between the end of initial driving performance and the beginning of redriving performance. Double-acting hammers generally perform at optimum levels during moderately hard driving (i.e., 5 to 15 blows per 25 mm {1 in.}), but may experience cylinder lift at higher driving resistances which necessitates some throttle reduction. Most double-acting hammers must be raised up to full throttle in several steps; thus, the first several hammer blows at the start of redriving usually occur at much less than optimum energy. Single-acting diesel hammers may

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experience some loss of transmitted energy during hard driving due to increased frictional losses related to heating of the hammer. All of these hammer performance characteristics may influence the execution and/or interpretation of redrives, depending upon the differences in driving resistances and hammer condition between the end of initial driving and redriving.

The different hammer types may be approximately ranked with respect to the amount of performance variation and resultant difficulty in interpreting redrive data. Ranked from least to most difficult, the following sequence is indicated: single-acting air/steam, single-acting diesel, double-acting air/steam and, double-acting diesel. The designer should recognize the potential difficulties in using a particular hammer type for redriving operations and, where appropriate, should restrict hammer types to those which will minimize problems in obtaining usable information.

10.7.3.8.3e Capacity Prediction

Nominal axial pile resistances shall be estimated for monitored piles throughout the range of driving resistances appropriate for the type of pile(s) and support mechanism(s). Resistance estimates based on dynamic monitoring shall be representative of an average value of at least five hammer blows.

C10.7.3.8.3e

Use of an average capacity value over a range of at least two blows per cm is suggested to minimize the effect of the variations in hammer performance and capacity estimation, which typically occur between hammer blows during driving.

10.7.3.8.4 Wave Equation Analysis

A wave equation analysis may be used to establish the driving criteria. In this case, the wave equation analysis shall be performed based on the hammer and pile driving system to be used for pile installation. To avoid pile damage, driving stresses shall not exceed the values obtained in D10.7.8, using the resistance factors specified or referred to in Table D10.5.5.2.3-1. Furthermore, the blow count needed to obtain the maximum driving resistance anticipated shall be less than the maximum value established based on the provisions in D10.7.8.

A wave equation analysis should also be used to evaluate pile drivability.

10.7.3.8.4a General

The constructability of the pile foundation design shall be evaluated using GRLWEAP 2003. Wave equation analyses shall be performed on all pile/hammer/soil/rock combinations pertaining to the foundation design. The wave equation shall be used to confirm that the design pile section can be installed to the designed depth, nominal axial resistance, and within the driving stress levels specified in D10.7.8 using the resistance factors specified or referred to in Table D10.5.5.2.3-1. The wave equation analyses will be performed by the Department based on hammer data

C10.7.3.8.4a

Detailed discussion of the wave equation analysis program background and use are presented in the various user manuals currently available, Goble and Rausche (1987), and the Department's Wave Equation Analysis Manual. Publication 15A provides a compilation of wave equation analyses conducted by the Department in conjunction with previous work.

Wave equation analyses generally predict driving stresses within about 20% low to 10% high, Gannett Fleming, et al, (1985) and Thompson and Thompson

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submitted by the contractor. Revised wave equation analyses incorporating actual field conditions (e.g., as determined by pile installation, static load tests, dynamic monitoring, or CAPWAP analyses) shall be performed during the design phase if directed by the Department. Each analysis shall include a completed "Hammer Approval" form. Analyses shall reflect optimized driveability as set forth in D10.7.8.

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(1983). Static pile strengths are usually predicted with more varying degrees of accuracy, depending upon the reliability of quake and damping factors and the load carrying mechanism assumed in the analysis. The wave equation analyses are usually more useful in estimating pile driving stresses than in estimating nominal axial resistance.

10.7.3.8.4b Pile Driving Systems

Wave equation analyses shall incorporate appropriate hammer performance parameters which are furnished by the contractor when properly completed pile hammer data sheets are submitted. For hammers with variable throttle settings and strokes, the settings and strokes to be used shall be identified. Cushion, capblock and drive helmet materials, and the associated weights and stiffness and restitution coefficients shall be identified.

C10.7.3.8.4b

Where dynamic monitoring or field observations indicate appreciable differences between driving system parameters used in wave equation analyses and those actually occurring in the field, the analyses should be rerun using the revised input, representative of field conditions, or operating procedures should be made to achieve the appropriate relationship between driving resistance and pile capacity.

The values of hammer efficiencies used in the current wave equation programs generally reflect optimum hammer performance, whereas, actual hammer performance is typically less efficient. Therefore, the magnitude of transmitted hammer energy predicted by wave equation analyses typically exceeds that observed in the field. The lower efficiency results in lower driving stresses compared to those predicted by wave equation analyses.

10.7.3.8.4c Pile Characteristics

The analyzed pile shall be modeled with respect to the anticipated soil and rock strata penetrated, length of pile above and below ground, and cross-sectional variations. Input data shall accurately reflect the presence of splices, tip protection, soil plugs (in open-end pipe piles) and similar features. The properties and dimensions of the mandrel shall be used for analysis of mandrel-driven piles.

C10.7.3.8.4c

10.7.3.8.4d Quake and Damping Factors

Wave equation analyses shall incorporate quake and damping factors appropriate to the anticipated character and relative position of the soil and rock strata penetrated. Quake and damping factors suitable for preliminary analyses are given in Tables 1 and 2. If available, values of quake and damping factors based on dynamic monitoring and/or CAPWAP analysis shall be used.

C10.7.3.8.4d

Quake and damping factor(s) are used in wave equation analyses to model the elasto-plastic behavior and dynamic impedance of the soil and rock profile through and into which the pile is driven. Variations in the values used in analyses, particularly for damping factors, may exert appreciable influence on the static pile strength predicted as a function of driving resistance. The higher the values of quake and damping factor used in the analysis the lower the capacity predicted at a particular driving resistance. In cases where the pile is driven through layers of greatly differing damping characteristics (e.g., granular and cohesive layers, or granular material overlying soft rock), effects of soil layering should be considered in the prediction of pile performance.

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Table 10.7.3.8.4d -1 – Preliminary Quake Factors for Impact Driven Piles (Pile Dynamics, Inc. 2003).

SOIL TYPE	PILE TYPE OR SIZE	SKIN QUAKE	TOE QUAKE
All soil types	Non-displacement piles* i.e. driving unplugged	2.5 mm {0.10 in}	2.5 mm {0.10 in}
Very dense or hard soils	Displacement piles**of diameter or width D	2.5 mm {0.10 in}	D/120
Loose or soft soils	Displacement piles** of diameter or width D	2.5 mm {0.10 in}	D/60
Soft Rock	Non-displacement piles* i.e. driving unplugged	-	2.5 mm {0.10 in}
Hard Rock	All types	-	1.0 mm {0.04 in}

* Non-displacement piles are sheet pile, H-Piles, or open-ended pipe piles which are not plugging during driving. Normally it can be assumed that pipe piles with diameters of 750 mm {30 inches} or more will not plug during driving while H-Piles and pipe piles of diameter 500 mm {20 inches} or less will plug during driving into a bearing layer. Between 500 and 750 mm (20 to 30 inches), pipe piles may or may not plug.

** Displacement piles are closed-ended pipe piles, pipe piles, or H-Piles that are plugged during driving and solid concrete piles. Normally, H-Piles and pipe piles with diameters 500 mm {20 inches} or less would be modeled as displacement piles.

Table 10.7.3.8.4d -2 – Preliminary Damping Factors for Impact Driven Piles (Pile Dynamics, Inc. 2003).

SOIL TYPE	SKIN DAMPING*	TOE DAMPING
Non-cohesive soils	0.16 sec/m {0.05 sec/ft}	0.50 sec/m {0.15 sec/ft}
Cohesive soils	0.65 sec/m {0.20 sec/ft}	0.50 sec/m {0.15 sec/ft}

* For mixed soils, intermediate values may be appropriate.

10.7.3.8.4e Nominal Axial Resistance

The nominal axial pile resistance can be predicted from the relationship between driving resistance and nominal axial resistance developed from the wave equation analysis. Wave equation analyses used for nominal axial resistance prediction shall meet the optimized driveability requirements of D10.7.8 at the driving resistances specified for the various pile support mechanisms (point bearing, end bearing or friction).

10.7.3.8.5 Dynamic Formula

A Dynamic Formula should not be used to establish the driving criterion unless pre-approved by the Chief Bridge Engineer.

C10.7.3.8.5

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10.7.3.8.6 Static Analysis

10.7.3.8.6a General

Where a static analysis prediction method is used to determine pile installation criteria, i.e., for bearing resistance, the nominal pile resistance shall be factored at the strength limit state using the resistance factors in Table D10.5.5.2.3-1 associated with the method used to compute the nominal bearing resistance of the pile. The factored bearing resistance of piles, R_R, may be taken as:

$$R_R = \varphi R_n \quad (10.7.3.8.6a\text{-}1)$$

or:

$$R_R = \varphi R_n = \varphi_{stat} R_p + \varphi_{stat} R_s \quad (10.7.3.8.6a\text{-}2)$$

in which:

$$R_p = q_p A_p \quad (10.7.3.8.6a\text{-}3)$$

$$R_s = q_s A_s \quad (10.7.3.8.6a\text{-}4)$$

where:

φ_{stat} = resistance factor for the bearing resistance of a single pile specified in D10.5.5.2.3

R_p = pile tip resistance (N){kips}

R_s = pile side resistance (N){kips}

q_p = unit tip resistance of pile (MPa){ksf}

q_s = unit side resistance of pile (MPa){ksf}

A_s = surface area of pile side (mm^2) {ft.^{2}}

A_p = area of pile tip (mm^2) {ft.^{2}}

Both total stress and effective stress methods may be used, provided the appropriate soil strength parameters are available. The resistance factors for the skin friction and tip resistance, estimated using these methods, shall be as specified in Table D10.5.5.2.3-1. The limitations of each method as described in DC10.5.5.2.3 should be applied in the use of these static analysis methods.

10.7.3.8.6b α-Method

The α-method, based on total stress, may be used to relate the adhesion between the pile and clay to the

COMMENTARY

C10.7.3.8.6a

For use of static analysis methods for contract pile quantity estimation, see D10.7.3.3.

The nominal axial resistance of piles in cohesive soils may be calculated using a total stress method (e.g., Tomlinson (1957)) for undrained loading conditions, or an effective stress method (e.g., Meyerhof (1976)), for drained loading conditions. The nominal axial resistance may also be calculated from in situ testing methods, such as the cone penetration (e.g., Schmertmann (1978)).

The nominal axial resistance of piles in cohesionless soils may be calculated using an empirical effective stress method (e.g., Nordlund) or from in-situ testing methods and analysis, such as the cone penetration (e.g., Schmertmann (1978)).

C10.7.3.8.6b

The α-method has been used for many years and gives reasonable results for both displacement and

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undrained strength of the clay. For this method, the nominal unit skin friction, in MPa, shall be taken as:

$$q_s = \alpha S_u \qquad (10.7.3.8.6b\text{-}1)$$

where:

S_u = undrained shear strength (MPa){ksf}

α = adhesion factor applied to S_u (dim.)

The adhesion factor for this method, α, shall be assumed to vary with the value of the undrained strength, S_u, as shown in Figure 1.

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nondisplacement piles in clay.

In general, this method assumes that a mean value of S_u will be used. It may not always be possible to establish a mean value, as in many cases, data are too limited to reliably establish the mean value. The Engineer should apply engineering judgment and local experience as needed to establish an appropriate value for design (see DC10.4.6).

If the undrained shear strength is not reasonably constant along the pile length, the side resistance should be estimated along discrete lengths of the pile.

For H-piles, the perimeter or "box" area should generally be used to compute the surface area of the pile side.

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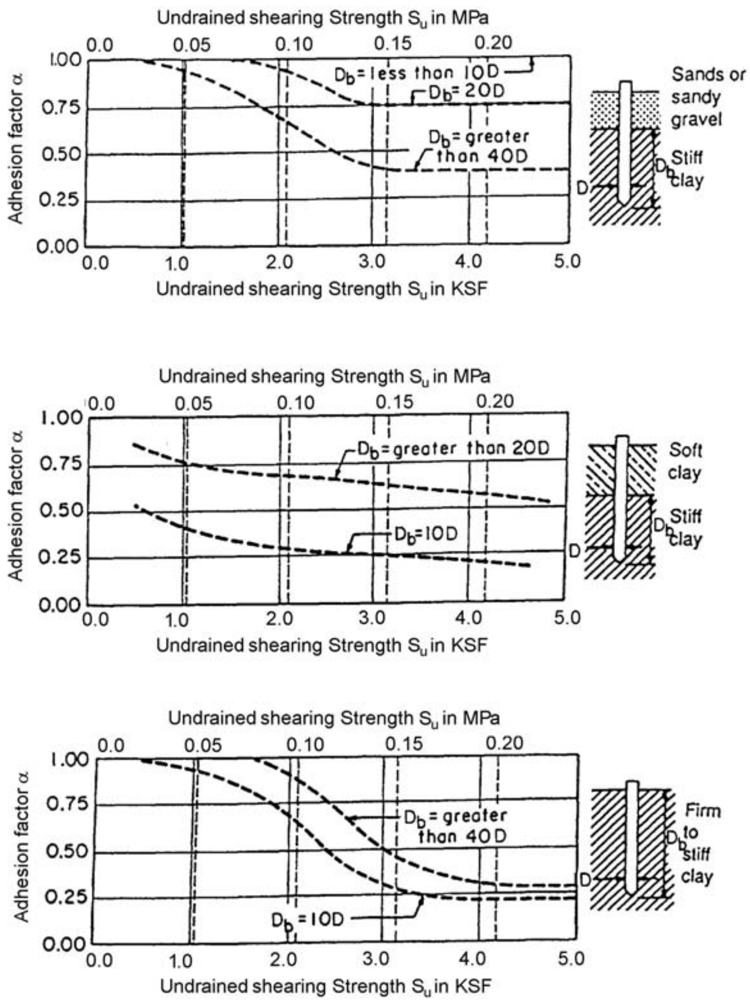

Figure 10.7.3.8.6b-1 Design Curves for Adhesion Factors for Piles Driven into Clay Soils after Tomlinson (*1980*).

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10.7.3.8.6c β-Method

The β-method, based on effective stress, may be used for predicting skin friction of prismatic piles. The nominal unit skin friction for this method, in MPa, shall be related to the effective stresses in the ground as:

$$q_s = \beta\sigma'_v \qquad (10.7.3.8.6c\text{-}1)$$

where:

σ'_v = vertical effective stress (MPa){ksf}

β = a factor taken from Figure 1

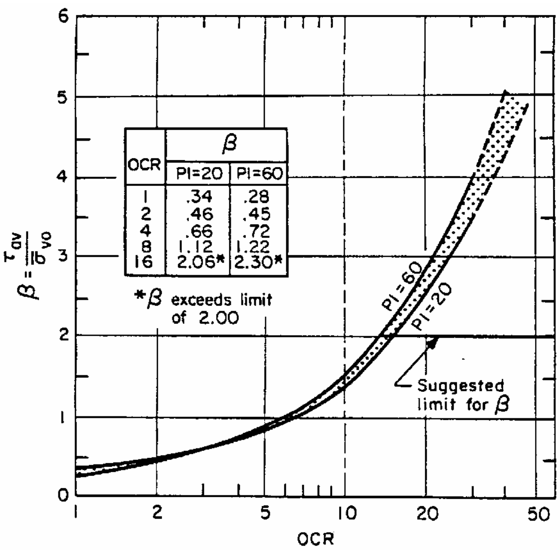

Figure 10.7.3.8.6c-1 β Versus OCR for Displacement Piles after Esrig and Kirby (*1979*)

COMMENTARY

C10.7.3.8.6c

The β-method has been found to work best for piles in normally consolidated and lightly overconsolidated clays. The method tends to overestimate skin friction of piles in heavily overconsolidated soils. Esrig and Kirby (*1979*) suggested that for heavily overconsolidated clays, the value of β should not exceed 2.

The term β implicitly includes the effects of variations in interface angle of shearing resistance and the in-situ horizontal stress state, and may be taken as:

$$\beta = k \tan \delta \qquad (C10.7.3.3.2b\text{-}1)$$

where:

k = post-construction lateral earth pressure coefficient (dim)

δ = effective stress angle of shearing resistance between soil and pile (deg)

10.7.3.8.6d λ-Method

The λ-method, based on effective stress (though it does contain a total stress parameter), may be used to relate the unit skin friction, in MPa {ksf}, to passive earth pressure. For this method, the unit skin friction shall be taken as:

$$q_s = \lambda(\sigma'_v + 2S_u) \qquad (10.7.3.8.6d\text{-}1)$$

C10.7.3.8.6d

The value of λ decreases with pile length and was found empirically by examining the results of load tests on steel pipe piles.

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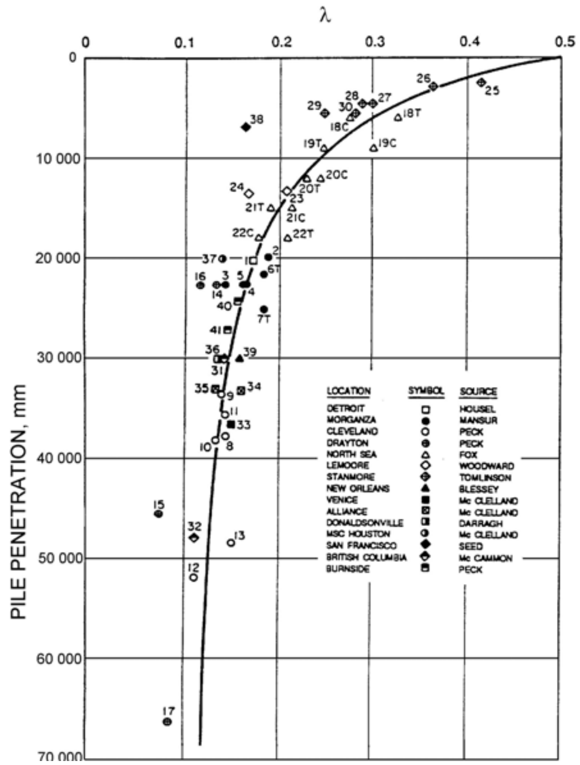
where:

$\sigma'_v + 2S_u$ = passive lateral earth pressure (MPa){ksf}

σ'_v = the effective vertical stress at midpoint of soil layer under consideration (MPa){ksf}

λ = an empirical coefficient taken from Figure 1 (dim.).

Metric Units:



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U.S. Customary Units:

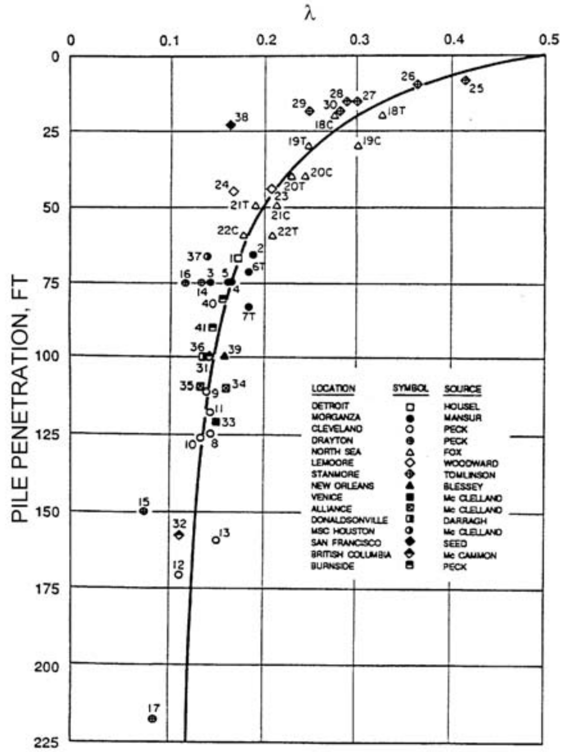

Figure 10.7.3.8.6d-1 λ Coefficient for Driven Pipe Piles after Vijayvergiya and Focht (*1972*).

10.7.3.8.6e Tip Resistance in Cohesive Soils

The nominal unit tip resistance of piles in saturated clay, in MPa, shall be taken as:

$$q_p = 9S_u \quad (10.7.3.8.6e\text{-}1)$$

where:

S_u = undrained shear strength of the clay near the pile base (MPa){ksf}

COMMENTARY

C10.7.3.8.6e

Equation 1 is a simplified expression for the nominal unit tip resistance of a pile for undrained loading which is applicable only when the following criteria are met:

- the weight of the pile is approximately equal to the weight of soil it displaces,
- $D_f/D \geq 5$, and
- $E_s/3S_u \geq 8$.

10.7.3.8.6f Nordlund/Thurman Method in Cohesionless Soils

The Department's preferred method for predicting side resistance of piles in cohesionless soils is the Nordlund/Thurman Method.

This effective stress method should be applied only to sands and nonplastic silts. The nominal unit side resistance, q_s, for this method, in MPa, shall be taken as:

C10.7.3.8.6f

Detailed design procedures for the Nordlund/Thurman method are provided in Hannigan et al., (*2005*). This method was derived based on load test data for piles in sand. In practice, it has been used for gravelly soils as well.

The effective overburden stress is not limited in Eq. 1.

For H-piles, the perimeter or "box" area should generally be used to compute the surface area of the pile side.

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$$q_s = K_\delta C_F \sigma'_v \frac{\sin(\delta + \omega)}{\cos \omega} \qquad (10.7.3.8.6f\text{-}1)$$

where:

K_δ = coefficient of lateral earth pressure at mid-point of soil layer under consideration from Figures 1 through 4 (dim.)

C_F = correction factor for K_δ when $\delta \neq \phi_f$, from Figure 5

σ'_v = effective overburden stress at midpoint of soil layer under consideration (MPa){ksf}

δ = friction angle between pile and soil obtained from Figure 6 (deg.)

ω = angle of pile taper from vertical (deg.)

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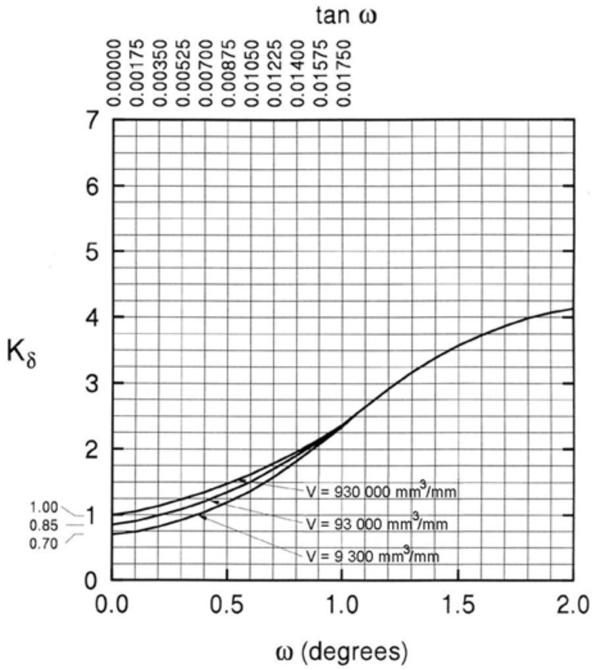
COMMENTARY

Metric Units:



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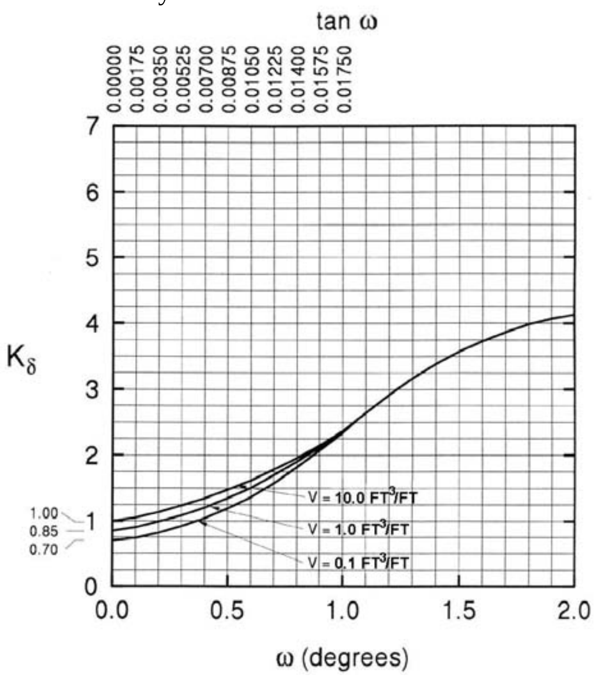

Figure 10.7.3.8.6f-1 Design Curve for Evaluating K_δ for Piles where $\phi_f = 25^o$ (*Hannigan et al., 2005* after Nordlund, *1979*).

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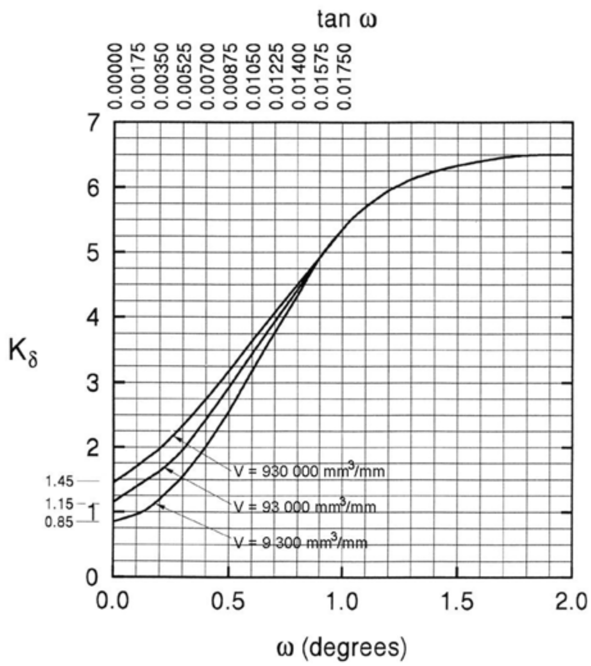
Metric Units:



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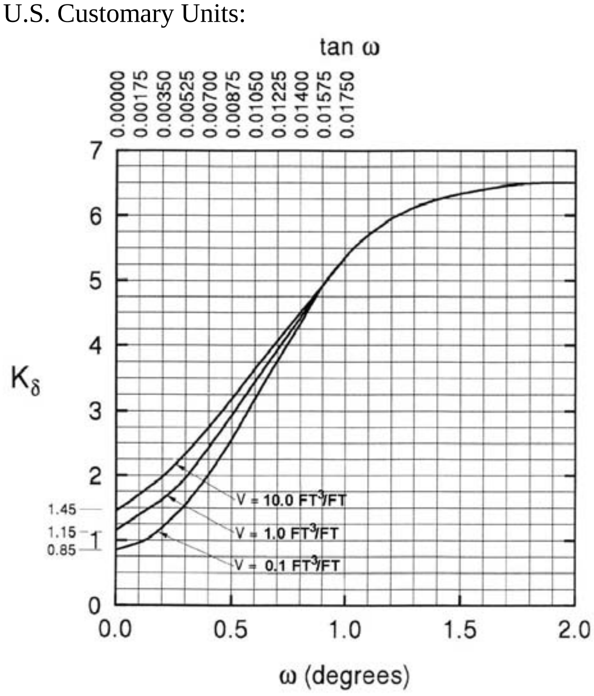

Figure 10.7.3.8.6f-2 Design Curve for Evaluating K_δ for Piles where $\phi_f = 30^o$ (*Hannigan et al., 2005* after *Nordlund, 1979*).

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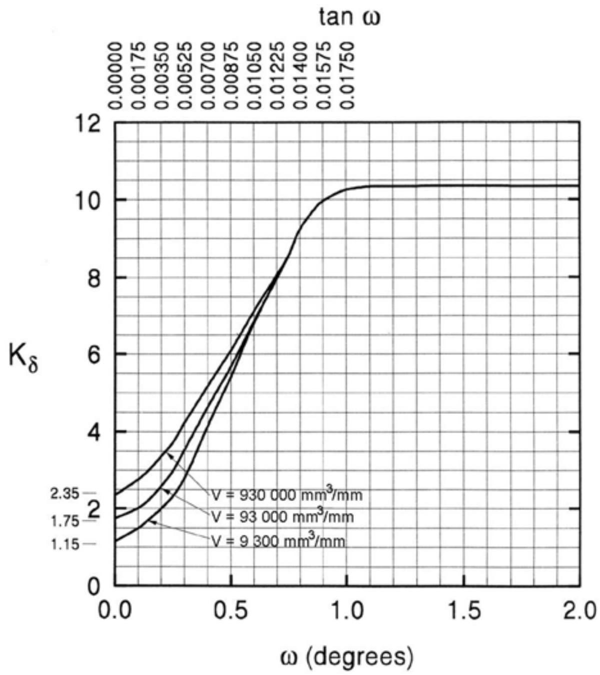
Metric Units:



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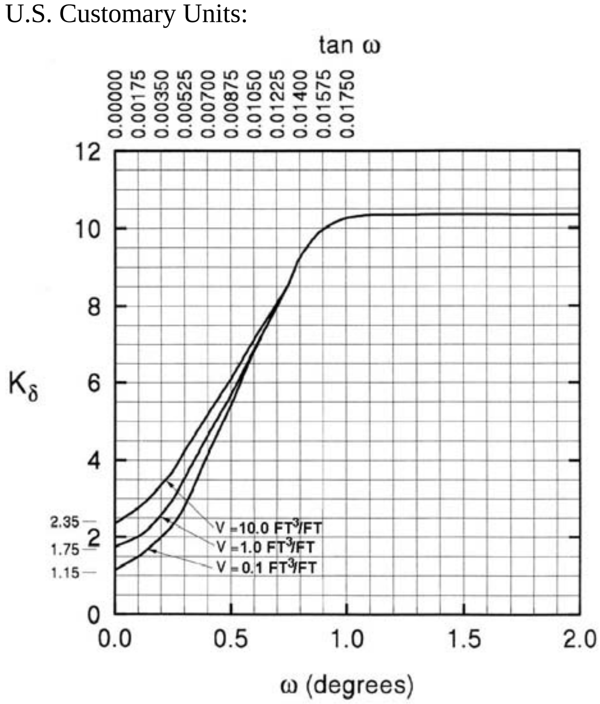

Figure 10.7.3.8.6f-3 Design Curve for Evaluating K_δ for Piles where $\phi_f = 35^o$ (Hannigan et al., 2005 after *Nordlund, 1979*).

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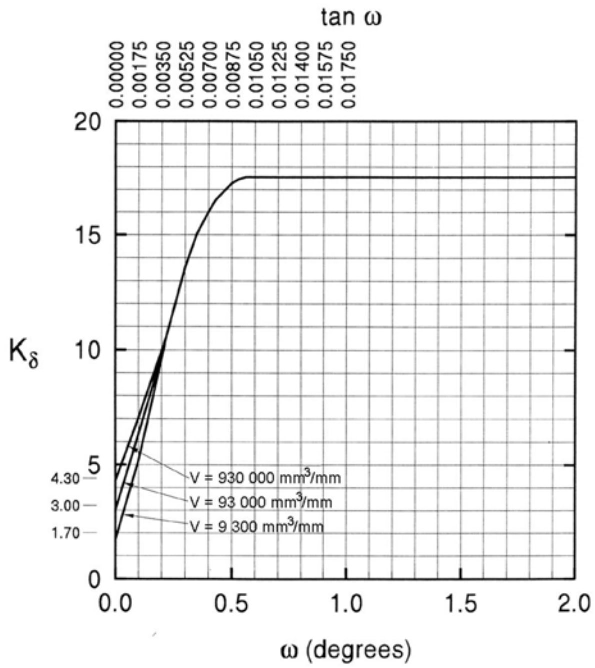
COMMENTARY

Metric Units:



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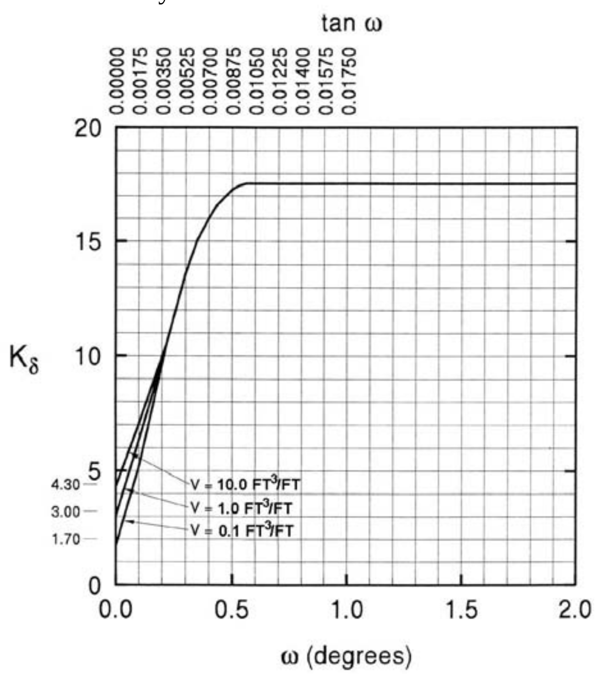

Figure 10.7.3.8.6f-4 Design Curve for Evaluating K_δ for Piles where $\phi_f = 40^o$ (*Hannigan et al., 2005* after *Nordlund, 1979*).

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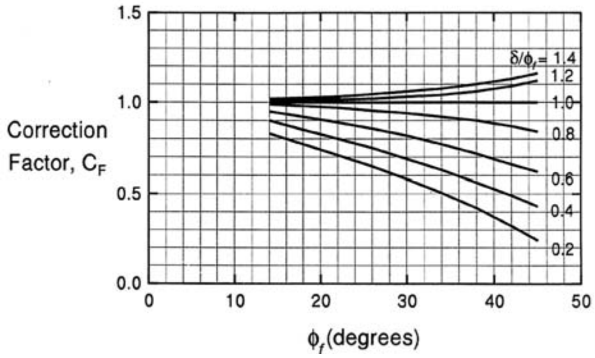
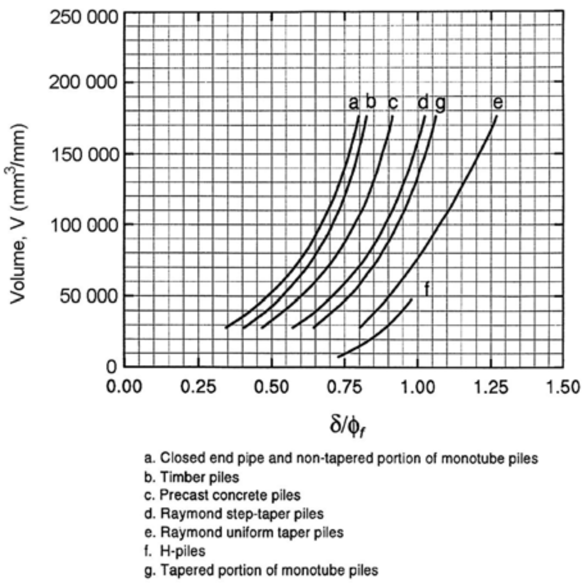


Figure 10.7.3.8.6f-5 Correction Factor for K_δ where $\delta \neq \phi_f$ (*Hannigan et al., 2005* after *Nordlund, 1979*).

Metric Units:



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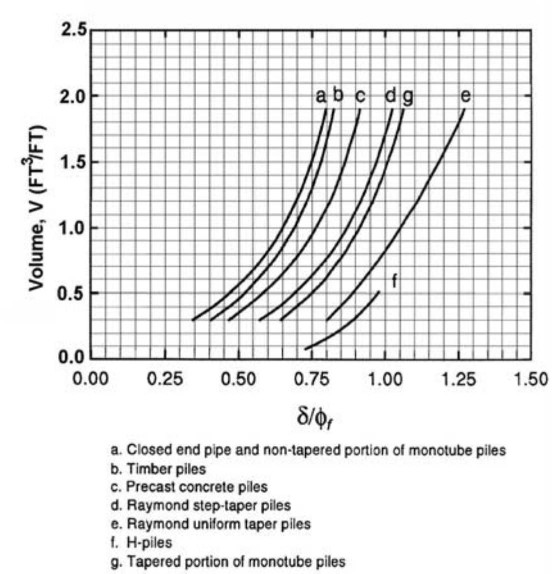

Figure 10.7.3.8.6f-6 Relationship Between δ/ϕ_f and Pile Displacement, V, for Various Types of Piles (*Hannigan et al., 2005* after *Nordlund, 1979*).

The nominal unit tip resistance, q_p, in MPa by the Nordlund/Thurman method shall be taken as:

$$q_p = \alpha_t N'_q \sigma'_v \leq q_L \qquad (10.7.3.8.6f\text{-}3)$$

where:

α_t = coefficient from Figure 7 (dim.)

N'_q = bearing capacity factor from Figure 8

σ'_v = effective overburden stress at pile tip (MPa){ksf} ≤0.15 MPa {≤ 3.2 ksf}

q_L = limiting unit tip resistance from Figure 9

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If the friction angle, ϕ_f, is estimated from average, corrected *SPT* blow counts, $N1_{60}$, the $N1_{60}$ values should be averaged over the zone from the pile tip to 3 diameters below the pile tip.

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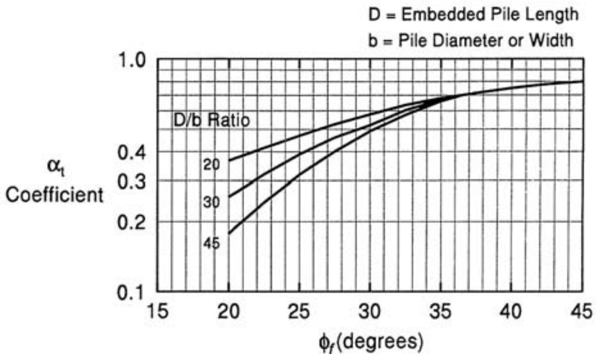


Figure 10.7.3.8.6f-7 α_t Coefficient (*Hannigan et al., 2005* modified after *Bowles, 1977*).

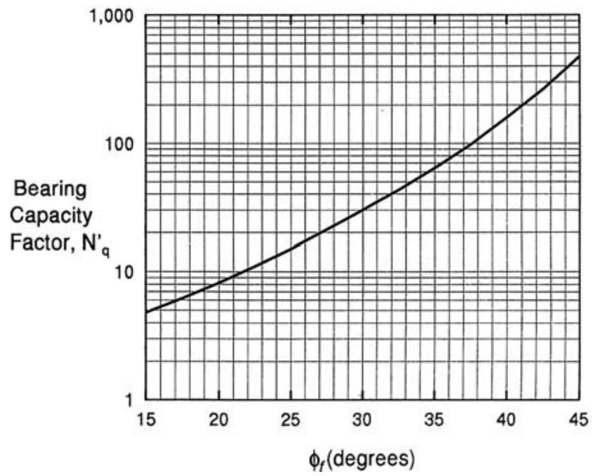
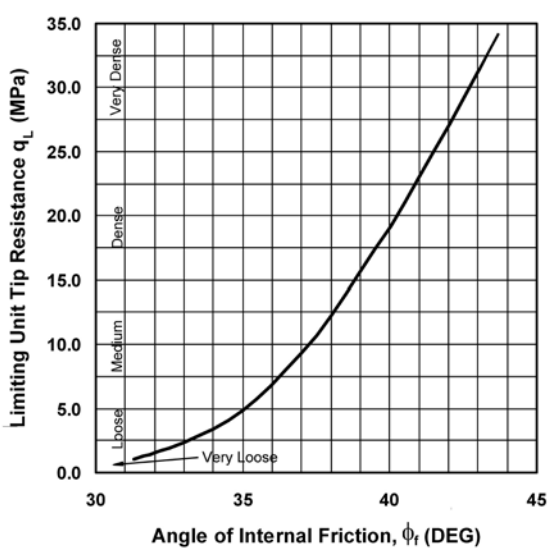


Figure 10.7.3.8.6f-8 Bearing Capacity Factor, N'_q, (*Hannigan et al., 2005* modified after *Bowles, 1977*).

Metric Units:



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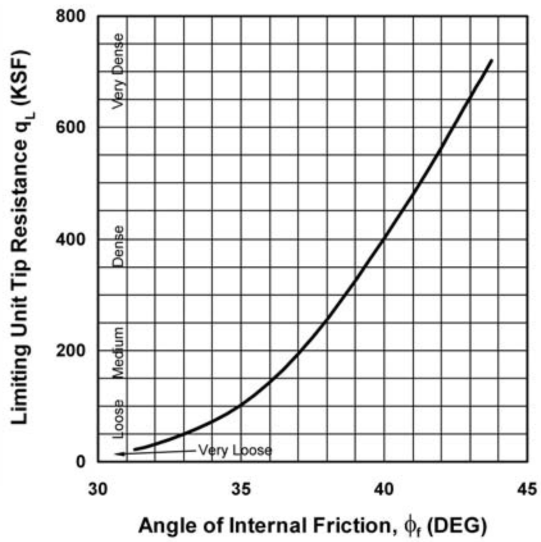

Figure 10.7.3.8.6f-9 Limiting Unit Pile Tip Resistance (*Hannigan et al., 2005* after *Meyerhof, 1976*).

10.7.3.8.6g Using SPT or CPT in Cohesionless Soils

These methods shall be applied only to sands and nonplastic silts.

The nominal unit tip resistance for the Meyerhof method, in MPa {ksf}, for piles driven to a depth D_b into a cohesionless soil stratum shall be taken as:

Metric Units:

$$q_p = \frac{0.038(N1_{60})D_b}{D} \leq q_\ell \qquad (10.7.3.8.6g\text{-}1)$$

U.S. Customary Units:

$$q_p = \frac{0.8(N1_{60})D_b}{D} \leq q_\ell$$

where:

$N1_{60}$ = representative *SPT* blow count near the pile tip corrected for overburden pressure as specified in D10.4.6.2.4 (blows/300 mm){blows/ft.}

D = pile width or diameter (mm){ft.}

D_b = depth of penetration in bearing strata (mm){ft.}

q_ℓ = limiting tip resistance taken as eight times the value of $0.4N1_{60}$ { $N1_{60}$} for sands and six

COMMENTARY

C10.7.3.8.6g

In-situ tests are widely used in cohesionless soils because obtaining good quality samples of cohesionless soils is very difficult. In-situ test parameters may be used to estimate the tip resistance and skin friction of piles.

Two frequently used in-situ test methods for predicting pile axial resistance are the standard penetration test (*SPT*) method (*Meyerhof, 1976*) and the cone penetration test (*CPT*) method (*Nottingham and Schmertmann, 1975*). CPT is typically only used in fine sands and silts. It is not practical for coarse sands and gravels.

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times the value of $0.3N1_{60}$ $\{N1_{60}\}$ for nonplastic silt (MPa){ksf}

The nominal skin friction of piles in cohesionless soils for the Meyerhof method, in MPa, shall be taken as:

- For driven displacement piles:

Metric Units: $q_s = 0.0019\bar{N}1_{60}$ (10.7.3.8.6g-2)

U.S. Customary Units: $q_s = \dfrac{\bar{N}1_{60}}{25}$

- For nondisplacement piles, e.g., steel H-piles:

Metric Units: $q_s = 0.00096\bar{N}1_{60}$ (10.7.3.8.6g-3)

U.S. Customary Units: $q_s = \dfrac{\bar{N}1_{60}}{50}$

where:

q_s = unit skin friction for driven piles (MPa){ksf}

$\bar{N}1_{60}$ = average corrected *SPT*-blow count along the pile side (blows/300 mm){blows/ft.}

Tip resistance, q_p, for the Nottingham and Schmertmann method, in MPa {ksf}, shall be determined as shown in Figure 1.

In which:

$$q_p = \frac{q_{c1} + q_{c2}}{2} \quad (10.7.3.8.6g\text{-}4)$$

where:

q_{c1} = average q_c over a distance of yD below the pile tip (path a-b-c); sum q_c values in both the downward (path a-b) and upward (path b-c) directions; use actual q_c values along path a-b and the minimum path rule along path b-c; compute q_{c1} for y-values from 0.7 to 4.0 and use the minimum q_{c1} value obtained (MPa){ksf}

q_{c2} = average q_c over a distance of $8D$ above the pile tip (path c-e); use the minimum path rule as for path b-c in the q_{c1}, computations; ignore any minor "x" peak depressions if in sand but include in minimum path if in clay (MPa){ksf}

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Displacement piles, which have solid sections or hollow sections with a closed end, displace a relatively large volume of soil during penetration. Nondisplacement piles usually have relatively small cross-sectional areas, e.g., steel H-piles and open-ended pipe piles that have not yet plugged. Plugging occurs when the soil between the flanges in a steel H-pile or the soil in the cylinder of an open-ended steel pipe pile adheres fully to the pile and moves down with the pile as it is driven.

CPT may be used to determine:

- The cone penetration resistance, q_c, which may be used to determine the tip resistance of piles, and
- Sleeve friction, f_s, which may be used to determine the skin friction resistance.

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The minimum average cone resistance between 0.7 and 4 pile diameters below the elevation of the pile tip shall be obtained by a trial and error process, with the use of the minimum-path rule. The minimum-path rule shall also be used to find the value of cone resistance for the soil for a distance of eight pile diameters above the tip. The two results shall be averaged to determine the pile tip resistance.

The nominal skin friction resistance of piles for this method, in N {kips}, shall be taken as:

$$R_s = K_{s,c}\left[\sum_{i=1}^{N_1}\left(\frac{L_i}{8D_i}\right) f_{si} a_{si} h_i + \sum_{i=1}^{N_2} f_{si} a_{si} h_i\right] \quad (10.7.3.8.6g\text{-}5)$$

where:

$K_{s,c}$ = correction factors: K_c for clays and K_s for sands from Figure 2 (dim.)

L_i = depth to middle of length interval at the point considered (mm){ft.}

D_i = pile width or diameter at the point considered (mm){ft.}

f_{si} = unit local sleeve friction resistance from *CPT* at the point considered (MPa){ksf}

a_{si} = pile perimeter at the point considered (mm){ft.}

h_i = length interval at the point considered (mm){ft.}

N_1 = number of intervals between the ground surface and a point 8*D* below the ground surface

N_2 = number of intervals between 8*D* below the ground surface and the tip of the pile

COMMENTARY

This process is described in Nottingham and Schmertmann (*1975*).

For a pile of constant cross-section (nontapered), Eq. 5 can be written as:

$$R_s = K_{s,c}\left[\frac{a_s}{8D}\sum_{i=1}^{N_1} L_i f_{si} h_i + a_s \sum_{i=1}^{N_2} f_{si} h_i\right] \quad (C10.7.3.8.6g\text{-}1)$$

If, in addition to the pile being prismatic, f_s is approximately constant at depths below 8*D*, Eq. C1 can be simplified to:

$$R_s = K_{s,c}\left[a_s f_s (Z - 4D)\right] \quad (C10.7.3.8.6g\text{-}2)$$

where:

Z = total embedded pile length (mm){ft.}

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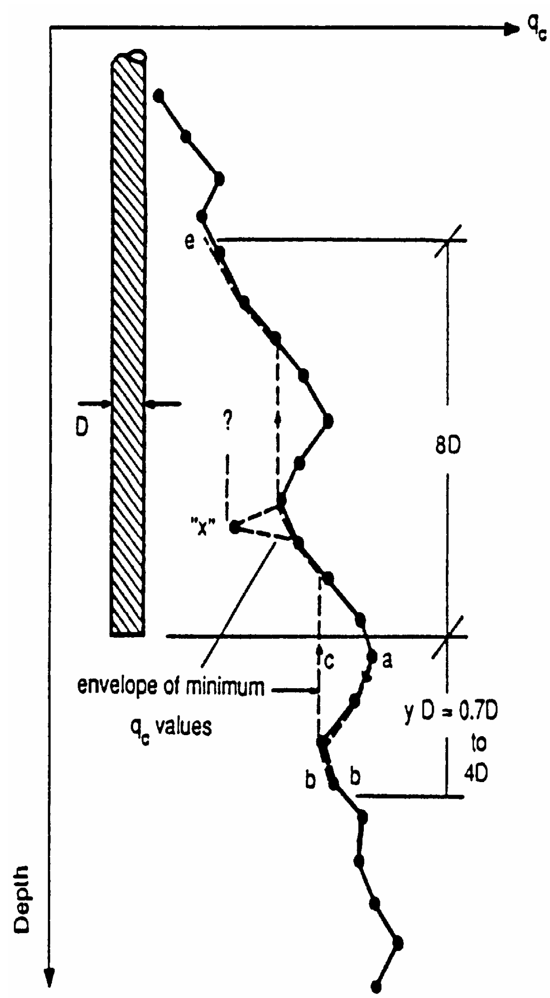

Figure 10.7.3.8.6g-1 Pile End-Bearing Computation Procedure after Nottingham and Schmertmann (*1975*).

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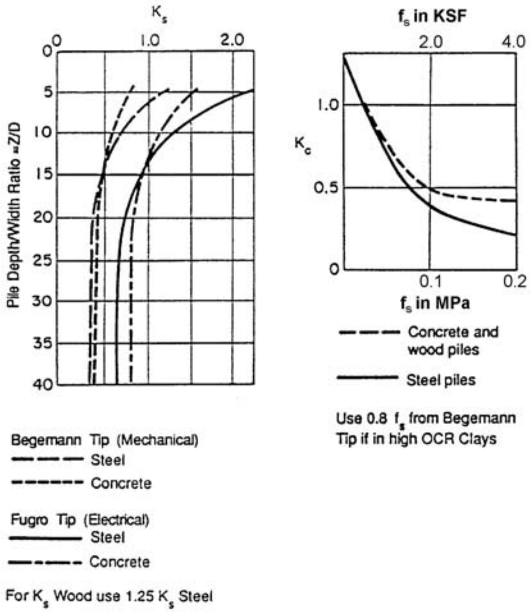
Metric Units:



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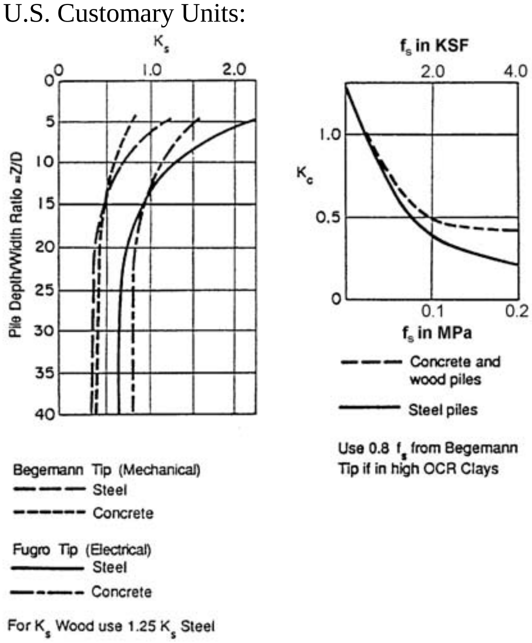

Figure 10.7.3.8.6g-2 Side Friction Correction Factors K_s and K_c after Nottingham and Schmertmann (*1975*).

10.7.3.9 RESISTANCE OF PILE GROUPS IN COMPRESSION

For pile groups in clay, the nominal axial resistance of the pile group shall be taken as the lesser of:

- The sum of the individual nominal resistances of each

COMMENTARY

C10.7.3.9

The equivalent pier approach checks for block failure and is generally only applicable for pile groups within cohesive soils. For pile groups in sand, the sum of the nominal resistances of the individual piles always controls the group resistance.

When analyzing the equivalent pier, the full shear

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pile in the group, or

- The nominal resistance of an equivalent pier consisting of the piles and the block of soil within the area bounded by the piles.

If the cap is not in firm contact with the ground and if the soil at the surface is soft, the individual resistance of each pile shall be multiplied by an efficiency factor η, taken as:

- η = 0.65 for a center-to-center spacing of 2.5 diameters,
- η = 1.0 for a center-to-center spacing of 6.0 diameters.
- For intermediate spacings, the value of η may be determined by linear interpolation.

If the cap is in firm contact with the ground, no reduction in efficiency shall be required. If the cap is not in firm contact with the ground and if the soil is stiff, no reduction in efficiency shall be required.

The resistance factor for an equivalent pier or block failure shall be as given in Table D10.5.5.2.3-1. The resistance factors for the group resistance calculated using the sum of the individual resistances are the same as those for the single pile resistance as given in Table D10.5.5.2.3-1.

The bearing resistance of pile groups in cohesionless soil shall be the sum of the resistance of all the piles in the group. The efficiency factor, η, shall be 1.0 where the pile cap is or is not in contact with the ground for a center-to-center pile spacing of 2.5 diameters or greater. The resistance factor is the same as that for single piles, as specified in Table D10.5.5.2.3-1.

For pile groups in clay or sand, if a pile group is tipped in a strong soil deposit overlying a weak deposit, the block bearing resistance shall be evaluated with consideration to pile group punching as a group into the underlying weaker layer. The methods in D10.6.3.1.2a of determining bearing resistance of a spread footing in a strong layer overlying a weaker layer shall apply, with the notional footing located as shown in D10.7.2.3.

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strength of the soil should be used to determine the friction resistance. The total base area of the equivalent pier should be used to determine the end bearing resistance. The additional resistance of the cap shall be ignored.

The efficiency of pile groups in cohesive soil may be diminished from the individual pile due to overlapping zones of shear deformation in the soil surrounding the piles. In cohesive soils, the resistance of a pile group depends on whether the cap is in firm contact with the ground beneath. If the cap is in firm contact, the soil between the pile and the pile group behave as a unit.

At small pile spacings, a block type failure mechanism may prevail, whereas individual pile failure may occur at larger pile spacings. It is necessary to check for both failure mechanisms and design for the case that yields the minimum capacity.

For a pile group of width *X*, length *Y*, and depth *Z*, as shown in Figure C1, the nominal resistance for block failure, in N {kips}, is given by:

$$R_g = \left(2X + 2Y\right) Z\overline{S}_u + XYN_cS_u \qquad \text{(C10.7.3.9-1)}$$

in which:

for $\dfrac{Z}{X} \le 2.5$:

$$N_c = 5\left(1 + \frac{0.2X}{Y}\right)\left(1 + \frac{0.2Z}{X}\right) \qquad \text{(C10.7.3.9-2)}$$

for $\dfrac{Z}{X} > 2.5$:

$$N_c = 7.5\left(1 + \frac{0.2X}{Y}\right) \qquad \text{(C10.7.3.9-3)}$$

where:

$\overline{S_u}$ = average undrained shear strength along the depth of penetration of the piles (MPa){ksf}

S_u = undrained shear strength at the base of the group (MPa){ksf}

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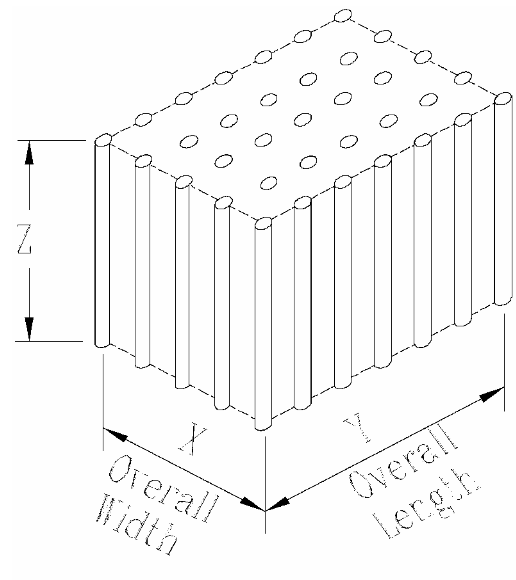

Figure C10.7.3.9-1 Pile Group Acting as a Block Foundation.

10.7.3.10 UPLIFT RESISTANCE OF SINGLE PILES

Uplift resistance at the Service Limit State shall not be used without approval by the Chief Bridge Engineer.

Uplift on single piles shall be evaluated when tensile forces are present. The factored nominal tensile resistance of the pile due to soil failure shall be greater than the factored pile loads.

The uplift resistance of a single pile should be estimated in a manner similar to that for estimating the skin friction resistance of piles in compression specified in D10.7.3.8.6.

Factored uplift resistance in N {kips} shall be taken as:

$$R_R = \varphi R_n = \varphi_{up} R_s \qquad (10.7.3.10\text{-}1)$$

where:

R_s = nominal uplift resistance due to side resistance (N){kips}

φ_{up} = resistance factor for uplift resistance specified in Table D10.5.5.2.3-1

Uplift resistance of single piles may be determined by static load test. If a static uplift test is to be performed, it shall follow the procedures specified in ASTM D 3689.

The pile load test(s) should be used to calibrate the static analysis method, i.e., back calculate soil properties, to adjust the calculated uplift resistance for variations in the stratigraphy. The minimum penetration criterion to obtain the desired uplift resistance should be based on the

C10.7.3.10

The factored load effect acting on any pile in a group may be estimated using the traditional elastic strength of materials procedure for a cross-section under thrust and moment. The cross-sectional properties should be based on the pile as a unit area.

Note that the resistance factor for uplift already is reduced to 80 percent of the resistance factor for static skin friction resistance. Therefore, the skin friction resistance estimated based on D10.7.3.8.6 does not need to be reduced to account for uplift effects on skin friction.

Static uplift tests should be evaluated using a modified Davisson Method as described in Hannigan et al. (*2005*).

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calculated uplift resistance using the pile load test results.

10.7.3.11 UPLIFT RESISTANCE OF PILE GROUPS

Uplift resistance shall not be used without approval by the Chief Bridge Engineer.

The nominal uplift resistance of pile groups shall be evaluated when the foundation is subjected to uplift loads.

Pile group factored uplift resistance, in N, shall be taken as:

$$R_R = \varphi R_n = \varphi_{ug} R_{ug} \quad (10.7.3.11\text{-}1)$$

where:

φ_{ug} = resistance factor specified in Table D10.5.5.2.3-1

R_{ug} = nominal uplift resistance of the pile group (N){kips}

The uplift resistance, R_{ug}, of a pile group shall be taken as the lesser of:

- the sum of the individual pile uplift resistance, or
- the uplift resistance of the pile group considered as a block.

For pile groups in cohesionless soil, the weight of the block that will be uplifted shall be determined using a spread of load of 1*H* in 4*V* from the base of the pile group taken from Figure 1. Buoyant unit weights shall be used for soil below the groundwater level.

In cohesive soils, the block used to resist uplift in undrained shear shall be taken from Figure 2. The nominal group uplift resistance may be taken as:

$$R_n = R_{ug} = (2XZ + 2YZ)\overline{S}_u + W_g \quad (10.7.3.11\text{-}2)$$

where:

X = width of the group, as shown in Figure 2 (mm){ft.}

Y = length of the group, as shown in Figure 2 (mm){ft.}

Z = depth of the block of soil below pile cap taken from Figure 2 (mm){ft.}

$\overline{S}_u$ = average undrained shear strength along the sides of the pile group (MPa){ksf}

W_g = weight of the block of soil, piles, and pile cap (N){kips}

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A net uplift force can act on the foundation. An example of such a load is the construction load induced during the erection of concrete segmental girder bridges.

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The resistance factor for the nominal group uplift resistance, R_{ug}, determined as the sum of the individual pile resistance, shall be taken as the same as that for the uplift resistance of single piles as specified in Table D10.5.5.2.3-1.

The resistance factor for the uplift resistance of the pile group considered as a block shall be taken as specified in Table D10.5.5.2.3-1 for pile groups in clay and in sand.

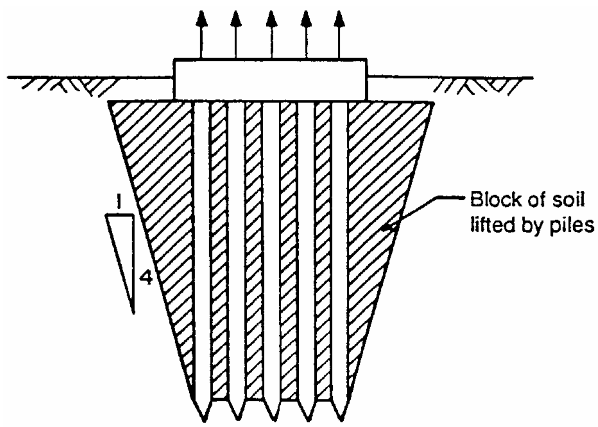

Figure 10.7.3.11-1 Uplift of Group of Closely Spaced Piles in Cohesionless Soils after Tomlinson (*1987*).

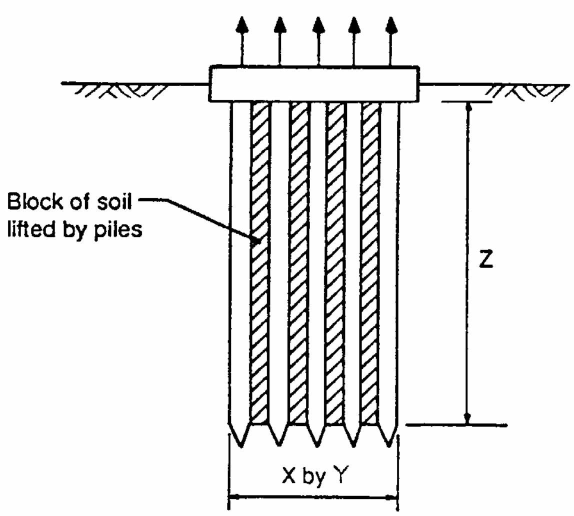

Figure 10.7.3.11-2 Uplift of Group of Piles in Cohesive Soils after Tomlinson (*1987*).

10.7.3.12 NOMINAL HORIZONTAL RESISTANCE OF PILE FOUNDATIONS

10.7.3.12.1 Batter Piles

The batter piles in a pile group shall resist the entire

C10.7.3.12.1

This method is used in the Department's computer

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horizontal load through the horizontal component of their axial capacity, unless an alternate method is approved by the Chief Bridge Engineer.

Refer to D6.15.1 for design procedures for pile groups containing battered piles.

10.7.3.12.2 Vertical Piles

The nominal resistance of pile foundations to horizontal loads shall be evaluated based on both geomaterial and structural properties. The horizontal soil resistance along the piles should be modeled using *P-y* curves developed for the soils at the site.

The applied loads shall be factored loads and they must include both horizontal and axial loads. The analysis may be performed on a representative single pile with the appropriate pile top boundary condition or on the entire pile group. Minimum embedment of piles into the pile footing shall be in accordance with D10.7.1.2 and Standard Drawing BD-621M.

For this embedment, the piles shall be designed assuming both full pile-head fixity and 50% pile-head fixity. The 50% pile-head fixity condition shall be simulated by application of one-half of the fixed-head moment (as a negative moment) to the top of a free-head pile.

The passive resistance of soil in front of the footing shall be neglected.

Contractors shall be required to replace any disturbed soil or fill voids created during driving of the pile with compacted granular material.

The final design of laterally loaded vertical piles (i.e., no batter piles in the pile pattern) shall be based on the results of COM624P computer analyses (see Wang and Reese, (1993); and Reese, (1984)), LPILE 5.0 (see ENSOFT, Inc. 2004 for LPILE 5.0), or other methods of analysis (e.g., Borden and Gabr (1987), for the case of a sloping ground surface), if approved by the Chief Bridge Engineer, which account for the effects of soil/rock-structure interaction between the pile and ground. Other methods of analysis to evaluate the nominal horizontal resistance or deflection of laterally loaded piles may be used for preliminary design only as a means to determine approximate shaft dimensions.

Design of axially- and laterally-loaded piles using COM624P, Wang and Reese (1993), or LPILE 5.0 shall be performed according to the following steps (for integral abutments, refer to Appendix G):

1. Select a preliminary pile section based on settlement and axial load considerations (D10.7.2.3

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programs.

C10.7.3.12.2

The negative moment used to model partial pile-head fixity acts in a direction opposite that of the actual moment.

The degree of pile-head fixity is a function primarily of the pile-head embedment into the pile footing. Based on analytical studies and full-scale load test results, an embedment length of two to four pile diameters is required to provide full fixity, Shahawy and Issa, (1992); Castilla, et al, (1984).

The lateral loading of pile groups containing vertical and/or batter piles may be analyzed using soil structure methods of analysis, such as Reese, et al, (1994) and Poulos and Davis (1980).

When this analysis is performed, the loads are factored since the strength limit state is under consideration, but the resistances as represented by the *P-y* curves are not factored since they already represent the ultimate condition.

COM624P or LPILE 5.0 requires the engineer to make a decision as to whether the applied loading is static or cyclic in nature. This decision affects the P-y curves generated internally by the program in determining the level of soil resistance. The following guideline is suggested for typical applications encountered on Department projects:

> When the predominate lateral load on the pile/caisson is sustained and non-transient in nature (e.g. earth pressure on retaining walls, wingwalls and abutments), use static soil modulii and static loading condition;
>
> When the predominate lateral load on the pile/caisson is transient in nature (e.g. wind, centrifugal and braking loads on piers), use cyclic soil modulii and cyclic loading condition.

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and D10.7.3).

2. Using factored axial and lateral loads, compute the maximum groundline lateral deflection and the maximum factored moment in the pile section using COM624P or LPILE 5.0.

3. If the groundline lateral deflection exceeds 25 mm {1 in.} or the maximum factored moment exceeds the factored moment resistance of the pile obtained from D5.13.4 and A5.13.4, and D6.15P, select a new trial section, and repeat Step 2.

4. If neither the groundline lateral deflection nor the factored moment criteria in Step 3 is exceeded, compute the maximum groundline lateral deflection of the pile for the service limit state using COM624P or LPILE 5.0.

5. If the groundline lateral deflection exceeds 13 mm {1/2 in.}, select a new trial section and repeat Step 2.

For all COM624P or LPILE 5.0 analyses, lateral load deflection relationships used to determine deflections should be unfactored, whether input or default relationships are used.

The minimum penetration of the piles below ground (see D10.7.6) required in the contract should be established such that fixity is obtained. For this determination, the loads applied to the pile are factored as specified in Section 3, and a soil resistance factor of 1.0 shall be used as specified in Table D10.5.5.2.3-1.

If fixity cannot be obtained, additional piles should be added, larger diameter piles used if feasible to drive them to the required depth, or a wider spacing of piles in the group should be considered to provide the necessary lateral resistance. Batter piles may be added to provide the lateral resistance needed, unless downdrag is anticipated. If downdrag is anticipated, batter piles should not be used. The design procedure, if fixity cannot be obtained, should take into consideration the lack of fixity of the pile.

Lateral resistance of single piles may be determined by static load test. If a static lateral load test is to be performed, it shall follow the procedures specified in ASTM D 3966.

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The strength limit state for lateral resistance is only structural (see Sections 5 and 6 for structural limit state design requirements), though the determination of pile fixity is the result of soil-structure interaction. A failure of the soil does not occur; the soil will continue to displace at constant or slightly increasing resistance. Failure occurs when the pile reaches the structural limit state, and this limit state is reached, in the general case, when the nominal combined bending and axial resistance is reached.

For information on analysis and interpretation of load tests, see D10.7.2.4.

Refer to Publication 15A, Compilation of Pile Load Test and Wave Equation Analysis (1989).

10.7.3.12.3 Group Lateral Load Resistance

Laterally loaded pile groups containing battered piles shall be designed assuming that all lateral load is resisted by the horizontal component of the axial resistance of the battered piles. Laterally loaded pile groups containing only vertical piles shall be designed based on lateral load analysis of individual vertical piles in accordance with D10.7.3.8.2, assuming that the lateral load on the pile group is evenly

C10.7.3.12.3

Driven piles in a group are considered to act individually when the center-to-center spacing is greater than 3B in the direction perpendicular to the applied lateral load, and greater than or equal to 8B parallel to the direction of loading. For the case of closely-spaced piles in a group, the interaction behavior is typically accounted for indirectly, using empirical procedures (e.g., Reese, et al, (1994) and

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distributed among the piles. The distribution of vertical loads within both mixed and all vertical pile groups shall be determined based on elastic theory in accordance with the Department's "Users Manual for Computer Program, LRFD, Abutment and Retaining Wall Design".

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Prakash and Sharma (1990)). One such procedure assumes a reduction in the coefficient of lateral subgrade reaction for a pile in a group from that of a single pile, using the following ratios, Prakash and Sharma (1990):

Table C10.7.3.12-1 - Ratio of Lateral Resistance of Pile in Group to Lateral Resistance of Single Pile

Center-to-Center Shaft Spacing	Ratio of Lateral Resistance of Pile in Group to Single Pile	
	Cohesive Soil	Cohesionless Soil
8D	1.00	1.00
6D	0.65	0.70
4D	0.50	0.60
3D	0.40	0.50

For typical Department designs, in which the center-to-center spacing is 3B or greater and the lateral displacement is limited to 25 mm {1 in.} at the strength limit state, the factors in Table C1 need not be applied because there is no significant stress overlap between adjacent piles for this condition. Reductions of the magnitudes indicated in Table C1 are consistent with displacements at passive soil failure (i.e., about 75 mm to 125 mm {3 in. to 5 in.}).

10.7.3.13 PILE STRUCTURAL RESISTANCE

10.7.3.13.1 Steel Piles

The nominal compressive resistance in the structural limit state for piles loaded in compression shall be as specified in A6.9.4.1 for noncomposite piles and A6.9.5.1 and D6.9.5.1 for composite piles. If the pile is fully embedded, λ shall be taken as 0.

The nominal axial resistance of horizontally unsupported noncomposite piles that extend above the ground surface in air or water shall be determined from Eqs. A6.9.4.1-1 or A6.9.4.1-2. The nominal axial resistance of horizontally unsupported composite piles that extend above the ground surface in air or water shall be determined from Eqs. A6.9.5.1-1 or A6.9.5.1-2.

The effective length of laterally unsupported piles should be determined based on the provisions in D10.7.3.13.4.

The resistance factors for the compression limit state are specified in D6.5.4.2.

C10.7.3.13.1

Composite members refer to steel pipe piles that are filled with concrete.

The effective length given in D10.7.3.13.4 is an empirical approach to determining effective length. Computer methods are now available that can determine the axial resistance of a laterally unsupported compression member using a P-Δ analysis that includes a numerical representation of the lateral soil resistance (*Williams et al., 2003*). These methods are preferred over the empirical approach in D10.7.3.13.4.

10.7.3.13.2 Concrete Piles

The nominal axial compression resistance for concrete

C10.7.3.13.2

A5.7.4 and D5.7.4 include specified limits on

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piles and prestressed concrete piles shall be as specified in A5.7.4.4 and D5.7.4.4.

The nominal axial compression resistance for concrete piles that are laterally unsupported in air or water shall be determined using the procedures given in A5.7.4.3 D5.7.4.3, and A4.5.3.2. The effective length of laterally unsupported piles should be determined based on the provisions in D10.7.3.13.4.

The resistance factor for the compression limit state for concrete piles shall be that given in D5.5.4.2.1 for concrete loaded in axial compression.

10.7.3.13.3 Timber Piles

The nominal axial compression resistance for timber piles shall be as specified in A8.8.2. The methods presented there include both laterally supported and laterally unsupported members.

The effective length of laterally unsupported piles should be determined based on the provisions in D10.7.3.13.4.

10.7.3.13.4 Buckling and Lateral Stability

In evaluating stability, the effective length of the pile shall be equal to the laterally unsupported length, plus an embedded depth to fixity.

The potential for buckling of unsupported pile lengths and the determination of stability under lateral loading should be evaluated by methods that consider soil-structure interaction as specified in D10.7.3.12.

For preliminary design, the depth to fixity below the ground, in mm {ft}, may be taken as:

- For clays:

$$1.4\ [E_p\ I_w / E_s\ ]^{0.25} \qquad (10.7.3.13.4\text{-}1)$$

- For sands:

$$1.8\ [E_p\ I_w / n_h\ ]^{0.2} \qquad (10.7.3.13.4\text{-}2)$$

where:

E_p = modulus of elasticity of pile (MPa){ksi}

I_w = weak axis moment of inertia for pile (mm^4) {ft.^{4}}

E_s = soil modulus for clays = 0.67 S_u (MPa) { = 0.465 S_u ksi}

S_u = undrained shear strength of clays (MPa) {ksf}

n_h = rate of increase of soil modulus with depth for sands as specified in Table DC10.4.6.3-2

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longitudinal reinforcement, spirals and ties. Methods are given for determining nominal axial compression resistance but they do not include the nominal axial compression resistance of prestressed members. AC5.7.4.1 notes that compression members are usually prestressed only where they are subjected to high levels of flexure. Therefore, a method of determining nominal axial compression resistance is not given.

A5.7.4.5 specifically permits an analysis based on equilibrium and strain compatibility. Methods are also available for performing a stability analysis (*Williams et al., 2003*).

C10.7.3.13.3

A8.5.2.3 requires that a reduction factor for long term loads of 0.75 be multiplied times the resistance factor for Strength Load Combination IV.

C10.7.3.13.4

This procedure is taken from Davisson and Robinson (*1965*) and should only be used for preliminary design.

In Eqs. 1 and 2, the loading condition has been assumed to be axial load only, and the piles are assumed to be fixed at their ends. Because the equations give depth to fixity from the ground line, the Engineer must determine the boundary conditions at the top of the pile to determine the total unbraced length of the pile. If other loading or pile tip conditions exist, see Davisson and Robinson (*1965*).

The effect of pile spacing on the soil modulus has been studied by Prakash and Sharma (*1990*), who found that, at pile spacings greater than 8 times the pile width, neighboring piles have no effect on the soil modulus or buckling resistance. However, at a pile spacing of 3 times the pile width, the effective soil modulus is reduced to 25 percent of the value applicable to a single pile. For intermediate spacings, modulus values may be estimated by interpolation.

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(MPa/mm) {ksi/ft.}

For a pile group composed of only vertical piles, which is subject to lateral loads, the pile structural analysis shall include explicit consideration of soil-structure interaction using a COM624P or LPILE 5.0 analysis, as specified in D10.7.3.12.

10.7.3.13.5 Point Attachments on Steel H-Piles

If pile penetration through cobbles, boulders, debris fill or obstructions is anticipated and for all piles driven to bedrock, pile tips shall be reinforced with structural shapes or with prefabricated cast steel points. Cast steel points shall meet the requirements of ASTM A 27/A 27M.

See Standard Drawing BC-757M and Publication 408, Section 1005, for details.

10.7.4 Extreme Event Limit State

The provisions of D10.5.5.3 shall apply.

For the applicable factored loads, including those specified in D10.7.1.6, for each extreme event limit state, the pile foundations shall be designed to have adequate factored axial and lateral resistance. For seismic design, all soil within and above the liquefiable zone, if the soil is liquefiable, shall not be considered to contribute axial compressive resistance. Downdrag resulting from liquefaction induced settlement shall be determined as specified in A3.11.8 and included in the loads applied to the foundation. Static downdrag loads should not be combined with seismic downdrag loads due to liquefaction.

The pile foundation shall also be designed to resist the horizontal force resulting from lateral spreading, if applicable, or the liquefiable soil shall be improved to prevent liquefaction and lateral spreading. For lateral soil resistance of the pile foundation, the *P-y* curve soil parameters should be reduced to account for liquefaction. To determine the amount of reduction, the duration of strong shaking and the ability of the soil to fully develop a liquefied condition during the period of strong shaking should be considered.

When designing for scour, the pile foundation design shall be conducted as described in D10.7.3.6, except that the check flood and resistance factors consistent with D10.5.5.3.2 shall be used.

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C10.7.4

See DC10.5.5.3.

10.7.5 Corrosion and Deterioration

10.7.5.1 GENERAL

The effects of corrosion and deterioration from environmental conditions shall be considered in the selection of the pile type and in the determination of the required pile cross-section.

C10.7.5.1

More detail on design for corrosion is contained in Hannigan et al. (*2005*).

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10.7.5.2 CORROSION AND DETERIORATION

Evaluation of protective measures shall include consideration of the soil and groundwater conditions surrounding the pile and the loading characteristics of the pile. The evaluation of protective measures shall be performed for each situation based on the level of deterioration anticipated, the practicality of applying a particular protective measure and cost.

See D10.7.5.7 for the definition of conditions indicative of potentially corrosive soil and groundwater.

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C10.7.5.2

Soil and groundwater characteristics, such as pH, resistivity, sulfate content, chloride content and bacteria level are necessary for determining the deterioration potential of a pile.

Methods of pile deterioration prevention should be compared for practicality and cost. For a small job in which piles are driven through a layer of mildly corrosive soil, the deduction of 1.5 mm {1/16 in.} in determining the pile capacity may be practical and the cost of excess material in the non corrosive area insignificant. On a large job, applying a protective coating to only that section of the pile exposed to the corrosive layer may result in cost savings. In some cases, the choice of pile type (i.e., steel vs. concrete) may be made on the basis of deterioration considerations.

High velocity water flows containing suspended sediments can abrade piles and remove protective coatings above the scour depth.

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10.7.5.3 STEEL PILES

The following measures shall be compared for protection of steel piles against deterioration by corrosion.

a. Deduct 1.5 mm {1/16 in.} (minimum) from the exposed surface of the pile used to compute section capacity if anticipated corrosion losses will be less than 1.5 mm {1/16 in.}.

b. Apply a coating which has good dielectric strength, is resistant to abrasive forces during driving, and has a proven service in the type of corrosive environment anticipated. Electrostatically applied epoxies have proven to be effective in many cases.

c. Place a minimum 100 mm {4 in.} thick concrete encasement jacket around the pile of the same quality concrete as that recommended for concrete

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A steel pile foundation design should consider that steel piles may be subject to corrosion, particularly in fill soils, low ph soils (acidic) and marine environments. A field electric resistivity survey, or resistivity testing and ph testing of soil and groundwater samples should be used to evaluate the corrosion potential.

Corrosion rates for piles in undisturbed soil are generally negligible, Schwerdtfeger and Romanoff (1972). In fills, the corrosion rates range from negligible to severe (averages range from 0.05 mm to 0.2 mm {2 to 8 mils} per year) depending on the various factors. Corrosion usually takes place in the form of pitting which is not as serious as a uniform reduction in thickness over a considerable area of the pile. The rate of corrosion slows up considerably as the steel takes on a film of corrosion products which tends to protect the steel from further corrosion. Where steel piles are driven into sand, conditions are particularly favorable to the formation of an impervious, insoluble coating of ferrosilicate as soon as the steel corrodes slightly, U. S. Steel (1986).

Steel piles in fresh water have a slow initial corrosion rate of 0.02 mm to 0.08 mm {1 to 3 mils} per year, decreasing with time as a protective coating forms, U. S. Steel (1986). The corrosion rate of steel piles in polluted water is highly variable.

Although coal-tar epoxies are commonly used on piles, the Department has experienced some problems with their use. Plain epoxy coatings have proven to be more successful. Other coatings, such as metalized zinc and aluminum with top coats, are usually more expensive, but may be applicable in some cases. Coatings, such as vinyls, epoxies, urethanes and coal tar epoxies have been found to be effective in controlling corrosion in water.

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piles. The portion of the steel embedded in the bottom few feet of the concrete jacket shall be coated with an electrostatically applied epoxy.

d. Install a cathodic protection system, and coat the pile with a coating resistant to cathodic disbondment to reduce cost of the system. Piles exposed to unpolluted fresh water shall be electrically connected so that a cathodic protection system can be easily installed if corrosion is discovered during future inspections.

10.7.5.4 CONCRETE PILES

In any corrosive medium, a dense, impervious concrete shall be used. The following measures shall be taken on all concrete piles used in corrosive environments:

- Minimum concrete cover as follows:

  (1) Cast-in-place reinforced concrete, 100 mm {4 in.}

  (2) Precast reinforced concrete, 75 mm {3 in.}

  (3) Prestressed concrete - Prestressed strands, 65 mm {2 1/2 in.}; secondary reinforcement, 40 mm {1 1/2 in.}

- Maximum water/cement ratio of 0.45 (by weight)

- Air entrainment

- No concrete additives containing chlorides

The following protective measures may also be required in particular cases.

- Sulfate resistant cement as follows:

Water Soluble Sulfate in Soil (%)	Sulfate in Water (ppm)	Cement Type
0.10 - 0.20	150 - 1,500	II
0.20 - 2.00	1,500 - 10,000	V
> 2.00	> 10,000	V plus Pozzolan

- Epoxy-coated reinforcement

- Cathodic protection with electrical continuity between all reinforcement. Cathodic protection should not be used for prestressed piles.

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Cathodic protection systems are very expensive due to required maintenance. These systems, in combination with coatings, are used in buried pipeline applications where the effects of small corrosion pits at defects in the coating can lead to leaks. In piles, however, small pitting is not as critical, and a good coating without cathodic protection is generally effective in preventing significant uniform corrosion loss on the pile section.

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A concrete pile foundation design should consider that deterioration of concrete piles can occur due to sulfates in soil, groundwater or sea water, chlorides in soils and chemical wastes, acidic groundwater and organic acids. Laboratory testing of soil and groundwater samples for sulfates and ph is usually sufficient to assess pile deterioration potential. A full chemical analysis of soil and groundwater samples is recommended when chemical wastes are suspected.

Sulfates are present in groundwater in fills containing blast furnace slag, cinders, or pyritic shale. Sulfates react with chemicals present in the concrete, such as hydrated lime and gypsum. These reactions result in an increase in solid volume, with a subsequent spalling of concrete.

The principle cause of reinforcing steel corrosion is chloride ion. Good corrosion protection in concrete piles can be accomplished by limiting the amount of chloride ion in the concrete mix. ACI Committee 222 has suggested the following limits for chloride ion in concrete prior to service exposure, expressed as a percent by weight of cement:

- Prestressed concrete 0.06%

- Reinforced concrete exposed to chloride in service 0.10%

- Reinforced concrete in a moist environment, but not exposed to chloride 0.15%

The use of accelerating admixtures containing calcium chloride is discouraged.

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10.7.5.5 TIMBER PILES

Untreated timber piles shall be used only for temporary construction. Timber piles for permanent construction shall be protected as follows:

a. Treatment with creosote in accordance with Publication 408, Section 1005, or

b. Concrete jacketing, in accordance with recommendations for concrete piles

COMMENTARY

C10.7.5.5

The principle causes of deterioration of timber piles are decay, insect attack, and marine-borer attack. Deterioration of timber piles typically occurs in areas where oxygen and moisture are present. Therefore, piles driven below the permanent groundwater table are generally unaffected by decay and insect attack. Treatment with creosote is the most common method of protecting timber piles. The advantages of creosote as a preservative are its high toxicity to wood-destroying organisms, relative insolubility, availability, good depth of penetration and good performance record.

10.7.5.6 PRELIMINARY TESTING

If driven pile foundations are anticipated, the soils investigation shall provide the following minimum information to determine pile deterioration potential:

- Soil pH, sulfate content in soil and groundwater and moisture content
- General soil profile, including type, variation, depth and layering of fill and undisturbed natural soils, and groundwater level
- Previous land use
- Soil resistivity (laboratory test on soil samples) - If evaluation of data with respect to criteria in D10.7.5.7 indicates a potential corrosion problem, a field resistivity survey may be performed.

If piles extend through open water, the water shall be tested for chlorides, sulfates, bacteria and pH, and its velocity shall be measured.

10.7.5.7 CORROSIVE ENVIRONMENTS

Conditions which are indicative of potentially corrosive soil and groundwater and require consideration of protective measures:

- Resistivity less than 2000 ohm cm in soil
- Resistivity between 2000 and 5000 ohm-cm and combined with:
    - sulfate concentration greater than 200 ppm, or
    - chloride concentration greater than 100 ppm
- pH less than 5.5
- pH between 5.5 and 8.5 in soils with high organic

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content

- Sulfate concentration greater than 1000ppm in soil or greater than 150 ppm in groundwater
- Landfills and cinder fills
- Soils subject to mine or industrial drainage
- Mixtures of high resistivity soils and low resistivity high-alkaline soils

Conditions which are low in corrosion potential and which generally do not require protective measures include:

- Undisturbed natural soils with no free draining layers, regardless of conditions, noted above as indications of high corrosion potential
- pH greater than 5.5 with no organic content
- Soils with resistivities greater than 5000 ohm cm and uniform in profile
- Well-aerated loose soils of uniform composition (i.e., sand)

Water shall be considered corrosive if it contains any of the following:

- Chloride content greater than 1000 ppm
- Sulfate content greater than 150 ppm
- Mine or industrial runoff
- High organic content
- pH less than 5.5

Water with high velocity is generally more damaging than standing water.

Piles exposed to air containing sulfur dioxide, chlorine concentrations, or other pollutants require protection against deterioration.

10.7.5.8 STRAY CURRENTS

Steel and concrete piles located near sources of direct currents (i.e., electric transit systems, welding shops, cathodic protection systems) may be subject to damage from stray currents. To protect against stray current damage, steel piles shall be electrically connected and grounded to the current source. Concrete piles shall be similarly grounded with electrical continuity between all reinforcement. The effects of stray currents on prestressed piles can lead to pile failure and prestressed piles should not

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be used in areas of potential stray currents.

10.7.6 Pile Penetration

The minimum pile penetration, if required for the particular site conditions and loading, shall be based on the maximum depth needed to meet the following requirements as applicable:

- Single and pile group settlement (service limit state)
- Lateral deflection (service limit state)
- Uplift (strength limit state)
- Depth into bearing soils needed to resist downdrag loads resulting from static consolidation stresses on soft soil or downdrag loads due to liquefaction (strength and extreme event limit state, respectively)
- Depth into bearing soils needed to provide adequate pile axial (compression and uplift) and lateral resistance after scour (strength and extreme event limit states)
- Nominal soil shear resistance and fixity for resisting the applied lateral loads to the foundation (strength limit state)
- Axial uplift, and lateral resistance to resist extreme event limit state loads

The contract documents should indicate the minimum pile penetration, if applicable, as determined above. The contract documents should also include the required nominal axial compressive resistance, R_{ndr} as specified in D10.7.7 and the method by which this resistance will be verified, if applicable, such that the resistance factor(s) used for design are consistent with the construction field verification methods of nominal axial compressive pile resistance.

In general, unless refusal is encountered, the design penetration for any pile should be not less than 3000 mm {10.0 ft.} into hard cohesive or dense granular material and not less than 6000 mm {20.0 ft.} into soft cohesive or loose granular material.

Unless refusal is encountered, piles for trestle or pile bents shall penetrate a distance equal to at least one-third the unsupported length of the pile.

Piling used to penetrate a soft or loose upper stratum overlying a hard or firm stratum, shall penetrate the firm stratum by a distance sufficient to limit movement of the piles and attain sufficient bearing capacities.

Penetration shall be controlled so that pile damage during driving is avoided. A maximum blow count limited by driving stresses shall be determined using a Wave Equation analysis (see D10.7.3.8.4 and DC10.7.3.8.4). The

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C10.7.6

A minimum pile penetration should only be specified if necessary to ensure that all of the applicable limit states are met. A minimum pile penetration should not be specified solely to meet axial compression resistance, i.e., bearing, unless field verification of the pile nominal bearing resistance is not performed as described in D10.7.3.8.

Driving points or shoes may be necessary to achieve penetration or to provide adequate lateral tip restraint.

The generally acceptable minimum length of 3000 mm {10 ft.} is based upon driveability and lateral resistance

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maximum blow count shall not be exceeded.

Although the minimum design length of piles in hard or dense soils is generally 3000 mm {10 ft.}, reduced penetrations as low as 1800 mm {6 ft.} may be permitted for special conditions. The minimum penetration length of pile shall be 3000 mm {10 ft.} unless otherwise approved by the Chief Bridge Engineer for a specific project.

For friction and end bearing piles, the designer shall require predrilling through the resisting soil layer to the bottom of the estimated scour depth prior to the pile driving.

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concerns. As pile length decreases, the potential for damage during driving increases, particularly for point bearing and end bearing piles. Because very short piles are nearly always point bearing or end bearing piles, damage potential is a prime consideration. Lateral resistance is severely lessened in short piles due to reduced tip fixity and decreased soil resistance adjacent to the pile.

Because of these concerns, the designer shall conduct a thorough analysis of driveability (10.7.8) and lateral load resistance (D10.7.2.4 and D10.7.3.1.2) during design to evaluate the feasibility of short piles for a particular application.

Predrilling will reduce side resistance through the scour zone during pile driving so as to reduce the probability of obtaining a false indication of adequate pile capacity.

10.7.7 Determination of R_{ndr} Used to Establish Contract Driving Criteria for Bearing

The value of R_{ndr} used for the construction of the pile foundation to establish the driving criteria to obtain the design bearing resistance shall be the value that meets or exceeds the following limit states, as applicable:

- Strength limit state compression resistance specified in D10.7.3.8
- Strength limit state compression resistance, including downdrag specified in D10.7.3.7
- Strength limit state compression resistance, accounting for scour specified in D10.7.3.6
- Extreme event limit state compression resistance for seismic specified in D10.7.4
- Extreme event limit state compression resistance for scour specified in D10.7.4

10.7.8 Pile Driveability

10.7.8.1 GENERAL

Driveability shall be optimized using the procedure specified in D10.7.3.8.4 when the design resistance is achieved with the least expensive pile and most efficient hammer sized for maximum set without damaging the pile.

C10.7.8.1

Because dynamic resistance to pile penetration must be overcome during driving, but does not influence subsequent static capacity, the highest stress levels experienced by a pile will occur during driving.

Optimized pile design generally results in piles being driven at near-maximum permissible stresses to the driving criteria for the various load carrying mechanisms. Some exceptions include the cases of friction/displacement piles where pile length reductions in conjunction with less than maximum possible loads may provide an economically attractive combination. Such cases are relatively rare, however, and are difficult to anticipate during the design

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The establishment of the installation criteria for driven piles should include a drivability analysis. Except as specified herein, the drivability analysis shall be performed by the Engineer using a wave equation analysis, and the driving stresses (σ_{dr}) anywhere in the pile determined from the analysis shall be less than the limits contained herein:

The following permissible driving stresses, as determined by wave equation analysis at 20 blows per 25 mm {1 in.}, are specified for particular pile types.

a. Driven Steel Piles Driven without Mandrel

- $\sigma_{dr} = \varphi_{da} f_y$ for f_y = 250 MPa {36 ksi}
- $\sigma_{dr} = 0.8\varphi_{da} f_y$ for f_y = 345 MPa {50 ksi} (for point bearing piles provided the piles have verification of material strength greater than or equal to 345 MPa {50 ksi}

where:

f_y = yield strength for steel (MPa) {ksi}

φ_{da} = 1.0

b. Steel Piles Driven with Mandrel - Driving stresses will be controlled by shell damage, dependent on geologic conditions and obstructions

c. Concrete Piles

Compression:

$$\sigma_{dr} = \varphi_{da}\left(0.85 f_c' - f_{pe}\right) \qquad (10.7.8.1\text{-}1)$$

Tension:

Metric Units:

$$\sigma_{dr} = \varphi_{da}\left(0.25\sqrt{f_c'} + f_{pe}\right) \qquad (10.7.8.1\text{-}2)$$

U.S. Customary Units:

$$\sigma_{dr} = \varphi_{da}\left(0.095\sqrt{f_c'} + f_{pe}\right)$$

where:

f'_c = compressive structural design strength of concrete at 28 days (MPa) {ksi}

f_{pe} = concrete compression stress due to

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phase of a project without field testing.

Wave equation analyses should be conducted during design using a range of likely hammer/pile combinations, considering the soil and installation conditions at the foundation site. See D10.7.3.8.4 for additional considerations for conducting wave equation analyses. These analyses should be used to assess feasibility of the proposed foundation system and to establish installation criteria with regard to driving stresses to limit driving stresses to acceptable levels. For routine pile installation applications, e.g., smaller diameter, low nominal resistance piles, the development of installation criteria with regard to the limitation of driving stresses, e.g., minimum or maximum ram weight, hammer size, maximum acceptable driving resistance, etc., may be based on local experience, rather than conducting a detailed wave equation analysis that is project specific. Local experience could include previous drivability analysis results and actual pile driving experience that are applicable to the project specific situation at hand. Otherwise, a project specific drivability study should be conducted.

Drivability analyses may also be conducted as part of the project construction phase. When conducted during the construction phase, the drivability analysis shall be conducted using the contractor's proposed driving system. This information should be supplied by the contractor. This drivability analysis should be used to determine if the contractor's proposed driving system is capable of driving the pile to the maximum resistance anticipated without exceeding the factored structural resistance available, i.e., σ_{dr}.

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prestressing after all losses (MPa) {ksi}

For reinforced concrete piles (non-prestressed piles), f_{pe} equals zero.

d. Timber Piles - See Publication 408, Section 1005

For routine pile installation applications where significant local experience can be applied to keep the risk of pile installation problems low, a project specific drivability analysis using the wave equation may be waived.

This drivability analysis shall be based on the maximum driving resistance needed:

- To obtain minimum penetration requirements specified in D10.7.6,
- To overcome resistance of soil that cannot be counted upon to provide axial or lateral resistance throughout the design life of the structure, e.g., material subject to scour, or material subject to downdrag, and
- To obtain the required nominal bearing resistance.

Maximum allowable stress values recommended by various engineers, manufacturers and contractors may differ considerably, sometimes by a factor of 2. The values of maximum permissible driving stress presented reflect, where possible, past experience in the Commonwealth regarding driving stresses, and, in general, tend to fall in the median area within the range of recommended values for each pile type.

The designer should be aware that under certain conditions pile driveability tends to be controlled by phenomena not accurately predicted by wave equation analysis of driving stress. Piles driven through extremely loose or soft soil to point bearing on competent rock may experience localized buckling due to lack of lateral support and strong wave reflection from the pile tip. Also, piles driven into or through buried debris or boulders may experience damage due to high localized stresses where obstructions are first encountered. Refer to D10.7.8.1 for additional discussion regarding these conditions.

10.7.8.2 EVALUATION OF DRIVEABILITY

Preliminary evaluation of driveability shall be made by wave equation analysis using the procedure specified in D10.7.3.8.4. Driving stresses, transmitted hammer energy and developed pile capacity shall be verified during test pile construction. Changes in driving equipment, procedures, or driving criteria may be directed by the Department if monitored driving parameters vary appreciably from those assumed in the preliminary analyses.

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Wave equation analysis provides the most accurate evaluation of driving stresses, hammer energy and static pile strength available to the designer prior to the actual field testing and/or dynamic monitoring. Although appreciable predictive inaccuracies may occur (depending upon the particular program used, accuracy of assumed hammer efficiencies, damping factors) in wave equation analyses, they generally provide suitable information for near-optimization of pile driveability. The most common cause

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of inaccuracy in predicting pile driveability by wave equation analysis is use of inaccurate hammer efficiencies. Currently available wave equation programs incorporate optimistic values of hammer efficiency (typical values are 95% for diesel and 80% for air/steam hammers), which in turn lead to higher predicted transmitted hammer energies and driving stresses than are observed in the field. Use of more realistic hammer efficiencies in wave equation analyses will provide more accurate evaluation of pile driveability, and is, thus, recommended. A FHWA report, Vanikar (1985), presents average measured efficiencies of 72% for diesel, 67% for single-acting air/steam and 50% for double-acting air/steam hammers.

A driveability analysis begins with the selection of a pile type (or types) which is suitable for the soil and rock conditions present at the job site(s), and which has a structural capacity (at the time of driving) approximately 10 to 20% greater than the design capacity. A hammer is then selected which should develop optimum driving stresses in the pile. As a starting point, the following guidelines may be used to select a minimum rated hammer energy for preliminary analysis:

(a) Steel H-Piles: 16,000 J {12,000 ft·lbs} times the cross-sectional area (in square meters) {in square inches} of the pile.

(b) Concrete Piles (solid section): 16,000 J {12,000 ft·lbs} times the pile width (in meters) {in inches}.

(c) Timber Piles: 9500 J {7000 ft·lbs}

After choosing a pile/hammer combination, an initial wave equation analysis should be performed in accordance with D10.7.3.8.4 to evaluate the feasibility of the preliminary pile/hammer combination. The results of the initial wave equation analysis will probably indicate that pile driveability is not optimized. Successive analyses are then conducted altering the relevant parameters as required to meet the criteria for optimized driveability.

10.7.8.3 DRIVING CRITERIA

Driving shall be in accordance with Publication 408, Section 1005. Test piles shall be driven to absolute refusal. If a test pile does not achieve absolute refusal within 600 mm {2 ft.} of the estimated tip elevation, driving should be stopped and the Chief Bridge Engineer shall be contacted. If absolute refusal is not obtained, special studies may be required during construction. Past experience, actual driving data, wave equation analyses and possible load tests shall be correlated to obtain driving parameters.

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10.7.8.4 GEOLOGIC CONDITIONS AFFECTING DRIVEABILITY

The presence of boulders or construction debris, buried concrete slabs, or hard layers (above a required bearing strata) may either damage piles or hinder advancement to a required elevation or bearing stratum. Such geologic conditions shall be disclosed by the subsurface investigation. Measures to increase pile driveability shall be taken in accordance with D10.7.8.5 appropriate to the location, size, or thickness of the obstructing feature(s).

COMMENTARY

C10.7.8.4

In most cases, geologic conditions which will influence pile driveability are readily discernible from boring records; however, in some instances subsurface obstructions are more difficult to recognize. For example, the presence of boulders immediately overlying bedrock may be misinterpreted as a layer of fractured rock overlying sound rock. The presence of buried foundation slabs may be missed, or misinterpreted as cobbles, boulders, etc., if the exploratory drilling personnel and recovered samples are not closely monitored.

10.7.8.5 METHODS OF INCREASING DRIVEABILITY

Increasing driveability is generally achieved by either increasing the strength or stiffness of the pile (or portions thereof) or by penetrating an obstruction prior to pile installation. Steel tip protection shall be used to increase the strength of pile tips where moderate or severe damage potential conditions exist. Tip protection shall be required for all point bearing and end bearing piles driven into bedrock. Where piles must penetrate a dense layer of appreciable thickness to obtain a specified tip elevation or bearing stratum, the layer can be penetrated by jetting during pile installation or by predrilling a pilot hole prior to pile installation.

C10.7.8.5

Increasing the stiffness of the pile section is the most common method of increasing driveability when wave equation analysis indicates overstressing will occur with a particular hammer/pile combination. When potentially damage-causing obstructions such as boulders, foundation slabs, or construction debris must be penetrated, steel tip protection will generally provide adequate protection. However, predrilling may be necessary in cases in which obstructions are unusually massive or located relatively near the ground surface where piling will have little lateral support when they are encountered. Because extensive predrilling through rock/obstructions adds considerably to foundation expense, consideration should be given to eliminating predrilling in conjunction with lower pile strengths to allow for some damage or using drilled shaft foundations.

10.7.9 Test Piles

Test piles should be driven at several locations on the site to establish order length. If dynamic measurements are not taken, these test piles should be driven after the driving criteria have been established.

If dynamic measurements during driving are taken, both order lengths and driving criteria should be established after the test pile(s) are driven. Dynamic measurements obtained during test pile driving, signal matching analyses, and wave equation analyses should be used to determine the driving criteria (bearing requirements) as specified in D10.7.3.8.2, D10.7.3.8.3, and D10.7.3.8.4.

When piles are specified, each substructure unit must have at least one test pile and no less than one test pile for each 15 000 mm {50 ft.} of footing length. Additional test piles may be specified if test borings indicate that irregular pile lengths could be anticipated. In special cases in which the soil conditions are relatively uniform, one test pile for each 30 000 mm {100 ft.} of footing length will be acceptable. The specified length of test piles shall be the same as the estimated lengths of bearing piles when driven

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Test piles are sometimes known as Indicator Piles. It is common practice to drive test piles at the beginning of the project to establish pile order lengths and/or to evaluate site variability whether or not dynamic measurements are taken.

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to bedrock or dense end bearing strata. For friction piles, test pile lengths shall be based on driving to absolute refusal. Test piles may be used as bearing piles, if approved.

The lengths of test piles shall be based on geotechnical data and analysis and/or previous experience (e.g., actual driving and/or load test data), if available. See D10.7.8.3 for driving criteria.

10.7.10 Evaluation of Predetermined Tip Elevations

10.7.10.1 GENERAL

Predetermined tip elevations shall be determined in accordance with PP1.7.5 and Publication 408, Section 1005.

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C10.7.10.1

Predetermined tip elevations are determined after driving test and/or load test piles at the foundation location. These elevations, thus determined, reflect an estimate of the length of pile embedment required to develop the nominal axial pile resistance used in design. The pile embedment length is based on the load carrying mechanism (point-bearing, end-bearing, or friction) used to develop the resistance of the pile. Because of variations in soil and rock conditions existing across a project site, tip elevations of installed piles will vary from the predetermined tip elevation. It is, therefore, necessary to interpret the data obtained from the driving and/or load testing of piles, and to evaluate the acceptability of bearing piles attaining the specified driving criterion above the predetermined tip elevation.

Guidelines are presented below for evaluating predetermined tip elevations from test and load test piles and for evaluating bearing piles which do not attain the predetermined tip elevation. Because each piling project and the subsurface conditions encountered are unique, the following guidelines should be applied with an understanding of the special conditions relating to the particular projects.

(a) Point-Bearing Piles - Predetermined tip elevations for point bearing piles should be readily apparent by the behavior of the hammer and the observed driving resistance. The hammer will quickly approach optimum performance levels as the bearing layer is reached and may experience cylinder lift or other erratic behavior if refusal is achieved very quickly at full throttle.

If boulders are known or suspected to be present at a piling location, care shall be taken to ensure that a valid predetermined tip elevation is established. If the boulders are located close to the bearing stratum, it is often difficult to determine whether a pile has reached the bearing stratum or has encountered a boulder. The number of test and/or load test piles should be increased in areas where boulders overlie the bearing stratum. The engineer

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shall evaluate the driving and/or testing results to determine if bearing piles may be expected to penetrate the boulders and reach the bearing stratum. If a significant number of piles fail to penetrate the boulders, the engineer shall evaluate several factors before proceeding with piling installation. This may require reduction of the design resistance or revision of the driving criteria.

(b) End Bearing Piles - End bearing piles typically reach refusal more gradually than point bearing piles. End bearing piles are typically driven to rock through a relatively thin layer of dense soils or driven a relatively short distance into a layer of very dense soil or soft rock. In the case of end bearing piles driven to competent rock, the driving resistance will typically increase to refusal quickly upon reaching bedrock. In this case, the predetermined tip elevation usually reflects the estimated top of rock. End bearing piles driven into very dense soil layers or soft rock will exhibit driving resistances which increase gradually to refusal. In these cases, the predetermined tip elevation reflects the minimum penetration into the bearing stratum needed to develop the required resistance. If load testing is performed, the piles penetrating the least into the bearing stratum shall be tested. This will allow strength verification for piles representative of the least bearing stratum embedments to be anticipated during driving of bearing piles. If boulders are encountered above or within the bearing stratum, refer to the guidelines above for point bearing piles.

(c) Friction Piles - The driving resistance for friction piles increases gradually as the pile penetrates into the bearing stratum. Typically, the predetermined tip elevation reflects the elevation of the top of the bearing stratum minus the length of bearing stratum embedment at which test or load test piles achieve the required resistance.

Monitoring hammer performance during installation of test and/or load test piles provides a basis of comparison useful in evaluating piles which attain the specified driving criterion above the predetermined tip elevation. Thus, it is important that as much information as possible regarding hammer performance be obtained during the installation of test and load test piles.

10.7.10.2 EVALUATION OF BEARING PILES NOT REACHING THE PREDETERMINED TIP ELEVATION

The resistance of piles not reaching the predetermined

C10.7.10.2

Very few project sites exhibit such uniform soil and

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tip elevation shall be reevaluated based on available subsurface information and driving records.

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rock conditions, and few pile hammers operate at such uniform levels that all bearing piles driven at the site to a particular criterion will attain the predetermined tip elevation.

The following procedures can be used to evaluate the suitability of piles not reaching the predetermined tip elevation.

a. Point Bearing Piles

1. Review Subsurface Information - Review subsurface information to determine the variation in the top of rock elevation. The pile may be acceptable if the tip elevation is within the variation in rock elevations noted during the investigation. If boulders are present, the pile may be unacceptable, pending the results of additional considerations below.

2. Review Driving Record of Suspect Pile - The driving resistance of the suspect pile should be compared to the driving resistance of adjacent piles. If the driving resistance increased rapidly over the last few feet of driving for the suspect pile, as well as adjacent piles, the driving record of the suspect pile would indicate cause for special concern. If the suspect pile exhibited a gradually increasing driving resistance in contrast to adjacent piles, it is likely that the suspect pile did not reach the same bearing stratum.

3. Review Driving Records of Adjacent Piles - Check the driving records of nearby piles with regard to the tip elevations achieved. If a trend toward higher tip elevations has occurred in the direction of the suspect pile, the top of bearing stratum elevation may be increasing in elevation, such that the suspect pile may have reached the intended stratum.

4. Drive Additional Piles - It may be prudent to reserve judgment on the suspect pile until several piles have been driven adjacent to it in locations which place the suspect between piles previously driven and the additional piles. If the additional piles behave similarly to most previously driven piles, the suspect pile is likely not acceptable. If the additional piles compare closely to the suspect pile, the suspect pile may be acceptable.

b. End Bearing Piles

1. Review Subsurface Information - If appreciable variations in elevation of the bearing stratum are indicated, the suspect pile tip elevation may be reflecting such a variation. Where end bearing

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piles obtain a relatively large proportion of their axial resistance from skin friction in the lower portion of the pile, tip elevations may be sensitive to variations in the density or stiffness of the bearing stratum which may result in axial resistance development at lesser embedments (i.e., higher tip elevations).

2. Review Driving Record of Suspect Pile - Compare the driving resistance of the suspect pile with those of other piles. An end bearing pile usually exhibits relatively low driving resistances until the bearing stratum is reached. As the pile advances into the bearing stratum, driving resistance increases rapidly, which is an indicator of the top of bearing stratum elevation. If the driving resistance of the suspect pile increased at a higher elevation than adjacent piles, the suspect pile may have an embedment length in the bearing stratum similar to adjacent piles which attained the predetermined tip elevation.

3. Review Driving Records of Adjacent Piles - Check for a trend towards higher tip elevations.

4. Drive Additional Piles - Observe the behavior of several additional piles driven.

5. Check Hammer Performance - Review the records of hammer performance kept during driving of the suspect pile. If hammer performance (e.g., bounce chamber pressure, blow rate, stroke, etc., depending on hammer type and degree of instrumentation) was appreciably lower during driving of the suspect pile than during driving of adjacent piles, then the higher tip elevation may be a result of low energy input. The pile shall be redriven at optimum hammer performance to achieve the predetermined tip elevation.

6. Check Suspect Pile Axial Resistance by Dynamic Monitoring - If the suspect pile was monitored during initial driving, the estimated axial resistance may be compared to the estimated axial resistance of piles reaching the predetermined tip elevation. If the pile was not monitored when first driven, it may be monitored while being redriven. When monitoring piles being redriven, make certain that hammer performance has reached optimum levels before stopping redriving, since less than optimum hammer performance will likely be reflected by reduced nominal strength estimates (see D10.7.3.8.3d, Redriving). A favorable comparison between the nominal strength of the suspect pile and the strength of piles driven to the predetermined tip elevation indicates the

acceptability of the suspect pile.

7. Check Potential for Temporary Increase in Driving Resistance - When piles are driven into dense silts or glacial tills below the water table, or into soft shales, siltstones or claystones, the driving resistance may be temporarily increased due to the behavior of the bearing stratum. After a 24-hour period has elapsed following initial driving, driving resistances have been observed to decrease by half as piles are redriven. It is possible that a pile failing to achieve the predetermined tip elevation in soil or rock conditions, as described above, encountered temporary resistance, rather than actual bearing. Suspect piles may achieve the predetermined tip elevation if redriven a day or more following initial driving.

c. Friction Piles

1. Check Hammer Performance - Compare the level of performance to that observed during test pile driving and driving of nearby bearing piles. If the suspect pile was installed at lower hammer energies, it should be redriven at optimum energy to attain the predetermined tip elevation. Embedment lengths of friction piles are often very sensitive to fluctuations in hammer performance.

2. Review Subsurface Information - Where the bearing stratum is encountered at a higher elevation than at previous piling locations, or where a greater density was indicated during subsurface investigation, friction piles may be expected to attain a specified driving criterion at corresponding higher elevations.

3. Review Driving Record of Suspect Pile - If the suspect pile encountered appreciable driving resistance at a higher elevation than piles which attained the predetermined tip elevation, the suspect pile may have an embedment length in suitable material similar to piles driven to the predetermined tip elevation.

4. Drive Additional Piles - Observe the driving behavior and tip elevations of additional piles to evaluate the possible continuation of conditions encountered by suspect pile.

5. Review Driving Records of Adjacent Piles - Check for a trend toward higher tip elevations.

6. Check Suspect Pile Axial Resistance by Dynamic Monitoring - If dynamic monitoring equipment is being used on the project, check the axial

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resistance of the suspect pile.

7. Check Potential for Temporary Increase in Driving Resistance - If soil conditions "artificially" elevate, driving resistances at the end of initial driving, redriving the suspect pile after a waiting period may advance the pile to the predetermined tip elevation.

10.7.11 Foundation Submission

As part of the foundation approval process, a foundation submission letter shall be submitted to the Department for use in foundation review, in compliance with PP1.9.4 and D11.4.4P. The letter shall include the basis for the pile capacity (e.g., load tests, wave equation analyses, dynamic monitoring, past experience) and the relevance of field conditions and construction procedures used to develop the pile capacity. In addition to the basic requirements of PP1.9.4 and D11.4.4P, the foundation submission letter shall include the following, as a minimum:

a. Pile type and size (including alternates),

b. Geotechnical axial pile capacity,

c. Structural pile capacity,

d. Basis for pile capacity determination,

e. Bottom of pile cap elevation,

f. Estimated tip elevations and maximum pile lengths,

g. Description of anticipated bearing stratum,

h. Anticipated load carrying mechanism (point bearing, end bearing, or friction),

i. Relevant soil and/or rock conditions,

j. If requested, in special and/or unusual situations, preliminary wave equation analysis, for a trial driving system including soils and driving system input parameters,

k. Where applicable, foundation type of existing structure and other pertinent information,

l. Downdrag analysis, if anticipated,

m. COM624P or LPILE 5.0 analysis results and lateral pile capacities, and

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n. Evaluation of corrosion potential and design measures for protection against deterioration.

10.8 DRILLED SHAFTS

This article presents design specifications for drilled shafts. Refer to Appendix B for design specifications for micropiles.

10.8.1 General

10.8.1.1 SCOPE

The provisions of this Section shall apply to the design of drilled shafts. Throughout these provisions, the use of the term "drilled shaft" shall be interpreted to mean a shaft constructed using either drilling (open hole or with drilling slurry) or casing plus excavation equipment and technology.

These provisions shall also apply to shafts that are constructed using casing advancers that twist or rotate casings into the ground concurrent with excavation rather than drilling.

The provisions of this Section shall not be taken as applicable to drilled piles, e.g., augercast piles, installed with continuous flight augers that are concreted as the auger is being extracted.

Drilled shafts shall be considered for foundation support when spread footings cannot be founded on suitable soil or rock strata within a reasonable depth (e.g., 3000 mm {10 ft.}), in areas of karst conditions, and when piles are not economically feasible due to high loads or obstructions to driving. Drilled shafts shall also be considered when high lateral or uplift loads must be resisted and deformation tolerances are small, or as a direct support element for columns used as pier bents. As an alternate to drilled shafts, foundation pedestals or columns may be considered when suitable rock bearing is within 3000 mm {10 ft.} of the ground surface (see D10.6.4.2P).

Drilled shafts not founded on or socketed into rock are generally not permitted by the Department and, if used, must be approved by the Chief Bridge Engineer.

The terminology used in the design of drilled shafts is shown in Figure 1.

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C10.8.1.1

Drilled shafts may be an economical alternative to spread footing or pile foundations, particularly when spread footings cannot be founded on suitable soil or rock strata within a reasonable depth or when driven piles are not viable. Drilled shafts may be an economical alternative to spread footings where scour depth is large. Drilled shafts may also be considered to resist high lateral or axial loads, or when deformation tolerances are small. For example, a movable bridge is a bridge where it is desirable to keep deformations small.

Drilled shafts are classified according to their primary mechanism for deriving load resistance either as floating (friction) shafts, i.e., shafts transferring load primarily by side resistance, or end-bearing shafts, i.e., shafts transferring load primarily by tip resistance.

It is recommended that the shaft design be reviewed for constructability prior to advertising the project for bids.

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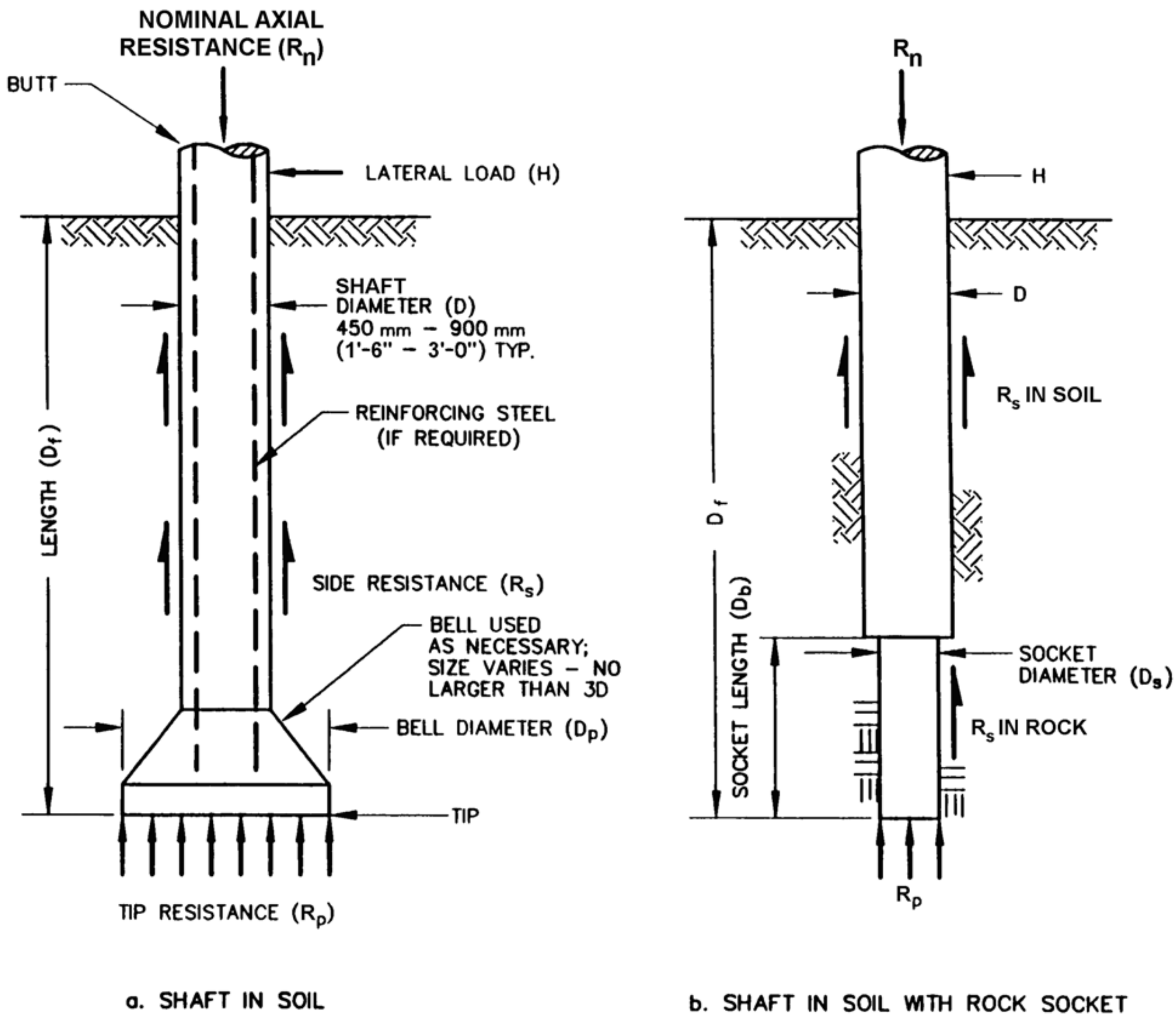

Figure 10.8.1.1-1 - Design Terminology for Drilled Shaft Foundations

10.8.1.2 SHAFT SPACING, CLEARANCE, AND EMBEDMENT INTO CAP

If the center-to-center spacing of drilled shafts is less than 4.0 diameters, the interaction effects between adjacent shafts during construction shall be evaluated. If the center-to-center spacing of drilled shafts is less than 6.0 diameters, the sequence of construction should be specified in the contract documents.

Shafts used in groups should be located such that the distance from the side of any shaft to the nearest edge of the cap is not less than 300 mm. {1 ft.} Shafts shall be embedded sufficiently into the cap to develop the required structural resistance. Shaft reinforcement shall extend sufficiently into the cap to overlap horizontal cap reinforcement in order to develop the required stress transfer.

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Larger spacing may be required to preserve shaft excavation stability or to prevent communication between shafts during excavation and concrete placement.

Shaft spacing may be decreased if casing construction methods are required to maintain excavation stability and to prevent interaction between adjacent shafts.

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10.8.1.3 SHAFT DIAMETER AND ENLARGED BASES

For drilled shaft requiring inspection, the shaft dimensions shall not be less than 900 mm {3 ft.}. All shafts shall be sized in 150 mm {6 in.} increments with a minimum shaft diameter of 450 mm {1'-6"}. Inclined or battered shafts shall not be used without the prior approval of the Chief Bridge Engineer. The diameter of shafts with rock sockets shall be sized a minimum of 150 mm {6 in.} larger than the diameter of the socket. The diameter of columns supported by shafts shall be less than D. See Figure 1 for dimensional guidelines.

In stiff cohesive soils, an enlarged base (bell, or underream) may be used at the shaft tip to increase the tip bearing area to reduce the unit end bearing pressure or to provide additional resistance to uplift loads.

Where the bottom of the drilled hole is dry, cleaned and inspected prior to concrete placement, the entire base area may be taken as effective in transferring load.

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C10.8.1.3

Nominal shaft diameters used for both geotechnical and structural design of shafts should be selected based on available diameter sizes. Most drilling tools and casings used in the United States are sized in 150 mm {6 in} size intervals. Therefore, unless special project requirements dictate the use of unconventional shaft dimensions, it is most economical to size shaft diameters in 150 mm {6 in} increments.

If the shaft and the column are the same diameter, it should be recognized that the placement tolerance of drilled shafts is such that it will likely affect the column location. The shaft and column diameter should be determined based on the shaft placement tolerance, column and shaft reinforcing clearances, and the constructability of placing the column reinforcing in the shaft. A horizontal construction joint in the shaft at the bottom of the column reinforcing will facilitate constructability. Making allowance for the tolerance where the column connects with the superstructure, which could affect column alignment, can also accommodate this shaft construction tolerance.

In drilling rock sockets, it is common to use casing through the soil zone to temporarily support the soil to prevent cave-in, allow inspection and to produce a seal along the soil-rock contact to minimize infiltration of groundwater into the socket. Depending on the method of excavation, the diameter of the rock socket may need to be sized at least 150 mm {0.5 ft.} smaller than the nominal casing size to permit seating of casing and insertion of rock drilling equipment.

Where practical, consideration should be given to extension of the shaft to a greater depth to avoid the difficulty and expense of excavation for enlarged bases.

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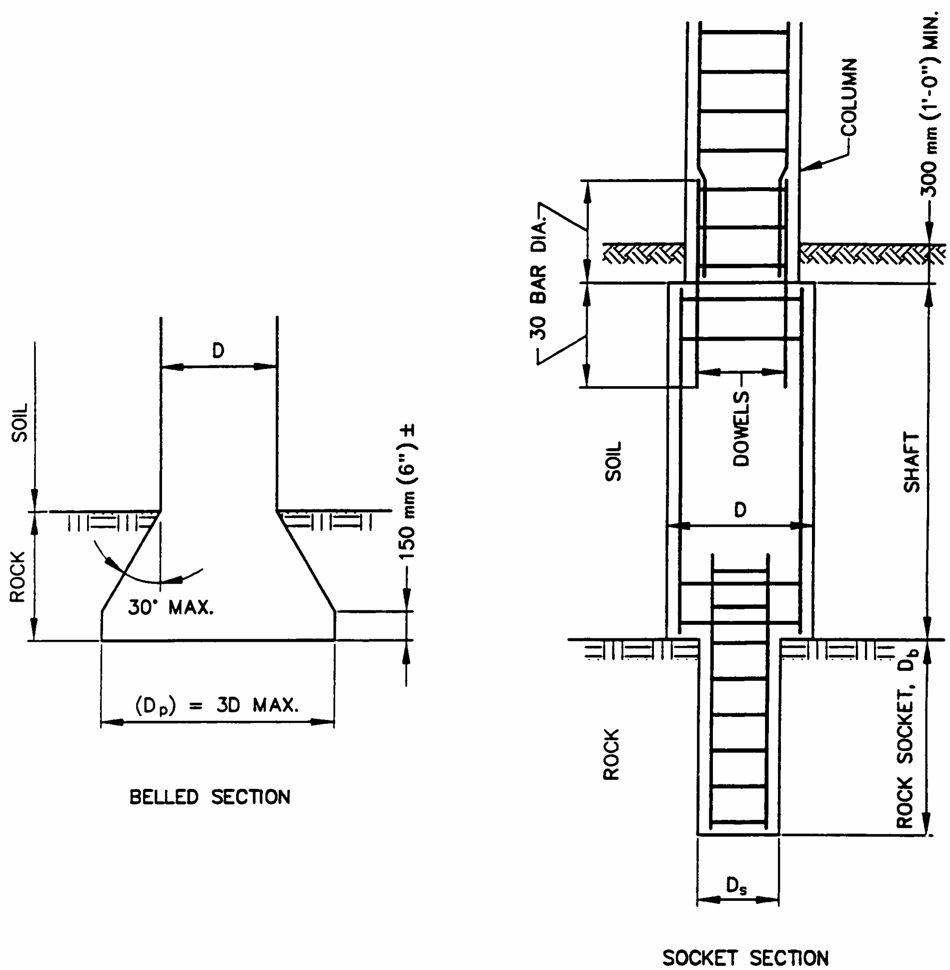

Figure 10.8.1.3-1 - Dimensional Guidelines for Drilled Shafts

10.8.1.4 BATTERED SHAFTS

Battered shafts should be avoided. Where increased lateral resistance is needed, consideration should be given to increasing the shaft diameter or increasing the number of shafts.

C10.8.1.4

Due to problems associated with hole stability during excavation, installation, and with removal of casing during installation of the rebar cage and concrete placement, construction of battered shafts is very difficult.

10.8.1.5 DRILLED SHAFT RESISTANCE

Drilled shafts shall be designed to have adequate axial and structural resistances, tolerable settlements, and tolerable lateral displacements.

The axial resistance of drilled shafts shall be determined through a suitable combination of subsurface investigations, laboratory and/or in-situ tests, analytical methods, and load tests, with reference to the history of past performance. Consideration shall also be given to:

- The difference between the resistance of a single shaft

C10.8.1.5

The drilled shaft design process is discussed in detail in Drilled Shafts: Construction Procedures and Design Methods (O'Neill and Reese, 1999).

The performance of drilled shaft foundations can be greatly affected by the method of construction, particularly side resistance. The designer should consider the effects of ground and groundwater conditions on shaft construction operations and delineate, where necessary, the general method of construction to be followed to ensure the expected performance. Because shafts derive their resistance

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and that of a group of shafts;

- The resistance of the underlying strata to support the load of the shaft group;

- The effects of constructing the shaft(s) on adjacent structures;

- The possibility of scour and its effect;

- The transmission of forces, such as downdrag forces, from consolidating soil;

- Minimum shaft penetration necessary to satisfy the requirements caused by uplift, scour, downdrag, settlement, liquefaction, lateral loads and seismic conditions;

- Satisfactory behavior under service loads;

- Drilled shaft nominal structural resistance; and

- Long-term durability of the shaft in service, i.e., corrosion and deterioration.

Resistance factors for shaft axial resistance for the strength limit state shall be as specified in Table D10.5.5.2.4-1.

The method of construction may affect the shaft axial and lateral resistance. The shaft design parameters shall take into account the likely construction methodologies used to install the shaft. Drilled shafts shall be constructed using the dry, casing or wet method of construction or a combination of these methods.

In every case, hole excavation, concrete placement and all other aspects of shaft construction shall be performed in conformance with the provisions of the specifications, Publication 408 and applicable Special Provisions.

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from side and tip resistance, which is a function of the condition of the materials in direct contact with the shaft, it is important that the construction procedures be consistent with the material conditions assumed in the design. Softening, loosening, or other changes in soil and rock conditions caused by the construction method could result in a reduction in shaft resistance and an increase in shaft displacement. Therefore, evaluation of the effects of the shaft construction procedure on resistance should be considered an inherent aspect of the design. Use of slurries, varying shaft diameters, and post grouting can also affect shaft resistance.

Soil parameters should be varied systematically to model the range of anticipated conditions. Both vertical and lateral resistance should be evaluated in this manner.

Procedures that may affect axial or lateral shaft resistance include, but are not limited to, the following:

- Artificial socket roughening, if included in the design nominal axial resistance assumptions.

- Removal of temporary casing where the design is dependent on concrete-to-soil adhesion.

- The use of permanent casing.

- Use of tooling that produces a uniform cross-section where the design of the shaft to resist lateral loads cannot tolerate the change in stiffness if telescoped casing is used.

It should be recognized that the design procedures provided in these Specifications assume compliance to construction specifications that will produce a high quality shaft. Performance criteria should be included in the construction specifications that require:

- Shaft bottom cleanout criteria,

- Appropriate means to prevent side wall movement or failure (caving) such as temporary casing, slurry, or a combination of the two,

- Slurry maintenance requirements including minimum slurry head requirements, slurry testing requirements, and maximum time the shaft may be left open before concrete placement.

If for some reason one or more of these performance

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criteria are not met, the design should be reevaluated and the shaft repaired or replaced as necessary.

Past practice in the Commonwealth is that temporary or permanent casing is used to support the sides of the drilled shaft hole.

10.8.1.6 DETERMINATION OF SHAFT LOADS

10.8.1.6.1 General

The factored loads to be used in shaft foundation design shall be as specified in Section 3. Computational assumptions that shall be used in determining individual shaft loads are also specified in Section 4.

C10.8.1.6.1

The specification and determination of top of cap loads is discussed extensively in Section 3. It should be noted that A3.6.2.1 states that dynamic load allowance need not be applied to foundation elements that are below the ground surface. Therefore, if shafts extend above the ground surface to act as columns the dynamic load allowance should be included in evaluating the structural resistance of that part of the shaft above the ground surface. The dynamic load allowance may be ignored in evaluating the geotechnical resistance.

10.8.1.6.2 Downdrag

The provisions of D10.7.1.6.2 and A3.11.8 shall apply.

C10.8.1.6.2

See DC10.7.1.6.2 and AC3.11.8.

Downdrag loads may be estimated using the α-method, as specified in D10.8.3.5.1b, for calculating negative shaft resistance. As with positive shaft resistance, the top 1500 mm {5 ft.} and a bottom length taken as one shaft diameter should be assumed to not contribute to downdrag loads.

When using the α-method, an allowance should be made for a possible increase in the undrained shear strength as consolidation occurs. Downdrag loads may also come from cohesionless soils above settling cohesive soils, requiring granular soil friction methods be used in such zones to estimate downdrag loads.

10.8.1.6.3 Uplift

The provisions in D10.7.1.6.3 shall apply.

Tension in drilled shafts is not permitted at Service Limit States. At Strength Limit States, the uplift capacity may be taken as 10 percent of the axial structural capacity.

C10.8.1.6.3

See DC10.7.1.6.3.

10.8.2 Service Limit State Design

10.8.2.1 TOLERABLE MOVEMENTS

The requirements of D10.5.2.2 shall apply.

C10.8.2.1

See DC10.5.2.2.

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10.8.2.2 SETTLEMENT

10.8.2.2.1 General

The settlement of a drilled shaft foundation involving either single-drilled shafts or groups of drilled shafts shall not exceed the movement criteria selected in accordance with D10.5.2.2.

10.8.2.2.2 Settlement of Single-Drilled Shaft

The settlement of single-drilled shafts shall be estimated in consideration of:

- Short-term settlement,
- Consolidation settlement if constructed in cohesive soils, and
- Axial compression of the shaft.

The normalized load-settlement curves shown in Figures 1 through 4 should be used to limit the nominal shaft axial resistance computed as specified for the strength limit state in D10.8.3 for service limit state tolerable movements. Consistent values of normalized settlement shall be used for limiting the base and side resistance when using these figures. Long-term settlement should be computed according to D10.7.2 using the equivalent footing method and added to the short-term settlements estimated using Figures 1 though 4.

Other methods for evaluating shaft settlements that may be used are found in O'Neill and Reese (*1999*).

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C10.8.2.2.2

O'Neill and Reese (*1999*) have summarized load-settlement data for drilled shafts in dimensionless form, as shown in Figures 1 through 4. These curves do not include consideration of long-term consolidation settlement for shafts in cohesive soils. Figures 1 and 2 show the load-settlement curves in side resistance and in end bearing for shafts in cohesive soils. Figures 3 and 4 are similar curves for shafts in cohesionless soils. These curves should be used for estimating short-term settlements of drilled shafts.

The designer should exercise judgment relative to whether the trend line, one of the limits, or some relation in between should be used from Figures 1 through 4.

The values of the load-settlement curves in side resistance were obtained at different depths, taking into account elastic shortening of the shaft. Although elastic shortening may be small in relatively short shafts, it may be substantial in longer shafts. The amount of elastic shortening in drilled shafts varies with depth. O'Neill and Reese (*1999*) have described an approximate procedure for estimating the elastic shortening of long- drilled shafts.

Settlements induced by loads in end bearing are different for shafts in cohesionless soils and in cohesive soils. Although drilled shafts in cohesive soils typically have a well-defined break in a load-displacement curve, shafts in cohesionless soils often have no well-defined failure at any displacement. The resistance of drilled shafts in cohesionless soils continues to increase as the settlement increases beyond 5 percent of the base diameter. The shaft end bearing R_p is typically fully mobilized at displacements of 2 to 5 percent of the base diameter for shafts in cohesive soils. The unit end bearing resistance for the strength limit state (see D10.8.3) is defined as the bearing pressure required to cause vertical deformation equal to 5 percent of the shaft diameter, even though this does not correspond to complete failure of the soil beneath the base of the shaft.

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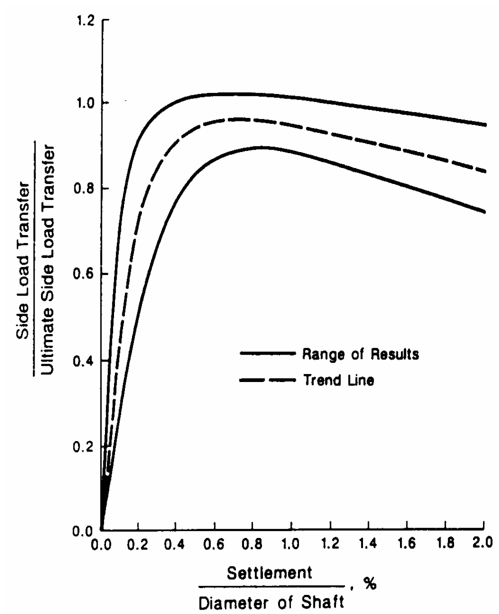

Figure 10.8.2.2.2-1 Normalized Load Transfer in Side Resistance Versus Settlement in Cohesive Soils (from *O'Neill and Reese, 1999*).

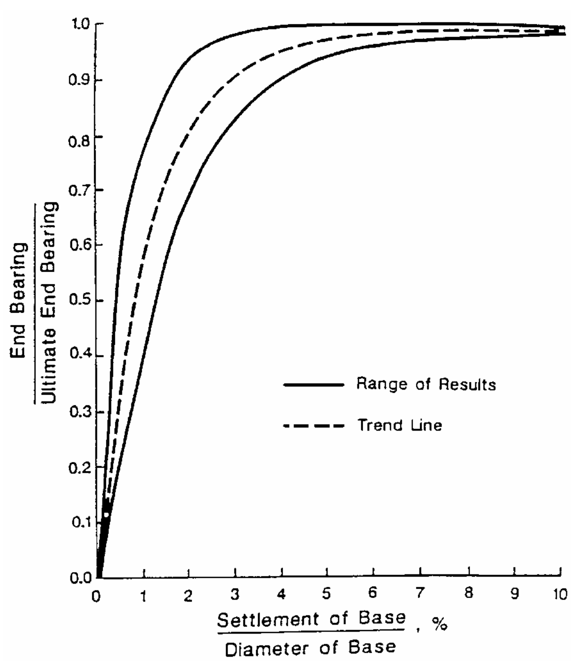

Figure 10.8.2.2.2-2 Normalized Load Transfer in End Bearing Versus Settlement in Cohesive Soils (from *O'Neill and Reese, 1999*).

COMMENTARY

The curves in Figures 1 and 3 also show the settlements at which the side resistance is mobilized. The shaft skin friction R_s is typically fully mobilized at displacements of 0.2 percent to 0.8 percent of the shaft diameter for shafts in cohesive soils. For shafts in cohesionless soils, this value is 0.1 percent to 1.0 percent.

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The deflection-softening response typically applies to cemented or partially cemented soils, or other soils that exhibit brittle behavior, having low residual shear strengths at larger deformations. Note that the trend line for sands is a reasonable approximation for either the deflection-softening or deflection-hardening response.

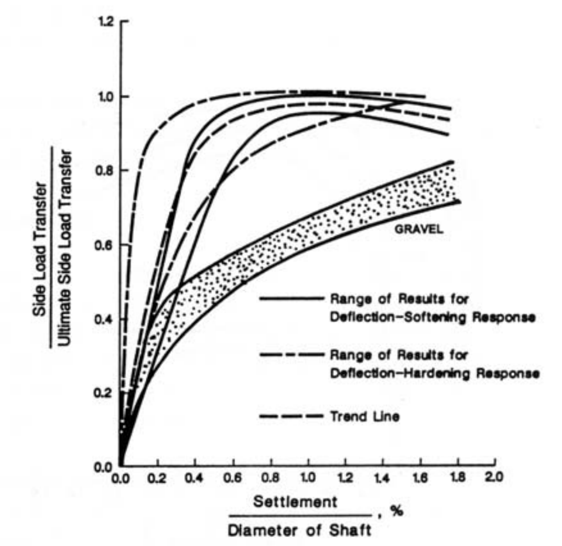

Figure 10.8.2.2.2-3 Normalized Load Transfer in Side Resistance Versus Settlement in Cohesionless Soils (from *O'Neill and Reese, 1999*).

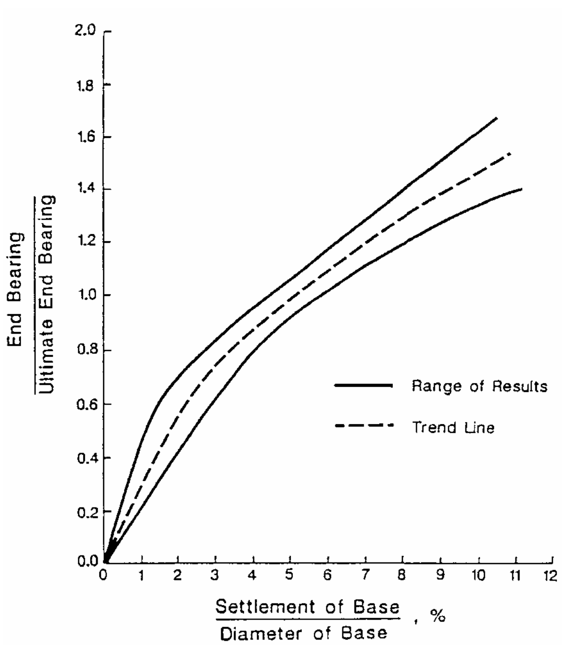

Figure 10.8.2.2.2-4 Normalized Load Transfer in End Bearing Versus Settlement in Cohesionless Soils (from *O'Neill and Reese*, *1999*).

10.8.2.2.3 Intermediate Geo Materials (IGMs)

For detailed settlement estimation of shafts in IGMs, the procedures provided by O'Neill and Reese (*1999*) should be used.

C10.8.2.2.3

IGMs are defined by O'Neill and Reese (*1999*) as follows:

- *Cohesive IGM*—clay shales or mudstones with an S_u of

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0.25 to 2.5 MPa {5 to 50 ksf}, and

- *Cohesionless*—granular tills or granular residual soils with $N1_{60}$ greater than 50 blows/300 mm {50 blows/ft.}.

10.8.2.2.4 Group Settlement

The provisions of D10.7.2.3 shall apply. Shaft group effect shall be considered for groups of 2 shafts or more.

C10.8.2.2.4

See DC10.7.2.3.

O'Neill and Reese (*1999*) summarize various studies on the effects of shaft group behavior. These studies were for groups that consisted of 1 × 2 to 3 × 3 shafts. These studies suggest that group effects are relatively unimportant for shaft center-to-center spacing of 5*D* or greater.

10.8.2.3 HORIZONTAL MOVEMENT OF SHAFTS AND SHAFT GROUPS

The provisions of D10.5.2.2 and D10.7.2.4 shall apply.

C10.8.2.3

See DC10.5.2.2 and DC10.7.2.4.

10.8.2.4 SETTLEMENT DUE TO DOWNDRAG

The provisions of D10.7.2.5 shall apply.

C10.8.2.4

See DC10.7.2.5.

10.8.2.5 LATERAL SQUEEZE

The provisions of D10.7.2.6 shall apply.

C10.8.2.5

See DC10.7.2.6.

10.8.3 Strength Limit State Design

10.8.3.1 GENERAL

The nominal shaft resistances that shall be considered at the strength limit state include:

- axial compression resistance,
- axial uplift resistance,
- punching of shafts through strong soil into a weaker layer,
- lateral geotechnical resistance of soil and rock stratum,
- resistance when scour occurs,
- axial resistance when downdrag occurs, and
- structural resistance of shafts.

10.8.3.2 GROUND WATER TABLE AND BOUYANCY

The provisions of D10.7.3.5 shall apply.

C10.8.3.2

See DC10.7.3.5.

10.8.3.3 SCOUR

C10.8.3.3

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The provisions of D10.7.3.6 shall apply.

Scour investigations and design of bridge structure foundations to resist scour shall be in accordance with PP7.2.

10.8.3.4 DOWNDRAG

The provisions of D10.7.3.7 shall apply.

10.8.3.5 NOMINAL AXIAL COMPRESSION RESISTANCE OF SINGLE DRILLED SHAFTS

The factored resistance of drilled shafts, R_R, shall be taken as:

$$R_R = \varphi R_n = \varphi_{qp} R_p + \varphi_{qs} R_s \qquad (10.8.3.5\text{-}1)$$

in which:

$$R_p = q_p A_p \qquad (10.8.3.5\text{-}2)$$

$$R_s = q_s A_s \qquad (10.8.3.5\text{-}3)$$

where:

R_p = nominal shaft tip resistance (N){kips}

R_s = nominal shaft side resistance (N){kips}

φ_{qp} = resistance factor for tip resistance specified in Table D10.5.5.2.4-1

φ_{qs} = resistance factor for shaft side resistance specified in Table D10.5.5.2.4-1

q_p = unit tip resistance (MPa){ksf}

q_s = unit side resistance (MPa){ksf}

A_p = area of shaft tip (mm^2) {$ft.^2$}

A_s = area of shaft side surface (mm^2) {$ft.^2$}

COMMENTARY

See DC10.7.3.6.

C10.8.3.4

See DC10.7.3.7.

C10.8.3.5

The nominal axial compression resistance of a shaft is derived from the tip resistance and/or shaft side resistance, i.e., skin friction. Both the tip and shaft resistances develop in response to foundation displacement. The maximum values of each are unlikely to occur at the same displacement, as described in D10.8.2.2.2.

The load transfer and deformation characteristics of drilled shafts are a function of the ratios of shaft depth to diameter (D_f /D), the shaft diameter (D) to base diameter (D_p), and the relative stiffness between the shaft and the soil. The following observations are applicable:

1. As D_f /D and soil stiffness decrease, the proportion of load supported by the tip (R_p) increases.

2. Shafts with large values of D_f /D develop appreciable R_s before mobilizing any R_p.

3. Butt displacements of the order of 8 to 10% of D are necessary to mobilize R_p in cohesionless soils and stiff clays.

4. The ultimate shaft resistance (R_s) is typically fully mobilized at displacements of about 5 to 10 mm {0.2 to 0.4 in.}.

Drained analyses using effective soil shear strengths (c' and ϕ'_f) are appropriate for cohesionless soils and most rock types, and for cohesive soils which are permitted to drain and consolidate under loading. Such analyses are appropriate for bridge pier foundations which are subjected to large dead loads and relatively small live loads. Undrained loading should be considered where very high live loads are anticipated in cohesive soils. Live loads are of short duration, and do not provide sufficient time for draining of cohesive soils.

For consistency in the interpretation of both static load tests (D10.8.3.5.6) and the normalized curves of D10.8.2.2.2, it is customary to establish the failure criterion at the strength limit state at a gross deflection equal to 5 percent of the base diameter for drilled shafts.

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COMMENTARY

O'Neill and Reese (*1999*) identify several methods for estimating the resistance of drilled shafts in cohesive and granular soils, intermediate geomaterials, and rock. The most commonly used methods are provided in this article. Methods other than the ones provided in detail in this article may be used provided that adequate local or national experience with the specific method is available to have confidence that the method can be used successfully and that appropriate resistance factors can be determined. At present, it must be recognized that these resistance factors have been developed using a combination of calibration by fitting to previous allowable stress design (ASD) practice and reliability theory (see *Allen, 2005*, for additional details on the development of resistance factors for drilled shafts). Such methods may be used as an alternative to the specific methodology provided in this article, provided that:

- The method selected consistently has been used with success on a regional or national basis.
- Significant experience is available to demonstrate that success.
- As a minimum, calibration by fitting to allowable stress design is conducted to determine the appropriate resistance factor, if inadequate measured data are available to assess the alternative method using reliability theory. A similar approach as described by Allen (*2005*) should be used to select the resistance factor for the alternative method.

The methods for estimating drilled shaft resistance provided in this article should be used. Shaft strength limit state resistance methods not specifically addressed in this article for which adequate successful regional or national experience is available may be used with approval of the Chief Bridge Engineer, provided adequate information and experience is also available to develop appropriate resistance factors.

10.8.3.5.1 Estimation of Drilled Shaft Resistance in Cohesive Soils

10.8.3.5.1a General

Drilled shafts in cohesive soils should be designed by total and effective stress methods for undrained and drained loading conditions, respectively.

10.8.3.5.1b Side Resistance

The nominal unit side resistance, q_S, in MPa, for shafts in cohesive soil loaded under undrained loading conditions by the α-Method shall be taken as:

$$q_s = \alpha S_u \qquad (10.8.3.5.1b\text{-}1)$$

in which:

$$\alpha = 0.55 \text{ for } \frac{S_u}{p_a} \le 1.5 \qquad (10.8.3.5.1b\text{-}2)$$

$$\alpha = 0.55 - 0.1\left(S_u / p_a - 1.5\right)$$
$$\text{for } 1.5 \le S_u / p_a \le 2.5 \qquad (10.8.3.5.1b\text{-}3)$$

C10.8.3.5.1b

The α-method is based on total stress. For effective stress methods for shafts in clay, see O'Neill and Reese (*1999*).

The adhesion factor is an empirical factor used to correlate the results of full-scale load tests with the material property or characteristic of the cohesive soil. The adhesion factor is usually related to S_u and is derived from the results of full-scale pile and drilled shaft load tests. Use of this approach presumes that the measured value of S_u is correct and that all shaft behavior resulting from construction and loading can be lumped into a single parameter. Neither presumption is strictly correct, but the approach is used due to its simplicity.

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where:

S_u = undrained shear strength (MPa){ksf}

α = adhesion factor (dim.)

p_a = atmospheric pressure (= 0.101 MPa) {2.12 ksf}

The following portions of a drilled shaft, illustrated in Figure 1, should not be taken to contribute to the development of resistance through skin friction:

- at least the top 1500 mm {5 ft} of any shaft;
- for straight shafts, a bottom length of the shaft taken as the shaft diameter;
- periphery of belled ends, if used; and
- distance above a belled end taken as equal to the shaft diameter.

When permanent casing is used, the side resistance shall be adjusted with consideration to the type and length of casing to be used, and how it is installed.

Values of α for contributing portions of shafts excavated dry in open or cased holes should be as specified in Eqs. 2 and 3.

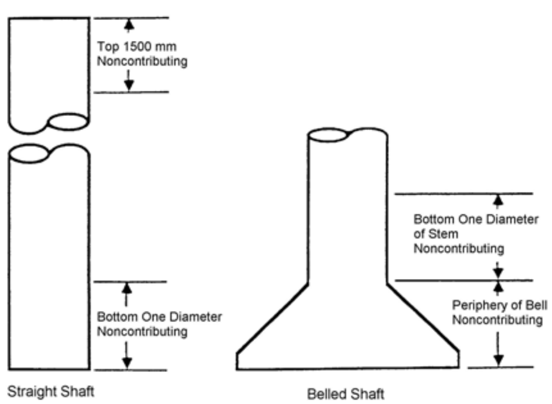

Figure 10.8.3.5.1b-1 Explanation of Portions of Drilled Shafts Not Considered in Computing Side Resistance (*O'Neill and Reese, 1999*).

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Steel casing will generally reduce the side resistance of a shaft. No specific data is available regarding the reduction in skin friction resulting from the use of permanent casing relative to concrete placed directly against the soil. Side resistance reduction factors for driven steel piles relative to concrete piles can vary from 50 to 75 percent, depending on whether the steel is clean or rusty, respectively (*Potyondy, 1961*). Greater reduction in the side resistance may be needed if oversized cutting shoes or splicing rings are used.

If open-ended pipe piles are driven full depth with an impact hammer before soil inside the pile is removed, and left as a permanent casing, driven pile static analysis methods may be used to estimate the side resistance as described in D10.7.3.8.6.

The upper 1500 mm {5 ft}of the shaft is ignored in estimating R_s, to account for the effects of seasonal moisture changes, disturbance during construction, cyclic lateral loading, and low lateral stresses from freshly placed concrete. The lower 1.0-diameter length above the shaft tip or top of enlarged base is ignored due to the development of tensile cracks in the soil near these regions of the shaft and a corresponding reduction in lateral stress and side resistance.

Bells or underreams constructed in stiff fissured clay often settle sufficiently to result in the formation of a gap above the bell that will eventually be filled by slumping soil. Slumping will tend to loosen the soil immediately above the bell and decrease the side resistance along the lower portion of the shaft.

The value of α is often considered to vary as a function of S_u. Values of α for drilled shafts are recommended as shown in Eqs. 2 and 3, based on the results of back-analyzed, full-scale load tests. This recommendation is based on eliminating the upper 1500 mm {5 ft.} and lower 1.0 diameter of the shaft length during back-analysis of load test results. The load tests were conducted in insensitive cohesive soils. Therefore, if shafts are constructed in sensitive clays, values of α may be different than those obtained from Eqs. 2 and 3. Other values of α may be used if based on the results of load tests.

The depth of 1500 mm {5 ft.}at the top of the shaft may need to be increased if the drilled shaft is installed in expansive clay, if scour deeper than 1500 mm {5 ft.} is anticipated, if there is substantial groundline deflection from lateral loading, or if there are other long-term loads or construction factors that could affect shaft resistance.

A reduction in the effective length of the shaft contributing to side resistance has been attributed to horizontal stress relief in the region of the shaft tip, arising from development of outward radial stresses at the toe during mobilization of tip resistance. The influence of this effect may extend for a distance of 1*B* above the tip (*O'Neill and Reese, 1999*). The effectiveness of enlarged bases is

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limited when L/D is greater than 25.0 due to the lack of load transfer to the tip of the shaft.

The values of α obtained from Eqs. 2 and 3 are considered applicable for both compression and uplift loading.

10.8.3.5.1c Tip Resistance

For axially loaded shafts in cohesive soil, the nominal unit tip resistance, q_p, in MPa {ksf} by the total stress method as provided in O'Neill and Reese (*1999*) shall be taken as:

Metric Units:

$$q_p = N_c S_u \leq 4.0 \qquad (10.8.3.5.1c\text{-}1)$$

U.S. Customary Units:

$$q_p = N_c S_u \leq 80.0$$

in which:

$$N_c = 6\left[1 + 0.2\left(\frac{Z}{D}\right)\right] \leq 9 \qquad (10.8.3.5.1c\text{-}2)$$

where:

D = diameter of drilled shaft (mm){ft.}

Z = penetration of shaft (mm){ft.}

S_u = undrained shear strength (MPa){ksf}

The value of S_u should be determined from the results of in-situ and/or laboratory testing of undisturbed samples obtained within a depth of 2.0 diameters below the tip of the shaft. If the soil within 2.0 diameters of the tip has S_u <0.024 MPa {0.50 ksf}, the value of N_c should be multiplied by 0.67.

C10.8.3.5.1c

These equations are for total stress analysis. For effective stress methods for shafts in clay, see O'Neill and Reese (*1999*).

The limiting value of 4.0 MPa for q_p is not a theoretical limit but a limit based on the largest measured values. A higher limiting value may be used if based on the results of a load test.

Use of enlarged bases requires the prior approval of the Chief Bridge Engineer. The tip resistance of an enlarged base shall be determined assuming that the entire base area is effective in transferring load. Allowance of full effectiveness of the enlarged base shall be permitted only when cleaning of the bottom of the drilled excavation is specified and can be acceptably completed before concrete placement.

An enlarged base may be used at the tip of a shaft to increase the tip bearing area, or to provide additional resistance to uplift loads. Due to the difficulty of excavation and support of enlarged bases, consideration should be given instead to extending the shaft to a greater depth to provide additional resistance. This avoids the construction difficulties and high additional cost of shafts with enlarged bases relative to straight-sided shafts.

10.8.3.5.2 Estimation of Drilled Shaft Resistance in

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Cohesionless Soils

10.8.3.5.2a General

Shafts in cohesionless soils should be designed by effective stress methods for drained loading conditions.

10.8.3.5.2b Side Resistance

The nominal unit side resistance, q_s in MPa {ksf} of drilled shafts in cohesionless soils by the β-method shall be taken as:

$$q_s = \beta\sigma'_v \leq 0.19 \text{ for } 0.25 \leq \beta \leq 1.2 \quad (10.8.3.5.2b\text{-}1)$$

in which, for sandy soils:

- for $N_{60} \geq 15$:

  Metric Units:

  $$\beta = 1.5 - \left(7.7\times10^{-3}\sqrt{z}\right) \quad (10.8.3.5.2b\text{-}2)$$

  U.S. Customary Units:

  $$\beta = 1.5 - \left(0.135\sqrt{z}\right)$$

- for $N_{60} < 15$:

  Metric Units:

  $$\beta = \frac{N_{60}}{15}(1.5 - 7.7\times10^{-3}\sqrt{z}) \quad (10.8.3.5.2b\text{-}3)$$

  U.S. Customary Units:

  $$\beta = \frac{N_{60}}{15}(1.5 - 0.135\sqrt{z})$$

where:

σ'_v = vertical effective stress at soil layer mid-depth (MPa){ksf}

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C10.8.3.5.2a

The factored resistance should be determined in consideration of available experience with similar conditions.

Although many field load tests have been performed on drilled shafts in clays, very few have been performed on drilled shafts in sands. The shear strength of cohesionless soils can be characterized by an angle of internal friction, ϕ_f, or empirically related to its *SPT* blow count, *N*. Methods of estimating shaft resistance and end bearing are presented below. Judgment and experience should always be considered.

C10.8.3.5.2b

O'Neill and Reese (*1999*) provide additional discussion of computation of shaft side resistance and recommend allowing β to increase to 1.8 in gravels and gravelly sands, however, they recommend limiting the unit side resistance to 0.19 MPa in all soils.

O'Neill and Reese (*1999*) proposed a method for uncemented soils that uses a different approach in that the shaft resistance is independent of the soil friction angle or the *SPT* blow count. According to their findings, the friction angle approaches a common value due to high shearing strains in the sand caused by stress relief during drilling.

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β = load transfer coefficient (dim.)

z = depth below ground, at soil layer mid-depth (mm){ft.}

N_{60} = average *SPT* blow count (corrected only for hammer efficiency) in the design zone under consideration (blows/300 mm){blows/ft.}

Higher values may be used if verified by load tests.

For gravelly sands and gravels, Eq. 4 should be used for computing β where $N_{60} \geq 15$. If $N_{60} < 15$, Eq. 3 should be used.

Metric Units:

$$\beta = 2.0 - 0.00082\left(z\right)^{0.75} \qquad (10.8.3.5.2b\text{-}4)$$

U.S. Customary Units:

$$\beta = 2.0 - 0.06\left(z\right)^{0.75}$$

When permanent casing is used, the side resistance shall be adjusted with consideration to the type and length of casing to be used, and how it is installed.

COMMENTARY

The detailed development of Eq. 4 is provided in O'Neill and Reese (*1999*).

Steel casing will generally reduce the side resistance of a shaft. No specific data is available regarding the reduction in skin friction resulting from the use of permanent casing relative concrete placed directly against the soil. Side resistance reduction factors for driven steel piles relative to concrete piles can vary from 50 to 75 percent, depending on whether the steel is clean or rusty, respectively (*Potyondy, 1961*). Casing reduction factors of 0.6 to 0.75 are commonly used. Greater reduction in the side resistance may be needed if oversized cutting shoes or splicing rings are used.

If open-ended pipe piles are driven full depth with an impact hammer before soil inside the pile is removed, and left as a permanent casing, driven pile static analysis methods may be used to estimate the side resistance as described in D10.7.3.8.6.

Table 10.8.3.5.2b-1 - Friction Angles and Unit Weights of Sands

			Metric Units	U.S. Customary Units
CONSISTENCY	ϕ_f	Nl_{60}	γ(kg/m^3)	γ(kcf)
Very Loose	25°- 30°	0 - 4	1100-1600	0.018-0.025
Loose	27°-32°	4-10	1400-1800	0.022-0.028
Medium	30°-35°	10-30	1800-2100	0.028-0.032
Dense	35°-40°	30-50	1900-2200	0.030-0.035
Very Dense	38°-43°	>50	2100-2300	0.032-0.038

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10.8.3.5.2c Tip Resistance

The nominal tip resistance, q_p, in MPa {ksf}, for drilled shafts in cohesionless soils by the O'Neill and Reese (*1999*) method shall be taken as:

Metric Units:

$$\text{for } 0.057N_{60} \le 50,\ q_p = 1.2N_{60} \qquad (10.8.3.5.2c\text{-}1)$$

U.S. Customary Units:

$$\text{for } N_{60} \le 50,\ q_p = 1.2N_{60}$$

where:

N_{60} = average *SPT* blow count (corrected only for hammer efficiency) in the design zone under consideration (blows/300 mm){blows/ft.}

The value of q_p in Eq. 1 should be limited to 3.0 MPa {60 ksf}, unless greater values can be justified though load test data.

Cohesionless soils with *SPT*-N_{60} blow counts greater than 50 shall be treated as intermediate geomaterial (IGM) and the tip resistance, in MPa {ksf}, taken as:

$$q_p = 0.59\left[N_{60}\left(\frac{p_a}{\sigma'_v}\right)\right]^{0.8}\sigma'_v \qquad (10.8.3.5.2c\text{-}2)$$

where:

p_a = atmospheric pressure (= 0.101 MPa) {2.12 ksf}

σ'_v = vertical effective stress at the tip elevation of the shaft (MPa){ksf}

N_{60} should be limited to 100 in Eq. 2 if higher values are measured.

COMMENTARY

C10.8.3.5.2c

O'Neill and Reese (*1999*) provide additional discussion regarding the computation of nominal tip resistance.

See O'Neill and Reese (*1999*) for background on IGMs.

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10.8.3.5.3 Shafts in Strong Soil Overlying Weaker Compressible Soil

Where the tip of a shaft could bear on a thin firm soil layer underlain by a softer soil unit, the shaft shall be extended through the softer soil unit to eliminate the potential for a punching shear failure into the softer soil deposit.

Where a shaft is tipped in a strong soil layer overlying a weaker layer, the base resistance shall be reduced if the shaft base is within a distance of 1.5D of the top of the weaker layer. A weighted average should be used that varies

COMMENTARY

C10.8.3.5.3

Punching shear failure is a failure mode typically associated with drilled shafts bearing on soils which behave plastically, but it is also of concern where shafts bear on a thin firm soil layer underlain by a softer deposit. In such cases, the influence of the bearing load at the surface of the soft layer shall be analyzed.

The distance of 1.5D represents the zone of influence for general bearing capacity failure based on bearing capacity theory for deep foundations.

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linearly from the full base resistance in the overlying strong layer at a distance of 1.5D above the top of the weaker layer to the base resistance of the weaker layer at the top of the weaker layer.

10.8.3.5.4 Estimation of Drilled Shaft Resistance in Rock

10.8.3.5.4a General

Drilled shafts in rock subject to compressive loading shall be designed to support factored loads in:

- Side-wall shear comprising skin friction on the wall of the rock socket; or
- End bearing on the material below the tip of the drilled shaft; or
- A combination of both, with approval of the Chief Bridge Engineer

The side resistance from overlying soil deposits and weak rock shall be ignored. The difference in the deformation required to mobilize skin friction in rock versus what is required to mobilize end bearing shall be considered when estimating axial compressive resistance of shafts embedded in rock. Where end bearing in rock is used as part of the axial compressive resistance in the design, the contribution of skin friction in the rock shall be reduced to account for the loss of skin friction that occurs once the shear deformation along the shaft sides is greater than the peak rock shear deformation, i.e., once the rock shear strength begins to drop to a residual value.

COMMENTARY

C10.8.3.5.4a

Methods presented in this article to calculate drilled shaft axial resistance require an estimate of the uniaxial compressive strength of rock core. Unless the rock is massive, the strength of the rock mass is most frequently controlled by the discontinuities, including orientation, length, and roughness, and the behavior of the material that may be present within the discontinuity, e.g., gouge or infilling. The methods presented are semi-empirical and are based on load test data and site-specific correlations between measured resistance and rock core strength.

Design based on side-wall shear alone should be considered for cases in which the base of the drilled hole cannot be cleaned and inspected or where it is determined that large movements of the shaft would be required to mobilize resistance in end bearing.

Design based on end-bearing alone should be considered where sound bedrock underlies low strength overburden materials, including highly weathered rock. In these cases, however, it may still be necessary to socket the shaft into rock to provide lateral stability.

Where the shaft is drilled some depth into sound rock, a combination of sidewall shear and end bearing can be assumed (*Kulhawy and Goodman, 1980*).

If the rock is degradable, use of special construction procedures, larger socket dimensions, or reduced socket resistance should be considered.

For drilled shafts installed in karstic formations, exploratory borings should be advanced at each drilled shaft location to identify potential cavities. Layers of compressible weak rock along the length of a rock socket and within approximately three socket diameters or more below the base of a drilled shaft may reduce the resistance of the shaft.

For rock that is stronger than concrete, the concrete shear strength will control the available side friction, and the strong rock will have a higher stiffness, allowing significant end bearing to be mobilized before the side wall shear strength reaches its peak value. Note that concrete typically reaches its peak shear strength at about 250 to 400 microstrain (for a 3000 mm {10 ft.}long rock socket, this is approximately 10 mm {0.5 in.} of deformation at the top of the rock socket). If strains or deformations greater than the value at the peak shear stress are anticipated to mobilize the

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desired end bearing in the rock, a residual value for the skin friction can still be used. D10.8.3.5.4d provides procedures for computing a residual value of the skin friction based on the properties of the rock and shaft.

Rock stratification should be considered in the design of rock sockets as follows:

- Sockets embedded in alternating layers of weak and strong rock should be designed using the strength of the weaker rock.

- The side resistance provided by soft or weathered rock should be neglected in determining the required socket length where a socket extends into more competent underlying rock. Rock is defined as soft when the uniaxial compressive strength of the weaker rock is less than 20% of that of the stronger rock or weathered when the RQD is less than 20%.

- Where the tip of a shaft would bear on thin rigid rock strata underlain by a weaker unit, the shaft should be extended into or through the weaker unit (depending on load capacity or deformation requirements) to eliminate the potential for failure due to flexural tension or punching failure of the thin rigid stratum.

- Shafts designed to bear on strata in which the rock surface is inclined should extend to a sufficient depth to ensure that the shaft tip is fully bearing on the rock.

- Shafts designed to bear on rock strata in which bedding planes are not perpendicular to the shaft axis should extend a minimum depth of 2B into the dipping strata to minimize the potential for shear failure along natural bedding planes and other slippage surfaces associated with stratification.

10.8.3.5.4b Side Resistance

For drilled shafts socketed into rock, shaft side resistance, in MPa {ksf}, may be taken as (*Horvath and Kenney*, *1979*):

$$q_s = 0.65\alpha_E\, p_a \left(q_u / p_a\right)^{0.5} < 7.8\, p_a \left(f_c'/p_a\right)^{0.5} \qquad (10.8.3.5.4b\text{-}1)$$

where:

q_u = uniaxial compressive strength of rock (MPa){ksf}

| p_a = atmospheric pressure (= 0.101 MPa) {2.12 ksf }

α_E = reduction factor to account for jointing in rock as provided in Table 2

C10.8.3.5.4b

Eq. 1 applies to the case where the side of the rock socket is considered to be smooth or where the rock is drilled using a drilling slurry. Significant additional shaft resistance may be achieved if the borehole is specified to be artificially roughened by grooving. Methods to account for increased shaft resistance due to borehole roughness are provided in Section 11 of O'Neill and Reese (*1999*).

The procedure for calculating side resistance in rock is as follows:

Step 1. Evaluate the ratio of rock mass modulus to intact rock modulus, i.e., E_m/E_i, using Table 1.

Step 2. Evaluate the reduction factor, α_E, using Table 2.

Step 3. Calculate q_s according to Eq. 1.

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f'_c = concrete compressive strength (MPa){ksi}

Table 10.8.3.5.4b-1 Estimation of E_m based on *RQD* (after *O'Neill and Reese, 1999*).

RQD (percent)	E_m/E_i	
	Closed Joints	Open Joints
100	1.00	0.60
70	0.70	0.10
50	0.15	0.10
20	0.05	0.05

Table 10.8.3.5.4b-2. Estimation of α_E (*O'Neill and Reese, 1999*).

E_m/E_i	α_E
1.0	1.0
0.5	0.8
0.3	0.7
0.1	0.55
0.05	0.45

10.8.3.5.4c Tip Resistance

End-bearing for drilled shafts in rock may be taken as follows:

- If the rock below the base of the drilled shaft to a depth of 2.0*B* is either intact or tightly jointed, i.e., no compressible material or gouge-filled seams, and the depth of the socket is greater than 1.5*B* (*O'Neill and Reese, 1999*):

$$q_p = 2.5 q_u \qquad (10.8.3.5.4c\text{-}1)$$

- If the rock below the base of the shaft to a depth of 2.0*B* is jointed, the joints have random orientation, and the condition of the joints can be evaluated as:

$$q_p = \left[\sqrt{s} + \sqrt{(m\sqrt{s} + s)}\right] q_u \qquad (10.8.3.5.4c\text{-}2)$$

where:

s, m = fractured rock mass parameters and are specified in Table D10.4.6.4-4

q_u = unconfined compressive strength of rock (MPa){ksf}

C10.8.3.5.4c

If end bearing in the rock is to be relied upon, and wet construction methods are used, bottom clean-out procedures such as airlifts should be specified to ensure removal of loose material before concrete placement.

The use of Eq. 1 also requires that there are no solution cavities or voids below the base of the drilled shaft.

For further information see O'Neill and Reese (*1999*).

Eq. 2 is a lower bound solution for bearing resistance for a drilled shaft bearing on or socketed in a fractured rock mass. This method is appropriate for rock with joints that are not necessarily oriented preferentially and the joints may be open, closed, or filled with weathered material. Load testing will likely indicate higher tip resistance than that calculated using Eq. 2. Resistance factors for this method have not been developed and must therefore be estimated by the designer with review and approval by the Chief Bridge Engineer.

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10.8.3.5.4d Combined Side and Tip Resistance

Design methods that consider the difference in shaft movement required to mobilize skin friction in rock versus what is required to mobilize end bearing, such as the methodology provided by O'Neill and Reese (*1999*), shall be used to estimate axial compressive resistance of shafts embedded in rock.

COMMENTARY

C10.8.3.5.4d

Typically, the axial compression load on a shaft socketed into rock is carried solely in shaft side resistance until a total shaft movement on the order of 10 mm occurs.

Designs which consider combined effects of side friction and end-bearing of a drilled shaft in rock require that side friction resistance and end bearing resistance be evaluated at a common value of axial displacement, since maximum values of side friction and end-bearing are not generally mobilized at the same displacement.

Where combined side friction and end-bearing in rock is considered, the designer needs to evaluate whether a significant reduction in side resistance will occur after the peak side resistance is mobilized. As indicated in Figure C1, when the rock is brittle in shear, much shaft resistance will be lost as vertical movement increases to the value required to develop the full value of q_p. If the rock is ductile in shear, i.e., deflection softening does not occur, then the side resistance and end-bearing resistance can be added together directly. If the rock is brittle, however, adding them directly may be unconservative. Load testing or laboratory shear strength testing, e.g., direct shear testing, may be used to evaluate whether the rock is brittle or ductile in shear.

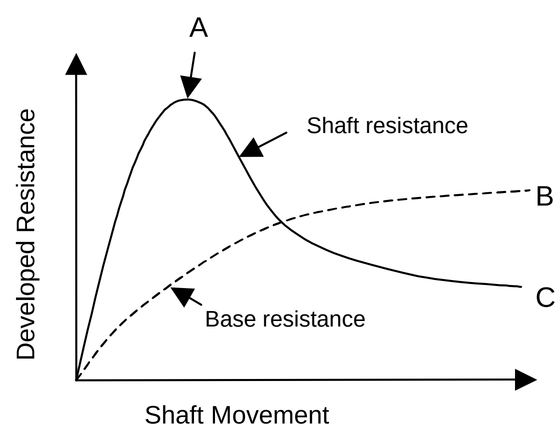

Figure C10.8.3.5.4d-1 Deflection Softening Behavior of Drilled Shafts under Compression Loading (after *O'Neill and Reese*, *1999*).

The method used to evaluate combined side friction and end-bearing at the strength limit state requires the construction of a load-vertical deformation curve. To accomplish this, calculate the total load acting at the head of the drilled shaft, Q_{T1}, and vertical movement, w_{T1}, when the nominal shaft side resistance (Point *A* on Figure C1) is mobilized. At this point, some end bearing is also mobilized. For detailed computational procedures for estimating shaft resistance in rock, considering the

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combination of side and tip resistance, see O'Neill and Reese (*1999*).

10.8.3.5.5 Estimation of Drilled Shaft Resistance in Intermediate Geo Materials (IGMs)

For detailed base and side resistance estimation procedures for shafts in IGMs, the procedures provided by O'Neill and Reese (*1999*) should be used.

C10.8.3.5.5

See D10.8.2.2.3 for a definition of an IGM.

For convenience, since a common situation is to tip the shaft in a cohesionless IGM, the equation for tip resistance in a cohesionless IGM is provided in DC10.8.3.5.2c.

10.8.3.5.6 Shaft Load Test

When used, load tests shall be conducted in representative soil conditions using shafts constructed in a manner and of dimensions and materials identical to those planned for the production shafts. The load test shall follow the procedures specified in ASTM D 1143. The loading procedure should follow the Quick Load Test Method, unless detailed longer-term load-settlement data is needed, in which case the standard loading procedure should be used.

The nominal resistance shall be determined according to the failure definition of either:

- "plunging" of the drilled shaft, or
- a gross settlement or uplift of 5 percent of the diameter of the shaft if plunging does not occur.

The resistance factors for axial compressive resistance or axial uplift resistance shall be taken as specified in Table D10.5.5.2.4-1.

Regarding the use of shaft load test data to determine shaft resistance, the load test results should be applied to production shafts that are not load tested by matching the static resistance prediction to the load test results. The calibrated static analysis method should then be applied to adjacent locations within the site to determine the shaft tip elevation required, in consideration of variations in the geologic stratigraphy and design properties at each production shaft location. The definition of a site and number of load tests required to account for site variability shall be as specified in D10.5.5.2.3.

C10.8.3.5.6

Load tests should be conducted following prescribed written procedures that have been developed from accepted standards and modified, as appropriate, for the conditions at the site. The Quick Test Procedure is desirable because it avoids problems that frequently arise when performing a static test that cannot be started and completed within an eight-hour period. Tests that extend over a longer period are difficult to perform due to the limited number of experienced personnel that are usually available. The Quick Test has proven to be easily performed in the field, and the results usually are satisfactory. However, if the formation in which the shaft is installed may be subject to significant creep settlement, alternative procedures provided in ASTM D 1143 should be considered.

Load tests are conducted on full-scale drilled shaft foundations to provide data regarding nominal axial resistance, load-displacement response, and shaft performance under the design loads, and to permit assessment of the validity of the design assumptions for the soil conditions at the test shaft(s).

Tests can be conducted for compression, uplift, lateral loading, or for combinations of loading. Full-scale load tests in the field provide data that include the effects of soil, rock, and groundwater conditions at the site; the dimensions of the shaft; and the procedures used to construct the shaft.

The results of full-scale load tests can differ even for apparently similar ground conditions. Therefore, care should be exercised in generalizing and extrapolating the test results to other locations.

For large diameter shafts, where conventional reaction frames become unmanageably large, load testing using Osterberg load cells (O-Cells) may be considered. An advantage of an O-Cell load test is that the load is applied from the bottom of the rock socket and data on the load carrying capacity of the rock socket can be directly obtained. An O-Cell load test was performed on a large diameter technique shaft at a site located at the borders of Fayette and Somerset Counties. The rock strata providing bearing for the test shaft is the Freeport Formation. A summary of the drilled shaft design and test results is presented in Table DC10.8.3.5.6-1.

The maximum test load on the rock socket in side shear

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was 51.09 MN {11,486 kips}, over 2.6 times the anticipated maximum factored axial load. The estimated average ultimate net unit side shear over the full 6100-mm {20-foot}long rock socket was on the order of 0.986 MPa {20.6 ksf}. This is in good agreement with the design average ultimate unit side resistance of 0.919 MPa {19.2 ksf}. The O-Cell results indicated that the ultimate net unit side shear of the stronger siltstone at the top and bottom of the rock socket was higher than the calculated design value. However, the O-Cell results indicated that, as the test load increased, no load was carried in the weaker argillaceous siltstone sandwiched between the stronger siltstone strata.

Additional discussion regarding load tests is provided in O'Neill and Reese (*1999*).

Plunging occurs when a steady increase in movement results from incrementally small increases in load, e.g., 1×10^4 N {2.0 kips}.

Table C10.8.3.5.6-1 Summary Of Drilled Shaft Design And Osterberg Cell Load Test Results Fayette And Somerset Counties

<u>SHAFT DATA</u>

Shaft Diameter = 2745 mm {9 ft.}
Rock Socket Diameter = 2592 mm {8.5 ft.}
Rock Socket Length = 6100 mm {20 ft.}

Maximum Factored Axial Load = 19.3 MN {4347 kips}

- Design socket length controlled by lateral capacity
- Design capacity is based on side resistance only; no end bearing

Rock Stratum	Upper Siltstone	Argillaceous Siltstone	Lower Siltstone
Approximate Thickness in Socket of Test Shaft, mm {ft.}	2440 {8.0}	1220 {4.0}	2440 {8.0}
Average RQD, %	71	89	90
Average RMR	56	65	64
Design Compressive Strength, MPa {psi}	24.1[1] {3500}	7.0 {1018}	24.1[1] {3500}
Design Ultimate Unit Side Resistance, MPa {ksf}	0.58 {12.2}	0.55 {11.4}	0.58 {12.2}
Design Factored Unit Side Resistance[2], MPa {ksf)	0.56 {11.6}	0.30 {6.2}	0.56 {11.6}
Estimated Ultimate Unit Side Shear from Load Test, MPa {ksf}	1.29 {27.0}	0	1.17 {24.5}

(1) Rock compressive strength is greater than concrete compressive strength of 24.1 MPa {3500 psi}. 24.1 MPa {3500 psi} used in design.
(2) Resistance Factor = 0.55.

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10.8.3.6 SHAFT GROUP RESISTANCE

10.8.3.6.1 General

Reduction in resistance from group effects shall be evaluated.

C10.8.3.6.1

In addition to the overlap effects discussed below, drilling of a hole for a shaft less than three shaft diameters from an existing shaft reduces the effective stresses against both the side and base of the existing shaft. As a result, the resistances of individual drilled shafts within a group tend to be less than the corresponding capacities of isolated shafts.

If casing is advanced in front of the excavation heading, this reduction need not be made.

10.8.3.6.2 Cohesive Soil

The provisions of D10.7.3.9 shall apply.

The resistance factor for the group resistance of an equivalent pier or block failure provided in Table D10.5.5.2.4-1 shall apply where the cap is, or is not, in contact with the ground.

The resistance factors for the group resistance calculated using the sum of the individual drilled shaft resistances are the same as those for the single-drilled shaft resistances.

C10.8.3.6.2

The efficiency of groups of drilled shafts in cohesive soil may be less than that of the individual shaft due to the overlapping zones of shear deformation in the soil surrounding the shafts.

10.8.3.6.3 Cohesionless Soil

Regardless of cap contact with the ground, the individual nominal resistance of each shaft should be reduced by a factor η for an isolated shaft taken as:

- η = 0.65 for a center-to-center spacing of 2.5 diameters,
- η = 1.0 for a center-to-center spacing of 4.0 diameters or more.
- For intermediate spacings, the value of η may be determined by linear interpolation.

C10.8.3.6.3

The bearing resistance of drilled shaft groups in sand is less than the sum of the individual shafts due to overlap of shear zones in the soil between adjacent shafts and loosening of the soil during construction. The recommended reduction factors are based in part on theoretical considerations and on limited load test results. See O'Neill and Reese (*1999*) for additional details and a summary of group load test results. It should be noted that most of the available group load test results were obtained for sands above the water table and for relatively small groups, e.g., groups of 3 to 9 shafts. For larger shaft groups, or for shaft groups of any size below the water table, more conservative values of η should be considered.

10.8.3.6.4 Shaft Groups in Strong Soil Overlying Weak Soil

For shaft groups that are collectively tipped within a strong soil layer overlying a soft, cohesive layer, block bearing resistance shall be evaluated in accordance with D10.7.3.9.

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10.8.3.7 UPLIFT RESISTANCE

10.8.3.7.1 General

Uplift resistance shall not be used without approval of the Chief Bridge Engineer. Uplift resistance shall be evaluated when upward loads act on the drilled shafts. Drilled shafts subjected to uplift forces shall be investigated for resistance to pullout, for their structural strength, and for the strength of their connection to supported components.

10.8.3.7.2 Uplift Resistance of Single Drilled Shaft

The uplift resistance of a single straight-sided drilled shaft should be estimated in a manner similar to that for determining side resistance for drilled shafts in compression, as specified in D10.8.3.

In determining the uplift resistance of a belled shaft, the side resistance above the bell should conservatively be neglected if the resistance of the bell is considered, and it can be assumed that the bell behaves as an anchor.

The factored nominal uplift resistance of a belled drilled shaft in a cohesive soil, R_R, in N {kips}, should be determined as:

$$R_R = \varphi R_n = \varphi_{up} R_{sbell} \qquad (10.8.3.7.2\text{-}1)$$

in which:

$$R_{s\ bell} = q_{s\ bell} A_u \qquad (10.8.3.7.2\text{-}2)$$

where:

q_{sbell} = $N_u S_u$ (MPa){ksf}

A_u = $\pi(D_p^2 - D^2)/4$ (mm^2) {ft.^{2}}

N_u = uplift adhesion factor (dim.)

D_p = diameter of the bell (mm){ft.}

D_b = depth of embedment in the founding layer (mm){ft.}

D = shaft diameter (mm){ft.}

S_u = undrained shear strength averaged over a distance of 2.0 bell diameters ($2D_p$) above the base (MPa){ksf}

φ_{up} = resistance factor specified in Table D10.5.5.2.4-1

COMMENTARY

C10.8.3.7.2

The resistance factors for uplift are lower than those for axial compression. One reason for this is that drilled shafts in tension unload the soil, thus reducing the overburden effective stress and hence the uplift side resistance of the drilled shaft. Empirical justification for uplift resistance factors is provided in DC10.5.5.2.3, and in Allen (*2005*).

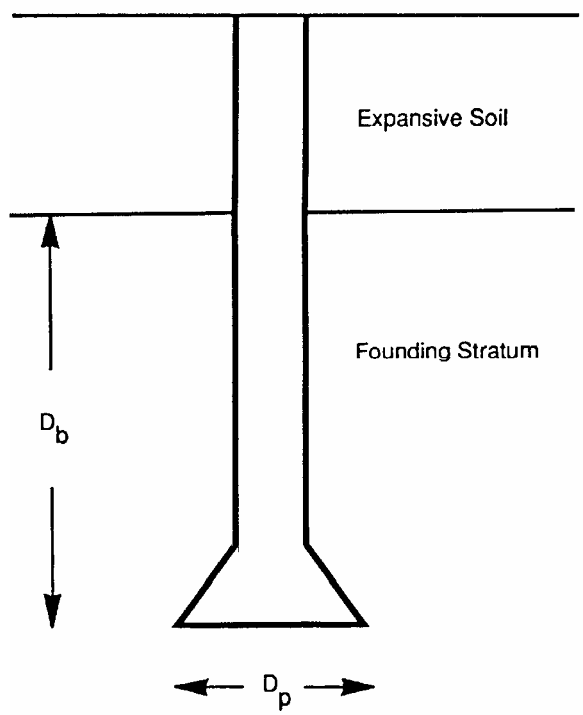

Figure C10.8.3.7.2-1 Uplift of a Belled Drilled Shaft.

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If the soil above the founding stratum is expansive, S_u should be averaged over the lesser of either $2.0D_p$ above the bottom of the base or over the depth of penetration of the drilled shaft in the founding stratum.

The value of N_u may be assumed to vary linearly from 0.0 at D_b/D_p = 0.75 to a value of 8.0 at D_b/D_p = 2.5, where D_b is the depth below the founding stratum. The top of the founding stratum should be taken at the base of zone of seasonal moisture change.

COMMENTARY

The assumed variation of N_u is based on Yazdanbod et al. (*1987*).

This method does not include the uplift resistance contribution due to soil suction and the weight of the shaft.

The effect of an enlarged base on uplift displacements is usually ignored because mobilization of any uplift resistance associated with enlarged bases typically occurs at displacements outside the tolerable limits of shaft displacement.

10.8.3.7.3 Group Uplift Resistance

The provisions of D10.7.3.11 shall apply.

10.8.3.7.4 Uplift Load Test

The provisions of D10.7.3.10 shall apply.

C10.8.3.7.4

See DC10.7.3.10.

10.8.3.8 NOMINAL HORIZONTAL RESISTANCE OF SHAFT AND SHAFT GROUPS

The provisions of D10.7.3.12 apply.

The design of horizontally loaded drilled shafts shall account for the effects of interaction between the shaft and ground, including the number of shafts in the group.

The design of laterally loaded drilled shafts shall account for the effects of soil layering, variable groundwater level, loss of lateral ground support (e.g., scour), cyclic loading, combined axial and lateral loading and sloping ground.

The final design of laterally loaded drilled shafts shall be based on the results of COM624P computer analyses, Wang and Reese (1993), and Reese (1984) or LPILE 5.0 (see ENSOFT, Inc. 2004 for LPILE 5.0). Other methods of analysis to evaluate the nominal horizontal resistance or deflection of laterally loaded shafts may be used for preliminary design only as a means to determine approximate shaft dimensions.

For shafts used in groups, the drilled shaft head shall be fixed into the cap.

The effects of group action shall be considered in the design of laterally loaded drilled shafts.

C10.8.3.8

See DC10.7.3.12.

Refer to O'Neill and Reese (1999) for methods of analysis to estimate nominal resistance of laterally loaded drilled shafts for preliminary design.

The major portion of lateral load resistance is mobilized within a depth equal to the five to eight shaft diameters from the ground surface.

10.8.3.9 SHAFT STRUCTURAL RESISTANCE

10.8.3.9.1 General

The structural design of drilled shafts shall be in accordance with the provisions of Section 5 for the design of reinforced concrete.

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10.8.3.9.2 Buckling and Lateral Stability

The provisions of D10.7.3.13.4 shall apply.

10.8.3.9.3 Reinforcement

Where the potential for lateral loading is insignificant, drilled shafts may be reinforced for axial loads only. Those portions of drilled shafts that are not supported laterally shall be designed as reinforced concrete columns in accordance with A5.7.4 and D5.7.4. Reinforcing steel shall extend a minimum of 3000 mm {10 ft.} below the plane where the soil provides fixity.

Where the potential for lateral loading is significant, the unsupported portion of the shaft shall be designed in accordance with A5.10.11, D5.10.11, A5.13.4.6 and D5.13.4.6.

The minimum spacing between longitudinal bars, as well as between transverse bars or spirals, shall be sufficient to allow free passage of the concrete through the cage and into the annulus between the cage and the borehole wall.

The minimum clear distance between reinforcement shall not be less than 1.5 times the bar diameter nor 1.5 times the maximum aggregate size, provided the concrete can be vibrated. If the concrete cannot be vibrated, the minimum distance between reinforcement shall not be less than three times the bar diameter nor three times the maximum aggregate size. See Figure D10.8.1.3-1 for guidelines for layout of steel reinforcement including minimum bar lap.

The minimum requirements to consider the steel shell to be load carrying shall be as specified in A5.13.4.5.2

Permanent steel casing, if used, shall have a thickness sufficient to withstand installation stresses, stresses due to lateral earth pressure and groundwater, and corrosion.

10.8.3.9.4 Transverse Reinforcement

Transverse reinforcement may be constructed as hoops of spiral steel.

Seismic provisions shall be in accordance with A5.13.4.6 and D5.13.4.6.

COMMENTARY

C10.8.3.9.2

See DC10.7.3.13.4.

C10.8.3.9.3

Shafts constructed using generally accepted procedures are not normally stressed to levels such that the allowable concrete stress is exceeded. Exceptions include:

- Shafts with sockets in hard rock,
- Shafts subjected to lateral loads,
- Shafts subjected to uplift loads from expansive soils or direct application of uplift loads, and
- Shafts with unreinforced bells.

Maintenance of the spacing of reinforcement and the maximum aggregate size requirements are important to ensure that the high-slump concrete mixes normally used for drilled shafts can flow readily between the steel bars during concrete placement. See A5.13.4.5.2 for specifications regarding the minimum clear spacing required between reinforcing cage bars.

A shaft can be considered laterally supported:

- below the zone of liquefaction or seismic loads,
- in rock, or
- 1500 mm {5 ft.}below the ground surface or the lowest anticipated scour elevation.

Laterally supported does not mean fixed. Fixity would occur somewhat below this location and depends on the stiffness of the supporting soil.

The out-to-out dimension of the assembled reinforcing cage should be sufficiently smaller than the diameter of the drilled hole to ensure free flow of concrete around the reinforcing as the concrete is placed. See A5.13.4 and D5.13.4.

See DC10.7.5 regarding assessment of corrosivity. In addition, consideration should be given to the ability of the concrete and steel shell to bond together.

SPECIFICATIONS

10.8.3.9.5 Concrete

The maximum aggregate size, slump, wet or dry placement, and necessary design strength should be considered when specifying shaft concrete. The concrete selected should be capable of being placed and adequately consolidated for the anticipated construction condition, and shaft details should be specified. The maximum size aggregate shall meet the requirements of D10.8.3.9.3.

10.8.3.9.6 Reinforcement into Superstructure

Sufficient reinforcement shall be provided at the junction of the shaft with the shaft cap or column to make a suitable connection. The embedment of the reinforcement into the cap shall comply with the provision for cast-in-place piles in Section 5.

10.8.3.9.7 Enlarged Bases

Enlarged bases shall be designed to ensure that the plain concrete is not overstressed. The enlarged base shall slope at a side angle not greater than 30° from the vertical and have a bottom diameter not greater than three times the diameter of the shaft. The thickness of the bottom edge of the enlarged base shall not be less than 150 mm {6.0 in}.

10.8.4 Extreme Event Limit State

The provisions of D10.5.5.3 and D10.7.4 shall apply.

10.8.5 Foundation Submission

As part of the foundation approval process, a foundation submission letter shall be submitted to the Department for use in foundation review, in compliance with PP1.9.4 and D11.4.4P. The letter shall include the basis for the drilled shaft resistance (e.g., load tests, resistance analyses and past experience) and the relevance of field conditions and construction procedures used to develop the shaft resistance. In addition to the basic requirements of PP1.9.4 and D11.4.4P, the foundation submission letter shall include the following, as a minimum:

a. Shaft type and size (including alternates)

b. Geotechnical axial shaft resistance

c. Structural shaft resistance

d. Basis for shaft resistance determination

e. Bottom of cap elevation

COMMENTARY

C10.8.3.9.5

When concrete is placed in shafts, vibration is often not possible except for the uppermost cross-section. Vibration should not be used for high slump concrete.

C10.8.4

See DC10.5.5.3 and DC10.7.4.

SPECIFICATIONS COMMENTARY

f. Estimated tip elevations and maximum shaft lengths

g. Description of anticipated bearing stratum

h. Anticipated load carrying mechanism (point bearing, end bearing, or friction)

i. Relevant soil and/or rock conditions

j. Where applicable, foundation type of existing structure and other pertinent information

k. Downdrag analysis, if anticipated

l. COM624P or LPILE 5.0 analysis results and lateral shaft resistances

m. Evaluation of corrosion potential and design measures for protection against deterioration.

APPENDIX

A10.1 INVESTIGATION

Slope instability, liquefaction, fill settlement, and increases in lateral earth pressure have often been major factors contributing to bridge damage in earthquakes. These earthquake hazards may be significant design factors for peak earthquake accelerations in excess of 0.1 *g* and should form part of a site-specific investigation if the site conditions and the associated acceleration levels and design concepts suggest that such hazards may be of importance. Because liquefaction has contributed to many bridge failures, methods for evaluating site liquefaction potential are described in more detail below.

Liquefaction Potential—Liquefaction of saturated foundation soils has been a major source of bridge failures during historic earthquakes. For example, during the 1964 Alaska earthquake, nine bridges suffered complete collapse, and 26 suffered severe deformation or partial collapse. Investigations indicated that liquefaction of foundation soils contributed to much of the damage, with loss of foundation support leading to major displacements of abutments and piers. A study of seismically-induced liquefaction and its influence on bridges has been completed by Ferritto and Forest (*1977*) in a report to the Federal Highway Administration. A brief review of seismic design considerations for bridge foundations related to site liquefaction potential is given in Martin (*1979*). From the foundation failure documented in these reports and in the literature in general, it is clear that the design of bridge foundations in soils susceptible to liquefaction poses difficult problems. Where possible, the best design measure is to avoid deep, loose to medium-dense sand sites where liquefaction risks are high. Where dense or more competent soils are found at shallow depths, stabilization measures such as densification may be economical. The use of long ductile vertical steel piles to support bridge piers could also be considered. Calculations for lateral resistance should assume zero support from the upper zone of potential liquefaction, and the question of axial buckling would need to be addressed. Overall abutment stability would also require careful evaluation, and it may be preferable to use longer spans and to anchor abutments well back from the end of approach fills.

An additional design philosophy of bridges in liquefaction-susceptible areas might be one of “calculated risk,” at least for those bridges regarded as being less essential for communication purposes immediately after an earthquake. It may not be economically justifiable to design some bridges to survive a large earthquake without significant damage in a liquefaction environment. However, it may be possible to optimize a design so that the cost of repair of potential earthquake damage to those bridges does not exceed the cost of remedial measures and additional construction needed to avoid the damage. The approaches for determining the liquefaction potential at a site are outlined below.

A recent review of methodologies (*Seed 1979*) identifies two basic approaches for evaluating the cyclic liquefaction potential of a deposit of saturated sand subjected to earthquake shaking:

1. Empirical methods based on field observations of the performance of sand deposits in previous earthquakes and correlations between sites that have not liquefied and Relative Density of Standard Penetration Test (*SPT*) blow counts.

2. Analytical methods based on the laboratory determination of the liquefaction strength characteristics of undisturbed samples and on the use of dynamic site response analysis to determine the magnitude of earthquake-inducing shearing stresses.

Both empirical and analytical methods require the level of ground acceleration at a site to be defined as a prerequisite for assessing liquefaction potential. This is often established from relationships between earthquake magnitude, distance from the epicenter, and peak acceleration.

For conventional evaluations using a “total stress” approach, the two methods are similar and differ only in the manner in which the field liquefaction strength is determined. In the “total stress” approach, liquefaction strengths are normally expressed as the ratio of an equivalent uniform or average cyclic shearing stress, $(\tau_h)_{av}$, acting on horizontal surfaces of the sand to the initial vertical effective stress σ'_o. As a first approximation, the cyclic stress ratio, developed in the field because of earthquake ground shaking, may be computed from an equation (*Seed and Idriss 1971*), namely:

$$\frac{(\tau_h)_{av}}{\sigma'_o} = 0.65 r_d \left(\frac{a_{max}}{g}\right)\left(\frac{\sigma_o}{\sigma'_o}\right) \qquad \text{(A10.1-1)}$$

where:

a_{max} = maximum or effective peak ground acceleration at the ground surface (m/sec.^{2})

σ_o = total overburden pressure on sand layer under consideration (MPa){ksf}

σ'_o = initial effective overburden pressure on sand layer under consideration (MPa){ksf}

r_d = stress reduction factor varying from a value of 1 at the ground surface to 0.9 at a depth of 9 m

Empirical Methods—Values of the cyclic stress ratio defined by Eq. 1 have been correlated for sites that have and have not liquefied, with parameters such as relative density based on *SPT* Data *(Seed et al. 1975*; *Castro 1975*). The latest form of this type of correlation (*Seed*) is expressed in Figures 1 and 2. N_1 is the measured standard penetration resistance of the sand corrected to an effective overburden pressure of 0.096 MPa using the relationship:

$$N_1 = NC_N \qquad \text{(A10.1-2)}$$

where:

N = measured penetration resistance (blows/300 mm){blows/ft.}

C_N = correction factor from Figure 2

Thus, for a given site and a given maximum ground surface acceleration, the average stress ratio developed during the earthquake, $(\tau_h)_{av}/ \sigma'_o$, at which liquefaction may be expected to occur is expressed by the empirical correlations shown by Figure 1. The correlations for different magnitudes reflect the influence of earthquake duration on liquefaction potential. The factor of safety against liquefaction can be determined by comparing the stress ratio required to cause liquefaction with that induced by the design earthquake. It is suggested that a factor of safety of 1.5 is desirable to establish a reasonable margin of safety against liquefaction in the case of important bridge sites.

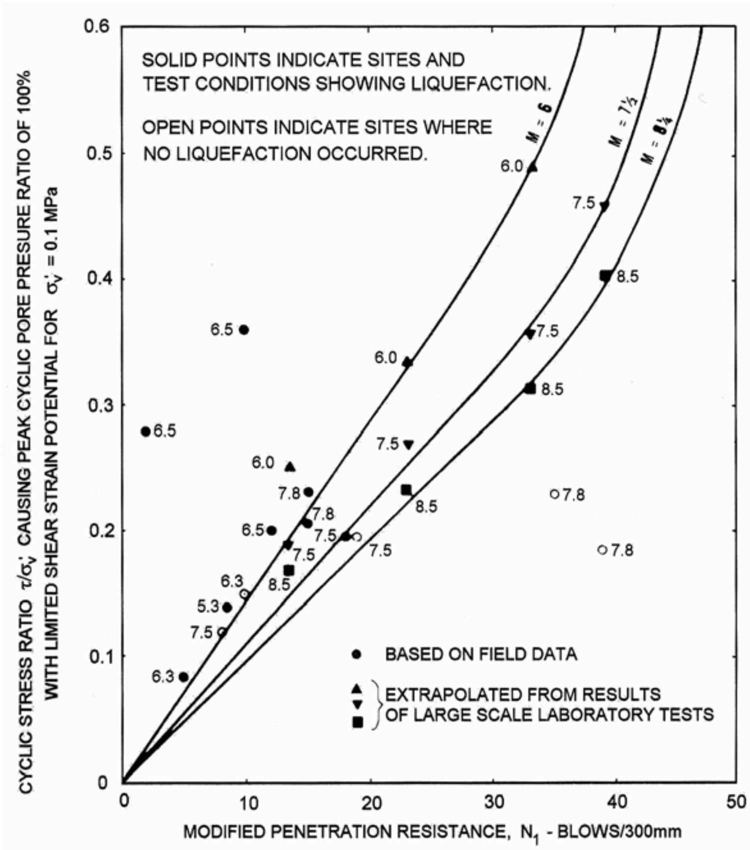

Figure A10.1-1 Correlation Between Field Liquefaction Behavior and Penetration Resistance.

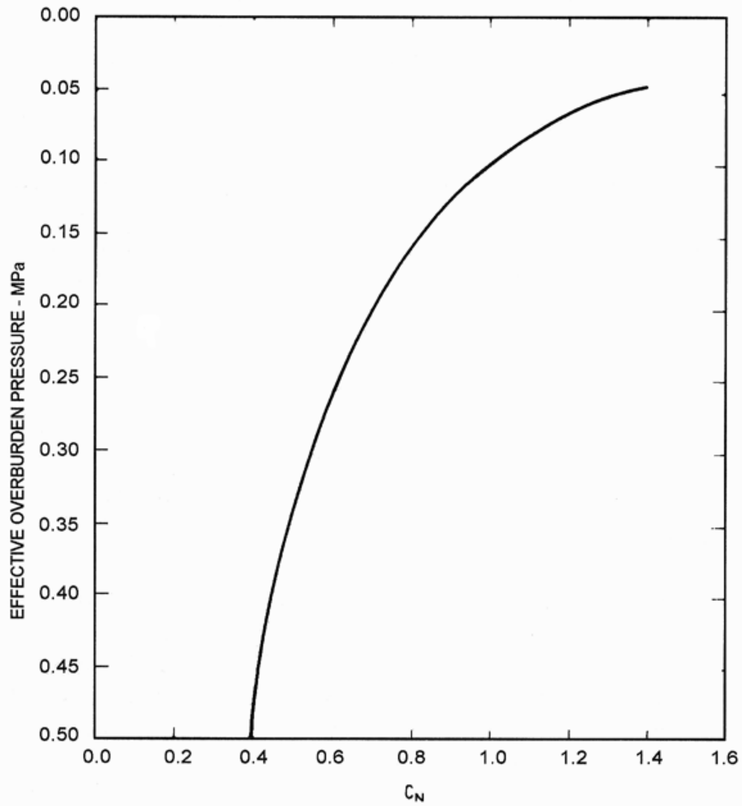

Figure A10.1-2 Relationship Between C_N and Effective Overburden Pressure.

A further extension of the empirical approach has recently been described by Dezfulian and Prager (*1978*), where a correlation between cone penetrometer tests (*CPT*) and standard penetration tests (*SPT*) has enabled *CPT* measurements in sands (expressed as point resistance q_c) to be used as a measure of liquefaction potential. *CPT* have the advantage of being more economical than *SPT*, and because they can provide a continuous record of penetration resistance with depth, potentially liquefiable thin seams of sands can be identified more readily.

Although penetration tests have the clear advantage of being a field-owned liquefaction evaluation procedure, it must always be remembered that the empirical correlation has been established from a very limited database restricted to sites comprising primarily deposits of fine silty sand. The correlation may break down for sandy silts and gravelly soils (where blow count data are difficult to interpret) and for coarser sands where partial drainage of excess pore pressures may occur during an earthquake. Furthermore, for situations where additional stresses are imposed by construction operations, care is needed in interpreting the correlation.

Analytical Methods—The analytical approach for evaluating liquefaction potential is based on a comparison between field liquefaction strengths established from cyclic laboratory tests on undisturbed samples and earthquake-induced shearing stresses. In this approach, it must be recognized that the development of a field liquefaction strength curve from laboratory tests results requires data adjustment to account for factors such as correct cyclic stress simulation, sample disturbance, aging effects, field cyclic stress history, and the magnitude of in-situ lateral stresses. These adjustments require a considerable degree of engineering judgment. Also, in many cases it is impossible to obtain undisturbed sand samples.

Once a liquefaction strength curve has been established, if a total stress analysis is used, liquefaction potential is evaluated from comparisons with estimated earthquake-induced shear stresses as shown in Figure 3.

The earthquake-induced shear stress levels may be established from a simplified procedure (*Seed and Idriss 1971*) or from more sophisticated assessments using one-dimensional “equivalent linear” dynamic response programs such as SHAKE. Average stress levels are established using the equivalent-number-of-cycles concept (approximately 10 for M7 and 30 for M8.5 earthquakes). More recently, nonlinear programs have been introduced for response calculations.

An improved representation of the progressive development of liquefaction is provided by the use of an effective stress approach (*Finn et al. 1978, 1977*; *Martin and Seed 1979*), where pore water pressure increases are coupled to nonlinear dynamic response solutions, and the influence of potential pore water pressure dissipation during an earthquake is taken into account. This approach provides data on the time history of pore water pressure increases during an earthquake, as shown in Figure 4.

It is of interest to note that a rough indication of the potential for liquefaction may be obtained by making use of empirical correlations established between earthquake magnitude and the epicentral distance to the most distant field manifestations of liquefaction. Such a relationship has been described by Youd and Perkins (*1977*) (Figure 5) and has been used as a basis for preparation of liquefaction-induced ground failure susceptibility maps.

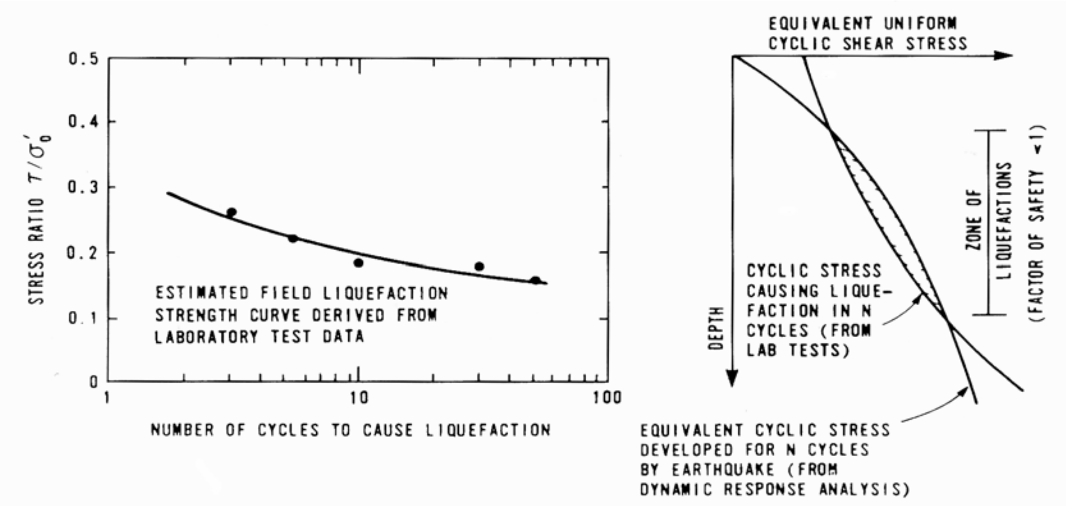

Figure A10.1-3 Principles of Analytical Approach (Total Stress) to Liquefaction Potential Evaluation.

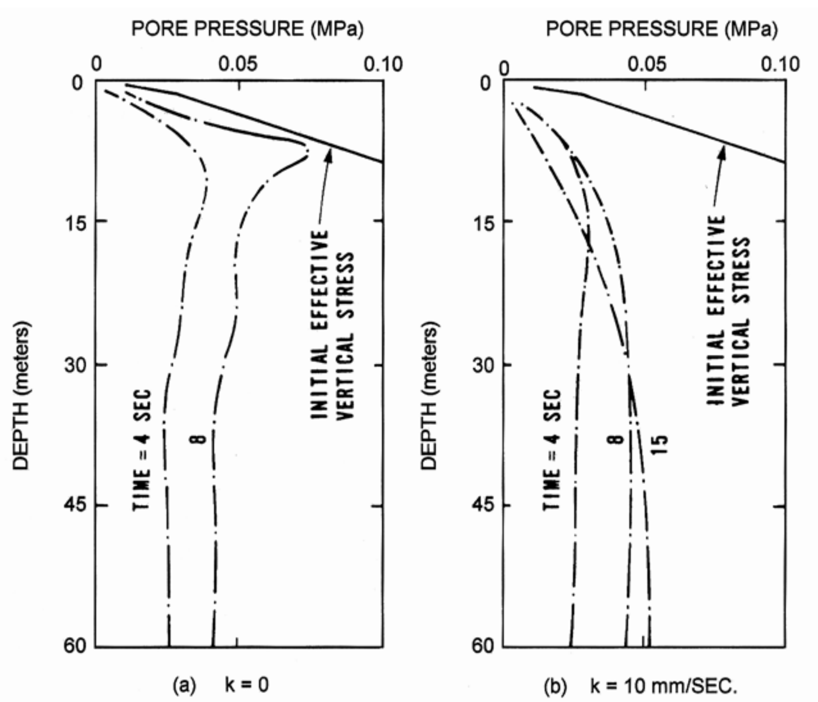

Figure A10.1-4 Effective Stress Approach to Liquefaction Evaluation Showing Effect of Permeability after Finn et al. (*1977*).

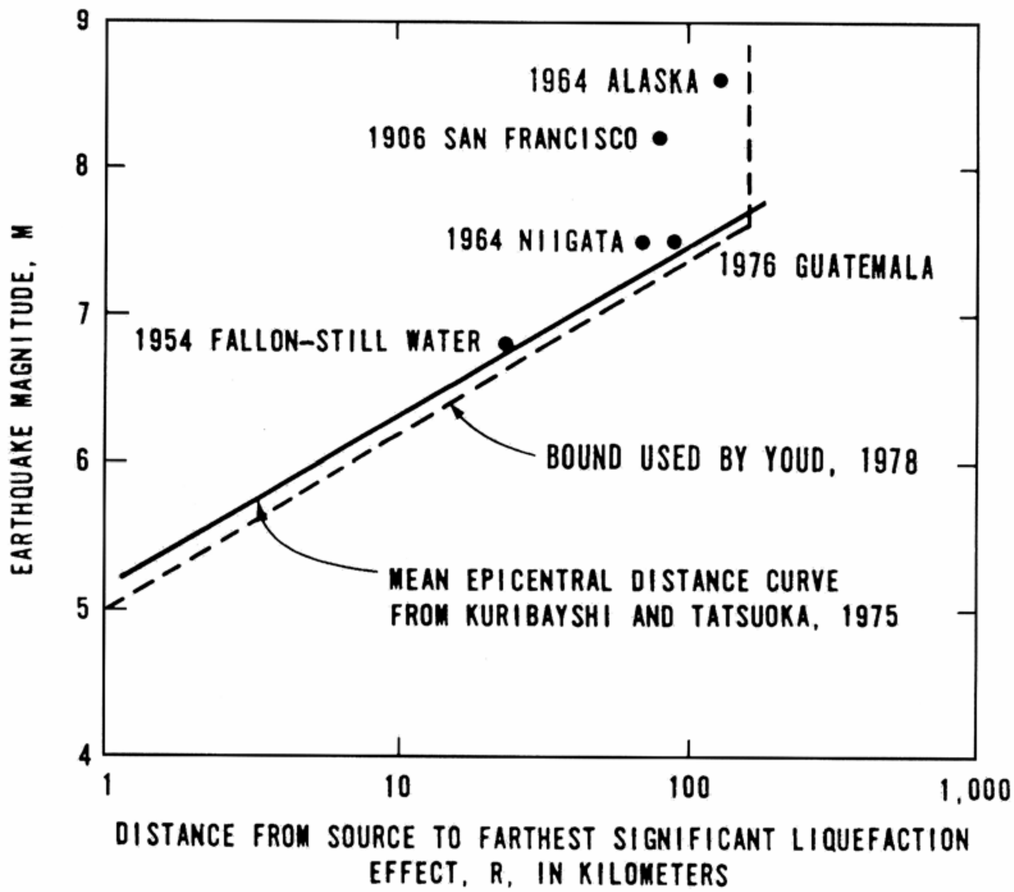

Figure A10.1-5 Maximum Distance to Significant Liquefaction as a Function of Earthquake Magnitude.

A10.2 FOUNDATION DESIGN

The commonly accepted practice for the seismic design of foundations is to utilize a pseudo-static approach, where earthquake-induced foundation loads are determined from the reaction forces and moments necessary for structural equilibrium. Although traditional bearing capacity design approaches are also applied, with appropriate capacity reduction factors if a margin of safety against "failure" is desired, a number of factors associated with the dynamic nature of earthquake loading should always be borne in mind.

Under cyclic loading at earthquake frequencies, the strength capable of being mobilized by many soils is greater than the static strength. For unsaturated cohesionless soils, the increase may be about 10 percent, whereas for cohesive soils, a 50 percent increase could occur. However, for softer saturated clays and saturated sands, the potential for strength and stiffness degradation under repeated cycles of loading must also be recognized. For bridges classified as Zone 2, the use of static soil strengths for evaluating ultimate foundation capacity provides a small implicit measure of safety and, in most cases, strength and stiffness degradation under repeated loading will not be a problem because of the smaller magnitudes of seismic events. However, for bridges classified as Zones 3 and 4, some attention should be given to the potential for stiffness and strength degradation of site soils when evaluating ultimate foundation capacity for seismic design.

As earthquake loading is transient in nature, "failure" of soil for a short time during a cycle of loading may not be significant. Of perhaps greater concern is the magnitude of the cyclic foundation displacement or rotation associated with soil yield, as this could have a significant influence on structural displacements or bending moments and shear distributions in columns and other members.

As foundation compliance influences the distribution of forces or moments in a structure and affects computation of the natural period, equivalent stiffness factors for foundation systems are often required. In many cases, use is made of various analytical solutions that are available for footings or piles where it is assumed that soil behaves in an elastic medium. In using these formulae, it should be recognized that equivalent elastic moduli for soils are a function of strain amplitude, and for seismic loads modulus values could be significantly less than those appropriate for low levels of seismic loading. Variation of shear modulus with shearing strain amplitude in the case of sands is shown in Figure 1.

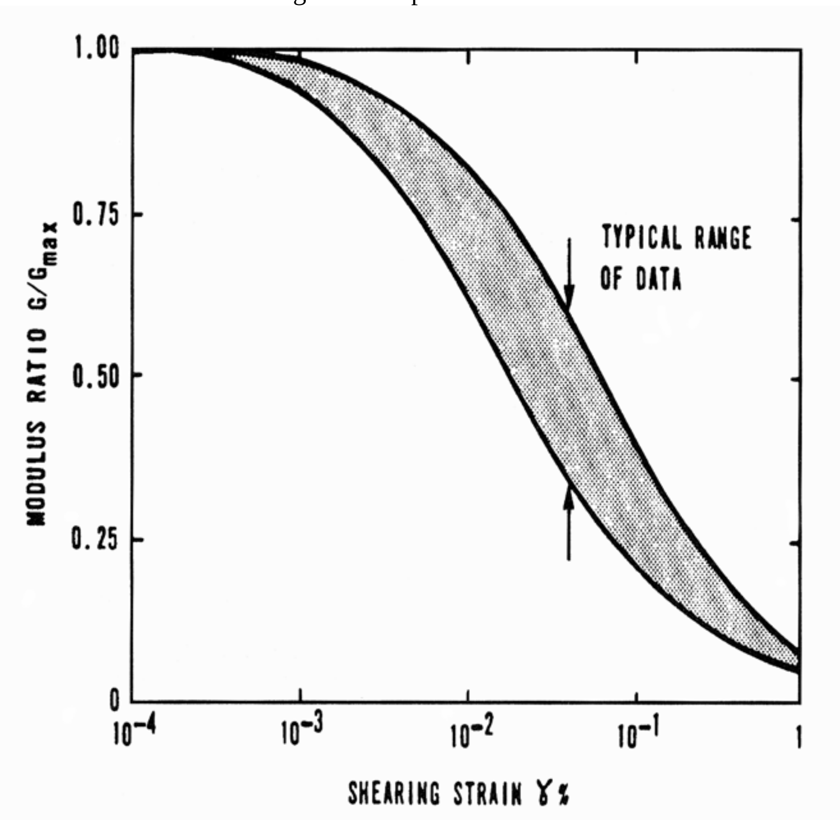

Figure A10.2-1 Variation of Shear Modulus with Shearing Strain for Sands.

On the basis of field and experimental observations, it is becoming more widely recognized that transient foundation uplift or rocking during earthquake loading, resulting in separation of the foundation from the subsoil, is acceptable provided that appropriate design precautions are taken (*Taylor and Williams 1979*). Experimental studies suggest that rotational yielding beneath rocking foundation can provide a useful form of energy dissipation. However, care must be taken to avoid significant induced vertical deformations accompanying possible soil yield during earthquake rocking as well as excessive pier movement. These could lead to design difficulties with relative displacements.

Lateral Loading of Piles—Most of the well-known solutions for computing the lateral stiffness of vertical piles are based on the assumption of elastic behavior and utilize equivalent cantilever beam concepts (*Davisson and Gill 1960*), the beam on an elastic Inkler foundation method (*Matlock and Reese 1960*), or elastic continuum solutions (*Poulos 1971*). However, the use of methods incorporating nonlinear subgrade reaction behavior that allows for soil failure may be important for high lateral loading of piles in soft clay and sand. Such a procedure is encompassed in the American Petroleum Institute (API) recommendations for offshore platform design. The method utilizes nonlinear subgrade reaction or *P-y* curves for sands and clays that have been developed experimentally from field loading tests.

The general features of the API analysis in the case of sands are illustrated in Figure 2. Under large loads, a passive failure zone develops near the pile head. Test data indicate that the ultimate resistance, p_u, for lateral loading is reached for pile deflections, y_u, of about $3d/80$, where d is the pile diameter. Note that most of the lateral resistance is mobilized over a depth of about $5d$. The API method also recognizes degradation in lateral resistance with cyclic loading, although in the case of saturated sands, the degradation postulated does not reflect pore water pressure increases. The degradation in lateral resistance due to earthquake-induced, free-field pore water pressure increases in saturated sands has been described by Finn and Martin (*1979*). A numerical method that allows the use of API *P-y* curves to compute pile stiffness characteristics forms the basis of the computer program BMCOL 76 described by Bogard and Matlock (*1977*).

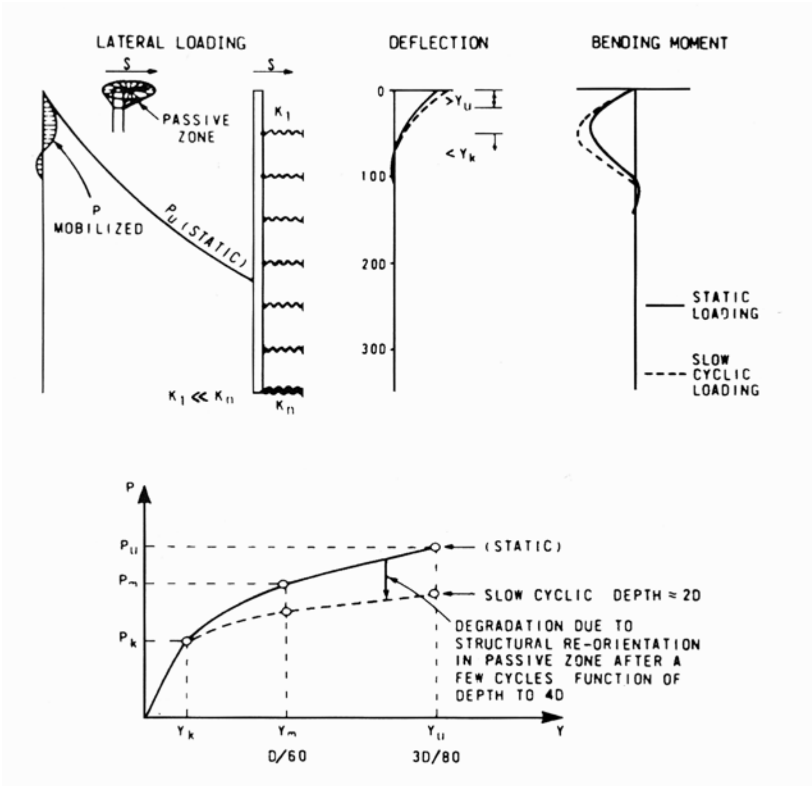

Figure A10.2-2 Lateral Loading of Piles in Sand Using API Criteria.

The influence of group action on pile stiffness is a somewhat controversial subject. Solutions based on elastic theory can be misleading where yield near the pile head occurs. Experimental evidence tends to suggest that group action is not significant for pile spacings greater than 4*d* to 6*d*.

For batter pile systems, the computation of lateral pile stiffness is complicated by the stiffness of the piles in axial compression and tension. It is also important to recognize that bending deformations in batter pile groups may generate high reaction forces on the pile cap.

It should be noted that although battered piles are economically attractive for resisting horizontal loads, such piles are very rigid in the lateral direction if arranged so that only axial loads are induced. Hence, large relative lateral displacements of the more flexible surrounding soil may occur during the free-field earthquake response of the site (particularly if large changes in soil stiffness occur over the pile length), and these relative displacements may in turn induce high pile bending moments. For this reason, more flexible vertical pipe systems where lateral load is resisted by bending near the pile heads are recommended. However, such pile systems must be designed to be ductile because large lateral displacements may be necessary to resist the lateral load. A compromise design using battered piles spaced some distance apart may provide a system that has the benefits of limited flexibility and the economy of axial load resistance to lateral load.

Soil-Pile Interaction—The use of pile stiffness characteristics to determine earthquake-induced pile bending moments based on a pseudo-static approach assumes that moments are induced only by lateral loads arising from inertial effects on the bridge structure. However, it must be remembered that the inertial loads are generated by interaction of the free-field earthquake ground motion with the piles and that the free-field displacements themselves can influence bending moments. This is illustrated in an idealized manner in Figure 3. The free-field earthquake displacement time histories provide input into the lateral resistance interface elements, which in turn transfer motion to the pile. Near the pile heads, bending moments will be dominated by the lateral interaction loads generated by inertial effects on the bridge structure. At greater depth (e.g., greater than 10*d*), where soil stiffness progressively increases with respect to pile stiffness, the pile will be constrained to deform in a manner similar to that of the free field, and pile bending moments become a function of the curvatures induced by free-field displacements.

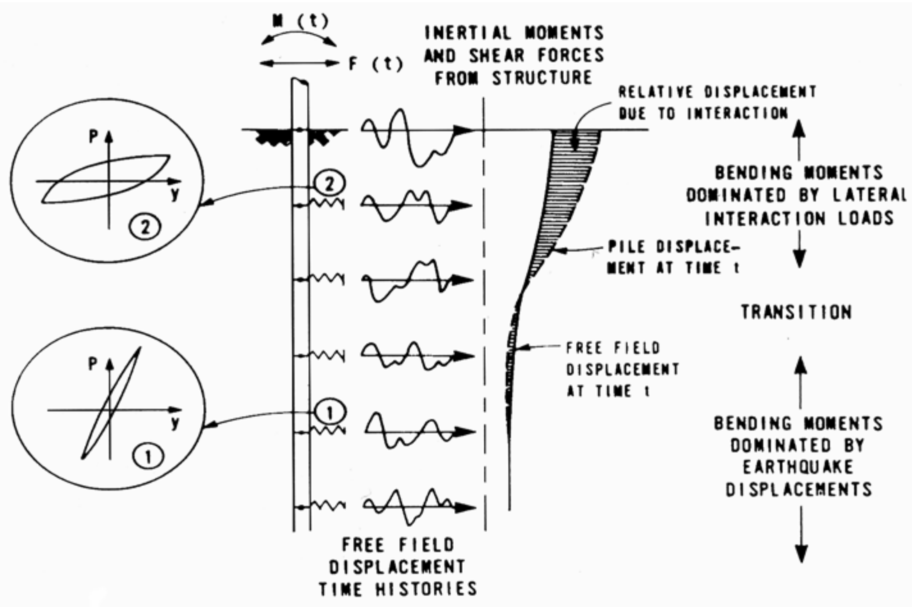

Figure A10.2-3 Mechanism of Soil-Pile Interaction During Seismic Loading.

To illustrate the nature of free-field displacements, reference is made to Figure 4, which represents a 200-ft. deep cohesionless soil profile subjected to the El Centro earthquake. The free-field response was determined using a nonlinear, one-dimensional response analysis. From the displacement profiles shown at specific times, curvatures can be computed and pile bending moments calculated if it is assumed that the pile is constrained to displace in phase with the free-field response.

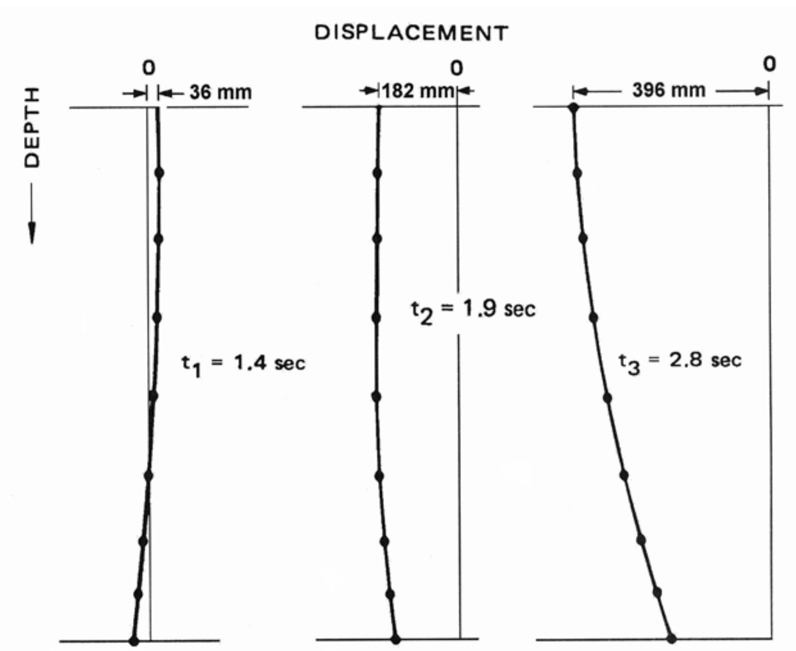

Figure A10.2-4 Typical Earthquake Displacement Profiles.

Large curvatures could develop at interfaces between soft and rigid soils and, clearly, in such cases emphasis should be placed on using flexible ductile piles. Margason (*1979*) suggests that curvatures of up to 2.36×10^{-5} mm^{-1} could be induced by strong earthquakes, but these should pose no problem to well-designed steel or prestressed concrete piles.

Studies incorporating the complete soil-pile structure interaction system, as presented in Figure 3, have been described by Penzien (*1970*) for a bridge piling system in a deep soft clay. A similar but somewhat simpler soil-pile structure interaction system (SPASM) to that used by Penzien has been described by Matlock et al. (*1978*). The model used is, in effect, a dynamic version of the previously mentioned BMCOL program.

A10.3 SPECIAL PILE REQUIREMENTS

The uncertainties of ground and bridge response characteristics lead to the desirability of providing tolerant pile and foundation systems. Toughness under induced curvature and shears is required, and hence piles such as steel H-sections and concrete filled steel-cased piles are favored for highly seismic areas. Unreinforced concrete piles are brittle in nature, so nominal longitudinal reinforcing is specified to reduce this hazard. The reinforcing steel should be extended into the footing to tie elements together and to assist in load transfer from the pile to the pile cap.

Experience has shown that reinforced concrete piles tend to hinge or shatter immediately below the pile cap. Hence, tie spacing is reduced in this area so that the concrete is better confined. Driven precast piles should be constructed with considerable spiral confining steel to ensure good shear strength and tolerance of yield curvatures should these be imparted by the soil or structural response. Clearly, it is desirable to ensure that piles do not fail below ground level and that flexural yielding in the columns is forced to occur above ground level. The additional pile design requirements imposed on piles for bridges classified as Zones 3 and 4, for which earthquake loading is more severe, reflect a design philosophy aimed at minimizing below-ground damage that is not easily inspected following a major earthquake.

SPECIFICATIONS COMMENTARY

B10.1 SCOPE

Provisions of this section shall apply for the design of micropiles used for structure foundations.

B10.2 DEFINITIONS

Battered Pile - Pile installed at an angle inclined to the vertical to provide higher resistance to lateral loads.

Bonded Length – The length of the micropile that is bonded to the rock and which is conceptually used to transfer the applied axial loads to the surrounding rock. Also known as the load transfer length.

Casing – Steel pipe introduced during the drilling process to temporarily stabilize the drill hole. Depending on the details of the micropile construction and composition, this casing may be fully extracted during or after grouting, or may remain partially or completely in place, as part of the final pile configuration.

Centralizer – A device to centrally locate the reinforcing element(s) within the borehole.

Competent Rock - A rock mass with discontinuities that are open not wider than 3.2 mm (0.13 inches).

Core Steel – Reinforcing bars or pipes used to strengthen or stiffen the pile, excluding any left-in drill casing.

Design Load (DL) – Anticipated maximum service load in the micropile.

Free (unbonded) length – The designed length of the micropile that is not bonded to the surrounding ground.

Geotechnical Bond Strength - The nominal grout-to-ground bond strength.

Load Test – Incremental loading of a micropile, recording the total movement at each increment.

Micropile – A small diameter (less than 300 mm {12 in.}), bored, cast-in-place pile, in which most of the applied load is resisted by the steel reinforcement and bonded length.

Overburden – Non-lithified material, natural or placed, which normally requires cased drilling methods to provided an open borehole to underlying strata.

Pile Bent - A type of bent using piles as the column members.

Primary Grout – Portland cement based grout that is injected into the micropile hole prior to, or after the installation of the reinforcement to provide the load transfer to the surrounding ground along the micropile and afford a degree of corrosion protection for a micropile loaded in compression.

Reinforcement – The steel component of the micropile which accepts and/or resists applied loadings.

Residual Movement – The non-elastic (non-recoverable) movement of a micropile measured during load testing.

Test Load (TL) – The maximum load to which the micropile is subjected during testing.

SPECIFICATIONS COMMENTARY

B10.3 NOTATION

The units shown after the description of each term are suggested units. Other units that are consistent with the expressions being evaluated may be used.

A_b = cross-sectional area of steel reinforcing bar (mm^2) {in^2}
A_c = cross-sectional area of steel pipe/casing (mm^2) {in^2}
A_{ct} = cross-sectional area of steel casing considering reduction for threads (mm^2) {in^2}
A_g = cross-sectional area of grout within micropile (mm^2) {in^2}
D_b = micropile bond length in bearing stratum (mm) {in}
d_b = grouted bond length diameter (mm) {in}
f_c' = specified unconfined compressive strength of micropile grout at 28 days, unless another age is specified (MPa){ksi}
f_y = specified minimum yield strength of reinforcing bar or steel casing, whichever is less (MPa){ksi}
Q_n = nominal resistance (N) {k}
Q_R = factored resistance (N) {k}
Q_s = nominal micropile shaft resistance (N){k}
Q_{ug} = nominal uplift resistance of a micropile group(N){k}
R_{cc} = factored structural axial compression resistance of cased micropile segments (N) {k}
R_{cu} = factored structural axial compression resistance of uncased micropile segments (N) {k}
R_n = nominal structural axial compression or tension resistance (N) {k}
R_{tc} = factored structural axial tension resistance of cased micropile segments (N) {k}
R_{tu} = factored structural axial tension resistance of uncased micropile segments (N) {k}
α_b = nominal micropile grout-to-ground bond strength (MPa){ksi}
φ = resistance factor
φ_{cc} = structural resistance factor for cased micropiles segments in axial compression (DIM)
φ_{cu} = structural resistance factor for uncased micropiles segments in axial compression (DIM)
φ_s = resistance factor for the shaft capacity of a micropile (DIM)
φ_{tc} = structural resistance factor for cased micropiles segments in axial tension (DIM)
φ_{tu} = structural resistance factor for uncased micropiles segments in axial tension (DIM)
φ_u = resistance factor for the uplift capacity of a single micropile (DIM)
φ_{ug} = resistance factor for the uplift capacity of micropile group (DIM)

B10.4 LIMIT STATES AND RESISTANCE FACTORS

See D10.5.1 and D10.5.2

B10.4.1 Strength Limit States

Design of micropile foundations at the strength limit state shall include:

- Axial load resistance;
- Lateral resistance; and
- Structural resistance.

Foundations shall be proportioned such that the factored resistance is not less than the effects of factored loads specified in Section 3 of the AASHTO LRFD Specifications.

SPECIFICATIONS COMMENTARY

B10.4.2 Extreme Events Limit States

Foundations shall be designed for extreme events per D10.5.5.3.

B10.4.3 Resistance factors

Resistance factors for micropile foundation systems at the strength limit state shall be taken as specified in Table 1. Resistance factors for the service limit state shall be taken as 1.0.

Table 10.4.5-1 – Resistance Factors for Geotechnical Strength Limit State for Axially Loaded Micropiles

METHOD/SOIL/CONDITION		RESISTANCE FACTOR
Axial Load Resistance of Single Micropile, φ_s	Shaft Resistance: Presumptive Values (1) Load Test (2)	 Rock = 0.60 Rock = 0.80
Uplift Resistance of Single Micropile, φ_u	Shaft Resistance: Presumptive Values (1) Load Test (2)	 Rock = 0.60 Rock = 0.80
Group Uplift Resistance, φ_{ug}	Shaft Resistance: Presumptive Values (1) Load Test (2)	 Rock = 0.60 Rock = 0.80

(1) Apply to Presumptive grout-to-ground bond values for preliminary design only in AC7.3.2
(2) Apply where preproduction load tests are conducted to a load of 1.0 or greater times the factored design load on individual micropiles
(3)

Table 10.4.5-2 – Resistance for Structural Strength Limit State for Axially Loaded Micropiles

METHOD/SOIL/CONDITION		RESISTANCE FACTOR
Pile Cased Length	Compression, φ_{cc}	0.65
	Tension, φ_{tc}	0.80
Pile Uncased Length	Compression, φ_{cu}	0.65
	Tension, φ_{tu}	0.80

B10.5 GENERAL

B10.5.1 Materials

Micropiles are to utilize steel casing that meets ASTM A-252 Grade 3, but have minimum yield strength of 550 MPa {80 ksi}. The material may be mill secondary API

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drill casing. The core reinforcing bar is to have a minimum yield of 520 MPa {75 ksi}. The neat cement grout used for the micropiles is to have a minimum 28-day unconfined compressive strength, f_c', = 28MPa {4ksi}. Micropiles may range from 125mm {5 inches} to 300mm {12 inches} in diameter.

B10.5.2 Micropile Types

In Pennsylvania, micropiles are constructed by placing a sand-cement mortar or neat cement grout in the pile under a gravity head only.

B10.5.3 Micropile Penetration

Required micropile penetration should be determined based on the resistance to vertical and lateral loads and the displacement of both the micropile and the subsurface materials.

Micropiles for trestle or pile bents shall penetrate a distance equal to at least one-third the unsupported length of the micropile.

Micropiles used to penetrate a soft or loose upper stratum overlying a hard or firm stratum, shall penetrate the firm stratum by a distance sufficient to limit movement of the micropiles per D10.5.2.2 and attain sufficient bearing resistance.

B10.5.4 Resistance

The resistance of micropiles should be determined by static analysis methods based on soil structure interaction. The resistance of micropiles should be determined through a subsurface investigation, laboratory and/or in-situ tests, analytical methods, micropile load tests, and reference to the past performance of micropiles in similar ground conditions. Consideration shall also be given to:

- The difference between the resistance of a single micropile and that of a group of micropiles;
- The capacity of the underlying strata to support the load of the micropile group;
- The possibility of scour and its effect; and
- The transmission of forces, such as negative skin friction or downdrag forces, from consolidating soil.

Micropiles are to transfer their load through grout to ground friction in the bonded length without contribution of end bearing.

Micropiles shall develop capacity in rock.

COMMENTARY

CB10.5.4

Micropiles which develop capacity in soil shall only be considered with approval from the Chief Bridge Engineer.

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B10.5.5 Effect of Settling Ground and Down Drag Loads

See D10.7.1.6.2 and D10.7.3.7.

B10.5.6 Micropile Spacing, Clearances and Embedment

Micropile spacing, clearances and embedment into the foundation is in accordance with D10.7.1.2. Group effects must be considered for micropiles spaced closer than 750mm {30 in.} center to center.

The center to center spacing of micropiles should be greater than 3.0 micropile diameters or the spacing required to avoid interaction between adjacent micropiles. Larger spacings may be required where drilling operations are anticipated to be difficult.

If closer spacings are required, the sequence of construction shall be specified in the contract documents, and the interaction effects between adjacent micropiles shall be evaluated.

The connection between micropiles and footings shall be designed to distribute structure loads and overturning moments to all micropiles in a group. Where a reinforced concrete beam is cast-in-place and used as a bent cap supported by micropiles, the concrete cover at the sides of the micropiles shall be greater than 150 mm {6 in.}.

B10.5.7 Battered Micropiles

Battered micropiles shall be used in the foundation to resist lateral loads.

B10.5.8 Groundwater Table and Buoyancy

Resistance shall consider the effects of groundwater level consistent with that used to calculate load effects. The effect of hydrostatic pressure shall be considered in the design.

B10.5.9 Protection Against Deterioration

As a minimum, the following types of deterioration shall be evaluated:

- Corrosion of steel, particularly in fill soils, low pH soils, and marine environments;
- Sulfate, chloride, and acid attack of concrete and cement grout.

Refer to D10.7.5 for determination of corrosive environment. Minimum corrosion protection/sacrificial thickness for micropiles shall be in accordance with

COMMENTARY

CB10.5.6

The factored load effect acting on any micropile in a group may be estimated using the traditional elastic strength of material procedure for a cross-section under thrust and moment.

CB10.5.7

Use of vertical micropiles to resist lateral loads shall only be considered with the approval from the Chief Bridge Engineer.

SPECIFICATIONS

COMMENTARY

D10.7.5.2 Steel Piles. For measures to protect piles against deterioration refer to D10.7.5.

B10.5.10 Uplift

Micropile foundations designed to resist uplift forces should be checked for resistance to pullout and structural ability to carry tensile stress in accordance with D10.7.1.6.3 and D10.7.3.10.

CB10.5.10

Uplift forces can be caused by lateral loads, buoyancy effects, and expansive soils. The connection of the micropile to the footing is part of its structural ability to resist uplift and should also be investigated.

B10.5.11 Estimated Micropile Lengths

Estimated micropile lengths for each substructure shall be shown on the plans in accordance with PP1.6.4.11, and shall be based on careful evaluation of available subsurface information, static and lateral capacity calculations, and/or past experience.

B10.5.12 Estimated and Minimum Tip Elevations

See A10.7.6.

Estimated and minimum micropile tip elevations for each substructure should be shown on the contract plans.

B10.5.13 Micropiles Through Embankment Fill

Micropiles extending through embankments shall penetrate a minimum of 3 m {10 ft.} through original ground unless competent bearing occurs at a lesser penetration.

B10.5.14 Test Micropiles and Load Tests

Test micropiles shall be installed at each substructure to determine micropile installation characteristics, evaluate micropile capacity with depth, and establish contractor micropile order lengths. One static load test per substructure unit is to be performed. The maximum test load must be no less than two (2) times the micropile unfactored design load (DL). The number of test micropiles required may be increased in non-uniform subsurface conditions. The number of test micropiles may be reduced where previous experience exists with the same micropile type and ultimate micropile capacity, similar subsurface conditions, similar installation methods and contractor personnel experience. However, the number of test micropiles and load test may only be reduced with the Chief Bridge Engineer's approval.

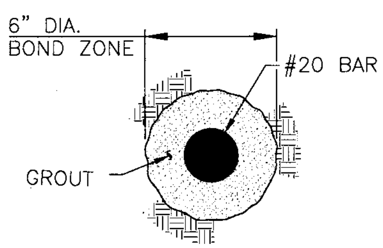
SPECIFICATIONS COMMENTARY

B10.5.15 Micropiles Notes and Sketches

The following notes shall be placed on the General Notes sheet of the contract drawings:

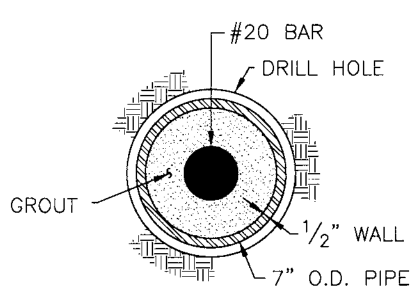
- Each individual pile bond zone must contain an accumulation of X' of rock.
- Each bond zone must be a minimum of Y' in length.
- No one soil seam in excess of 5' will be acceptable.
- No bond zone shall be terminated without 2.0' (min.) of rock at the bottom of the bond zone.
- Each bond zone must be extended as necessary to a length that includes an accumulation of Z' of rock.

{The X, Y, and Z values are based on calculations and subsurface conditions}



SECTION B–B'

NO SCALE



SECTION A–A'

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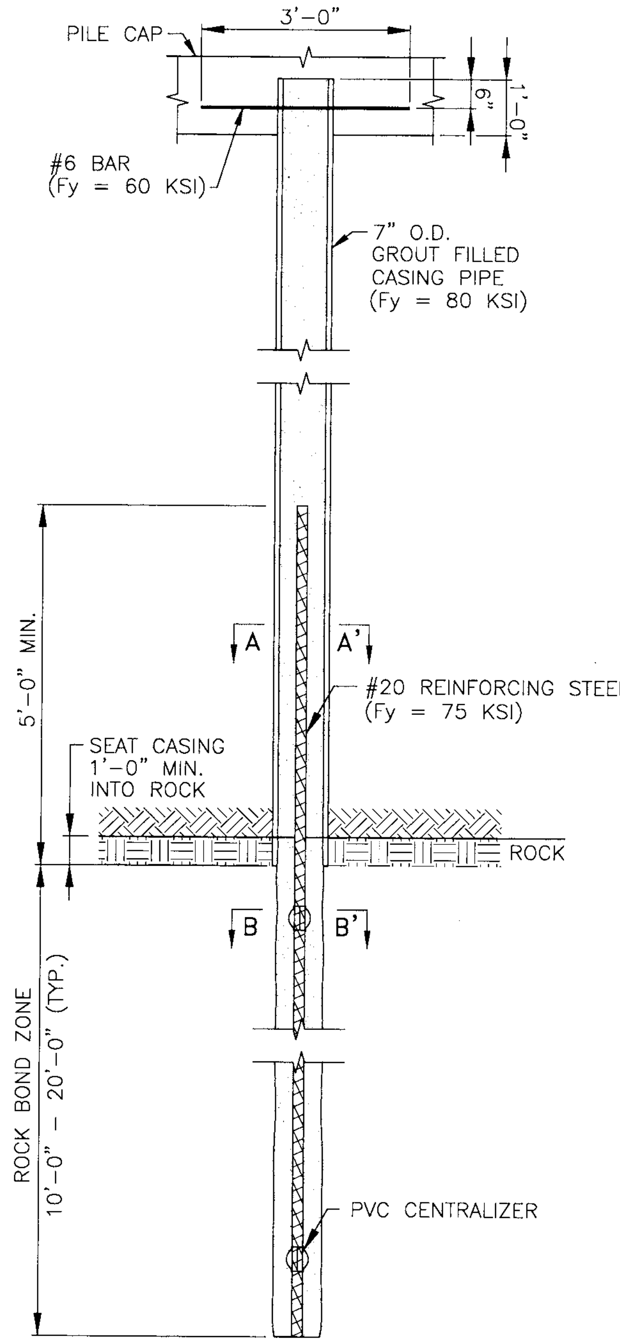

MICRO–PILE IN ROCK

NO SCALE

SPECIFICATIONS

B10.6 MOVEMENT AND BEARING RESISTANCE AT THE SERVICE LIMIT STATE

B10.6.1 Settlement

See D10.7.2

The settlement of a micropile foundation shall not exceed the tolerable settlement, as selected according to DM-4. Refer to D10.7.2.2 for criteria for horizontal displacement.

B10.6.2 Lateral Displacement

The lateral displacement of micropile groups shall be estimated using procedures that consider soil structure interaction, in accordance with D10.7.2.4.

B10.6.3 Uplift

Tension in micropiles is not permitted at the service limit state.

B10.7 GEOTECHNICAL RESISTANCE AT THE STRENGTH LIMIT STATE

B10.7.1 General

Consider the following:

- Axial load resistance of micropiles,
- Uplift resistance of micropiles, and
- Structural resistance of the micropiles.

B10.7.2 Axial Loading of Micropiles

Tip resistance shall be neglected for micropile design.

Structural resistance (compression strength) of the bonded zone generally controls the maximum load.

B10.7.3 Estimates of Micropile Axial Resistance

B10.7.3.1 GENERAL

Micropiles shall be designed to resist failure of the bonded length in rock. The resistance factors for the shaft resistance (grout-to-ground bond) are specified in Table B10.4.5-1.

COMMENTARY

CB10.7.1

See DC10.7.3.1, and as specified herein.

CB10.7.3.1

In Pennsylvania, micropiles are designed considering only grout-to-ground bond (shaft) resistance.

SPECIFICATIONS

B10.7.3.2 SHAFT RESISTANCE

The shaft resistance over the uncased, bonded length below casing tip of a micropile is computed as:

$$Q_R = \varphi Q_n = \varphi_s Q_s = \varphi_s (\pi d_b \alpha_b D_b) \qquad \text{(B10.7.3.2-1)}$$

where:

Q_s = nominal micropile shaft resistance (N){k}.

d_b = diameter of micropile drill hole through bonded length (mm) {in}.

α_b = nominal micropile grout-to-ground bound strength (MPa){ksi}.

D_b = micropile bonded length (mm){in.}.

φ_s = resistance factor for shaft resistance specified in Table A4.5-1

The shaft resistance of micropiles may be based on the results of micropile load test; estimates based on a review of geologic and boring data, soil and rock samples, laboratory testing and previous experience; or estimated using published grout to ground bond guidelines. For final design, micropile capacity shall be verified through the performance of micropile load tests as described in B10.5.14.

COMMENTARY

CB10.7.3.2

The value of nominal unit grout-to-ground bond strength, either estimated empirically or determined through load testing, is typically taken as the average value over the entire bond length.

Micropile grout-to-ground bond strength is influenced by soil and rock conditions, method of micropile drilling and installation, and grouting pressure. As a guide, Table CB10.7.3.2-1 may be used to estimate the nominal (ultimate) unit grout-to-ground bond strength.

Table CB10.7.3.2-1 – Summary of Typical α_b Values (Grout-to-Ground Bond) for Preliminary Micropile Design (modified after Armour, et al. 2000)

Rock Description	Typical Range of Grout-to-Ground Bond Strength (MPa){ksi}
Soft Shales (fresh-moderate fracturing, little to no weathering)	0.20 – 0.55 {0.03 – 0.08}
Slates and Hard Shales (fresh-moderate fracturing, little to no weathering)	0.52 – 1.38 {0.08 – 0.20}
Limestone (fresh-moderate fracturing, little to no weathering)	1.04 – 2.07 {0.15 – 0.30}
Sandstone (fresh-moderate fracturing, little to no weathering)	0.52 – 1.72 {0.08 – 0.25}
Granite and Basalt (fresh-moderate fracturing, little to no weathering)	1.38 – 4.20 {0.20 – 0.61}

B10.7.3.3 TIP RESISTANCE ON ROCK

Tip resistance on rock shall not be considered.

B10.7.4 Uplift

B10.7.4.1 GENERAL

Uplift shall be considered when the force effects, calculated based on the appropriate strength limit state load combinations, are tensile.

SPECIFICATIONS

When micropiles are subjected to uplift, they should be investigated for both resistances to pullout and structural ability to resist tension and transmit it to the footing.

B10.7.4.2 SINGLE MICROPILE UPLIFT RESISTANCE

The uplift resistance of a single micropile shall be estimated based on the shaft resistance of micropiles specified in B10.7.3.2.

Factored uplift resistance shall be taken as:

$$Q_R = \varphi Q_n = \varphi_u Q_s \quad \text{(B10.7.4-2-1)}$$

where:

φ_u = resistance factor for uplift capacity specified in Table B10.4.5-1
Q_s = nominal micropile shaft resistance (N){k}(7.3.2)

B10.7.4.3 MICROPILE GROUP UPLIFT RESISTANCE

Micropile group factored uplift resistance shall be taken as:

$$Q_R = \varphi Q_n = \varphi_{ug} Q_{ug} \quad \text{(B10.7.4.3-1)}$$

where:
φ_{ug} = Resistance factor specified in Table B10.4.5-1
Q_{ug} = Nominal uplift resistance of the group (N){k}

The uplift resistance, Q_{ug} of a micropile group shall be taken as the lesser of:

- The sum of the individual micropile uplift resistances, or

- The uplift capacity of the micropile group considered as a block.

B10.7.5 Lateral Resistance

Micropiles subject to lateral loads are to be designed in accordance with D10.7.3.12.2 and D6.15P. Lateral resistance of micropile groups is to be in accordance with D10.7.3.12.3.

COMMENTARY

CB10.7.4.2

The preliminary design of micropiles subjected to tension loading may be based on the estimated nominal unit grout-to-ground bond strengths presented in Table CB10.7.3.2-1.

CB10.7.4.3

Group Uplift resistance in rock should consider depth of soil overburden, rock discontinuity spacing and condition, and rock mass shear strength, as well as bond between micropiles and rock.

SPECIFICATIONS COMMENTARY

B10.8 STRUCTURAL RESISTANCE AT THE STRENGTH LIMIT STATE

B10.8.1 General

The cased and uncased length of each micropile shall be designed to resist the forces distributed to the micropile based on the micropile inclination and spacing. The resistance factors for structural design shall be as specified in Table B10.4.5-2.

B10.8.1.1 AXIAL COMPRESSION RESISTANCE

The upper, cased section of a micropile subjected to compression loading shall be designed structurally to support the full factored load on the micropile.

For micropiles extending through a weak upper soil layer, extending above ground, subject to scour, extending through mines/caves or extending through soil that may liquefy, the effect of any laterally unsupported length shall be considered in the determination of axial compression resistance.

B10.8.1.1.1 Cased Length

A portion of the cased length of the micropile will consist of the casing and grout. The lower portion will contain the reinforcing bar which extends through the bonded zone.

The factored structural resistance of the upper cased length of a micropile having no unsupported length and loaded in compression, Rcc, may be taken as:

$$R_{cc} = \varphi_c R_n = \varphi_{cc} \left[0.85\, f'_c\, A_g + f_y \left(A_b + A_c \right) \right] \quad \text{(B10.8.1.1.1-1)}$$

where:

f_c' = specified compressive strength of micropile grout at 28 days unless another age is specified. Maximum for design purposes is 28MPa {4ksi}, regardless of actual grout compressive strength. (MPa){ksi}

A_g = cross-sectional area of grout within micropile pipe/casing $(mm^2)\{in^2\}$

F_y = specified minimum yield strength of reinforcement bar or steel pipe/casing, whichever is less (MPa){ksi}

A_b = cross-sectional area of steel reinforcing bar $(mm^2)\{in^2\}$

A_c = cross-sectional area of steel pipe/casing $(mm^2)\{in^2\}$

SPECIFICATIONS

B10.8.1.1.2 Uncased Length

The uncased length consists of the bonded zone of the micropile and contains grout and reinforcing bar. The factored structural resistance of the lower uncased length of a micropile having no unsupported length and loaded in compression, R_{cu}, may be taken as:

$$R_{cu} = \varphi_{cu} R_n = \varphi_u (0.85 f_c' A_g + f_y A_b) \qquad \text{(B10.8.1.1.2-1)}$$

where:

- φ_{cu} = resistance factor specified in Table B10.4.5-2 for structural resistance of the uncased section of micropile subjected to compression loading
- f_c' = specified compression strength of micropile grout at 28 days unless another age is specified (MPa){ksi}
- A_g = cross-sectional area of grout in bonded length (mm^2){in^2}
- f_y = specified minimum yield strength of reinforcement bar (MPa){ksi}
- A_b = cross-sectional area of steel reinforcing bar (mm^2){in^2}

B10.8.1.2 AXIAL TENSION RESISTANCE

The upper, cased section of a micropile subjected to tension loading shall be designed structurally to support the full factored load on the micropile. The lower uncased section of a micropile subjected to tension loading shall be designed structurally to support the full factored load on the micropile.

B10.8.1.2.1 Cased Length

The factored structural resistance of the upper, cased length of a micropile subjected to tensile loading, R_{tc}, may be taken as:

$$R_{tc} = \varphi_{tc} R_n = \varphi_{tc} [f_y(A_b + A_{ct})] \qquad \text{(B10.8.1.2.1-1)}$$

where:

- φ_{tc} = resistance factor specified in Table B10.4.5-2 for structural resistance of the cased section of a micropile subjected to tension loading
- f_y = specified minimum yield strength of reinforcement bar or steel pipe/casing, whichever is less (MPa){ksi}
- A_b = cross-section area of steel reinforcing bar (mm^2){in^2}
- A_{ct} = cross-section area of steel pipe/casing, considering reduction for threads (mm^2){in^2}

COMMENTARY

CB10.8.1.2.1

Micropiles used to resist axial tension loads in excess of 20 percent of the pipe/casing thread strength generally incorporate a full-length reinforcement bar designed to support the full factored load.

SPECIFICATIONS

B10.8.1.2.2 Uncased Length

The factored structural resistance of the lower, uncased length of a micropile having no unsupported length and loaded in tension, R_{tu}, may be taken as:

$$R_{tu} = \varphi_{tu} R_n = \varphi_{tu} f_y A_b \quad \text{(B10.8.1.2.2-1)}$$

where:

φ_{tu} = resistance factor specified in Table B10.4.5-2 for structural resistance of the uncased section of a micropile subjected to tension loading

f_y = specified minimum yield strength of reinforcement bar (MPa){ksi}

A_b = cross-sectional area of steel reinforcing bar (mm^2){in^2}

B10.8.1.3 PLUNGE LENGTH

A one foot plunge length (embedment of casing into bond length) is to be provided. However, no additional load transfer due to the plunge length may be incorporated into the design.

B10.8.1.4 GROUT-TO-STEEL BOND

Casing-to-grout bond shall be checked, and reinforcement bar development length shall be in accordance with the provisions of Section 5 of the AASHTO LRFD Specifications.

B10.8.2 Buckling of Micropiles

Micropiles that extend through water or air shall be checked for buckling in accordance with D5.13.4.7.4P

COMMENTARY

CB10.8.1.4

Grout-to-steel bond does not typically govern micropile design, except for overlap of reinforcing bars into upper pipe/casing.

The bond between the cement grout and the reinforcing steel is the mechanism for transfer of the pipe load from the reinforcing steel to the ground. Typical ultimate bond values range from 1.0 to 1.75 Mpa {0.15 to 0.25 ksi} for smooth bars and pipe/casing, and 2.0 to 3.5 MPa {0.30 to 0.50 ksi} for deformed bars. Refer to Section 5 of the AASHTO LRFD Specifications for bar development requirements.

As is the case with any requirement, the surface condition will affect the attainable bond. A film of rust may be beneficial, but the presence of loose debris or lubricant or paint is not desirable. Normal methods for the handling and storage of reinforcing bars applies to micropile construction. For the permanent pipe/casing that is also used to drill the hole, cleaning of the pipe/casing surface can occur during drilling, particularly in granular soil.

C10.1 SCOPE

Estimated values for allowable bearing pressure on rock developed for working stress design are presented in Table 1. These values may be used for preliminary sizing of foundations.

TABLE C10.1-1 Summary of Allowable Bearing Values for Rock

Lithology	Uniaxial Compressive Strength C_o (tsf)	Range of Allowable Bearing Pressure, q_{all} (tsf)				
		RQD %				
		$\leq$ 60	70	80	90	100
Sandstone	720-1,800	5-15	15-46	30-75	40-80	55-100
Siltstone	145-575	2-5	4-15	6-25	8-35	12-50
Shale	70-370	1-4	2-10	3-15	5-25	6-30
Claystone	15-70	0.1-0.6	0.4-2	0.5-3	1-4	1-6
Limestone	1,080-2,160	8-15	25-50	45-90	65-100	100
Gneiss	720-2,160	6-15	15-50	30-90	40-100	40-100
Schist	145-720	1-6	4-15	6-30	8-40	12-55

Notes:

1. The allowable bearing values are applicable for rock core having recoveries of 90 percent or greater. Lower recoveries could indicate the presence of clay seams, very weak rock zones or voids which may control the bearing capacity of the rock mass. If the recovery is less than 90 percent, lower bearing values shall be used to reflect these conditions.

2. The range of allowable bearing values corresponds to the range in uniaxial compressive strength of the rock, with lower bearing values corresponding to the lower rock strength, and higher bearing values corresponding to the higher rock strength limited to the allowable bearing stress in the concrete. The bearing values were developed by assuming a rock mass cohesion, c, equal to 10 percent of C_o, a rock mass angle of internal friction, ϕ_{fm}, equal to zero, and a factor of safety of 2.5.

3. The allowable bearing values shall be determined based on the condition of material within 2B below isolated rectangular footings and within 4B below strip footings (i.e., L > 5B). Where rock strata within these zones of influence of footings are variable, the material having the weakest capacity will control the allowable bearing pressure of footings.

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PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 1
PART B: DESIGN SPECIFICATIONS

SECTION 11 - ABUTMENTS, PIERS AND WALLS

SECTION 11 - TABLE OF CONTENTS

SPECIFICATIONS COMMENTARY

11.1 SCOPE

The following shall replace A11.1:

Provisions of this section shall apply for the design of abutments, piers and retaining walls. Retaining wall types addressed include:

- Rigid gravity and semi-gravity walls,
- Anchored walls,
- Mechanically-stabilized earth (MSE) walls,
- Prefabricated modular walls, and
- Nongravity cantilevered walls

Abutments, piers and retaining walls shall be designed for all applicable loads, including, but not limited to, lateral earth and water pressures, including any live and dead load surcharge, impact loads, the self weight of the wall, temperature and shrinkage effects, and earthquake loads.

Retaining walls shall be designed considering the potential long-term effects of corrosion, seepage, stray currents and other potentially deleterious environmental factors on each of the material components comprising the wall. Retaining walls are considered permanent if the service life is more than 36 months. Permanent retaining walls shall be designed for a minimum service life of 100 years to retain an aesthetically pleasing appearance, and be essentially maintenance free throughout their design service life. Retaining walls for temporary applications are designed for a service life of 36 months or less.

11.1.1P Use

Selection of wall type shall be based on an assessment of the magnitude and direction of loading, depth to suitable foundation support, potential for earthquake loading, presence of deleterious environmental factors, proximity of physical constraints, tolerable and differential settlements, facing appearance, and ease and cost of construction and maintenance.

Only approved MSE and prefabricated modular wall systems shall be used. See Bulletin 15 and Appendix K for approved wall systems.

SPECIFICATIONS COMMENTARY

11.1.1.1P ABUTMENTS

11.1.1.1.1P Stub Abutment

Stub abutments are located at or near the top of approach fills, with a backwall depth sufficient to accommodate the structure depth and bearings which sit on the bearing seat.

In general, a stub abutment without a slope wall shall have a minimum 1200 mm {4 ft.} wide bench in the fill immediately in front of the abutment. At locations where a slope wall is desirable, the bench shall be omitted, and construction of the slope wall shall conform to the requirements of the Department's standard specifications and standard drawings. See Figure 1 for typical details.

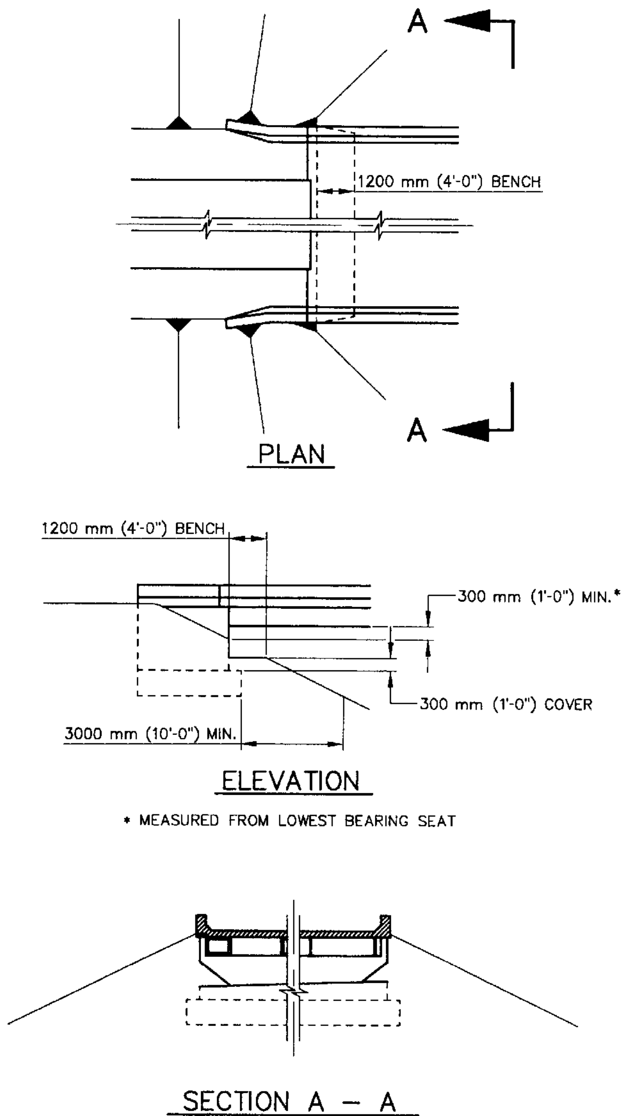

Figure 11.1.1.1.1P-1 - Recommended Details for Stub Abutments

SPECIFICATIONS COMMENTARY

11.1.1.1.2P Partial Depth Abutment

Partial depth abutments are located approximately at mid-depth of the front slope of the approach embankment. The higher backwall and wingwalls may retain fill material, or the embankment slope may continue behind the backwall. In the latter case, a structural approach slab or end span design must bridge the space over the fill slope, and curtain walls are provided to close off the open area. Inspection access should be provided for this situation.

11.1.1.1.3P Full-Depth Abutment

Full-depth abutments are located at the approximate front toe of the approach embankment, restricting the opening under the structure.

11.1.1.1.4P Integral Abutment

Integral abutments are rigidly attached to the superstructure and are supported on a deep foundation capable of permitting necessary horizontal movements.

11.1.1.2P PIERS

11.1.1.2.1P Solid Wall Piers

Solid wall piers are designed as columns for forces and moments acting about the weak axis and as piers for those acting about the strong axis. They may be pinned, fixed or free at the top, and are conventionally fixed at the base. Short, stubby types are often pinned at the base to eliminate the high moments which would develop due to fixity. Earlier, more massive designs were considered gravity types.

11.1.1.2.2P Double Wall Piers

More recent designs consist of double walls, spaced in the direction of traffic, to provide support at the continuous soffit of concrete box superstructure sections. These walls are integral with the superstructure and must also be designed for the superstructure moments which develop from live loads and erection conditions. The use of double wall piers shall be subject to the approval of the Chief Bridge Engineer.

11.1.1.2.3P Bent Piers

Bent-type piers consist of two or more transversely spaced columns of various solid cross-sections, and these types are designed for frame action relative to forces acting about the strong axis of the pier. They are usually fixed at the base of the pier and are either integral with the superstructure or with a pier cap at the top. The columns may be supported on a spread- or pile-supported footing, or a solid wall shaft, or they may be extensions of the piles or shaft above the ground line.

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11.1.1.2.4P Single Column Piers

Single column piers, often referred to as "T" or "Hammerhead" piers, are usually supported at the base by a spread- or pile-supported footing, and may be either integral with, or provide independent support for, the superstructure. Their cross-section may be of various shapes and the column can be prismatic or flared to form the pier cap or to blend with the sectional configuration of the superstructure cross-section. This type pier can avoid the complexities of skewed supports if integrally framed into the superstructure and their appearance reduces the massiveness often associated with superstructures.

11.1.1.2.5P Tubular Piers

The configuration may be as described in D11.1.1.2.1P through D11.1.1.2.4P. Because of their vulnerability to lateral loadings, tubular piers shall be of sufficient wall thickness to sustain the forces and moments for all loading situations as are appropriate. Prismatic configurations may be sectionally precast or prestressed as erected. Tubular piers of hollow core section may be of steel, reinforced concrete or prestressed concrete, of such cross-section to support the forces and moments acting on the elements. The use of tubular piers shall be subject to the approval of the Chief Bridge Engineer.

11.1.1.3P RIGID GRAVITY AND SEMI-GRAVITY WALLS

Rigid retaining walls of stone masonry or concrete construction may be used for bridge substructures to support earth slopes adjacent to roadways, or for grade separations. Rigid retaining walls include gravity, semi-gravity, cantilevered and counterfort walls. These wall types shall be used for permanent applications.

11.1.1.4P ANCHORED WALLS

Anchored walls are applicable for temporary and permanent support of stable and unstable soil and rock masses. Anchors are usually required for support of both temporary and permanent nongravity cantilevered walls higher than about 5000 mm {15 ft.}, depending on soil conditions, and for stabilizing gravity structures, where applicable.

Anchored walls are typically constructed in cut situations

COMMENTARY

C11.1.1.2.5P

When reinforced concrete tubular piers are subjected to seismic loading, the pier stem may not provide adequate confinement of the vertical reinforcing steel.

C11.1.1.4P

Anchored walls may be used for widening roads or construction grade separations in stable ground masses and for stabilizing land slides in unstable ground masses.

Anchored walls are not applicable for all ground conditions, and consideration should be given to limitations when evaluating the feasibility of using an anchored wall for a particular site. In general, poor wall performance has been associated with unsatisfactory behavior of anchors installed in caving materials (if casing is not used), organic soils, and cohesive soils with a Plasticity Index greater than 20%, which may be susceptible to creep. The feasibility of using anchors bonded in creep-sensitive soils should be evaluated in a precontract test program. Anchors should not be bonded in organic soils.

Anchored walls have been used to support earth fills.

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in which construction occurs from the top down to the base of the wall. Anchored walls in fill situations require the approval of the Chief Bridge Engineer.

Anchors may be prestressed ground anchors or dead-man-type elements comprised of tendons or bars extending from the wall face to a grouted zone or mechanical anchorage located beyond the zone of soil applying load to the wall.

Anchors can be installed in existing rigid gravity and semi-gravity retaining walls to provide additional resistance to sliding and/or overturning.

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However, because of the difficulties which arise in constructing anchored walls to support earth fills, their use is restricted and normally other wall types are used. The design of anchored walls for fill application must include provisions for protection of the anchors from damage during backfill operations or due to subsequent backfill and foundation soil settlements. This can be accomplished by providing ungrouted protective casing or pipe around the anchors through their unbounded length to isolate the anchors from the surrounding soil.

To prevent excessive lateral wall deflections due to anchor stressing prior to the completion of backfill placement (i.e., when the backfill does not provide sufficient passive resistance to prevent undesirable wall deflections), grouted casing or struts can be used between the wall and natural ground or rock as a reaction mechanism to prevent distress.

11.1.1.5P MECHANICALLY STABILIZED EARTH WALLS

MSE walls may be used where conventional gravity, cantilever, or counterforted concrete retaining walls are feasible. These walls are particularly well suited where substantial total and differential settlements are anticipated. The allowable settlement of MSE walls is limited by the longitudinal deformability of the facing and the ultimate purpose of the structure.

When constructed on fills, the embankment between original ground and the leveling pad shall be composed of a granular fill in conformance with Publication 408, Section 206.2.2(b), or rock.

The backfill requirements must be per Standard Special Provision c80201.

C11.1.1.5P

All designs must be based on equal, sound and compatible design principles. MSE walls are subject to the same external stability design criteria as conventional retaining walls, independent of the type of reinforcing system utilized. The structure must be stable with respect to sliding, overturning, foundation bearing failure and overall slope stability.

In spite of some unique design features associated with each proprietary system, several design similarities exist among all systems. A mechanically stabilized retaining wall system has three major components: reinforcements, backfill and facing elements. The reinforcement is described by the type of material used and the reinforcement geometry. The backfill used within the reinforced zone is granular to meet stress transfer, durability and drainage requirements. Facing elements are provided to retain fill material at the face. Typical facing elements include precast concrete panels with or without architectural treatments.

11.1.1.6P PREFABRICATED MODULAR WALLS

Prefabricated modular wall systems, whose elements may be proprietary, generally employ interlocking soil-filled reinforced concrete or steel modules or bins, which resist earth pressures by acting as gravity retaining walls. Prefabricated modular systems may be used where conventional gravity, cantilever or counterfort concrete retaining walls are considered.

The proprietary precast modular systems which are approved for use, subject to the design requirements and

C11.1.1.6P

Concrete modular systems are designed as conventional gravity retaining walls developing lateral resistance primarily from self weight. In general, the full weight of the soil in the module cannot be relied upon in estimating resistance against overturning. The portion of soil weight effective in resisting overturning is limited to that weight which can be transferred in friction to the surrounding modules. Concrete modular retaining walls approximate the elements of solid gravity walls in their usual proportions, and satisfy Coulomb's lateral earth

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limitations contained in this manual, are given in Appendix K.

Metallic modular systems shall not be used.

The backfill requirements must be per Standard Special Provision c80221.

11.1.1.7P NONGRAVITY CANTILEVERED WALLS

Nongravity cantilevered walls may be used for the same applications as rigid gravity and semi-gravity walls, as well as temporary or permanent support of earth slopes, excavations, or unstable soil and rock masses. Nongravity cantilevered walls are generally limited to a maximum height of 5000 mm {15 ft.}, unless they are provided with additional support by means of anchors. For these cases, a complete structural analysis is required. Use of permanent nongravity cantilevered walls in fill situations requires the approval of the Chief Bridge Engineer.

Permanent nongravity cantilevered walls may be constructed of reinforced concrete and/or metals. Temporary nongravity cantilevered walls may be constructed of reinforced concrete, metal and/or timber. Suitable metals generally include steel and galvanized steel for components, such as piles, anchor head assemblies, brackets and plates, lagging and concrete reinforcement.

11.1.1.8P TEMPORARY EXCAVATION SUPPORT FOR STRUCTURES

The following are some guidelines for preparing the contract documents for projects that will require temporary excavation support for structures.

- The engineer should verify that shoring is required and that a shoring system can be designed and constructed at the locations indicated.

- The engineer should show the approximate locations of the shoring with a simple line diagram (i.e. do not show as sheet piling). Do not show stations and offsets for the wall limits, lengths or heights or other dimensions regarding the wall. The plans must show constraints that the contractor must follow in designing and detailing the wall such as R.O.W., horizontal clearances to highway or railroad, etc.

- The engineer is to provide the applicable soil properties in the standard special provision "TEMPORARY EXCAVATION AND SUPPORT PROTECTION SYSTEM." For soil properties that are not applicable indicate as "Not Applicable".

- The contractor is responsible for the design, limits, location, structure type, details, etc. (essentially this is a design-build item). The contractors should be given the

COMMENTARY

pressure criteria which require that the failure wedge be bounded on one side by the pressure surface and on the other side by the surface of rupture.

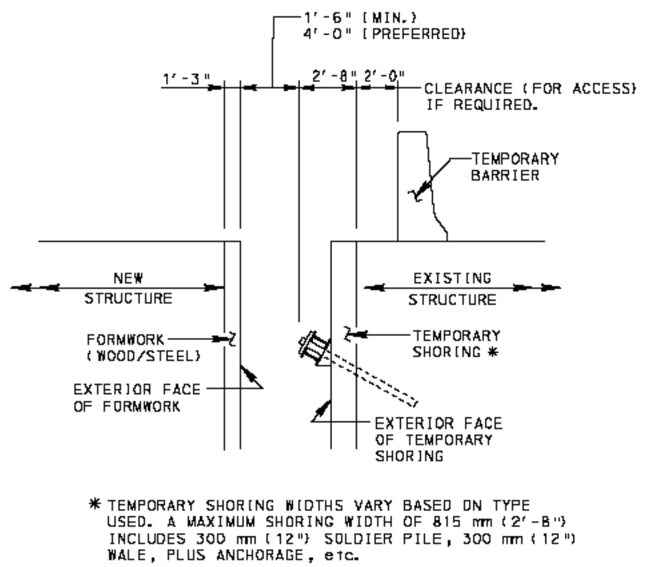
C11.1.1.7P

Typical nongravity cantilevered walls include soldier-pile and lagging and sheet pile walls.

Stiffer wall elements, such as concrete posts or large steel beams up to 5000 mm {15 ft.} in height and embedded properly in concrete caissons of sufficient designed diameter in very stiff subgrades such as dense sand and gravel or stiff clays or rock have been performing satisfactorily

C11.1.1.8P

To facilitate necessary construction activities such as installing formwork, the designer shall provide adequate working clearances when locating temporary support structures.



The amount or working clearance will vary by structure and should be based on the environment of the related structure(s). Common practices have established that allowing for a minimum of 460 mm {1'-6"}, the preferred value is 1220

SPECIFICATIONS

freedom to install the most economical shoring system based on safe and economical methods and operations.

COMMENTARY

mm {4’-0”} clear between temporary support construction, and any adjacent structure/feature, is considered adequate for construction of formwork and the movement of equipment. Special circumstances may increase this clear distance.

Conventional practices for the construction of temporary support structures located near or adjacent to railroad facilities indicate clearances of no less than 3660 mm {12 ft.} clear from centerline of tangent trackage up to 4570 mm {15 ft.} clear from centerline for non-tangent trackage.

11.2 DEFINITIONS

The following shall replace the 4th bullet under Abutment

Integral Abutment-Integral abutments are rigidly attached to the superstructure and are supported on deep foundations capable of permitting necessary horizontal movements.

11.4 SOIL PROPERTIES AND MATERIALS

11.4.1 General

The following shall replace A11.4.1.

Cohesionless soils with a maximum fines content of 5% by weight shall be used for backfill. This criteria can be met by AASHTO No. 57 coarse aggregate or open graded subbase (OGS) conforming to the requirements of Publication 408, Section 703.

C11.4.1

Delete AC11.4.1.

11.4.3P Subsurface Exploration and Testing Programs

The provisions of D10.4 shall apply.

For major retaining walls (i.e., H. 9000 mm) {30 ft.}), borings shall be taken at approximately 30 000 mm {100 ft.} intervals along their alignment and at selected locations in back of the wall face to determine the nature and strength of the soils encountered. For anchored walls, borings must include sampling of soil or rock into which the anchors will be bonded.

11.4.4P Foundation Submission

Sufficient information shall be developed to comply with PP1.9.4. As a minimum, the foundation submission letter shall address the following:

- Interpretation of all subsurface and laboratory investigation results to develop foundation and earth support design parameters
- Basis for estimation of all soil/rock strengths and other design parameters
- Suitable type and depth of foundations and bearing resistance of foundation soil and rock, and method used to estimate ultimate bearing resistance

SPECIFICATIONS

- Required foundation improvement(s) and recommended method(s) (e.g., extent of unsuitable material to be removed)
- Drainage requirements
- Acceptable alternative retaining wall systems
- Maximum estimated settlement during construction and service life
- Scour depth for each substructure unit at stream crossings
- Presence of corrosive soil or groundwater conditions

11.5 LIMIT STATES AND RESISTANCE FACTORS

11.5.1 General

The following shall supplement A11.5.1.

The design service life for retaining walls, abutments and piers is 100 years.

11.5.2 Service Limit States

The following shall replace A11.5.2

Abutments, piers and walls shall be investigated for excessive displacement at the service limit state in accordance with D10.6.2, D10.7.2, D11.6.2, D11.9.3, A11.10.4 and A11.11.3. For design, the horizontal movement at the top of abutment footings shall not exceed 12 mm {1/2 in.}.

COMMENTARY

C11.5.2

Delete the last sentence of the 3rd paragraph of AC11.5.2.

The following shall supplement AC11.5.2.

A MSE wall with welded wire or geosynthetic facing is not permitted for permanent walls.

11.5.4 Resistance Requirement

The following shall replace the first paragraph of A11.5.4.

Abutments, piers, retaining structures, their foundations and other supporting elements shall be proportioned, as specified in A11.6, A11.7, A11.9, A11.10, A11.11, D11.6, D11.7, D11.8, D11.9, D11.10, D11.11 and D11.12P, so that their resistance satisfies A11.5.5.

C11.5.4

The following shall replace AC11.5.4.

Procedures for calculating nominal resistance are provided in A11.6, A11.7, A11.9, A11.10, A11.11, D11.6, D11.7, D11.8, D11.9, D11.10, D11.11 and D11.12P for abutments and retaining walls, piers, anchored walls, mechanically-stabilized earth walls, prefabricated modular walls, gabion walls, and nongravity cantilevered walls, respectively.

11.5.6 Resistance Factors

The following shall replace Table 11.5.6-1 in A11.5.6.

Abutments, piers, retaining structures, their foundations and other supporting elements shall be proportioned, as specified in A11.6, A11.7, A11.8, A11.9, A11.10, D11.6, D11.7, D11.8, D11.9, D11.10, D11.11P, and D11.12P, so that their resistance satisfied A11.5.5.

C11.5.6P

Refer to Appendix A of LRFD, Section 11, in Barker, et al, (1991) regarding the selection of performance factors as a function of reliability.

Discrete vertical elements, such as soldier piles, should be treated as individual deep foundation elements. Continuous vertical elements, such as tangent piles and slurry trench concrete walls, should be treated as shallow strip footings.

Table 11.5.6-1 - Resistance Factors for Retaining Walls

WALL TYPE AND CONDITION		RESISTANCE FACTOR
Abutments and Conventional Walls		
Bearing resistance		D10.5.5 applies
Sliding resistance		D10.5.5 applies
Sliding Resistance (Construction Condition Only)	Precast concrete placed on sand • using φ_f estimated from SPT data • using φ_f estimated from CPT data	 1.00 1.00
	Concrete cast-in-place on sand • using φ_f estimated from SPT data • using φ_f estimated from CPT data	 0.90 0.90
	Sliding on clay is controlled by the strength of the clay when the clay shear is less than 0.5 times the normal stress, and is controlled by the normal stress when the clay shear strength is greater than 0.5 times the normal stress (see Figure 1). Clay (where shear resistance is less than 0.5 times normal pressure) • using shear resistance measured in lab tests • using shear resistance measured in field tests • using shear resistance estimated from CPT data Clay (where the resistance is greater than 0.5 times normal pressure)	 0.95 0.95 0.90 0.95
Nongravity Cantilevered and Anchored Walls(1)		
Bearing resistance of vertical elements		D10.5.5 applies
Passive resistance of vertical elements	Passive resistance of vertical elements in soil or rock • active earth pressure loading • at-rest earth pressure loading	 1.00 0.90
Pullout resistance of anchors	Anchor pullout resistance • Cohesionless Soil correlation with SPT resistance – corrected for overburden pressure pullout load tests • Cohesive Soil correlation with unconfined compressive strength using shear strength from lab tests using shear strength from field tests pullout load tests • Rock presumptive values using minimum shear resistance measured in lab tests - soft rock only laboratory rock-grout bond tests pullout load tests	 0.55 0.65 0.55 0.55 0.55 0.55 0.45 0.50 0.60 0.65

<table>
<tr><th colspan="2">WALL TYPE AND CONDITION</th><th>RESISTANCE FACTOR</th></tr>
<tr><td>Abutments and Conventional Walls</td><td>• Reinforced Concrete (A5.5.4.2.1)
• Steel (A6.5.4.2)
• Timber (A8.5.2.2)</td><td>0.90
1.00
0.85</td></tr>
<tr><td colspan="3">Mechanically Stabilized Earth Walls[(1)]</td></tr>
<tr><td colspan="2">Bearing resistance</td><td>D10.5.5 applies</td></tr>
<tr><td colspan="2">Sliding (Soil on Soil)</td><td>1.00</td></tr>
<tr><td>Tensile resistance of metallic reinforcement</td><td>Strip reinforcements
• Yielding of gross section less sacrificial area
• Fracture of net section less sacrificial area

Grid reinforcements
• Yield of gross section less sacrificial area
• Fracture of net section less sacrificial area

Connectors
• Yielding of gross section less sacrificial area
• Fracture of net section less sacrificial area</td><td>
0.85
0.70

0.75
0.60

0.75
0.60</td></tr>
<tr><td colspan="2">Ultimate pullout resistance</td><td>0.9</td></tr>
<tr><td colspan="3">Prefabricated Modular Walls[(1)]</td></tr>
<tr><td colspan="2">Bearing</td><td>D10.5.5 applies</td></tr>
<tr><td colspan="2">Sliding (Concrete on Soil or Rock)</td><td>D10.5.5 applies</td></tr>
<tr><td colspan="2">Sliding (Soil on Soil)</td><td>1.00</td></tr>
<tr><td colspan="2">Passive resistance</td><td>D10.5.5 applies</td></tr>
</table>

(1)Proprietary wall types or wall components on conventional wall types must be approved in accordance with PP1.14 "System Approval" prior to incorporation into any project.

SPECIFICATIONS COMMENTARY

11.6 ABUTMENTS AND CONVENTIONAL RETAINING WALLS

11.6.1 General Considerations

11.6.1.2 LOADING

The following shall replace the first paragraph of A11.6.1.2.

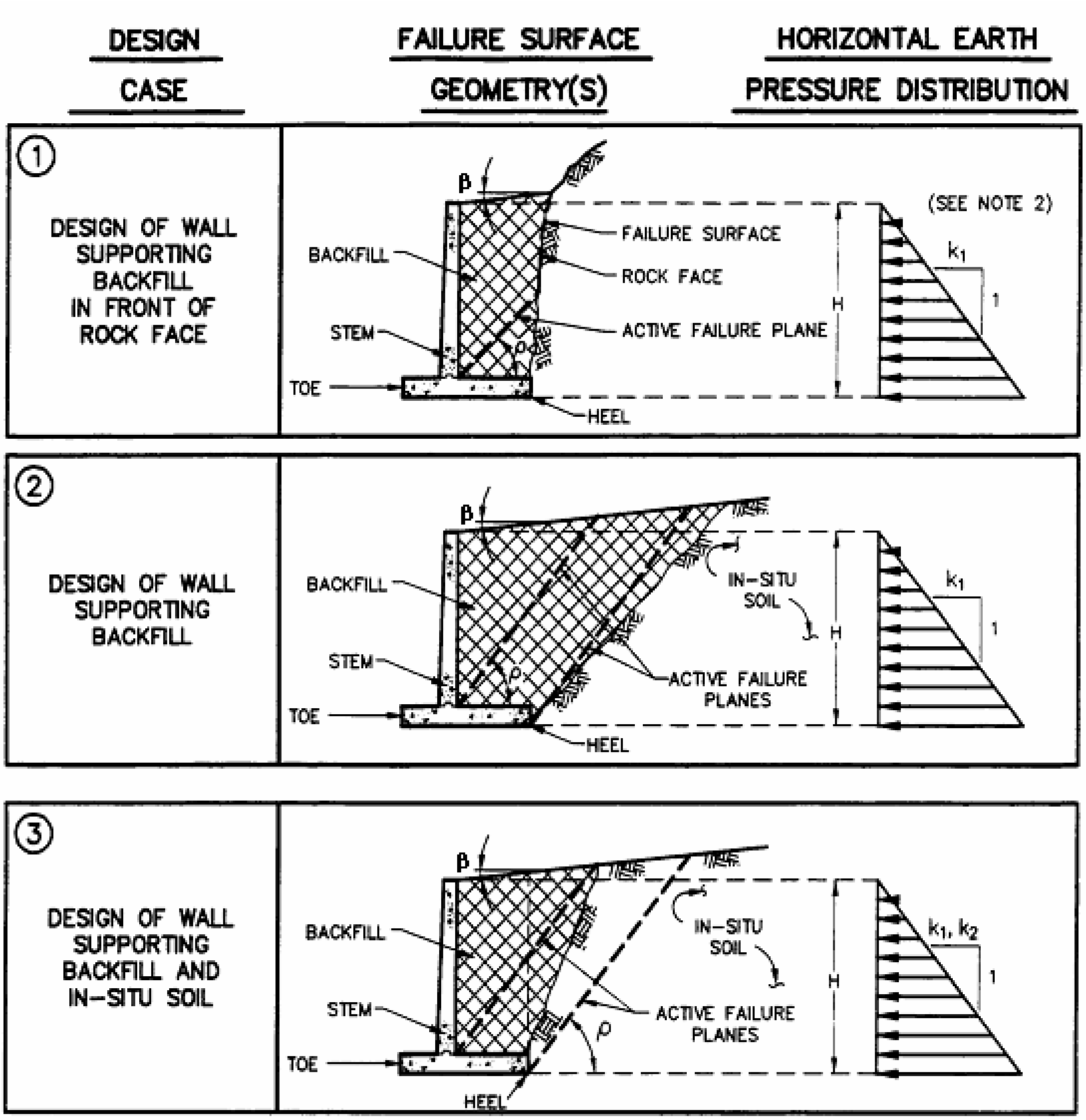
Abutments and retaining walls shall be investigated for all applicable load combinations from A3.4.1, including, as a minimum:

- lateral earth and water pressures, including any live and dead load surcharge,
- the self weight of the wall,
- temperature and shrinkage deformation effects, and
- earthquake loads, as specified herein, in AASHTO and DM-4, Section 3, and elsewhere in these Specifications.

The following shall supplement A11.6.1.2.

The magnitude of lateral earth pressure appropriate for evaluating safety against structural and soil failure is controlled by the geometry of the soil backfill as illustrated in Figure 1. In some instances (e.g., Case 3 in Figure 1), structural design of the stem is controlled by the backfill, whereas evaluation of soil failure is controlled by the in-situ soil. Where a wall is located in front of a rock face (e.g., Case 1 in Figure 1), the magnitude of lateral earth pressure may be less than that determined using the methods in A3.11.5 and D3.11.5. For such cases, Culmann's graphical procedure may be used to determine the magnitude and location of the lateral earth pressure resultant (Terzaghi and Peck, 1967).

SPECIFICATIONS COMMENTARY



LEGEND

k_1 = UNIT HORIZONTAL SOIL PRESSURE DUE TO BACKFILL

k_2 = UNIT HORIZONTAL SOIL PRESSURE DUE TO IN-SITU SOIL

ρ = ANGLE BETWEEN ACTIVE FAILURE PLANE AND HORIZONTAL

ϕ'_f = WEIGHTED AVERAGE EFFECTIVE STRESS ANGLE OF INTERNAL FRICTION ALONG FAILURE PLANE

$\rho \approx (45° + \phi'_f/2) - \beta$ (SEE HUNT, 1986)

NOTES:

(1) DETERMINE EARTH PRESSURE IN ACCORDANCE WITH A3.11.5.

(2) THE EARTH PRESSURE RESULTANT FOR THIS CONDITION CAN BE MORE ACCURATELY DETERMINED BY CULMANN'S GRAPHICAL CONSTRUCTION [SEE TERZAGHI AND PECK (1967)].

(3) ADD PRESSURES DUE TO WATER AND SURCHARGE (INCLUDING LIVE LOAD SURCHARGE) IN ACCORDANCE WITH A3.11.3 AND A3.11.6, RESPECTIVELY.

Figure 11.6.1.2-1 - Failure Surface and Horizontal Earth Pressure Distribution for Cantilever Retaining Walls

For temporary (construction) conditions, wind load on the abutment may be neglected, and loads considered shall include the following:

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- Dead load of abutment up to bridge seat elevation.

- Dead load of backfill up to bridge seat elevation.

- Lateral earth pressure and live load surcharge.

Walls shall be designed for a minimum live load surcharge (LS) equal to 900 mm {3 ft.} of soil, or the actual surcharge determined in accordance with A3.11.6.4, whichever is greater.

11.6.1.3 INTEGRAL ABUTMENTS

The following shall supplement A11.6.1.3.

The design of integral abutments, including applicable limits is to be in accordance with D11.6.4.6P. Construction specifications for integral abutments shall preclude placement of backfill until the superstructure is erected and connected to the abutments.

11.6.1.5.2 Wingwalls

Delete A11.6.1.5.2

No key joint is required between wingwall and abutment wall per the Standard Drawings.

11.6.1.6 EXPANSION AND CONTRACTION JOINTS

The following shall replace A11.6.1.6

Construction joints shall be provided at intervals not exceeding 9000 mm {30 ft.} and expansion joints at intervals not exceeding 27 000 mm {90 ft.} for gravity or reinforced concrete walls. All joints shall be filled with approved filling material to ensure the function of the joint. Joints in abutments shall be located approximately midway between the longitudinal members bearing on the abutments.

11.6.1.7P BACKWALLS AND END DIAPHRAGMS

Backwalls shall be provided as follows:

- For each abutment where the girder depth exceeds 1800 mm {6 ft.}

- For the fixed end abutment if the top of deck movement caused by rotation due to live and impact loads exceeds 15 mm {0.5 in.}, or

- For the expansion end abutment if the top of deck movement caused by temperature change relative to fall only from 20º C {68º F}, plus rotation due to live and impact loads, exceeds 15 mm {0.5 in.}

Provide full-depth end diaphragms where backwalls are

COMMENTARY

C11.6.1.6P

An exception may be made for BRADD generated drawings, where construction joints shall be provided at intervals not exceeding 13 500 mm {45 ft.}.

C1.6.1.7P

A 15 mm {0.5 in.} (maximum) crack between the full-depth diaphragm and approach flexible pavement or approach slab is easily maintainable if there is a crack sealing program.

Generally, an 1800 mm {6 ft.} girder depth (approximately 2100 mm {7 ft.} total superstructure depth) with a 30-degree skew will result in an approximate 15 mm {0.5 in.} movement at the fixed end. Similarly, a 27 000 mm {90 ft.} steel span with a 1500 mm {5 ft.} superstructure depth and a 30º skew will result in an approximate15 mm {0.5 in.} movement at the expansion end. For a prestressed concrete bridge, an approximate 15 mm {0.5 in.} movement at the expansion end will control for a bridge having a superstructure depth of 2100 mm {7 ft.}, span of 45 000 mm {150 ft.}, and skew of 30º.

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COMMENTARY

not required.

Deck deflections shall be estimated in accordance with D2.5.2.6.2.

The construction specifications shall require that deck concrete be placed prior to construction of the end diaphragms or top of the backwall to permit compensation for the beam end rotation caused by the weight of the deck.

11.6.2 Movement at the Service Limit State

The following shall replace A11.6.2.1 and A11.6.2.2.

Criteria for tolerable movement of abutments and retaining walls shall be developed based on the function and type of wall, anticipated service life and consequences unacceptable movements. Vertical displacements shall be estimated in accordance with the provisions of D10.6.2.4, D10.7.2.3 and D10.8.2.2, as applicable, and the maximum vertical displacement shall be limited to 25 mm {1.0 in.}. Horizontal displacements at the top of the foundation shall be estimated in accordance with the provisions of D10.7.2.4 and D10.8.2.3, as applicable, and the maximum horizontal displacement shall be limited to 12 mm {0.5 in.}. Tilting or translation of walls above the foundation level may be estimated using information provided in AC3.11.1.

11.6.2.3 OVERALL STABILITY

The following shall supplement A11.6.2.3.

The provisions of D10.6.2.5 shall apply.

11.6.3 Bearing Resistance and Stability at the Strength Limit State

SPECIFICATIONS

11.6.3.1 GENERAL

The following shall replace A11.6.3.1.

Abutments and retaining walls shall be proportioned to ensure stability against bearing capacity failure, overturning and sliding. Where a wall is supported by clayey foundation, safety against deep-seated foundation failure shall also be investigated.

When considering temporary conditions during construction (i.e., backfill placed prior to superstructure erection), cantilevered abutments may be evaluated for stability against overturning and sliding failure using the reduced loadings and increased resistance factors defined in D11.6.1.2, D11.5.6, D11.6.3.3, and D11.6.3.6.

COMMENTARY

C11.6.3.1

Common dimensions for conventional retaining walls are shown in Figure C1.

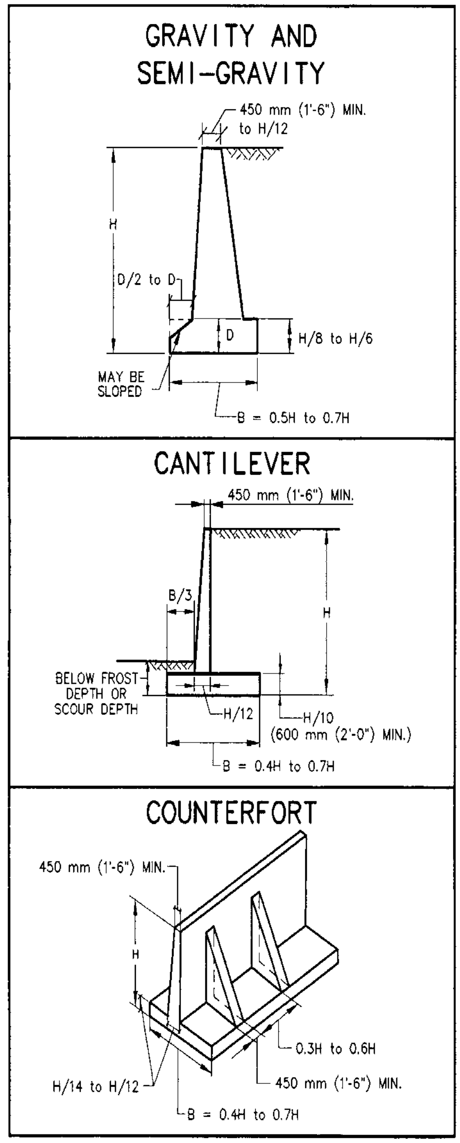

Figure C11.6.3.1-1 - Common Dimensions for Conventional Retaining Walls (after Hunt, 1986)

11.6.3.2 BEARING RESISTANCE

The following shall replace A11.6.3.2.

Bearing resistance shall be investigated at the strength limit state, assuming the following soil pressure distributions:

- if the wall is supported by a soil foundation, or rock modeled as soil:

  a uniformly distributed pressure over the effective base

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area, as shown in Figure 1.

if the wall is supported by a hard rock foundation:

a linearly varying distribution of pressure over the effective base area, as shown in Figure D10.6.5-1.

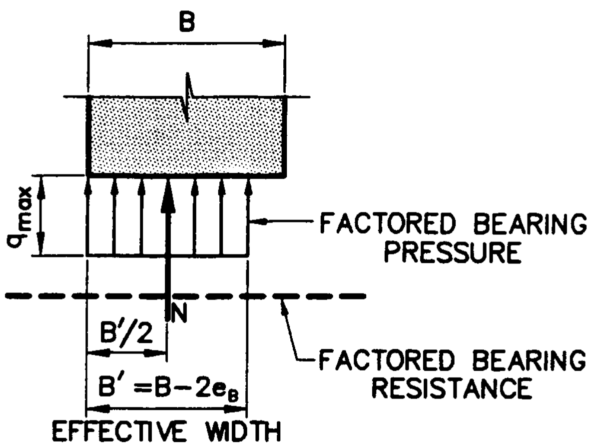

Figure 11.6.3.2-1 - Bearing Resistance Criteria for Walls with Granular Backfills and Foundations on Soil and Soft Rock, Modified after Duncan, et al, (1990)

11.6.3.3 OVERTURNING

The following shall supplement A11.6.3.3.

For evaluation of temporary conditions during construction of cantilevered abutments on soil, the location of the resultant of reaction forces shall be within the middle two-thirds of the base.

11.6.3.4 SUBSURFACE EROSION

The following shall replace A11.6.3.4.

Refer to PP7.2 for scour requirements.

For cases where differential water levels may occur in front of and behind a wall, the potential for piping shall be evaluated. The hydraulic gradient for such cases shall not exceed the following (Harr, 1962):

COMMENTARY

C11.6.3.3

The following shall supplement AC11.6.3.3

Plastic deformation of foundation soils at the strength limit state results in a redistribution of the contact stress to a more uniform bearing pressure.

Base pressure resultants are maintained within the specified limits to provide the maximum possible effective foundation bearing area and minimum bearing pressures. For resultants outside of the middle third of the base for unfactored loads, a portion of the base is separated from the foundation soil or rock and is not effective in bearing. The base pressure is then redistributed over the remaining area of the base, which is in compression, resulting in higher bearing pressures and possibly tensile stresses in the concrete. The indicated limits reflect that, for identical unfactored loads, the resultant eccentricity will increase when the loads are factored.

C11.6.3.4

The following shall supplement AC11.6.3.4

The use of protective filters to protect against piping failure of soils includes use of geotextile materials in conformance to Publication 408, Section 212.

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- Fine sand and nonplastic silt (SP, SM, ML) 0.15
- Medium to coarse sand (SP) 0.20
- Sand and gravel (GW, SW) 0.30

11.6.3.6 SLIDING

The following shall supplement A11.6.3.6.

For evaluation of temporary conditions during construction of cantilevered abutments, the resistance factors in Table D11.5.6-1 shall apply.

11.6.4. Safety Against Structural Failure

The following shall supplement A11.6.4.

11.6.4.1P BASE OR FOOTING SLABS

The rear projection or heel of base slabs shall be designed to support the entire weight of the superimposed materials, unless a more exact method is used. The base slabs of cantilever walls shall be designed as cantilever supported by the wall. The base slabs of counterforted and buttressed walls shall be designed as fixed or continuous beams of spans equal to the distance between counterforts or buttresses.

The critical sections for bending moments in footings shall be taken at the face and back of the stem. The critical sections for shear in footings shall be taken at a distance d(d=effective depth) from the face of the stem for the toe section and at the back of the stem for the heel section.

The minimum footing thickness shall be 600 mm {2’-0”} for footings bearing directly on soil or rock. For footings supported on piles, refer to D10.7.1.2.

11.6.4.2P WALL STEMS

The upright stems of cantilever walls shall be designed as cantilevers supported at the base. The upright stems or face

COMMENTARY

C11.6.3.5 PASSIVE RESISTANCE

The following shall supplement AC11.6.3.5.

The earth pressure on the back of a retaining wall will generally be active or at rest. Wall movements or rotations required to develop passive resistance in front of the wall are typically much larger (as much as 20 times larger) than movements required to develop active earth pressure behind the wall. Furthermore, the foundation for a rigid wall will usually be shallow (just deep enough to protect against frost heave and/or scour) and potentially subject to future exposure by utility excavations, road reconstruction, or other activities. Because full mobilization of passive resistance is unlikely and because soil in front of the wall may be disturbed sometime after construction, the passive resistance of soil in front of a rigid wall is neglected, unless the foundation is unusually deep or a structural foundation key is provided.

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walls of counterfort and buttress walls shall be designed as fixed or continuous beams. The face walls (or stems) shall be securely anchored to the supporting counterforts or buttresses by means of adequate reinforcement.

Wall stems shall be designed for combined axial load (including the weight of the stem and friction due to backfill acting on the stem), and bending due to eccentric vertical loads, surcharge loads and earth pressure.

The minimum thickness of wall stems shall be 450 mm {1'-6"} at the top and may be of constant thickness, or with battered front and/or rear face, as required. Normally, the front face shall be vertical.

11.6.4.3P COUNTERFORTS AND BUTTRESSES

Counterforts shall be designed as T-beams. Buttresses shall be designed as rectangular beams. In connection with the main tension reinforcement of counterforts, there shall be a system of horizontal and vertical bars or stirrups to anchor the face walls and base slab to the counterfort. These stirrups shall be anchored as near to the outside faces of the face walls and as near to the bottom of the base slab as practicable.

11.6.4.4P REINFORCEMENT

Reinforcement requirements herein are in addition to these specified in applicable portions of D5.10., D5.11, A5.10 and A5.11.

The reinforcement in each construction panel (i.e., between vertical construction joints) of walls, with height varying uniformly from one end to another, shall be designed for the loading condition acting at one-third of the panel length from the high end of the panel. If practical, the thickness of footings shall be maintained constant in each panel or in each group of panels. The width of footings, however, may vary according to the height of wall as required by design.

Tension reinforcement at the bottom of the heel shall be provided if required during the construction stage prior to wall backfill. The adequacy of reinforcement shall be checked due to the dead load of the stem and any other vertical loads applied to the stem prior to backfilling.

11.6.4.5P ABUTMENTS

Abutment stems shall be designed for 50 mm {2 in.} longitudinal eccentricity from the theoretical centerline of bearing to compensate for incidental field adjustments in the locations of the bearings. The eccentricity does not need to be considered for footing design.

In general, pile-supported footings for abutments and retaining walls shall be provided with a minimum of two rows of piles. Stub abutments may be designed with one or two rows of piles, with piles battered as necessary.

Refer to criteria in D11.6.1.7P to determine whether a backwall or a full-depth concrete diaphragm wall shall be

SPECIFICATIONS

specified for an abutment. In addition to the earth, surcharge and water pressures, prescribed in D3.11.5 and A3.7, respectively, the backwalls of abutments shall be designed to resist loads due to expansion joints, and loads due to design live and impact loads. For design purposes, it shall be assumed that wheel loads are positioned so as to generate the maximum tensile stresses at the back of the backwall when combined with stresses caused by the backfill.

11.6.4.6P INTEGRAL ABUTMENTS

Integral abutments shall be designed to resist the forces generated by thermal movements of the superstructure against the pressure of the fill behind the abutment. Integral abutments should not be constructed on spread footings founded or keyed into rock. Movement calculations shall consider temperature, creep and long-term prestress shortening in determining potential movements of abutments.

Maximum span lengths and design considerations shall comply with the requirements of Appendix G, "Integral Abutments". For integral abutment details, refer to BD-667M Standard Drawing.

11.6.5 Seismic Design Provisions

The following shall replace A11.6.5

For typical bridges, the seismic loading condition should be evaluated using the Coulomb or Rankine earth pressure as appropriate for the structure type and geometry. The inertial effects of the abutment and the surrounding soil need not be included. For this loading condition, the strength and stability of the structure shall be evaluated.

Consideration of an increased earth pressure and inertial effects may bed appropriate for unusual bridges and/or soil conditions.

PP1.5.1 provides some guidance to be used in determining whether a bridge should be classified as unusual.

COMMENTARY

C11.6.5

The following shall replace AC11.6.5

The provisions herein are based on the state of practice at CALTRANS. CALTRANS' current design policy does not consider Mononobe-Okabe earth pressure effects. They have found that the earth pressures resulting from the Mononobe-Okabe method are generally unreasonable. For abutments in which seismic forces can be transmitted from the superstructure to the substructure, the following recommendations have been made:

i) The horizontal superstructure seismic force is applied at the bearings.

ii) Inertial effects of the structure and surrounding soil should be neglected.

iii) The dead loads (DC, EV, EH…) should be computed and factored based on the load factors provided for the seismic load combinations given in A3.4..1 and D3.4.1. In rare cases, the geotechnical engineer may specify an additional factor to be applied to EH to account for unusual conditions.

iv) The strength of the footing and stem should be investigated, as well as the stability of the structure which includes overturning, sliding, bearing and pile capacity as appropriate.

v) The length of the abutment seat must be checked to ensure adequate support length.

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In the case of unusual bridges, consideration of seismic forces should be made on a case-by-case basis by the design engineers, with input from seismic experts with knowledge and experience in seismology and geotechnical engineering.

C11.6.6 Drainage

The following shall supplement AC11.6.6

Where a retaining wall is located in a stream environment, the wall will be subjected to varying stream levels. If the soil behind the wall cannot or is not permitted to drain freely (i.e., the backfill or retained soil is fine grained or the weep holes are clogged), the wall will be subjected to loading by differential water pressure.

11.6.7P Submittals

The following information shall be submitted by the designer and included with the construction documents for abutments and conventional retaining walls:

- Foundation submission, as required by D11.4.4P
- Earth pressures, water pressures and surcharge loadings (to be included with final plan submission)
- Geometric considerations, including beginning and ending wall stations, wall profile and alignment, right-of-way limits, utility locations, construction considerations, such as traffic restrictions or required construction sequences, and location of wall appurtenances, such as drainage outlets, overhead signs and lights, and traffic barriers (to be included in the plans)
- References and methods used for analysis for all appropriate loading conditions including all calculations (with applicable load and resistance factors), computer analyses, assumptions, input, and explanation of all symbols, notations, and formulas (to be included with final plan submission)
- Details, dimensions and schedules of all concrete and reinforcing steel, if applicable (to be included in the final plans)
- Limitations on backfill placement for integral abutments (to be included in the final plans)

11.7 PIERS

The following shall supplement A11.7.

Piers shall be designed for 50 mm {2 in.} longitudinal eccentricity from the theoretical centerline of bearing to compensate for incidental field adjustments in the locations of the bearings. The eccentricity does not need to be considered

C11.7

The PAPIER program carries the eccentricity throughout the substructure design, including the footing design, to ensure that a proper mathematical model where the moment, shear and axial force at the bottom of the column are equal to the moment, shear and axial force at the top of the footing is

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for footing design.

The geotechnical design of pier foundations shall be in accordance with the provisions of D10. Design of piers supporting lateral earth loads shall incorporate applicable provisions of A11.6 and D11.6. The foundation submission shall conform to the requirements of D11.4.4P.

11.7.0P General

Piers shall be of the open-bent (columns with cantilevered cap beams) or hammerhead-type, except at stream crossings where hammerhead or solid-section piers with rounded or protected ends shall be used. For stream crossings, the rounded or protected ends of the piers should be a minimum of 300 mm {1 ft.} above design flood elevation. For structures with piers skewed to the flow of a stream, circular pier stems shall be used.

Circular stems should also be considered to reduce turbulence where scour is a problem.

For pier designs, adhere to the following criteria:

- Pier columns shall be round, square, or rectangular, with a minimum diameter or thickness of 900 mm {3'-0"}, unless a smaller column is required for aesthetic reasons. The minimum depth increment shall be 150 mm {6"}. Solid piers shall have a minimum thickness of 600 mm {2'-0"} and may be widened at the top to accommodate the bridge seat when required. Minimum footing thickness is 600 mm {2'-0"}, unless the footing is on piles, in which case 750 mm {2'-6"} is the minimum thickness

- Ends of pier cap shall project beyond the sides of columns, when possible, to balance positive and negative moments in the cap.

- Cap width shall be at least 150 mm {6 in.} wider than the thickness or diameter of the column and may be up to 300 mm {1'-0"} wider than the thickness or diameter of the column. Caps which support prestressed concrete beams made truly continuous may be increased in width based upon the increase in the gap between beam ends up to a maximum of 2'-0" wider than the thickness or diameter of the column, provided the centerline of bearing falls a minimum of 3 inches within the column/shaft section. Two layers of reinforcement bars are allowed to minimize cap dimensions.

- If the height of the pedestal exceeds 450 mm (1'-6"),, the cap shall be slanted.

- Hoops of No. 13 bars at 300 mm {No. 4 bars at 12 in.} shall be used in round pier columns. For arrangement of hoops and auxiliary ties in square and rectangular columns or shafts, see Figure 1. Spiral reinforcement

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provided.

C11.7.0P

Consideration of freeboard may allow passage of ice flows and debris. An evaluation of whether freeboard (clearance above design flood elevation) is needed should be determined on a case by case basis.

The Department prefers round columns for seismic design considerations.

Cap width with a minimum of 150 mm {6 in.} wider than the column thickness or diameter is specified to avoid interference between the column reinforcement that projects from the column into the cap, and the cap reinforcement. An increased gap between end face of the beams, to develop the positive moment reinforcement bars in the cast-in-place diaphragm, requires a cap width up to 600 mm {2'-0"} wider than the column thickness or diameter.

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may be used in lieu of column/pier tie reinforcement. The spiral reinforcement requirements must be designed and shown on the contract drawings.

- Bars subject to tensile stresses in cantilevered ends of cap beams shall be provided with 90º or 180º standard hooks.

- Wide columns with long internal cross ties may need lap splices at a minimum spacing of 2400 mm {8 ft} detailed for a few locations to allow worker access during concrete placement.

- The vertical bars in columns shall extend into the cap beam a minimum of 20 bar diameters.

- Columns of pier bents shall have individual or continuous footings, depending on economy and soil conditions. Continuous footings shall be used, unless founded on rock.

- Shrinkage keys shall not be used in pier caps to eliminate partial or total shrinkage stresses.

- No. 16 {No. 5} reinforcement bars at a maximum spacing of 300 mm {12 in.} shall be provided at bottom of caps of hammerhead piers.

- For columns of pier bents located in the sloped portion of an embankment, the earth pressure against the back of the footing and column shall be increased 100% to include the effect of the adjacent embankment. The effect of the embankment in front of the column shall be neglected. Piers located in the embankment shall be investigated for stability not considering superstructure loads.

- Pier bents with continuous footings may be analyzed using the following procedure in lieu of a more exact analysis:

  (1) Analyze the pier bent above the footing assuming the bottom of columns to be fully fixed.

  (2) Analyze footing continuously supported by columns and loaded by soil reactions due to loads on the pier bent.

- For hammerhead piers, all the calculated cantilever reinforcement shall be extended throughout the entire length of the cap. Additional No. 16 {No. 5} stirrups spaced at 600 mm {24 in.} shall be placed in the cap within the limits of the shaft. The stirrups shall be more closely spaced near the ends of the shaft than in the interior region. In deep caps, additional longitudinal bars

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shall be placed at intervals throughout the depth of the cap.

- Reinforce pier caps adequately to control cracking when concrete cover of 75 mm {3 in.} is used in high tensile zone and corrosive environment. Alternately, build the top of the pier level or with a constant slope from one end to the other and maintain 50 mm {2 in.} concrete cover over the top reinforcement. If it is necessary, provide pedestals for beam seats.

- Provide closely spaced horizontal bars in the bottom half of the pier cap of hammerhead piers at each face to control shrinkage cracks. The amount of the rebars provided in this area should not be less than No. 16 at 300 mm {No. 5 at 12 in.}, and a bar spacing greater than 300 mm {12 in.} should not be used.

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Excessive concrete cover at pier caps for the main (top) reinforcement is sometimes included in the design without recognizing its detrimental effect. In some instances, details do not reflect the design assumptions or calculations. Excessive concrete cove is observed when beam seats are sloped parallel to the deck slope or are stepped.

The LRFD serviceability criteria (A5.7.3.4) permits controlled cracking of concrete. The crack width is approximately proportional almost to the square of the concrete cover over the reinforcement.

Vertical cracks in the pier cap of hammerhead piers have been observed at the intersection of the cap and the stem, generally near the middle of the cap. These cracks generally occur for wide stem (stem width 3000 mm {10 ft.} and more) piers due to shrinkage of the pier cap concrete. The hardened concrete of the stem and protruding rebars resist free shrinkage of the cap. These cracks can be controlled by providing closely spaced horizontal bars at each face in the bottom half of the pier cap of hammerhead piers.

11.7.3P Pier Design

Piers shall be designed for longitudinal and transverse superstructure loads.

For the purpose of designing piers of multi-fixed-pier bridges, the design temperature, Δt, shall be taken as the larger of the temperature rise and temperature fall defined in Table D3.12.2.1-1. This design temperature change shall be used for both the expansion and the contraction of the structure.

The thermal movement at any pier location shall be determined using Equation 1

$$\Delta = \alpha \Delta_t L \qquad (11.7.2P\text{-}1)$$

where:

Δ = design displacement at the top of the pier

Δ_t = design temperature change

L = length of superstructure between the pier under consideration and the theoretical fixed center of structure.

α = coefficient of thermal expansion of the girders

The displacement Δ shall be assumed to act parallel to the longitudinal axes of the superstructure at the pier location.

The theoretical fixed center of the structure shall be determined by assuming that the abutments are free to expand.

C11.7.3

The design displacement represents the movement due to thermal expansion or thermal contraction. Thus, for the purpose of analysis, it may be taken in either direction such that the total forces in the pier are maximized.

The theoretical center of the structure is the point that does not move when the structure is subjected to uniform

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temperature change. Only one such point exist in a jointless superstructure. For other bridges, one such point exist in each segment of the structure between two expansion joints or between an expansion joint and the end of the bridge. For each segment of the structure, the sum of the thermal forces on all piers to one side of the theoretical center is equal, and opposite in direction, to the sum of the thermal forces on all piers on the other side.

Pier forces caused by thermal movements of the superstructure shall be calculated assuming the pier acting elastically and assuming a concrete modulus of elasticity equal to one third the elastic modulus of elasticity.

Pier forces calculated using the specified reduced modulus of elasticity are one third the forces calculated assuming elastic behavior. This reduction accounts for both the inelastic behavior of the concrete under long term deflections and the moment redistribution due to the expected rotations of the foundations.

For the strength limit state analysis, the reduction may be taken while at the same time allowing the thermal force with a 0.5 Load Factor for the conventional piers and 1.0 load factor for the integral piers of the segmental construction bridge. If a designer wants to prevent cracking at the service limit state, a Load Factor of 1.0 may be used.

The type of connection to the superstructure shall be considered in determining pier moments and shears.

Elastic pier forces are a function of the restraint of the rotation at the ends of the pier and the direction of movement relative to the major axis of the pier.

For bridges with skew angle of 90º, the elastic forces should be calculated as follows:

Piers supporting fixed bearings should be assumed to act as cantilevers. The column elastic base shear, P, and elastic base moment, M_B, are calculated as:

$$P = 3EI\frac{\Delta}{l^3} \quad \text{(C11.7.3P-1)}$$

$$M_B = 3EI\frac{\Delta}{l^2} \quad \text{(C11.7.3P-2)}$$

where:

Δ = design thermal movement at the pier location

E = the concrete modulus of elasticity for long term deflection

I = moment of inertia of the pier about an axis perpendicular to the longitudinal axis of the superstructure

l = the length of the column from the top of the footing or pile cap to the bottom of the bearings

For piers integral with the superstructure, the piers are assumed to act as a beam with restrained rotation (fixed) at both ends. The column elastic base shear, P, and elastic base moment, M_B, are calculated as:

$$P = 12EI\frac{\Delta}{l^3} \quad \text{(C11.7.3P-3)}$$

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$$M_B = 6EI\frac{\Delta}{l^2} \tag{C11.7.3P-4}$$

where:

l = the length of the column from the top of the footing or pile cap to the bottom of the superstructure.

In case of skewed bridges, the components of the thermal movements should be calculated in the direction of both major axes of the pier. The appropriate moment of inertia should be used to determine the pier forces in each direction. For the component of the thermal movement in the plane of the pier, the pier should always be assumed to act as a cantilever fixed at the top of the footing or the pile cap. For the component of the thermal movements perpendicular to the plane of the pier, the pier forces will be calculated according to Equations 1 through 4. In this case, Δ is the component of the thermal movement perpendicular to the pier.

Providing a hinge at the bottom of the columns shall not be allowed.

If a column or pier bent is located in the sloped portion of an embankment, the earth pressure against the back of the footing and column shall be taken as the at rest earth pressure computed in accordance with A3.11 and D3.11. The resistance due to the passive earth pressure of the embankment in front of the column or pier bent shall be neglected.

11.8 NONGRAVITY CANTILEVERED WALLS

11.8.1 General

The following shall supplement A11.8.1.

Hot rolled steel sheet piling may be used for permanent and temporary applications. Cold rolled steel sheet piling may be used only for temporary applications.

C11.8.1

The following shall replace AC 11.8.1.

Depending on soil conditions, nongravity cantilevered walls less than about 5000 mm {15 ft.} in height are usually feasible.

Cold rolled steel sheet piling may be used for temporary applications, provided that the inherent strength deficiencies are adequately considered. Research has shown cold rolled sheet piling sections to have substantially less elastic moment capacity than hot rolled sections with equivalent section moduli. The difference in capacity was found to be due to the geometry of the cold rolled sections rather than the cold rolling process. The cold rolled sections were wider, deeper, and thinner than hot rolled sections with equivalent section moduli.

11.8.5 Safety Against Structural Failure

The provisions of D11.9.5.2, A11.9.5.2, D11.9.5.3 and A11.9.5.3 shall apply.

C11.8.5.2 FACING

Soil arching shall not be considered.

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11.8.6 Seismic Design

The provisions of A11.6.5 shall apply.

11.8.7 Corrosion Protection

The provisions of A11.9.7 shall apply.

11.8.8 Drainage

The following shall replace A11.8.8.

The provisions of A3.11.3 shall apply.

Seepage shall be controlled by installation of a drainage medium (e.g., preformed drainage panels, sand or gravel drains or wick drains) behind the facing with outlets at or near the base of the wall. Drainage panels shall maintain their drainage characteristics under the design earth pressures and surcharge loadings, and shall extend from the base of the wall to a level 300 mm {1 ft.} below the top of the wall. Only Department-approved drainage panel materials shall be specified.

11.8.9P Submittals

The following information shall be submitted by the Designer to the Department for review and shall be shown in the construction documents for nongravity cantilevered walls:

- Foundation Submission as required by D11.4.4P.
- Earth pressures, water pressures and surcharge loadings (to be included with final plan submission)
- Lateral deflection at top of wall including assessment of lateral deflection effects on adjacent features or facilities supported by the wall.
- Geometric considerations including beginning and ending wall stations, wall profile and alignment, right-of-way limits, utility locations, construction considerations such as traffic restrictions or required construction sequences, and location of wall appurtenances such as drainage outlets, overhead signs and lights, and traffic barriers (to be included in the plans)

COMMENTARY

C11.8.8

The following shall replace AC 11.8.8.

In general, the potential for development of hydrostatic pressures behind walls with discrete vertical elements and lagging is limited due to the presence of openings in the lagging, and the disturbance of soil behind lagging as the wall is constructed. However, the potential for leakage through the wall should not be counted upon where the ground water level exceeds one-third the height of the wall because of the potential for plugging and clogging of openings in the wall with time by migration of soil fines. It is probable that, under such conditions, a wall with continuous vertical elements (i.e., a cutoff wall) constructed with a drainage system designed to handle anticipated flows will be required.

Water pressures may be ignored in design only if positive drainage (e.g., drainage blanket, performed drainage panels, sand drains, wick drains, etc.) with outlet pipes is provided to prevent buildup of hydrostatic pressure behind the wall.

For CIP facing rigidly attached to soldier piles, extend facing to 1.2m {4 ft} below ground elevation to prevent freezing of drainage blanket.

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- References and methods used for analysis for all appropriate loading conditions including all calculations (with applicable load and resistance factors), computer analyses, assumptions, input and explanation of all symbols, notations and formulas (to be included in the plans)

- Vertical wall element types, sizes and spacings; and erection sequence (to be included with Final Plan Submission)

- Details, dimensions, connections and schedules of all structural steel and reinforcing steel for vertical wall elements and facing (to be included in the plans)

- Drainage requirements (to be included in the plans)

- Corrosion protection and/or accommodation details for the wall elements and hardware (to be included in the plans)

11.9. ANCHORED WALLS

11.9.1 General

Figure 1 shall replace Figure A11.9.1-1.

COMMENTARY

C11.9.1

The following shall supplement AC11.9.1.

The inclination and spacing of anchors will be dependent mostly on the soil and rock conditions, the presence of geometric constraints, and the required anchor capacity. For tremie-grouted anchors, a minimum angle of inclination of about ten and a minimum overburden cover of about 4500 mm {15 ft.} are typically required to ensure that grout fills the hole through the entire bonded length, and that adequate load transfer is achieved as a result of the confinement of the bond zone. For pressure-grouted anchors, the angle of inclination is generally not critical, and is governed primarily by geometric constraints. Also, because pressure-grouted anchors do not rely as heavily on confinement for load transfer, their bonded lengths may be located at shallower depths (e.g., 2000 to 4500 mm) {6 to 15ft.}) than is typical for tremie-grouted anchors. Very flat or steep inclination angles (ranging from 0 to 45) may be required to avoid anchorage in unsuitable soil or rock, to reach deep strata for anchorage, or to avoid underground obstructions. The minimum horizontal spacing specified between anchors (see D11.9.5.1) is intended to reduce the potential for overlap of stresses between adjacent anchors. If necessary, the inclination of adjacent anchors can be adjusted to limit the potential for stress overlap.

The minimum depth of embedment should be sufficient to provide adequate bearing resistance to support the vertical component of the anchor load(s). In addition, consideration should be given to the effects of freezing and thawing, scour, weathering and other shallow ground disturbance (e.g., utility excavations or pavement replacement) in front of or below the

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wall.

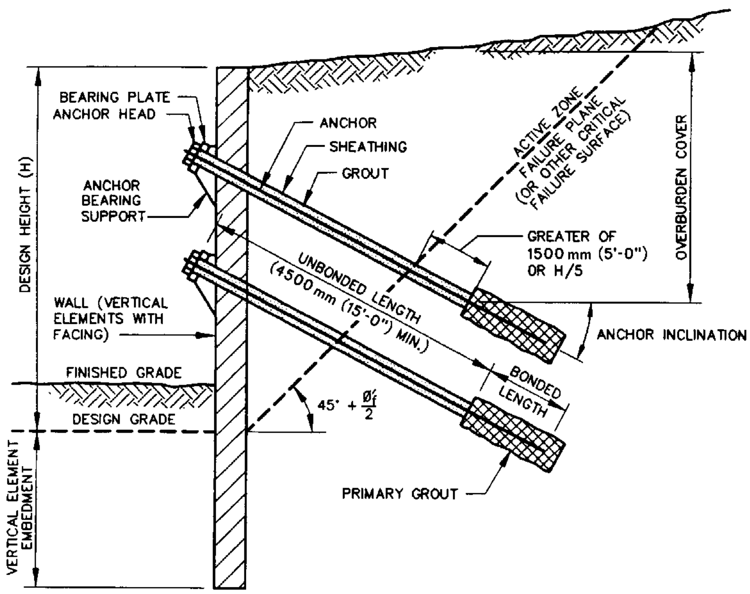

Figure 11.9.1-1 - Anchored Wall Nomenclature and Anchor Embedment Guidelines

The following shall supplement A11.9.1.

With the approval of the Chief Bridge Engineer, anchored walls may be constructed in fill situations (i.e., wall construction from the bottom up) with the following restrictions:

Unprotected (uncased) anchors penetrating fill which can settle due to compression of underlying soil layers may bend as a result of settlement. Such bending is difficult to predict and would result in an unpredictable increase in the anchor load.

- The full-length of all anchors extending through fill materials shall be placed within a steel casing (or strut) to minimize the effects of fill settlement on the anchors. The casing shall be of adequate size to preclude the potential for the inside top diameter of the casing from coming in direct contact with the anchor assembly as a result of settlement of the fill and/or subgrade. The casing shall extend from the back face of the wall, through the full width of backfill, and to a depth into original ground to preclude the application of differential forces on the anchor at the backfill/original ground contact. The anchor/casing assembly shall be installed by placement on the top of the compacted backfill so that the casing is in full bottom contact with the backfill.

- Solid bar anchors shall not be permitted.

Solid bars are especially intolerant of bending and are prone to brittle failure.

- The bond length of each anchor shall extend into stable, original ground beyond the potential failure zone.

- All fill material shall consist of structural backfill and shall be compacted in conformance with

Publication 408, Sections 1001.3(t) and 206.3(b), respectively. When the level of backfill is less than 300 mm {1 ft.} above the top of the casing or within 1200 mm {4 ft.} of the inside face of the wall, only hand-operated compaction equipment shall be permitted.

- Stressing of any anchor level shall not proceed until backfill is placed to at least the next anchor level or to within 300 mm {1 ft.} of the top of the wall. To prevent transfer of load from backfill on the casing to the anchor, the unbonded length of the anchor shall not be grouted within the casing. The annulus between the casing and anchor shall be filled with grease or other approved materials for corrosion protection. The strut shall be designed to resist the required design prestress force and bending due to settlement of the fill material or live load influence.

11.9.3 Movement Under the Service Limit State

11.9.3.1 MOVEMENT

The following shall replace A11.9.3.1

The provisions of D10.6.2, D10.7.2, and D10.8.2 shall apply for evaluation of vertical element settlements due to the vertical component of anchor forces.

The effects of lateral wall movements on adjacent facilities shall be considered in the development of the design earth pressure in accordance with the provisions of A3.11.5.6 and D3.11.5.6

11.9.4 Safety Against Soil Failure

11.9.4.1 BEARING RESISTANCE

The following shall replace A11.9.4.1.

The provisions of D10.6.3, D10.7.3 and D10.8.3 shall apply.

Loads at the base of vertical wall elements, including the vertical component of contributing anchor loads, shall be determined assuming that all vertical components of loads are transferred to the base of the elements. Side friction of wall elements shall not be included in the resistance to vertical loads.

11.9.4.2 ANCHOR PULLOUT CAPACITY

The following shall supplement A11.9.4.2.

Anchor embedment and inclination for straight shaft anchors installed in small diameter holes using low grout pressure shall follow the guidelines in Figure D11.9.1-1, unless otherwise approved by the Chief Bridge Engineer.

Final determination of the anchor pullout capacity and required bond length shall be the responsibility of the anchor wall specialty contractor.

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11.9.5 Safety Against Structural Failure

11.9.5.0P GENERAL

Anchored walls shall be designed with sufficient redundancy to protect against catastrophic wall failure in the event of the failure of an anchor.

The procedure for anchored wall design depends on the number of anchor rows and the construction sequence. For a typical wall with two or more rows of anchors constructed from the top down, the procedure requires design for the final structure with multiple rows of anchors and checking the design for the various stages of wall construction.

The required horizontal component of each anchor force shall be computed using the apparent earth pressure distributions in A3.11.5.7 and D3.11.5.7, or other approved earth pressure distributions, and any other horizontal water pressure, surcharge or seismic forces acting on the wall. The total anchor force shall be determined based on the anchor inclination. The horizontal anchor spacing and anchor capacity shall be selected to provide the required total anchor force.

The vertical wall elements shall be designed to resist all applicable loads including, but not limited to, horizontal earth pressure, surcharge, water pressure, anchor and seismic loadings, as well as the vertical component of the anchor loads and any other vertical loads. Supports may be assumed at each anchor location and at the bottom of the wall if the vertical element is extended below the bottom of the wall.

The stresses in and the design of the wall facing shall be computed in accordance with the requirements of A11.9.5.3.

All components of the anchored wall system shall be designed for the various earth pressure distributions and other loading conditions which are anticipated during construction.

COMMENTARY

C11.9.5.0P

Soil arching shall not be considered. One method of providing redundancy is to use a structural connection (e.g., bars, facing, or wales) between vertical wall elements to permit transfer and redistribution of load to adjacent anchors in the event of the failure of an anchor. Tie-back soldier pile walls with cast-in-place facing should be designed for their facings to distribute load in the event of an anchor failing. For tie-back soldier pile walls with precast facing the designer must provide a positive means to provide this required redundancy. This could be done by use of a continuous reinforced concrete cap beam or by horizontal steel tie rods run between the piles or additional anchors. Where individual soldier piles contain multiple anchors, the adjacent anchors can be designed to handle the additional load caused by this occurrence.

The procedure for design of a nongravity anchored wall depends on the number of rows of anchors utilized and the sequence of construction. For a typical wall with two or more rows of anchors constructed from the top down, the procedure requires design for multiple rows of anchors and checking the construction conditions prior to and after the installation of each row of anchors. The procedures for design or analysis of each of these cases may be as follows:

(a) No anchors:

Evaluate earth pressure distributions and perform analysis and design in accordance with the requirement for flexible cantilevered walls in D11.12P.

(b) One row of anchors:

(1) Select an elevation and inclination for anchors (often dictated by physical constraints such as underground utilities).

(2) Develop a lateral earth pressure diagram (including surcharge and water pressures) using the simplified procedures of D3.11.5.6 as outlined in Figures D3.11.5.6-1 through D3.11.5.6-4. Any deviation must be approved in writing by the Chief Bridge Engineer before initiating the design. Develop an expression for the lateral pressure on the embedded portion of the vertical wall elements as a function of the embedment depth, D.

(3) Sum moments about the anchor elevation (neglecting the force F in Figures D3.11.5.6-2

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and D3.11.5.6-4), and solve for the depth of embedment, D, required to provide equilibrium. Sum horizontal forces to determine the horizontal component of the anchor force required for equilibrium and calculate the associated total anchor force based on the anchor inclination. Select anchor spacing and capacity to provide the required total anchor force.

(4) Determine the maximum bending moments in the vertical wall elements in accordance with A11.9.5.2 and select a vertical element size and spacing (typically 2000 to 3000 mm {6 to 10 ft.}).

(5) Design the wall facing in accordance with the requirements of A11.9.5.3.

(6) Check the combined axial-bending capacity and bearing capacity and estimate the settlement of the vertical wall elements under the vertical component of the anchor forces and other vertical loads in accordance with D10.6, D10.7 and D10.8.

(7) Check the overall stability of the wall system, retained soil and foundation in accordance with A11.9.3.2. For soft clays with S_u . 0.3 x $10^{-9}\gamma'_s gH$ (H in mm) {S_u . 0.3 $\gamma'_s H$ (H in ft.)}, continuous vertical elements extending well below the exposed base of the wall are generally required to prevent heave in front of the wall [U. S. Steel (1984)]. Otherwise, the vertical elements are embedded as required for bearing capacity.

(Where significant embedment of the wall is required to prevent bottom heave, the lowest section of wall below the lowest row of anchors must be designed to resist the moment induced by the pressure acting between the lowest row of anchors and the base of the exposed wall, and the force $P_b = 0.7(10^{-9}\gamma_s gHB_e - 1.4cH - \pi cB_e)$, in N/mm {$P_b = 0.7(\gamma_s HB_e - 1.4cH - \pi cB_e)$, in kip/ft} acting at the mid-height of the embedded depth of the wall.).

(c) Two or more rows of anchors.

(1) Develop an apparent lateral earth pressure diagram (including surcharge and water pressures) in accordance with A3.11.5.7 and D3.11.5.7.

(2) Select a number of anchor level elevations based on the anticipated wall height. Vertical spacings

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between rows of anchors of 2500 to 4000 mm {8 to 12 ft.} are common.

(3) Calculate the required horizontal components of the anchor forces by the proportional methods using the apparent earth pressure distribution developed under Step 1, or by other suitable methods. If the proportional method is used, the top row of anchors is assumed to support the tributary area of pressure from the top of the wall down to a point midway between the first and second anchor levels. The bottom row of anchors is assumed to support the pressure from the base of the wall up to a point midway between the two lowest rows of anchors. Alternatively, the embedded portion of the vertical wall element may be assumed to support the pressure between the base of the exposed wall and a point midway up to the lowest anchor level. The magnitude of available support may be computed in accordance with A3.11.5.9. Intermediate rows of anchors are assumed to support the pressure between the midway points to the next higher and lower rows of anchors. Calculate the total anchor force based on the anchor inclination. Select the horizontal anchor spacing and anchor capacity to provide the required total anchor force.

(4) Determine the maximum bending moments in the vertical wall elements in accordance with A11.8.9.2, and select a vertical element size and spacing (typically 2000 to 3000 mm {6 to 10 ft.}).

(5) Design the wall facing in accordance with the requirements of A11.9.5.3 and D11.9.5.3.

(6) Check the combined axial-bending capacity and bearing capacity, and estimate the settlement of the vertical wall elements under the vertical component of the anchor forces and other vertical loads in accordance with D10.6, D10.7 and D10.8.

(7) Check the overall stability of the wall system, retained soil and foundation in accordance with A11.9.4.3.

For soft clays with $S_u \leq 0.3 \times 10^{-9}\gamma'_s gH$ (H in mm) {$S_u \leq 0.3\ \gamma'_s H$ (H in ft.)} continuous vertical elements extending well below the exposed base of the wall are generally required to prevent heave in front of the wall [U. S. Steel (1984)];otherwise, the vertical elements are

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embedded as required for bearing capacity.

(8) Estimate wall deflections under the design loadings and support conditions where deflections are critical.

11.9.5.1 ANCHORS

The following shall supplement A11.9.5.1.

The optimum vertical spacing between anchors shall be determined by minimizing the bending moment in the vertical wall elements between anchor levels with consideration of the construction sequence to be followed. Vertical spacings between anchors of 2500 to 4000 mm {8 to 12 ft.} are common.

Horizontal spacings of 2000 to 3000 mm {6 to 10 ft.} between anchors are common.

11.9.5.2 VERTICAL WALL ELEMENTS

The following shall supplement A11.9.5.2.

The combined axial-bending and bearing capacity, and the settlement of the vertical wall elements under the vertical component of the anchor forces and other vertical loads shall be evaluated in accordance with D10.6, D10.7 and D10.8.

The overall stability of the wall system, retained soil, and foundation shall be evaluated in accordance with A11.9.4.3.

The use of welded threaded studs on soldier piles is not permitted at either the design or construction phase of a project.

C11.9.5.2P

Structural analysis of vertical wall elements may be performed using COM624P, LPILE 5.0 or other Department approved method suitable for evaluation of laterally loaded deep foundations.

On a recent Department project, a tie-back wall design utilized an angle bolted on to threaded studs welded to the soldier piles in order to attach the precast panels used for its permanent facing. Several of the welds of the studs failed during installation of the precast panels. Possible alternates to this detail includes fillet welded or bolted connections of the angle to the soldier pile.

11.9.5.3 FACING

The following shall supplement A11.8.5.3.

Do not consider temporary lagging as permanent resistance in the design of facing.

11.9.7 Corrosion Protection

The following shall replace A11.9.7.

Corrosion protection requirements will be as follows:

Prestressed anchors and anchor heads shall be protected against corrosion in a manner consistent with the conditions at the site. The level and extent of corrosion protection shall be a function of whether the anchor is intended for temporary or permanent applications of the ground environment and of the potential consequences of an anchor failure. For permanent wall applications, a minimum of double corrosion protection shall be provided, regardless of the ground environment.

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Table 11.9.7-1 Criteria for Aggressive Ground Environments

PARAMETER	LIMITING VALUES
Resistivity	< 2000 ohm-cm
pH	< 5

All anchors for permanent walls shall be fully encapsulated over their entire length. For temporary walls, anchors shall be encapsulated whenever one or more of the limiting values specified in Table 1 is exceeded. Corrosion protection may not be required for temporary walls having a design life of less than one year. Other potentially corrosive conditions, including stray currents, shall be identified and evaluated, and appropriate means of corrosion protection shall be designed by the anchor-wall specialty contractor and shall be approved by the Department.

Corrosion protection for anchor hardware required for permanent and temporary walls shall be consistent with the level of protection required for the anchors. Structural steel elements shall be provided with additional sacrificial thickness, painted or coated to accommodate or prevent corrosion, in accordance with Department construction specifications. For structural design, sacrificial thickness shall be computed by assuming a carbon steel loss equal to 9 μm/year after zinc coating depletion. Corrosion resistant coatings, if specified, shall be of the electrostatically applied resin-bonded epoxy type with a minimum application thickness of 0.38 mm {15 mils} in conformance with the requirements of AASHTO M 284/M 284M. For corrosion protection of concrete elements, see PP3.4.

11.9.8 Construction and Installation

11.9.8.1 ANCHOR STRESSING AND TESTING

The following shall replace the first sentence of A11.9.8.1.

All production anchors shall be subjected to load testing and stressing in accordance with the special provisions.

11.9.9 Drainage

The following shall supplement A11.9.9.

Only Department-approved drainage panel materials shall be specified.

11.9.10P Submittals

In addition to the information in D11.8.9P for nongravity cantilevered walls, the following items shall be submitted by the Designer to the Department for review and shall be shown

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in the construction documents for anchored walls.

- Anchor type and estimated capacity, required capacity, minimum bonded and unbonded anchor lengths, anchor inclination, and anchor locations and spacings.

- Description of anchor installation procedures including drilling and grouting.

- Corrosion protection details for the anchors and anchor hardware.

- Detailed plans for proof, performance, creep (if applicable), and lift-off testing of anchors including specified load measuring devices, test locations, and testing procedures.

- Analysis of the stresses in vertical wall elements, facing, and anchors at critical stages of construction.

11.10 MECHANICALLY STABILIZED EARTH WALLS

11.10.1 General

Supplement the first paragraph of A11.10.1 to read.

Mechanically stabilized earth (MSE) systems, whose elements may be proprietary, should use strip or grid-type inextensible tensile reinforcements in the soil mass, and a discrete modular precast concrete facing which is vertical or near vertical.

Supplement the second paragraph of A11.10.1 as follows:

Also, mechanically stabilized earth walls shall not be used under the following conditions:

(a) Height greater than 12 000 mm {40 ft.} when mesh reinforcing is used and height greater than 17 000 mm {55 ft.} when strip reinforcement is used

(b) On curves with a radius of less than 18 000 mm {60 ft.}

(c) When longitudinal differential settlements along the face of the wall are expected to be greater than shown in Table AC11.10.4-1

(d) When floodplain erosion is anticipated to undermine the reinforced fill zone, or where depth of scour cannot be reliably determined. Chief Bridge Engineer approval is required for this use

(e) Where upstream floodplain or downstream drawdown creates unstable conditions of the backfill material and retaining wall

COMMENTARY

Selection of anchor installation procedures is usually made by the anchor wall contractor. In general, however, anchors in rock, clayey soils, silts and fine sands are pressure grouted, whereas anchors in coarser grained soils are usually grouted by gravity backfilling.

C11.10.1

Delete from Figure AC11.10.1-1 the schematic Titled

- MSE Wall with CIP concrete or shotcrete facing

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COMMENTARY

The proprietary mechanically stabilized earth wall systems which are approved for use subject to the design requirements and limitations contained in this manual are given in Appendix K.

For walls supporting roadways which are de-iced with chemical additives, an impervious membrane shall be placed above the reinforced zone and sloped to a collector drain to preclude infiltration of corrosion-causing elements, as shown in Standard Drawing BC-799M.

For walls constructed in side-hill cut and fill geometries or cut, a drainage blanket shall be constructed to intercept groundwater as shown in Standard Drawing BC-799M.

Where manholes for surface drainage must be constructed within the zone of reinforcing, reinforcing elements may be skewed from their normal position perpendicular to the wall face up to a maximum skew of 25º. (Refer to Standard Drawing BC-799M). Where this is not possible, an independent wall shall be constructed in back of the drainage structure. Panels in front of such drainage structures shall be structurally connected to adjacent panels.

When constructed on fills, the embankment between the original ground and the leveling pad shall be composed of a granular material (in conformance with Publication 408, Section 206.2.1(b)) or rock.

The following shall supplement A11.10.1.

Typical MSE wall cut and fill sections are given on Standard Drawing BC-799M.

An allowable range of 90 to 120 pcf is permitted for unit weight of specified backfill. MSE wall designs must consider both the upper and lower limits of allowable backfill unit weights. This includes calculations for bearing capacity, settlement, sliding, overturning and reinforcement pullout. If a particular unit weight is desired (i.e. due to strap length restrictions for right-of-way or availability of specified backfill material), the MSE wall design may be performed for this particular weight or range of unit weights. The unit weight or range of unit weights used in the design must be noted on the drawings.

11.10.2 Structure Dimensions

11.10.2.1 MINIMUM LENGTH OF SOIL REINFORCEMENT

Revise the second paragraph of A11.10.2.1 as follows.

The horizontal force used to design the connections to the panels may be taken as equal to, but no less than, 85% of the maximum horizontal force at a given level to a depth of 0.6H from the top of the wall. From there it increases linearly to 100% at the bottom of the wall.

11.10.2.2 MINIMUM FRONT FACE EMBEDMENT

The following shall supplement A11.10.2.2.

Consideration may be given to alternate methods of scour protection including sheetpile walls driven to below potential scour levels and/or riprap of sufficient size and placed to

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sufficient depth to preclude scour.

11.10.2.3 FACING

The following shall supplement A11.10.2.3.

Minimum panel reinforcement in accordance with A5.5 shall be provided. See D5.4.3.6P for corrosion protection of panel reinforcement and module reinforcement where salt spray is anticipated.

11.10.2.3.2 Flexible Wall Facing

Delete A11.10.2.3.2

11.10.4.3 OVERALL STABILITY

The following shall supplement A11.10.4.3.

For structures loaded with sloping surcharges, general stability analyses shall be performed using Swedish circle methods and using a minimum performance factor as outlined in D10.5.

11.10.5 Safety Against Soil Failure (External Stability)

The following shall supplement A11.10.5.

In addition, saturated soil condition must be considered in determining the external stability of the walls.

For external stability computations, live load surcharges shall be applied from a vertical plane beginning at the back of the reinforced zone.

For calculation of horizontal design forces behind the reinforced mass, consider and apply all the properties of the random backfill, which includes 300 mm {1 ft.} of specified backfill material.

11.10.5.2 LOADING

The following shall supplement A11.10.5..2.

For external stability computations, maximum EH and EV Load Factors govern design.

COMMENTARY

C11.10.4 Movements Under Service Limit State

The following shall supplement AC11.10.4.

For MSE walls, the limiting differential settlement criteria based on joint spacing is empirically derived from observations on completed structures. Structures subjected to greater differential settlements may be damaged by concrete spalling from panel corners. Full height panels (2.8 x 10^6 mm^2 {30 ft^2} or more), when used, have been subject to longitudinal and vertical cracking and visible bending. Therefore, their use is not recommended without substantial revision to the design and erection procedures presently in force.

Where greater differential settlements are anticipated, a system of open vertical slip joints should be provided at suitable intervals or the foundation improved by various ground improvement techniques (such as over-excavation and replacement with compacted backfill using select material).

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11.10.5.3 SLIDING

The following shall replace the first paragraph of A11.10.5.3.

The provisions of D10.6.3.4 shall apply. The vertical force due to the surcharge load shall not be used in calculating the sliding resistance given in D10.6.3.4.

11.10.6 Safety Against Structural Failure (Internal Stability)

11.10.6.2 LOADING (Internal Stability)

The following shall supplement A11.10.6.2.

For internal stability computations, maximum EH and EV Load Factors govern the design for maximum stress in the reinforcement and minimum EV for pullout computations.

11.10.6.3 REINFORCEMENT PULLOUT

11.10.6.3.2 Reinforcement Pullout Design

The following shall supplement A11.10.6.3.2.

Where pullout tests are performed, the reinforcement element configuration shall be the same as for the actual project condition. Minimum length of embedment shall be 1000 mm {3'-6"} and a constant rate of strain of 1.25 mm/mm {1.25 in/in} shall be used.

Perform specific pullout tests to determine ultimate resistance for fully saturated soil condition.

11.10.6.4.2 Design Life Considerations

11.10.6.4.2a Steel Reinforcement

The following shall supplement A11.10.6.4.2a

Steel reinforcement elements in MSE walls shall be designed to have a corrosion resistance-durability to ensure a minimum design life of 100 years for permanent structures. Delete the third paragraph of A11.9.8.1 which relates to epoxy coating.

11.10.6.4.2b Geosynthetic Reinforcement

Delete A11.10.6.4.2b.

11.10.6.4.5P Redundancy

MSE walls shall be designed with sufficient redundancy to protect against catastrophic failure of each discrete facing panel in the event of the failure of one strip or one longitudinal bar per grid mesh in each discrete panel. Adequacy under this condition shall be evaluated using a load factor of 1.1 for EH and 1.0 for EV for both pullout and rupture of the soil reinforcements.

11.10.8 Drainage

The following shall supplement the second paragraph of A11.10.8.

COMMENTARY

C11.10.6.4.2a

The following shall supplement AC11.10.6.4.2a

Where stray ground currents are anticipated within a 60 000 mm {200 ft.} distance of a structure, the potential for stray current corrosion exists when metallic reinforcements are used. Induced-current cathodic protection measures have not been successful in the past and have caused two known failures. Therefore, the use of these corrosion-mitigation systems are not recommended.

The use of alloys, such as aluminum and stainless steel, is not recommended under any circumstances. Substantial deterioration/corrosion has been observed in structures constructed with these alloys.

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COMMENTARY

Refer to Standard Drawing BC-799M for typical drainage blanket detail.

The following shall supplement the third paragraph of A11.10.8. Refer to Standard Drawing BC-799M for impervious membrane detail.

11.10.10.2 TRAFFIC LOADS AND BARRIERS

The following shall replace A11.10.10.2.

Traffic loads shall be considered in accordance with the criteria outlined in A13.

Barriers, constructed over or in line with the front face of the panel shall be designed to resist overturning moments by their own mass. Base slabs shall not have any transverse joints, except construction joints. The horizontal load shall be deemed to be transferred by horizontal shear stress to the reinforced mass and distributed to the upper row of reinforcement only. The upper row of soil reinforcement shall be sized to resist an additional load of 30 kN per linear meter {2 kips per linear foot} of wall. The full reinforcement length shall be considered effective in resisting the impact horizontal load.

Barrier reinforcement shall be in accordance with the appropriate Standard Drawings. Other barrier reinforcement requires Chief Bridge Engineer approval. The anchoring slab shall be strong enough to resist the ultimate strength of the standard barrier.

Flexible post and beam barriers, when used, shall be placed at a minimum distance of 1000 mm {3 ft.} from the wall face, driven 1500 mm {5 ft.} below grade, and spaced to miss the reinforcements. The upper two rows of reinforcement shall be designed for an additional horizontal load of 4 kN per linear meter {0.3 kips per linear foot} of wall.

When constructed over in line with the front face of the panels or precast modules, barriers shall be designed to meet the ultimate strength of the Department's standard barrier by their own mass and ability to resist overturning moments. Base slabs shall not have any transverse joints, except construction joints. Alternately, the wall can be moved out to provide a U-wing tie of barrier from the abutment of standard shape and strength.

C11.10.10.2

Delete AC11.10.10.2.

C11.10.10.3 Hydrostatic Pressures

Delete AC11.10.10.3.

11.10.10.5P DESIGN DETAILS

The juncture of mechanically stabilized earth walls and cast-in-place structures shall be protected from loss of fines and differential settlements in accordance with the detail shown in Standard Drawing BC-799M. Dissimilar wall types shall not be constructed immediately adjacent to each other if anticipated differential settlements at the juncture are greater than 25 mm {1 in.}. For mechanically stabilized earth walls, geotextile fabric shall be placed behind all open joints of panel facing.

SPECIFICATIONS COMMENTARY

11.10.11 MSE Abutments

Delete A11.10.11.

11.10.11.1P MSE ABUTMENTS ON PILE FOUNDATIONS

The design of bridge abutment footings and connecting backwall, supported on mechanically stabilized earth walls, shall be based on bridge loading developed by the LRFD method.

The MSE wall abutments (stub abutment) shall be supported on steel H-beam piles encased in smooth or corrugated galvanized steel pipe, filled with coarse sand and the MSE walls shall be designed as an earth retaining structure as shown in Standard Drawing BC-799M.

The minimum distance from the centerline of the bearing on the abutment to the outer edge of the facing shall be 1000 mm {3'-6"}. The minimum distance between the back face of the panel and the footing shall be 150 mm {6 in.}.

To prevent runoff of potentially chemically active water from entering the reinforced soil embankment, the gap between the facing panels and abutment footing shall be sealed with an impervious liner, as shown in Standard Drawing BC-799M.

Use the following pile design guidelines:

a. Piles should be designed for vertical loads, i.e., (DL + LL + I + DL, stub abutment)

b. Piles should be either point bearing or end bearing.

c. Allowable bearing pressure on piles should be in accordance with foundation approval.

d.. For construction sequences for abutments supported in piles refer to special provisions and construction specifications.

e. In some instances, pile locations interfere with soil reinforcing grid (Foster Geotechnical walls) or soil reinforcing strips (R.E. wall) behind the MSE walls. Therefore, develop specific method for the field installation to avoid interference of grids or strips, with the piles and show the details of pile locations and arrangement of MSE wall soil reinforcing elements. Cutting of reinforcing strips, steel mesh or grids at pile locations, vertical obstacles or utilities is not acceptable.

f. Show complete drainage behind the wall as shown in Standard Drawing BC-799M as required by the field condition and provide weep holes also.

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11.11 PREFABRICATED MODULAR WALLS

11.11.1 General

The following shall supplement A11.11.1.

Prefabricated modular wall systems are particularly well suited in side-hill cut applications, along stream channels, and where limited space is available between the wall line and the right-of-way limits. Typically, the width of the bottom module is approximately 50% of the wall height. When constructed on fills, the embankment between the original ground and the footing shall be composed of a granular material (in conformance with Publication 408, Section 206.2.1(b)) or rock.

Concrete modular systems shall not be used under the following conditions:

(a) When wall heights exceed 11 000 mm {35 ft.}

(b) For abutments where flared wingwalls are not at 30, 45, 60 or 90 degrees to the abutment wall or with open front-face modules

(c) When calculated longitudinal differential settlements along the face of the wall are greater than 1/200

11.11.2 Loading

The following shall supplement A11.11.2.

Where the back of the prefabricated modules forms an irregular, stepped surface, the earth pressure shall be computed on a plane surface drawn from the upper back corner of the top module to the lower back heel of the bottom module using Coulomb earth pressure theory.

Maximum EH and EV Load factors govern stability computations.

Neglect the soil weight on the module beyond an average plane (apparent) surface behind the modules as shown in Figure D3.11.5.9-2. Consider the weight of the concrete modules and soils within the concrete modules even if it is beyond the average plane surface.

11.11.4 Safety Against Soil Failure

11.11.4.1 GENERAL

The second paragraph of A11.11.4.1 shall be revised to read only:

Passive pressures shall be neglected in stability computations.

The following shall supplement A11.11.4.1

The foundation footing shall be proportioned in accordance with the factored loads, bearing performance factors and the applicable provisions of D10.6.

Refer to Table A3.11.5.3-1 for the coefficients of the backfill or the foundation soil.

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C11.11P

Precast modular systems are sensitive to longitudinal differential settlements which may cause cracking of connecting interior members. The limiting criteria derived are empirical and based on description in the literature of bin-type wall failures.

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For structures loaded with sloping surcharges, general stability analyses shall be performed using Swedish circle methods and using a minimum performance factor as outlined in D10.5.

If a computer program is submitted for the design of a proprietary wall, the submission shall include design and/or analysis methodology assumptions, and one copy of hand calculations for the most critical bottom module of the wall to demonstrate compliance of all the design requirements and the results of the computer output.

11.11.4.3 BEARING RESISTANCE

The following shall supplement A11.11.4.3.

Alternatively, bearing pressures may be computed using a uniform base pressure distribution over an effective footing width ($B' = B\text{-}2e_B$) in accordance with D11.6.3.2

11.11.4.5 SUBSURFACE EROSION

The following shall replace A11.11.4.5.

Bin walls, T-walls, and any prefabricated modular walls may be used in scour-sensitive areas only where their suitability has been documented to the satisfaction of the Owner.

11.11.6 Seismic Design

The provisions of A11.6.5 shall apply.

11.11.7 Abutments

Delete A11.11.7.

11.11.8 Drainage

The following shall supplement A11.11.8.

Refer to Standard Drawing BC-799M for drainage blanket details.

11.12P GABION RETAINING WALLS

All gabion retaining walls shall be designed based on the following guidelines:

1. General

   a. Gabion walls may be constructed for non-structural applications, such as erosion control and slope protection against erosion for any length and height in rural areas

   b. As a retaining wall (structural application), gabion walls are permitted for up to 4000 mm {12 ft.} maximum height (base to top) in rural areas only and ADT of 750 or less.

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2. Design Specifications

a Use gabion manufacturer's specifications for design considerations, except that load and resistance factors shall be as in D10.5.

b. Check external stability of the gabion wall (overturning and sliding) for all applicable forces due to live load surcharge and fill slope being retained by the wall.

c. Design the wall using Coulomb Wedge Theory and the following backfill soil characteristics.

- Unit Weight of Backfill = 1925 kg/m^3 {0.120 kcf}

- Horizontal Soil Pressure = 560 kg/m^3 {0.035 kcf}

In the event that a clear interpretation of design cannot be resolved, the Chief Bridge Engineer will be the arbiter and hisdecision will be final.

d. Where gabions will be exposed to corrosive environment (such as salt spray due to splashing or drainage into the wall) or industrial fumes and effluents, the basket should be constructed of galvanized and plastic coated wire.

e. The gabions baskets in gabion walls shall be designed to have a corrosion resistance - durability to ensure a minimum design life of 50 years

f. Place footing not less than 900 mm {3 ft.} below finished ground elevation.

g. Provide at regular intervals along the gabion walls a suitable drainage system (Drain Pipe) to carry water away from the foundation.

h. Check foundation pressure and settlement to ensure that no foundation failure occurs. District Bridge and Soil Engineers' approval should be secured before incorporating gabion walls in any project.

3. Construction Guidelines:

Show the following notes on the plans:

a. Provide materials and perform work in accordance with Publication 408, supplements

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thereto, and/or Special Provisions as indicated.

b. Provide geotextile fabric Class 2, Type B along all interface areas with backfill and the gabion walls. The minimum lap of fabric = 300 mm {12 in.}.

c. The structural backfill behind the gabion walls shall be in accordance with Standard Drawing RC-12M.

COMMENTARY

REFERENCES

Barker, R. M., Duncan, J. M., Rojiani, K. B., Ooi, P.S.K., Tan, C. K., and Kim, S. G., "Load Factor Design Criteria for Highway Structure Foundations", Final Report NCHRP 24-4, National Research Council, Washington, DC, 292 pp, 1991

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Hunt, R. E., "Geotechnical Engineering Techniques and Practices", McGraw-Hill, Inc., New York, New York, 729 pp, 1986

Terzaghi, K., and Peck, R. B., "Soil Mechanics in Engineering Practice", 2nd ed., John Wiley & Sons, New York, New York, 729 pp, 1967

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PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 1
PART B: DESIGN SPECIFICATIONS

SECTION 12 - BURIED STRUCTURES AND TUNNEL LINERS

SECTION 12 - TABLE OF CONTENTS

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12.1 SCOPE

The following shall supplement A12.1.

Culverts or buried structures refer to flexible (corrugated metal and thermoplastic) and rigid (concrete) structures used for conveyance of water or traffic below earth embankments. The structures for which the design procedures in this section are applicable range in span from 1200 to 12 200 mm {4 to 40 ft.}. Although pipe sizes are included herein for design consideration, pipes less than 2400 mm {8 ft.} in diameter are generally a roadway item (see Design Manual, Part 2, Section 2.10).

12.2 DEFINITIONS

The following shall supplement A12.2.

Tied Arch - An arch section designed and constructed with an integral base slab.

12.3 NOTATION

The following shall supplement A12.3.

BD = load factor for dead load (dim)

BL = load factor for live load (dim)

d = depth of corrugation (mm) {in.} (12.6.2.1)

D_h = horizontal span of metal pipe (mm) {in.} (12.6.2.1)

DL = dead load (N) {kips}

D_L = deflection lag factor (dim)

D_v = vertical rise of metal pipe (mm) {in.} (12.6.2.1)

i = interest rate (dim)

I = inflation rate (dim)

K = bedding constant (dim)

LCC = present worth life cycle cost ($)

LL = live load (N) {kips}

MC = maintenance cost ($)

n = service life (yr)

n_p = material design life (yr)

N_f = flexibility number (dim)

P	=	design load (MPa) {ksi}
PC	=	present cost ($)
P_{DL}	=	design dead load (N/mm) {kip/in}
P_{LL}	=	design live load (N/mm) {kip/in}
PS	=	pipe stiffness (N/mm/mm) {kip/in/in}
S	=	mean pipe diameter = 2 C + ID/2)
SV	=	salvage value ($)
W_c	=	vertical load per unit of pipe length (N/mm) {kip/in}
δ_c	=	vertical crown deflection with respect to the invert (mm) {in.} (12.6.2.1)
ε_b	=	bending strain in thermoplastic pipe (mm/mm) {in/in}
ε_c	=	hoop strain in thermoplastic pipe (mm/mm) {in/in}
ε_T	=	total strain in thermoplastic pipe (mm/mm) {in/in}
μ_m	=	corrugated metal pipe shape factor (dim) (12.6.2.1)

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12.4 SOIL AND MATERIAL PROPERTIES

12.4.1 Determination of Soil Properties

12.4.1.1 GENERAL

The following shall supplement A12.4.1.1.

The provisions of D10.4.1 shall apply.

The foundation materials shall be sampled and tested to a depth below the anticipated foundation level approximately equal to the rise plus the height of cover, unless rock is encountered at a shallower depth. Where soft, compressible deposits are encountered, all exploration shall extend to a minimum depth below the anticipated foundation level equal to twice the rise, and at least one boring shall extend a minimum depth below the anticipated foundation level equal to twice the rise plus the depth of cover. The maximum drilling depth should be 15 200 mm {50 ft.}, except for high fills (i.e., fill heights exceeding 15 200 mm {50 ft.}), for which the maximum drilling depth should be 15 200 mm {50 ft.} plus two times the rise.

12.4.1.3 ENVELOPE BACKFILL SOILS

The following shall replace A12.4.1.3.

The type, compacted density and strength properties of the soil envelope immediately adjacent to the pipe shall be established. The structural backfill shall conform to the requirements of Standard Drawings RC-12M and RC-30M and Publication 408.

C12.4.1.3

The following shall replace AC12.4.1.3.

In-situ soil along the sides of the structure need not be excavated (except for placement and compaction of the minimum width of structural backfill) and recompacted unless the quality of the in situ-soil is not comparable to the proposed compacted side fill.

12.4.2 Materials

12.4.2.2 CONCRETE

The following shall replace A12.4.2.2.

For concrete requirements, see D5.4.2.1.

12.4.2.3 PRECAST CONCRETE PIPE

The following shall replace A12.4.2.3.

Precast concrete pipe shall comply with the requirements of Publication 280M.

12.4.2.4 PRECAST CONCRETE STRUCTURES

The following shall replace A12.4.2.4.

Precast concrete arch, elliptical and box structures shall comply with the requirements of Publication 408.

12.4.2.8P BOLTS FOR STRUCTURAL PLATE STRUCTURES

The construction of structural plate structures requires field assembly of the individual plates to form the metal

SPECIFICATIONS COMMENTARY

shell around which the soil envelope is to be compacted. The fasteners used for assembly shall be 19.1 mm {3/4 in.} diameter galvanized steel, stainless steel, or aluminum bolts. The underside of the bolt heads and nuts to be used for assembling structural plate structures shall be uniformly rounded to permit installation on either the crown or valley of the corrugations without stress variation. Washers shall not be required for the assembly of corrugated metal plate structures.

Bolts used for fastening aluminum structural plates shall meet the requirements of ASTM A 307 for steel bolts, and ASTM F 468 for aluminum bolts. Bolts used for fastening steel structural plates shall meet the requirements of ASTM A 449. The galvanizing used on bolts for either aluminum or steel structural plate shall meet the requirements of ASTM A 153. In severely corrosive environments, stainless steel bolts may be used to assemble corrugated aluminum structural plates. Stainless steel nuts and bolts shall meet the requirements of ASTM A 193M.

Hook and anchor bolts shall be used to provide a connection between the ends of a structural plate structure and the headwalls, where applicable. Hook or anchor bolts shall be placed in alternate bolt holes. Hook and anchor bolts shall not be used as fasteners on the end of a longitudinal seam. These bolts and their galvanized coatings shall meet the same criteria as the bolts used to assemble the structure. The nuts for hook and anchor bolts shall meet the requirements of ASTMA 563, Grade C, for steel fasteners and ASTM F 467 for aluminum fasteners.

12.5 LIMIT STATES AND RESISTANCE FACTORS

12.5.5 Resistance Factors

The following shall replace Table A12.5.5-1.

Table 12.5.5-1 – Resistance Factors for Buried Structures

STRUCTURE TYPE	RESISTANCE FACTOR
Metal Pipe, Arch and Pipe Arch Structures	
Helical pipe with lock seam or fully-welded seam: • Minimum wall area and buckling	1.0
Annular pipe with spot-welded, riveted or bolted seam: • minimum wall area and buckling • minimum seam strength	1.0 0.67
Structural plate pipe: • minimum wall area and buckling • minimum seam strength • bearing resistance of pipe arch foundations	1.0 0.67 refer to D10.5.5
Long-Span Structural Plate and Tunnel Liner Plate Structures	
• minimum wall area • minimum seam strength • bearing resistance of pipe arch foundations	0.67 0.67 Refer to D10.5.5
Structural Plate Box Structures	
• plastic moment strength • bearing resistance of pipe arch foundations	1.0 refer to D10.5.5
Reinforced Concrete Pipe	
Direct design method: • flexure • shear • radial tension	0.90 0.85 0.85
Reinforced Concrete Cast-in-Place Box Structures	
• flexure • shear	0.90 0.85
Reinforced Concrete Precast Box Structures	
• flexure • shear	1.00 0.90
Thermoplastic Pipe	
PE and PVC pipe: • minimum wall area and buckling • flexure	1.00 1.00

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12.5.6 Flexibility Limits and Construction Stiffness

The following shall supplement A12.5.6.

The flexibility factor, FF, in mm/N {in/kip}, of metal pipe shall be calculated as:

$$FF = \frac{D_e^2}{EI} \quad (12.5.6\text{-}1)$$

where:

I = moment of inertia (mm^4/mm) {in^4/in}

E = long-term modulus of elasticity of pipe material (MPa) {ksi}

D_e = effective diameter of pipe (mm) {in.}

The construction stiffness factor, C_s, in N/mm {kip/in}, for steel tunnel liner plate shall be calculated as:

$$C_S = \frac{EI}{D_e^2} \quad (12.5.6\text{-}2)$$

where:

I = moment of inertia (mm^4/mm) {in^4/in}

E = long-term modulus of elasticity of the steel tunnel liner plate (MPa) {ksi}

D_e = effective diameter of pipe (mm) {in.}

12.5.6.2 SPIRAL RIB METAL PIPE AND PIPE ARCHES

The following shall replace Table A12.5.6.2-1.

COMMENTARY

C12.5.6

An upper limit of FF and a lower limit of C_s are used as an indication of the minimum stiffness of the pipe wall so that it can be handled and backfilled without inducing yield. The upper limits of FF in A12.5.6.1, A12.5.6.2 and A12.5.6.3 are based on empirical evaluation of successful pipe installation provided by industry.

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Table 12.5.6.2-1 - Flexibility Factor Limits

Metric Units			
MATERIAL	CONDITION	CORRUGATION SIZE (mm)	FLEXIBILITY FACTOR (mm/N)
Steel	Embankment	19 x 19 x 190 19 x 25 x 290	$0.049\ I^{1/3}$ $0.031\ I^{1/3}$
Aluminum	Embankment	19 x 19 x 190 19 x 25 x 290	$0.076\ I^{1/3}$ $0.039\ I^{1/3}$

U.S. Customary Units			
MATERIAL	CONDITION	CORRUGATION SIZE (in.)	FLEXIBILITY FACTOR (in/kip)
Steel	Embankment	0.75 x 0.75 x 7.5 0.75 x 1.0 x 11.5	$217\ I^{1/3}$ $140\ I^{1/3}$
Aluminum	Embankment	0.75 x 0.75 x 7.5 0.75 x 1.0 x 11.5	$340\ I^{1/3}$ $175\ I^{1/3}$

12.6 GENERAL DESIGN FEATURES

12.6.2 Service Limit State

12.6.2.1 TOLERABLE MOVEMENT

The following shall supplement A12.6.2.1.

For elliptical and round corrugated metal pipe, the crown deflection shall not exceed δ_c, given by:

$$\delta_C = \mu_m \frac{D_h^2}{d} \qquad (12.6.2.1\text{-}1)$$

where:

δ_c = crown deflection (mm) {in.}

μ_m = shape factor (dim)

D_h = horizontal span of pipe (mm) {in.}

d = depth of corrugation (mm) {in.}

The value of the shape factor is obtained from Figure 1.

For long-span structures, the following deformation criteria shall apply:

- For horizontal ellipse shapes having a ratio of top to

COMMENTARY

C12.6.2.1P

During the initial stages of backfilling, when the pipe is not fully contained by the soil, the pipe wall undergoes high deformations. These, if allowed to exceed certain limits, will include permanent set in the wall. The provision in this clause is aimed at restricting such excessive deformations.

The limits of vertical crown deflections for round and elliptical pipes were derived by an analysis in which the pipe walls, with a 152 mm x 51 mm {6 in. x 2 in.} corrugation profile, were subjected to uniform lateral pressures. Limits of the crown deflections, which could be characterized by the span-to-rise ratios of the pipe, were established so that the stresses anywhere in the cross-section remained within 90% of the yield stress of the steel. Values of the non-dimensional parameter μ_m, given in Figure 1, were calculated from the results of the above analysis. For shapes other than round and elliptical, the limit of the vertical deflection is based mostly on practical considerations, rather than on analysis.

The crown deflection limits, specified in the clause, are meant for construction rather than design control. If the deformations begin to exceed the specified limits, measures should be taken to contain them. With the present state-of-the-art, it is difficult to "design" a pipe wall so that it can

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side radii of three or less, the span and rise shall not deviate from the specified dimensions by more than 2%.

- For arch shapes having a ratio of top to side radii of three or more, the rise shall not deviate from the specified dimensions by more than 1% of the span.

- For all other long-span structures, the span and rise shall not deviate from the specified dimensions by more than 2%, nor more than 120 mm {5 in.}, whichever is less.

For structural plate box structures, the rise shall not deviate from the specified dimensions by more than 1% of the span.

For other corrugated metal pipe shapes and thermoplastic pipes, the crown deflection shall not exceed 5% of the rise, unless otherwise approved.

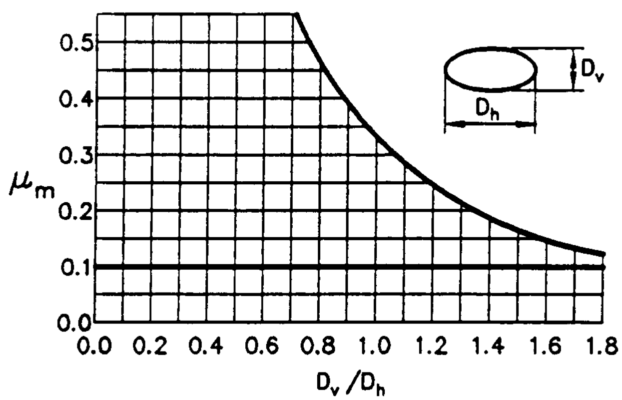

Figure 12.6.2.1-1 - Values of μ_m for Elliptical and Round Corrugated Metal Pipe (Ontario Ministry of Transportation, 1992)

COMMENTARY

comply with the provision of this article.

12.6.2.2 SETTLEMENT

12.6.2.2.1 General

The following shall supplement A12.6.2.2.1.

- differential settlement between the pipe and endwall

When the foundations for soil-steel structures have markedly non-uniform settlement properties within the extent of the pipe and its structural backfill, appropriate measures, including the removal of unsuitable materials, shall be taken to avoid their detrimental effects on the structure.

C12.6.2.2.1P

Foundations with reasonably uniform settlement properties are necessary to avoid situations where the invert may be founded partially on compressible materials and partially on incompressible materials. Lack of uniformity along the invert bedding could induce undesirable stress concentrations in the pipe wall above the incompressible areas. A condition in which the foundation under the pipe is less compressible than that of the adjacent areas should be avoided. In such conditions, columns of soil adjacent to the pipe settle more than the column of soil above the pipe, thus inducing negative arching which, in turn, increases the thrust. Foundations with fairly uniform settlement

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12.6.2.2.2 Longitudinal Differential Settlement

The following shall replace the second sentence in the first paragraph of A12.6.2.2.2.

Pipes and culverts subjected to longitudinal differential settlements shall be fitted with positive joints to resist disjointing forces.

12.6.2.2.4 Footing Settlement

The following shall supplement A12.6.2.2.4.

Metal box culverts shall not be used where foundation soil or loading conditions are likely to lead to differential settlement between footings.

12.6.2.2.5 Unbalanced Loading

The following shall supplement A12.6.2.2.5.

Due to the complexity of determining the actual load distribution on a structure subjected to unbalanced loading, the problem can be modeled using numerical methods or approximated as an edge beam. If the edge beam method is used, the provisions of A4.6.2.1.4b for longitudinal edges shall be applied and checked along the skewed section (see Figure 1) and the normal main reinforcement shall be provided as shown in Figure 1.

Refer to Appendix H "Pennsylvania Installation Direct Design (PAIDD) for Concrete Pipes" for the design of concrete pipes subjected to unbalanced loading.

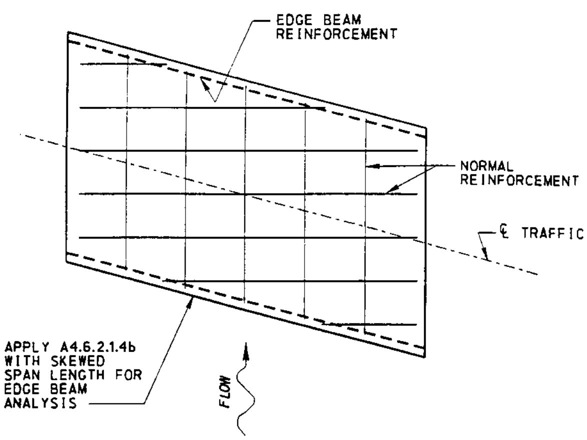

Figure 12.6.2.2.5-1 - Reinforcement Requirement for Box Culvert with Skew Ends

COMMENTARY

properties can be provided by adjusting the bedding thickness, replacing compressible materials, shattering bedrock, and other similar treatments.

C12.6.2.2.5

The following shall supplement AC12.6.2.2.5.

Skewed culverts are subjected to horizontally unbalanced soil pressures in the sloped embankment range. This unbalanced distributed force has to be carried by the culvert and by the headwall if it is shear connected to the culvert. In the case of a skewed culvert with shear connected headwall, the culvert is restrained by the headwall from deflecting laterally to carry the unbalanced horizontal load by bending. Instead, a support system (as shown in Figure AC12.6.2.2.5-1) may develop which carries the unbalanced distributed load, E(x), by a horizontal shear component, F, acting between headwall and culvert, and shear forces, S_1 and S_2, acting along the culvert bearing lines.

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12.6.2.2.6P Structure on Unyielding Foundation

Buried structures shall not be placed directly on an unyielding foundation. Unless determined by a special analysis (e.g., finite element analysis), a minimum 600 mm {2 ft.} thickness of soil bedding shall be placed between the foundation and the bottom of the structure.

12.6.2.2.7P Endwall Settlement

The design of headwalls shall provide sufficient restraint to maintain the shape of the structure under the design loads. The footings of headwalls and wingwalls shall be proportioned so that the calculated settlement of the corrugated metal shell shall be the same as the calculated settlement of the headwalls and endwalls.

12.6.2.3 UPLIFT

The following shall supplement A12.6.2.3.

To satisfy this condition, the dead load on the crown of the structure shall exceed the buoyancy of the culvert, drainage shall be provided to maintain the groundwater at a level below the culvert, or the structure shall be anchored to resist uplift forces. Whenever practical, the drainage option shall be used.

12.6.5 Scour

The following shall supplement A12.6.5.

Cutoff walls or scour curtains shall extend to a minimum depth of 750 mm {2'-6"} below all pipe inlets and outlets and arch structure footings placed over erodible deposits in accordance with Standard Drawing BC-791M.

12.6.6 Soil Envelope

C12.6.6.1 TRENCH INSTALLATIONS

12.6.6.2 EMBANKMENT INSTALLATIONS

The following shall supplement A12.6.6.2.

Refer to Standard Drawing RC-30M for guidance regarding minimum soil envelope widths.

12.6.6.3 MINIMUM SOIL COVER

The following shall supplement A12.6.6.3.

The minimum soil cover for thermoplastic pipe shall be 600 mm {2 ft.}, the pipe ID, or the tables in Chapter 10 of

COMMENTARY

C12.6.2.2.6P

The effect of constructing a buried structure directly on an unyielding foundation is an amplification of the vertical earth load on the culvert.

C12.6.2.3

Delete AC12.6.2.3.

C12.6.6.1

The following shall replace the first paragraph of AC12.6.6.1.

Refer to Standard Drawing RC-30M for guidance regarding minimum trench width.

C12.6.6.2

Delete AC12.6.6.2.

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Design Manual Part 2, whichever is greater.

For construction requirements for thermoplastic pipes, refer to Standard Drawing RC-30M (based on ASTM D 2321).

12.6.6.4P MAXIMUM SOIL COVER

See Standard Drawing BD-635M and DM-2 for height of cover limits for metal culverts and thermoplastic pipes.

12.6.7 Minimum Spacing Between Multiple Lines of Pipe

The following shall supplement A12.6.7.

Allow 600mm {2 ft.} distance for minimum spacing requirements between multiple lines of pipe. Refer to Standard Drawing BD-636M and Appendix H, "Pennsylvania Installation Direct Design (PAIDD) for Concrete Pipes" for minimum spacing requirements between multiple lines of concrete pipe.

12.6.8 End Treatment

12.6.8.1 GENERAL

The following shall supplement A12.6.8.1.

Refer to Standard Drawings BD-631M, BD-632M, BD-633M, BD-634M, RC-30M, RC-31M and RC-33M for end treatment requirements for buried structures.

A culvert with a skew of 75 degrees and a two horizontal to one vertical (2H:1V) bevel shall not be exceeded without end reinforcement. Square ends may be designed with side plates beveled up to a maximum 2H:1V slope without reinforcement. Skew ends up to 75 degrees with no bevel are permissible. Skew ends on spans over 6000 mm {20 ft.} shall be protected with a reinforced concrete headwall or structural steel collar. When partial headwalls are skewed, the offset portion of the metal structure shall be supported by the headwall. The maximum skew shall be limited to 55 degrees.

For hydraulic structures, additional reinforcement of the end is recommended to secure the metal edges at the inlet and outlet against hydraulic forces. Reinforced concrete, structural steel collars, tension tiebacks or anchors in soil, partial headwalls, or cutoff walls below the invert elevation are methods which may be considered. See Figure 1 for typical slope wall for metal culverts.

Beveled ends shall be anchor-bolted to headwalls or slope walls at approximately 450 mm {18 in.} intervals. Square-end structures may be anchored to a headwall with bolts in alternate standard circumferential holes.

Reinforced concrete headwalls shall be provided for concrete pipes greater than 2400 mm {8 ft.} in diameter.

End treatments for thermoplastic pipe are presented in Standard Drawing RC-30M.

The type of end treatment for culverts shall be approved

COMMENTARY

C12.6.6.4P

Analysis runs are available for maximum soil cover criteria for thermoplastic pipes.

C12.6.7

Delete AC12.6.7.

C12.6.8.1

The following shall supplement AC12.6.8.1.

Square ends are the most common and generally most economical ends, and are used when the structure is installed normal to a roadway. Unless required by the velocity of water through the conduit, no further inlet or outlet treatment is needed.

Because the structural capacity of a long-span structure is derived from the interaction of the corrugated metal shell and the surrounding soil, it is very important to evaluate the loading effects at the free ends. Whenever possible, the free ends should be located at a distance equal to the height of fill from the edge of a trafficable shoulder.

End structures for culvert inlets serve to improve the hydraulic capacity of the structure and reduce the potential for scour and piping. Inlets should provide a smooth transition from the channel to the culvert to accomplish the following:

- Minimize hydraulic head losses due to turbulence
- Prevent or minimize the contraction of the water channel entering the culvert barrel

Potential methods for achieving a smooth transition and inlet protection include the following:

- Riprap extending to the level of the maximum flow elevation
- Recessing the invert slightly below the streamed elevation to prevent undercut
- Construction of a headwall, or headwall and wingwall

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by the District Bridge Engineer.

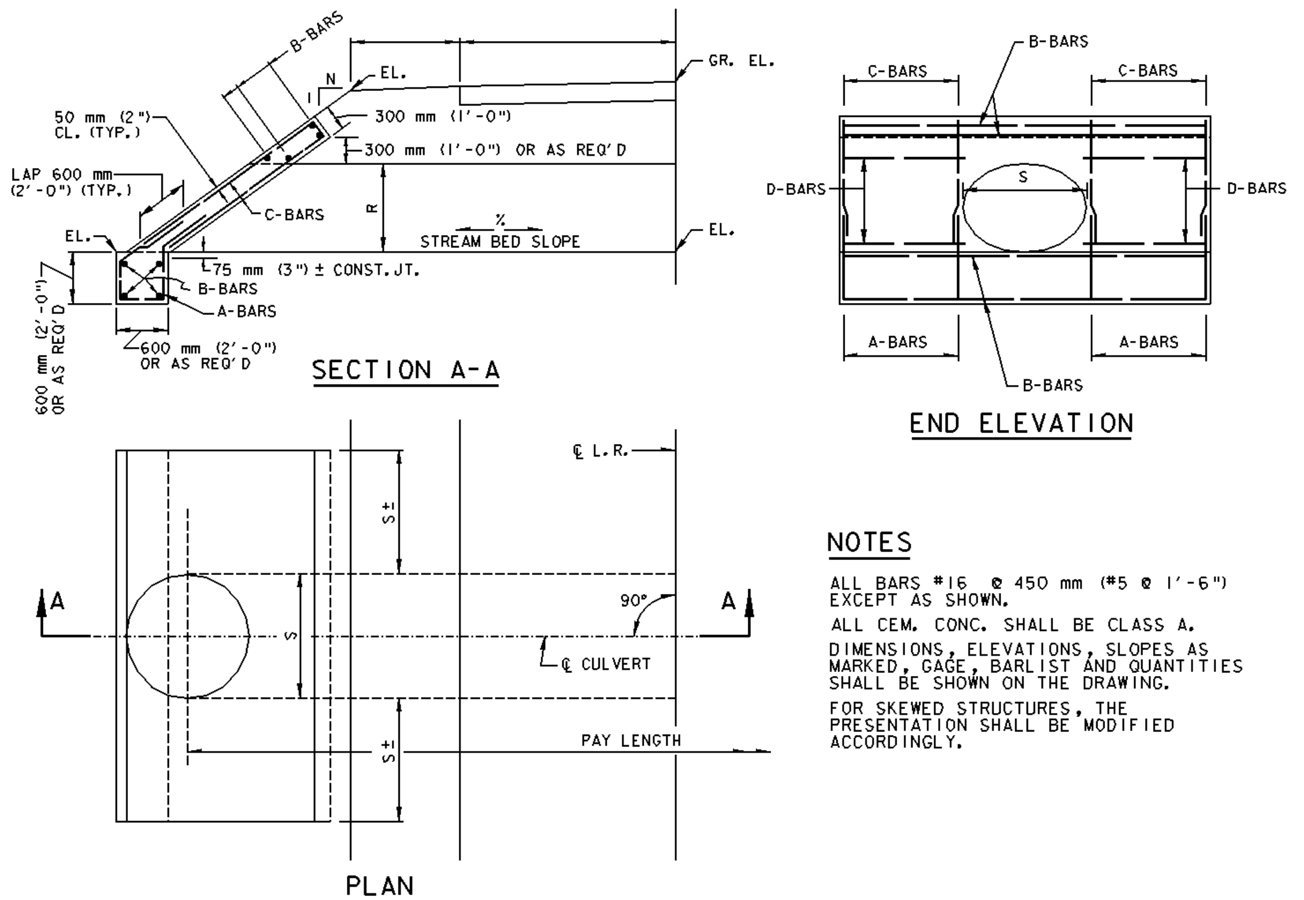

Figure 12.6.8.1-1 - Slope Wall for Metal Culverts

12.6.8.2 FLEXIBLE CULVERTS CONSTRUCTED ON SKEW

The following shall supplement A12.6.8.2.

Disregarding the effect of lateral unbalanced forces during headwall design may lead to failure of the headwall and adjacent culvert sections.

Unless otherwise directed, the angle of end walls shall be in accordance with Table 1. The angle of end walls shall be shown on the preliminary plans submitted for type, size and location approval.

When the end wall is not parallel to the edge of shoulder, the embankment slope shall be warped to cover the exposed portion of the structure. Show on the plans when warping of embankment slope is necessary, similar to the grading shown in Figure AC12.6.8.2-1.

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Table 12.6.8.2P-1 – End Wall Angle

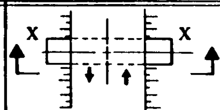
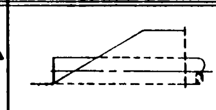
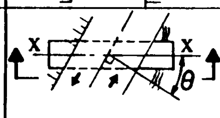
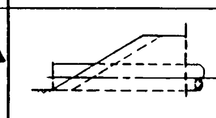
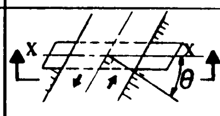
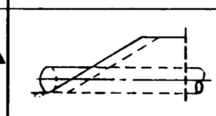
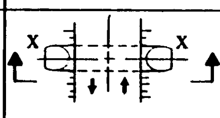
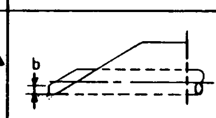
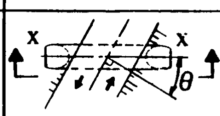
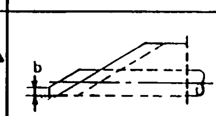
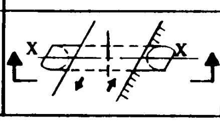
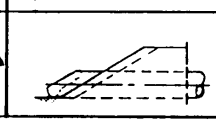
Fill Height at Edge Shoulders	Skew Angle (deg)	Angle of End Wall (deg)
≤ 1.2 m {≤ 4 ft.}	Any	Parallel to edge of shoulders
1.2 to 2.4 m {4 to 8 ft.}	90 – 75 75 – 60 less than 60	90* 75* 60
>2.4 m {>8 ft.}	90 – 60 less than 60	90* 60

*If shallow fills do not permit warping of the embankment slopes to eliminate exposure of the culvert, end walls shall be constructed parallel to edge of shoulders.

The structural design provisions, herein, are intended for square-ended structures with uniform backfill and surface loading. Deviation from these conditions shall be evaluated. Table 2 provides guidance for a variety of skew and bevel conditions.

Table 12.6.8.2P-2 - Requirements for Cut Ends without Headwalls (Kaiser Aluminum, 1985)

REQUIREMENTS FOR CUT ENDS WITHOUT HEAD WALLS
(Kaiser Aluminum, 1985)

No.	Cut End	Typical View of Installation		Requirements
		Plan	Section XX	
1	square end with roadway parallel to tranverse direction			
2	square end with roadway skew to transverse direction			θ shall be less than 40°. For θ > 20°, the earth pressure imbalance shall be accommodated either by structural reinforcement of conduit wall or by contour grading of the embankment slope.
3	skew end			As for No. 2.
4	square bevel with roadway parallel to transverse direction			b shall not be less than (rise/8). The ends shall be treated as a retaining structure and shall be designed accordingly. b - bottom step
5	square bevel with roadway skew to transverse direction			As for Nos. 3 and 4.
6	skew bevel			Requires approval.

12.6.8.3P PRECAST AND CAST-IN-PLACE BOX CULVERTS

The ends of box sections shall be normal to the walls and centerline of the box sections. When a beveled end is specified at the culvert end, an edge beam shall be required.

12.6.8.4P FISH PASSAGE THROUGH CULVERTS

Fish passage shall be provided in culverts in fishable streams having continuous flow.

See Design Manual, Part 2, Chapter 10, for guidance regarding the design of low-flow fish passage systems through culverts.

12.6.9 Corrosive and Abrasive Conditions

The following shall supplement A12.6.9.

Flexible corrugated metal structures shall be designed to provide the structural capacity required by the appropriate design loads throughout the design life of the structure. The corrugated metal plates shall include either a sacrificial

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thickness equal to the expected metal loss (due to corrosion and abrasion) or a protective coating which is capable of sustaining the metal structure.

Refer to D12.6.9.4P for acceptable protective coatings.

12.6.9.1P CORROSION

Corrosion of corrugated metal or reinforcing steel in concrete drainage structures is a major concern in the selection of the materials to be used for a buried structure. Indicators of corrosion potential include soil and water pH, resistivity, oxidation-reduction potential, chemical composition of backfill soils, precipitation, and flow velocity. As a minimum, the following information shall be developed for the culvert design:

(a) pH of soil, surface water and groundwater

(b) resistivity of soil, surface water and groundwater

(c) sulfate content of the stream flow or runoff

See Table 1 for typical resistivity values for soil and water. For general guidelines regarding soil corrosion, see Table 2. For cases in which abrasion is not a problem, cathodic protection can be provided to reduce the corrosion potential of metal.

Table 12.6.9.1P-1 - Typical Resistivity Values

Soil		Water	
Classification	ohm-cm	Source	ohm-cm
Clay	750-2000	Seawater	25
Loam (silts)	3000-10 000	Brackish	2000
Gravel	10 000-30 000	Drinking Water	4000+
Sand	30 000-50 000	Surface Water	5000+
Rock	50 000-Infinity*	Distilled Water	Infinity*

*Theoretical

Source: Adapted from AASHTO (1993)

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Table 12.6.9.1P-2 - Corrosiveness of Soils

Soil Type	Description of Soil	Aeration	Drainage	Color	Water Table
I Lightly Corrosive	Sands or sandy loams Light textured silt loams Porous loams or clay loams thoroughly oxidized to great depths	Good	Good	Uniform Color	Very Low
II Moderately Corrosive	Sandy Loams Silt Loams Clay Loams	Fair	Fair	Slight Mottling	Low
III Badly Corrosive	Clay Loams Clays	Poor	Poor	Heavy Texture Moderate Mottling	600 to 900 mm {2 to 3 ft.} below surface
IV Unusually Corrosive	Muck Peat Tidal Marsh Clays and Organic Soils	Very Poor	Very Poor	Bluish-Gray Mottling	At surface or extreme impermeability

Source: (Hurd, 1984)

12.6.9.2P ABRASION

Waters with suspended bed loads (i.e., sand-, gravel- and cobble-size particles) flowing at high velocities (i.e., greater than about 3000 mm/s {10 ft/sec}) and creating appreciable turbulence can cause severe localized deterioration resulting from the combined action of mechanical abrasion and corrosion. Known as corrosion-abrasion, corrosion-erosion, or erosion-corrosion, this includes both impingement attack and cavitation. These effects are very difficult to separate from corrosion in metal culverts as the two often work in conjunction with each other.

12.6.9.3P MATERIALS

Those materials subject to deterioration by corrosion and abrasion which are used for buried structures include steel, aluminum and concrete. Other culvert materials which are not considered in A12.6.9 and D12.6.9 here include vitrified clay, stainless steel, cast iron and thermoplastic.

12.6.9.3.1P Steel

Recommended conditions for the installations of corrugated galvanized or aluminized steel buried structures include:

(a) Soil and water pH within the range of 5.5 to 8.5

C12.6.9.3.1P

For installation in mildly acidic environments, uncoated galvanized steel is less suitable than aluminum.

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(b) Resistivity greater than 6000 ohm-cm

12.6.9.3.2P Aluminum

Aluminum is suitable for use in neutral to mildly acidic environments and performs well in organic acid environments. Recommended conditions for the installation of corrugated aluminum drainage structures include:

(a) Soil and water pH within the range of 4 to 8.5

(b) Resistivity greater than 500 ohm-cm

12.6.9.3.3P Concrete

Concrete culverts, because of their rigidity, are susceptible to foundation movements which can expose steel reinforcement to corrosion. Concrete surfaces exposed to flow and soil are susceptible to sulfate attack and abrasion. The use of high compressive strength concrete and durable, hard aggregate increases abrasion resistance. Concrete is resistant to many chemicals, but is subject to acid attack, especially sulfuric and sulfurous acids from acid mine drainage. Acceptable effluent limitations for concrete culvert include:

(a) pH greater than 4.0

(b) Sulfate content in solutions within the range of 100 to 1000 ppm

Rubber gaskets for circular pipe connections may be used in the following situations:

(a) Whenever foundation conditions are conducive to differential movement between sections

(b) If the culvert is placed under more than 4500 mm {15 ft.} of fill

(c) If the culvert is constructed on an embankment

(d) If joint failure could be difficult or expensive to correct

Whenever box culvert or vertical or horizontal elliptical culvert sections are specified, foundation conditions shall be stabilized to reduce the potential of differential settlement between culvert sections. Vitrified clay liner plates may also be specified in accordance with PP3.4.5 for corrosion protection.

12.6.9.4P PROTECTIVE COATINGS

Because environmental conditions at drainage structure

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C12.6.9.3.2P

On aluminum surfaces, as with other metals, an oxide film develops which increases the metal's resistance to corrosion. Abrasion will remove this film and allow an acceleration of corrosion which may or may not reform the oxide film.

C12.6.9.4P

The Department does not generally use protective

SPECIFICATIONS

locations do not always meet the recommended criteria for aluminum, concrete, or steel buried structures, coatings can be added to protect the structure from corrosion and abrasion.

Acceptable protective coatings include the following:

- Shop- or field-applied coatings applied in accordance with AASHTO M 190, M 224 and M 245M for metal and concrete pipe products.
- Sacrificial metal plate thickness for corrugated metal structures.
- Class A cement concrete paving in the bottom of pipes or pipe arches in accordance with Publication 408, Section 603.

Refer to Design Manual, Part 2, and Standard Drawing BD-635M for guidance regarding specific applications and limitations.

No service life credit shall be given for unreinforced concrete lining of corrugated metal culverts.

COMMENTARY

coatings on concrete pipe.

For concrete culverts with less than 600 mm {2 ft.} of cover, protective coating may be needed and should be applied to the entire outside face of the top and sides of the culvert

Concrete lining of corrugated metal pipe provided to improve hydraulic efficiency has been observed to crack or break up after installation. This thin lining is not expected to extend the service life of the culvert.

12.6.9.5P PERFORMANCE ESTIMATES

Mathematical formulas and graphical charts have been developed, on the basis of field observations, to estimate the metal loss rate of galvanized corrugated steel culverts and structural steel plate culverts, as presented in Figures 1 and 2. The minimum metal loss rates shall be 0.050 mm/year {2 mil/year} for galvanized metal pipes and 0.025 mm/year {1 mil/year} for aluminized and aluminum metal pipe. The minimum predicted metal loss rate specified in Figures 1 and 2 shall be added to the required structural wall thickness to provide the required service life of the corrugated steel culverts.

When pH and resistivity data is available, Figure 3 can be used to estimate the years to perforation and to indicate the inspection and maintenance interval of corrugated steel culverts.

Similarly, mathematical formulas and graphical charts have been developed for concrete culverts to estimate the culvert service life as presented in Figure 4. A correction factor is to be applied to the culvert service life because of the effect of sediment depth on the life of the concrete culverts. This correction factor is presented in Figure 5.

C12.6.9.5P

Extensive field studies by New York State Department of Transportation and other states show that the minimum metal loss rates are reasonable.

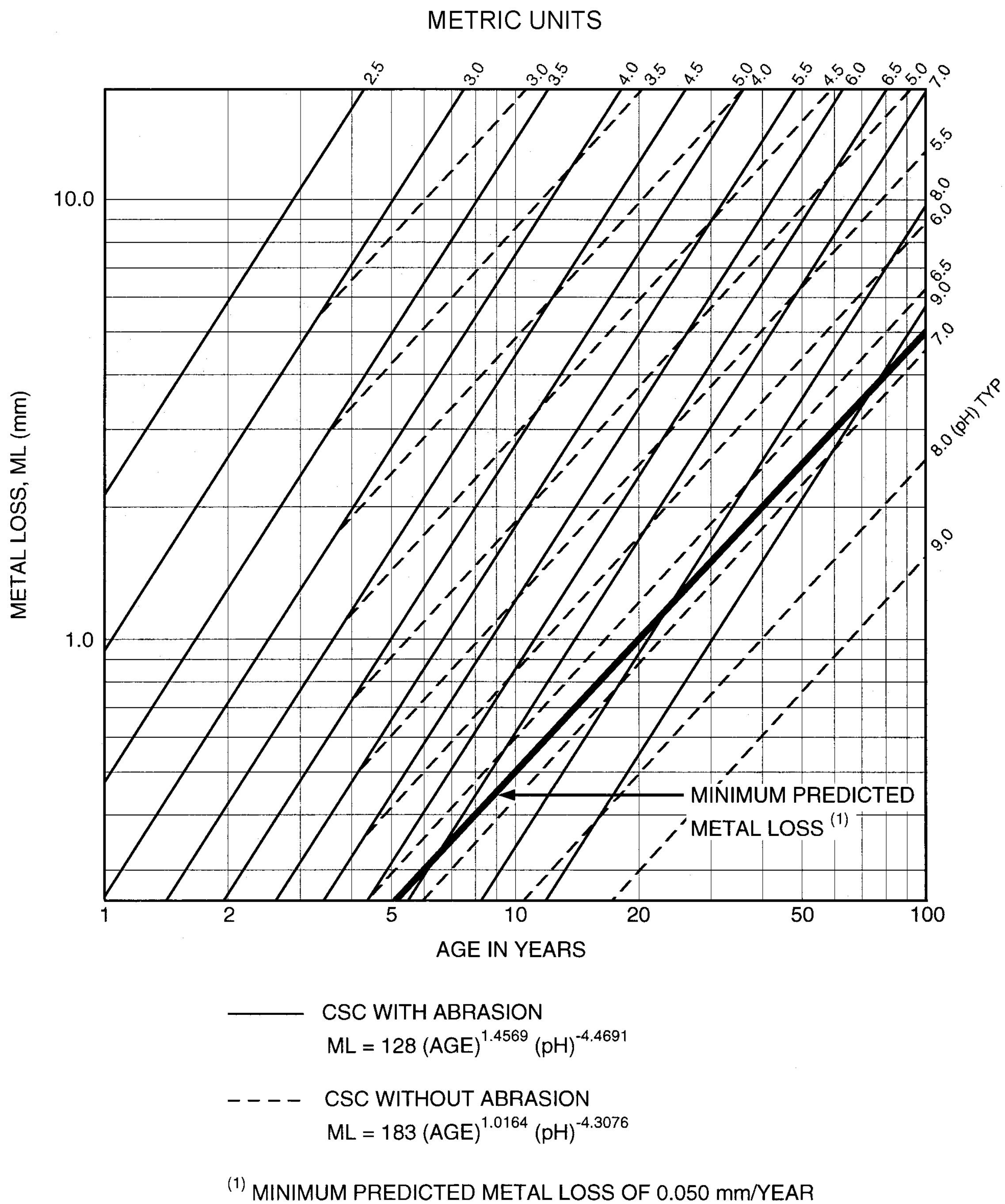


Figure 12.6.9.5P-1 - Predicted Metal Loss Rate for Corrugated Steel Culvert (CSC), AASHTO M 36/M 36M (modified after Hurd, 1984)

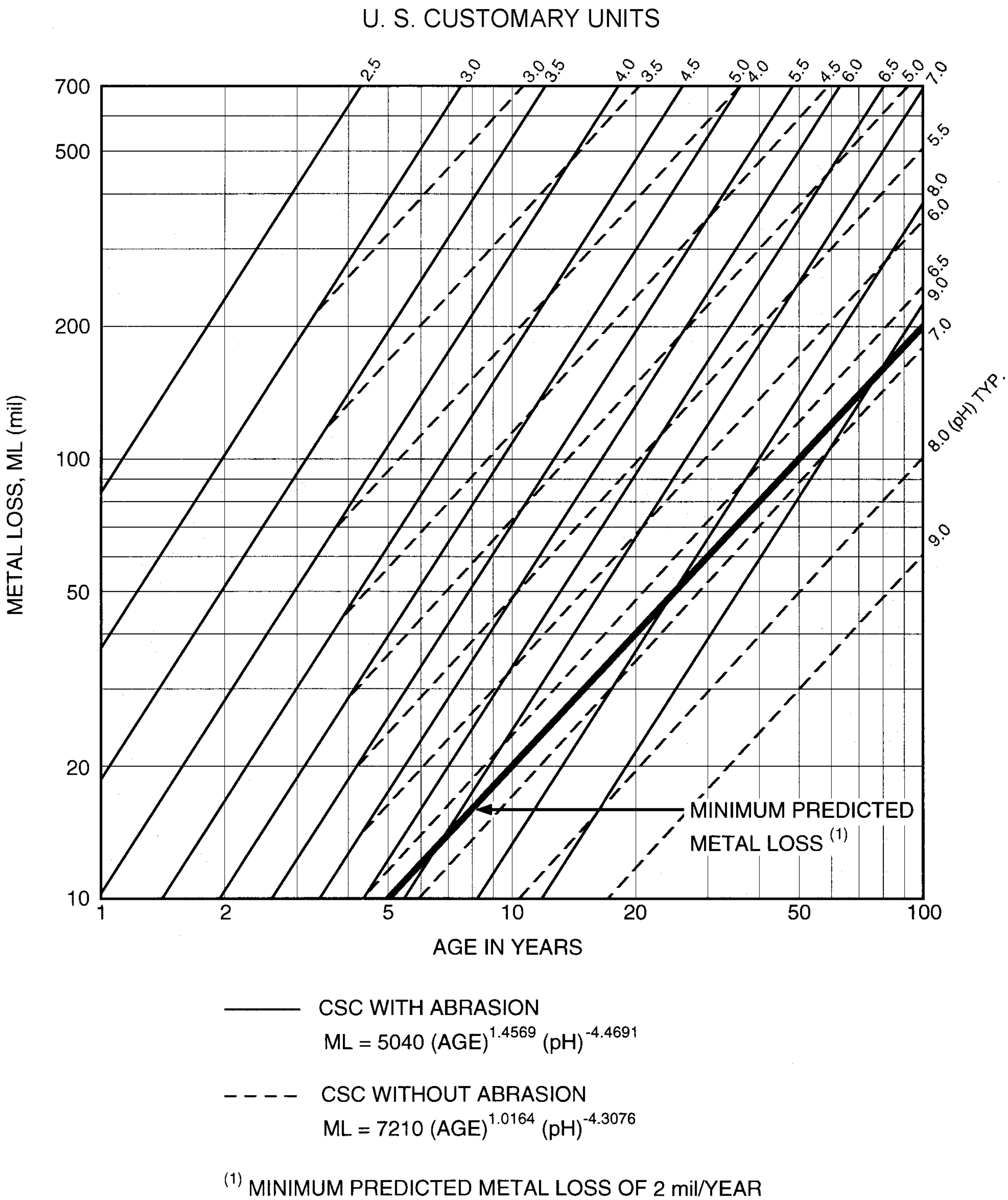


Figure 12.6.9.5P-1 – Predicted Metal Loss Rate for Corrugated Steel Culvert (CSC), AASHTO M 36/M 36M (modified after Hurd, 1984) (continued)

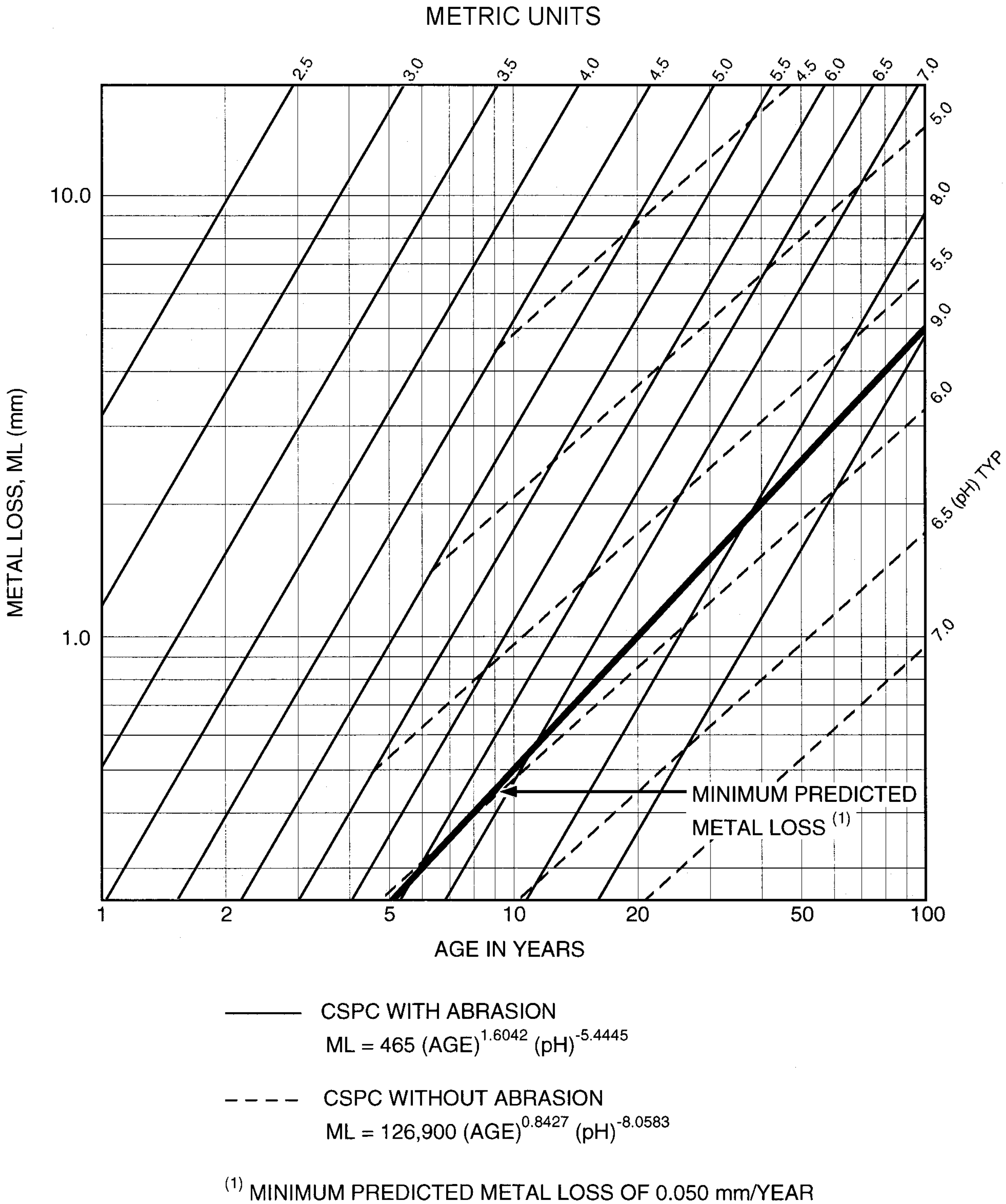


Figure 12.6.9.5P-2 - Predicted Metal Loss Rate for Corrugated Steel Plate Culvert (CSPC), AASHTO M 167/M 167M (modified after Hurd, 1984)

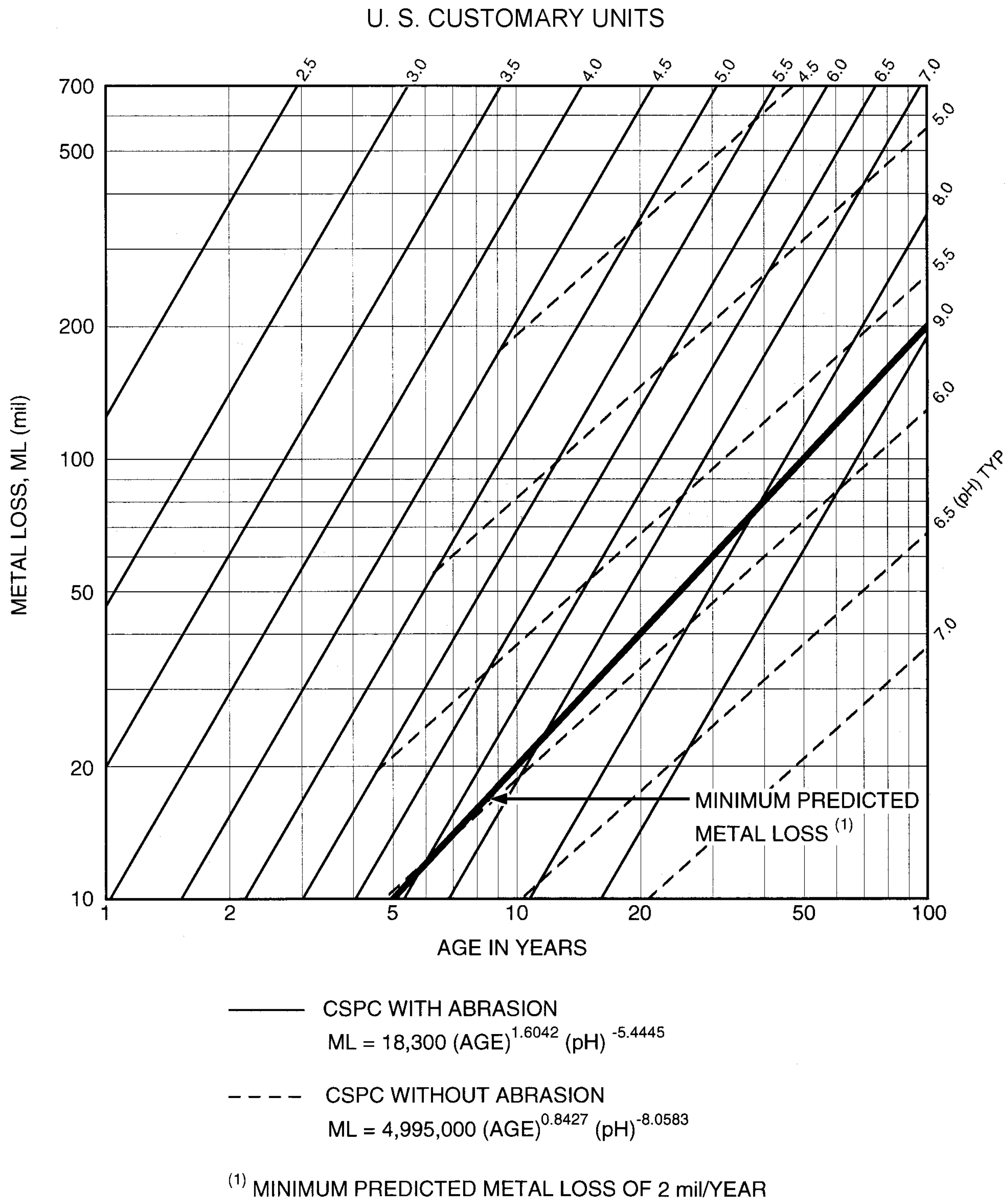


Figure 12.6.9.5P-2 – Predicted Metal Loss Rate for Corrugated Steel Plate Culvert (CSPC), AASHTO M 167/M 167M (modified after Hurd, 1984) (continued)

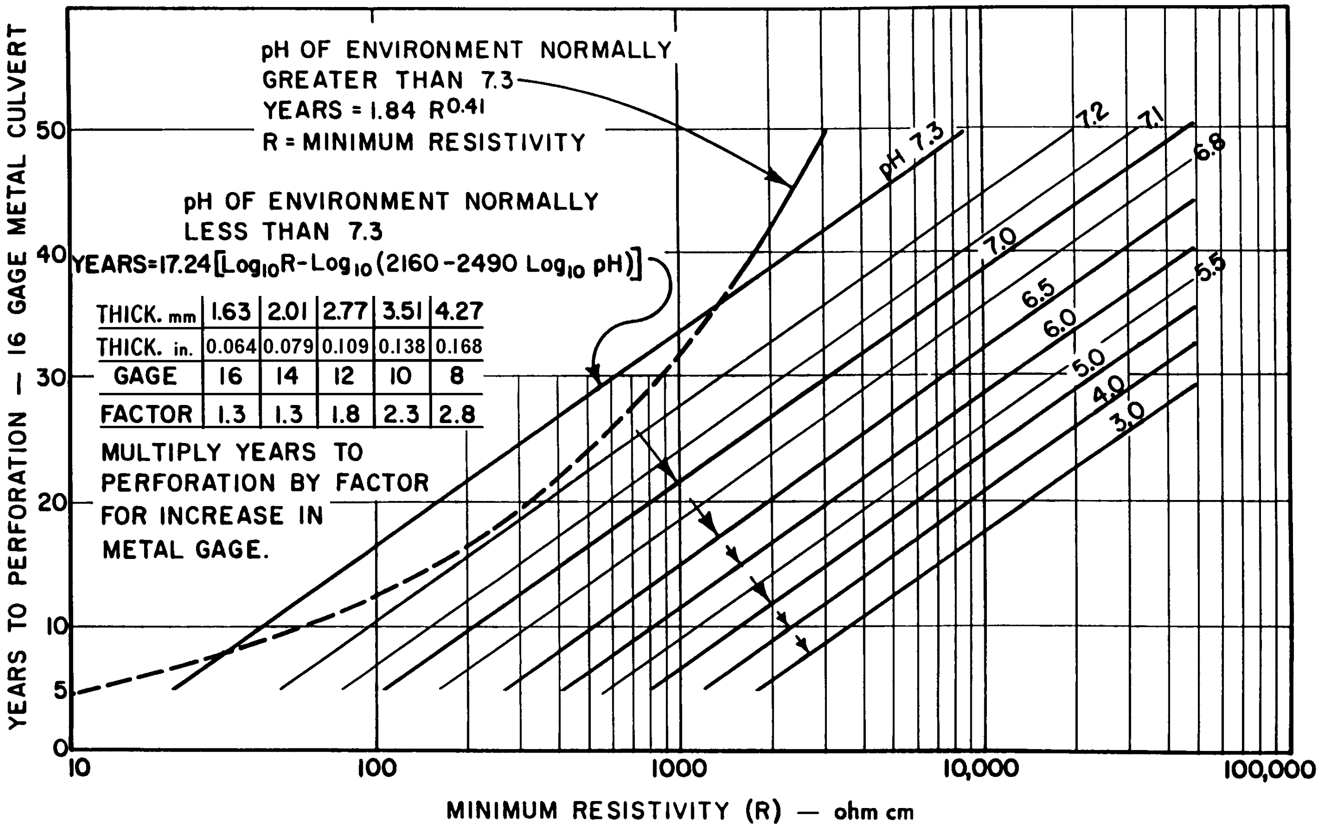
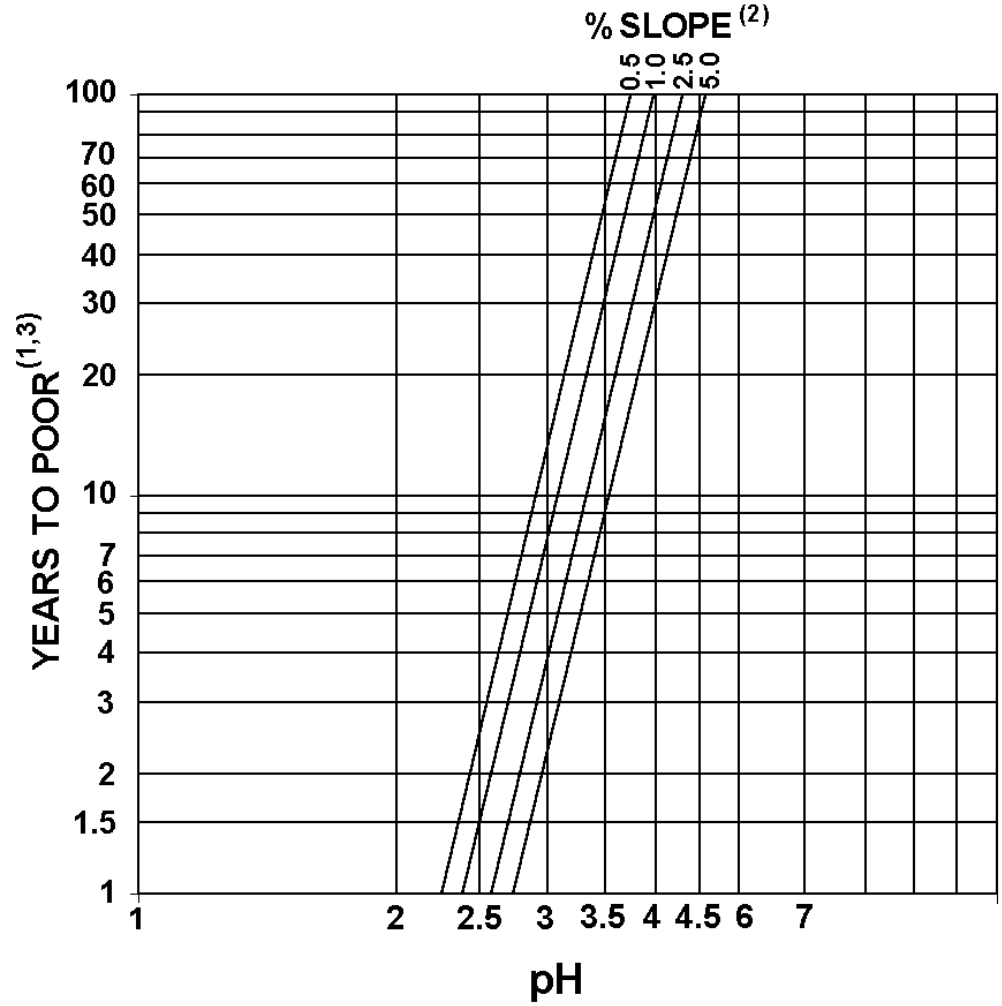


Figure 12.6.9.5P-3 - Estimated Maintenance Interval of Corrugated Steel Pipe (modified after TRB, 1978)



(1) Significant loss of mortar and aggregate; complete loss of invert; concrete in softened condition

(2) Relationships assume no accumulation of sediment

(3) Years to Poor = $\dfrac{\left(0.3509(\text{pH})^{1.205}\right)^{7.457}\left(1+\dfrac{\text{sediment depth}}{\text{rise}}\right)^{8.630}}{(\text{slope})^{0.767}}$

Figure 12.6.9.5P-4 – Predicted Concrete Culvert Life (Meacham, et al, 1982)

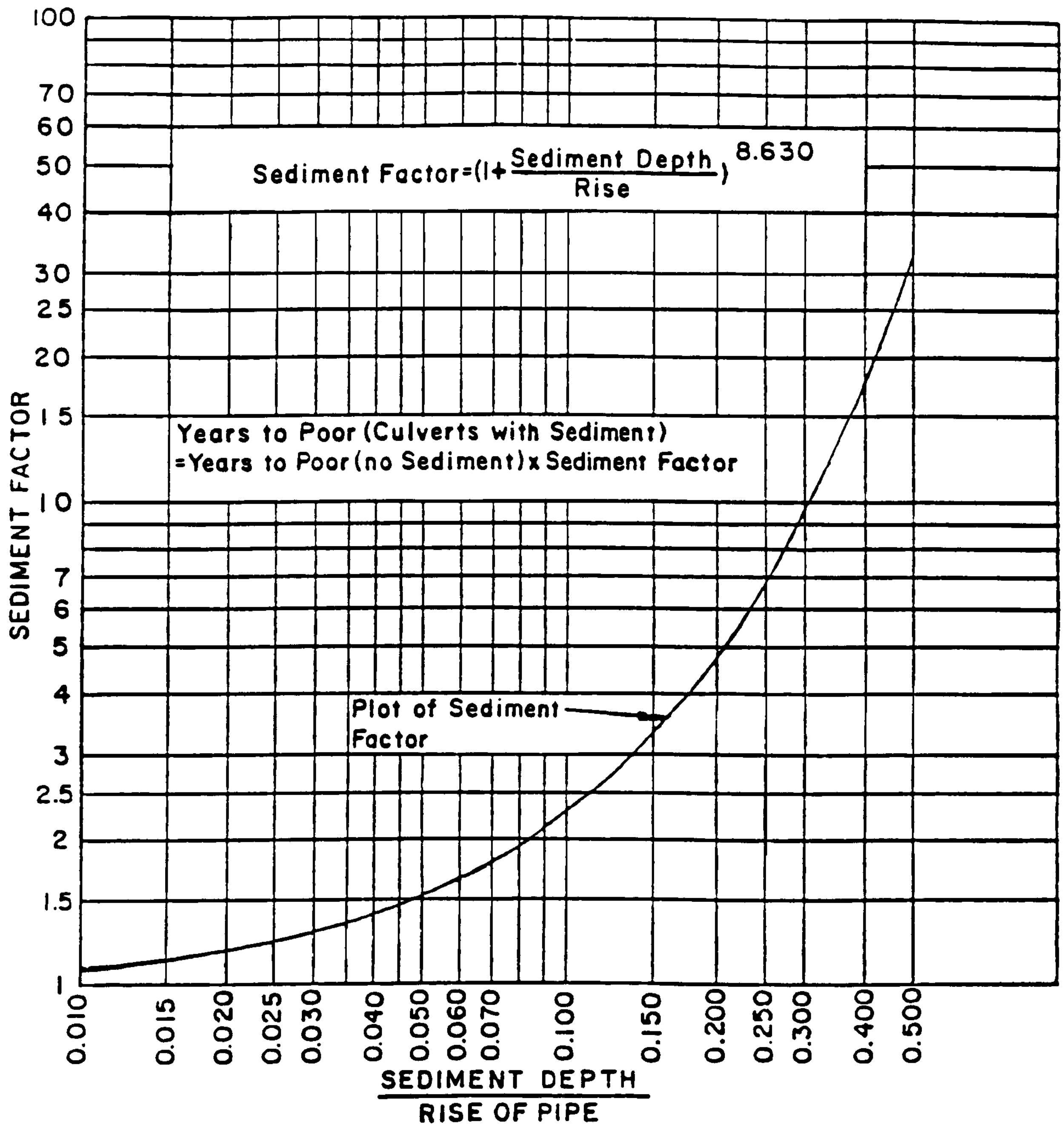


Figure 12.6.9.5P-5 - Effect of sediment depth on life of concrete culverts (Meacham, et al, 1982)

SPECIFICATIONS COMMENTARY

An abrasion resistance performance rating schedule was developed with respect to culvert diameter, mean culvert water velocity, rock size, and culvert slope to estimate aluminum drainage structure service life (see Figure 6 and Table 1). The mean culvert water velocity, V_w, for this analysis is presented in Table 2.

A minimum allowable metal loss rate of 0.025 mm/year {1 mil/year} per year shall be added to the required structural wall thickness to provide the required service life of aluminum and aluminized culverts.

Table 12.6.9.5P-1 - Abrasion Performance Rating Schedule (Koepf and Ryan, 1986)

Performance Zone Ratings	Effect on Surface of Crown of Corrugation, Invert Only*
A	**No surface effect** - No reduction in service life due to bed load abrasion. Projected Abrasion Service Life, 100 years or more.
B	**Non-erosive** - Some slight roughening of the metal surface, but no metal removal by erosion action. No reduction in normal service life of aluminum culvert. Projected Abrasion Service Life, 75 years or more.
C	**Erosion** - Surface roughening and slight progressive removal of metal from culvert. Some gouging may be noted if rocks tend to be large. Projected Abrasion Service Life, 50 years or more.
D	**Abrasion** - Surface roughening and slow removal of metal from culvert. Definite reduction in pipe life due to abrasion. Gouging of surface may be expected. Projected Abrasion Service Life, 25 to 50 years.
E	**Abusive** - Surface roughening and rapid removal of metal from culvert. Definite reduction in pipe life due to abrasion. Projected Abrasion Service Life, 25 years or less.

*Abrasion affects only this portion of the surface. The remainder of the culvert is usually unaffected.

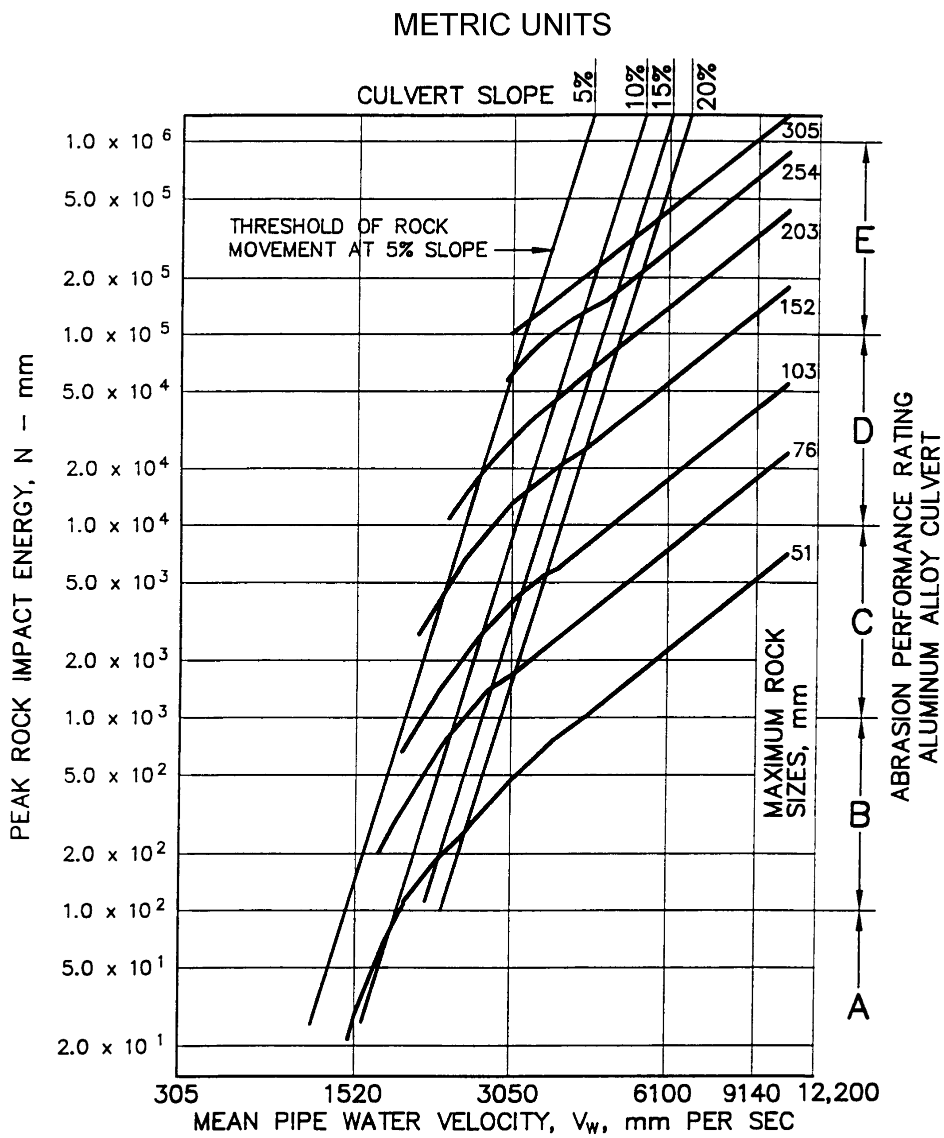


Figure 12.6.9.5P-6 - Abrasion Performance Rating for Aluminum Culvert
(modified after Koepf and Ryan, 1986)

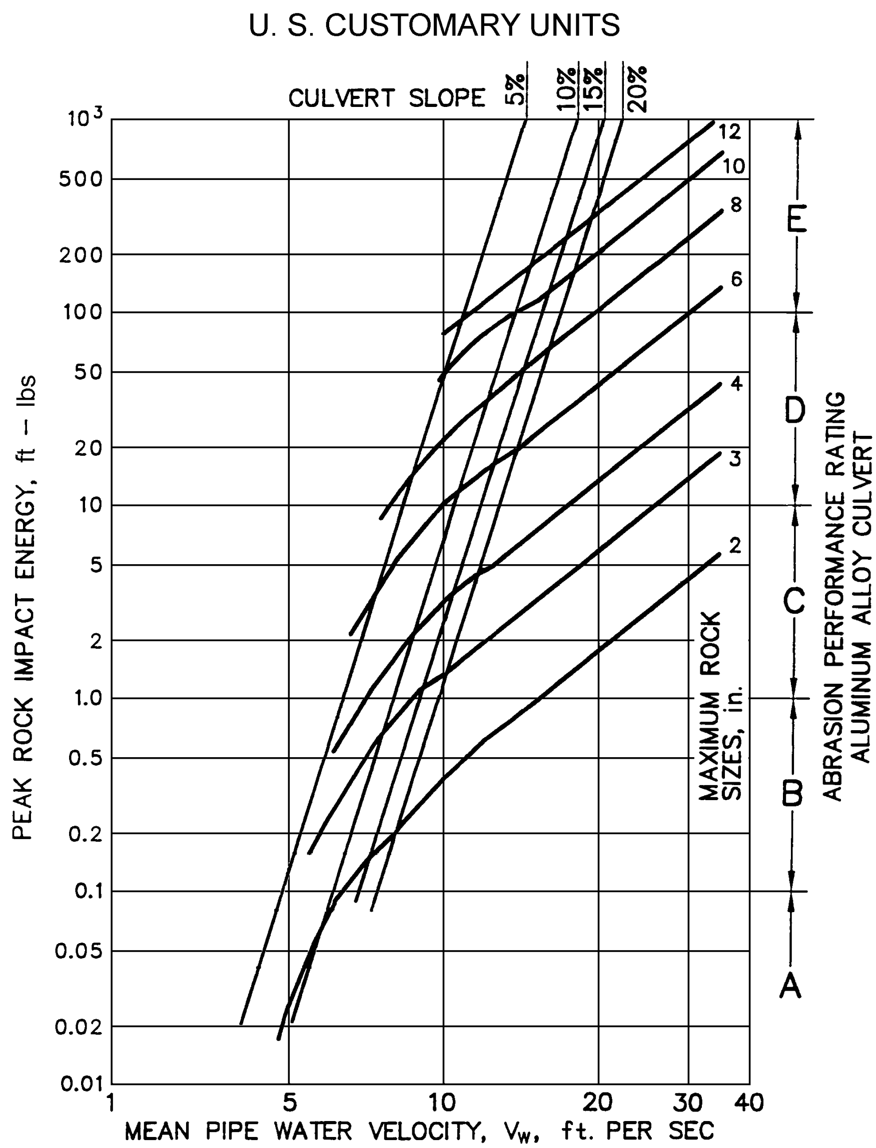


Figure 12.6.9.5P-6 – Abrasion Performance Rating for Aluminum Culvert (modified after Koepf and Ryan, 1986) (Continued)

Table 12.6.9.5P-2 – Assumed Mean Culvert Water Velocity for Abrasion Performance Rating for Circular Aluminum Culvert. Manning n-value is 0.24 (modified after Koepf and Ryan, 1986)

Metric Units							
		Culvert Slope in %					
Size (mm)	Q (m^3/s)	5	10	15	20	25	30
457	0.06	1.68	2.01	2.32	2.53	2.77	3.05
610	0.12	1.89	2.44	2.83	3.23	3.51	3.66
762	0.23	2.19	2.87	3.26	3.66	3.90	4.27
914	0.37	2.44	3.14	3.66	4.11	4.33	4.60
1070	0.57	3.05	3.66	4.27	4.60	5.03	5.43
1220	0.82	3.08	4.11	4.57	5.03	5.49	5.94
1520	1.47	3.57	4.72	5.33	6.10	6.61	7.01
1830	2.35	4.02	5.18	6.10	6.49	7.32	7.77
2130	3.68	4.51	5.79	6.55	7.32	7.92	8.44
2440	5.27	4.88	6.40	7.32	8.17	8.84	9.45
		Table gives mean culvert velocity in mps					

U.S. Customary Units							
		Culvert Slope in %					
Dia. (in.)	Q (cfs)	5	10	15	20	25	30
18	2.0	5.5	6.6	7.6	8.3	9.1	10.0
24	4.4	6.2	8.0	9.3	10.6	11.5	12.0
30	8.0	7.2	9.4	10.7	12.0	12.8	14.0
36	13.0	8.0	10.3	12.0	13.5	14.2	15.1
42	20.0	10.0	12.0	14.0	15.1	16.5	17.8
48	29.0	10.1	13.5	15.0	16.5	18.0	19.5
60	52.0	11.7	15.5	17.5	20.0	21.7	23.0
72	83.0	13.2	17.0	20.0	21.3	24.0	25.5
84	130	14.8	19.0	21.5	24.0	26.0	27.7
96	186	16.0	21.0	24.0	26.8	29.0	31.0
		Table gives mean culvert velocity in fps					

SPECIFICATIONS COMMENTARY

12.6.10P Life-Cycle Cost

The life-cycle cost of a structure is a method used to compare various alternative buried structures to determine the comparative cost of each material and the method of installation. Life-cycle cost comparison methods involve the principles of engineering economics to determine the "present worth" of a structure, with consideration given to initial cost, maintenance costs, replacement costs, and the residual value of the structure at the end of the project design life. This method of analysis utilizes the parameters service life, interest rate and inflation rate to determine the most economical structure.

12.6.10.1P SERVICE LIFE ESTIMATION

Service life is defined as the number of years of good performance with minor maintenance. A structure may have reached its service life, but still be years from ultimate failure. Determination of the service life of a structure is dependent on durability, which is a function of the intended use of the structure; the relationship of the structure to its environment; the geometric constraints of the installation; and the site and environmental conditions, as discussed in D12.6.9.

The estimation of service life should include allowances for progressive changes in the culvert materials due to corrosion, abrasion and age, and in methods of installation, inspection and maintenance. Use of corrugated steel or aluminum culverts shall be restricted to secondary roads. For secondary road facilities, the minimum service life shall be 50 years. For primary highway facilities or for secondary road facilities where replacement access is limited (e.g., below high embankment fills), the minimum service life shall be 100 years. Pipe selection criteria are provided in Design Manual, Part 2.

Methods used to estimate the service life include the following:

(a) Field performance surveys

(b) Field prototype tests

(c) Laboratory test methods

(d) Analytical methods

12.6.10.1.1P Corrugated Metal

The design procedure for determining the gage or thickness of corrugated steel, steel structural plate and aluminum buried structures is based on the estimated metal loss field performance studies, pH, resistivity, and service life. Determination of the structural and hydraulic requirements for structure installation will result in a metal thickness necessary to support local loading conditions. The

SPECIFICATIONS

durability design requirements will utilize the complete and uniform predicted metal loss determined from Figures D12.6.9.5P-1 and D12.6.9.5P-2 for corrugated steel culverts and structural steel plate culverts, respectively, which correlates the required culvert age with the pH and abrasive nature of the water it will convey. This corresponds to the metal loss along the length of the culvert invert or flow line. The minimum predicted metal loss will be 0.050 mm/year {2 mil/year} as shown in Figures D12.6.9.5P-1 and D12.6.9.5P-2. The minimum predicted metal loss for aluminum and aluminized culverts is 0.025 mm/year {1 mil/year}. Once the minimum required gage of the metal structure is determined with respect to durability, the metal thickness is added to the structural design metal thickness requirements to determine the minimum design thickness.

Adjustment of the predicted metal loss due to abrasion has been incorporated into Figures D12.6.9.5P-1 and D12.6.9.5P-2. A similar relationship has yet to be developed for aluminum and aluminized culverts; however, the minimum predicted metal loss rate of 0.025 mm/year {1 mil/year} should be used for the adjustment.

12.6.10.1.2P Reinforced Concrete

A correlation of durability factors to service life of reinforced concrete culverts is presented in Figure D12.6.9.5P-5. Adjustment of the service life to account for abrasion in aluminum culverts is presented in Figure D12.6.9.5P-6. Service life (years-to-poor for culverts with sediment) is equal to the service life (years-to-poor, no sediment) multiplied by a sediment factor. Years-to-poor, as defined by the field performance study, occurs when only 50% of the invert thickness remains.

12.6.10.2P PRESENT WORTH ANALYSIS

Present worth analysis is a method for adjusting all annual and future costs to a present cost, to permit economic comparison between alternatives. When two or more alternatives are capable of performing the same functions, the more cost-effective alternative will have the least present worth. The present worth method of analysis is restricted to alternatives that perform the same function.

The present worth life-cycle cost for culvert installations shall be determined in accordance with the following relationship (TRB, 1985)

$$LCC = PC + MC\left(\frac{1+I}{1+i}\right)^n - SV\left[\left(\frac{n_p - n}{n_p}\right)\left(\frac{1+I}{1+i}\right)^n\right] \quad (12.6.10.2P\text{-}1)$$

where PC is used to compute the initial cost of the culvert, including the cost for culvert materials, backfill and labor costs, and MC $[(1 + I)/(1 + i)]^n$ is used to compute the annual cost for maintenance over the anticipated service life

COMMENTARY

C12.6.10.2P

Because of the range of costs associated with the various alternative methods that can be used to satisfy a particular design, life-cycle cost comparison can be made to determine the alternative which is most economically feasible. The following example may be helpful in understanding the application of life-cycle cost analysis.

Three alternatives are available for improvement of an existing culvert which crosses below a rural route in central Pennsylvania. Alternatives A and B involve replacement of the existing structure. Both alternatives will require maintenance at some time during the service life of the buried structure. Alternative C has an annual maintenance cost for continued use of the existing structure and, therefore, no salvage value. Determine the most economical alternative.

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of the culvert. If a uniform annual maintenance cost is required over the service life of the culvert, the latter term shall be revised as follows:

$$MC\left[\frac{\left(\frac{1+I}{1+i}\right)\left[1-\left(\frac{1+I}{1+i}\right)^{n}\right]}{1-\left(\frac{1+I}{1+i}\right)}\right] \qquad (12.6.10.2P\text{-}2)$$

The term SV $\{[(n_p - n)/n_p]\,[(1 + I)/(1 + i)]^n\}$ is used to compute the value remaining in the culvert at the end of the anticipated service life of the culvert. The removal cost shall be included in the salvage value term; therefore, the value of this last term could be negative. If the material life is equal to the project design life, the present worth of each alternative is based on a comparison of present or initial costs. When the material life is less than the project design life, alternative comparisons are based on initial costs and replacement or maintenance costs. Similarly, if the material life exceeds the design life, the present worth of the structure is a function of the initial costs with an adjustment for the future value of the structure. Note that PC, MC, and SV values must be in terms of the present worth of that cost.

For life-cycle cost comparisons, the projected inflation and interest rates are difficult to estimate. For preliminary analysis purposes, an average ratio of the inflation to the interest factor (i.e., $[(1 + I)/(1 + i)]$) of 0.9853 may be used, based on a review of Federal treasury bonds from 1953 to 1983.

COMMENTARY

	Alternatives		
	A	B	C
Present Material and Installation Cost (PC)	\$22,000	\$16,000	\$0
Material Design Life (n_p)	100 years	75 years	50 years
Maintenance Cost (MC) Year 30 Year 40 Annual	 \$0 \$3,000 \$0	 \$10,000 \$0 \$0	 \$0 \$0 \$600/year
Salvage Value (SV)	\$7,000	\$3,000	\$0
Service Life (n)	50 years	50 years	50 years
$\frac{\text{Inflation}}{\text{InterestFactor}}\left(\frac{1+I}{1+i}\right)$	0.9853	0.9853	0.9853

$$LCC = PC + MC\left(\frac{1+I}{1+i}\right)^{n} - SV\left[\left(\frac{n_p - n}{n_p}\right)\left(\frac{1+I}{1+i}\right)^{n}\right]$$

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COMMENTARY

Alternative A

(a) Step 1

Convert maintenance cost in Year 40 to present worth cost.

$\$3,000\ (0.9853)^{40} = \$1,659.05$

(b) Step 2

Convert salvage value in Year 50 to present worth cost. (Note that the salvage value includes the structure removal cost.)

$$\$7,000\left(\frac{100-50}{100}\right)(0.9853)^{50} = \$1,669.14$$

LCC=$22,000+$1,659.05-$1,669.14=$21,989.91

Alternative B

(a) Step 1

Convert maintenance cost in Year 30 to present worth cost.

$\$10.000(0.9853)^{30} = \$6,412.90$

(b) Step 2

Convert salvage value in Year 50 to present worth cost. (Note that the salvage value includes the structure removal cost.)

$$\$3,000\left(\frac{75-50}{75}\right)(0.9853)^{50} = \$476.90$$

LCC= $16,000 + $6,412.90 - $476.90 =$21,936.00

Alternative C

(a) Step 1

Convert uniform annual cost to present worth cost.

$$\$600\left(\frac{0.9853\left[1-(0.9853)^{50}\right]}{1-0.9853}\right) = \$21,037.30$$

(b) Step 2

LCC=$0 + $21,037.30 + $0 = $21,037.30

SPECIFICATIONS COMMENTARY

From the above analysis, Alternative C is the most economical alternate since it requires the least LCC.

12.6.10.3P OTHER FACTORS

Various alternates, such as culvert coatings, as discussed in D12.6.9.4P, may be considered to enhance culvert durability. Lining a culvert may also be more effective and cost-efficient than replacement and shall be considered part of the LCC analysis.

12.7 METAL PIPE, PIPE ARCH AND ARCH STRUCTURES

12.7.2 Safety Against Structural Failure

12.7.2.7P STANDARD DESIGNS

Use Standard Drawing BD-635M entitled "Design Tables for Metal Culverts", and note that it includes consideration for corrosion protection.

Value engineering or alternate design by the contractor should not be permitted, unless it is approved by the District Executive in consultation with the District Bridge Engineer.

It is not required to prepare structure drawings for pipes having an inside span length of less than 2400 mm {8 ft.}. Thus, the designs under 2400 mm {8 ft.} in span are roadway items and shall be included in the construction plan.

12.7.4 Stiffening Elements for Structural Plate Structures

The following shall supplement A12.7.4.

The maximum allowable spacing for circumferential stiffening elements shall be 1370 mm {54 in.}.

12.7.5 Construction and Installation

The following shall replace A12.7.5.

The contract documents shall require that construction and installation conform to Publication 408 and Standard Drawing RC-30M.

C12.7.5P

Deflections of metal box culverts are dependent on many factors, including backfill characteristics, compaction, pavement type, vehicle weight and number of load cycles. The use of higher quality backfill, improved backfill compaction and stiff pavements will result in smaller culvert deflections. Live load deflections increase in proportion to increases in the axle load. Deflections will be larger under first loading. After many load applications, deflections are likely to be about one-quarter of those under the first loading of the same load.

12.8 LONG-SPAN STRUCTURAL PLATE STRUCTURES

12.8.2 Service Limit State

The following shall replace A12.8.2.

SPECIFICATIONS

The provisions of D12.6.2.1 shall apply.

12.8.3 Safety Against Structural Failure

12.8.3.1 SECTION PROPERTIES

12.8.3.5 ACCEPTABLE SPECIAL FEATURES

12.8.3.5.2 Reinforcing Ribs

The following shall supplement A12.8.3.5.2.

When required to satisfy the structural design, reinforcing ribs shall be attached to the structural plate corrugation crown at a bolt spacing of not more than 300 mm {12 in.}.

When required only to control structure shape during installation, reinforcing ribs shall be spaced and attached to the corrugated plates as required by the manufacturer and with the approval of the Chief Bridge Engineer.

12.8.8 Construction and Installation

The following shall supplement A12.8.8.

If approved by the Chief Bridge Engineer, the contract documents shall require that construction and installation of long-span structural plate structures conform to the requirements of Publication 408 and Standard Drawing RC-30M.

12.9 STRUCTURAL PLATE BOX STRUCTURES

12.9.1 General

The following shall supplement A12.9.1.

Metal box culverts shall be designed with cement concrete or steel spread footings, or a full steel invert.

12.9.2 Loading

The following shall replace A12.9.2.

For live loads, the provisions of A3.6.1 and D3.6.1 shall apply.

12.9.3 Service Limit State

The following shall supplement A12.9.4.2.

The provisions of D12.6.2.1 shall apply.

COMMENTARY

C12.8.3.1.1 Cross-Section

The following shall supplement AC12.8.3.1.1.

Because no rational design procedure exists for the structural design of long-span structures with acceptable special features in Table A1, their design is based on the successful performance of completed structures. AASHTO and ASTM technical committees are currently developing recommended design procedures for these structures.

C12.9.2

Delete AC12.9.2.

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COMMENTARY

12.9.4 Safety Against Structural Failure

12.9.4.2 MOMENTS DUE TO FACTORED LOADS

The following shall supplement A12.9.4.2.

The standard dead and live load conditions shall be taken as:

Dead Load: Density of soil backfill,
γ_s = 2240 kg/m^3 {0.140 kcf}

Live Load: 180 kN {40 kip} four-wheel single axle (HS25 loading)
A_L = 180 000 N {40 kip}

C12.9.4.2

The following shall supplement AC12.9.4.2.

The designs based on this provision are controlled by the crown and haunch moments. The critical live load position for both crown and haunch moments is at or near the center of the span.

C12.9.4.3 PLASTIC MOMENT RESISTANCE

The following shall supplement AC12.9.4.3.

Equations A12.9.4.2-1 and A12.9.4.2-2 provide the unfactored dead and live load moments for design. These moments are distributed between the crown and haunch of a box culvert. Theoretically, this distribution could be as unbalanced as 0 to the haunch and 100% to the crown; however, this provision limits distribution to the crown to between 45% and 70%. Analyses, tests and applications have validated the range of distributions prescribed in Table A12.9.4.3-1. Hence, plastic moment capacities of crown and haunch shall comply with the ratios of distribution in Table A12.9.4.3-1.

To account for longitudinal spreading of live loads and resulting reduction in maximum haunch moment. The term R_h is used in equation A12.9.4.3-2.

The specification is structured to permit the user to make adjustments in many of the parameters. However, it is possible to generate a simplified design table for standard conditions. Maximum, median and minimum moments, proportioned in accordance with values found in Table A12.9.4.3-1, are shown for various span ranges and cover depths. The haunch moment values have been adjusted by the appropriate values for R_h found in Table A12.9.4.3-2.

12.9.4.6 CONCRETE RELIEVING SLABS

The following shall supplement A12.9.4.6.

If a cement concrete relieving slab is used immediately above an aluminum culvert, the aluminum shall be physically separated by an inert material to preclude chemical reaction with the concrete.

Replace the second paragraph of A12.9.4.6 with the following.

The length of the cement concrete relieving slab shall be at least 3000 mm {10 ft.} greater than the culvert span and shall project 1500 mm {5 ft.} beyond the haunch on each side of the culvert. Slab projections in excess of

C12.9.4.6

The following shall supplement AC12.9.4.6.

It is not desirable to locate the concrete relieving slab at top of the roadway as pavement. This type of relieving slab could be inadvertently removed in the future by the maintenance force or under pavement rehabilitation contracts without realizing the structural importance.

The slab adjustment factor, R_f, is based on the results of finite elements analyses comparing stresses in slabs on soil without underlying culverts.

SPECIFICATIONS

300 mm {1 ft.} will result in further reduction in culvert moments due to live load. The relieving slab shall extend across the width subject to traffic loading.

The following shall supplement the third paragraph of A12.9.4.6.

Relieving slab thickness shall be determined using R_{AL} = 1.05 for HS25 loading, R_C = 1.19 for Class A Cement Concrete, and t_b = 225 mm {9 in.}. Construction documents shall ensure that granular material is specified under the concrete slab and compacted as per Publication 408, Section 206.3(b)1. Construction documents shall also specify a minimum slab thickness of 330 mm {13 in.}, minimum reinforcement of No. 13 bars at 300 mm {No. 4 bars at 12 in.} in both directions, top and bottom, and use of epoxy-coated bars if the slab is under less than 600 mm {2 ft.} of fill.

For construction loading, the relieving slab thickness shall be checked for a 225 kN {50 kip} axle load for self-propelled permit load construction equipment (R_{AL} = 1.15).

12.9.5 Construction and Installation

The following shall replace A12.9.5.

The contract documents shall require that construction and installation conform to Publication 408.

12.10 REINFORCED CONCRETE PIPE

The following shall replace A12.10.

Refer to Appendix H "Pennsylvania Installation Direct Design (PAIDD) for Concrete Pipes" and Standard Drawing BD-636M for structural design criteria and to Publication 280M for manufacturing specifications.

COMMENTARY

C12.10

The following shall replace AC12.10

Refer to Appendix H.

12.11 REINFORCED CONCRETE CAST-IN-PLACE AND PRECAST BOX CULVERTS AND REINFORCED CAST-IN-PLACE ARCHES

12.11.1 General

The following shall supplement A12.11.1.

Box structures shall be designed using the computer program BXLRFD, entitled LRFD Box Culvert Design and Rating.

The haunch shall be dimensioned to satisfy design, transportation and construction requirements. The haunch dimensions for precast box culverts shall not be less than 150 mm x 150 mm {6 in. x 6 in.}.

C12.11.1

The following shall supplement AC12.11.1.

The designs for precast box culverts in AASHTO M 259M and M 273M are for an HS20 live load and a maximum span length of 3650 mm {12 ft.}. For other live load conditions, the designs should be modified.

The documentation for BXLRFD includes narrative on when the haunch is considered or neglected when using that program.

12.11.2 Loads and Live Load Distribution

12.11.2.1 GENERAL

The following shall replace A12.11.2.1.

Loads and load combinations specified in Table 3.4.1-1 shall apply. Live load shall be as specified in Article 3.6.1.3.

C12.11.2.1

The following shall supplement AC12.11.2.1.

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Distribution of wheel loads and concentrated loads for culverts with less than within 600 mm {24 in.} of fill shall be taken as specified in D4.6.2.12P. For traffic traveling parallel to the span, box culverts shall be designed for a single loaded lane with the single lane multiple presence factor applied to the load. Requirements for bottom distribution reinforcement in top slabs of such culverts shall be as specified in A9.7.3.2 for mild steel reinforcement and A5.14.4.1 for prestressed reinforcement.

Edge beams shall be provided as specified in A4.6.2.1.4 as follows:

- At ends of culvert runs where wheel loads travel within 600 mm {24 in.} from the end of culvert.
- At expansion joints of cast-in-place culverts where wheel loads travel over or adjacent to the expansion joint.

Distribution of wheel loads to culverts with 600 mm {2 ft.} or more of cover shall be as specified in D3.6.1.2.6.

The dynamic load allowance for buried structures shall conform to A3.6.2.2.

Reinforced concrete box culverts at grade shall be designed for an additional dead load of 1.4×10^{-3} MPa {0.030 ksf} of slab to account for the placement of future wearing surfaces. The thickness of the top slab shall include a 20 mm {1/2 in.} integral wearing surface which shall not be considered in the design of the effective depth of the slab. This additional thickness shall be neglected for slabs incorporating a bituminous wearing surface.

12.11.2.2 MODIFICATION OF EARTH LOADS FOR SOIL-STRUCTURE INTERACTION

The following shall replace A12.11.2.2.

The total earth load, W_E, on the box section shall be determined as specified in Equation A12.10.2.1-1. The value of B_c shall be taken as specified in Figure 1.

For embankment or trench installations, the soil-structure interaction factor, F_e, shall be determined using Equation 1.

$$F_e = 1 + 0.20\ H/B_c \qquad (12.11.2.2\text{-}1)$$

where:

H = height of fill over pipe or culvert.

F_e shall not exceed 1.15 for installations with compacted fill along the sides of the box section, or 1.40 for installations with uncompacted fill along the sides of the box section.

COMMENTARY

This equivalent strip width policy results in force effects that are generally consistent with the effects computed using the AASHTO Standards Specifications where the strip widths are based on wheel loads rather than axle loads. This policy relies on the interlocking mechanism of the precast units to distribute wheel loads across culvert joints when the equivalent strip width exceeds the segment length.

Research into live load distribution on box culverts (McGrath et al., 2004) has shown that design for a single loaded lane with a multiple presence factor of 1.2 on the live load and using the live load distribution widths in D4.6.2.12P will provide adequate design loading for multiple loaded lanes with multiple presence factors of 1.0 or less when the traffic direction is parallel to the span.

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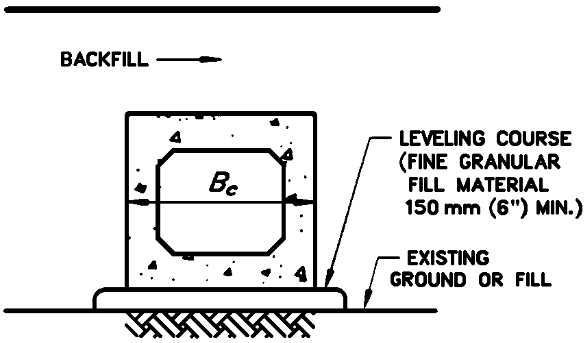

Figure 12.11.2.2-1 - Design Nomenclature for Embankment or Trench Condition Precast Concrete Box Sections

C12.11.2.3 DISTRIBUTION OF CONCENTRATED LOADS TO BOTTOM SLAB OF BOX CULVERT

The following shall supplement AC12.11.2.3.

Restricting the live load distribution width for the bottom slab to the same width used for the top slab provides designs suitable for multiple loaded lanes, even though analysis is only completed for a single loaded lane (As discussed in DC12.11.2.1).

While typical designs assume a uniform pressure distribution across the bottom slab, a refined analysis that considers the actual soil stiffness under box sections will result in pressure distributions that reduce bottom slab shear and moment forces (McGrath et al., 2004). Such an analysis requires knowledge of in-situ soil properties to select the appropriate stiffness for the supporting soil. A refined analysis taking this into account may be beneficial when analyzing existing culverts.

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Figure 12.11.2.2-1 - Design Nomenclature for Embankment or Trench Condition Precast Concrete Box Sections

12.11.2.5P ENDS OF BOX SECTION

The ends of box sections shall be normal to the walls and centerline of the box section. When a beveled end is specified, an edge beam design shall be required.

12.11.4 Safety Against Structural Failure

12.11.4.1 GENERAL

The following shall supplement A12.11.4.1.

If feasible and economical, trussed bars may be used in the design of reinforced concrete box culvert slabs when required to resist diagonal tension. The use of trussed bars shall be approved by the District Bridge Engineer as part of TS&L approval. Stirrups shall not be used.

12.11.4.1.1P Cast-in-Place Arches

Arches of clear normal span up to 4600 mm {15 ft.} may be designed with semi-circular or segmental intrados. Larger arches shall be designed with multi-centered intrados derived from an axis conforming as nearly as possible to the equilibrium polygon for full dead load or for full dead load and one-half live load over full span, including lateral earth pressures, to reduce bending moments to a minimum under critical loading conditions.

Culverts on yielding foundation, suitable for either open or tied arch, shall be investigated to determine the most economical type.

12.11.4.4 MINIMUM COVER FOR PRECAST BOX STRUCTURES

The following shall replace the last sentence of A12.11.4.4.

If the height of fill is greater than 600 mm {2 ft.}, the minimum cover shall be 40 mm {1 1/2 in.} and in accordance with BD-632M.

12.11.4.5P CONSTRUCTION AND EXPANSION JOINTS

Construction joints shall be provided at approximate 9000 mm {30 ft.} intervals. Expansion joints (in tied arch and box sections, including the bottom slab and in the ring of open arches) shall be provided at approximate 27 000 mm {90 ft.} intervals, except as noted subsequently. All joints shall be normal to centerline of culvert.

When appreciable settlement of foundation material is anticipated, and the use of cambered grade for the flow line of a tied arch or box culvert is considered, only construction joints shall be used throughout the entire length of the

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C12.11.2.5P

The short side of an end segment should be at least 600 mm {2 ft.} in length. The designer must review the detail to ensure proper structural function of the skewed ends.

C12.11.4.1P

When trussed bars are used, the reinforcement and concrete should be placed in a manner that provides for adequate cover and bonding of the reinforcement.

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culvert. Additional longitudinal reinforcing steel (reinforcing steel installed parallel to the centerline of culvert) shall be provided, as required.

For cast-in-place arches, construction joints between the arch ring and footing or the tie slab shall be shown on the drawings.

For precast box culverts, the joints between the box sections shall be sealed with waterproofing sealer. Waterproofing materials shall be in accordance with Publication 408, Section 680, and shall be placed at the top and the outside of the box at every joint as per Publication 408, Section 680.

12.11.4.6P REINFORCEMENT DETAILS

For tied arches bars on outside surface of arch shall be bent to the design radius and shall be anchored sufficiently into the bottom of the slab. Main circumferential reinforcement bars for arches shall be placed normal to the centerline of the arch. In skewed end panels, main reinforcement bars shall be cut to fit and shall be anchored properly into the end wall. If appropriate, end panels of skewed arches may be constructed with square ends. The required area of longitudinal reinforcement steel (parallel to the centerline of the culvert) shall be the same as for all shrinkage and temperature reinforcement as specified in A5.10.8.2. It shall not, however, be less than No. 10 bars at 300 mm {No. 4 bars at 12 in.} each face. Minimum W4 (5.74 mm {0.226 in.} diameter) annealed iron wire ties shall be used to prevent buckling of longitudinal bars due to axial thrust in the arch ring. Ties shall be staggered by placement at alternate intersections of longitudinal and transverse bars.

Main circumferential steel reinforcement in cast-in-place box culverts shall be placed normal to the centerline of the culvert. In skewed end panels, main circumferential reinforcement shall be cut to fit and shall be anchored properly into headwalls or edge beams at the top and bottom. If practical, end panels of skewed box culverts may be built with square ends. The required area of longitudinal steel reinforcement (parallel to box culvert centerline) shall be the same as shrinkage and temperature reinforcement specified in A5.10.8.2, except for the top slab of boxes with 600 mm {2 ft.} or less cover where A9.7.3.2 controls the required amount of distribution reinforcement. For main reinforcement perpendicular to traffic, the specified amount of distribution reinforcement shall be used in the middle half of the slab span, and not less than 50% of the specified amount shall be used in the outer quarters of the slab span.

Welded deformed wire fabric for precast box culverts shall be furnished in flat sheets. The center-to-center spacing of main circumferential wire shall not be less than 50 mm {2 in.} and not more than 100 mm {4 in.} (75 mm {3 in.} is preferred).

12.11.4.7P MINIMUM THICKNESS

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Refer to Standard Drawing BD-633M for minimum thickness requirements for cast-in-place arches and Standard Drawing BD-632M for minimum thickness requirements for cast-in-place and precast box culverts.

12.11.4.8P MINIMUM CONCRETE COVER

Refer to D5.12.3 for minimum concrete cover for steel reinforcement in cast-in-place and precast concrete box culverts.

12.11.4.9P MULTI-CELL INSTALLATIONS

For multi-cell installations of precast box culverts, positive lateral bearing by continuous contact between the sides of adjacent boxes shall be provided by means such as compacted soil fill, granular backfill, grouting or concreting. The method selected shall be shown on the contract drawings.

Standard details for the joint between cells of multi-cell installations shall be in accordance with BD-632M.

12.11.5 Construction and Installation

The following shall replace A12.11.5.

The contract documents shall require that construction and installation conform to the requirements of Publication 408M, Special Provisions and Standard Drawings RC-11M and RC-12M.

To account for irregularities in culvert excavation and scour, the thickness of the bottom slab shall be increased in accordance with Standard Drawing BD-632M.

12.12 THERMOPLASTIC PIPES

12.12.1 General

The following shall supplement A12.12.1.

Thermoplastic pipe properties vary with temperature and duration of loading. This aspect shall be considered in the selection of material properties for design.

12.12.2 Service Limit States

The following shall supplement A12.12.2.

The provisions of D12.6.2.1 shall apply.

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12.12.2.1P DEFLECTIONS

The total horizontal deflection of thermoplastic pipe due to earth and live loads shall be estimated as:

$$\Delta x = \frac{K\left(D_L W_E + W_L\right)}{0.149PS + 0.061M_S} \quad (12.12.2.1P\text{-}1)$$

where:

Δx = total horizontal deflection (mm) {in.}

D_L = deflection lag factor (dim)

K = bedding constant (dim)

W_E = vertical earth load per unit length of pipe (N/mm) {kip/in}

W_L = vertical live load per unit length of pipe (N/mm) {kip/in}

PS = pipe stiffness (N/mm/mm) {kip/in/in}

M_s = modulus of soil reaction (MPa) {ksi}

For thermoplastic pipes installed in accordance with Standard Drawing RC-30M, the value of M_s shall be taken as 10 MPa {1.5 ksi}.

$$PS = \frac{EI}{0.019S^3} \quad (12.12.2.1P\text{-}2)$$

where:

E = long-term modulus of elasticity of pipe material at 50 years from Table A12.12.3.3-1 for earth loading and initial modulus of elasticity of pipe material from Table A12.12.3.3-1 for live loading (MPa) {ksi}

I = minimum moment of inertia of pipe (mm^4/mm) {in^4/in}

S = mean pipe diameter (mm) {in.}

The vertical deflection shall be estimated as

$$\Delta y = \Delta x \left(\text{ATV}\right) \quad (12.12.2.1P\text{-}3)$$

where:

ATV = B. J. Schrock's Correction Factor from Figure 1

COMMENTARY

C12.12.2.1P

The effects of live load are not included in Equation 1. If the effects of live load are significant (such as for shallow soil cover less than 1500 mm {5 ft.}), deflection estimates should include the effects of live load on the pipe, such as described in ASTM D 3839 using the initial modulus from Table A12.12.3.3-1 for the live load deflection component.

After soil has been initially loaded, it continues to deform (consolidate) with time. The deflection lag factor converts the immediate deflection of the pipe to the deflection of the pipe after many years. Limited investigations concerning this factor have been carried out, but it appears to vary with the type of soil used for the primary zone backfill material and its degree of compaction, native soil characteristic and trench width. For shallow burial depths, a value up to 2.0 appears appropriate when moderate or higher degrees of compaction are attained. With the greater degrees of compaction, the initial pipe deflection is usually very small, and even a slight increase in deflection over a period of time may result in a long-term deflection which can be twice the initial deflection.

The bedding constant reflects the degree of support, provided by the soil at the bottom of the pipe and over which the bottom reaction is distributed. Table C1 gives the recommended values for different types of installation.

Table C12.12.2.1P-1 – Values for Bedding Constant

Type of Installation	Value of K
Shaped bottom with tamped primary pipe zone backfill, 95% Proctor density or greater	0.083
Compacted coarse-grained bedding and primary pipe zone backfill, 70 to 100% relative density	0.083
Shaped bottom, moderately compacted primary pipe zone backfill, 85 to 95% Proctor density	0.103

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The vertical soil load on the pipe may be considered as the weight of the rectangular prism of soil directly above the pipe. The soil prism would have a height equal to the depth of earth cover, a width equal to the pipe outside diameter, and a unit length of 1 mm {1 in.}. In the absence of specific soil information, the unit weight of soil may be assumed to be 2250 kg/m^3 {0.140 kcf}.

The values in Table C2 will result in a calculated average deflection so there will be a 50% probability that the actual average deflection will be less than the calculated value. To obtain an estimated average deflection so there will be a 95% probability that the actual average deflection will be less than the calculated value, two procedures are available:

- For H>4800 mm {16 ft.} – Use an M_S value equal to 0.75 times the value obtained from Table C2.
- For H $\leq$ 4800 mm {16 ft.} - Use the M_s value from Table C2 and add the percentage deflections shown as follows to the deflection value so calculated:

  Degree of Compaction

Moderate	High
+1%	+0.5%

  For pipe installed with a high degree of compaction (95% Proctor, 70% relative density), there is a maximum of 0.5% deflection difference in the average deflections predicted by either procedure.

SPECIFICATIONS COMMENTARY

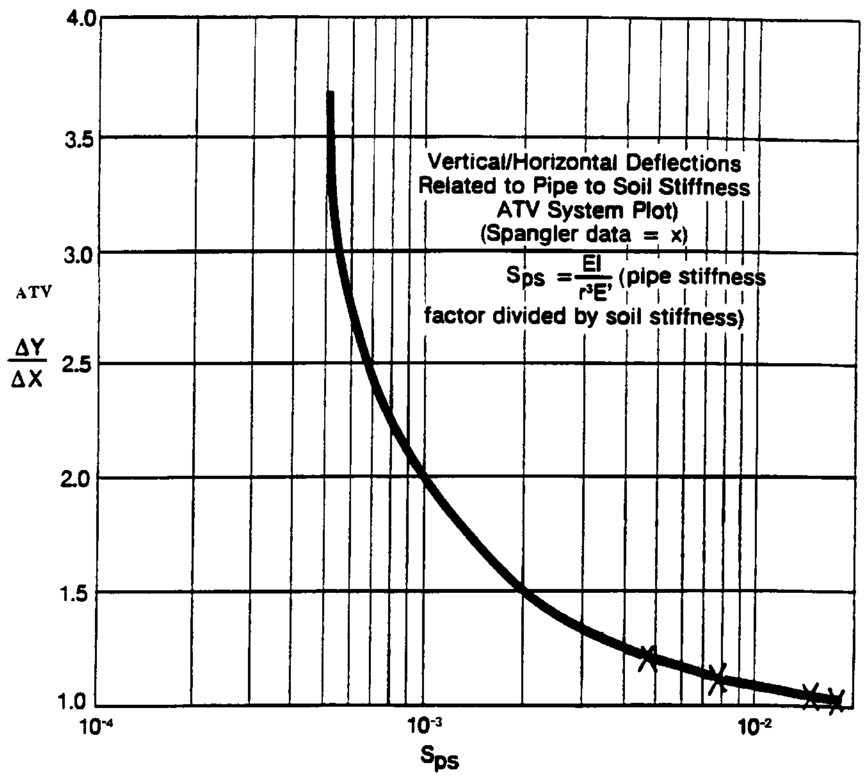

Figure 12.12.2.1P-1 - B. J. Schrock's Correction Factors

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Table C12.12.2.1P-2 - Values of Soil Modulus

Soil Type - Primary Pipe Zone Backfill Material	M_s for Degree of Compaction of Pipe-Zone Material	
	Moderate: 85 to 95% Proctor, 40 to 70% Relative Density	High > 95% Proctor, > 70% Relative Density
Fine-grained soils (LL < 50)*: Soil with medium to no plasticity: CL, ML with less than 25% coarse-grained particles	2.8 MPa {0.4 ksi}	6.9 MPa {1.0 ksi}
Fine-grained soils (LL < 50)*: Soils with medium to no plasticity: CL, ML with more than 25% coarse-grained particles	6.9 MPa {1.0 ksi}	13.8 MPa {2.0 ksi}
Coarse-grained soils with fines: GM, GC, SM, SC containing more than 12% fines	6.9 MPa {1.0 ksi}	13.8 MPa {2.0 ksi}
Coarse-grained soils with little or no fines: GW, GP, SW, SP containing less than 12% fines	13.8 MPa {2.0 ksi}	20.7 MPa {3.0 ksi}
Crushed rock	20.7 MPa {3.0 ksi}	20.7 MPa {3.0 ksi}

*LL - Liquid Limit

12.12.3 Safety Against Structural Failure

12.12.3.2 SECTION PROPERTIES

The following shall supplement A12.12.3.2.

The section properties presented in Appendix A12, Tables A12-11 through A12-13, shall be verified by testing in an independent laboratory, certified by the manufacturer and approved by the Department.

12.12.3.3 CHEMICAL AND MECHANICAL REQUIREMENTS

The following shall supplement A12.12.3.3.

Use actual long-term (50-year) values of tensile strength (f_u) and Modulus of Elasticity (E) tested by an independent laboratory, certified by the manufacturer, approved by the Department and meeting minimum requirements as shown.

12.12.3.5 WALL RESISTANCE

The following shall supplement A12.12.3.5.

The value of F_u in Equation A12.12.3.5.1-1 shall be the specified minimum tensile strength at 50 years from Table A12.12.3.3-1.

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12.12.3.5.2 BUCKLING

The following shall supplement A12.12.3.5.2.

For side fills conforming to A12.6.6.3, the value of M_s in Equation A12.12.3.5.2-1 shall be based on Sn-95 of Table A12.12.3.4-1.

12.12.4P Submittals

Design submittal shall include detailed section properties with adequate references to standards and tabulated design calculations.

12.13 STEEL TUNNEL LINER PLATE

12.13.1 General

Delete the second sentence of the first paragraph of A12.13.1.

12.14 PRECAST REINFORCED CONCRETE THREE SIDED STRUCTURES

Delete the provisions of A12.14.

12.14.5.2 DISTRIBUTION OF CONCENTRATED LOAD EFFECTS IN SIDES

Revise entire section, including the title of the article as follows.

12.14.5.2 DISTRIBUTION OF CONCENTRATED LOAD EFFECTS IN TOP SLAB AND SIDES

Distribution of wheel loads and concentrated loads for the top slab and sides of three-sided structures with less than 600mm {2 ft.} of fill shall taken as specified in D12.11.2.1.

Distribution of wheel loads and concentrated loads for the top slab and sides for three sided structures with depths of fill 600mm {2 ft.} or greater shall be taken as specified in D3.6.1.2.6.

COMMENTARY

C12.14.5.2

Delete commentary.

REFERENCES

American Association of State Highway and Transportation Officials (AASHTO), Standard Specifications for Transportation Materials and Methods of Sampling and Testing, Part I, Specification, 15th ed., Washington, DC, 1990 (see AASHTO Interims for more recent updates)

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Transportation Research Board (TRB), "Durability of Drainage Pipe", NCHRP Synthesis No. 50, Washington, DC, 1978

Transportation Research Board (TRB), Life-Cycle Cost Analysis of Pavements, NCHRP Synthesis No. 122, Washington, DC, 1985

APPENDIX

The following shall replace Tables A12-2, A12-3, A12-5 and A12-8 of the AASHTO Appendix to Section 12.

Table A12-2 - Spiral Rib Steel Pipe - Cross-Section Properties

Metric Units			
19 x 19 x 190 mm CORRUGATION			
Thickness (mm)	A (mm^2/mm)	R (mm)	I (mm^4/mm)
1.63	1.08	6.55	46.2
2.01	1.51	6.35	60.6
2.77	2.51	6.02	90.7
3.51	3.63	5.79	121.8

U.S. Customary Units			
3/4 x 3/4 x 7-1/2 in. CORRUGATION			
Thickness (in.)	A (in^2/ft)	R (in)	I x 10^{-3} (in^4/in)
0.064	0.509	0.258	2.821
0.079	0.712	0.250	3.701
0.109	1.184	0.237	5.537
0.138	1.717	0.228	7.433

Metric Units			
20 x 26 x 292 mm CORRUGATION			
Thickness (mm)	A (mm^2/mm)	R (mm)	I (mm^4/mm)
1.63	0.79	9.73	75.1
2.01	1.11	9.47	99.6
2.77	1.87	9.02	152

U.S. Customary Units			
3/4 x 1 x 11-1/2 in. CORRUGATION			
Thickness (in.)	A (in^2/ft)	R (in.)	I x 10^{-3} (in^4/in)
0.064	0.374	0.383	4.58
0.079	0.524	0.373	6.08
0.109	0.883	0.355	9.26

Note: Effective section properties are taken at full yield stress.

Table A12-3 - Steel Structural Plate Cross-Section Properties

Metric Units			
150 x 50 mm CORRUGATIONS			
Thickness (mm)	A (mm^2/mm)	R (mm)	I (mm^4/mm)
2.79	3.294	17.3	990
3.56	4.240	17.4	1280
4.32	5.184	17.4	1580
4.78	5.798	17.5	1770
5.54	6.771	17.5	2080
6.32	7.726	17.6	2400
7.11	8.719	17.7	2720
8.08	9.887	17.7	3115
9.65	11.881	17.9	3800

U.S. Customary Units			
6 x 2 in. CORRUGATIONS			
Thickness (in.)	Area (in^2)	Radius of Gyration, r (in.)	Moment of Inertia, I ($in^4/in \times 10^{-3}$)
0.110	1.556	0.682	60.4
0.140	2.003	0.684	78.2
0.170	2.449	0.686	96.2
0.188	2.739	0.688	108.0
0.218	3.199	0.690	126.9
0.249	3.650	0.692	146.2
0.280	4.119	0.695	165.8
0.318	4.671	0.698	190.0
0.380	5.613	0.704	232.0

Table A12-5 - Aluminum Spiral Rib Pipe - Cross-Section Properties

Metric Units			
19 x 19 x 190 mm CORRUGATION			
Thickness (mm)	A (mm^2/mm)	r (mm)	I (mm^4/mm)
1.52	0.88	6.91	41.9
1.91	1.20	6.78	55.3
2.67	1.93	6.55	83.1
3.43	2.73	6.40	111.9

U.S. Customary Units			
3/4 x 3/4 x 7-1/2 in. CORRUGATION			
Thickness (in.)	A (in^2/ft)	r (in.)	I x 10^{-3} (in^4/in)
0.060	0.415	0.272	2.558
0.075	0.569	0.267	3.372
0.105	0.914	0.258	5.073
0.135	1.290	0.252	6.826

Metric Units			
20 x 26 x 292 mm CORRUGATION			
Thickness (mm)	A (mm^2/mm)	r (mm)	I (mm^4/mm)
1.52	0.66	10.06	66.9
1.91	0.90	9.93	89.3
2.67	1.48	9.65	137
3.43	2.14	9.37	188

U.S. Customary Units			
3/4 x 1 x 11-1/2 in. CORRUGATION			
Thickness (in.)	A (in^2/ft)	r (in.)	I x 10^{-3} (in^4/in)
0.060	0.312	0.396	4.08
0.075	0.427	0.391	5.45
0.105	0.697	0.380	8.39
0.135	1.009	0.369	11.48

Note: Effective section properties are taken at full yield stress.

Table A12-8 - Minimum Longitudinal Seam Strengths Steel and Aluminum Structural Plate Pipe - Bolted

Metric Units				
150 x 50 mm STEEL STRUCTURAL PLATE PIPE				
Thickness (mm)	Bolt Diameter (mm)	13 Bolts/m (N/mm)	20 Bolts/m (N/mm)	26 Bolts/m (N/mm)
2.77	19.1	628	-	-
3.51	19.1	905	-	-
4.27	19.1	1180	-	-
4.78	19.1	1360	-	-
5.54	19.1	1640	-	-
6.32	19.1	1930	-	-
7.11	19.1	2100	2630	2830
8.08	22.2	-	-	3430
9.65	22.2	-	-	4160

U.S. Customary Units				
6 x 2 in. STEEL STRUCTURAL PLATE PIPE				
Bolt Thickness (in.)	Bolt Diameter (in.)	4 Bolts/ft (kip/ft)	6 Bolts/ft (kip/ft)	8 Bolts/ft (kip/ft)
0.109	3/4	43.0	-	-
0.138	3/4	62.0	-	-
0.168	3/4	81.0	-	-
0.188	3/4	93.0	-	-
0.218	3/4	112.0	-	-
0.249	3/4	132.0	-	-
0.280	3/4	144.0	180.0	194.0
0.318	7/8	-	-	235.0
0.380	7/8	-	-	285.0

Table A12-8 - Minimum Longitudinal Seam Strengths Steel and Aluminum Structural Plate Pipe - Bolted (Continued)

Metric Units			
230 x 64 mm ALUMINUM STRUCTURAL PLATE PIPE			
Thickness (mm)	Bolt Diameter (mm)	Steel Bolts 18 Bolts/m (N/mm)	Aluminum Bolts 18 Bolts/m (N/mm)
2.54	20	409	385
3.18	20	599	508
3.81	20	790	648
4.45	20	930	771
5.08	20	1070	771
5.72	20	1220	771
6.35	20	1360	771

U.S. Customary Units			
9 x 2-1/2 in. ALUMINUM STRUCTURAL PLATE PIPE			
Thickness (in.)	Bolt Diameter (in.)	Steel Bolts 5.5 Bolts/ft (kip/ft)	Aluminum Bolts/ft 5.5 Bolts (kip/ft)
0.100	3/4	28.0	26.4
0.125	3/4	41.0	34.8
0.150	3/4	54.1	44.4
0.175	3/4	63.7	52.8
0.200	3/4	73.4	52.8
0.225	3/4	83.2	52.8
0.250	3/4	93.1	52.8

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 1
PART B: DESIGN SPECIFICATIONS

SECTION 13 - RAILINGS

SECTION 13 - TABLE OF CONTENTS

SPECIFICATIONS

13.4 GENERAL

The following shall supplement A13.4.

The bridge railing and transition type chosen during the design field view process shall be taken from the bridge railings given on the BD or BC Standard Drawings.

Other than at deck expansion joints, open joints in the concrete portion of barriers are not permitted unless approved by the Chief Bridge Engineer.

COMMENTARY

C13.4

The following shall supplement AC13.4.

The bridge railings given on the BD or BC Standard Drawings have been approved by the FHWA.

Generally, the concrete barrier shown on the Typical Concrete Barrier Detail on Standard Drawing BD-601M will be specified for bridges.

PennDOT has adopted other bridge barrier designs. The PA HT barrier (shown on Standard Drawing BD-615M) and the PA barrier (shown on Standard Drawing BD-610M) may be used in place of the 1070 mm {42 in.} F-shape barrier. The 810 mm {32 in.} Alternate F-shape barrier and the PA 10M (shown on Standard Drawing BD-617M) may be used only if the guidelines in Design Manual Part 2, Chapter 12.9 for TL-4 barriers are met.

The barrier shown on BD-609M may be used on precast or cast-in-place reinforced concrete box culverts on non-limited access arterials, collectors, and local roadways.

Structure-related environmental commitments for bridge guide rail shall be carefully considered and justified. Use of weathering steel is permitted only if it is absolutely necessary. Also, note that the weathering steel may deteriorate at a faster rate when it is subject to deicing chemicals. Keep all possible options open for the designer to provide a structurally sound and economical bridge guide rail design during the final design phase. Where it is necessary to deviate from the established standards, justifications and special approvals should be well documented.

13.7 TRAFFIC RAILING

13.7.1 Railing System

13.7.1.1 GENERAL

The following shall supplement A13.7.1.1.

Protective fence in accordance with Standard Drawing BC-701M shall be provided on structures with sidewalks over Interstate and other limited access freeways, except where protective barrier is required. Also, provide protective fence on existing bridges, if unsafe incidents have occurred.

In addition, protective fence on structures with sidewalks shall be considered at the following locations:

- On an overpass near a school, playground, or other site where it would be expected that the overpass would be frequently used by children.
- On all overpasses in large urban areas used exclusively by pedestrians and not easily kept under surveillance by police.
- On overpasses with sidewalks, where experience on

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nearby structures indicates a need for protective fence.

For bridges with sidewalks over electrified railroads, protective barrier shall be used on affected spans or portions of spans. Use the appropriate protective barrier details as shown on Standard Drawings BC-711M, BC-707M, BC-713M or BC-709M for the type of barrier chosen for the structure.

If required by the Railroad or the Public Utility Commission, bridges without sidewalks over electrified railroads may require the appropriate protective barrier as shown on Standard Drawings BC-711M, BC-707M, BC-713M or BC- 709M for the type of barrier chosen for the structure.

13.7.2 Test Level Selection Criteria

The following shall replace the second paragraph of A13.7.2.

Design Manual Part 2, Chapter 12.9, Railing System Test Level Selection For Bridges provides the criteria for the selection of Test Level for the project. Under any circumstance that Design Manual Part 2 does not provide criteria for the selection of the Test Level, then Test Level Five, TL-5, shall be used, except when otherwise directed by the Department.

For bridges on Very Low Volume Roads, it is permissible to utilize a reduced Test Level provided the barrier is crash tested and acceptable to the FHWA. The bridge and bridge site must be checked to verify the Test Level used meets AASHTO guidelines.

13.7.3 Railing Design

13.7.3.1 GENERAL

The following shall supplement A13.7.3.1.

Table 1 lists barrier types, the performance level they were designed for and the location of standard details. The barriers were designed using the method provided in the Appendix to AASHTO, Section 13.

The Vertical Wall Bridge Barrier and Alternate Vertical Wall Bridge Barrier in Table 1 is only to be used in rehabilitation projects where providing new substructure safety wings is beyond the scope of work as follows:

a) for the transition sections at the end of the structure where the structure length is greater than 30 000 mm {100 ft.} in length and the typical concrete barrier is used; and

b) for the entire structure when structure length is less than 30 000 mm {100 ft.}.

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13.7.2

The following shall supplement AC13.7.2.

The Department recognizes the performance criteria contained in NCHRP Project 22-7, Report 350, Recommended Procedures for Safety Performance Evaluation of Highway Features, including criteria for crash tested barrier installations, as evidence of appropriately assigned Test Level designations. The standard 42 inch height F-shape bridge barrier is designated as a TL-5 barrier and is listed on the FHWA Approved list of Crash Tested Barriers contained in NCHRP Report 350, Appendix B - Update of the 1997 FHWA Bridge Rail Memorandum.

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Table 13.7.3.1-1 - Test Level for Barrier Type

Barrier Type	TL	Details on Standard Drawing
Typical Concrete Barrier	5	BD-601M, Sheet 2
Alternate Concrete Barrier	4	BD-601M, Sheet 2
Split Concrete Glare Screen Median Barrier Detail	4	BD-601M, Sheet 2
Alternate Split Concrete Median Barrier Detail	4	BD-601M, Sheet 2
Concrete Glare Screen Median Barrier Detail	4	BD-601M, Sheet 3
Concrete Median Barrier Detail	4	BD-601M, Sheet 3
Vertical Wall Bridge Barrier at Alternate Sidewalk Detail	5	BD-601M, Sheet 4
PA Bridge Barrier	5	BD-610M
PA HT Bridge Barrier	5	BD-615M
PA Type 10M Bridge Barrier	4	BD-617M
Vertical Wall Bridge Barrier	5	BD-618M
Alternate Vertical Wall Bridge Barrier	4	BD-618M
Vertical Wall Bridge Barrier for Non-Composite Adjacent Box Beams	4	BD-618M
Alternate Concrete Barrier for Plank Beams	4	BD-661M, Sheet 3
Typical Barrier Reinforcement for Composite Adjacent Box Beams	5	BD-661M, Sheet 4
Vertical Wall Bridge Barrier for Composite Adjacent Box Beams	5	BD-661M, Sheet 5
Alternate Concrete Barrier for Non-Composite Adjacent Box Beams	4	BD-661M, Sheet 5

13.7.3.2 HEIGHT OF TRAFFIC BARRIER OR RAILING

The following shall supplement A13.7.3.2.

The required railing heights are reflected in the BD-601M, BD-610M, BD-615M, BD-617M and BD-618M Standard Drawings.

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13.8 PEDESTRIAN RAILING

13.8.1 Geometry

The following shall supplement A13.8.1.

Unless otherwise directed by the Department, pedestrian railing shall be provided as given in the BC Standard Drawings.

The minimum height of a pedestrian railing shall be 1070 mm {42 in.} measured from the top of the walkway.

A pedestrian rail may be composed of horizontal and/or vertical elements. The clear opening between elements shall be such that a 150 mm {6 in.} diameter sphere shall not pass through.

When both horizontal and vertical elements are used, the 150 mm {6 in.} clear opening shall apply to the lower 685 mm {27"} of the railing, and the spacing in the upper portion shall be such that a 200mm {8 in.} diameter sphere shall not pass through. A safety toe rail or curb should be provided. Rails should project beyond the face of posts as shown in Figure AA13.1.1-2.

The rail spacing requirements given above should not apply to chain link or metal fabric fence support rails and posts. Mesh size in chain link or metal fabric fence should have openings no larger than 50 mm {2 in.}

A13.2 TRAFFIC RAILING DESIGN FORCES

The following shall replace Table AA13.2-1.

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Table A13.2-1 - Design Forces for Traffic Railings

Design Forces and Designations	Railing Test Levels					
	TL-1	TL-2	TL-3	TL-4	TL-5	TL-6
F_t Transverse (N) {kip}	60 000 {13.5}	120 000 {27}	240 000 {54}	240 000 {54}	550 000 {124}	780 000 {175}
F_L Longitudinal (N) {kip}	20 000 {4.5}	40 000 {9.0}	80 000 {18}	80 000 {18}	183 000 {41}	260 000 {58}
F_v Vertical Down (N) {kip}	20 000 {4.5}	20 000 {4.5}	20 000 {4.5}	80 000 {18}	355 000 {80}	355 000 {80}
L_t and L_L (mm) {ft.}	1200 {4.0}	1220 {4.0}	1220 {4.0}	1070 {3.5}	2440 {8.0}	2440 {8.0}
L_v (mm) {ft.}	5500 {18.0}	5500 {18.0}	5500 {18.0}	5500 {18.0}	12 200 {40.0}	12 200 {40.0}
H_e (min) (mm) {in.}	460 {18}	510 {20}	610 {24}	810 {32}	1070 {42}	1420 {56}
Minimum H Height of Rail (mm){in.}	685 {27}	685 {27}	685 {27}	810 {32}	1070 {42}	2290 {90}

A13.3 DESIGN PROCEDURE FOR RAILING TEST SPECIMENS

A13.3.1 Concrete Railings

The following shall replace the definition for M_w in A13.3.1.

M_w = flexural resistance of the wall about its vertical axis (N-mm) {kip-ft}

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DESIGN MANUAL

PART 4

VOLUME 1
PART B: DESIGN SPECIFICATIONS

SECTION 14 - JOINTS AND BEARINGS

SECTION 14 - TABLE OF CONTENTS

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14.1 SCOPE

The following shall supplement A14.1.

The materials, fabrication and installation of the bearings shall be accordance with Publication 408.

14.2 DEFINITIONS

The following shall supplement A14.2.

Guided Expansion Bearings - Bearings which allow rotation and longitudinal movement in the bearing plane; transverse movement shall be restricted.

Non-Guided Expansion Bearings - Bearings which allow rotation, longitudinal movement and transverse movement in the bearing plane.

14.3 NOTATION

The following shall supplement A14.3.

A_b = bonded area of rubber

A_r = reduced net bonded area of rubber
= $A_b (1 - \Delta/B)$

Δ = the shear deflection of the bearing appropriate to the calculation

B = plan dimension in loaded direction of rectangular bearing or diameter of circular bearing

D = superstructure depth from bearing to top of deck (mm) {in.} (D14.5.3.2)

d_i = lateral displacement under earthquake loads as specified in D4.7.4.5.2P

k = material constant

P = maximum vertical load resulting from the Extreme Event I with the γ_p factors equal to 1.0

ε_{eq} = shear strain due to d_i, the seismic design displacement

ε_{sc} = shear strain due to vertical loads

ε_{sh} = shear strain due to maximum horizontal displacement resulting from creep, post-tensioning, shrinkage, and thermal effects computed between the installation temperature and the least favorable extreme temperature

ε_{sr} = shear strain due to imposed rotation

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In rare cases, some guided bearings may restrict longitudinal movement and allow transverse movement in the bearing plane.

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ε_c = compression strain in total bearing due to vertical loads

ε_u = minimum elongation-at-break of rubber

θ = rotation imposed on bearing

14.4 MOVEMENTS AND LOADS

14.4.2 Design Requirements

The following shall supplement A14.4.2.

Thermal movements and rotations for all bearings and joints shall be designed using Service-I limit state with γ_{TU}=1.

14.4.2.2 HIGH LOAD MULTIROTATIONAL (HLMR) BEARINGS

The following shall supplement A14.4.2.2.

The service limit state rotation, θ_s, for bearings such as pot bearings, disc bearings, and curved sliding surfaces that may develop hard contact between metal components shall be taken as the sum of:

- The rotations due to all applicable Service I loads;
- The maximum rotation caused by fabrication and installation tolerances, which shall be taken as 0.01 RAD unless an approved quality control plan justifies a smaller value; and
- An allowance for uncertainties, which shall be taken as 0.01 RAD unless an approved quality control plan justifies a smaller value.

14.4.2.2.1 POT BEARINGS AND CURVED SLIDING SURFACE BEARINGS

Delete A14.4.2.2.1.

14.4.2.2.2 DISC BEARINGS

Delete A14.4.2.2.2.

14.5 BRIDGE JOINTS

14.5.1 Requirements

14.5.1.1 GENERAL

The following shall supplement A14.5.1.1.

Superstructure joints shall be in accordance with the provisions in A14.5, D14.5 and the Standard Drawings. Any other type must be evaluated and approved through a

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C14.4.2

The Department does not use γ_{TU}=1.2 as shown in Table A3.4.1-1. Structures in Pennsylvania have been successfully designed using the temperature ranges listed in Table D3.12.2.1-1 under Service-I Limit State with γ_{TU}=1 and construction tolerances

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process using performance evaluation under heavy truck traffic criteria.

14.5.2 Selection

14.5.2.1 NUMBER OF JOINTS

The following shall supplement A14.5.2.1.

The number of expansion joints shall be minimized by designing continuous steel or prestressed concrete structures. Refer to Appendix G for Integral Abutments. For integral abutment details, refer to BD-667M Standard Drawing. For bridge approach slab details, refer to BD-628M Standard Drawing.

14.5.3 Design Requirements

14.5.3.2 MOVEMENTS IN SERVICE

The following shall supplement A14.5.3.2.

Minimum movement classification shall be 75 mm {3 in.}, even if less movement is anticipated either at fixed or expansion ends. Round off movement results to the next highest 10 mm {1/2 in.}.

The designer shall calculate the end rotation for all joints. End rotation due to applied dead and live loads of girders at the fixed end shall be considered in determining movement classification. End rotation is extremely important at fixed-fixed bearings and to a lesser extent at fixed-expansion bearings. End rotation can be ignored at expansion-expansion bearings, except at skewed joints.

The effect of live load end rotation for a 90° skew fixed end can be roughly approximated by

$$\delta = \frac{4\Delta D}{L} \qquad (14.5.3.2\text{-}1)$$

where:

Δ = L/800 or L/1000 (mm) {ft.}

L = length of end span (mm) {ft.}

D = superstructure depth from bearing to top of deck (mm) {in.}

δ = change in joint opening (mm) {in.}

The expansion dams shall accommodate the longitudinal movement due to temperature change, end rotation of superstructure due to deflection, substructure movement and skew effect of the structure. A 10 mm {1/2 in.} contingency allowance (allow 0.66 × 10 mm {0.5 in.} for low temperature and 0.33 × 10 mm {0.5 in.} for higher temperature) for joints at abutments, and a 5 mm {1/4 in.} contingency for joints at piers shall be made unless larger

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The following shall supplement AC14.5.3.2.

The end rotation for right angle and skewed bridges is explained below.

- End rotation and joint openings of right angle bridges

  At a deck joint on the fixed end of a girder, the increase in joint opening is directly proportional to the amount of end rotation of the girder and to the vertical height from the bearing to the joint. At a similar deck joint on the expansion end of a girder, the amount of joint opening is a function of temperature and span length, as well as the amount of end rotation of the girder and the relative position of the neutral axis with respect to the bearing and the joint. Note that the neutral axis of a girder is unstressed and will not change length under load. However, the compression flange will shorten and the tension flange will lengthen under load.

  In the case where the neutral axis is midway between the bearing and the joint, when the girder is loaded the expansion bearing will move away from the fixed bearing by an amount equal to the amount that the joint over the fixed bearing opens, and the joint at the expansion end will remain stationary. If the neutral axis is closer to the joint than to the bearing, when the girder is loaded the expansion bearing will move away from the fixed bearing by an amount greater than the amount that the joint over the fixed bearing opens, and the joint at the expansion end will actually close. If the neutral axis happens to be closer to the bearings than to the joint, the expansion bearing will move a smaller amount than will the joint at the fixed bearing, and the joint at the expansion end will open.

  If the guidelines given above are properly considered, the displacement of bearings and joints for normal

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movements are anticipated in the foundation report.

A minimum 50 mm {2 in.} movement classification shall be used for the fixed-fixed condition. Opening requirements from end rotation of the fixed-fixed condition must be evaluated.

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bridges can be predicted with reasonable accuracy, and provisions can be made for presetting the bearings and joints prior to placing the deck slab so that they will be in proper position in the finished structure. In the past, however, the presetting of dams on skewed structures has often resulted in misfits so bad as to require the removal of the dam from the hardened concrete and the resetting of it.

- End rotation and joint openings of skewed bridges

  Figure C1 represents a schematic plan view of the end of a girder at skewed substructure units. Point A represents a point on a dam directly above the intersection of the centerline of the girder and the centerline of fixed bearings, the girder being undeflected. Point A' represents the position of Point A on the deflected girder as calculated on the basis of the girder rotating about an axis normal to the girder. Point A" represents the position of Point A on the actual deflected structure. Point E represents a point on a dam directly above the intersection of the centerline of the girder and the centerline of expansion bearings on the same undeflected girder. Point E' represents the position of Point E after a temperature drop that changes the span length by the amount "e". Point E" represents the position of Point E (due to deflection only) on the actual deflected structure, assuming that the neutral axis of the girder is midway between the bearing and the dam. The location of Point A" relative to Point A, and Point E" relative to Point E, is based on the assumption that the ends of the girder will rotate about an axis on the centerline of bearing rather than on an axis normal to the girder. A stiff end diaphragm system validates this assumption.

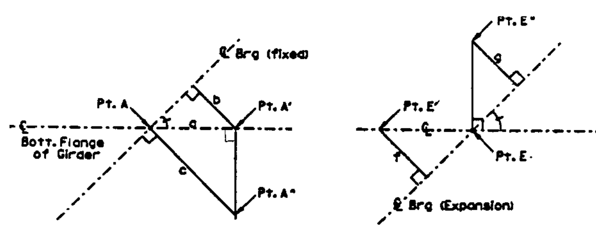

Figure C14.5.3.2-1 - End Rotation and Joint Openings of Skewed Bridges

For thermal movement, e, the change in the expansion dam opening (normal to the dam) will always be

f = e sin (skew angle)

For a skewed structure with little or no torsional stiffness, but having an end diaphragm system, the following relationships will hold true for the normal

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dam openings:

c = a / sin (skew angle)

All structures have varying degrees of torsional stiffness, which in the case of a skewed structure affects the amount of deflection that will occur under a particular loading condition. The greater the torsional stiffness, the smaller the deflection that will occur. There are no readily available methods of accurately predicting the deflection and subsequent end rotation of a torsionally stiff, severely skewed structure other than sophisticated finite element methods. Therefore, for such cases, a gap (block-out) should be left near the dam in the deck slab when it is poured so that final adjustment of the dam can be made after the major portion of dead load deflection has occurred.

14.5.5 Installation

14.5.5.1 ADJUSTMENT

The following shall supplement A14.5.5.1.

Superstructure joints shall be carefully detailed on the design drawings. Provisions shall be made to block out 600 mm {2 ft.} of the deck slab adjacent to a joint so that the joint opening may be adjusted after the deck slab has been placed. The Contractor shall be given the option to eliminate the block-out on 90-degree bridges and for other than tooth dams on skewed bridges, provided the ability to achieve a satisfactory joint in the completed structure is demonstrated with appropriate calculations. Blockouts may also be eliminated if the end rotation due to dead load is less than 5 mm {1/4 in.}.

14.5.6 Considerations for Specific Joint Types

14.5.6.10P SELECTION OF JOINT TYPE

Expansion joint types: If expansion joints are needed, the following shall be specified:

- Strip seals shall be specified for anticipated movements between 0 mm {0 in.} minimum and 100 mm {4 in.} maximum. For construction requirements, refer to Standard Drawing BC-767M.

- Compression seal (unarmored) may be provided for bridges with a design ADTT of 100 or fewer trucks per day. If compression seal is permitted for a bridge, its use shall be specifically indicated either on the bridge plans or in the contract specifications; otherwise, a strip seal shall be used. Compression seals shall be specified to have a 30 mm {1 in.} minimum movement classification.

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Table C14.5.6.10P shows the field measurements of various types of proprietary expansion dams. This table can be used to check to ensure a system is available for the computed final design movements.

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- Tooth dam with 3 mm {1/8 in.} thick reinforced sheet neoprene trough (50 durometer hardness) shall be specified for movements over 100 mm {4 in.}. Drainage arrangement for the trough, including downspouting, shall be specified as a part of the tooth expansion dam. A minimum of a 760 mm {2'-6"} (but not to exceed shoulder width) plate shall be provided in the gutter area for the safety of bicyclists.

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Table C14.5.6.10P-1 - Experimentally Determined Perpendicular Movement Capabilities of Evaluated Systems vs. Angle of Crossing

	Perpendicular Movement Capability (mm)							Perpendicular Movement Capability (in.)						
	Angle of Crossing (degrees)							Angle of Crossing (degrees)						
Joint System	90	80	70	60	50	40	30	90	80	70	60	50	40	30
Onflex 25	45	45	45	45	45	45	45	1.7	1.7	1.7	1.7	1.7	1.7	1.7
Onflex 20	45	45	45	45	45	45	40	1.8	1.8	1.8	1.8	1.8	1.8	1.6
Pro-Span 50 mm {2 in.} system (low profile)	50	50	50	50	45	40	35	2.0	2.0	2.0	2.0	1.7	1.5	1.4
Acme 75 mm {3 in.} Strip Seal (AS 300)	75	75	75	70	50	50	30	3.0	3.0	3.0	2.7	1.9	1.9	1.2
Acme 75 mm {3 in.} Trojan (TR 300)	75	75	75	75	70	50	35	3.0	3.0	3.0	3.0	2.7	2.0	1.4
Watson Bowman 75 mm {3 in.} system (S 300)	75	75	75	75	75	55	35	3.0	3.0	3.0	3.0	2.9	2.2	1.4
Acme 100 mm {4 in.} Trojan (TR 400)	80	80	75	65	40	25	15	3.2	3.2	3.0	2.5	1.5	1.0	0.6
Onflex 40	95	95	95	95	90	70	50	3.8	3.8	3.8	3.8	3.6	2.8	2.0
Acme 100 mm {4 in.} Strip Seal (AS 400)	100	100	100	50	45	35	25	4.0	4.0	4.0	2.0	1.7	1.3	1.0
Watson Bowman 100 mm {4 in.} system (S 400)	100	100	100	95	70	45	35	4.0	4.0	4.0	3.8	2.8	1.8	1.3
Pro-Span 100 mm {4 in.} system (low profile)	100	100	100	95	80	70	55	4.0	4.0	4.0	3.8	3.2	2.8	2.2
Onflex 45	105	105	100	95	90	70	50	4.1	4.1	4.0	3.7	3.6	2.8	2.0

Source: F. J. Bashore, et al, "Determination of Allowable Movement Ratings for Various Proprietary Bridge Deck Expansion Joint Devices at Various Skew Angles", Michigan Transportation Commission

14.6 REQUIREMENTS FOR BEARINGS

14.6.1 General

The following shall replace the third paragraph of A14.6.1.

Bearings subject to net uplift at Service I limit state shall be secured by tie-downs, anchorages, or counterweights. The tie-downs, anchorages, or counterweights must be designed to resist the factored net uplift force at Strength I Limit State. If the counterweight extends beyond a distance equal to the depth of the girder from center line of bearing, it should be included in the girder design. Reaction due to counterweights should also be included in the bearing design. Detail the counterweight on the contract drawings.

For additional provisions on bearing anchorages see

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A14.8.3 and D14.8.3.

14.6.1.1P FIXED AND EXPANSION BEARINGS

For single span bridges with a span of 10 000 mm {35 ft.} and less, provide a fixed condition at both abutments. For other single span bridges, arrange the fixed and expansion conditions so that the bridge is expanding uphill, wherever practical.

Where bearings are used to prevent or restrict movement in any direction, a minimum of two such bearings shall be employed on any superstructure unit.

Laminated bearings, known as “Masticord” bearings, are not allowed in Pennsylvania.

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Generally, fixed bearings prevent translation in any direction, and expansion bearings permit translation longitudinally, but are fixed against lateral translation. However, these simple arrangements do not always satisfy the required restraints and freedom of movement required for certain types and sizes of bridge superstructures. For instance, thermally-induced movement radiates from a point which is coincident with the center of gravity of the structure. For a bridge on tangent, that point will lie on the longitudinal centerline at the mid-point between the ends of the bridge; regardless of which of the bearings are fixed, the thermal movement will always be parallel to the centerline. However, for a bridge on curved alignment, the center of gravity of the superstructure will be toward the radius of curvature from the centerline; thermal movement will be diagonal to the centerline with respect to that point.

Inspection of Masticord bearings in one Pennsylvania structure revealed completely unsatisfactory performance. The laminated bearings debonded in all layers, i.e., Teflon debonded from stainless steel, Teflon/stainless steel sandwich plate debonded from Masticord material, and the Masticord debonded from 6 mm steel sole plate. According to the manufacturer, these bearings are supposed to be particularly forgiving when subjected to non-uniform loading. This has not been experienced in Pennsylvania.

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14.6.1.2P MULTI-ROTATIONAL BEARINGS

All bearings must be capable of permitting rotation in at least one direction. For bridges on tangent alignment, bearing rotation in a single direction is adequate. However, for certain applications such as curved bridges, sharply skewed bridges, and cross girders, the bearings must be able to rotate in more than one direction. In such cases, multi-rotational bearings which utilize an elastomeric material or spherical sliding surfaces shall be used.

The use of multi-rotational bearings shall be indicated where:

- Low profile, high load bearings are required.
- Curved or skewed bridges and other similar structures of complex design are required.
- Long slender columns or light frames and members exhibit minimum stiffness or rigidity.
- The direction of rotation varies.
- The direction of rotation cannot be precisely determined.

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This section has been revised to include various types of High Load Multi-Rotational Bearings other than just pot bearings. This specification was prepared for the broad range of normal applications and the limits of loads, forces and movements stated. The design and manufacture of multi-rotational bearings relies heavily on the principles of engineering mechanics and extensive practical experience in bearing design and manufacture. Therefore, in special cases in which structural requirements fall outside the limits of this specifications, a bearing manufacturer shall be consulted.

This specification treads a fine line between the need to produce high-quality, long-life bearings and the need to compromise with what is actually possible in design and manufacturer. For this reason, every item in the specification is vital to some aspect of bearing function and cannot be readily changed in the interest of conservativeness or deemed unnecessary and left out.

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- Self aligning capabilities are required.
- Load and rotation eccentricity does not significantly alter the net distribution of stress through the bearing and into the substructure and superstructure.
- It is desirable to reduce the moment applied to truss or space frame panels.
- Large movements are required.
- Economical, long life, zero or low maintenance bearings are desirable.

Circular neoprene bearings meeting all applicable design requirements may be used as multi-rotational bearings.

Bearings shall be designed for the total movement capacity specified under "Design Movement" in the bearing schedule plus 25 mm {1 in.} additional movement in each direction. Spacing between the guides of the bearing does not require this additional movement capacity. The centerline of all bearing components are symmetrical about the bearing stiffener. The temperature value used for the calculation of longitudinal design movement shall be as per D3.12.2.1.

Only <u>one</u> fixed or guided expansion bearing <u>shall be assumed</u> to resist the sum of all the transverse horizontal forces at each abutment, bent, column, hinge or pier. Seismic forces are an exception as these forces may be resisted by all fixed or guided expansion bearings located at a given substructure unit. Longitudinal loads are resisted <u>only at fixed bearings</u>, and transverse loads are resisted by fixed and guided expansion bearings.

Provide <u>at least two</u> fixed or guided expansion bearings, <u>each</u> able to resist all transverse horizontal forces at each abutment, column, hinge or pier for design redundancy.

The substructure and superstructure shall be designed so that the sole and masonry plates remain rigid under all service conditions in areas around and in contact with the bearings.

14.6.1.2.1P BEARING SCHEDULE

Contract documents shall contain a "bearing schedule" indicating the following information:

- Provide a schedule of all minimum and maximum vertical and horizontal loads for LRFD Load Combinations as shown in Figure 1 (this includes all longitudinal and transverse forces, as well as seismic forces) (not applicable for elastomeric bearings).
- Indicate minimum design rotation requirements of the bearing, including a construction tolerance.

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Using a value for longitudinal design movement equal to twice the amount of contraction provides more movement than theoretically needed. The purpose of using this value is to ensure that adequate movement is provided when the bridge is erected at temperatures greater than 20°C (68°F).

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Design rotation, movement and other requirements in the bearing schedule shall only refer to the requirements of the structure where the bearings are to be used. The design specification apply very conservative safety factors to the design requirements.

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- Show magnitude and direction of movements at all bearing support points including thermal, creep and shrinkage movements (see D14.6.1).

- Show the location, quantity and type of each bearing (fixed, expansion, or guided expansion), and the location of all bearing units (an actual bearing layout is preferred or use a bearing framing plan to show this data).

- Indicate the nominal stresses and nominal upper and lower bearing contact pressure to be used in the bearing design. They are provided in DM-4 and LRFD Specifications.

- Indicate and properly detail all anchorage details and/or requirements.

- Provide all details and indicate all grades, bevels and slopes at each bearing location.

- Indicate the coefficient of friction used in design of the sliding surfaces. (They should also be used in the substructure design.)

- Highlight any special details needed for earthquake requirements, such as uplift details, temporary attachments, or other requirements.

- Define the surface coating requirements, including the coating specifications and specific surfaces to be coated.

- Field welding of the sole plate to the beam may be done provided a special provision is developed to assure qualified welders and proper welding inspection.

- Show beam seat elevations based on an assumed total bearing thickness detailed in the plans.

SUGGESTED FORMAT FOR LOAD PORTION OF BEARING SCHEDULE

LOAD COMBINATION	FACTORED LOADS (kN) {kips}							
	VERTICAL				HORIZONTAL			
	DL		LL+I		TRANSVERSE		LONGITUDINAL	
	MIN	MAX	MIN	MAX	MIN	MAX	MIN	MAX

To the designer:

This table is required for every bearing type (not applicable for elastomeric bearings). Engineering judgment can be used to eliminate groups which obviously will not control the bearing design in order to limit the table size.

Figure 14.6.1.2.1P-1 – Suggested Format for Load Portion of Bearing Schedule

14.6.3 Force Effects Resulting from Restraint of Movement at the Bearing

14.6.3.1 HORIZONTAL FORCE AND MOVEMENT

The following shall replace the definition of G and Δ_u for Equation A14.6.3.1-2.

G = shear modulus of the elastomer (use the highest value of G within the range for elastomer hardness selected, see Table A14.7.5.2-1) (MPa) {ksi}

Δ_u = factored shear deformation (using the appropriate load factors from A3.4 and D3.4, factor the service value shear deformation, Δ_s, determined from A14.7.5.3.4) (mm) {in.}

The following shall replace the last sentence of A14.6.3.1.

For rocker bearings, the factored longitudinal rolling and sliding force is given by

$$H_u = 0.25\gamma\left(D_l\right)\frac{r}{R} \qquad (14.6.3.1\text{-}3)$$

where:

D_l = dead load reaction on the rocker (N) {kips}

γ = load factor from A3.4 or D3.4

r = average radius of the web and sole plates or the pin (mm) {in.}

R = radius of the rocker (mm) {in.}

The following shall supplement A14.6.3.1.

Frictional resistance of bearings slide surfaces shall be

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excluded when specifying horizontal load requirements.

Minimum horizontal load capacity for fixed or expansion bearings shall be 10 percent of the vertical load capacity.

14.6.4 Fabrication, Installation, Testing and Shipping

The following shall replace A14.6.4.

The provisions for fabrication, installation, testing and shipping of bearings, specified in Publication 408, shall apply.

14.6.4.1P CONSECUTIVELY FIXED PIERS

Whenever it is advantageous to the overall design, consecutively fixed piers should be utilized. Generally it will be advantageous for tall, slender piers. However, an analysis should be made, taking into account the stiffness of piers, thermal movements and distribution of horizontal forces. The determination of the number of piers to be consecutively fixed must be based on cost-effectiveness.

When consecutively fixed piers are utilized in a design, instructions for jacking the required deflection into the piers for proper positioning of the bearings under the beams shall be shown on the drawings. A table of dimensions shall be included showing the relative distance that each pier must be moved for each 5º C {10º F} temperature variation from the mid-range of the anticipated temperature extremes.

The theoretical fixed point on the bridge, based on the relative stiffness and heights of the piers that are fixed, shall also be shown on the drawings.

14.6.4.2P JACKING

Provision shall be made on the superstructure and substructure units to place jacks in order to jack the superstructure for bearing repair or replacement.

The jacking load shall consist of dead load only.

14.7 SPECIAL DESIGN PROVISIONS FOR BEARINGS

14.7.1 Metal Rocker and Roller Bearings

14.7.1.1 GENERAL

The following shall replace the third paragraph of A14.7.1.1.

Steel rocker and roller bearings are not permitted on new structures and should be replaced on rehabilitation projects.

Approval from the Chief Bridge Engineer is required when it is desired to leave rocker bearings and/or roller bearings in place on rehabilitation projects.

COMMENTARY

C14.7.1.1

Delete the third paragraph of AC14.7.1.1.

The following shall supplement AC14.7.1.1.

Figure PP5.5.2.8.2-1 indicates the bearing types to be replaced, and Figure PP5.5.2.8.2-2 indicates a schematic example of a bearing replacement.

SPECIFICATIONS COMMENTARY

14.7.2 PTFE Sliding Surfaces

14.7.2.1 PTFE SURFACE

14.7.2.1.1P Spherical Element - Concave Surface - PTFE

- The spherical radius shall be determined such that the resulting geometry of the bearing is capable of withstanding the greatest ratio of horizontal load to vertical load under all loading conditions to prevent unseating the concave element.

- If required during construction, mechanical safety restraints shall be incorporated to prevent overturning.

- Maximum design rotation of the structure itself, plus 0.02 radians, shall be considered in the bearing design to prevent overturning or uplift.

- Calculations showing the determination of the radius shall be submitted for approval.

- The concave surface shall face down.

Refer to Figure 1 for spherical bearings.

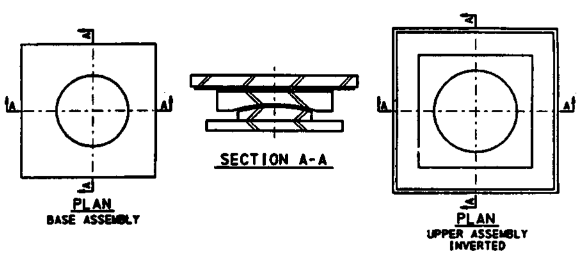

Figure 14.7.2.1.1P-1 - Spherical Bearings

14.7.2.2 MATING SURFACE

The following shall supplement A14.7.2.2.

For PTFE/Stainless and Bronze/Stainless sliding surfaces, the stainless surface shall be one of the following:

- ASTM A 240/A 240M, Type 304, 1.5 to 2.3 mm {0.060 to 0.090 in.} thick with a 0.4 µm {20 µin.} rms finish or less.

- Solid stainless steel ASTM A 240/A 240M, Type 304 or 304L, with a 0.4 µm {20 µin.} rms finish or less.

- Stainless steel weld overlay a minimum of 2.4 mm {3/32 in.} thick with a 0.4 µm {20 µin.} rms finish or less.

SPECIFICATIONS

14.7.2.3 MINIMUM THICKNESS

14.7.2.3.1 PTFE

The following shall supplement A14.7.2.3.1.

PTFE fabric shall be in accordance with ASTM D 1777.

Any edges other than the selvedge shall be oversown or recessed so that no cut fabric edges are exposed.

14.7.2.3.2 Stainless Steel Mating Surface

The following shall supplement A14.7.2.3.2.

The stainless steel surface shall cover the mating surface in all operating positions plus 25 mm {1 in.} in each direction of movement or extend to the end of sole plate whichever is greater.

Stainless steel sliding surfaces shall face down.

Stainless steel welded overlay shall be a minimum of 2.4 mm {3/32 in.} thick after welding, grinding, and polishing, and produced using Type 309L electrodes.

Guided members shall have their contact area within the guide bars in all operating positions. Guiding off the fixed base or any extensions of it is not permitted.

14.7.2.5 COEFFICIENT OF FRICTION

The following shall supplement A14.7.2.5.

Friction coefficients must be used in conjunction with the substructure design.

Design friction factors shall correspond to the type of PTFE specified.

Design friction factors given in Table A14.7.2.5-1 may be used for pot bearings if a 0.2 µm {8 µin.} rms surface finish is specified for the mating surface. In using Table A14.7.2.5-1, a temperature of -25ºC {-13ºF} shall be assumed.

COMMENTARY

C14.7.2.5

A temperature of -25ºC {-13º F} is the closest temperature equating to the cold end of the DM-4's design temperature range for structures.

14.7.3 Bearings with Curved Sliding Surfaces

14.7.3.1 GENERAL

The following shall supplement A14.7.3.1.

Complete design calculations must be provided by the designer for all aspects of spherical bearings. For high load multi-rotational bearing design plan presentation, see D14.6.1.2.1P. Bearings shall be designed for the temperature range found in Table D3.12.2.1-1.

These provisions are directed primarily toward spherical or cylindrical bearings with bronze or PTFE sliding surfaces.

The minimum center thickness of concave spherical surfaces must be at least 19 mm {3/4 in.}, and the minimum vertical clearance between the rotating and non-rotating parts is given by Equations 1 or 2.

C14.7.3.1

Delete AC14.7.3.1.

The first two paragraphs of AC14.7.3.1 have been moved into the specific D14.7.3.1. This third paragraph has been replaced by some Department requirements and given in the last paragraph of D14.7.3.1.

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For rectangular spherical or curved bearings:

Metric Units: $c = 0.7D\theta_u + 3.2$ (14.7.3.1-1)

U.S. Customary Units: $c = 0.7D\theta_u + 0.125$

For round spherical or round bearings:

Metric Units: $c = 0.5D\theta_u + 3.2$ (14.7.3.1-2)

U.S. Customary Units: $c = 0.5D\theta_u + 0.125$

where:

D = diameter of the projection of the loaded surface of the bearing in the horizontal plane (mm) {in.}

θ_u = factored design rotation (RAD)

Minimum edge and center thicknesses shall be no less than

- OD x 0.06 for bearings directly on concrete
- OD x 0.045 for bearings on steel masonry plates
- 12.5 mm {1/2 in.}

14.7.4 Pot Bearings

14.7.4.1 GENERAL

The following shall supplement A14.7.4.1.

BD-613M shall be used for pot bearings with vertical service loads between 890 kN (200 kips) and 5293 kN (1500 kips) with 0.03 radians maximum rotation. If the standard is not used, complete design calculations must be provided prior to submission of shop drawings on all projects for all aspects of pot bearings, see A14.6.1 and D14.6.1.

Pot bearings are not stiff against bending in their plane. A sole plate, beveled if necessary, on top and a masonry plate at the bottom of the bearing shall be provided. Pot bearings have a limited capacity of rotation; they shall not be mixed with other types of bearings at common superstructure and substructure units.

For high load multi-rotational bearing design plan presentation, see D14.6.1.2.1P.

As a part of final design, designers are responsible for the design of all aspects of the pot bearing as well as the pot

COMMENTARY

C14.7.3.3 RESISTANCE TO LATERAL LOAD

To prevent the accumulation of debris, the concave surface of the bearing shall face down.

SPECIFICATIONS

base thickness. These items are all included in BD-613M. If the standard is not used, these items must be designed:

- pot diameter
- pot wall thickness and height
- piston shaft diameter (Min.), piston bottom diameter (Min.), and sealing rings
- masonry plate size and thickness
- sole plate size and thickness
- guide plate thickness, length and width (Min.); piston to guide plate weld size (Min.)
- sliding surface dimensions; length of Teflon strip and width (Min.); number of flat head socket screws
- all connection requirements, including weld sizes and number and size of bolts, cap screens and anchor bolts

14.7.4.1.1P Types of Pot Bearings

There are two types of pot bearing designs allowed in Pennsylvania:

- Pot facing up with plates attached to the piston and guided by the edge of the sliding plate. Piston is to be self-aligning to direction of movement, but guides are to be carefully aligned. This pot bearing must be designed for PennDOT structures, unless approved otherwise by the Chief Bridge Engineer at the TS&L stage.
- Design as specified above, but with double elastomeric pad for greater rotational capacity.

14.7.4.2 MATERIALS

The following shall replace the first paragraph of A14.7.4.2.

The elastomeric disc shall be made from a compound based on virgin natural rubber or virgin neoprene conforming to the requirements of Publication 408.

14.7.4.4 ELASTOMERIC DISC

Delete the last sentence of the second paragraph of A14.7.4.4.

COMMENTARY

C14.7.4.4

Delete the last sentence of the second paragraph of AC14.7.4.4.

14.7.4.5 SEALING RINGS

14.7.4.5.3 Rings with Circular Cross-Sections

SPECIFICATIONS COMMENTARY

The following shall replace paragraph A14.7.4.5.3.

Sealing rings with a circular cross-section shall have the following minimum diameters:

- Up to 550 kN {125 kips} - 4 mm {5/32 in.}
- Up to 1780 kN {400 kips} - 5 mm {3/16 in.}
- Up to 3560 kN {800 kips} - 8 mm {5/16 in.}
- Up to 6230 kN {1400 kips} - 9.5 mm {3/8 in.}
- Up to 22 240 kN {5000 kips} - 12.5 mm {1/2 in.}

14.7.4.6 POT

The following shall supplement A14.7.4.6.

Pot inside diameter shall be the same as the elastomeric disc.

If BD-613M is not used, the minimum pot wall thickness shall be determined by analyzing horizontal loads, internal elastomer pressure, and piston force due to friction, shear, bending, and tension, but shall not be less than 19 mm {3/4 in.}.

Pots shall be connected to the masonry plate by either setting the pot in a recess designed for horizontal loads or by welding the pot to the masonry plate. If the pot is set in a recess proper drainage is required.

The following shall replace the second paragraph of A14.7.4.6.

The minimum thickness of a base bearing directly against concrete or grout shall satisfy:

- $t_{base} \geq 0.06\ D_p$ and (14.7.4.6-1)
- $t_{base} \geq 19$ mm {3/4 in.} (14.7.4.6-2)

The thickness of a base bearing directly on steel girders or load distribution plates shall satisfy:

- $t_{base} \geq 0.045\ D_p$ and (14.7.4.6-3)
- $t_{base} \geq 12.5$ mm {1/2 in.} (14.7.4.6-4)

14.7.4.7 PISTON

The following shall replace the fourth paragraph of A14.7.4.7 through equation 14.7.4.7-3.

Pot bearings subjected to lateral loads shall be proportioned to satisfy:

$$t_w \geq \sqrt{\frac{40 H_s \theta_s}{F_y}} \qquad (14.7.4.7\text{-}1)$$

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COMMENTARY

Pot bearings that transfer load through the piston shall satisfy:

$$w \geq \frac{2.5H_s}{D_p F_y} \qquad (14.7.4.7\text{-}2)$$

$$w \geq 3.2\text{mm}\,(0.125\text{in}) \qquad (14.7.4.7\text{-}1)$$

where:

H_s = horizontal service load on the bearing (N) {kips}

θ_s = maximum service rotation due to total load (rad)

F_y = yield strength of steel (MPa) {ksi}

D_p = internal diameter of pot (mm) {in}

w = height of piston rim (mm) {in}

t_w = pot wall thickness (mm) {in}

The following shall supplement A14.7.4.7.

Piston thickness shall be sufficient to provide clearance between the top of the pot and the sliding surface above as follows:

Metric Units:

- For square pots, clearance = (Design Rotation (radians)) × (0.7 × Pot side (mm)) + 3 ≥ 5 mm

- For round pots, clearance = (Design Rotation (radians)) × (Pot OD (mm)/2) + 3 ≥ 5 mm

U. S. Customary Units:

- For square pots, clearance = (Design Rotation (radians)) × (0.7 × Pot side (in.))+ 0.12 ≥ 0.2 in.

- For round pots, clearance = (Design Rotation (radians)) × (Pot OD (in.)/2) + 0.12 ≥ 0.2 in.

Where the seal is wholly within the piston thickness, pistons for round seals shall have the lower corner chamfered at 45° for a depth equal to 1.7 times the diameter of the seal and 1.2 times the diameter where the seal extends into the elastomer. Refer to Figures 1 and 2.

SPECIFICATIONS COMMENTARY

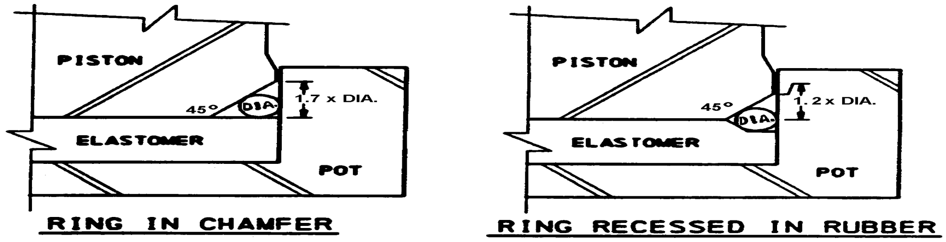

Figure 14.7.4.7-1P - Piston Chamfer for Sealing Rings

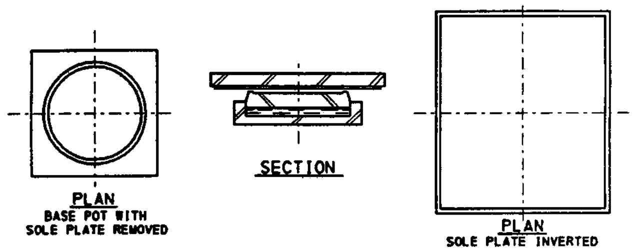

Figure 14.7.4.7-2P - Pot Bearings

14.7.5 Steel Reinforced Elastomeric Bearings - Method B

14.7.5.1 GENERAL

The following shall supplement A14.7.5.1.

The Department has decided to basically use Method A from AASHTO Standard Specifications for Highway Bridges, Section 14.4.1, Fifteenth Edition, 1992, which has been incorporated in D14.7.6. The Department has selected this method because it has successfully been used in the past and it does not require testing of bearings as does the methods given in the LRFD Specifications.

Elastomeric bearings utilized in implementing isolation design shall be designed by the procedures and specifications given in D14.7.6.3.9P and D14.7.6.3.10P. Circular neoprene bearings may be used as multi-rotational bearings on skewed bridges.

C14.7.5.1

Delete the second paragraph of AC14.7.5.1.

14.7.5.2 MATERIAL PROPERTIES

The following shall replace the first paragraph of A14.7.5.2.

The elastomer shall have a shear modulus between 0.60 and 1.3 MPa {0.080 and 0.189 ksi} and a nominal hardness between 50 and 60 on the Shore A scale, and shall conform to the requirements of Publication 408, Section 1107.02(n). Use a nominal hardness of 50 unless approved otherwise.

The following shall replace the first sentence of fourth paragraph of A14.7.5.2.

Bearings shall be made from low temperature elastomer grade 3 unless otherwise approved by the Chief Bridge Engineer.

SPECIFICATIONS

14.7.5.3 DESIGN REQUIREMENTS

14.7.5.3.1 Scope

Delete A14.7.5.3.1.

14.7.5.3.2 Compressive Stress

Delete A14.7.5.3.2.

14.7.5.3.3 Compressive Deflection

The following shall replace the last paragraph of A14.7.5.3.3.

Value for ε_i shall be obtained from design aids based on tests such as presented in Figure AC14.7.5.3.3-1 by testing or by an approved analysis method. Equations for ε_i, given in DC14.7.5.3.3, may also be used. Figure AC14.7.5.3.3-1 is for internal layers of reinforced bearings. It may be used for plain pads or cover layers of reinforced bearings if S is replaced by S/β'.

The following shall supplement A14.7.5.3.3.

The effects of creep of the elastomer shall be added to the instantaneous deflection when considering long-term deflections. They shall be computed from information relevant to the elastomeric compound used if it is available. If not, the material properties given in A14.7.5.2 shall be used.

COMMENTARY

C14.7.5.3.1

Delete AC14.7.5.3.1.

C14.7.5.3.2

Delete AC14.7.5.3.2.

C14.7.5.3.3

Delete the third paragraph of AC14.7.5.3.3.

The following shall supplement AC14.7.5.3.3.

A second order polynominal equation for ε_i was developed for use in the PennDOT bearing design computer program from Figure AC14.7.5.3.3-1 for each shape factor of 3, 4, 5, 6, 9 and 12, and for both 50 and 60 durometer elastomer. The basic equation, for instantaneous compressive strain, is:

$$\varepsilon_i = A\sigma^2 + B\sigma \qquad \text{(C14.7.5.3.3-1)}$$

where:

σ = average compressive load due to the load being investigated (MPa) {ksi}

A = a constant given in Table 1 ($1/MPa^2$) {$1/ksi^2$}

B = a constant given in Table 1 (1/MPa) {1/ksi}

Table C14.7.5.3.3 – Constants for Strain Equation

Metric Units				
Shape Factor	50 Durometer		60 Durometer	
	A	B	A	B
3	-0.379	3.292	-0.154	2.306
4	-0.088	1.784	-0.059	1.479
5	-0.029	1.218	-0.034	1.059
6	-0.015	0.856	-0.019	0.769
9	-0.015	0.754	-0.016	0.667
12	-0.021	0.725	-0.021	0.653

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U.S. Customary Units				
Shape Factor	50 Durometer		60 Durometer	
	A	B	A	B
3	-18.00	22.7	-7.30	15.9
4	-4.20	12.3	-2.80	10.2
5	-1.40	8.4	-1.60	7.3
6	-0.69	5.9	-0.90	5.3
9	-0.70	5.2	-0.78	4.6
12	-1.00	5.0	-0.99	4.5

The PennDOT bearing design computer program will use a minimum shape factor of 3.

The maximum relative compressive deflection should be less than 20 mm {3/4 in.}. The method for relative compressive deflection computations for end joints and pier joints is given below.

Relative deflection at end joints is the sum of creep deflection and live load deflection.

Relative deflection at pier joints shall be the greater of the followings:

(1) The difference of the sum of the dead load deflection, creep deflection and live load deflection on the right side of the joint, and the sum of the dead load deflection and creep deflection on the left side of the joint.

(2) The difference of the sum of the dead load deflection, creep deflection and live load deflection on the left side of the joint, and the sum of dead load deflection and creep deflection on the right side of the joint.

14.7.5.3.4 Shear Deformation

The following shall supplement A14.7.5.3.4.

For expansion bearings, Δ_s shall include longitudinal movement due to end rotation of beams. A procedure for computing longitudinal movement due to end rotation is provided in the LRFD Bearing Pad Program Manual.

For prestressed concrete bridges, the time when bearing fixity occurs in the construction sequence shall be considered. The construction plans shall indicate a construction sequencing for bearing fixity if a special sequencing is assumed in the design.

Longitudinal movement due to end rotation and beam lengthening for all dead loads (except loads to be applied in the future, e.g., future wearing surface and utility loads) may be eliminated from the pad design for permanent conditions provided that the bridge is jacked after dead loads are

C14.7.5.3.4

The following shall replace AC14.7.5.3.4.

The inclusion of applicable shear deformations with the temperature ranges specified in D3.12.2.1 allow contractors to erect beams at expected construction condition temperatures.

Consideration of creep and shrinkage deflection as it was previously computed can be eliminated because its direction of movement is opposite from the longitudinal movement due to end rotation. The magnitude of longitudinal movement due to end rotation will be greater than the previously set value of 5 mm {0.25 in.} for creep and shrinkage and it will thus control.

Longitudinal movement due to end rotation and beam lengthening may result in undesignable or excessively thick pads, especially with steel beam bridges. The requirement

SPECIFICATIONS

applied and the shear deformation in the bearing pads is relieved. The pad should be checked to verify that design criteria is met for the temporary condition prior to release of the dead load shear deformation. A temperature change of 20º C {35º F} shall be used to check for temporary conditions. The construction plans should clearly indicate where and when jacking is required, and provision for jacking points must be included in the design and detailing of the superstructure and substructure. The jacking forces should also be specified.

14.7.5.3.5 Combined Compression and Rotation

Delete A14.7.5.3.5.

14.7.5.3.6 Stability of Elastomeric Bearings

Delete A14.7.5.3.6.

14.7.5.3.7 Reinforcement

The following shall replace A14.7.5.3.7.

Fabric reinforcement shall not be used.

The thickness of the steel reinforcement, h_s, shall satisfy:

- at the service limit state"

Metric Units: $$h_s \geq \frac{h_{avg}(11.7MPa)}{0.55F_y} \qquad (14.7.5.3.7\text{-}1)$$

U.S. Customary Units: $$h_s \geq \frac{h_{avg}(1.7ksi)}{0.55F_y}$$

- at the fatigue limit state:

$$h_s \geq \frac{2h_{avg}\sigma_L}{\Delta F_{TH}} \qquad (14.7.5.3.7\text{-}2)$$

- at PennDOT minimum

$$h_s \geq 3.0\,mm\,\{1/8\,in.\}$$

where:

h_{avg} = the average thickness of the two layers of the elastomer bonded to the reinforcement (mm) {in.}

F_y = yield strength of steel reinforcement

ΔF_{TH} = constant amplitude fatigue threshold for Category A as specified in Table A6.6.1.2.5-3

COMMENTARY

for jacking the bridge will result in additional cost and construction time, but the reduction in the longitudinal movement for pad design may eliminate the need to use a more costly bearing type. The designer should evaluate the cost benefits and construction time effects for individual projects when jacking is proposed.

C14.7.5.3.5

Delete AC14.7.5.3.5.

C14.7.5.3.6

Delete AC14.7.5.3.6.

C14.7.5.3.7

The following shall replace AC14.7.5.3.7.

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(MPa) {ksi}

σ_L = service average compressive stress due to live load (MPa) {ksi}

If holes exist in the reinforcement, the minimum thickness calculated by Equations 1 or 2 shall be increased by a factor equal to twice the gross width divided by the net width.

The thickness of the steel reinforcement that is shown on the contract drawings shall be a thickness that is a manufactured steel plate size.

COMMENTARY

The Department has had problems in the past where a steel reinforcement thickness was used which was slightly less than or greater than the steel reinforcement thickness which was specified on the contract drawings.

14.7.5.3.8 Seismic Provisions

Delete A14.7.5.3.8.

C14.7.5.3.8

Delete AC14.7.5.3.8.

14.7.6 Elastomeric Pads and Steel Reinforced Elastomeric Bearings - Method A

14.7.6.1 GENERAL

The following shall replace A14.7.6.1.

The provisions of this article apply to the design of plain elastomeric pads, PEP and steel reinforced elastomeric bearings.

Pads reinforced with discrete layers of fiberglass, FGP, and pads reinforced with closely spaced layers of cotton duck, CDP, shall not be used in elastomeric bearings. However, these two materials may be used as a bedding material between a pier or abutment concrete top surface and a bearing base plate.

The shape factor for pads and steel reinforced elastomeric bearings covered by this article is determined as specified in A14.7.5.1.

C14.7.6.1

Delete AC14.7.6.1.

14.7.6.2 MATERIAL PROPERTIES

The following shall replace A14.7.6.2.

The material properties shall be as given in A14.7.5.2 and D14.7.5.2.

14.7.6.3 DESIGN REQUIREMENTS

14.7.6.3.1 Scope

The following shall replace A14.7.6.3.1.

Bearings designed by provisions herein do not require any additional testing other than that required in Publication 408.

C14.7.6.3.1

Delete AC14.7.6.3.1.

14.7.6.3.1aP General Size and Construction Requirements

Reinforced elastomeric bearings shall be reinforced

C14.7.6.3.1aP

Shimmed neoprene expansion bearings shall generally

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using integrally bonded steel reinforcement. Fabric reinforcement is not permitted.

Plain pad thickness shall be a minimum of 20 mm {3/4 in.} and a maximum of 30 mm {1 1/4 in.}.

Laminated pads are most efficiently designed by placing shims near the top and bottom extremities of the pads, which are provided with a bonded neoprene cover of 7 mm {1/4 in.}. Inner layer thickness of neoprene may be specified from 10 mm to 15 mm {3/8 in. to 5/8 in.}. Requirements for minimum shim thickness are given in D14.7.5.3.7.

COMMENTARY

not exceed a total thickness of 150 mm {6 in.} The minimum thickness of shimmed neoprene bearings shall generally be not less than 50 mm {2 in.}. This minimum thickness will provide increased rotational tolerance for spans less than 30 000 mm {100 ft.}

A relatively new method to align the shims during the vulcanizing of laminated shim neoprene bearing pads has been developed. Two companies have been approved by the Department to use this method. A brief description of this method is given below.

Instead of using several pins around the perimeter of the pad to hold the shims in place during the pad manufacturing process, a single 40 mm x 15 mm {1.5 in. x 0.5 in.} rounded slot in the center of the pad, oriented longitudinally, may be used to accommodate a steel bar which holds and aligns the shims during vulcanizing.

Instead of exhibiting small slotted indentations around the perimeter of the completed pads, typical of laminated pads using the old manufacturing process, the new pads will show 40 mm x 15 mm {1.5 in. x 0.5 in.} "plugged" rounded slot in the center of the finished bearing.

Laminated shims neoprene bearing pads exhibiting either characteristics, slotted indentations on the perimeter or plugged slotted rounded holes in the center, oriented longitudinally, will be acceptable. The plug, or neoprene extrusion, must be of the same hardness as the pad, must be installed with lubricant adhesive, and must provide a tight fit. There is no allowance for a minus tolerance on the size of the plug.

14.7.6.3.2 Compressive Stress

The following shall replace A14.7.6.3.2.

Unless shear deformation is prevented, use the lower value of shear modulus G when computing the allowable compressive stress. In any elastomeric bearing layer, the average compressive stress at the Service I Limit State shall satisfy:

$$\sigma_c \leq GS/\beta' \qquad (14.7.6.3.2\text{-}1)$$

and

$$\sigma_c \leq 7 \text{ MPa } \{1 \text{ ksi}\} \text{ for steel-reinforced elastomeric bearings} \qquad (14.7.6.3.2\text{-}2)$$

or

$$\sigma_c \leq 5.5 \text{ MPa } \{0.8 \text{ ksi}\} \text{ for plain elastomeric bearings} \qquad (14.6.5.3.2\text{-}3)$$

where:

G = shear modulus of elastomeric pad (MPa) {ksi}

S = shape factor given in A14.7.5.1

C14.7.6.3.2

The following shall replace AC14.7.6.3.2.

From past PennDOT practice and previous design codes, it is desirable to maintain a minimum compressive stress on the elastomeric bearing of 1.5 MPa {0.2 ksi}.

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β' = Modifying factor having a value of 1.0 for internal layers of reinforced bearings, 1.4 for cover layers, and 1.8 for plain pads. If slip is prevented from occurring at the surfaces of plain pads or outer layers reinforced bearings under all circumstances, β' factors smaller than those defined above may be used at the discretion of the Chief Bridge Engineer. β' shall never be taken as less than 1.0.

In bearings containing layers of different thickness, the value of S used shall be that of the thickest layer. Allowable compressive stress may be increased for laminated pads by 10% where shear deformation is prevented.

The minimum average compressive stress under DL only shall satisfy:

$$\sigma_c > 0.7 \text{ MPa } \{0.1 \text{ ksi}\} \qquad (14.7.6.3.2\text{-}4)$$

14.7.6.3.3 Compressive Deflection

The following shall replace A14.7.6.3.3.

The provisions of A14.7.5.3.3 and D14.7.5.3.3 shall apply.

C14.7.6.3.3

Delete AC14.7.6.3.3.

14.7.6.3.4 Shear

The following shall replace A14.7.6.3.4.

The provisions of A14.7.5.3.4 and D14.7.5.3.4 shall apply.

C14.7.6.3.4

Delete AC14.7.6.3.4

14.7.6.3.5 Rotation

The following shall replace A14.7.6.3.5.

The rotational deformations about each axis shall be taken as the maximum possible rotation between the top and bottom of the bearing caused by initial lack of parallelism and girder end rotation. They shall be limited by:

- for rectangular bearings

$$\theta_{sx} \le \frac{2\delta}{L} \qquad (14.7.6.3.5\text{-}1)$$

and

$$\theta_{sz} \le \frac{2\delta}{W} \qquad (14.7.6.3.5\text{-}2)$$

- for circular bearings

$$\sqrt{\theta_{sx}^2 + \theta_{sz}^2} \le \frac{2\delta}{D} s \qquad (14.7.6.3.5\text{-}3)$$

where:

C14.7.6.3.5

The following shall replace AC14.7.6.3.5.

SPECIFICATIONS

L = length of a rectangular bearing (measured along actthe length of the beam) (mm) {in.}

W = width of a rectangular bearing (measured along the width of the beam) (mm) {in.}

D = diameter of a circular bearing (mm) {in.}

δ = instantaneous deflection specified in A14.7.5.3.3 (mm) {in.}

θ_{sx} = maximum rotation between the top and bottom bearing about the transverse axis (RAD)

θ_{sz} = maximum rotation between the top and bottom bearing about the longitudinal axis (RAD)

Live load rotation shall be based on the same load condition used for live load deflection which is given in D2.5.2.6.2 and D3.6.1.3.2.

If the live load deflection is known, the live load rotation may be estimated by:

$$\theta_{LL} = \frac{16}{5}\frac{\delta_{LL}}{L_S} \qquad (14.7.6.3.5\text{-}4)$$

where:

δ_{LL} = live load deflection (mm) {in.}

L_s = span length (mm) {in.}

In lieu of Equation 4, a more exact method may be used to compute live load rotation.

For spans under 30 000 mm {100 ft.}, an additional 0.003 RAD of rotation about the transverse axis of the pad shall be considered for construction tolerance.

For spans equal to or over 30 000 mm {100 ft.}, an additional 0.005 RAD of rotation about the transverse and longitudinal axes of the pad shall be considered for construction tolerance.

The rotational tolerance shall be applied when checking for rotational adequacy under dead load and dead load plus live load conditions. The dead load shall be computed without future wearing surface, utilities, or other non-permanent dead load.

Where the specified construction tolerances result in undesignable pads or excessively thick pads, the construction tolerances may be reduced. For spans under 30 000 mm {100 ft.}, the tolerance about the transverse axis of the pad may be reduced. For spans equal to or greater than 30 000 mm {100 ft.}, the tolerance about the longitudinal axis of the pad may be reduced.

The tolerance values used for the bearing pad design shall be shown on the construction plans.

For steel beam bridges, assess the potential for a lift-off

COMMENTARY

Equation 4 is based on assume parabolic shape which is associated with dead and live load deflections.

Rotational tolerance is added to account for permissible geometric deviations that occur in superstructure and substructure elements during fabrication and construction. The specified design tolerances do not account for the combined fabrication and construction tolerances that are permitted in Publication 408. To do so would result in a large percentage of bridges where neoprene bearing pads would be undesignable.

A parametric study conducted on single span prestressed concrete I-beam, spread box beam, and adjacent box beam bridges resulted in undesignable neoprene pads for some conditions when using the full construction tolerance values. The majority of instances occurred in adjacent box beams.

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condition (or gap between the bottom of beam sole plate and top of the bearing pad) when the beam is initially set on the pad. Lift-off can be expected when the end rotation occurring for the temporary condition exceeds the rotational capacity as computed using Equations 1, 2 and 3 with beam dead load only. If a lift-off condition is expected, provide the following note on the construction plans:

"A gap between the bottom of sole plate and top of neoprene bearing pad is expected to be present at (Abutment 1, Pier 1, etc.) when the beams are placed on the pads. The gap is expected to close after all dead loads are placed on the bridge."

Where a temporary lift-off condition is expected at pier bearings of continuous steel beam bridges, the location of the lift-off (ahead or back stations) should be identified.

14.7.6.3.6 Stability

The following shall replace A14.7.6.3.6.

To ensure bearing stability, the total thickness, H, of the bearing shall be limited by the smallest of:

- for plain rectangular bearings

$$H \leq L/5 \tag{14.7.6.3.6-1}$$

or

$$H \leq W/5 \tag{14.7.6.3.6-2}$$

- for plain circular bearings

$$H \leq D/6 \tag{14.7.6.3.6-3}$$

- for reinforced rectangular bearings

$$H \leq L/3 \tag{14.7.6.3.6-4}$$

or

$$H \leq W/3 \tag{14.7.6.3.6-5}$$

- for reinforced circular bearings

$$H \leq D/4 \tag{14.7.6.3.6-6}$$

- for all bearing types

$$H \leq 150 \text{ mm } \{6 \text{ in.}\} \tag{14.7.6.3.6-7}$$

where:

L = length of a rectangular bearing (mm) {in.}

W = width of a rectangular bearing (mm) {in.}

COMMENTARY

C14.7.6.3.6

Delete AC14.7.6.3.6.

SPECIFICATIONS

D = diameter of circular bearing (mm) {in.}

H = total height of bearing including thickness of shims in reinforced bearing (mm) {in.}

14.7.6.3.7 Reinforcement

The following shall replace A14.7.6.3.7.

Reinforcement for steel-reinforced elastomeric bearings designed in accordance with the provisions of this article shall conform to the requirements of D14.7.5.3.7.

14.7.6.3.8P Bearing Area

14.7.6.3.8aP General

Epoxy-coated bearing surfaces are not to be specified.

The contact surfaces above and below neoprene bearing pads shall be roughened to prevent excessive pad bulging and walking out. Sandblast clean the concrete bearing surface to achieve a rough texture.

14.7.6.3.8bP Prestressed Adjacent Box Beams

See D14.7.6.3.8dP for bearing area and beveled sole plate requirements.

14.7.6.3.8cP Prestressed Spread Box Beams and Prestressed I-Beams

See D14.7.6.3.8dP for bearing area and beveled sole plate requirements.

14.7.6.3.8dP Beam Seat and Bottom of Beam Bearing Area Requirements

Bearing pads require uniform pressure, insofar as practical, over the entire bearing area of the pads to assure long-term, maintenance-free performance. Therefore, it is essential to ensure that the planes formed by the bottom of the beam bearing area and the top of beam seat are parallel unless the bearing pad is specifically designed to accommodate the slope differential. This requirement applies to both the longitudinal and transverse axes of the bearing pad. Refer to Figure 1 for beam and bearing axes orientation.

COMMENTARY

C14.7.6.3.7

Delete AC14.7.6.3.7.

C14.7.6.3.8aP

The Department has experienced instantaneous failure of bearing pads when the bearing surfaces were epoxy-coated. To avoid the high costs of corrective measures, the Department no longer permits epoxy-coated bearing surfaces.

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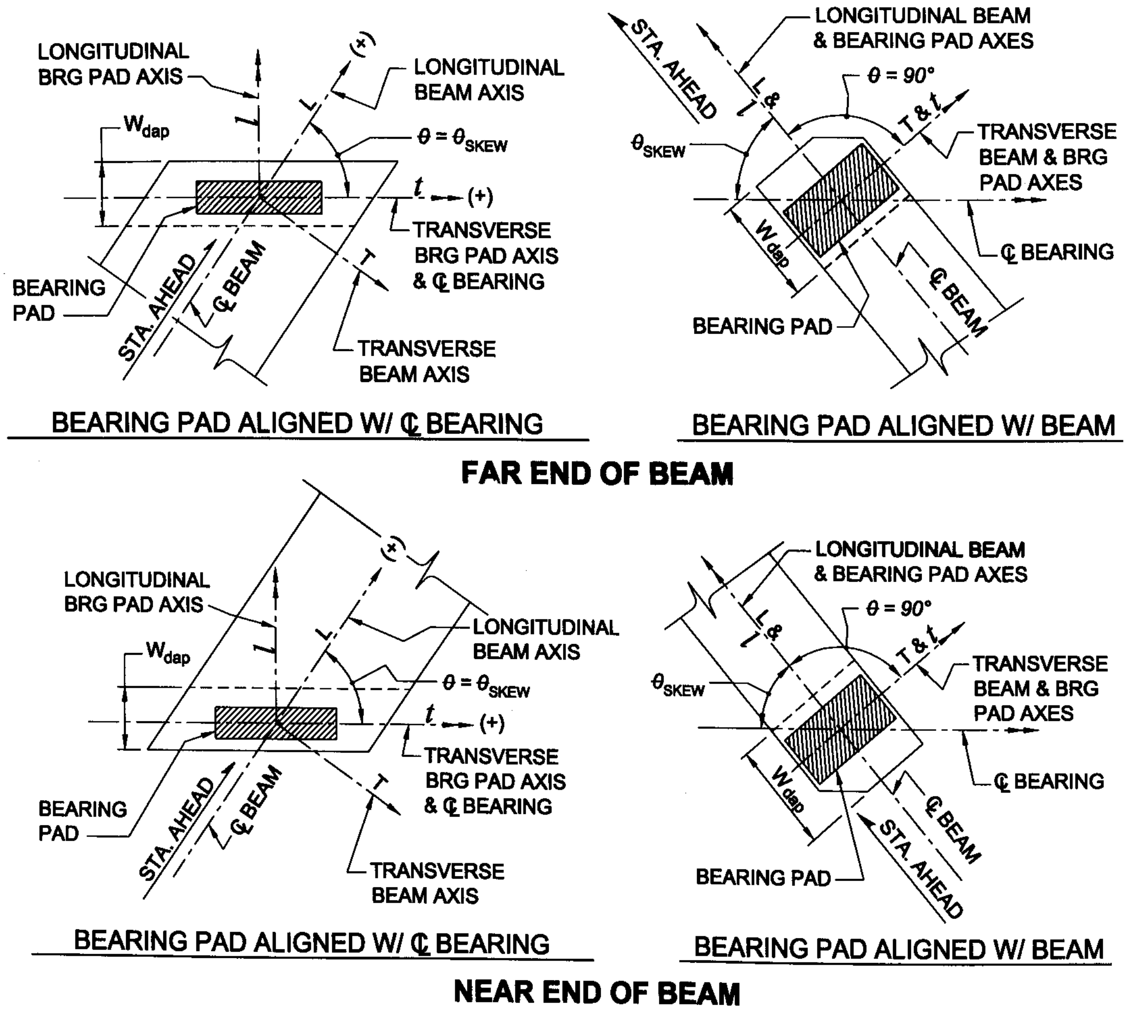

Figure 14.7.6.3.8dP-1 - Beam & Bearing Pad Axes Orientation

14.7.6.3.8d.1P Design of Beam Seat

14.7.6.3.8d.1.1P Direction Parallel to Longitudinal Axis of Beam

The beam seat in the direction parallel to the longitudinal axis of the beam may be constructed to a maximum slope of 1%. Any remaining differential slope between the beam seat and the bottom of beam bearing area must be accommodated for in the design of the bearing pad or by modifying the bottom of beam bearing area in accordance with D14.7.6.3.8d.2.2P.

14.7.6.3.8d.1.2P Direction Parallel to Transverse Axis of Bearing Pad

Establish the beam seat elevations/slopes in the direction parallel to the transverse axis of the bearing pad so that the beam seat is parallel to the bottom of beam. Doing so will ensure parallel top and bottom planes for the bearing

C14.7.6.3.8d.1.2P

Past experience has shown that the beams have a tendency to slide transversely and to distort the bearing pad if not temporarily braced.

pad in this direction. Furthermore, if modification of the bottom of beam bearing area is necessary, properly sloping the beam seat in the direction parallel to the transverse axis of the bearing pad will eliminate the need for two-dimensional sloping of the bottom of beam bearing area. For beams set truly vertical, such as I-beams, the beam seat will be level (see D14.7.6.3.8d.2.3P). There is no maximum slope to which the beam seat may be constructed in this direction. However, for box beams having a transverse beam seat slope, $s_{t,}$ exceeding 5% and placed on neoprene bearing pads thicker than 90 mm {3.5 in.}, providing a note on the design drawings requiring the contractor to provide temporary lateral support to the beam during construction until the end diaphragms are cast and the shear blocks or dowel bars are installed.

14.7.6.3.8d.2P Design of Bottom of Beam Bearing Area

14.7.6.3.8d.2.1P Direction Parallel to Longitudinal Axis of Beam

When the slope of the bottom of beam in the direction parallel to the longitudinal axis of the beam, S_L, cannot be accommodated for in the design of the bearing pad in conjunction with a maximum beam seat slope of 1%, modification of the bottom of beam bearing area is required. Acceptable methods for modifying the bottom of beam bearing area are required. Acceptable methods for modifying the bottom of beam bearing area include:

- For steel beams, provide a beveled steel sole plate;
- For all prestressed concrete beams, cast a beveled notch in the underside of the beam (referred to as dapping) unless the slope exceeds the maximum permitted slope, $(S_L)_{max}$ (see below), in which case provide a beveled steel sole plate. The slope of the beam dap/beveled steel sole plate is to match the actual bottom of the beam slope in the direction parallel to the longitudinal axis of the beam computed in accordance with D14.7.6.3.8d.2.4P.

The maximum slope in the direction parallel to the longitudinal axis of the beam that can be accommodated by dapping is variable and can be expressed as:

$$(S_L)_{max} = [(t_{dap})_{max} - (t_{dap})_{min}] \text{ x } \sin\theta/W_{dap} \quad (14.7.6.3.8d.2.1P\text{-}1)$$

where,

$(t_{dap})_{max}$ = 40mm {1.5 in.} based on a maximum distance from the bottom of the beam to the centerline of the prestressing strand bottom row of 85 mm {3.25 in.} and minimum concrete covers of 40 mm {1.5 in.} on 12.7 mm {1/2 in.}

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diameter prestressing strands and 25 mm {1 in.} on #13 {#4} confinement reinforcement bars in the beam dapping area. Do not increase the maximum dap thickness beyond 40 mm {1.5 in.} regardless of other methods used to increase concrete cover on the confinement reinforcement (such as using draped strand design.)

$(t_{dap})_{min}$ = 6 mm {0.25 in.} based on input from prestressed concrete beam fabrication industry regarding minimum practical thickness that shims used to form daps can be constructed.

W_{dap} = Maximum width of beam dap measured parallel to longitudinal axis of bearing pad.

= $d_{brg} + L_{bp}/2 + d_{clr}$ (Refer to Figures D14.7.6.3.8dP-1 & D14.7.6.3.8d.2.4P-1)

d_{brg} = distance from end of beam to C/L bearing measured parallel to longitudinal axis of bearing pad.

L_{bp} = length of bearing pad measured parallel to longitudinal axis of bearing pad.

d_{clr} = distance from edge of bearing pad to end of beam dap measured parallel to longitudinal axis of bearing pad. The minimum value for d_{clr} = 75 mm {3 in.} to account for construction tolerances of beam length and centerline of bearing location.

θ = Counterclockwise angle formed from the positive (+) transverse bearing pad axis to the positive (+) longitudinal beam axis (refer to Fig. D14.7.6.3.8dP-1).

Partial-width beam daps (see Figure 1) are **not** permitted. The formed notched area for the dap must extend completely to the end of the beam.

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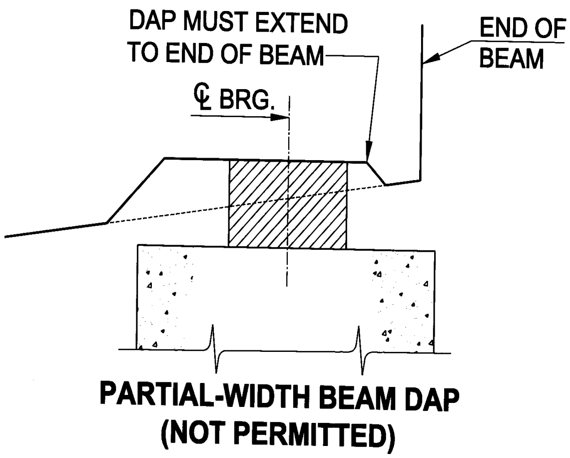

Figure 14.7.6.3.8d.2.1P-1 – Partial-Width Beam Dap

14.7.6.3.8d.2.2P Direction Parallel to Transverse Axis of Bearing Pad

Since the beam seat is to be constructed parallel to the actual slope of the bottom of the beam (see D14.7.6.3.8d.1.2P), there are no special construction requirements for the bottom of the beam bearing area in the direction parallel to the transverse axis of the bearing pad.

14.7.6.3.8d.2.3P Transverse Slope of Beams Relative to Beam Axis

Set the transverse beam slope, S_T, as follows (refer to Figure 1):

Steel and Prestressed Concrete I-beams	Set beams truly vertical in all cases.
Spread Box Beams	Set beams truly vertical or on a slope to conform to the deck cross-slope. When setting on slope, special considerations are required in areas of superelevation transition or within a vertical curve profile with skewed supports. When setting beams vertical, properly consider effects of haunch thickness on beam design and detailing, specifically, the additional weight of concrete and the need for haunch reinforcement.
Adjacent Box/Plank Beams	Set beams to conform as closely as practical to the deck cross-slope in order to minimize the haunch thickness and to align holes for the

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transverse post-tensioning tendons. In areas of superelevation transition or within a vertical curve profile with skewed supports, additional haunch or stepped beam seats may be required.

For box beams, ensure that the transverse beam slope is the same at each end of the beam to prevent inducing torsion.

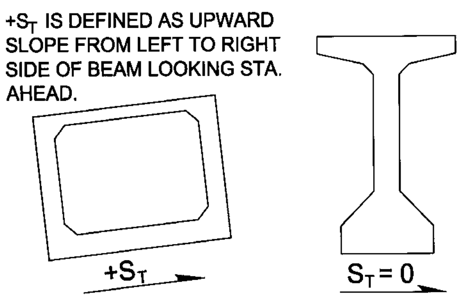

Figure 14.7.6.3.8d.2.3P-1 – Transverse Beam Slope

14.7.6.3.8d.2.4P Computation of Beam Dap Dimensions

Establish the beam dap dimensions to achieve a level bottom of beam bearing area in the direction parallel to the longitudinal axis of the beam. Compute the required beam dap thicknesses as follows (refer to Figure 1):

If S_L is positive (i.e., sloping upward in the stations ahead direction):

$$(t_{dap})_B = (t_{dap})_{min} + S_L \text{ x } W_{dap}/\sin\theta \qquad (14.7.6.3.8d.2.4P\text{-}1)$$

$$(t_{dap})_A = (t_{dap})_{min} \qquad (14.7.6.3.8d.2.4P\text{-}2)$$

If S_L is negative (i.e., sloping downward in the stations ahead direction):

$$(t_{dap})_B = (t_{dap})_{min} \qquad (14.7.6.3.8d.2.4P\text{-}3)$$

$$(t_{dap})_A = (t_{dap})_{min} - S_L \text{ x } W_{dap}/\sin\theta \qquad (14.7.6.3.8d.2.4P\text{-}4)$$

where:

$(t_{dap})_B$ = beam dap thickness at the back station end of the notch;

$(t_{dap})_A$ = beam dap thickness at the ahead station end of the notch;

S_L = bottom of beam slope at the end of the beam under consideration in the longitudinal beam axis direction,
= $(S_L)_G + (S_L)_C$

$(S_L)_G$ = longitudinal slope due to grade of beam,
= $[(E_b)_f - (E_b)_n]/L$

$(E_b)_n$ = Bot. of beam elev. at C/L brg. at near end (m) {ft.}

$(E_b)_f$ = Bot. of beam elev. at C/L brg. at far end (m) {ft.}

L = beam span length, c/c bearing (m) {ft.}

$(S_L)_C$ = longitudinal slope due to beam camber. $(S_L)_C$ is a positive (+) value at the near end of the beam and a negative (-) value at the far end of the beam.
= $4\ y_c/L$

y_c = beam net final camber at mid-span (m) {ft.}

Round dap thicknesses to the nearest 1 mm {1/16 in.}.

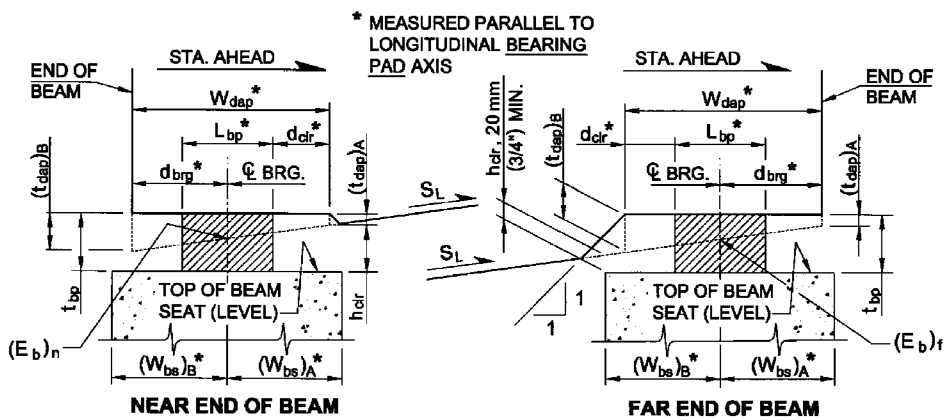

Figure 14.7.6.3.8d.2.4P-1 - Typical P/S Concrete Beam Dap Detail (Viewed Parallel to Longitudinal Beam Axis)

14.7.6.3.8d.2.5P Computation of Beam Seat Elevations when Providing Beam Dap

Establish the beam seat elevations/slope that the beam seat in the transverse direction of the bearing pad is parallel to the bottom of beam bearing area slope. Compute the beam seat elevations as follows (refer to Figure 1):

1) Compute beam dap thickness at C/L bearing:

For the **near end** of beam (back station):

$$(t_{dap})_C=(t_{dap})_B+[(t_{dap})_A-(t_{dap})_B]\mathrm{x}d_{brg}/W_{dap} \qquad (14.7.6.3.8d.2.5P\text{-}1)$$

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For the **far end** of beam (ahead station):

$$(t_{dap})_C=(t_{dap})_A+[(t_{dap})_B-(t_{dap})_A]xd_{brg}/W_{dap} \quad (14.7.6.3.8d.2.5P\text{-}2)$$

2) Beam seat elevation at C/L beam along C/L bearing;

$$(E_{bs})_C=(E_b)_{n\ or\ f}+[(t_{dap})_{cb}-(t_{dap})_{n\ or\ f} \quad (14.7.6.3.8d.2.5P\text{-}3)$$

where:

$(t_{bp})_{n\ or\ f}$ = thickness of bearing pad at the near/far end of beam.

3) Beam seat elevations at the four corners of the beam seat can be computed as follows:

$$(E_{bs})_{BR}=(E_b)_c+S_t\ (L_{bs}/2)-S_1(W_{bs})_B \quad (14.7.6.3.8d.2.5P\text{-}4)$$

$$(E_{bs})_{BL}=(E_b)_c-S_t\ (L_{bs}/2)-S_1(W_{bs})_B \quad (14.7.6.3.8d.2.5P\text{-}5)$$

$$(E_{bs})_{AR}=(E_b)_c+S_t\ (L_{bs}/2)+S_1(W_{bs})_A \quad (14.7.6.3.8d.2.5P\text{-}6)$$

$$(E_{bs})_{AL}=(E_b)_c-S_t\ (L_{bs}/2)+S_1(W_{bs})_A \quad (14.7.6.3.8d.2.5P\text{-}7)$$

where:

L_{bs} = Length of beam seat assumed centered about C/L beam and assumed rectangular in shape (plan view).

$(E_{bs})_{BR}$ = Beam seat elevation at right, back station corner.

$(E_{bs})_{BL}$ = Beam seat elevation at left, back station corner.

$(E_{bs})_{AR}$ = Beam seat elevation at right, ahead station corner.

$(E_{bs})_{AL}$ = Beam seat elevation at left, ahead station corner.

$(W_{bs})_A$ = Width of beam seat from C/L bearing to ahead station face/edge.

$(W_{bs})_B$ = Width of beam seat from C/L bearing to back station face/edge.

s_t = Bottom of beam slope in the transverse bearing pad axis direction,

= $S_L \cos\theta + S_T \sin\theta$

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S_T = Bottom of beam slope in transverse beam axis direction,
= 0 for beams set vertically (steel and prestressed concrete I-beams), Value is positive (+) if beam slopes upward from left-to-right looking stations ahead.

s_l = Bottom of beam slope within beam dap area in the longitudinal bearing pad axis direction,
= $-s_t/\tan\theta$

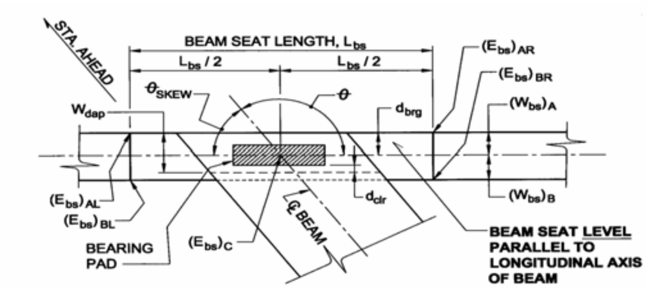

Figure 14.7.6.3.8d.2.5P-1 – Typical Beam Seat Detail (Far End of Beam Shown)

Note: For the case in which the bearing pads are aligned relative to the beam and not the C/L bearing (typically, steel & prestressed I-beams), the beam seat is to be constructed LEVEL in both the longitudinal and transverse directions. Therefore, only the beam seat elevation at C/L beam must be computed.

14.7.6.3.8d.2.6P Additional Box Beams Requirements When Dapping

When dapping is used on spread and adjacent box beams, ensure that a minimum concrete cover of 32 mm {1.25 in.} is provided on the prestressing strands adjacent to the inner void. Increase the thickness of the bottom flange in 13 mm {1/2 in.} minimum increments to meet this requirement.

14.7.6.3.8d.2.7P Minimum Clearance for Inspectability

Provide 20 mm {3/4 in.} minimum clearance from the underside of beam (without consideration of the beam dap) to the top of beam seat to ensure inspectability of the pad. Increase the thickness of the bearing pad in order to satisfy this requirement.

The actual clearance provided, h_{clr}, can be computed from the following equations:

For the **near end** of beam (back station):

If S_L is positive,

$$h_{clr} = t_{bp} - (t_{dap})_A \quad (14.7.6.3.8d.2.7P\text{-}1)$$

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If S_L is negative,

$h_{clr} = t_{bp} - (t_{dap})_C + (W_{bs})_A \times S_L/|\sin\theta_{SKEW}|$ (14.7.6.3.8d.2.7P-2)

For the **far end** of beam (ahead station):

If SL is positive,

$h_{clr} = t_{bp} - (t_{dap})_C - (W_{bs})_B \times S_L/|\sin\theta_{SKEW}|$ (14.7.6.3.8d.2.7P-3)

If SL is negative,

$h_{clr} = t_{bp} - (t_{dap})_B$ (14.7.6.3.8d.2.7P-4)

14.7.6.3.9P Shear Strain Components for Isolation Design

The various components of shear strain in the bearing are computed as follows:

(a) Shear strain, ε_{sc}, due to compression by vertical loads is given by

$$\varepsilon_{sc} = 6S\varepsilon_c \quad (14.7.6.3.9P\text{-}1)$$

For S ≥ 3

$$\varepsilon_C = \frac{\Delta_{ci}}{\sum h_{ri}} = \frac{\Delta_c}{h_{rt}} = \left[\frac{P}{A_r\left(5GS^2\right)}\right] \quad (14.7.6.3.9P\text{-}2)$$

For S < 3

$$\varepsilon_c = \frac{\Delta_{ci}}{\sum h_{ri}} = \frac{\Delta_c}{h_{rt}} = \left[\frac{P}{A_r E_c\left(1+2kS^2\right)}\right] \quad (14.7.6.3.9P\text{-}3)$$

where:

$$A_r = A_b\left(1-\Delta/B\right)$$

For most practical cases S ≥ 3

The effects of creep of the elastomer shall be added to the instantaneous compressive deflection, Δ_c, when considering long-term deflections. For compressive deflection requirements see D14.7.6.3.3. They shall be computed from information relevant to the elastomer compound used, if it is available. If not, the material properties given in A14.7.5.2 shall be used as a guide.

(b) Shear strain, ε_{sh}, due to imposed lateral displacement is given by

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C14.7.6.3.9P

The shear strain, ε_{sc}, due to compression by vertical loads is directly dependent on the compression strain, ε_c, due to the vertical loads. In computing ε_c, an apparent compressive modulus is used such that

$$\varepsilon_c = \frac{P}{A_r E_c} \quad (C14.7.6.3.9P\text{-}1)$$

For S ≥ 3

$$E_c = 5GS^2 \quad (C14.7.6.3.9P\text{-}2)$$

in which the elastomer shear modulus, G, depends on the specific bearing material with a range of 0.7-1.4 MPa {0.1-0.2 ksi}. For the rare case in which S < 3,

$$E_c = E\left(1+2kS^2\right) \quad (C14.7.6.3.9P\text{-}3)$$

in which E_c is the modulus of elasticity of the elastomer and k is a material property. The British Specification BE 1/76 (1) provides guidance for the selection of appropriate values for E and k.

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$$\varepsilon_{sh} = \frac{\Delta_s}{h_{rt}} \quad (14.7.6.3.9P\text{-}4)$$

(c) Shear strain, ε_{eq}, due to earthquake-imposed displacement is given by

$$\varepsilon_{eq} = \frac{d_i}{h_{rt}} \quad (14.7.6.3.9P\text{-}5)$$

(d) Shear strain, ε_{sr}, due to rotation is given by

$$\varepsilon_{sr} = \frac{B^2\theta}{(2h_{ri}h_{rt})} \quad (14.7.6.3.9P\text{-}6)$$

14.7.6.3.10P Limiting Criteria for Allowable Vertical Loads

The allowable vertical load on an elastomeric isolation bearing is not specified explicitly. The limits on vertical load are governed indirectly by limitations on the equivalent shear strain in the rubber due to different load combinations and to stability requirements. The permissible shear strain in the rubber is expressed as ϕ times the minimum specified elongation-at-break (ε_u). The value of ϕ is dependent on the load combination under consideration with specific values given in the following sections.

14.7.6.3.10aP Service Load Combinations

The following criteria shall be satisfied for service loads, which include dead load plus live load, thermal, creep, shrinkage and rotation:

(a) $0.5\varepsilon_u \geq \varepsilon_{sc} + \varepsilon_{sh} + \varepsilon_{sr}$ (14.7.6.3.10aP-1)

(b) $0.33\varepsilon_u \geq \varepsilon_{sc}$ (14.7.6.3.10aP-2)

The smaller load factors for TU, CR and SH should be used per D3.4.1.

14.7.6.3.10bP Seismic Load Combinations

The following criterion shall be satisfied for seismic loads which include dead load and seismic live load, seismic design displacements and rotation:

$$0.75\varepsilon_u \geq \varepsilon_{sc} + \varepsilon_{eq} + \varepsilon_{sr} \quad (14.7.6.3.10bP\text{-}1)$$

COMMENTARY

C14.7.6.3.10P

Since the primary design parameter for earthquake loading is the displacement, d_i, of the bearing, the design procedures shall be capable of incorporating this displacement in a logical and consistent manner. The requirements of D14.7.6.3 limit vertical loads by using a limiting compressive stress. Therefore, they do not have a mechanism for including the simultaneous effects of seismic displacements. The British Specification BE 1/76 (1976), and its more recent successor BS 5400 (1981), recognize that shear strains are induced in reinforced bearings by compression, shear deformation, and rotation. In these codes, the sum of these shear strains is limited to a proportion of the elongation-at-break of the rubber. The proportion (1/2 or 1/3 for service load combinations and 3/4 for seismic load combinations) is a function of the loading type.

SPECIFICATIONS

If thermal displacements are large, ε_{sh} shall be included on the right-hand side of the above inequality.

14.7.6.3.10cP Stability Against Overturning

Elastomeric isolation bearings shall be shown either by test or analysis to the capable of resisting the dead load, seismic live load, and any vertical load resulting from overturning at 1.5 times the seismic design displacement, d_i.

14.7.6.4 ANCHORAGE

The following shall replace A14.7.6.4.

If the factored shear force sustained by the deformed pad at the service limit state exceeds one-fifth of the compressive force, P_{sd}, due to permanent loads, the pad shall be secured against horizontal movement.

When the effects of live load are included in the factored shear force in the above requirements, the corresponding reaction from the live load may be included in the compressive force when checking the one-fifth limitation.

For spans less than 15 000 mm {50 ft.}, when the above requirements are not satisfied, holes for dowels in the bearings will be permitted to secure pads against displacement. The effect of holes must be accounted for in the design.

Securing pads against displacement by the use of adhesive is not permitted.

These provisions also apply to steel-reinforced elastomeric bearings designed using Method B

COMMENTARY

C14.7.6.4

The DM-4 is using service loads for anchorage check because the temperature range for the design of bearings in the DM-4 takes into account possible variations in setting temperature.

When checking the one-fifth anchorage limitation with live load effects, the live load reaction may be computed as:

Simple Spans: One-half the total live load force effect as specified in D2.5.2.6.2.

Continuous Spans and Simple Spans made Continuous: The determination of live load reaction concomitant with longitudinal movement in the bearing due to live load is a difficult task. As such, downward live load reaction should normally be neglected when checking anchorage requirements for continuous spans. Uplift reactions should be considered.

SPECIFICATIONS

14.7.6.5P DRAWING REQUIREMENTS

The contract drawings shall provide the following information:

1. Bearing pad length, width, thickness, edge cover, and layer thickness.
2. Total number of pads required for the project and number required for the testing.
3. Number of shims and shim thickness.
4. Project Specific Notes.
5. As applicable, sole plate sizes, material, finish, and type of coating.
6. Bearing seat or sole plate slope.
7. As applicable, anchor bolt sizes, lengths, and material requirements.
8. Reference to BC-755M.

COMMENTARY

C14.7.6.5P

The inclusion of this information on the contract drawings in conjunction with material and fabrication tolerance requirements specified in Publication 408 eliminate the need for shop drawings for elastomeric bearing pads.

SPECIFICATIONS

14.7.7 Bronze or Copper Alloy Sliding Surfaces

14.7.7.1 MATERIALS

The following shall supplement A14.7.7.1.

SPECIFICATIONS COMMENTARY

For bronze/carbon steel sliding surfaces, the surface finish shall not be more than 3.2 µm rms {125 µin.}.

14.7.7.5P ROTATIONAL ELEMENTS - CONCAVE - SPHERICAL

The spherical radius shall be determined such that the resulting geometry of the bearing is capable of withstanding the greatest ratio of horizontal force to vertical load under all loading conditions to prevent unseating the concave element.

If required, mechanical safety restraints shall be considered to prevent overturning.

Maximum design rotation of the structure, which includes a 0.02 radian construction tolerance, shall be considered in the bearing design to prevent overturning or uplift.

Calculations showing the determination of the radius shall be submitted for approval.

The bearing surfaces shall have lubricant recesses consisting of concentric rings with or without central circular recesses with a depth at least equal to the width of the rings or recesses.

The recesses or rings shall be arranged in a geometric pattern so that adjacent rows overlap in the direction of motion.

The entire area of all bearing surfaces that have provision for relative motion shall be lubricated by means of the lubricant-filled recesses.

The lubricant-filled areas shall comprise not less than 25% of the total bearing surface.

The lubricant compound shall be integrally molded at high pressure and compressed into the rings or recesses and project no less than 0.25 mm {0.01 in.} above the surrounding bronze plate.

14.7.8 Disc Bearings

14.7.8.1 GENERAL

The following shall supplement A14.7.8.1.

PADOT DWG #97-601-BQAD Standard Disc Bearing Details shall be used for disc bearings with vertical loads between 100 kips and 1000 kips with 0.02 radians maximum rotation. If the standard is not used, complete design calculations must be provided prior to submission of shop drawings on all projects for all aspects of disc bearings.

For high load multi-rotational bearing design plan presentation, see D14.6.1.2.1P.

14.7.8.3 ELASTOMERIC DISC

The following shall supplement A14.7.8.3.

The area of a disc shall be designed for a maximum nominal resistance of 25 MPa {3.7 ksi} for Polyether Urethane Compound A and 35 MPa {5 ksi} for Polyether

SPECIFICATIONS COMMENTARY

Urethane Compound B at the total dead and live loads of the structure.

14.7.8.5 STEEL PLATES

The following shall supplement A14.7.8.5.

A limiting ring shall be provided by a welded ring or by machining a recess in the bearing plate. The depth of the limiting ring shall be equal to or greater than ID X 0.014. The inside diameter of the retainer ring shall be greater than the diameter of the disc by 4% to 6% of the diameter.

Replace second paragraph of A14.7.8.5 with the following:

The section thickness of the plate beneath the disc where not limited by bending stresses shall not be less than

- Disc OD X 0.06 for bearing directly on concrete, but not less than 19 mm {3/4 in.}

- Disc OD X 0.045 for bearing directly on steel masonry plates, but not less than 12.5 mm {1/2 in.}

Bearing plates shall be connected to masonry plates by means of a fillet weld around the entire perimeter. Full clearance shall be maintained between the bearing parts for the following condition:

Design Rotation (radians) + vertical deflection

Refer to Figure 1 for disc bearings.

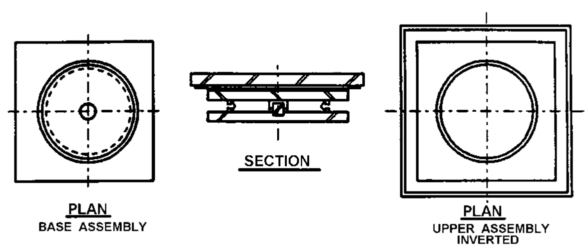


Figure 14.7.8.5-1P - Disc Bearings

14.7.9 Guides and Restraints

The following shall supplement A14.7.9.

Guided members shall have their contact area within the guide bars in all operating positions. Guiding off the fixed base or any extensions of it is not permitted. Do not use inner key to resist horizontal forces. Instead, use guide bars.

Alignment of bearing guiding systems relative to the anticipated movement direction of the structure shall be carefully considered to avoid bearing guide system failure. Special studies or designs may be required on curved or skewed structured to ensure correct alignment.

SPECIFICATIONS COMMENTARY

14.7.9.1 GENERAL

The following shall supplement A14.7.9.1.

Guide bars may be integral by machining from the solid or by welding or connecting with high-strength fasteners. High-strength fasteners shall be designed using .25 X Ultimate Shear Strength.

PTFE used on guide bars shall be pigmented.

14.7.9.4 GEOMETRIC REQUIREMENTS

The following shall supplement A14.7.9.4.

Total spacing should not be specified more than 1.6 mm {1/16 in.} between guides and guided components where possible (see A14.7.9).

Clearance between bearing guides or keys and guided members shall be 1.5 mm {1/16 in.}.

14.7.9.6 ATTACHMENT OF LOW-FRICTION MATERIAL

The following shall supplement A14.7.9.6.

PTFE must be used on guides and shall be bonded to and recessed in their substrate. The PTFE shall also be mechanically fastened by a minimum of two screws. The screws shall be recessed a minimum of 50% of the amount of protrusion of the PTFE above the guiding surface.

14.8 LOAD PLATES AND ANCHORAGE FOR BEARINGS

14.8.1 Plates for Load Distribution

The following shall supplement A14.8.1.

The stainless steel plate mounted on sole plates to permit full expansion and contraction for pot or disc bearings should be extended to within 6 mm {1/4 in.} of the edge of the sole plate. This will minimize a need for costly field adjustment in case of design or construction error.

14.8.2 Tapered Plates

The following shall supplement A14.8.2.

Sole plate above the bearings shall be tapered as needed so that the distance between the top of the pot and the sliding surface does not deviate from general uniformity by more than 3 mm {1/8 in.} under all dead loads.

14.8.3 Anchorage and Anchor Bolts

14.8.3.1 GENERAL

The following shall replace paragraphs three and four of A14.8.3.1.

Trusses, girders and rolled beams shall be securely

SPECIFICATIONS COMMENTARY

anchored to the substructure. Preferably, anchor bolts shall be cast in substructure concrete. Anchor bolts may be grouted in place. With the approval of the Chief Bridge Engineer, anchor bolts can be swedged or threaded to secure a satisfactory grip upon the material to embed them in the holes. Dowels shall be cast in the substructure concrete.

The factored tensile resistance of the anchor bolts shall be greater than the factored tensile force due to Strength I or II limit state.

The following shall supplement A14.8.3.1.

The factored shear resistance of anchor bolts and dowels shall be greater than the factored tensile horizontal or transverse force due to the Strength I or II limit state and all applicable extreme event load combinations.

The embedment length of the dowel or anchor bolt in shear may need to be further increased due to the bearing resistance of the concrete.

14.8.3.3P ANCHORAGE BOLTS

For each bearing, Table 1 provides minimum anchorage bolt requirements.

Table 14.8.3.3P-1 - Minimum Anchorage Bolt Requirements

Metric Units				
Type	Span Range (L) (mm)	No. of Bolts	Diameter of Bolts (mm)	Minimum Embedment Length (mm)
Rolled Beams	All	2	25.4	260
Girders and Trusses	$L \leq 15\,000$	2	25.4	260
Girders and Trusses	$15\,001 < L \leq 30\,000$	2	31.8	310
Girders and Trusses	$30\,001 < L \leq 45\,000$	2	38.1	380
Girders and Trusses	$45\,001 < L$	4	38.1	380

U.S. Customary Units				
Type	Span Range (L) (ft.)	No. of Bolts	Diameter of Bolts (in.)	Minimum Embedment Length (in.)
Rolled Beams	All	2	1	10
Girders and Trusses	L ≤ 50	2	1	10
Girders and Trusses	50 < L ≤ 100	2	1 1/4	12
Girders and Trusses	100 < L ≤ 150	2	1 1/2	15
Girders and Trusses	150 < L	4	1 1/2	15

14.8.3.4P DOWELS

For prestressed concrete girders when anchorage bolts are not used, dowels can be used to prevent longitudinal and transverse movements. Standard Drawings BD-656M, BD-664M and BD-665M provide minimum requirements for dowels.

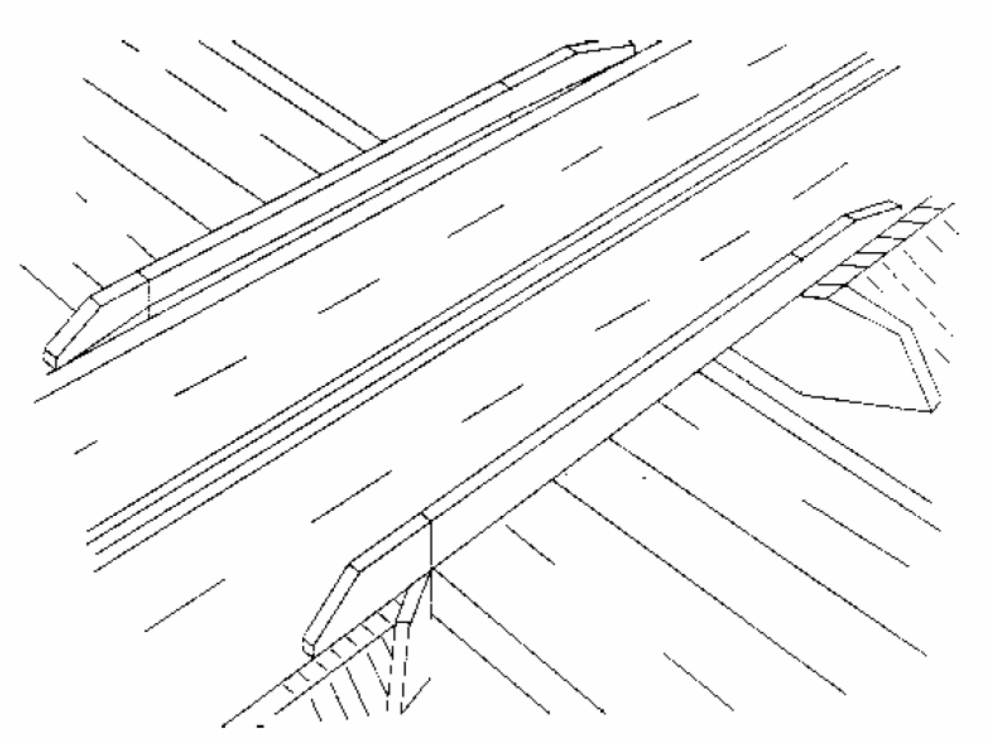
REFERENCES

British Standards Institution, Draft British Standard BD 5400: Steel, Concrete and Composite Bridges: Part 9A: Code of Practice for Design of Bearings, and Part B: Specifications for Materials, Manufacture and Installation of Bearings, Document 81/10/84, 1981

"Design Requirements for Elastomeric Bridge Bearings", Technical Memorandum BE 1/76, Highways Directorate, Department of Environment, Great Britain, 1976

DESIGN MANUAL
PART 4

Volume 2 of 2 Volumes



STRUCTURES

PROCEDURES – DESIGN – PLANS PRESENTATION
PDT – PUB No. 15M

SEPTEMBER 2007 EDITION
(Dual Unit)

COMMONWEALTH OF PENNSYLVANIA
DEPARTMENT OF TRANSPORTATION

PENNDOT

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 2

APPENDIX A - QUALITY ASSURANCE FORMS FOR BRIDGE DESIGN

TABLE OF CONTENTS

1.0 GENERAL

The intent of these forms is to self-correct critical omissions and errors during the design of bridge components. All applicable forms will be filled out by the actual bridge designer, either in-house or consultant, and will be included with the appropriate submission, as given below, to the reviewing office, either District or Central Office. It is anticipated that it will only require approximately 15 minutes to complete each form, and as such, should not pose any undue or additional burden on the designer. Thus, these forms shall be completed at no additional cost to the Department.

Form D-500, Engineering Agreements, must be completed for each structure and included as a part of the price proposal for an Engineering Agreement or as a part of the Work Order under an Open-End Contract. This form should be incorporated in an appendix along with the District's Scope of Work.

Form D-501 through D-504 must be completed, as appropriate, and included as a part of the TS&L submission.

Form D-505, Foundations, must be completed and included as part of the Foundation submission.

Forms D-506 through D-518 must be completed, as appropriate, and included as part of the Final Design submission. Form D-513, BRADD-3 Final Plans, must be completed and included with all BRADD-3 designed projects, either in-house or consultant, along with all the other pertinent forms required with the Final Design submission.

Any submission received by the approving authority, either District or Central Office, without all the completed applicable forms attached will be returned without any action taken.

These forms also apply to all submissions developed by the contractor for alternate designs in accordance with PP1.10.1.

2.0 QUALITY ASSURANCE FORMS

A copy of each form is given on the following pages of this appendix.

D-500 (1-96)
REPRODUCE LOCALLY

SHEET NO. 1 OF 5

QUALITY ASSURANCE FORM FOR ENGINEERING AGREEMENTS

(Note: Provide one form for each structure included under this technical proposal or work order)

Designer: ______________________________ Date: ___________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units: ☐ Metric ☐ U. S. Customary

1. PROJECT INFORMATION

County: ____________ S.R.: ________ Sec.: ________ Over: ______________
(STREAM, RAILROAD, OR ROAD)

Structure Identification: ______________________________

2. PROJECT TYPE

☐ Rehabilitation ☐ Replacement ☐ Bridge at New Location

3. PROJECT STAGE

☐ Preliminary Design ☐ Final Design ☐ Construction

☐ Other/Specify: ______________________________

4. PRELIMINARY DESIGN (Key Items Only)

(Check Applicable Box)

	Yes	No	N/A
A. Hydraulics			
Is Backwater Study Addressed?	☐	☐	☐
B. New Bridge			
Alternate Location and Bridge Type Study: Preliminary Geotechnical Investigation Required?	☐	☐	☐
Other: ______________________	☐	☐	☐

Comments ______________________________

____

C. Bridge Rehabilitation

Bridge Type and Spans: ______________________________

Are the Following Items Included?	Yes	No	N/A
Current Inspection Report Study to be Used	☐	☐	☐
Detailed Inspection Report Part of Agreement	☐	☐	☐
Fatigue Evaluation	☐	☐	☐

D-500 (1-96) SHEET NO. 2 OF 5

QUALITY ASSURANCE FORM FOR ENGINEERING AGREEMENTS

(Note: Provide one form for each structure included under this technical proposal or work order)

Designer: ______________________________ Date: ____________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units: ☐ Metric ☐ U. S. Customary

	(Check Applicable Box) Yes	No	N/A
Scour Evaluation	☐	☐	☐
Seismic Evaluation	☐	☐	☐
Preliminary Rehabilitation Alternate Study	☐	☐	☐
Fracture Criticality	☐	☐	☐
Bridge Paint Evaluation	☐	☐	☐
Other: ______________________	☐	☐	☐

Comments ______________________________

____

5. FINAL DESIGN

	Yes	No	N/A
Will BRADD-3 be Used?	☐	☐	☐
BRADD-3 Input will be Prepared and Output Reviewed?	☐	☐	☐

A. T.S.&L. Stage

Will the Following Key Items be Studied?

	Yes	No	N/A
Fatigue	☐	☐	☐
Scour	☐	☐	☐
Seismic	☐	☐	☐
Bearings	☐	☐	☐
Load Capacity Study	☐	☐	☐
Drainage (Superstructure & Substructure)	☐	☐	☐
Deck Type	☐	☐	☐
Elimination of Expansion Joints	☐	☐	☐
Structural Stability/Constructability	☐	☐	☐
Structure Inspectability and Maintainability	☐	☐	☐
Approach Slab and Pavement Relief Joints	☐	☐	☐

D-500 (1-96) SHEET NO. 3 OF 5

QUALITY ASSURANCE FORM FOR ENGINEERING AGREEMENTS

(Note: Provide one form for each structure included under this technical proposal or work order)

Designer: ________________________________ Date: ___________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units: ☐ Metric ☐ U. S. Customary

	(Check Applicable Box) Yes	No	N/A
Maintenance & Protection of Traffic	☐	☐	☐
Construction Sequencing of All Operations	☐	☐	☐

Analysis Method Used: ☐ PennDOT Approx. ☐ 2D ☐ 3D

Number of Alternates to be Studied: ☐ Three ☐ Four or More

Number of Spans (Existing) or (Anticipated) ____________________

	Yes	No	N/A
Other: ____________________	☐	☐	☐

Comments ____

____

B. Foundation

	Yes	No	N/A
Geotechnical Investigation	☐	☐	☐
Foundation Design	☐	☐	☐
Scour	☐	☐	☐
Other: ____________________	☐	☐	☐

Comments ____

____

C. Final Plans

Will the Following Key Items be Addressed, Designed and Detailed?

	Yes	No	N/A
Fatigue	☐	☐	☐
Seismic	☐	☐	☐
Bearings	☐	☐	☐
Deck Design	☐	☐	☐
Beam Design	☐	☐	☐
Substructure Design	☐	☐	☐
Drainage	☐	☐	☐

D-500 (1-96) SHEET NO. 4 OF 5

QUALITY ASSURANCE FORM FOR ENGINEERING AGREEMENTS

(Note: Provide one form for each structure included under this technical proposal or work order)

Designer: ______________________________ Date: __________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units: ☐ Metric ☐ U. S. Customary

	(Check Applicable Box) Yes	No	N/A
Continuity Design	☐	☐	☐
Jacking of Superstructure	☐	☐	☐
Concrete and Crack Repair	☐	☐	☐
Structural Stability/Constructability	☐	☐	☐
Structure Inspectability and Maintainability	☐	☐	☐
Approach Slab and Pavement Relief Joints	☐	☐	☐
Maintenance & Protection of Traffic	☐	☐	☐
Construction Sequencing of All Operations	☐	☐	☐
Other: ______________________________	☐	☐	☐

Comments ______________________________

____

NOTE: These are not all-inclusive lists. Refer to applicable Design Manuals or additional items.

D-500 (1-96) SHEET NO. 5 OF 5

QUALITY ASSURANCE FORM FOR ENGINEERING AGREEMENTS

(Note: Provide one form for each structure included under this technical proposal or work order)

Designer: ______________________________ Date: __________

(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

1. PROJECT INFORMATION

County: ______________ S.R.: __________ Sec.: __________ Over: ______________________

(STREAM, RAILROAD, OR ROAD)

Structure Identification: ______________________________

ENGINEERING MAN-HOURS ESTIMATE SUMMARY

	PROJECT PRINCIPAL	PROJECT MANAGER	PROJECT ENGINEER	SENIOR ENGINEER	• _______ ENGINEER	ENGINEER TECHNICIAN	DRAFT	CLERICAL	TOTAL
1. Prelim. Design									
H&H Report (If Applicable)									
Atl. Loc. & Br. Type Study									
2. Final Design									
TS&L									
Foundation									
Final Plan									
Page Number in the proposal where all bridge sheets with titles are listed: ______________									
3. PS&E									
4. Fatigue Analysis									
5. Seismic Analysis									
6. Load Capacity Analysis									
7. Other Special Item: __________ __________									
TOTAL									

NOTE: Some classifications may not be applicable for each task.

D-501 (1-96)
REPRODUCE LOCALLY

QUALITY ASSURANCE FORM FOR T.S.&L. OF STEEL BRIDGES

(New Structures)

Designer: ______________________________ Date: __________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

1. PROJECT INFORMATION

County: ______________ S.R.: ________ Sec.: ________ Over: ______________
(STREAM, RAILROAD, OR ROAD)

S-No.: __________ Design ADT: __________ ADTT: __________ Year: __________

2. BRIDGE GEOMETRY

Horizontal Geometry: ______________________________

Width: Out-Out __________; Curb-Curb ______________; Sidewalk ______________

Skew: __________ (if < 70º, why?): ______________________________

Minimum Clearance: Vertical → Provided = __________; Required = __________

Horizontal → Provided = __________; Required = __________

3. SUPERSTRUCTURE

Type: ______________________________

Design Methodology: ______________________________

Analysis Methodology (DM-4 Sec. 4): ______________________________

Fatigue Design Roadway Class: ______________________________

Fracture-Critical Elements: ______________________________

No. of Spans: ______________ Span Length(s): ______________

No. of Beams: ______________ (if > 4, why?): ______________

____

Straight, Curved, or Kinked: ______________________________

Beam Spacing: ______________ Max. Overhang: ______________

Web Size: ______________ Slab Thickness: ______________

BEARINGS	ABUT. 1	ABUT. 2	PIER 1	PIER 2	PIER 3	PIER 4
Function:	______	______	______	______	______	______
Type:	______	______	______	______	______	______

D-501 (1-96) SHEET NO. 2 OF 4

QUALITY ASSURANCE FORM FOR T.S.&L. OF STEEL BRIDGES
(New Structures)

Designer: ______________________________ Date: ___________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

DECK JOINTS	LOCATION	TYPE	MOVEMENT CLASSIFICATION
No. ______	______	______	______
______	______	______	______
______	______	______	______

(if > 2, why?): ______________________________

4. SUBSTRUCTURE

UNIT	ABUT. 1	ABUT. 2	PIER 1	PIER 2	PIER 3	PIER 4
Type:	____	____	____	____	____	____
Support:	____	____	____	____	____	____
Geology:	____	____	____	____	____	____
# Borings:	____	____	____	____	____	____

(**Note**: Type = Structure Type; Support = Shallow or Deep Foundation; Geology = Type of Geological Formation)

5. MISCELLANEOUS

Drainage: On Structure Number: ______________ and Type ______________

Off Structure Number: ______________ and Type ______________

Protective Fence: Yes ☐ No ☐

Navigation Lights: Yes ☐ No ☐

Inspection Facilities: Yes ☐, and Type ______________; or No ☐

Utilities: ______________________________

Bridge-Mounted Sign Structures: Yes ☐ No ☐

D-501 (1-96) SHEET NO. 3 OF 4

QUALITY ASSURANCE FORM FOR T.S.&L. OF STEEL BRIDGES
(New Structures)

Designer: ______________________________ Date: __________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

Bridge-Mounted Lighting: Yes ☐ No ☐

Was High Mast or Pier-Mounted Considered? Yes ☐ No ☐ If no, why? ____________

6. ALTERNATES SUMMARY

Is this a Streamlined submission? Yes ☐ No ☐

If yes, how many Alternates are required? 1 ☐ 2 ☐

	ALTERNATE #1	ALTERNATE #2	ALTERNATE #3	ALTERNATE #4
Description:	________	________	________	________
Total Cost:	________	________	________	________
Calculation Page:	________	________	________	________

Permitted Alternates by Contractors:

Prestressed Concrete ☐ Steel ☐ Other ☐

If other, specify: ______________________________

7. ALTERNATES

ITEM DESCRIPTION	ALTERNATE #1	ALTERNATE #2	ALTERNATE #3	ALTERNATE #4
Superstructure:				
Type:	________	________	________	________
No. of Spans:	________	________	________	________
Beam Spacing:	________	________	________	________
No. of Beams:	________	________	________	________
Deck Quantity:	________	________	________	________
Deck Cost	________	________	________	________
Beam Quantity:	________	________	________	________
Beam Cost:	________	________	________	________

D-501 (1-96) SHEET NO. 4 OF 4

QUALITY ASSURANCE FORM FOR T.S.&L. OF STEEL BRIDGES

(New Structures)

Designer: ____ Date: ___________

(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

ITEM DESCRIPTION	ALTERNATE #1	ALTERNATE #2	ALTERNATE #3	ALTERNATE #4
Substructure:				
Abutment: Type:	________	________	________	________
Quantity:	________	________	________	________
Cost:	________	________	________	________
Pier: Type:	________	________	________	________
Quantity:	________	________	________	________
Cost:	________	________	________	________
Foundation: Type:	________	________	________	________
Quantity:	________	________	________	________
Cost:	________	________	________	________
TOTAL COST:	________	________	________	________

8. COMMENTS

____

____

____

D-502 (1-96)
REPRODUCE LOCALLY

SHEET NO. 1 OF 4

QUALITY ASSURANCE FORM FOR T.S.&L. OF STEEL BRIDGES

(Rehabilitation Structures)

Designer: ______________________________ Date: ___________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

1. PROJECT INFORMATION

County: ____________ S.R.: ________ Sec.: ________ Over: ______________
(STREAM, RAILROAD, OR ROAD)

Structure Identification: ______________________________

S-No.: ______________________________

Bridge Type: ____________________ Year Built: ________________

Design ADT: ________ Year: ________ ADTT Present: __________Cumulative: __________

2. BRIDGE GEOMETRY

Proposed Width: Out-Out ____________; Curb-Curb ____________; Sidewalk ____________

Skew: ______________

Minimum Clearance: Vertical → Provided = ____________; Required = ____________

Horizontal → Provided = ____________; Required = ____________

3. SUPERSTRUCTURE

Structure to be Replaced: Deck ☐ Deck and Beams ☐ None ☐

Deck Condition Rating from NBIS report: ____________________ Year: __________

Proposed Deck Rehabilitation:______________________________

Calculation Page for Logic: ______________________________

Design Methodology: ______________________________

Analysis Methodology (DM-4 Sec. 4):______________________________

Remaining Fracture-Critical Elements: ______________________________

____

Remaining Fatigue Life: ____________________ Calculation Page:__________

Retrofitted Distortion-Induced Fatigue Details: ______________________________

____

D-502 (1-96) SHEET NO. 2 OF 4

QUALITY ASSURANCE FORM FOR T.S.&L. OF STEEL BRIDGES

(Rehabilitation Structures)

Designer: ______________________________ Date: __________

(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

No. of Spans: ____________ Span Length(s): ____________

No. of Beams: ____________ (If > 4, Why?): ____________

Straight, Curved, or Kinked: ____________

Spacing: ____________ Max. Overhang: ____________

Web Size: ____________ Slab Thickness: ____________

BEARINGS	ABUT. 1	ABUT. 2	PIER 1	PIER 2	PIER 3	PIER 4
Function:	____	____	____	____	____	____
Type:	____	____	____	____	____	____

DECK JOINTS	LOCATION	TYPE	MOVEMENT CLASSIFICATION
No. ____	____	____	____
____	____	____	____
____	____	____	____

(if > 2, why?): ____________

4. SUBSTRUCTURE

Parallel Eccentricity Limit ____________ ; Calc. Page ____________

Maximum Parallel Eccentricity ____________ ; Calc. Page ____________

Perpendicular Eccentricity Limit ____________ ; Calc. Page ____________

Maximum Perpendicular Eccentricity ____________ ; Calc. Page ____________

Sliding Resistance ____________; Maximum Horiz. Load ____________; Calc. Page ____________

Bearing Resistance ____________;Maximum Bearing Pressure ____________ ; Calc. Page ____________

Type of Scour Protection Proposed: ____________

Is Substructure Adequately Reinforced? Yes ☐ No ☐

D-502 (1-96) SHEET NO. 3 OF 4

QUALITY ASSURANCE FORM FOR T.S.&L. OF STEEL BRIDGES

(Rehabilitation Structures)

Designer: ______________________________ Date: ___________

(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

Type of Seismic Retrofit Proposed: ______________________________

____

5. RATINGS

Ratings after Rehabilitation, including Future Wearing Surface:

LOAD	PHL-93 (Factor)	H (MetricTonne) {Tons}	HS (Metric Tonne) {Tons}	ML-80 (Metric Tonne) {Tons}	P-82 (Metric Tonne) {Tons}
I.R.	________	________	________	________	N/A ________
O.R.	________	________	________	________	________

6. MISCELLANEOUS

Drainage: On Structure Number: ______________ and Type ______________

Off Structure Number: ______________ and Type ______________

Protective Fence: Yes ☐ No ☐

Navigation Lights: Yes ☐ No ☐

Inspection Facilities: Yes ☐ No ☐

Utilities: ______________________________

Bridge-Mounted Sign Structures: Yes ☐ No ☐

Bridge-Mounted Lighting: Yes ☐ No ☐

Was High Mast or Pier-Mounted Considered? Yes ☐ No ☐ If no, why? ______________

____

D-502 (1-96) SHEET NO. 4 OF 4

QUALITY ASSURANCE FORM FOR T.S.&L. OF STEEL BRIDGES
(Rehabilitation Structures)

Designer: ______________________________ Date: __________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

7. ALTERNATES SUMMARY

Is this a Streamlined submission? Yes ☐ No ☐

If yes, how many Alternates are required 1 ☐ 2☐

Permitted Alternates by Contractors:

Prestressed Concrete ☐ Steel ☐ Other ☐ None ☐

If other, specify: ______________________________

ITEM DESCRIPTION	ALTERNATE #1	ALTERNATE #2	ALTERNATE #3	ALTERNATE #4
Total Cost:	________	________	________	________
Calculation Page:	________	________	________	________

8. COMMENTS

____

____

____

____

D-503 (1-96)
REPRODUCE LOCALLY

SHEET NO. 1 OF 4

QUALITY ASSURANCE FORM FOR T.S.&L. OF PRESTRESSED CONCRETE BRIDGES

(New Structures)

Designer: ______________________________________ Date: __________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

1. PROJECT INFORMATION

County: ______________ S.R.: ________ Sec.: ________ Over: ______________
(STREAM, RAILROAD, OR ROAD)

S-No.: ____________ Design ADT: ____________ ADTT: ____________ Year: ____________

2. BRIDGE GEOMETRY

Horizontal Geometry: ______________________________________

Proposed Width: Out-Out ______________; Curb-Curb ______________; Sidewalk ____________

Skew: ______________ (if < 70º, why?): ______________________________

Minimum Clearance: Vertical → Provided = ______________; Required = ______________

Horizontal → Provided = ______________; Required = ______________

3. SUPERSTRUCTURE

Type: ______________________________________

Design Methodology: ______________________________________

Analysis Methodology (DM-4 Sec. 4): ______________________________

No. of Spans: ______________ Span Length(s): ______________________________

No. of Beams: ______________ (if > 4, why?): ______________________________

____

Beam Strength (f'_c): ______________________________________

Beam Spacing: ______________________ Max. Overhang: ______________________

Beam Size: ______________________ Slab Thickness: ______________________

BEARINGS	ABUT. 1	ABUT. 2	PIER 1	PIER 2	PIER 3	PIER 4
Function:	________	________	________	________	________	________
Type:	________	________	________	________	________	________

D-503 (1-96) SHEET NO. 2 OF 4

QUALITY ASSURANCE FORM FOR T.S.&L. OF PRESTRESSED CONCRETE BRIDGES

(New Structures)

Designer: ______________________________ Date: ___________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

DECK JOINTS	LOCATION	TYPE	MOVEMENT CLASSIFICATION
No. ______	______	______	______
______	______	______	______
______	______	______	______

(if > 2, why?): ______________________________

4. SUBSTRUCTURE

UNIT	ABUT. 1	ABUT. 2	PIER 1	PIER 2	PIER 3	PIER 4
Type:	______	______	______	______	______	______
Support:	______	______	______	______	______	______
Geology:	______	______	______	______	______	______
# Borings:	______	______	______	______	______	______

(**Note**: Type = Structure Type; Support = Shallow or Deep Foundation; Geology = Type of Geological)

5. MISCELLANEOUS

Are there any Superbeams? Yes ☐ No ☐

If yes, has the hauling permit been issued? Yes ☐ No ☐

Drainage: On Structure Number: ______________ and Type ______________

Off Structure Number: ______________ and Type ______________

Protective Fence: Yes ☐ No ☐

Navigation Lights: Yes ☐ No ☐

D-503 (1-96) SHEET NO. 3 OF 4

QUALITY ASSURANCE FORM FOR T.S.&L.
OF PRESTRESSED CONCRETE BRIDGES
(New Structures)

Designer: ______________________________ Date: ___________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

Inspection Facilities: Yes ☐ , and Type ____________________; or No ☐

Utilities: ______________________________

Bridge-Mounted Sign Structures: Yes ☐ No ☐

Bridge-Mounted Lighting: Yes ☐ No ☐

Was High Mast or Pier-Mounted Considered? Yes ☐ No ☐ If no, why? ________________________

____

6. ALTERNATES SUMMARY

Is this a Streamlined submission? Yes ☐ No ☐

If yes, how many Alternates are required? 1 ☐ 2 ☐

	ALTERNATE #1	ALTERNATE #2	ALTERNATE #3	ALTERNATE #4
Description:	________	________	________	________
Total Cost:	________	________	________	________
Calculation Page:	________	________	________	________

Permitted Alternates by Contractors:

Prestressed Concrete ☐ Steel ☐ Other ☐

If other, specify: ______________________________

7. ALTERNATES

ITEM DESCRIPTION	ALTERNATE #1	ALTERNATE #2	ALTERNATE #3	ALTERNATE #4
Superstructure:				
Type:	________	________	________	________
No. of Spans:	________	________	________	________
Beam Spacing:	________	________	________	________

D-503 (1-96) SHEET NO. 4 OF 4

QUALITY ASSURANCE FORM FOR T.S.&L. OF PRESTRESSED CONCRETE BRIDGES

(New Structures)

Designer: ______________________________ Date: ___________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

ITEM DESCRIPTION	ALTERNATE #1	ALTERNATE #2	ALTERNATE #3	ALTERNATE #4
Superstructure:				
No. of Beams:	________	________	________	________
Deck Quantity:	________	________	________	________
Deck Cost	________	________	________	________
Beam Quantity:	________	________	________	________
Beam Cost:	________	________	________	________
Substructure:				
Abutment: Type:	________	________	________	________
Quantity:	________	________	________	________
Cost:	________	________	________	________
Pier: Type:	________	________	________	________
Quantity:	________	________	________	________
Cost:	________	________	________	________
Foundation: Type:	________	________	________	________
Quantity:	________	________	________	________
Cost:	________	________	________	________
TOTAL COST:	________	________	________	________

8. COMMENTS

____

____

____

D-504 (1-96)
REPRODUCE LOCALLY

SHEET NO. 1 OF 4

QUALITY ASSURANCE FORM FOR T.S.&L. OF PRESTRESSED CONCRETE BRIDGES

(Rehabilitation Structures)

Designer: ______________________________ Date: __________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

1. PROJECT INFORMATION

County: __________ S.R.: ________ Sec.: ________ Over: ______________
(STREAM, RAILROAD, OR ROAD)

S-No.: ______________________________

Bridge Type: ______________ Year Built: ______________

Design ADT: ________ Year: ________ ADTT Present: ________ Cumulative: ________

2. BRIDGE GEOMETRY

Proposed Width: Out-Out __________; Curb-Curb __________; Sidewalk __________

Skew: __________

Minimum Clearance: Vertical → Provided = __________; Required = __________

Horizontal → Provided = __________; Required = __________

3. SUPERSTRUCTURE

Structure to be Replaced: Deck ☐ Deck and Beams ☐ None ☐

Deck Condition Rating from NBIS report: ______________ Year: __________

Proposed Deck Rehabilitation: ______________________________

Calculation Page for Logic: ______________________________

Design Methodology: ______________________________

Analysis Methodology (DM-4 Sec. 4): ______________________________

No. of Spans: __________ Span Length(s): ______________

No. of Beams: __________ (if > 4, why?): ______________

____

Beam Spacing: ______________ Max. Overhang: ______________

Beam Size: ______________ Slab Thickness: ______________

D-504 (1-96) SHEET NO. 2 OF 4

QUALITY ASSURANCE FORM FOR T.S.&L. OF PRESTRESSED CONCRETE BRIDGES

(Rehabilitation Structures)

Designer: ____ Date: ___________

(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

BEARINGS	ABUT. 1	ABUT. 2	PIER 1	PIER 2	PIER 3	PIER 4
Function:	________	________	________	________	________	________
Type:	________	________	________	________	________	________

DECK JOINTS	LOCATION	TYPE	MOVEMENT CLASSIFICATION
No. ________	________	________	________
________	________	________	________
________	________	________	________

(if > 2, why?): ____

4. SUBSTRUCTURE

Parallel Eccentricity Limit ______________________ ; Calc. Page ____________

Maximum Parallel Eccentricity ______________________ ; Calc. Page ____________

Perpendicular Eccentricity Limit ______________________ ; Calc. Page ____________

Maximum Perpendicular Eccentricity ______________________ ; Calc. Page ____________

Sliding Resistance ____________; Maximum Horiz. Load ____________; Calc. Page ________

Bearing Resistance __________ ; Maximum Bearing Pressure ________; Calc. Page ________

Type of Scour Protection Proposed: ____

Is Substructure Adequately reinforced? Yes ☐ No ☐

Type of Seismic Retrofit Proposed:____

____

D-504 (1-96) SHEET NO. 3 OF 4

QUALITY ASSURANCE FORM FOR T.S.&L. OF PRESTRESSED CONCRETE BRIDGES

(Rehabilitation Structures)

Designer: ______________________________ Date: __________

(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

5. RATINGS

Ratings after Rehabilitation, including Future Wearing Surface:

LOAD	PHL-93 (Factor)	H (MetricTonne) {Tons}	HS (Metric Tonne) {Tons}	ML-80 (Metric Tonne) {Tons}	P-82 (Metric Tonne) {Tons}
I.R.	______	______	______	______	N/A ______
O.R.	______	______	______	______	______

6. MISCELLANEOUS

Are there any Superbeams? Yes ☐ No ☐

If yes, has the hauling permit been issued? Yes ☐ No ☐

Drainage: On Structure Number: ______________ and Type ______________________

Off Structure Number: ______________ and Type ______________________

Protective Fence: Yes ☐ No ☐

Navigation Lights: Yes ☐ No ☐

Inspection Facilities: Yes ☐, and Type ______________; No ☐

Utilities:____

Bridge-Mounted Sign Structures: Yes ☐ No ☐

Bridge-Mounted Lighting: Yes ☐ No ☐

Was High Mast or Pier-Mounted Considered? Yes ☐ No ☐ If no, why? ____________________

____

D-504 (1-96) SHEET NO. 4 OF 4

QUALITY ASSURANCE FORM FOR T.S.&L. OF PRESTRESSED CONCRETE BRIDGES

(Rehabilitation Structures)

Designer: ______________________________ Date: ___________

(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

7. ALTERNATES SUMMARY

Is this a Streamlined submission? Yes ☐ No ☐

If yes, how many Alternates are required? 1 ☐ 2 ☐

Permitted Alternates by Contractors:

Prestressed Concrete ☐ Steel ☐ Other ☐

If other, specify: ______________________________

ITEM DESCRIPTION	ALTERNATE #1	ALTERNATE #2	ALTERNATE #3	ALTERNATE #4
Total Cost:	________	________	________	________
Calculation Page:	________	________	________	________

8. COMMENTS

____

____

____

____

D-505 (4-00)
REPRODUCE LOCALLY

SHEET NO. 1 OF 7

QUALITY ASSURANCE FORM FOR FOUNDATIONS

Designer: ____ Date: ___________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

1. PROJECT INFORMATION

County: ____________________ S.R.: __________ Sec.: __________ Over: ____________________
(STREAM, RAILROAD, OR ROAD)

S-No.: ____________________ Skew: ____________________ Grade: ____________________

Bridge Type: ____

No. of Substructure Units: ____

Note: **Bold-faced** parameters are those requiring approval of Central Office BQAD.

2. FOUNDATION TYPE

	ABUT. 1	ABUT. 2	PIER 1	PIER 2	PIER 3*
Spread Footing on Rock: (Complete Sections 3 & 4)	_______	_______	_______	_______	_______
Spread Footing on Soil: (Complete Sections 3 & 5)	_______	_______	_______	_______	_______
Point Bearing Pile Size: (Complete Sections 3 & 6)	_______	_______	_______	_______	_______
End Bearing Pile Size: (Complete Sections 3 & 6)	_______	_______	_______	_______	_______
Friction Pile Size: (Complete Sections 3 & 6)	_______	_______	_______	_______	_______
Caisson Size: (Complete Sections 3 & 6)	_______	_______	_______	_______	_______
Pedestal Size: (Complete Section 3)	_______	_______	_______	_______	_______
Depth of Footing Embedment**:	___/___	___/___	___/___	___/___	___/___

Comments (Special Treatments, etc.) ____

____

____

D-505 (4-00) SHEET NO. 2 OF 7

QUALITY ASSURANCE FORM FOR FOUNDATIONS

Designer: ____ Date: ___________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

3. BEARING STRATUM AND FOUNDATION PARAMETER INFORMATION

Any solution cavity, mining, pH, resistivity, sulfate, dissolved chloride, fill, or bacteria problems?

Yes ☐ No ☐

If yes, identify problem, location, and depth ______________________________________

	ABUT. 1	ABUT. 2	PIER1	PIER2	PIER 3*
Bearing Stratum:	_______	_______	_______	_______	_______
(1.5 x footing width below bottom of spread footings on soils) (15-pile dia. above tip elevation for end bearing & friction piles)					
RQD:	_______	_______	_______	_______	_______
% Gross Recovery:	_______	_______	_______	_______	_______
Average N Value (blows/300 mm){blows/ft}:	_______	_______	_______	_______	_______
Unconfined Comp. Test Result (Rock or Soil) (MPa){tsf}:	_______	_______	_______	_______	_______
Non-Saturated Soil Density (kg/m^3){tcf}:	_______	_______	_______	_______	_______
Saturated Soil Density (kg/m^3){tcf}:	_______	_______	_______	_______	_______
Soil Slope Inclination (if on slope) i (deg.):	_______	_______	_______	_______	_______
Depth from grade to groundwater, $D_f + Z_w$ (mm){ft}:	_______	_______	_______	_______	_______
Undrained Shear Strength, S_u (MPa){tsf}:	_______	_______	_______	_______	_______
Foundations & Loadings					
V*** (N){tons}:	_______	_______	_______	_______	_______
H*** (N){tons}:	_______	_______	_______	_______	_______
Bottom of Footing Elevation (mm){ft}:	_______	_______	_______	_______	_______
D_f (mm){ft}:	_______	_______	_______	_______	_______
B (mm)(ft):	_______	_______	_______	_______	_______
L (mm){ft}:	_______	_______	_______	_______	_______

D-505 (4-00) SHEET NO. 3 OF 7

QUALITY ASSURANCE FORM FOR FOUNDATIONS

Designer: ____ Date: ____________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

4. SPREAD FOOTINGS - ON ROCK BEARING STRATUM

				ABUT. 1	ABUT. 2	PIER1	PIER2	PIER 3*
	Coefficient of Friction between rock & footing:			_______	_______	_______	_______	_______
Show the appropriate information for A or B	(A) Semi-EmpiricalMethod	Use a or b	(a) RMR:	_______	_______	_______	_______	_______
			(b) NGI:	_______	_______	_______	_______	_______
			C_o (MPa){tsf}:	_______	_______	_______	_______	_______
			N_{ms}:	_______	_______	_______	_______	_______
	(B) Analytic Method		c (MPa){tsf}:	_______	_______	_______	_______	_______
			φ_{fm}:	_______	_______	_______	_______	_______
			Horiz. Joint Spacing, S (mm){ft}:	_______	_______	_______	_______	_______
		Closely Spaced Joints (D10.6.3.2.3aP)	_______	_______	_______	_______	_______	_______
			_______	_______	_______	_______	_______	_______
			_______	_______	_______	_______	_______	_______
			N_γ:	_______	_______	_______	_______	_______
			Y(kg/m^3){tcf}:	_______	_______	_______	_______	_______
		Widely Spaced Joints (D10.6.3.2.3bP)	J:	_______	_______	_______	_______	_______
			N_{cr}:	_______	_______	_______	_______	_______
			Vert. Joint Spacing, H_p (mm){ft}:	_______	_______	_______	_______	_______
			K_e:	_______	_______	_______	_______	_______
			q_{ult} - APPROX (MPa){tsf}.	_______	_______	_______	_______	_______
			q_{ult} - FINAL (MPa){tsf}:	_______	_______	_______	_______	_______
			φ:	_______	_______	_______	_______	_______
			q_r (MPa){tsf}:	_______	_______	_______	_______	_______
			Estimated Total Settlement (mm){in}:	_______	_______	_______	_______	_______
			Tolerable Settlement (mm){in}:	_______	_______	_______	_______	_______

For rocks defined as very poor quality in the Semi-Empirical Method, complete Section 5 with an equivalent soil mass

D-505 (4-00) SHEET NO. 4 OF7

QUALITY ASSURANCE FORM FOR FOUNDATIONS

Designer: ______________________________ Date: ___________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

5. SPREAD FOOTINGS - ON SOIL BEARING STRATUM

			ABUT. 1	ABUT. 2	PIER 1	PIER 2	PIER 3*
		Coefficient of Friction between soil & footing:	______	______	______	______	______
Show the appropriate information for A, B or C	(A) Theoretical EstimationMethod :	**c** (MPa){tsf}:	______	______	______	______	______
	(If on slope, substitute N_{cq} & $N_{\gamma q}$ for N_c and N_γ)	$\boldsymbol{\varphi_f}$:	______	______	______	______	______
		$\mathbf{N_\gamma}$:	______	______	______	______	______
		$\mathbf{N_c}$:	______	______	______	______	______
		$\mathbf{N_q}$:	______	______	______	______	______
	(B) SPT Method:	$\overline{N}_{corr}$:	______	______	______	______	______
	(C)CPT Method:	______	______	______	______	______	______
		$\mathbf{q_{ult}}$ **-APPROX** (MPa){tsf}:	______	______	______	______	______
		$\mathbf{q_{ult}}$ **- Final** (MPa){tsf}:	______	______	______	______	______
		$\boldsymbol{\varphi}$:	______	______	______	______	______
		$\mathbf{q_r}$ (MPa){tsf}:	______	______	______	______	______
		Estimated Total Settlement (mm){in}:	______	______	______	______	______
		Estimated Settlement After Beam Erection (mm){in}:	______	______	______	______	______
		Tolerable Net Settlement (mm){in}:	______	______	______	______	______

D-505 (4-00) SHEET NO. 5 OF 7

QUALITY ASSURANCE FORM FOR FOUNDATIONS

Designer: ____ Date: ___________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

6. PILE AND DRILLED SHAFT FOUNDATIONS

				ABUT. 1	ABUT. 2	PIER 1	PIER 2	PIER 3*
			Static Analysis Method:					
			Pile Size (e.g., HP 12 x 74) or Pile/Shaft Diameter (mm){ft}:					
			Pile/Shaft Length (mm){ft}:					
			Rock Socket Depth (mm){ft}:					
Show the appropriate information for A, B or C (friction and end-bearing piles and drilled shafts)	(A) Semi-Empirical Method:	Show the appropriate information for a, b, c or d	(a) Nordlund					
			K_δ					
			Pd(MPa){tsf}:					
			$\mathbf{C_F}$:					
			Δ (MPa){tsf}:					
			(b) **α**:					
			(c) **β**:					
			$\boldsymbol{\delta'_v}$(MPa){tsf}:					
			(d) **λ**:					
			$\boldsymbol{\delta'_v}$ (MPa){tsf}:					
	(B) SPT Method:		$\mathbf{N_{corr}}$:					
			$\mathbf{D_b}$ (mm):					
			D (mm):					
			$\boldsymbol{\delta'_v}$ (MPa){tsf}:					
	(C) CPT Method:(Requires approval of Chief Bridge Engineer)		$\mathbf{q_{c1}}$ (MPa){tsf}:					
			$\mathbf{q_{c2}}$(MPa){tsf}:					
			$\mathbf{Q_s}$ (MPa){tsf}:					
			Shaft Resistance $\mathbf{Q_s}$ (MPa){tsf}:					

D-505 (4-00) SHEET NO. 6 OF 7

QUALITY ASSURANCE FORM FOR FOUNDATIONS

Designer: ____ Date: ___________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

6. PILE AND DRILLED SHAFT FOUNDATIONS (continued)

	ABUT. 1	ABUT. 2	PIER 1	PIER 2	PIER 3*
Tip Resistance (MPa){tsf}:	______	______	______	______	______
Total Resistance (MPa){tsf}:	______	______	______	______	______
Side Resistance (MPa){tsf}:	______	______	______	______	______
φ:	______	______	______	______	______
Group Resistance (MPa){tsf}:	______	______	______	______	______
Is pile buckling a consideration?:	______	______	______	______	______
Pile Spacing (mm) {ft}:	______	______	______	______	______
Maximum Factored Vertical Load per Pile****(kN){k}:	______	______	______	______	______
Unfactored (Service I) Vertical Load per Pile (kN){k}:	______	______	______	______	______
Maximum Factored Lateral Load per Pile**** (kN){k}:	______	______	______	______	______
Unfactored (Service I) Lateral Load per Pile (kN){k}:	______	______	______	______	______
Estimated Total Settlement (mm){in}:	______	______	______	______	______
Estimated Lateral Settlement (mm){in}:	______	______	______	______	______
Estimated Settlement Before Beam Erection (mm){in}:	______	______	______	______	______
Tolerable Settlement (mm){in}:	______	______	______	______	______

* Use additional sheets if more than three piers.
** From the existing groundline to the bottom of the footing/From the finished groundline to the bottom of the footing.
*** H and V values are those from the load combination which controlled the strength design for bearing pressure.
**** Maximum factored pile loads are those from the load combination which controlled the strength design for bearing pressure.

7. SCOUR INFORMATION

Stream X-section at Bridge shown on Calculation Page: ______________________________

Value for projection into channel: A ______________________________________

Average Approach Velocity: Q_{100} = ________________ m/s{ft/sec}: Q_{500} = ________________ m/s{ft/sec}

Scour Computations Based on: FHWA Tech. Advisory ☐ HEC-18 ☐

D-505 (4-00) SHEET NO. 7 OF 7

QUALITY ASSURANCE FORM FOR FOUNDATIONS

Designer: ____ Date: ___________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

Computed Scour Depths:

ITEMS		SCOUR DEPTH IN MM									
		ABUT. 1		ABUT. 2		PIER 1		PIER 2		PIER 3*	
		Q_{100}	Q_{500}	Q_{100}	Q_{500}	Q_{100}	Q_{500}	Q_{100}	Q_{500}	Q_{100}	Q_{500}
AGGRADATION OR DEGRADATION SCOUR											
CONTRACTION SCOUR											
LOCAL SCOUR											
TOTAL SCOUR											
PROPOSED FOOTING ELEVATION	*TOP*										
	BOTTOM										
ADJACENT STREAM BED ELEVATION											
PROPOSED PILE OR SHAFT TIP ELEVATION											

Any known scour problems at or near the location of the proposed substructure unit(s)?

Yes ☐ No ☐

If yes, give pertinent information, such as scour depths: ________________________________

____

____

____

Scour protection reduction utilized? Yes ☐ No ☐

*Use additional sheets if more than three piers.

D-506 (1-96)
REPRODUCE LOCALLY

SHEET NO. 1 OF 2

QUALITY ASSURANCE FORM FOR GLULAM TIMBER BRIDGES

Designer: ______________________________________ Date: ___________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

1. PROJECT INFORMATION

County: ____________________ S.R.: __________ Sec.: __________ Over: ______________________
(STREAM, RAILROAD, OR ROAD)

S-No.:______________ Design ADT: ______________ ADTT: ______________ Year: ______________

2. BRIDGE GEOMETRY

Horizontal Geometry: ____

Width: Out-Out ____________; Curb-Curb ____________________; Sidewalk ____________________

Span Length: ____________________

Skew: ________________ (if < 70º, why?): ______________________________________

Minimum Clearance: Vertical → Provided = __________________; Required = ____________________

Horizontal → Provided = __________________; Required = ____________________

3. SUPERSTRUCTURE

Type: Glulam Deck ☐ Glulam Deck/Steel Beams ☐ Glulam Deck/Glulam Beams ☐

Design Live Load: PHL-93 ☐ H ________ ☐ HS ________ ☐ ML-80 ☐ P-82 ☐

Max. Flexural Stress in Deck: ______________________________ MPa{ksi},

Flexural Resistance: ___________________ MPa{ksi} (wet usage); Calc. Page: ____________________

Species of Hardwood: ____________________; Grade: ____________________

Deck Panel Interconnecting Device: Dowel ☐ Other ☐ , specify ______________________________

Beam Size: _____________________________ Beam Spacing: ______________________________

Max. Flexural Stress in Deck: ___________________ MPa{ksi},

Flexural Resistance: ___________________ MPa{ksi} (wet usage); Calc. Page: ___________________

Species of Hardwood: _____________________; Grade: ____________________

L.L. Deflection: Allowed = L/425 = _____________________; Designed: _____________________

Calc. Page: ____________________

D-506 (1-96) SHEET NO. 2 OF 2

QUALITY ASSURANCE FORM FOR GLULAM TIMBER BRIDGES

Designer: ____ Date: __________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

4. SUBSTRUCTURE

Abut. Type: __________________; Bot. of Ftg. Elev.: _____________ ; Scour Depth Elev.: ______________

Pile Type (Material) __________________________; Est. Pile Tip Elev. _____________________ ;

Bot. of Lagging Elev. __________________________; *Average N ________________________

Pier Type: _________________; Bot. of Ftg. Elev.: _____________; Scour Depth Elev.: ____________

Pile Type (Material) __________________________; Est. Pile Tip Elev. _______________________ ;

*Average N __________________________

Anticipated Debris Problem: Yes ☐ No ☐

*If timber piles are proposed, average N value is for the strata above the bearing stratum.

5. ALTERNATES SUMMARY

Is this a Streamlined submission? Yes ☐ No ☐

If yes, how many Alternates are required? 1 ☐ 2 ☐

Permitted Alternates by Contractors:

Prestressed Concrete ☐ Steel ☐ Other ☐

If other, specify: ____

ITEM DESCRIPTION	ALTERNATE #1	ALTERNATE #2	ALTERNATE #3	ALTERNATE #4
Total Cost:	____________	____________	____________	____________
Calculation Page:	____________	____________	____________	____________

6. COMMENTS

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D-507 (1-96)
REPRODUCE LOCALLY

SHEET NO. 1 OF 3

QUALITY ASSURANCE FORM FOR STRESSED TIMBER BRIDGES

Designer: ______________________________ Date: ___________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

1. PROJECT INFORMATION

County: ______________ S.R.: ________ Sec.: ________ Over: ______________
(STREAM, RAILROAD, OR ROAD)

S-No.:____________ Design ADT: ____________ ADTT: ____________ Year: ____________

2. BRIDGE GEOMETRY

Horizontal Geometry: ______________________________

Width: Out-Out __________; Curb-Curb ______________; Sidewalk ______________

Span Length: ______________

Skew: ____________ (if < 70º, why?): ______________________________

Minimum Clearance: Vertical → Provided = ____________; Required = ______________

Horizontal → Provided = ____________; Required = ______________

3. SUPERSTRUCTURE

Design Live Load: PHL-93 ☐ H ______ ☐ HS ______ ☐ ML-80 ☐ P-82 ☐

Species of Hardwood: ______________; Grade: ______________

Composite Stressed Timber Design: Yes ☐ No ☐

If yes, spacing of galvanized steel sandwich plates ______________ mm {in.} center-to-center

Max. Flexural Stress in Deck: ______________ MPa{ksi},

Flexural Resistance: ______________ MPa{ksi} (wet usage); Calc. Page: ______________

L.L. Deflection: Allowed = L/425 = ______________; Designed: ______________

Calc. Page: ______________

Does butt joint pattern meet criteria: Yes ☐ No ☐

Size of channel anchorage: ______________

Ultimate tensile strength of prestressing rods: ______________ MPa

Is triple corrosion protection specified: Yes ☐ No ☐

D-507 (1-96) SHEET NO. 2 OF 3

QUALITY ASSURANCE FORM FOR STRESSED TIMBER BRIDGES

Designer: ____ Date: __________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

Minimum prestress between laminae: __________________ MPa {ksi}

Interval of retensioning: 2nd ☐ 3rd ☐ 4th ☐ days

4. SUBSTRUCTURE

Abut. Type: ____________________; Bot. of Ftg. Elev.: ______________ ; Scour Depth Elev.: ______________

Pile Type (Material) ____________________________; Est. Pile Tip Elev. ________________________;

Bot. of Lagging Elev. ____________________________; *Average N ____________________________

Pier Type: ___________________; Bot. of Ftg. Elev.: ______________; Scour Depth Elev.: ______________

Pile Type (Material) ____________________________; Est. Pile Tip Elev. ____________________________;

*Average N ____________________________

Anticipated Debris Problem: Yes ☐ No ☐

*If timber piles are proposed, average N value is for the strata above the bearing stratum.

5. ALTERNATES SUMMARY

Is this a Streamlined submission? Yes ☐ No ☐

If yes, how many Alternates are required? 1 ☐ 2 ☐

Permitted Alternates by Contractors:

Prestressed Concrete ☐ Steel ☐ Other ☐

If other, specify: ____

ITEM DESCRIPTION	ALTERNATE #1	ALTERNATE #2	ALTERNATE #3	ALTERNATE #4
Total Cost:	______________	______________	______________	______________
Calculation Page:	______________	______________	______________	______________

D-507 (1-96) SHEET NO. 3 OF 3

QUALITY ASSURANCE FORM FOR STRESSED TIMBER BRIDGES

Designer: ____ Date: __________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

6. COMMENTS

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D-508 (1-96)
REPRODUCE LOCALLY

SHEET NO. 1 OF 3

QUALITY ASSURANCE FORM FOR COMPOSITE STEEL-GIRDER SUPERSTRUCTURE DESIGN

Designer: ____ Date: ___________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

1. PROJECT INFORMATION

County: ____________________ S.R.: __________ Sec.: __________ Over: ____________________
(STREAM, RAILROAD, OR ROAD)

S-No.:_______________ Design ADT: _______________ ADTT: _______________ Year: _______________

2. GENERAL INFORMATION

No. of Spans: _______________; Radius of Curvature: _______________; Max. Skew: _______________

Span Length(s) and Arrangement: ____

No. of Girders: ________________; Girder Spacing: ________________; Web Depth: _______________

Any deviations from T.S.&L. and Foundation approvals? Yes ☐ No ☐

If yes, indicate reasons:____

____

List restrictions on Alternate Design, if any: ____

Design Method: ______________________________; Analysis Method: ____________________________

3. SUPERSTRUCTURE

Section Properties Used for Calculation of Moments, Shears, and Deflections:

Max. Positive Moment I = ___________________________________ ; Calc. Page _________________

Max. Negative Moment I = __________________________________ ; Calc. Page _________________

Section Properties Used for Calculation of Girder Stresses:

Max. Positive Moment I = _____________________________________; Calc. Page _________________

Max. Negative Moment I = ____________________________________; Calc. Page _________________

Greatest Girder Moment and Which Live Load Governed: Interior ☐ Exterior ☐

Max. Positive Moment M = _______________; Live Load _______________; Calc. Page _______________

Max. Negative Moment M = ______________; Live Load _______________; Calc. Page _______________

D-508 (1-96) SHEET NO. 2 OF 3

QUALITY ASSURANCE FORM FOR COMPOSITE STEEL-GIRDER SUPERSTRUCTURE DESIGN

Designer: ____ Date: ___________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

For curved girders which are designed with a refined method of analysis, the ratio of the moment by the refined analysis for HS25 and that by the AASHTO Simplified Method for HS20 for the greatest girder moment (show in terms of calculated moments, i.e., HS25 moment/HS20 moment):

Positive Moment Ratio = ______________________ ; Calc. Page __________

Negative Moment Ratio = ______________________ ; Calc. Page __________

Lateral Bending Stress = ______________________ ; Calc. Page __________

Number of Deck Placements ______________________ ; Calc. Page __________

Greatest locked-in girder stresses (compression and tension) due to deck placement sequence:

Magnitude ______________________ ; Calc. Page __________

Location ______________________ ; Calc. Page __________

Resistance ______________________ ; Calc. Page __________

Prior to deck hardening:

Max. Comp. non-composite flange stress ______________________ ; Calc. Page __________

Location ______________________ ; Calc. Page __________

Resistance ______________________ ; Calc. Page __________

Max. Comp. non-composite web buckling stress ______________________ ,Calc. Page __________

Location ______________________ ; Calc. Page __________

Resistance ______________________ ; Calc. Page __________

Vertical placement of temporary overhang support bracket
(in terms of the depth of the girder web) ______________________ ; Calc. Page __________

Max. transverse stiffener spacing ______________________ ; Calc. Page __________

Max. service live-load + impact deflection as a ratio of span/deflection:

Deflection ______________________ ; Calc. Page __________

Location ______________________ ; Calc. Page __________

D-508 (1-96) SHEET NO. 3 OF 3

QUALITY ASSURANCE FORM FOR COMPOSITE STEEL-GIRDER SUPERSTRUCTURE DESIGN

Designer: ____ Date: ___________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

Load path chosen to carry transverse wind load to the bearings:

____ ; Calc. Page ____________

Category of the least fatigue resistant detail used ______________ ; Calc. Page ____________

Stress range at the most critical of these details ______________ ; Calc. Page ____________

Location of most critical detail ______________________ ; Calc. Page ____________

Ultimate fatigue resistance ______________________ ; Calc. Page ____________

Diaphragm design method ______________________ ; Calc. Page ____________

Bearing Types:

Expansion ______________________________ ; Calc. Page ____________

Fix ______________________________ ; Calc. Page ____________

Expansion Dam Type ______________________ ; Calc. Page ____________

Expansion Movement ______________________ ; Calc. Page ____________

D-509 (4-00) SHEET NO. 1 OF 3
REPRODUCE LOCALLY

QUALITY ASSURANCE FORM FOR PRESTRESSED CONCRETE BRIDGE DESIGN

Designer: ____ Date: ___________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

1. PROJECT INFORMATION

County: ____________________ S.R.: _________ Sec.: _________ Over: ___________________________
(STREAM, RAILROAD, OR ROAD)

S-No.:_______________ Design ADT: _______________ ADTT: _______________ Year: _______________

2. SUPERSTRUCTURE

No. of spans: ______________; Span length(s): ________________; Max. Skew:_____________________

Type and Size of Beams: ____

Any deviations from T.S.&L. and Foundation approvals? Yes ☐ No ☐

If yes, indicate reasons ____

List restrictions on Alternate Design, if any: ____

____

If multi-span, is it a jointless design? Yes ☐ No ☐ N/A ☐

If no, indicate reasons: ____

LL Distribution: PennDOT Approximate ☐ Finite Element ☐ Other ☐

Strands: Size____

1860 MPa{270ksi} Low Lax ________; Stress Rel ________; Straight ________; Draped____________

Debonded in lieu of draping _____________; % Debonded _____________

Max. Unfact. Pos. Moment _____________________ kN·m {kip·ft} (Calc. Page ___________)

Location ____

Max. Unfact. Neg. Moment (Slab @ Cont.) _____________________ kN·m {kip·ft} (Calc. Page ___________)

Location____

D-509 (4-00) SHEET NO. 2 OF 3

QUALITY ASSURANCE FORM FOR PRESTRESSED CONCRETE BRIDGE DESIGN

Designer: ____ Date: ___________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

Final Tensile Stresses in Precomp. Tensile Zone:

PHL-93 (actual) = ______________________ MPa {ksi} (Calc. Page ___________)

Resistance = ______________________ MPa {ksi} (Calc. Page ___________)

Maximum Prestressing Force ______________ kN {kips}; Eccentricity ______________ mm {in.}

For Critical Section*: Maximum Factored Flexural Resistance (M_r) ________________ kN·m {kip·ft};

Maximum Factored Moment (M_u) ________________ kN·m {kip·ft}

*Based on controlling vehicle, either PHL-93 or P-92.

Under-reinforced: ____________________; Over-reinforced: ____________________

Does Web Thickness meet the I-Beam minimum of 200 mm {8 in.}? Yes ☐ No ☐
(except 455/760, 455/835 and 455/915 beams, which is 155 mm)
{except 18/30, 18/33 and 18/36 beams, which is 6 in.}

Does Top Flange Thickness meet AASHTO Type V/VI Beam (minimum of 125 mm {5 in})? Yes ☐ No ☐

Transverse Tendon Layout as per BD-653M and 654M? Yes ☐ No ☐ N/A ☐

Deck Placement Sequence shown for Cont. Spans show on sheet: ___________(including diaphragm area)

If Draped I-Beam Design:

Is extra Shear Steel furnished at drape point? Yes ☐ No ☐

Is Epoxy-Coated Reinforcement provided:

In Non-Composite Adjacent Box Beams with Bituminous Surface? Yes ☐ No ☐

For 2700 mm {9ft.} length at ends of all beams adjacent to joints? Yes ☐ No ☐

Recessed Strand Detail Shown in Plans on Sheet ______________________

Are Strands Debonded in Bottom Row? Yes ☐ No ☐

For continuous designs: Are shear and moment envelopes or tables shown? Yes ☐ No ☐

Is positive moment reinforcement provided/required? Yes ☐ No ☐

D-509 (4-00) SHEET NO. 3 OF 3

QUALITY ASSURANCE FORM FOR PRESTRESSED CONCRETE BRIDGE DESIGN

Designer: ____ Date: ___________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

3. SUBSTRUCTURE

Bearing Types:

Expansion: ______________________________________ (Calc. Page ___________)

Fix: ____ (Calc. Page ___________)

Expansion Dam Type: __________________________________

Design Load for Shear Blocks: ___________________________ (Calc. Page ___________)

Was Live Load Considered in Design of Backwall as per DM-4? Yes ☐ No ☐ N/A ☐

Are Beam Seats at both substructure units sloped the same for adjacent Box Beams? Yes ☐ No ☐ N/A ☐

(Use camber values for longitudinal slopes)

4. COMMENTS

____

____

____

____

D-510 (1-96)
REPRODUCE LOCALLY

SHEET NO. 1 OF 2

QUALITY ASSURANCE FORM FOR ELASTOMERIC BEARING DESIGN

Designer: ____ Date: ___________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

1. PROJECT INFORMATION

County: ____________________ S.R.: _________ Sec.: _________ Over: ____________________
(STREAM, RAILROAD, OR ROAD)

S-No.:_______________ Design ADT: _______________ ADTT: _______________ Year: _______________

Calculation Page: __________________

2. DUROMETER: 50 ____________________; 60 ____________________; Other: ____________________

3. MINIMUM SHAPE FACTOR: ________________________

4. COMPRESSIVE STRESS

(a) Calculated Maximum Compressive Stress = _______________________________________ MPa {ksi}

(b) Compressive Stress Resistance = ____ MPa {ksi}

5. MAXIMUM CALCULATED COMPRESSIVE DEFLECTION _______________________ mm {in.}

6. ROTATION

(a) Construction Tolerance About Longitudinal Axis: ______________ radians (for spans $\geq$ 30 000 mm {100 ft.})
(b) Total Computed Maximum Rotation About Longitudinal Axis: ______________ radians
(c) Construction Tolerance About Transverse Axis: ______________ radians
(d) Total Computed Maximum Rotation About Longitudinal Axis: ______________ radians

7. BEARING PAD THICKNESS

Plain Bearing Pad:
(a) Maximum bearing pad thickness used on this bridge: __________________________________ mm {in.}
(b) Minimum bearing pad thickness used on this bridge: __________________________________ mm {in.}

Laminated Pad:
(a) Maximum inner layer thickness: ____ mm {in.}
(b) Minimum inner layer thickness: ____ mm {in.}
(c) Total Maximum Height: ____ mm {in.}

8. SHEAR

Calculated Maximum shear deflection = ____ mm {in.}

D-510 (1-96) SHEET NO. 2 OF 2

QUALITY ASSURANCE FORM FOR ELASTOMERIC BEARING DESIGN

Designer: ____ Date: __________

(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

9. STABILITY

(a) For Plain Rectangular Pad

$\frac{\text{Smallest Dimension}}{5}$ = _______________ mm {in.} ≳ Maximum Height of Bearing

(b) For Reinforced Rectangular Bearing Pad

$\frac{\text{Smallest Dimension}}{3}$ = _______________ mm {in.} ≳ Maximum Height of Bearing

(c) For Plain Circular Pad

$\frac{\text{Diameter}}{6}$ = _______________ mm {in.} ≳ Maximum Height of Bearing

(d) For Reinforced Circular Bearing Pad

$\frac{\text{Diameter}}{4}$ = _______________ mm {in.} ≳ Maximum Height of Bearing

10. BEARING SEAT SLOPE

Slope due to net camber and roadway grade in the direction parallel to the longitudinal axis of the beam = _______________ %

(If exceeds 1%, accommodate the remaining slope in the design of the bearing pad or by modifying the bottom of beam area.)

Slope of the bottom of the beam in the direction parallel to the transverse axis of the bearing pad = _______________ %

(For box beams, provide temporary lateral support to the beam during construction until the end diaphragms are cast and the shear blocks or dowel bars are installed if slope exceeds 5% and pads thicker than 90 mm {3.5 in.} are used.)

D-511 (1-96)
REPRODUCE LOCALLY

SHEET NO. 1 OF 3

QUALITY ASSURANCE FORM FOR HIGH LOAD MULTI-ROTATIONAL BEARING DESIGN

Designer: ______________________________ Date: __________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

1. PROJECT INFORMATION

County: ____________ S.R.: ________ Sec.: ________ Over: ______________
(STREAM, RAILROAD, OR ROAD)

S-No.:__________ Design ADT: __________ ADTT: __________ Year: __________

2. BEARING TYPES

Pot __________; Spherical __________; Disc __________; Other/Specify __________

If other, specify the specification used: ______________________________

____

____

3. REACTIONS USED FOR FIXED BEARINGS

Location: ______________________________; Calc. Page __________

Controlling Limit State ______________________________

Maximum Reaction:

Non-Seismic: Vertical: __________ kN {kips}; Horizontal: __________ kN {kips}

Seismic: Vertical: __________ kN {kips}; Horizontal: __________ kN {kips}

Maximum Total Horizontal Reaction at One (1) Substructure Unit: __________ kN {kips}

4. REACTIONS USED FOR EXPANSION BEARINGS

Location: ______________________________; Calc. Page __________

Maximum Reaction: Vertical: __________ kN {kips}; Horizontal: __________ kN {kips}

Minimum Reaction: Vertical: __________ kN {kips}; Horizontal: __________ kN {kips}

Minimum Number of Guided Bearings: ______________________________

5. MAXIMUM MOVEMENT

Thermal Expansion = __________ mm {in.}; Thermal Contraction = __________ mm {in.}

Camber Changes = __________ mm {in.}; Creep and Shrinkage = __________ mm {in.}

D-511 (1-96)
REPRODUCE LOCALLY

SHEET NO. 2 OF 3

QUALITY ASSURANCE FORM FOR HIGH LOAD MULTI-ROTATIONAL BEARING DESIGN

Designer: ______________________________ Date: ___________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

Construction Tolerance = _______________ mm {in.}

TOTAL Maximum Movement = _______________ mm {in.}

6. POT BEARINGS

	FIXED	GUIDED/ EXPANSION
Max. Bearing Pressure on Pot Wall*	________	________
Max. Principal Stress in Pot Wall due to hoop, bending, shear, and compression stresses*	________	________
Stress Resistance in Pot Wall*	________	________
Stress Resistance in Guide Key*..	________	________
Design Rotation	________	________
Depth of Pot Cavity	________	________
Max. Thickness of Pot Beneath Elastomer	________	________
Min. Thickness of Pot Beneath Elastomer	________	________
Thickness of Elastomeric Disc	________	________
Min. Thickness of Elastomeric Disc	________	________
Clear Between Top of Pot and Sliding Surface	________	________
Min. Clear Between Top of Pot and Sliding Surface	________	________

7. CONFIGURATION

Maximum size in mm {in.} and	________	________
Number of Sealing rings*	________	________
Minimum size in mm {in.} and	________	________
Number of Sealing rings*..	________	________

*N/A if BD-613M is used.

D-511 (1-96) SHEET NO. 3 OF 3

QUALITY ASSURANCE FORM FOR HIGH LOAD MULTI-ROTATIONAL BEARING DESIGN

Designer: ______________________________________ Date: ___________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

8. SPHERICAL BEARINGS

To be developed at a later date.

Calc. Page ____________________

9. DISC BEARINGS

To be developed at a later date.

Calc. Page ____________________

D-512 (1-96)
REPRODUCE LOCALLY

SHEET NO. 1 OF 2

QUALITY ASSURANCE FORM FOR PIPES AND CULVERTS

Designer: ____ Date: __________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

1. PROJECT INFORMATION

County: ____________________ S.R.: __________ Sec.: __________ Over: ____________________
(STREAM, RAILROAD, OR ROAD)

S-No.: ______________ Design ADT: ______________ ADTT: ______________ Year: ______________

2. CULVERT TYPE AND GEOMETRY

CULVERT TYPE:	CONCRETE PIPE	REINFORCED CONC. BOX	PLATE PIPE	PLATE PIPE ARCH	METAL BOX
Fill Height:	________	________	________	________	________
Skew:	________	________	________	________	________
Length:	________	________	________	________	________
Size:	________	________	________	________	________

Is fish passage provided? Yes ☐ No ☐ N/A ☐

3. GENERAL

Backfill Unit Density: ____

Railroad Live Load: Yes ☐ No ☐

pH: Foundation Material (Rock/Soil) ____________________; Water ____________________

Method of Corrosion Protection: ____

Anticipated removal of unsuitable material: Yes ☐ No ☐

Anticipated Settlement: ______________________________; Calculation Page______________

Location	Inlet	Outlet
Type of End Wall Provided:	________________	________________
Type of Scour Protection Provided:	________________	________________

D-512 (1-96) SHEET NO. 2 OF 2

QUALITY ASSURANCE FORM FOR PIPES AND CULVERTS

Designer: ______________________________ Date: ____________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

4. DESIGN REQUIREMENTS

Design Life: ______________________ ; Calculation Page ____________

Concrete Pipes:

Design Method: ☐ BD-636M; ☐ Other, specify ____________________

Installation Method: ☐ RC-30M; ☐ Other, specify ____________________

Method of Abrasion Protection: ____________________

Concrete Box Culverts:

Precast ____________________; Cast-in-Place (CIP) ____________________

(If CIP, why?)____________________

Computer program used: ____________________

Method of Abrasion Protection: ____________________

Metal Culverts:

Design Spec.: ____________________

How is Design Life accounted for: Additional Metal ☐ Concrete Paving ☐

Relieving Slab Provided: Yes ☐ No ☐

Tension Strut Required: Yes ☐ No ☐

5. ALTERNATES

Permitted Alternates by Contractor: ____________________

____

6. COMMENTS

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D-513 (1-96) SHEET NO. 1 OF 2

QUALITY ASSURANCE FORM FOR BRADD FINAL PLANS

Designer: ____ Date: ___________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

1. PROJECT INFORMATION

County: ______________ S.R.: __________ Sec.: __________ Over: ____________________
(STREAM, RAILROAD, OR ROAD)

S-No.:______________ Design ADT: ______________ ADTT: ______________ Year: ______________

2. GENERAL

(a) BRADD-3 Version No.: ______________________________________

(b) Which Designer Notes from Sheet Zero are not applicable? ______________________

Give reasons: ______________________________________

____

(c) Do details in contract drawings agree with current supplemental drawings? Yes ☐ No ☐

If no, justify: ______________________________________

____

(d) Indicate controlling rating Moment (M) or Shear (S), with FWS.

LOADING	HL-93	PHL-93	
Rating: (Factor)	IR____M____S____	IR____M____S____	
	OR____M____S____	OR____M____S____	
LOADING	HS	ML-80	P-82
Rating: (Metric Tonnes) {tons}	IR____M____S____	IR____M____S____	
	OR____M____S____	OR____M____S____	OR____M____S____

3. SUPERSTRUCTURE

(a) Beam Data: Beam Type: ____________________ ; No. of Beams: ____________________

Beam Spacing: ____________________; Max. Overhang: ____________________

(b) Bearing Data:

	Fixed	Expansion
Type:	____________________	____________________
Size: (W x L x H):	____________________	____________________

D-513 (1-96) SHEET NO. 2 OF 2

QUALITY ASSURANCE FORM FOR BRADD FINAL PLANS

Designer: ____ Date: ___________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

4. SUBSTRUCTURE

(a) Type of Abutment: Stub ☐ High ☐ Wall ☐

(b) Abutment Stem Steel/Linear m {ft.} ________________________________

5. OTHER

List all Non BRADD-3 Details and Sheet Nos.

Detail	Sheet No.	Detail	Sheet No.

NOTE: Other pertinent Q/A form will apply and must be completed.

D-514 (1-96)
REPRODUCE LOCALLY

SHEET NO. 1 OF 3

QUALITY ASSURANCE FORM FOR SUBSTRUCTURES

Designer: ______________________________ Date: ___________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

1. PROJECT INFORMATION

County: ________________ S.R.: __________ Sec.: __________ Over: ____________________
(STREAM, RAILROAD, OR ROAD)

S-No.: ________________ Skew: ________________ Span(s): ________________

Superstructure Type: ______________________________

2. SUBSTRUCTURE TYPE

	ABUT. 1	PIER 1	PIER 2	PIER 3*	ABUT. 2
Unit Type:	_______	_______	_______	_______	_______
Design Height:	_______	_______	_______	_______	_______
Footing Width:	_______	_______	_______	_______	_______
Footing Length:	N/A	_______	_______	_______	N/A
Footing Thickness:	_______	_______	_______	_______	_______
Parallel Eccentricity Limit:	_______	_______	_______	_______	_______
Maximum Parallel Eccentricity:	_______	_______	_______	_______	_______
Perpendicular Eccentricity Limit:	_______	_______	_______	_______	_______
Maximum Perpendicular Eccentricity:	_______	_______	_______	_______	_______
Maximum Horizontal Load:	_______	_______	_______	_______	_______
Sliding Resistance:	_______	_______	_______	_______	_______
Beam Seat Length: Required:	_______	_______	_______	_______	_______
Provided:	_______	_______	_______	_______	_______
Superstructure Bearings:					
Fixity (E/F):	_______	___/___	___/___	___/___	_______
Type:	_______	___/___	___/___	___/___	_______

D-514 (1-96) SHEET NO. 2 OF 3

QUALITY ASSURANCE FORM FOR SUBSTRUCTURES

Designer: ____ Date: __________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

Comments: ____

____

____

3. FOUNDATION DATA

Date of Foundation Approval: ____________________

	ABUT. 1	PIER 1	PIER 2	PIER 3*	ABUT. 2
Foundation Type:	______	______	______	______	______
Foundation (Load/Pressure)** Resistance:	______	______	______	______	______
Maximum (Load/Pressure)** Design:	______	______	______	______	______
Minimum (Load/Pressure)** Design:	______	______	______	______	______

Comments: ____

____

*Use additional sheets if more than three piers.
**For foundations supported on piles, provide a load value;
For foundations supported on spread footings, provide a pressure.

4. COMPUTER DESIGN PROGRAM

	ABUTMENT DESIGN	PIER DESIGN	SEISMIC ANALYSIS
Program Name:	______________	______________	______________
Version No.:	______________	______________	______________
Vendor:	______________	______________	______________

D-514 (1-96) SHEET NO. 3 OF 3

QUALITY ASSURANCE FORM FOR SUBSTRUCTURES

Designer: ____ Date: ___________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

5. SUBSTRUCTURE APPURTENANCES

Protection Systems ABUTMENTS PIERS

Corrosion: ____________________ ____________________

Collision: ____________________ ____________________

End Treatment

Railing/Fencing: ____

Barrier: ____

Special Backfill/Grading Requirements:

Structural Backfill: ____

Slope Protection: ____

Slope Benching: ____

Slope Grading: ____

Other: ____

Scour Protection

Abutments: ____

Piers: ____

Substructure Drainage:

Abutments: ____

Piers: ____

6. COMMENTS

____

____

____

D-515 (4-00)
REPRODUCE LOCALLY

SHEET NO. 1 OF 2

QUALITY ASSURANCE FORM FOR INTEGRAL ABUTMENT BRIDGES

Designer: ____ Date: ___________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

1. PROJECT INFORMATION

County: ____________________ S.R.: _________ Sec.: _________ Over: ____________________
(STREAM, RAILROAD, OR ROAD)

S-No.:_______________________ Skew: _______________________ Grade: _______________________

Bridge Type: ____

Design ADT: _______________ Year: ______________ ADTT: _______________Year: _______________

No. of Spans and Span Lengths: ____

2. PILE DESIGN

Pile Type/Size: ____________________________; No. of Piles/Beam _____________________________

Total expansion length/thermal movement:

Abutment 1 __________________; Abutment 2 __________________; Calc. Page________________

Estimated Pile Length: Abutment 1 _______________________; Abutment 2 _______________________

Pile Loads: Vertical Resistance ____________________________ Design _________________________

Lateral Resistance ______________________________ Design _________________________

Interaction Ratio for Axial + Bending (A6.9.2.2) ____

Pile Orientation: Abutment 1 ______________________; Abutment 2 _____________________________

3. INTEGRAL ABUTMENT DESIGN

Integral Abutment: Full ☐ Partial ☐

Lateral loads due to passive earth pressure: ____

Has stability of beam erection been checked? Yes ☐ No ☐

Backwall cushion material type/thickness: ____

Has subsurface drainage been addressed? Yes ☐ No ☐

D-515 (4-00) SHEET NO. 2 OF 2

QUALITY ASSURANCE FORM FOR INTEGRAL ABUTMENT BRIDGES

Designer: ____ Date: __________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

Has provision been made for approach slab sliding? Yes ☐ No ☐

Wingwall Design: Independent ☐ Moving with Bridge ☐

Expansion Dam: Type ______________________________; Movement Class ________________

Other special design details: ____

____

4. COMMENTS

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____

____

D-516 (1-96)
REPRODUCE LOCALLY

SHEET NO. 1 OF 3

QUALITY ASSURANCE FORM FOR FINAL DESIGN OF PROPRIETARY RETAINING WALLS

Designer: ____ Date: ___________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

1. PROJECT INFORMATION

County: ______________________ S.R.: __________ Sec.: __________ Over: ____________________________
(STREAM, RAILROAD, OR ROAD)

S-No.: ________________ Design ADT: _______________ ADTT: ________________Year: ________________

2. PROPRIETARY WALL INFORMATION

Wall Type: RE ☐ Foster Geotechnical ☐ Other ☐, indicate other ________________________________

____

Application: Retaining Wall ☐ Abutment ☐

Length: ____________ mm {ft.}; Max. Height ____________ mm {ft.}; Design height ____________ mm {ft.}

Traffic Surcharge = ____________ mm {ft.}

Fill Slope = ____________, Height = ____________ mm {ft.}

3. TECHNICAL INFORMATION

(a) Stability Criteria (External)

Eccentricity (on soil) = ______________________ < 3B/8

Eccentricity (on rock) = ______________________ < B/4

Slope Stability (if req'd) = ______________________ > 1.5

Maximum Horizontal Loads ______________________; Sliding Resistance = ______________________

(b) Max. Foundation Pressure (MPa) {ksf}: Design = ______________; Bearing Resistance = _____________

(c) Stability (Internal)

Pull out force = ______________________; Ultimate Pull Out Capacity = ______________________

Depth Level = ______________________ mm {ft.}

(d) Estimated Settlement (Max.) ______________________ mm {in.}

(e) Least ratio of soil reinforcement length to wall height ______________________;

Location ________________________________

D-516 (1-96) SHEET NO. 2 OF 3

QUALITY ASSURANCE FORM FOR FINAL DESIGN OF PROPRIETARY RETAINING WALLS

Designer: ____ Date: ___________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

(f) Backfill:

	REINFORCED ZONE	RANDOM
Drained Angle of Internal Friction	φ = _______________°;	φ = _____________°
Total Density of Material	= _______________kg/m³ {kcf}	_______________kg/m³ {kcf}

FOR WALL APPLICATION COMPLETE SECTION 4 ONLY

FOR ABUTMENT APPLICATION COMPLETE SECTIONS 4 AND 5

4. WALL APPLICATION

(a) Panel Thickness _______________ mm {in.}; Minimum Cover for Rebars _______________ mm {in.}

(b) Indicate any deviations from standard construction specifications: ________________________________

____

____

(c) Drainage Requirements:

Indicate design assumptions for drainage of backfill material, on top and under the wall: _______________

____

Is drainage provided to validate design assumptions: Yes ☐ No ☐

5. ABUTMENT APPLICATION

(a) Pile Type: Point Bearing ☐ Friction ☐ End Bearing ☐

(b) Pile Size: Steel __________________________________; Concrete ___________________________________

(c) Actual Max. Load/Pile __________________ kN {tons}; Allowable Load/Pile ____________________ kN {tons}

(d) Type of arrangement (Load Transfer Mechanism) of soil-reinforcing elements around piles: ____________

____

____

D-516 (1-96) SHEET NO. 3 OF 3

QUALITY ASSURANCE FORM FOR FINAL DESIGN OF PROPRIETARY RETAINING WALLS

Designer: ____ Date: ___________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

6. COMMENTS

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D-517 (1-96)
REPRODUCE LOCALLY

SHEET NO. 1 OF 5

QUALITY ASSURANCE FORM FOR FINAL DESIGN OF FLEXIBLE RETAINING WALLS

Designer: ______________________________________ Date: ___________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

1. PROJECT INFORMATION

County: ____________ S.R.: ________ Sec.: ________ Along: ______________
(STREAM, RAILROAD, OR ROAD)

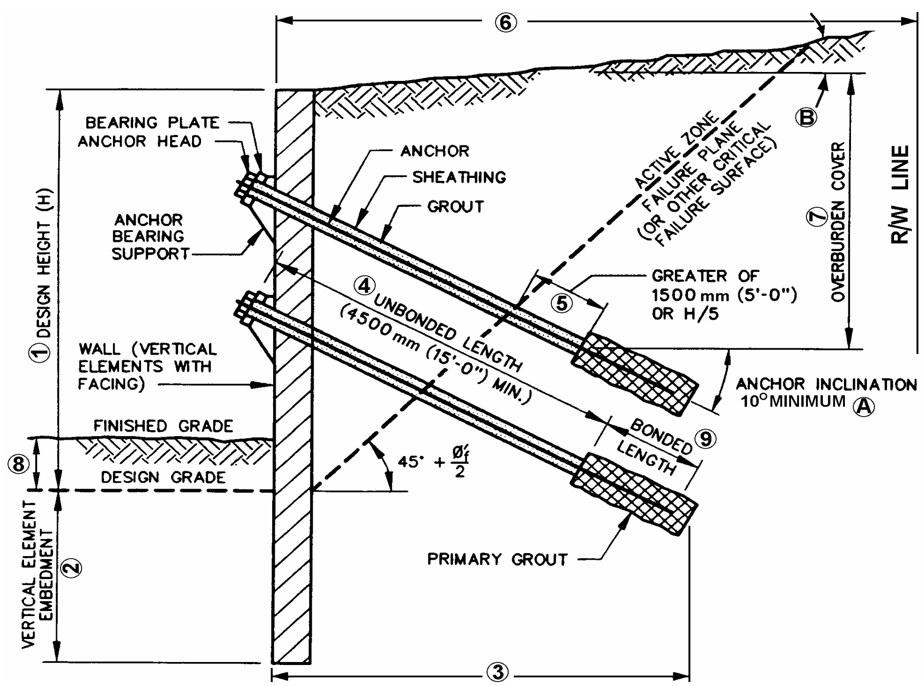
S-No.: __________ Stations: Begin Wall ______________; End Wall ______________

2. GEOMETRIC DESIGN INFORMATION

Wall Type: Permanent ☐ or Temporary ☐

Cantilever ☐ or Tie Back ☐

If Tie Back Wall, No. of Anchors per Vertical Element:



D-517 (1-96) SHEET NO. 2 OF 5

QUALITY ASSURANCE FORM FOR FINAL DESIGN OF FLEXIBLE RETAINING WALLS

Designer: ______________________________ Date: __________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

Design Dimensions —→ 1 = ______________ mm {ft.}; Calc. Page __________

2 = ______________ mm {ft.}; Calc. Page __________

3 = ______________ mm {ft.}; Calc. Page __________

4 = ______________ mm {ft.}; Calc. Page __________

5 = ______________ mm {ft.}; Calc. Page __________

6 = ______________ mm {ft.}; Calc. Page __________

7 = ______________ mm {ft.}; Calc. Page __________

8 = ______________ mm {ft.}; Calc. Page __________

9 = ______________ mm {ft.}; Calc. Page __________

Angles —→ A = ______________ Degrees; Calc. Page __________

B = ______________ Degrees; Calc. Page __________

3. SOIL AND FOUNDATION DATA

In-Situ Soil Type: ______________________; Calc. Page __________

Unit Density = ________ kg/m^3 {kcf}; Cohesion (c) = ______ kg/m^3 {kcf}; Calc. Page __________

Angle of Internal Friction = ______________________; Calc. Page __________

Stability Number (N) (D3.11.5.8P) = ______________ ≤ 3.0; Calc. Page __________

Foundation Material: Soil ☐ Rock ☐ Calc. Page ______________________

Type ______________________; Calc. Page __________

Unit Density = ________ kg/m^3 {kcf}; Cohesion (c) = ______ kg/m^3 {kcf}; Calc. Page __________

Angle of Internal Friction = ______________________; Calc. Page __________

Stability Number (N) (D3.11.5.8P) = ______________ ≤ 3.0; Calc. Page __________

Foundation Design:

Pressure: Resistance = ____________; Actual = ____________; Calc. Page __________

D-517 (1-96) SHEET NO. 3 OF 5

QUALITY ASSURANCE FORM FOR FINAL DESIGN OF FLEXIBLE RETAINING WALLS

Designer: ______________________________ Date: __________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

Settlement: Allowable = ______________; Actual = ______________; Calc. Page __________

Slope Stability Analysis Performed? Yes ☐ No ☐ ; Calc. Page__________

Bearing Resistance = ______________________; Calc. Page __________

Maximum Bearing Pressure = ______________________; Calc. Page __________

Live Load Surcharge Used = ______________________; Calc. Page __________

Earth Pressure Used: Active ☐ Passive ☐ ; Calc. Page__________

4. VERTICAL ELEMENT DATA

Type: ______________; Size: ______________; Calc. Page __________

Spacing: ______________; Embedment Length = ______________; Calc. Page __________

Design Checked for Staged Construction? Yes ☐ No ☐ ; Calc. Page __________

Corrosion Protection ______________________________

Special Details: ______________________________

5. ANCHOR DATA

Type: ______________________; Calc. Page __________

Size: ______________; Spacing: ______________; Calc. Page __________

Loads: Resistance = ______________; Design: ______________; Calc. Page __________

Bond Length based on: Soil ☐ Rock ☐ ; Calc. Page __________

Corrosion Protection ______________________________

Is Anchor Installation Procedure included? Yes ☐ No ☐

D-517 (1-96) SHEET NO. 4 OF 5

QUALITY ASSURANCE FORM FOR FINAL DESIGN OF FLEXIBLE RETAINING WALLS

Designer: ____ Date: ___________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

6. LAGGING AND FACING DATA

Lagging: Temporary ☐ Permanent ☐ ;Calc. Page ____________

Type: ____; Calc. Page ____________

Maximum Design Bending Moment = ______________________________; Calc. Page ____________

Facing: Cast-in-Place Concrete ☐ Precast Concrete ☐ ; Calc. Page ____________

Concrete Class = _____________; Thickness = ____________________; Calc. Page ____________

Reinforcement: Wire Mesh ☐ OR Bars ☐ ; Calc. Page ____________

Plain ☐ OR Epoxy-Coated ☐; Calc. Page ____________

Maximum Design Bending Moment = ______________________________; Calc. Page ____________

Attachment Details Designed? Yes ☐ No ☐; Calc. Page ____________

7. DRAINAGE DETAIL DATA

Are Drainage Panels provided behind wall? Yes ☐ No ☐

Do Drainage Panels extend full height of wall? Yes ☐ No ☐

Is Insulation provided to prevent freeze/thaw damage Yes ☐ No ☐

8. MISCELLANEOUS DATA

If Tie Back Wall is a fill situation, is the necessary approval from the
Chief Bridge Engineer included with the submission? Yes ☐ No ☐

For Tie Back Walls, do the plans and special provisions for this submission contain the proof, performance, creep, and lift off testing of the anchors? Yes ☐ No ☐

D-517 (1-96) SHEET NO. 5 OF 5

QUALITY ASSURANCE FORM FOR FINAL DESIGN OF FLEXIBLE RETAINING WALLS

Designer: ____ Date: ___________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

9. COMMENTS

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7

D-518 (8-05)
REPRODUCE LOCALLY

SHEET NO. 1 OF 4

QUALITY ASSURANCE FORM FOR FINAL DESIGN OF SOUND BARRIERS

Designer: ______________________________________ Date: __________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

1. PROJECT INFORMATION

County: ______________ S.R.: ________ Sec.: ________ Along/Over: ______________________
(ROADWAY, RAMP, ETC.)

S-No.: ____________ Stations: Begin Wall ________________; End Wall ____________________

2. SOUND BARRIER WALL - GENERAL:

Ground-Mounted: Post & Panel ☐ Offset ☐

Structure-Mounted: Bridge-Mounted ☐ Moment Slab-Mounted ☐ Retaining Wall-Mounted ☐

If Bridge-Mounted, Bridge Inspectability Provided? Yes ☐ No ☐

Total Barrier Length: ________________ mm {ft.}

Design Build? Yes ☐ No ☐

If Design Build, Foundation Parameters Provided by Designer? Yes ☐ No ☐

If Design Build, Foundation Parameters Provided by Contractor? Yes ☐ No ☐

Block and Note for District Environmental Manager Signature? Yes ☐ No ☐

Acoustic Profile Indicated? Yes ☐ No ☐

Antigraffiti Coating? Yes ☐ No ☐

If Yes: Limits Indicated: Yes ☐ No ☐

Federal Color Number: ________________

Penetrating Concrete Stain? Yes ☐ No ☐

If Yes: Limits Indicated: Yes ☐ No ☐

Federal Color Number: ________________

Integral Color Pigmentation for Precast Concrete? Yes ☐ No ☐

If Yes: Federal Color Number: ________________

D-518 (8-05) SHEET NO. 2 OF 4

QUALITY ASSURANCE FORM FOR FINAL DESIGN OF SOUND BARRIERS

Designer: ____ Date: __________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

3. POSTS:

Design done by? Standards ☐ New Product Evaluation (PE) ☐ If PE, indicate PE No ___________

Post Type: Steel ☐ Precast Concrete ☐ Other ☐, indicate other___________________________

Post Connection: Baseplate ☐ Embedded ☐ Other ☐, indicate other ________________________

Is Connection Patented? Yes ☐ No ☐

Design Wind Pressure: ________________ kPa {psf}

Design Post Spacing: Minimum ________________ mm {ft.}; Maximum: ________________ mm {ft.}

Design Wall Height: Minimum ________________ mm {ft.}; Maximum: ________________ mm {ft.}

If Concrete Post: Reinforcement? Uncoated ☐ Epoxy-Coated ☐ Galvanized ☐

If Steel Post: Federal Paint Color Number: ________________

4. PANELS:

Design done by? Standards ☐ New Product Evaluation (PE) ☐ If PE, indicate PE No.:___________

Panel Material: Concrete ☐ Steel ☐ Other ☐, indicate other______________________________

Sound Absorptive Material Required? Yes ☐ No ☐ If Yes, indicate PE No.:_________________

Steel Cables thru Panels and Connected to Steel Posts? Yes ☐ No ☐

Design Wind Pressure: ________________ kPa {psf}

Design Post Spacing: Minimum ________________ mm {ft.}; Maximum: ________________ mm {ft.}

Design Panel Height: Minimum ________________ mm {ft.}; Maximum: ________________ mm {ft.}

Structural Panel Thickness: ________________ mm {inch}

Architectural Surface Treatment:

Roadway Side: Surface Treatment: ________________ Average Thickness: ____________ mm {in.}

Residential Side: Surface Treatment: ________________ Average Thickness: ____________ mm {in.}

D-518 (8-05) SHEET NO. 3 OF 4

QUALITY ASSURANCE FORM FOR FINAL DESIGN OF SOUND BARRIERS

Designer: ____ Date: __________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

Total Average Panel Thickness: ____________________mm {in.}

Minimum Number of Lifting Inserts Indicated? Yes ☐ No ☐

Reinforcement? Uncoated ☐ Epoxy-Coated ☐ Galvanized ☐

Access Door Required and Detailed? Yes ☐ No ☐

Sleeve Openings Required and Detailed? Yes ☐ No ☐

5. FOUNDATIONS (for Ground-Mounted Walls only):

Design done by?

Standards ☐ New Product Evaluation (PE) ☐ If PE, indicate PE No.:________________________

Site Specific Design ☐ If Site Specific Design, Specify why? ____________________________

Type: Spread Footing ☐ Drilled Caisson ☐ Other , Specify: ____________________________

Ground Surface: Level ☐ Sloped ☐ If Sloped, Specify slope: ____________________________

Spread Footings:

Design Wind Pressure: __________________ kPa {psf}

Maximum Foundation Pressure (MPa){ksf}: Allowable _________________; Design __________________

Coefficient of Sliding Friction: __________________

Max. Dimensions: W __________________; L __________________; Thickness __________________

Max. Estimated Settlement __________________ mm {in.}

Overexcavation below bottom of footing required? Yes ☐ No ☐

If required, Indicated backfill material type: ____

D-518 (8-05) SHEET NO. 4 OF 4

QUALITY ASSURANCE FORM FOR FINAL DESIGN OF SOUND BARRIERS

Designer: ____ Date: ___________
(DESIGN OFFICE & NAME OF DESIGNER)

System of Units ☐ Metric ☐ U. S. Customary

Drilled Caissons:

Design Wind Pressure: ____________________ kPa {psf}

Soil Type (if per Standards): ____________________

Soil Properties (Site Specific):

Angle of Internal Friction: ____________________ Cohesion kg/m^2 {psf}: ____________________

Unit Weight kg/m^3 {pcf}: __________________ Modulus of Subgrade Reaction kg/m^3 pci}: ______________

e50: ____________________

Design Size: Maximum Diameter ____________________; Maximum Length ____________________

6. OFFSET GROUND MOUNTED WALLS:

Design done by? Standards ☐ New Product Evaluation (PE) ☐ If PE, indicate PE No.:____________

Cables Connections:

Minimum of two cable connections per panel-to-panel connection? Yes ☐ No ☐

Minimum of three cable connections for end panel to adjacent panel(s) connection? Yes ☐ No ☐

7. GENERAL:

Indicate any deviations from standard design, new product evaluation (PE), construction, or material specifications ____

____

____

8. COMMENTS:

____

____

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 2

APPENDIX B - TECHNICAL GUIDELINES FOR SHOP DRAWING REVIEW

The following contains technical guidelines for the review of shop drawings:

(a) For Policy and Procedure related to shop drawing review see PP1.10.2. Use the attached standard transmittal letters as appropriate.

(b) <u>Fabricated Structural Steel</u>

Check the Fabricator's geometrics for conformance to principal dimensions shown on the contract plans. This review does not include the check of detail dimensions.

Check all materials for conformance with the requirements of the contract plans and specifications. This includes painting requirements and material testing requirements.

Check that beams and girders are detailed to provide the camber as required by the contact plans.

Check that the profile of the girders are consistent with the vertical shape of the bridge.

The direction of rolling for splice plates or other small plates which may be easily disoriented should be shown on the shop drawings (see Publication 408, Section 1105.03(s)).

Check all splices, joints and connections for conformance with the intent of the contract plans and specifications. This check shall include conformance of weld types and sizes.

Check all web plates for thickness, including splice locations, and depths, including tapers and end haunches.

Check flange plates for thickness, including splice locations, width, and conformance to minimum length requirements.

Check stiffeners and connection plates for width, thickness, welding, and approximate spacing of intermediate stiffeners if any are required.

Check all bolted field splices for size of splice material and number and size of bolts in bolted material.

Check all cross frame connections for number of bolts in connection plate, length and size of welds, and size and type of members.

Check center of span, field splice, and interior pier ordinates.

Check fillet weld sizes for all weld connections for web to flange, stiffeners to web and flange, and built-up members.

Check number and size of bolts in floor beam, diaphragm, and cross girder connections.

For fracture-critical members, be sure that all necessary information on the design drawings and in the fracture control plan is properly shown on the shop drawings.

Nondestructive testing required for each welded joint type should be listed or identified on shop drawings to inform the shop inspector of appropriate testing and location.

Check that all details follow the latest revision to contract plans.

(c) <u>Grid Floor</u>

Materials furnished must be in accordance with the designer's selection from the manufacturer's catalog as stated in the Special Provisions of the contract. Moment of inertia, section modulus and density of the grid panels must be as required per design. The welding process and electrodes used must be shown. Full bearing at 90% of the grid intersections is required, and all intersections must be welded. These requirements should be noted on the shop drawings.

(d) Expansion and Fixed Dams

Assure that clip angles (or plates) used to support plate and fingerdams on the stringers and diaphragms are of sufficient quality and spacing, and of sufficient depth and correct vertical alignment to permit attachment to pre-drilled holes, inserts or attachment plates in the supporting members.

If adjustable support clips or plates are used, the assemblies must be welded after adjustment; this requirement must be shown on the shop drawings.

On shop drawings for armored neoprene compression seals, the location of the lug bars should be about 15mm {1/2 in.} or more below the compressed height as tested by the Materials and Testing Division of the Bureau of Construction and Materials.

Shop drawings for modular expansion dams, components of which are by themselves structural members (continuous beams), should show all shop splices, if any.

Splices must be made with full penetration welds or partial penetration welds augmented by improved section properties at the splices, if/or as approved by the Bridge Quality Assurance Division.

A "Temperature-Joint Opening" chart ranging from -23° C to 38° C in 5° increments {-10° F to 100° F in 10° increments} must be shown on the shop drawings for neoprene compression seals.

Ensure that the selected seal will be at least 20% compressed at full opening of the joint. Full opening consists of opening at construction temperature, effect of dead load rotation of bearings at the joint, and construction of the superstructure at lowest erection temperature.

The welding process and the electrodes must be shown on the shop drawings.

Special attention must be given to proprietary expansion dams, particularly the support systems. Approved proprietary dams will be listed in Bulletin 15, but such approval is intended primarily for the leakproof and movement capabilities of the dam. The support system for proprietary expansion dams must be equivalent to the support system shown in the Department's expansion dam standard drawings, unless otherwise detailed on the design drawings. When in doubt, contact the Bridge Quality Assurance Division for guidance.

(e) Railings and Barrier Protective Fences

There should be 225mm {9 in.} minimum distance between the centerlines of the railing post and adjacent deflection joints, this should be checked at the time of shop drawing review. Also check that railings are spliced at expansion joints with articulated splices.

(f) Drainage Items

Ascertain that fabricator uses correct piping details at expansion joints. Bolt holes in scupper bases must be compatible with bolt holes in supporting diaphragms or stringers.

(g) Bearings

Check that surface finishes on matching bearing plates are specified. If bearing assemblies require welding on plates which have specified finishes, the finishing should be done after welding is complete. This will assure surface flatness and eliminate the effects of warpage, if any, due to welding.

Check the orientation of the bearings, both relative to the girder as well as to bearing components.

Check that the materials, surface finishes and details for pot bearings are in conformance with contract plans.

(h) Sign Structures

Check for correct member sizes and welding and bolting details. Make sure hand holes are provided in sign structures that are illuminated. Weep holes must be provided at bottom of tower shafts. Ensure that needed camber for trusses is shown. Camber should be provided as per applicable Standard Drawings BD-643M, BD-644M and BD-645M.

(i) Light Poles

Check shop drawings and structural computations for structural capacity only. Electrical details are to be reviewed and approved by the District or Bureau of Design, Highway Lighting Unit. Check for prior light pole approval by light pole manufacture under a General Submission.

(j) Pre-Tensioned and Post-Tensioned Concrete Beams and Panels

The shop drawings must show a framing plan for the whole structure, including proper beam identification for each beam. The force and eccentricity for all beams must conform to the design drawings within reasonable tolerance. Major deviations must be substantiated by calculations submitted by the fabricator with the shop drawings. Check the beam lengths and continuity details against the design assumptions and construction plan.

Concrete release strength and 28-day strength must be shown on the shop drawings, as well as strand patterns and all cast-in hardware, voids or other components. Generally, the Department accommodates prestress fabricators and allows deviations from design plans if allowable stresses are not exceeded and the Department receives a product as good as or better than designed, at no extra cost.

Check that tensile stresses in the top fiber at centerline of bearings of box beams are within allowable stresses, or have been reduced to allowable stresses by either unbonding and/or unbonding supplemented with mild reinforcement.

Check that shear reinforcement is properly spaced in the beams and that epoxy-coated bars are identified.

Shipping weight for each beam should be shown. When the difference between cambers for the Department supplied design and that shown on the shop drawings is significant, which can affect beam seat elevations, the fabricators should inform the Bridge Contractor for corrective measures.

Check for plastic drains, drip notch (if required), chamfers and other miscellaneous details. While a certain amount of minimum end block reinforcing is required in accordance with the Department's Standards, unusual end block conditions, particularly in post-tensioned or prepost-tensioned beams, requires additional reinforcement to inhibit end cracking. Frequently, designers ignore end block details, and it is up to the fabricators to provide a reinforcing cage for heavily stressed end blocks to inhibit cracking and crack propagation.

Check for post-tensioning sequence.

Insert sizes and locations in the beams must be verified, i. e., inserts for attachment of diaphragms, utility supports, lighting fixtures and, occasionally, guide rail connections.

(k) Laminated Shim Neoprene Pads

Check for size, total thickness, layers of neoprene, number of shims, hardness of neoprene, and skew and clip, if any. Make sure quantities of pads include sample pads for testing by the Materials and Testing Division. Pads must be pre-molded.

(l) Permanent Metal Deck Forms

Check against applicable BC-732M that the furnished formwork meets both, section modulus and moment of inertia requirements for the span length shown on the designs drawings (C. to C. stringers spacing less flange width, etc.). If a different pitch, depth or gauge thickness is used, the fabricators shall supply Manufacturer's computations for section modulus, area and moment of inertia for the forms.

Non-composite compression flanges must have flanges encased.

Where the bearing of the form is either at or below the bottom of the top flange, lateral support for the beams flanges may be assumed, and additional flange encasement is not required.

In continuous structures, welding of form supports to flanges in tension zones is not permitted. Design plans for rehabilitation structures frequently do not show tension zones and must be estimated or the data obtained from the designer. Check Special Provisions and design plans for restrictions on the use of metal deck forms. Generally, permanent metal deck forms are only permitted underneath structures. They are not intended for forming the outside of barriers, etc.

Review closure details at joints to assure compliance with permissible details. Details that could result in voids in the slab or difficult concrete placement should not be permitted.

Verify adjustments for camber and beams haunch, minimum gage requirements and configuration of forms.

(m) Metal Plate Culverts

Ascertain that the manufacturer furnishes the culvert specified on the design drawings and the Special Provision. Span and rise should be verified, as well as the gage of the material.

(n) Precast Concrete Culverts

The design drawings and Special Provisions will specify which design standard, if any, is applicable. Wall and slab thicknesses, as well as reinforcement, must be specified from the applicable standard. If the fabricator furnishes the design, a review of the design and approval by the District or the Bridge Quality Assurance Division must precede the shop drawing review. Assure that proper 28-day strength, stripping and shipping strength of concrete are required prior to shipment from a fabrication plant.

(o) Timber Bridges

Check timber sizes, type and principal dimensions. Ascertain that proper hardware sizes are used and treatment of the timber is as specified in the Special Provisions.

(p) Impact Attenuators, R. C. Cribbing

These are proprietary items and shop drawings should be checked against the Design Drawings and Special Provisions to ensure that the item specified is in fact being furnished. Usually this will involve no more than a check of general configuration, i.e., layout, geometry and a check against catalogs furnished by the manufacturer.

(q) Proprietary Retaining Walls (Reinforced Earth, Retained Earth, Doublewall, etc.) and Anchored Pile Walls

Because of the Department's alternate bidding system, the design drawings may present these walls in concept only, with design parameters being given in the Special Provisions. In such cases, the shop drawings may also contain design aspects which must be reviewed. The reviewer should coordinate the shop drawing review with the District Bridge Engineer. Certain aspects may be subjected to a design review before the shop drawing review is started. Ensure that specified corrosion protective system is provided. The 28-day compressive strength is applicable generally for all precast elements.

(r) Quality Assurance

Adherence to the outlined procedures and guidelines will assure that structural materials specified by bridge designers in the contract documents are shown on fabrication drawings and reflect the intent of the design. It is up to the Department's inspection agents in the fabricating plants to assure that the materials and details shown on the shop drawings are, in fact, used during fabrication.

(s) Fabrication Errors

Frequently, errors are made during fabrication, especially in welded structures, which require changes from the original design. These changes must be approved either by the District or the Bridge Quality Assurance Division, and be thoroughly documented. This documentation may require changing the design plans.

(t) Welding Procedure

Routine welding procedures are approved by the Structural Materials Engineer, Materials and Testing Division, Bureau of Construction and Materials. Welding procedures must be reviewed as part of the shop drawing review, with particular attention to fracture-critical members.

(u) Erection Plans

Erection plans are usually part of the regular shop drawing review of structural steel or prestressed concrete shop drawings. Make sure that the erection plans provide data for setting expansion dam openings for temperature, and if found significant, end rotation.

(v) Alternate Design and Value Engineering

Prior to shop drawing review, plans and specifications for alternate designs and value engineered design allowed by the proposal must first be approved by the District or the Bridge Quality Assurance Division. It is not the responsibility of the shop drawing reviewer to make judgment on equivalency of design, except for simple obviously equivalent substitutions of materials, welds, etc.

When proprietary items are used, it is important to ascertain that these items, whether they are taken from catalogs or Manufacturer's standard plans, have prior Department approval. In addition, the shop drawings shall be of standard size and the drafting requirement shall meet the drafting standards in Design Manual, Part 3.

(w) Epoxy-coated Bars

The requirement of additional development length for the epoxy-coated bars should be verified during shop drawing review.

(x) Some suppliers may be trying to utilize unapproved Strip Seal Expansion Dam Retainers, use of which will not assure long-term performance of Strip Seal Dams. Unapproved material should not be permitted through the shop drawing approval process.

Any vendor desiring to have his retainer approved must submit all details to the Bureau of Design, Bridge Division, for review and approval. Once it is approved and incorporated in Bulletin 15, it may be permitted in any contract. Any deviation requires a written approval from the Chief Bridge Engineer.

(y) In related matters on shop drawing reviews, the reviewer must ensure that all material, technical requirements and details indicated in the contract drawings, standards, construction specifications and design specifications (if applicable) are strictly adhered to. Any deviation must be approved in writing by the Director, Bureau of Design.

Sample
COMMONWEALTH OF PENNSYLVANIA
DEPARTMENT OF TRANSPORTATION

PENNDOT

(Use for all shop drawings after first review)

IN REPLY REFER TO

County ______________________ Date __________________

Route ______________________

Station ______________________, S-______________________

Fabricator's Address:

____

____

____

Gentlemen:

With your letter of ____________________________, you submitted prints of shop drawings showing the ____________________________ for the subject structure.

Attached is one set of prints stamped (Accepted/Accepted as Noted, Returned for Corrections).

Please return seven sets of prints of corrected drawings for our file and distribution.

Very truly yours,

Attachment

cc: Contractor: (One set if requested)

____

____

____

Sample
COMMONWEALTH OF PENNSYLVANIA
DEPARTMENT OF TRANSPORTATION

PENNDOT

(Use for distribution of metal deck forms.)

IN REPLY REFER TO

County ______________________ Date __________________

Route ______________________

Station ______________________, S-______________________

Fabricator's Address:

____

____

____

Gentlemen:

With your letter of ____________________________, you submitted prints of shop drawings showing the ____________________________ for the subject structure. The drawings were accepted/accepted as noted on __________________. We have attached ____________ sets of prints.

Since there is no shop inspection of this material, we request that a thorough visual inspection be made at the job site.

Very truly yours,

Attachment

cc: General Contractor:

________________________________ (One Set)

____

____

District Bridge Engineer (One Set)

Sample
COMMONWEALTH OF PENNSYLVANIA
DEPARTMENT OF TRANSPORTATION

PENNDOT

(Use for distribution of all shop drawings, except metal deck forms.)

IN REPLY REFER TO

County ______________________ Date __________________

Route ______________________

Station ______________________, S-______________________

Fabricator's Address:

____

____

____

Gentlemen:

With your letter of ________________________________, you submitted prints of shop drawings sheets _____________ showing _____________________________ for the subject structure.

The drawings were accepted/accepted as noted on _____________________. We have attached ________ sets of prints. One set must be forwarded to the Department of Transportation Shop Inspector.

Very truly yours,

Attachments

cc: Department Shop Inspector

Contractor: (One Set)

____

____

____

Div. of Materials and Testing (One Set)

Attention: ________________________, P.E.

District Bridge Engineer (One Set)

Sample

PERFORMANCE REPORT
CONSULTING ENGINEERS
Confidential

Date of Visit: ___________________________

Visit Made By: ___________________________

Consultant Represented By: ________________

Consultant ____

Address ____

		CONTRACTS Include Agreements being Completed and Current One when Completed		
LEGEND P - Poor F - Fair G - Good E - Excellent Answer all questions and check applicable block.	County			
	S.R. Rte. & Sec.			
	Type (Work)	_____ Shop Drawings	_____ Shop Drawings	_____ Shop Drawings
	Agreement No.			
	Status Work Completed	_____ %	_____ %	_____ %
	Est. Volume of Work	_____ Sheets	_____ Sheets	_____ Sheets

ATTITUDE AND COOPERATION	P	F	G	E	P	F	G	E	P	F	G	E
1. Cooperation with Department												
2. Cooperation with Other Agencies												
3. Attitude Toward Public												
4. Attitude Toward the Work												
5. Capability to Follow and Apply Specifications and Standards												
ORGANIZATION AND MANAGEMENT	P	F	G	E	P	F	G	E	P	F	G	E
1. Adequacy of Personnel												
2. Caliber of Personnel												
3. Caliber of Supervision												
4. Adequacy of Supervision and Management												

WORK PERFORMANCE	Under remarks, explain "No" answers with fault and reasons.					
	YES	NO	YES	NO	YES	NO
1. Did Consultant start on time?						
2. Did Consultant prosecute work diligently?						
3. Did Consultant complete work on schedule?						
4. Were Consultant submissions complete?						
5. Was Consultant in possession of pertinent contract documents?						
6. Indicate status of visits, such as 1st, 2nd, 3rd for each project.						

Remarks:

____

____

____

File original with project file. cc: ______________________

Form completed by: ______________ Date: ______________

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 2

APPENDIX C - PAST AND CURRENT BRIDGE STANDARDS

The available standard drawings are divided into four major categories.

Utmost caution is advised in the use of Standard Drawings and Design Data discussed under a, b and c, since most of the data is based on the concept of working stress or load factor design methods.

(a) Standards for Old Bridges

These standards were assembled primarily for general inspection activities and for the analysis and rating of existing structures. They were issued as an internal publication (not listed in PennDOT Publication Sales Catalog) in five volumes by time period as follows:

(1) Volume 1, from 1918 to 1930, published March 1983
(2) Volume 2, from 1931 to 1940, published May 1983
(3) Volume 3, from 1941 to 1960, published September 1983
(4) Volume 4, from 1961 to 1965, published September 1989
(5) Volume 5, from 1965 to 1972, published November 1989

These standards may be seen as a collection of subject matter for this time period containing construction details, as well as design data.

A limited number of these volumes are inventoried by the Bureau of Design, Bridge Quality Assurance Division.

(b) Available Standard Drawings

Tables C-1 through C-8 present a comprehensive list of available standard drawings. It shall be noted, however, that many of the original designations have been changed. These tables can, therefore, be regarded as showing the historical changes of numbering and which standard drawings are most current.

In Table C-1, the drawing numbers in the columns on the left have an S-designation, and the equivalent ST-designation is shown in the columns on the right. The ST-symbol was used in the 1960's to designate standard drawing series.

Table C-2 lists the ST-100 series standards showing changes in approval dates, changes of drawing numbers, and elimination of individual drawings from the ST-series.

At a later date, the ST-100 series was divided into the ST-100 series for reinforced concrete and steel structures design-related items and into the ST-300 to BC-300 series for construction-related items (Tables C-3 and C-4).

In 1970, the above designations were changed from ST-100 to BD-100 series for design standards, and from ST-300 to BC-300 series for construction standards.

In January 1989, the BD-100 series and the BC-300 series were replaced by the BD-600 series and the BC-700 series, respectively.

The drawings listed in the attached tables were previously available under

(1) Publication No. 3, BD-100 Bridge Design Standards
(2) Publication No. 5, BC-300 Bridge Construction Standards
(3) Publication No. 219, BC-700 Series Standards for Bridge Construction (these supersede the BC-300 Series)
(4) Publication No. 218, BD-600 Series Standards for Bridge Design (these supersede the BD-100 Series)

Half-size drawings of the above are no longer available from the Department's Publication Sales Office At present time, specific old standards can be requested from the Bridge Quality Assurance Division. Eventually all old standards will be added to the Bridge Standards Archives included on BQAD’s Bridge Standards Web Site.

(c) Low Cost Bridge Standards - BLC Series

These standards are available as full-size, ready-to-use contract drawings (blank spaces provided for fill-in field data), as well as half-sized booklets containing a sample problem and instructions for the completion of the full-sized design drawings

These standards have been developed for use on single span bridges to eliminate cumbersome calculations and drafting. To further increase productivity, the entire BLC package, as listed below, has been incorporated into the CADD system. A procedure for statewide usage has been developed and is available on the district level. The application of the BLC Series will do much to expedite the design and detailing of structures.

These plans are approved for use on projects involving liquid fuels funds and conventional funding proportions.

Small modifications may be needed when these plans are used for bridges on higher classes of roadway (additional details for guardrail transition, accommodations for approach slabs).

Short and medium single span structures on tangent alignment are particularly suited for these plans, whereas structures on curvature alignments may require the conventional custom design approach.

In the past, full-sized ready-to-use contract drawings (mylars) had been available through the district offices. Only half-sized booklets for Pub. 6M are available from the Department's Publication Sales Office.

The BLC Series applies to the following span ranges and types of structures:

Pub. No.	Series	Span Range	Type of Structure
17	BLC-500	(90'-130')	Steel Girders, P/S I-Beams, P/S Box Beams
49	BLC-510	(30'-90'	Steel Girders, P/S Box Beams
76	BLC-520	(18'-35')	Steel Beams, P/S Box Beams, P/C Channel Beams
114	BLC-530		Metal Plate Pipe Metal Plate Box/Arch R/C Rigid Frame, P/C R/C Box, R/C Box
130	BLC-540	(30'-90')	Timber on Steel Girders
134	BLC-550	18'-35'	Timber Bridges
6M	BLC-560M	5.5m-30m (18’-98’-5”)	Hardwood Glulam Timber Bridge

(d) Current Standard Drawings

In January 1996, the metric standard drawings, BD-600M Series and BC-700M Series (shown in Tables C-9 and C-10, respectively) became the current standard drawings replacing BD-600 Series and BC-700 Series. These drawings have metric dimensions and are based on the LRFD Specification. However, the Metal Culverts and Concrete Pipe Standards, BD-635M and BD-636M, have been converted to metric dimensions, but their design is based on past AASHTO design specifications. The Overhead Sign Structures Standards (BD-641M through BD-645M) are based on the current AASHTO’s Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals with the latest Interims. This design specification is based on the older working stress design criteria.

In December 1999 and June 2000, many of the metric standard drawings were revised to include U.S. Customary units in parenthesis after the metric values. These dual units standard drawings remain part of the BD-600M Series and BC-700M Series. The corresponding U.S. Customary Standards are now discontinued as indicated in Tables C-7 and C-8.

The drawings listed in Table C-9 and C-10 are available in:

(1) Publication No. 218M - BD-600M Series Standards for Bridge Design

(2) Publication No. 219M - BC-700M Series Standards for Bridge Construction

Table C-1 – List of 1955-1968 Standards (OBSOLETE)

APPROVED DATE		NO.	DESCRIPTION	NO.	APPROVED DATE	
	3-1-61	S-2700 TO S-2703	R.C. SLAB BRIDGES	ST-100	2-25-65	
	11-30-62	SK-1188	R.C. T-BM. BRIDGES	ST-101	2-25-65	
	3-1-61	S-2711 TO S-2715	STEEL I-BM. BRIDGES			
	3-1-61	S-2716 TO S-2720	STEEL COMPOSITE I-BM. BRIDGES			
	8-2-63	S-2730 TO S-2736	STEEL I-BM. BRIDGES	ST-102	2-25-65	10-1-68
			STEEL I-BM. BRIDGES	ST-103	2-25-65	11-1-68
8-2-63	3-1-61	S-2724	STEEL I-BM. BRIDGES DIAPHRAGMS	ST-110	2-25-65	
8-2-63	3-1-61	S-2721	STEEL I-BM. BRIDGES BEARINGS	ST-111	2-25-65	1-2-68
8-2-63	3-1-61	S-2722	STEEL I-BM. BRIDGES PL. EXP. DAM	ST-112	2-25-65	
8-2-63	3-1-61 11-14-55	S-2722S- 1593	STEEL I-BM. BRIDGES TOOTH EXP. DAM	ST-113	2-25-65	
8-2-63	3-1-61	S-2723	BRIDGE SCUPPERS			
			BRIDGE DRAINAGE	ST-114	2-25-65	
8-2-63	3-1-61	S-2727	R.C. ABUT. WITH BACKWALL	ST-120	2-25-65	
8-2-63	11-18-61	S-2728	R.C. ABUT. LAYOUT & DETAILS	ST-120	2-25-65	
8-2-63	3-1-61	S-2710	R.C. ABUT. WITHOUT BACKWALL	ST-121	2-25-65	
	7-5-61	S-2704	R.C. ABUT. FOR CONC. BRIDGES	ST-121	2-25-65	
	12-7-56	S-1612†	R.C. ABUT. DETAILS	ST-122	2-25-65	
8-2-63	3-1-61	S-2726	R.C. RETAINING WALLS	ST-123	2-25-65	
8-2-63	3-1-61	S-2725	ENDWALLS FOR M. PL. CULVERTS	ST-130	2-25-65	
	10-3-62	S-2729	R.C. BOX CULVERTS	ST-131	2-25-65	
S-1614C 10-17-62	S-1614B 5-23-58	S-1614 & S-1614A	PARAPET & PARAPET RAILING			
			ALUM. BRIDGE RAILING	ST-140	2-25-65	
	3-2-59	S-3361	STEEL BRIDGE RAILING	ST-141	2-25-65	
	7-10-63	S-6500	METAL CRIBBING, UNCOATED	ST-142	2-25-65	

Table C-1 – List of 1955-1968 Standards (OBSOLETE) (continued)

APPROVED DATE		NO.	DESCRIPTION	NO.	APPROVED DATE	
	7-10-63	S-6500A	METAL CRIBBING, COATED	ST-143	2-25-65	
			CONCRETE CRIBBING, I & II	ST-144	2-25-65	
	2-13-58	S-1613	ELECTRICAL DETAILS	ST-145	2-25-65	12-10-65
			ALUM. BRIDGE RAILING	ST-146	11-4-65	12-10-65
			STEEL BRIDGE RAILING	ST-147	11-4-65	12-10-65
3-1-66	3-5-65	S-5657*	PARAPET PROTECTIVE FENCE	ST-148	1-2-68	
			ELECTRICAL DETAILS	ST-149	21-2-68	
			ALUM. BRIDGE RAILING	ST-150	1-2-68	
			STEEL BRIDGE RAILING	ST-150	1-2-68	
	9-1-59	S-3361A	SIDEWALK RAILING			

† TITLE = WALL DETAILS
* TITLE = ALUM. CHAIN LINK FENCE

Table C-2 - ST-Series Standards (OBSOLETE)

DESCRIPTION	APPROVED DATE						
MONTH	2	11	12	1	2	9	10
DAY	25	4	10	2	5	23	1
YEAR	65	65	65	68	68	68	68
TABLE OF CONTENTS	-						100
R.C. SLAB BRIDGES	100						E
DECK SLAB DETAILS	-						101
R.C. T-BM. BRIDGES	101						E
STEEL I-BM. BRIDGES	102						102
STEEL I-BM. BRIDGES	103						103
DIAPHRAGMS	110						111
BEARINGS	111			111			112
CLOSED JT. DETAILS	-						113
PL. EXP. DAM	112						114
TOOTH EXP. DAM	113						115
BRIDGE DRAINAGE	114						116
R.C. ABUT. WITH BACKWALL	120						121
R.C. ABUT. WITHOUT BACKWALL	121						122
R.C. ABUT. DETAILS	122						123
R.C. RETAINING WALLS	123						124
ENDWALLS FOR M. PL. CULVERTS	130						131
R.C. BOX CULVERTS	131						132
R.C. ARCH CULVERTS	-						133
ALUM. BRIDGE RAILING	140	146	146	150		312	
STEEL BRIDGE RAILING	141	147	147	151		313	
METAL CRIBBING	142				MC-2*		
METAL CRIBBING	143				MC-1*		
CONCRETE CRIBBING	144				CC-1*		
ELECTRICAL DETAILS	145		145	149		321	
PARAPET PROTECTIVE FENCE				148		301	

E = ELIMINATED
* = ELIMINATED FROM ST-SERIES AND INCLUDED IN THE ROADWAY STANDARD DRAWINGS

Table C-3 - ST-Series Standards (OBSOLETE)

DWG. NO.	DESCRIPTION		APPROVED DATE				
		MONTH	10	3	3	6	12
		DAY	1	18	24	12	17
		YEAR	68	69	69	69	69
ST-100	TABLE OF CONTENTS		X			X	X
ST-101	DECK SLAB DETAILS		X				X
ST-102	COMP. A36 STEEL I-BM. BRIDGES		X				
ST-103	COMP. A441 STEEL I-BM. BRIDGES		X				
ST-111	DIAPHRAGMS		X			X	X
ST-112	BEARINGS		X				X
ST-113	CLOSED JT. DETAILS		X				
ST-114	PLATE EXP. DAM		X				X
ST-115	TOOTH EXP. DAM		X				X
ST-116	BRIDGE DRAINAGE		X				
ST-121	R.C. ABUTMENT		X			X*	
ST-122	R.C. ABUTMENT		X			X*	
ST-123	R.C. ABUTMENT		X				
ST-124	R.C. RETAINING WALLS		X				
ST-131	METAL CULVERTS		X		X		
ST-132	R.C. BOX CULVERTS		X				X
ST-133	R.C. ARCH CULVERTS		X				X
ST-141	WATERSTOP DETAILS			X			

* SHEET 2 REVISED

Table C-4 - ST-Series Standards (OBSOLETE)

DWG. NO.	DESCRIPTION		APPROVED DATE		
		MONTH	9	6	12
		DAY	23	12	22
		YEAR	68	69	69
ST-300	TABLE OF CONTENTS		X	X	
ST-301	PROTECTIVE FENCE *		X	X	
ST-312	ALUM. BRIDGE RAILING		X	X	
ST-313	STEEL BRIDGE RAILING		X	X	
ST-314	ALUM. BRIDGE RAILING		X	X	
ST-315	STEEL BRIDGE RAILING		X	X	
ST-316	PEDESTRIAN RAILING				
ST-317	PEDESTRIAN-TRAFFIC BARRIER			X	
ST-321	ELECTRICAL DETAILS		X	X	
ST-322	LIGHTING POLE ANCHORAGE			X	
ST-331	PRECAST CEM. CONC. BLOCK SLOPE WALL		X		
ST-332	PERM. METAL DECK FORMS				X

*TITLE CHANGED FROM PARAPET PROTECTIVE FENCE TO PROTECTIVE FENCE, 6-12-69

Table C-5 (OBSOLETE)

DWG. NO.	DESCRIPTION	APPROVED DATE					
	MONTH	9	9	11	3	5	9
	DAY	1	1	1	1	12	28
	YEAR	70	72	72	73	76	82
BD-100	INDEX	X				X	
BD-101	CONCRETE DECK SLAB	X					X
BD-102	COMP. A36 STEEL I-BM. BRIDGES	X					
BD-103	COMP. A441 STEEL I-BM. BRIDGE	X				X	
BD-104	MOMENT OF INERTIA GRAPHS	X				X	
BD-105	LIVE LOAD DEFLECTION NOMOGRAPH	X					
BD-105	ALLOWABLE FATIGUE STRESS IN STEEL & GIRDERS						
BD-111	CONCRETE DIAPHRAGM DETAILS	X				X	
BD-112	BEARING PEDESTAL DETAILS					X	
BD-121	R.C. ABUTMENTS WITH BACKWALL	X				X	
BD-122	R.C. ABUTMENTS WITH BACKWALL	X				X	
BD-123	R.C. ABUTMENTS WITHOUT BACKWALL	X				X	
BD-124	R.C. ABUTMENTS WITHOUT BACKWALL	X				X	
BD-125	R.C. ABUTMENTS MISCELLANEOUS DETAILS	X				X	
BD-126	R.C. RETAINING WALLS	X				X	
BD-131	METAL CULVERTS END WALL DETAILS	X				X	
BD-132	R.C. BOX CULVERTS	X					
BD-133	R.C. ARCH CULVERTS	X				X	
BD-201	PRESTRESSED CONCRETE BRIDGE STANDARDS				X		
BD-211	PRESTRESSED CONCRETE BRIDGE STANDARDS – ADJACENT BOX BEAMS		X				

Table C-6 – (OBSOLETE)

DWG. NO.	DESCRIPTION	APPROVED DATE																
	MONTH	7	9	10	4	1	6	6	11	3	4	7	7	11	5	6	7	9
	DAY	1	18	20	1	10	1	20	10	2	1	1	8	4	10	21	12	1
	YEAR	70	70	70	71	72	72	80	80	81	81	81	81	81	82	82	82	83
BC-300	TABLE OF CONTENTS	X												X				
BC-301A	PROTECTIVE FENCE	X								X								
BC-309	PRECAST PARAPET											X						X
BC-311A	ALUMINUM PROTECTIVE BARRIER	X								X								
BC-312	ALUMINUM BRIDGE RAILING	X				X												
BC-313	STEEL BRIDGE RAILING	X				X												
BC-314	ALUMINUM BRIDGE RAILING	X					X											
BC-315	STEEL BRIDGE RAILING	X					X											
BC-316A	ALUMINUM PEDESTRIAN RAILING	X								X								
BC-317A	PEDESTRIAN-TRAFFIC BARRIER	X								X								
BC-318	GUARD RAIL CONNECTIONS TO PARAPETS	X					X											
BC-320	ALUM. OR STEEL BRIDGE HAND RAILING										X							
BC-321A	ELECTRICAL DETAILS	X						X										
BC-322A	LIGHTING POLE ANCHORAGE	X						X										
BC-331A	CEMENT CONC. BLOCK SLOPE WALL	X												X				
BC-332B	PERMANENT METAL DECK FORMS	X												X		X		
BC-334A	THREADED INSERT ANCHOR ASSEMBLY	X								X								
BC-335A	CONSTR. & EXPAN. JOINT DETAILS	X												X				

Table C-6 - (OBSOLETE) (continued)

DWG. NO.	DESCRIPTION	APPROVED DATE																
	MONTH	7	9	10	4	1	6	6	11	3	4	7	7	11	5	6	7	9
	DAY	1	18	20	1	10	1	20	10	2	1	1	8	4	10	21	12	1
	YEAR	70	70	70	71	72	72	80	80	81	81	81	81	81	82	82	82	83
BC-336A	REINF. BAR FABRICATION DETAILS	X												X				
BC-337A	RAILING ANCHOR SYSTEMS	X								X								
BC-338A	STRUC. MOUNTED GUARD RAIL & BARRIERS													X				
BC-339A	BRG. TERMINAL CONN. & INLET PLACEMENT													X				
BC-351	BRIDGE DRAINAGE	X																
BC-352	CONCRETE DECK SLAB DETAILS	X			X													
BC-353	STEEL GIRDER DETAILS				X													
BC-354	STEEL DIAPHRAGMS	X																
BC-355A	BEARINGS	X							X						X			
BC-356A	BEARINGS (FOR CURVED STL. BM. BRG.)								X									
BC-361	PLATE EXPANSION DAM	X																
BC-362	TOOTH EXPANSION DAM	X																
BC-363	ARMORED PREFORMED NEOPRENE COMPRESSION DAM			X														
BC-364	REINFORCED ELASTOMERIC EXPANSION DAM		X															
BC-365	CLOSED JOINT DETAILS				X													
BC-381	RANDOM STONE SLOPE WALL												X					
BC-391A	LOW COST BRIDGE																X	

Table C-7 – Listing of U.S. Customary BD Standard Drawings

Standard	Title	Signature Date
BD-600	INDEX OF STANDARDS FOR BRIDGE DESIGN	1/20/1989
		7/1/1993
		9/30/1994
		1/4/1999
		12/24/1999
		6/30/2000
		12/29/2000
		7/11/2001
		1/21/2003
BD-601	CONCRETE DECK SLAB, DESIGN & DETAILS FOR BEAM BRIDGES	1/20/1989
		8/10/1989
		7/1/1993
		9/30/1994
		6/9/1997
		12/24/1999
BD-602	SOUND BARRIER - CONCRETE POSTS ON SPREAD FOOTINGS, GEOMETRY AND REINFORCEMENT LAYOUT	3/8/1996
		1/2/1998
		6/30/2000
BD-604	GRID REINFORCED CONCRETE BRIDGE DECK	1/4/1999
		6/30/2000
BD-611	CONCRETE DIAPHRAGMS FOR STEEL I-BEAMS	7/1/1993
		9/30/1994
		12/24/1999
BD-621	R/C ABUTMENTS TYPICAL SECTIONS & DETAILS	4/4/1989
		7/1/1993
		8/1/1993
		9/30/1994
		12/24/1999
BD-622	R.C. ABUTMENTS WITH BACKWALLS, LAYOUT & DETAILS	4/4/1989
		7/1/1993
		9/30/1994
		6/30/2000
BD-624	R/C ABUTMENTS WITHOUT BACKWALLS, LAYOUT & DETAILS	4/4/1989
		7/1/1993
		9/30/1994
		6/30/2000
BD-625	R/C ABUTMENTS, MISC. DETAILS	4/4/1989
		7/1/1993
		9/30/1994
		12/24/1999
BD-631	END WALL DETAILS FOR METAL CULVERTS	1/19/1990
		7/1/1993
		9/30/1994
		6/30/2000
BD-632	R/C BOX CULVERTS	1/19/1990
		7/1/1993
		9/30/1994
		6/30/2000
BD-633	R/C ARCH CULVERTS	1/19/1990
		7/1/1993
		9/30/1994
		12/29/2000

Last Approval Date listed indicates when Standard was discontinued.

Table C-7 – Listing of U.S. Customary BD Standard Drawings (continued)

Standard	Title	Signature Date
BD-634	GABION END WALLS	1/19/1990
		7/1/1993
		9/30/1994
		12/29/2000
BD-635	DESIGN TABLES FOR METAL CULVERTS	1/31/1992
		7/1/1993
		9/30/1994
		6/30/2000
BD-636	REINFORCED CONCRETE PIPES	5/23/1994
		6/21/1994
		9/30/1994
		6/30/2000
BD-641	OVERHEAD SIGN STRUCTURES - CANTILEVER & CENTER-MOUNT STRUCTURES STRUT LENGTHS UP TO 40'	10/20/1995
		1/21/2003
BD-642	OVERHEAD SIGN STRUCTURES - TAPERED TUBE STRUCTURES SPANS FROM 30' TO 80'	8/1/1995
BD-643	OVERHEAD SIGN STRUCTURES - 2 POST PLANAR TRUSS SPANS FROM 9144 TO 30 480 (30' TO 100')	8/1/1995
		7/11/2001
BD-644	OVERHEAD SIGN STRUCTURES - 2 POST AND 4 POST TRI-CHORD TRUSS SPANS FROM 60' TO 240'	8/1/1995
BD-651	P/S BRIDGES: TENDONS, DOWELS, SHEAR BLOCKS, DIAPHRAGMS & SKEW	1/20/1989
		6/1/1991
		7/1/1993
		9/30/1994
		12/24/1999
BD-652	P/S BRIDGES: BEAM SIZES & SECTION PROPERTIES	1/20/1989
		9/11/1989
		6/1/1991
		7/1/1993
		9/30/1994
		6/30/2000
BD-653	P/S BRIDGES: FRAMING & DETAILS	1/20/1989
		7/1/1993
		9/30/1994
		6/30/2000
BD-654	P/S BRIDGES: ADJACENT BOX BEAM DETAILS	1/20/1989
		1/16/1990
		6/1/1991
		7/1/1993
		9/30/1994
		6/30/2000
BD-655	P/S BRIDGES, SIMPLE SPANS: TYPICAL SUPERSTR. SECTION	1/20/1989
		6/1/1991
		7/1/1993
		9/30/1994
		12/24/1999
BD-656	P/S BRIDGES, SIMPLE SPANS: TYPICAL LONGITUDINAL SECTIONS	1/20/1989
		9/11/1989
		1/16/1990
		6/1/1991
		7/1/1993
		9/30/1994
		6/30/2000

Last Approval Date listed indicates when Standard was discontinued.

Table C-7 – Listing of U.S. Customary BD Standard Drawings (continued)

Standard	Title	Signature Date
BD-657	P/S BRIDGES: ABUTMENTS (PLANS)	1/20/1989
		9/11/1989
		6/1/1991
		7/1/1993
		9/30/1994
		12/24/1999
BD-658	P/S BRIDGES: PIERS (PLANS)	1/20/1989
		9/11/1989
		7/1/1993
		9/30/1994
		12/24/1999
BD-659	P/S BRIDGES, SIMPLE SPANS: WATERPROOF. AT ABUTMENTS	1/20/1989
		7/1/1993
		9/30/1994
		12/24/1999
BD-660	P/S BRIDGES: DECK SLAB REINF.	1/20/1989
		9/11/1989
		6/1/1991
		7/1/1993
		9/30/1994
		12/24/1999
BD-661	P/S BOX BEAMS: REINF. - DETAILS	1/20/1989
		1/16/1990
		6/1/1991
		7/1/1993
		9/30/1994
		12/24/1999
BD-662	P/S I-BEAMS: REINF. - DETAILS	1/20/1989
		9/11/1989
		7/1/1993
		9/30/1994
		12/24/1999
BD-664	P/S I-BEAMS: CONTINUITY DETAILS FOR LIVE LOAD	1/20/1989
		9/11/1989
		1/16/1990
		6/1/1991
		7/1/1993
		9/30/1994
		12/24/1999
BD-665	P/S BOX BEAMS: CONTINUITY DETAILS FOR LIVE LOAD	1/20/1989
		9/11/1989
		1/16/1990
		6/1/1991
		7/1/1993
		9/30/1994
		12/24/1999
BD-666	P/S BEAMS: DEBONDING DATA & DETAILS	1/20/1989
		7/1/1993
		9/30/1994
		6/30/2000

Last Approval Date listed indicates when Standard was discontinued.

Table C-8 - Listing of U.S. Customary BC Standard Drawings

Standard	Title	Signature Date
BC-700	INDEX OF STANDARDS FOR BRIDGE CONSTRUCTION	1/20/1989
		6/1/1991
		7/1/1993
		9/30/1994
		12/24/1999
		6/30/2000
		12/29/2000
BC-701	PROTECTIVE FENCE	11/15/1989
		7/1/1993
		9/30/1994
		6/30/2000
BC-702	SOUND BARRIER WALLS	9/3/1991
		7/1/1993
		9/30/1994
		1/2/1998
		6/30/2000
BC-711	ALUMINUM PROTECTIVE BARRIER	11/15/1989
		7/1/1993
		9/30/1994
		12/24/1999
BC-716	ALUMINUM PEDESTRIAN RAILING	11/15/1989
		7/1/1993
		9/30/1994
		12/24/1999
BC-720	ALUMINUM OR STEEL BRIDGE HAND RAILING	11/15/1989
		7/1/1993
		9/30/1994
		12/24/1999
BC-721	ELECTRICAL DETAILS	1/20/1989
		7/1/1993
		9/30/1994
		6/30/2000
BC-722	LIGHTING POLE ANCHORAGE	1/20/1989
		7/1/1993
		9/30/1994
		12/24/1999
BC-731	CEMENT CONCRETE SLOPE WALL	1/20/1989
		6/1/1991
		7/1/1993
		9/30/1994
		12/24/1999
BC-732	PERMANENT METAL DECK FORMS	1/20/1989
		7/1/1993
		9/30/1994
		12/29/2000
BC-734	ANCHOR SYSTEMS	11/15/1989
		7/1/1993
		9/30/1994
		12/24/1999

Last Approval Date listed indicates when Standard was discontinued.

Table C-8 - Listing of U.S. Customary BC Standard Drawings (continued)

Standard	Title	Signature Date
BC-735	WALL CONSTRUCTION & EXPANSION JOINT DETAILS	1/20/1989
		7/1/1993
		9/30/1994
		12/29/2000
BC-736	REINFORCEMENT BAR FABRICATION DETAILS	3/1/1989
		6/1/1991
		7/1/1993
		9/30/1994
		12/24/1999
BC-738	STRUCTURE MOUNTED GUIDE RAIL AND CONCRETE BARRIER	11/15/1989
		7/1/1993
		9/30/1994
		12/24/1999
BC-739	BRIDGE TERMINAL CONNECTION & INLET PLACEMENT	6/1/1991
		9/3/1991
		7/1/1993
		9/30/1994
		12/24/1999
BC-751	BRIDGE DRAINAGE DETAILS	1/20/1989
		11/15/1989
		6/1/1991
		7/1/1993
		9/30/1994
		12/24/1999
BC-752	CONCRETE DECK SLAB DETAILS	1/20/1989
		7/1/1993
		9/30/1994
		12/24/1999
BC-753	STEEL GIRDER DETAILS	1/20/1989
		6/1/1991
		7/1/1993
		9/30/1994
		12/24/1999
BC-754	STEEL DIAPHRAGMS	5/9/1989
		6/1/1991
		7/1/1993
		9/30/1994
		12/24/1999
BC-755	BEARINGS	1/20/1989
		7/1/1993
		9/30/1994
		6/30/2000
BC-757	PILE TIP REINFORCEMENT	1/20/1989
		6/1/1991
		12/2/1991
		7/1/1993
		9/30/1994
		12/29/2000

Last Approval Date listed indicates when Standard was discontinued.

Table C-8 - Listing of U.S. Customary BC Standard Drawings (continued)

Standard	Title	Signature Date
BC-758	FIELD SPLICES FOR ROLLED BEAMS	1/20/1989
		5/9/1989
		7/1/1993
		9/30/1994
		12/29/2000
BC-762	TOOTH EXPANSION DAM	1/20/1989
		5/3/1989
		7/1/1993
		9/30/1994
		12/29/2000
BC-766	PREFORMED NEOPRENE COMPRESSION SEAL DAM	1/20/1989
		5/3/1989
		7/1/1993
		9/30/1994
		12/29/2000
BC-767	NEOPRENE STRIP SEAL DAM	1/20/1989
		5/3/1989
		7/1/1993
		9/30/1994
		6/30/2000
BC-781	RANDOM STONE SLOPE WALL	7/1/1993
		9/30/1994
		12/24/1999
BC-782	GABION SLOPE WALL	1/19/1990
		7/1/1993
		9/30/1994
		12/29/2000
BC-783	R.C. BRIDGE DECK REPAIRS	7/1/1993
		7/1/1994
		9/30/1994
		12/29/2000
BC-784	R.C. PARAPET MODIFICATIONS	7/1/1993
		9/30/1994
		12/29/2000
BC-785	BACKWALL REPLACEMENT	7/1/1993
		9/30/1994
		12/29/2000
BC-791	STANDARD LOW COST BRIDGE (P/C DECK & CULVERT)	1/20/1989
		7/1/1993
		7/1/1994
		9/30/1994
		12/29/2000
BC-792	CURB DRAIN DETAILS	1/20/1989
		7/1/1993
		9/30/1994
		6/30/2000
BC-793	PRECAST CHANNEL BEAM	1/20/1989
		7/1/1993
		9/30/1994
		12/24/1999

Last Approval Date listed indicates when Standard was discontinued.

Table C-8 - Listing of U.S. Customary BC Standard Drawings (continued)

Standard	Title	Signature Date
BC-794	UTILITY ATTACHMENT & SUPPORT DETAILS	1/20/1989
		7/1/1993
		9/30/1994
		12/29/2000
BC-795	GEN. NOTES & LEGENDS FOR SOIL/ ROCK DESCRIPTION	1/20/1989
		1/20/1990
		7/1/1993
		9/30/1994
		6/30/2000
BC-798	PRECAST & R.C. BOX CULVERT	1/19/1990
		7/1/1993
		9/30/1994
		12/29/2000
BC-799	PREFABRICATED RETAINING WALL DETAILS	7/1/1993
		9/30/1994
		6/9/1997
		6/30/2000

Last Approval Date listed indicates when Standard was discontinued.

Table C-9 – Listing of Metric/Dual Units BD Standard Drawings

Standard	Title	Signature Date	Status Indicator
BD-600M	INDEX OF STANDARDS FOR BRIDGE DESIGN	1/2/1996	
		1/4/1999	
		12/24/1999	
		6/30/2000	
		12/29/2000	
		7/11/2001	
		5/10/2002	
		1/21/2003	
		4/15/2004	
		7/29/2005	
		7/24/2006	
BD-601M	CONCRETE DECK SLAB	1/2/1996	
		12/24/1999	
		12/29/2000	
		7/11/2001	
		1/21/2003	
		4/15/2004	
		7/29/2005	
		7/24/2006	
BD-602M	SOUND BARRIER WALLS	6/30/2000	
		1/21/2003	
	Replaced by BD-676M to BD-679M	7/29/2005	D
BD-604M	GRID REINFORCED CONCRETE BRIDGE DECK	1/4/1999	
		6/30/2000	
		1/21/2003	
BD-606M	PRECAST SLAB BRIDGES & CULVERTS	12/29/2000	
		1/21/2003	
		7/24/2006	
BD-610M	PA BRIDGE BARRIER	4/15/2004	
		7/24/2006	
BD-611M	CONCRETE DIAPHRAGM DETAILS FOR STEEL I-BEAM STRUCTURES	12/24/1999	
		1/21/2003	
		7/29/2005	
BD-612M	UTILITY ATTACHMENT TO SUPERSTRUCTURE	12/29/2000	
		1/21/2003	
		7/29/2005	
BD-613M	HIGH LOAD MULTI-ROTATIONAL POT BEARINGS	5/10/2002	
		1/21/2003	
BD-614M	R.C. BARRIER MODIFICATION	12/24/1999	
		12/29/2000	
		1/21/2003	D
BD-615M	PA HT BRIDGE BARRIER	1/21/2003	
		7/29/2005	
BD-616M	FIELD SPLICE	7/11/2001	
		1/21/2003	

“D” indicates standard was discontinued on last Approval Date listed.

Table C-9 – Listing of Metric/Dual Units BD Standard Drawings (continued)

Standard	Title	Signature Date	Status Indicator
BD-617M	PA TYPE 10M BRIDGE BARRIER	1/21/2003	
		7/24/2006	
BD-618M	PA VERTICAL WALL BRIDGE BARRIER	1/21/2003	
		7/24/2006	
BD-620M	STEEL GIRDER BRIDGES LATERAL BRACING CRITERIA AND DETAILS	4/15/2004	
BD-621M	REINFORCED CONCRETE ABUTMENTS	1/2/1996	
		12/24/1999	
		1/21/2003	
		4/15/2004	
		7/29/2005	
BD-622M	R.C. ABUTMENTS WITH BACKWALL	1/2/1996	
		6/30/2000	
		1/21/2003	
		7/29/2005	
		7/24/2006	
BD-624M	R.C. ABUTMENTS WITHOUT BACKWALL	1/2/1996	
		6/30/2000	
		1/21/2003	
		7/29/2005	
		7/24/2006	
BD-625M	WINGWALL LENGTH	1/2/1996	
		12/24/1999	
		1/21/2003	
BD-631M	END WALL DETAILS	1/2/1996	
		6/30/2000	
		12/29/2000	
		1/21/2003	
		7/24/2006	
BD-632M	R.C. BOX CULVERT	1/2/1996	
		6/30/2000	
		7/11/2001	
		1/21/2003	
		7/29/2005	
		7/24/2006	
BD-633M	R.C. ARCH DETAILS	1/2/1996	
		12/29/2000	
		1/21/2003	
BD-634M	GABION END WALLS	1/2/1996	
		12/29/2000	
		1/21/2003	
BD-635M	DESIGN TABLES FOR METAL CULVERTS	1/2/1996	
		6/30/2000	
		1/21/2003	

“D” indicates standard was discontinued on last Approval Date listed.

Table C-9 – Listing of Metric/Dual Units BD Standard Drawings (continued)

Standard	Title	Signature Date	Status Indicator
BD-636M	REINFORCED CONCRETE PIPES	1/2/1996	
		6/30/2000	
		1/21/2003	
BD-641M	OVERHEAD SIGN STRS - CANTILEVER & CENTER-MOUNT STRUCTURES STRUT LENGTHS UP TO 12 192 (40')	1/21/2003	
BD-642M	OVERHEAD SIGN STRUCTURES - TAPERED TUBE STRUCTURES SPANS FROM 9 144 TO 24 384 (30' TO 80')	1/21/2003	
BD-643M	OVERHEAD SIGN STRUCTURES - 2 POST PLANAR TRUSS SPANS FROM 9144 TO 30 480 (30' TO 100')	7/11/2001	
BD-643M	OVERHEAD SIGN STRUCTURES - 2 POST PLANAR TRUSS SPANS FROM 9144 TO 30 480 (30' TO 100')	1/21/2003	
BD-644M	OVERHEAD SIGN STRUCTURES- 2 POST & 4 POST TRI-CHORD TRUSS SPANS FROM 18 288 TO 73 152 (60' TO 240')	1/21/2003	
BD-645M	OVERHEAD SIGN STRUCTURES - 4 POST 4 CHORD TRUSS SPANS FROM 30 480 TO 60 960 (100' TO 200')	1/21/2003	
BD-651M	REQUIREMENTS FOR TENDONS, DOWELS, SHEAR BLOCK, DIAPHRAGMS, SKEW LIMITATIONS AND BACKWALLS	1/2/1996	
		12/24/1999	
		12/29/2000	
		1/21/2003	
		7/29/2005	
BD-652M	BEAM SIZES AND SECTION PROPERTIES	1/2/1996	
		6/30/2000	
		1/21/2003	
		4/15/2004	
BD-653M	TYPICAL FRAMING PLANS AND DETAILS	1/2/1996	
		6/30/2000	
		1/21/2003	
		7/29/2005	
BD-654M	ADJACENT BOX BEAM DETAILS	1/2/1996	
		6/30/2000	D
BD-655M	TYPICAL SUPERSTRUCTURE SECTIONS	1/2/1996	
		12/24/1999	
		7/11/2001	
		1/21/2003	
		7/29/2005	
		7/24/2006	
BD-656M	TYPICAL LONGITUDINAL SECTIONS	1/2/1996	
		6/30/2000	
		7/11/2001	
		1/21/2003	
		7/29/2005	
		7/24/2006	
BD-657M	I-BEAM AND BOX BEAM BRIDGES	12/24/1999	
		1/21/2003	
		7/24/2006	

"D" indicates standard was discontinued on last Approval Date listed.

Table C-9 – Listing of Metric/Dual Units BD Standard Drawings (continued)

Standard	Title	Signature Date	Status Indicator
BD-658M	SHEAR BLOCK DETAILS AT PIER - P/S CONCRETE I-BEAMS AND BOX BEAM BRIDGES	1/2/1996 12/24/1999 1/21/2003 7/29/2005	
BD-659M	~~TYPICAL ABUTMENT WATERPROOFING DETAILS~~ (Details moved to BC-788M) REINFORCED CONCRETE PIERS	~~1/2/1996~~ ~~12/24/1999~~ 7/24/2006	~~D~~
BD-660M	DECK SLAB AND STEEL REINFORCEMENT PLACEMENT	1/2/1996 12/24/1999 1/21/2003 7/29/2005 7/24/2006	
BD-661M	BOX BEAM REINFORCEMENT DETAILS	1/2/1996 12/24/1999 1/21/2003 4/15/2004 7/29/2005 7/24/2006	
BD-662M	I-BEAM REINFORCEMENT DETAILS	1/2/1996 12/24/1999 12/29/2000 1/21/2003 4/15/2004 7/29/2005 7/24/2006	
BD-664M	CONTINUITY FOR LIVE LOAD DETAILS - I-BEAM BRIDGES	1/2/1996 12/24/1999 12/29/2000 1/21/2003 4/15/2004 7/29/2005	
BD-665M	CONTINUITY FOR LIVE LOAD DETAILS - BOX BEAM BRIDGES	1/2/1996 12/24/1999 1/21/2003 7/29/2005	
BD-667M	INTEGRAL ABUTMENT	12/24/1999 12/29/2000 1/21/2003 7/29/2005 7/24/2006	
BD-668M	PRECAST CHANNEL BEAM BRIDGES	12/24/1999 1/21/2003	
BD-676M	GROUND MOUNTED SOUND BARRIERS - PRECAST CONCRETE PANELS	7/29/2005	

“D” indicates standard was discontinued on last Approval Date listed.

Table C-9 – Listing of Metric/Dual Units BD Standard Drawings (continued)

Standard	Title	Signature Date	Status Indicator
BD-677M	GROUND MOUNTED SOUND BARRIERS - PRECAST CONCRETE POSTS	7/29/2005	
BD-678M	GROUND MOUNTED SOUND BARRIERS - STEEL POSTS	7/29/2005	
BD-679M	STRUCTURE MOUNTED SOUND BARRIER WALLS	7/29/2005	
BD-680M	OFFSET SOUND BARRIER WALLS	7/29/2005	

"D" indicates standard was discontinued on last Approval Date listed.

Table C-10 – Listing of Metric/Dual Units BC Standard Drawings

Standard	Title	Signature Date	Status Indicator
BC-700M	INDEX OF STANDARDS FOR BRIDGE CONSTRUCTION	1/2/1996	
		12/24/1999	
		6/30/2000	
		12/29/2000	
		7/11/2001	
		1/21/2003	
		9/4/2003	
		4/15/2004	
		7/29/2005	
		7/24/2006	
BC-701M	PROTECTIVE FENCE	1/2/1996	
		6/30/2000	
		1/21/2003	
		7/29/2005	
		7/24/2006	
BC-702M	SOUND BARRIER WALLS	1/2/1996	
		6/30/2000	
		1/21/2003	
	Replaced by BC-776M to BC-780M	4/15/2004	
		7/29/2005	D
BC-703M	THRIE-BEAM TO VERTICAL WALL BRIDGE BARRIER TRANSITION CONNECTION	1/21/2003	
		7/29/2005	
BC-707M	PA HT BRIDGE BARRIER MISC. DETAILS	1/21/2003	
		7/29/2005	
		7/24/2006	
BC-708M	THRIE-BEAM TO PA TYPE 10M BRIDGE BARRIER TRANSITION CONNECTION	1/21/2003	
		7/29/2005	
		7/24/2006	
BC-709M	PA TYPE 10M BRIDGE BARRIER MISC. DETAILS	1/21/2003	
		7/29/2005	
		7/24/2006	
BC-711M	ALUMINUM PROTECTIVE BARRIER	1/2/1996	
		12/24/1999	
		1/21/2003	
		7/24/2006	
BC-712M	PA BRIDGE BARRIER TO THRIE BEAM GUIDERAIL TRANSITION	4/15/2004	
		7/29/2005	
BC-713M	PA BRIDGE BARRIER - MISCELLANEOUS DETAILS	4/15/2004	
		7/29/2005	
		7/24/2006	
BC-716M	ALUMINUM PEDESTRIAN RAILING	1/2/1996	
		12/24/1999	
		1/21/2003	
		4/15/2004	
		7/24/2006	

"D" indicates standard was discontinued on last Approval Date listed.

Table C-10 – Listing of Metric/Dual Units BC Standard Drawings (continued)

Standard	Title	Signature Date	Status Indicator
BC-718M	ALTERNATE RAILING DETAILS	1/2/1996	
		12/24/1999	
		1/21/2003	
		7/29/2005	
BC-719M	TEMPORARY CONCRETE BARRIER, STRUCTURE MOUNTED	1/21/2003	
		9/4/2003	
		4/15/2004	
		7/29/2005	
BC-720M	ALUMINUM OR STEEL BRIDGE HAND RAILING	1/2/1996	
		12/24/1999	
		1/21/2003	
		4/15/2004	
BC-721M	ELECTRICAL DETAILS	1/2/1996	
		6/30/2000	
		7/11/2001	
		1/21/2003	
		7/29/2005	
BC-722M	LIGHTING POLE ANCHORAGE	1/2/1996	
		12/24/1999	
		1/21/2003	
		7/24/2006	
BC-723M	BRIDGE ANTI-ICING SYSTEM	7/29/2005	
BC-731M	CEMENT CONCRETE SLOPE WALL	1/2/1996	
		12/24/1999	
		1/21/2003	
		7/29/2005	
BC-732M	PERMANENT METAL DECK FORMS	1/2/1996	
		12/29/2000	
		1/21/2003	
		4/15/2004	
		7/29/2005	
		7/24/2006	
BC-734M	ANCHOR SYSTEMS	1/2/1996	
		12/24/1999	
		1/21/2003	
		4/15/2004	
BC-735M	WALL CONSTRUCTION & EXPANSION JOINT DETAILS	1/2/1996	
		12/29/2000	
		1/21/2003	
BC-736M	REINFORCEMENT BAR FABRICATION DETAILS	1/2/1996	
		12/24/1999	
		1/21/2003	

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Table C-10 – Listing of Metric/Dual Units BC Standard Drawings (continued)

Standard	Title	Signature Date	Status Indicator
BC-738M	STRUCTURE MOUNTED GUIDE RAIL AND CONCRETE BARRIER	1/2/1996	D
		12/24/1999	
BC-739M	BRIDGE BARRIER TO GUIDE RAIL TRANSITION	1/2/1996	
		12/24/1999	
		12/29/2000	
		7/11/2001	
		1/21/2003	
		7/29/2005	
		7/24/2006	
BC-741M	OVERHEAD SIGN STRS - CANTILEVER & CENTER-MOUNT STRUCTURES STRUT LENGTHS UP TO 12 192 (40')	1/21/2003	
BC-742M	OVERHEAD SIGN STRUCTURES - TAPERED TUBE STRUCTURES SPANS FROM 9 144 TO 24 384 (30' TO 80)'	1/21/2003	
BC-743M	OVERHEAD SIGN STRUCTURES - 2 POST PLANAR TRUSS SPANS FROM 9144 TO 30 480 (30' TO 100')	1/21/2003	
BC-744M	OVERHEAD SIGN STRUCTURES - 2 POST & 4 POST TRI-CHORD TRUSS SPANS FROM 18 288 TO 73 152 (60' TO 240')	1/21/2003	
BC-745M	OVERHEAD SIGN STRUCTURES - 4 POST 4 CHORD TRUSS SPANS FROM 30 480 TO 60 960 (100' TO 200')	1/21/2003	
BC-751M	BRIDGE DRAINAGE	1/2/1996	
		12/24/1999	
		7/11/2001	
		1/21/2003	
		4/15/2004	
		7/29/2005	
BC-752M	CONCRETE DECK SLAB DETAILS	1/2/1996	
		12/24/1999	
		1/21/2003	
		7/29/2005	
BC-753M	STEEL GIRDER DETAILS	1/2/1996	
		12/24/1999	
		12/29/2000	
		1/21/2003	
		7/29/2005	
BC-754M	STEEL DIAPHRAGMS	1/2/1996	
		12/24/1999	
		1/21/2003	
		7/29/2005	
		7/24/2006	
BC-755M	BEARINGS	1/2/1996	
		6/30/2000	
		1/21/2003	
		7/29/2005	
BC-757M	STEEL PILE TIP REINFORCEMENTS & SPLICES	1/2/1996	
		12/29/2000	
		1/21/2003	

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Table C-10 – Listing of Metric/Dual Units BC Standard Drawings (continued)

Standard	Title	Signature Date	Status Indicator
BC-762M	TOOTH EXPANSION DAM	1/2/1996 12/29/2000 1/21/2003 7/29/2005 7/24/2006	
BC-766M	PREFORMED NEOPRENE COMPRESSION SEAL JOINT	1/2/1996 12/29/2000 1/21/2003	
BC-767M	NEOPRENE STRIP SEAL DAM FOR PRESTRESSED CONCRETE & STEEL I-BEAM BRIDGES	6/30/2000 7/11/2001 1/21/2003 7/29/2005 7/24/2006	
BC-771M	STEEL INTERMEDIATE DIAPHRAGMS FOR PA. I-BEAM PRESTRESSED BRIDGES	1/2/1996 12/29/2000	D
BC-775M	MISCELLANEOUS PRESTRESS DETAILS	1/2/1996 6/30/2000 7/11/2001 1/21/2003 4/15/2004 7/29/2005	
BC-776M	GROUND MOUNTED SOUND BARRIERS - PRECAST CONCRETE PANELS	7/29/2005	
BC-777M	GROUND MOUNTED SOUND BARRIERS - PRECAST CONCRETE POSTS	7/29/2005	
BC-778M	GROUND MOUNTED SOUND BARRIERS - STEEL POSTS	7/29/2005	
BC-779M	STRUCTURE MOUNTED SOUND BARRIER WALLS	7/29/2005	
BC-780M	OFFSET SOUND BARRIER WALLS	7/29/2005	
BC-781M	RANDOM STONE SLOPE WALL	1/2/1996 12/24/1999 7/11/2001 1/21/2003	
BC-782M	GABION SLOPE WALL DETAILS	1/2/1996 12/29/2000 1/21/2003	
BC-783M	REINFORCED CONCRETE REPAIR	1/2/1996 12/29/2000 1/21/2003	
BC-784M	R.C. PARAPET MODIFICATIONS	1/2/1996 12/29/2000	D
BC-785M	BACKWALL MODIFICATION DETAILS	1/2/1996 12/29/2000	D

“D” indicates standard was discontinued on last Approval Date listed.

Table C-10 – Listing of Metric/Dual Units BC Standard Drawings (continued)

Standard	Title	Signature Date	Status Indicator
BC-788M	TYPICAL WATERPROOFING AND EXPANSION DETAILS	12/24/1999	
		6/30/2000	
		12/29/2000	
		1/21/2003	
BC-791M	LOW COST BRIDGE	1/2/1996	
		12/29/2000	D
BC-792M	CURB DRAIN DETAILS	1/2/1996	
		6/30/2000	D
BC-793M	PRECAST CHANNEL BEAM BRIDGES	1/2/1996	
		12/24/1999	D
BC-794M	UTILITY ATTACHMENT & SUPPORT DETAILS	1/2/1996	
		12/29/2000	
		1/21/2003	
BC-795M	GENERAL NOTES AND LEGENDS FOR SOIL / ROCK DESCRIPTION	1/2/1996	
		6/30/2000	
		1/21/2003	
BC-798M	MECHANICAL CONNECTION DETAILS	1/2/1996	
		12/29/2000	
		7/11/2001	
		1/21/2003	
		4/15/2004	
		7/29/2005	
		7/24/2006	
BC-799M	MECHANICALLY STABILIZED EARTH RETAINING WALLS	6/30/2000	
		12/29/2000	
		1/21/2003	
		4/15/2004	
		7/29/2005	
		7/24/2006	

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APPENDIX D - CHECKLIST FOR CONTRACTOR'S DESIGN ALTERNATE - CONCEPTUAL DESIGN REVIEW

Contractor's Design Alternates -- Conceptual Design

The following items shall be addressed during conceptual design reviews:

(a) The completeness of the submitted package.

(b) The inclusion of comments from the reviewing Consultant and the District. Central Office review shall be complete by the time the comments from the District and Consultant come in if simultaneous submissions are made.

(c) The computer program used in the analysis - state clearly which program is being used. The agreement of Design/Analysis Methodology with Special Provision Design Methodology (line girder, 2-D, 3-D).

(d) The pier/abutment arrangement - has it been changed from the original scheme (check Special Provisions).

(e) The provision of core boring data if required by the contract.

(f) The provision of substructure design data - approved soil bearing resistance parameters, and approved pile resistance parameters shall be same as in original design.

(g) The clear indication of pouring sequence - conceptual drawings shall show it.

(h) Debonding - the use of up-to-date criteria.

(i) Bearings - have they been altered.

(j) Joint layout - has it been altered. (Number of joints may be decreased, but not increased).

(k) Diaphragm arrangement is to be checked closely; it shall meet LRFD requirements.

(l) Approval letter must contain the following sentence: "Approval of concept is subject to review of detail plans/design."

(m) Utility arrangement is also to be compared to original design.

(n) Type of expansion joints shall be approved at conceptual stage.

(o) Expansion joints - check versus standard drawings.

(p) The submission shall be treated as a combination of TS&L and foundation with emphasized attentions toward the original design.

(q) Deck and end structure drainage shall be the same as in as-designed bridge plans.

(r) Approval letter shall contain all TS&L items, girder spacing, foundation items (not as detailed as for regular approval) and design methodology.

(s) Construction methods for untypical structures.

(t) Awareness of problem areas - ask for early partial submittals on problem areas.

(u) Flood clearance.

(v) Commitment of the Contractor to his contract pile quantity in the approval letter, if alternate design uses as-designed piles. Pile quantity shall be approved in accordance with the Special Provisions. Contractor shall be notified if the required quantities are different from his bid quantity so he can withdraw his concept if he must install a significant amount of piling without compensation.

(w) Steel structures - verify that web buckling criteria and deck pouring sequence were examined as required.

(x) Corrosion protection of substructure in accordance with current Department criteria.

(y) Pile driving requirements shall be checked if as-designed piles were not used.

(z) Introduction of fatigue prone details - do not approve without approval of Chief Bridge Engineer.

(aa) Fracture-critical concepts - do not allow unless included in as-designed bridge plans.

(bb) Department criteria shall be checked to determine whether a backwall is required or an end diaphragm may be provided.

(cc) A list must accompany the submissions that states all variations from the as-designed bridge plans. Anything not listed must conform to the as-designed plans.

(dd) If light poles are located on the structure, the rotation and deflection of the structure at the light pole locations for the Contractor's alternate must be less than the structure rotation and deflection at the light pole locations for the as-designed structure. Calculations must show this.

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APPENDIX E – BRIDGE MANAGEMENT WHITE PAPER

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ASSET DEFINITION

PennDOT owns 25,353 bridges greater than or equal to 8 feet in length, which is equivalent to more than 110 million square feet (MSF) of deck area. Of the 25,353 bridges, 15,405 (105 MSF) are greater than 20 feet in length which is the Federal Highway Administration's (FHWA) definition of a bridge. 2,681 bridges are on the Interstate system and 3,290 more are on the National Highway System. PennDOT's bridge inventory consists of most all structure types including significant arch or truss bridges crossing major rivers, but the predominant bridge type is an I-shape girder constructed of either steel or concrete. As an indication of the value of the bridge assets, the replacement cost of all bridges is more than $33 billion for the construction alone. PennDOT's Bridge Improvement and Preservation Construction Program for 2006 was approximately $528 million. Bridges comprise the majority of structural assets; however other structural assets that are managed (inventoried and inspected) are sign structures, walls and tunnels.

GOALS AND POLICIES

The overarching goals of PennDOT's bridge program include:

- Ensuring the bridges are safe for the efficient movement of people and goods.
- Reducing the backlog of bridge deficiencies to the national average by CY 2025 - more specifically, as measured by the deck area of bridges that are Structurally Deficient.
- Using good practice in design, construction, and maintenance to sustain the continual improvement of our bridges in a cost effective manner and move towards a bridge life of 100 years.

Our holistic approach to bridge management includes timely inspection, load rating analysis, on-demand structural repairs, preservation, and routine maintenance to maximize the life of a bridge. For bridge improvements, PennDOT is incorporating a design approach and details that will result in a 100-year service life. (See Table 1)

The policies governing the bridge program are:

Safety - Inspection and load rating analysis

Federal Requirement - National Bridge Inspection Standards (NBIS), Code of Federal Regulations Chapter 23 Part 650, Subpart c.

PennDOT Requirements - Publication 238 Bridge Safety Inspection Manual

100 Year Life - Design and Construction

Federal Requirement - AASHTO LRFD Bridge Design Specification, Title 23 Section 144 Highway Bridge Program

PennDOT Requirements - Publication 15M Design Manual Part 4, Publication 218M Standard Drawings for Bridge Design (BD-600M series), Publication 219M Standard Drawings for Bridge Construction (BC-700M series) and Publication 408

Bridge Improvements and Preservation - Planning and Programming

Federal Requirement - Federal Highway Bridge Program (HBP), Code of Federal Regulations Chapter 23 Part 650, Subpart d.

Maintenance - Address critical structural repairs, repair of leaking deck joints; routine cleaning and flushing of joints, decks, scuppers, and beam seats.

Federal Requirement - 23 USC (United States Code) 116

PennDOT Requirements - Publication 55 Procedures and Standards for Bridge Maintenance, Publication 23 Maintenance Manual, ASHMA Budget Allocation

INVENTORY

Inventory and location data is collected by PennDOT and stored in its Bridge Management System 2 (BMS2) for all state-owned bridges greater than or equal to 8 feet in length. PennDOT collects and stores all data as required by the FHWA's "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges," (December, 1995) for compliance with the Federal NBIS regulations. Examples of the inventory data are: Structure Type, Structure Length, Structure Width, Deck Type, Year Built, Latitude, Longitude, Features Intersected Data, Hydrologic Data and Posting Data. All bridge inventory data is entered into BMS2 prior to the initial inspection. All bridges are inspected at a maximum frequency of every 2 years, and changes in the inventory data are noted and collected in the field.

The Department also collects additional information for bridges that is not required by FHWA. Examples include: Design and Shop Drawing Numbers, State Senatorial and Congressional Districts, Detailed Railroad Information, Design and Material Information and Fracture Critical Information. Information is also collected for sign structures, walls and tunnels and is stored in BMS2.

PennDOT has just implemented its new Pontis based BMS2 in November 2006. New data fields have been added to store more information in order to meet the Department's needs. The new fields include: additional Inspection Planning Information, Material Testing Results, and additional Load Rating Information.

CONDITION ASSESSMENT AND PERFORMANCE MODELING

Inspection and appraisal data for bridges is also collected by PennDOT and stored in BMS2 for all state-owned bridges greater than or equal to 8 feet in length. PennDOT collects and stores all inspection data as required by the FHWA's "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges," (December, 1995).

Examples of the inspection data quantifying the condition of a bridge are:

- Superstructure Rating
- Substructure Rating
- Deck Rating
- Observed Scour Rating
- Load Capacity Rating

In addition to the FHWA required inspection data, PennDOT collects more detailed condition data on its bridges, such as Paint Condition.

The Department also requires bridge inspectors to recommend maintenance items from a pre-determined list of repair items.

The frequency of bridge inspections is every 2 years or less, depending on the condition of the structure. Inspectors collect and enter the inspection data into electronic data collectors (EDC). The use of EDCs has greatly expedited the transfer of data from the field to the office through an automated process. Once the data is entered and then approved by the Bridge Inspection Supervisors, it is available in BMS2 for Department personnel to run reports for their use. District Bridge Units typically query selected inspection, inventory and maintenance data items for their planning and programming of bridge work.

The Department has initiated a process to develop predictive models using Pontis/BMS2. The first step in this process was to define bridge elements at a more detailed level than those currently inspected, and train PennDOT inspectors to conduct element-level inspections. The Department will conduct its initial element-level inspections in the spring of 2007. Accumulating condition history data on homogeneous elements of structures will allow the Department to build predictive deterioration models. The next steps are to develop interim models based on expert judgment that can be improved over time. These predictive models are required in order to utilize the simulation and optimization features of Pontis.

ALTERNATIVES EVALUATION AND OPTIMIZATION (DECISION MAKING PROCESSES)

Bridge improvements are identified and prioritized at the District level based on structural condition, sufficiency rating and repair needs identified from recent inspections.

Bridge preservation projects are prioritized based on work type (scour correction, leaking joints, elimination of the most urgent maintenance priorities – e.g. maintenance items with priorities of 0’s and 1’s) and road category as defined by the Business Plan Network.

For bridge improvements the funding is developed with Planning Organizations, Transportation Improvement Program (TIP), Bridge Bill and Twelve Year Plan (TYP).

The Pontis capabilities of BMS2 will be used in the future for alternatives evaluation and optimization.

IMPLEMENTATION (WHAT IS DONE AND HOW)

The major treatment categories for bridges are as follows:

- Maintenance - on-demand repairs to restore damaged or deteriorated members to a minimum level of safety performance, or small-scale preventive maintenance actions to ensure the bridge members realize their expected service life. Maintenance is generally limited to cleaning activities and smaller scale preventative maintenance items (e.g. sealing joints, drainage repairs) at each bridge. Work may be done by Department forces or by outside contractors.
- Preservation - actions to extend the life of a bridge without improving bridge functionality or performance (keeping good bridges good). The scale of work tends to be much larger than Maintenance work, with a longer extension of bridge life expected. Examples of preservation work items include deck joint replacement, beam end repairs, deck overlays, painting and fatigue retrofits. Most preservation projects are performed by contractors; some are performed by Department Maintenance crews.
- Improvement - major rehabilitation or replacement of bridges that require an improvement in bridge safety and/or performance over an extended period of time (30+ years). The need for improvement may be dictated by structural concerns and/or functional needs of the highway system. Improvements include structure replacement or major rehabilitation. Most Improvement projects are typically performed by contractors, but there is an occasional bridge replacement or rehabilitation performed by County Maintenance crews.

All bridge work to be performed takes into account the needs of other assets (e.g. roadways, signs, etc.) so that work for these items may be coordinated and implemented at the same time to reduce costs (e.g. mobilization, traffic control, etc.).

PERFORMANCE MONITORING

The following measures and targets are included in the annual Business Plans and DE Dashboard or Scorecard:

- Achieve the Bridge Program Lets in Dollar Amount- Set Yearly by Executive Staff
- Achieve the Number Bridge Program Lets - Set Yearly by Executive Staff
- Percent of Structurally Deficient (SD) Bridges, Measured by Deck Area for State-Owned Bridges ≥ 8 feet - Reduce SD Deck Area to 10% (national average) by 2025. A linear decrease of 0.5% of SD deck area per year is required to achieve this goal.
- Percentage of completed 0 and 1 Maintenance Priorities for Bridge Maintenance Category – Goal is to complete 10% of the items in a year.
- Percentage of completed 0 and 1 Maintenance Priorities – Goal is to complete 5% of the items in a year.
- Compliance with NBIS Inspection Frequency - 98% within 24 months, 100% within 25 months.
- Monitor Bridge Analysis Backlog - 90 day cycle time on load rating analysis

Improvements to performance monitoring under consideration are: 85% of bridge improvement construction let dollars are SD bridges, improvement of Interstate SD bridges, project delivery streamlining, effectiveness of bridge preservation, quality of bridge inspections and quality of load rating analysis.

NEXT STEPS

Next steps for improved bridge management include:

- **Inspection - Element Level Data Collection and Support for Predictive Modeling**
  Bridge inspections will collect more refined and quantified condition information such as lineal feet of deteriorated beams or square feet of deteriorated deck. This refined data will be used in predictive modeling. PennDOT has begun the task of collecting element level condition state assessments.

  The model will predict condition performance factors and costs for an optimal mix of various actions (including preservation, rehabilitation and improvement) over time. A minimum level of maintenance is assumed. Some maintenance (e.g. cleaning, etc.) and preservation treatments (e.g. deck seals) can be achieved through specified treatment cycles. During the initial phase of element data collection, the Department will have sufficient time to develop its agency's rules for modeling. Default Pontis condition models need to be modified to better represent PA bridges. Initially, the PA models may be developed based on expert judgment that can be improved over time from actual element condition data. These predictive models are required in order to utilize the simulation and optimization features of Pontis.

- **Risk Assessment**
  The Department is evaluating the risk to bridge safety and long-term performance by several factors, including scour, fracture-critical members, load capacity, maintenance needs, as well as component condition. This assessment is being done to ensure that work critical to a bridge's performance is adequately considered in new bridge investment optimization methods and programming decisions.

- **Interstate Management Program**
  The Interstate Management Program Guidelines have been published which include baseline indicators and target values. The Districts will use this information in project planning and programming, and we will monitor the program and resultant system improvement.

- **Support for Enterprise Asset Management**
  Bridge analyses is to provide needed bridge costs and performance information, including "what-if" scenarios, to assist in trade-off analysis between asset categories. Analysis methodology must be consistent with Department standards.

IMPLEMENTATION AND CONTINUATION OF NEXT STEPS

To ensure the successful implementation and continuation of the initiatives for asset management of bridges, the following steps need to be taken and/or continued:

Communication

The intent and plan for Asset Management of Bridges will be communicated throughout the Department in the following forums:

- Annual Business Plans
  Topics include plans, goals, accomplishments, bridge performance data, new issues etc. to be presented annually in Chief Engineer's Business Plan.

- Annual Bridge Program Audits of the Districts
  Conducted by the Chief Bridge Engineer to review accomplishments, programmed projects, bridge needs and other operational issues with the District Bridge Engineer.

- Various Department–wide leadership meetings held throughout the year
  Including: Bridge Engineers meeting, Structure Control/Bridge Engineers meetings, Bridge Maintenance Coordinators meetings, selected DE/ADE meetings, bridge inspection training, manager meetings for other PennDOT systems (RMS, Plant Maintenance, etc.), Construction Management Training Workshops.

Implementation

The various components of Asset Management of Bridges will be incorporated into appropriate sections of the following Department documents:

- Design Manual Part 4 – Structures (Pub 15M)

- Bridge Safety Inspection Manual (Pub 238)

- Software – The Department is to continue to develop and improve state-of-art software for various activities, such as:
  - Design – BRADD, other bridge design software
  - Planning and Programming – BMS2 (Pontis)
  - Asset Management – AASHTOware's AssetManager®
  - Inspection – I Forms

Quality Assurance

Various aspects of the quality of the overall management of PA bridges program are reviewed for quality in the following ways:

- Design – QA review of District design processes/plans is performed by BQAD staff each year.

- Bridge Programming and Delivery – the projects selected and delivered by the Districts for the TIP are monitored on an on-going basis by the Bridge Champion to ensure program meets overall bridge initiatives and schedule. Compliance with NBIS inspection frequency is checked monthly by BQAD.

- Inspection – Statewide Inspection QA program reviews selected bridge inventory, inspection, and capacity rating data from bridges in each District through blind inspections by consultant.

Reporting and Monitoring Plans

Various aspects of the management of the bridges are monitored and reported through:

- Annual Business Plans

- Dashboard and Scorecard Performance measures

Table 1. 100 Year Bridge Life Matrix

<table>
<tr><th>Forces</th><th>Design and Construction Solutions</th><th>Era of Implementation</th><th>Construction Cost</th><th>Construction Quality</th><th>Inspections</th><th>Maintenance/Preservation</th><th>Rehabilitation</th></tr>
<tr><td rowspan="7">Corrosion</td><td>1. Minimum number of Deck Joints

2. Eliminate deck joints. No joints on bridges (put joints beyond approach slabs for bridge skews greater than 70^0)</td><td>Mid 80s

• Implement policy late 2006</td><td>Minimal net increase</td><td rowspan="11">Construction Quality - Do not accept defective material such as buckled steel or cracked prestressed beams. Install properly designed riprap, install piles in accordance with specifications, set bottom of footing elevation on proper bearing material</td><td rowspan="7">Focus on beam ends and bearing areas

Element level inspection and reporting (BMS II)

Deck concrete sounding

Perform accurate and timely inspections</td><td rowspan="7">Repair leaking joints

Clean/flush deck, scupper, bearing areas and beam seats

Deck Patching

Latex overlays extends deck life 20 years

Membrane waterproofing and bituminous overlays

Alternate deck joint systems - low cost, easily maintained and repairable such as XJS for retrofits

Spot and Zone Paint

Perform identified maintenance/repairs of structural components

Beam repairs</td><td rowspan="7">Replace ruptured joint glands

Hydro-demolition and deep overlay

Replace decks after 40-50 years

Blast and Repaint Steel Bridges (Environmental challenge for lead base paint)</td></tr>
<tr><td>Eliminate Barrier Deflection Joints</td><td>Mid 80s</td><td>Minimal net increase</td></tr>
<tr><td>Epoxy coated deck reinforcement</td><td>Early to mid 70s</td><td>$2000-$5000 per bridge ($0.05-$0.10/lbs.)</td></tr>
<tr><td>Prestressed Beams with epoxy coated reinforcement</td><td>Mid 80s</td><td>Minimal increase</td></tr>
<tr><td>Prestressed Beams exposed to salt spray from traffic underneath chemical additives and sealants</td><td>• Under development</td><td>• Under development</td></tr>
<tr><td>Steel Beams:
A588 Weathering Steel in certain environmental conditions with painted ends under joints

Bridge Painting - Inorganic zinc rich paint system</td><td>Mid to late 80s

Mid to late 80s</td><td>$0.05/lbs

$0.15/lbs</td></tr>
<tr><td>Epoxy coated J-bars
(Abutments and Piers)</td><td>Mid 80s</td><td>$500-$1000 per bridge ($0.05-$0.10/lbs.)</td></tr>
<tr><td rowspan="2">Fatigue</td><td>Improved Structural Detailing - Welding of connection plates</td><td>Mid 80s</td><td>$1500-$2000 per bridge</td><td rowspan="2">Fracture critical bridge inspection is critical

Major structures susceptible connections at ends of floorbeams

Pre mid 80s constructed bridges - web distortion</td><td rowspan="2">Drill tips of fatigue cracks prior to significant crack growth</td><td rowspan="2">Address floorbeam connections for major structures

Address out of plane distortion connection</td></tr>
<tr><td>Design Equations for 100 year life</td><td>Mid to late 80s</td><td>Minimal increase in material cost</td></tr>
<tr><td rowspan="2">Scour</td><td>Properly designed riprap</td><td>Early 90s</td><td>$15,000 per bridge</td><td rowspan="2">Footing undermining of older bridges
Movement of riprap
Stream bed scour holes</td><td rowspan="2">Install counter scour measures - riprap, stream bed paving, curtain walls
Removal of debris in streams</td><td rowspan="2">Install counter scour measures – riprap
Removal of debris in streams</td></tr>
<tr><td>Appropriate Bottom footing elevations</td><td>Early 90s</td><td>$30,000 per substructure</td></tr>
</table>

<u>Ongoing Activities</u>
Eliminate use of Adjacent Noncomposite Prestressed Box Beams bridge
Maintenance –Develop new performance measurement metrics.
All structures must have redundancy (implemented in 1980’s)

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APPENDIX F - REFERENCE MATERIAL FOR LATERALLY LOADED PILES AND DRILLED SHAFTS

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1.1 LATERAL LOADS ON PILES

The following Appendix lists information on the development of design curves used to determine the lateral deflection and moment on a pile when given a soil profile and a lateral load. The curves were originally developed in US units for service load design. Since the method used was not dependant on the design method used (LRFD or Service Load Design), the curves can be used with any design method as long as the specified units are consistent with the loads used. Therefore, the curves were converted to SI units for use as a preliminary design tool. The design curves should be used as described in D 10.7.3.8.2P.

1.1.1 General

Lateral loads for piles supporting retaining walls, abutments, and piers are generally governed by deflections rather than an ultimate lateral load. Two approaches are available for analyzing laterally loaded piles [1,2,3]. In the Winkler, or subgrade reaction approach, the soil is modeled as a series of unconnected springs. In the elastic continuum approach the soil is assumed to be an ideal elastic material. Both methods of analysis deal with the lateral resistance of soil and the variation of resistance with depth as the pile is pushed against the soil. The major uncertainties of both methods are related to the assumptions used to determine the lateral soil resistance, which is represented by the definition of the soil spring in the Winkler approach and the modulus of elasticity for the elastic continuum approach. At this time the Winkler approach is considered more practical and versatile for routine analyses. The key factor in the successful application of the Winkler approach is the definition of the soil springs.

1.1.2 P-Y Curves and the COM624G Computer Program

The behavior of the soil spring at any point along the length of the pile can be modeled by the relationship between the soil pressure per unit length (p) and lateral deflection (y) at that point. This relationship is typically expressed as a p-y curve and is linear only for a small range of deflections. In general, p-y curves are nonlinear and depend on several parameters, including depth, shear strength, coefficient of friction, unit weight, and the number of load cycles.

A computer program has been developed by Reese and Sullivan to analyze the behavior of a laterally loaded pile utilizing p-y curves to model the soils springs [4,5]. A modified version of this program [6], the COM624G, developed to be used with an IBM Personal Computer, has been used to develop the design charts in this manual. The COM624G program includes all the features of the original Reese program. (Note that the current version of the program is COM624P.)

In the COM624G program the pile length is divided into many segments, each acting independently. P-Y curves are developed for each segment. The governing differential equation is then solved by the finite difference method. An iterative procedure is used until satisfactory compatibility is obtained between the predicted behavior of the soil and the load-deflection relationships required by an elastic pile.

The COM624G program is equipped to develop p-y curves for the following soil conditions:

(a) Soft clay below the water table
(b) Stiff clay below the water table
(c) Stiff clay above the water table
(d) Sand, above or below the water table

As an example, Figure 1.1.2-1 contains the plots of p-y curves computed for various depths for a loose to medium dense sand (φ=32) above the water table under static and cyclic loading. For sands, the slope of the initial straight line portion of the curve is the secant modules of soil reaction, E_s, where:

$$E_S = \frac{-P}{y}$$

and E_s is related to depth by the constant k:

$$E_s = kx$$

It can be seen from the p-y curve plots that the extent of the straight line portion of the curve varies with depth and mode of loading (cyclic or static). The level portion of the curves represents the yield point of the soil at a given depth. For this case, a cyclic loading condition only influences the p-y curves near the ground surface.

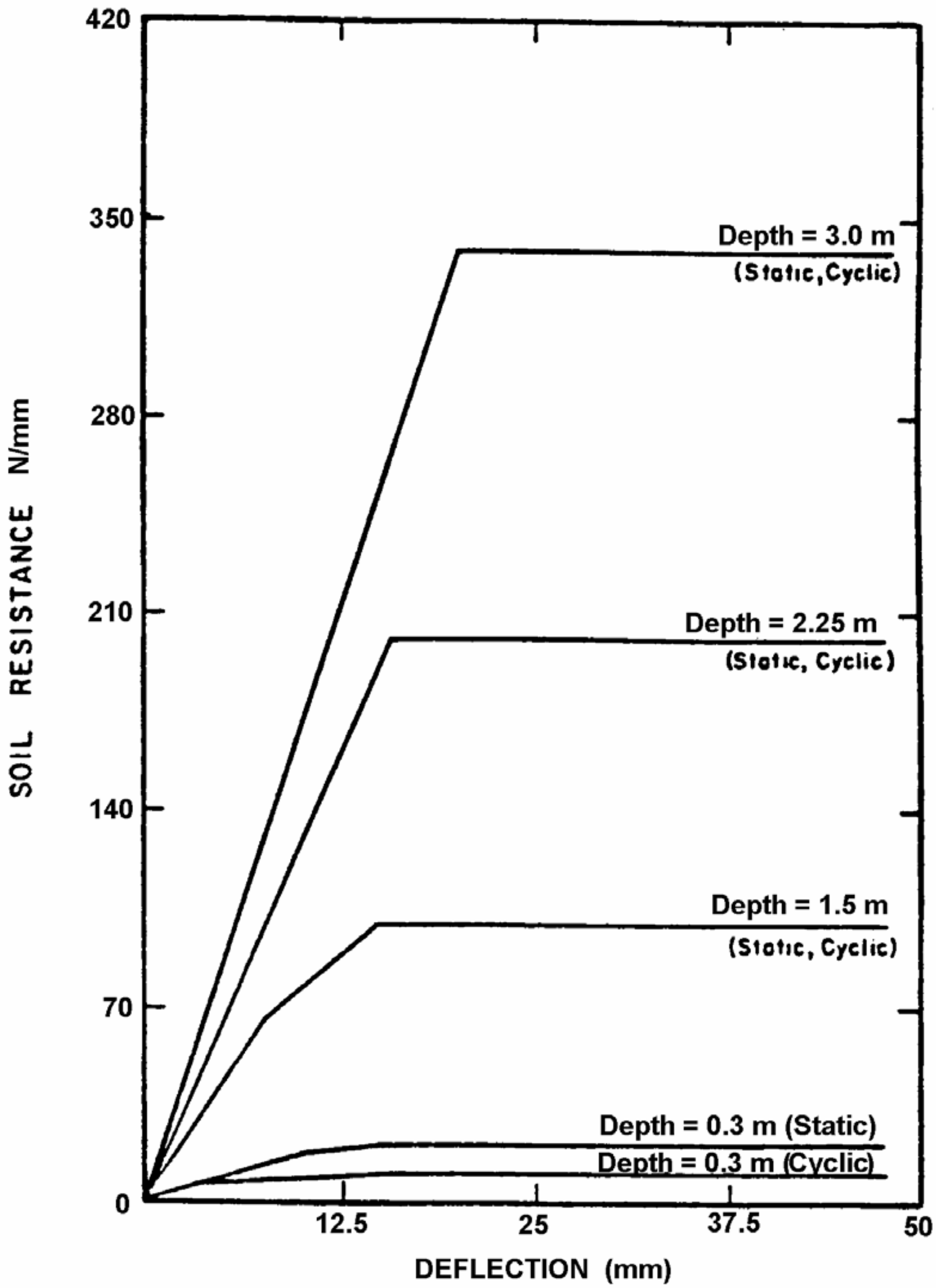


Figure 1.1.2-1 - P-Y Curves in Loose Sand

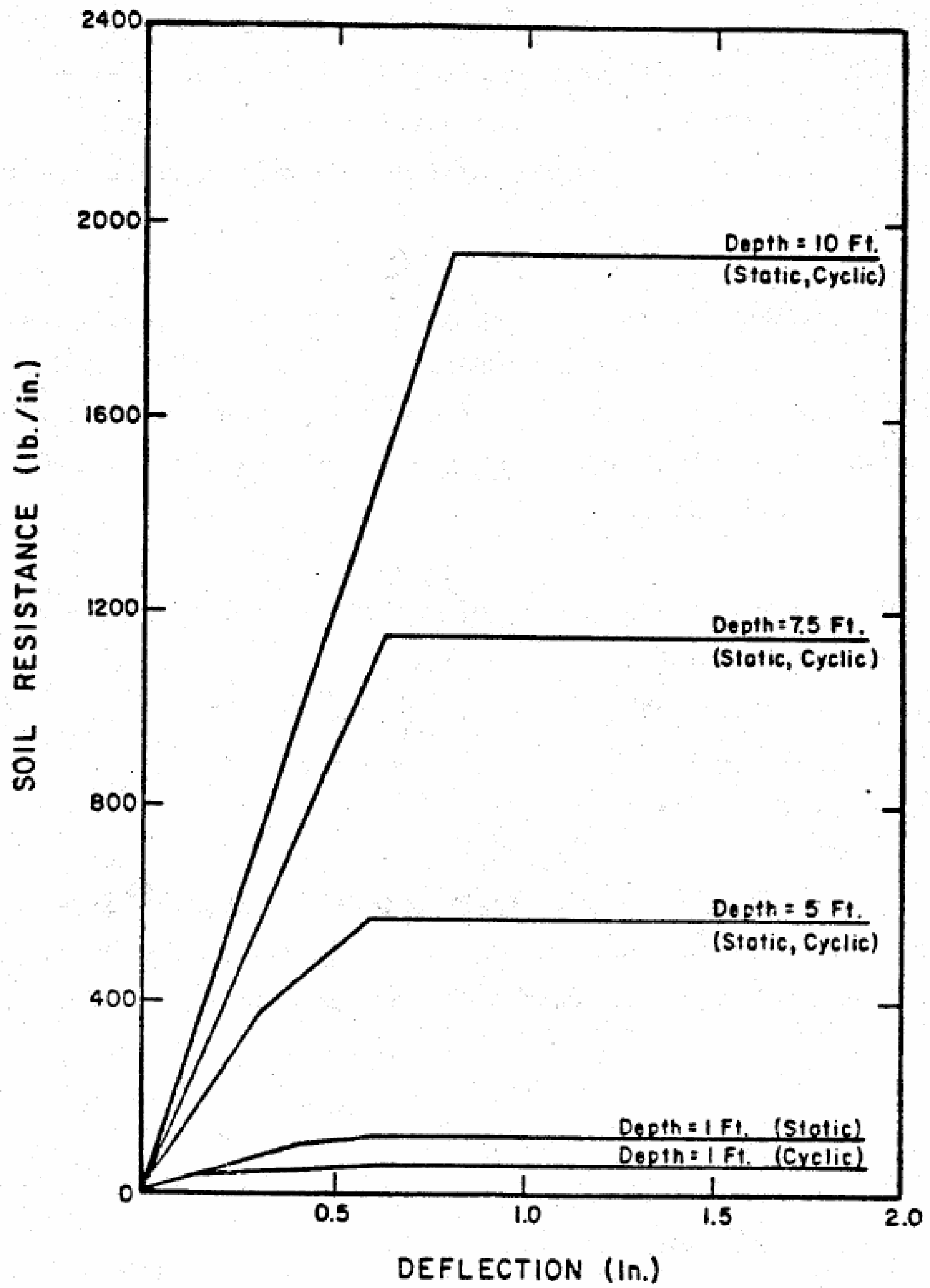


Figure 1.1.2-1 - P-Y Curves in Loose Sand

The methods used to develop the p-y curves in the program are empirical, based on the results of a limited number of load tests. Other p-y curves can be developed and input into the program. The COM624G program is also equipped to evaluate fixed, free, or partially restrained pile head; static or cyclic loading; layered soils; axial loads; and variations in the flexural rigidity (EI) of the pile with depth.

1.1.3 Parametric Studies

The effects of changing various parameters were studied prior to developing the design cases. The results of these studies may be useful in applying the computed COM624G results to actual conditions. A 400 mm pipe pile was generally analyzed for two soil conditions, loose sand (φ = 32 degrees, γ = 1760 kg/m^3 {110 pcf}, and k=5500 kN/m^3) {35 kcf}) and soft clay (c = 0.024 MPa {500 psf}, γ=1602 kg/m^3 {100 pcf},, ε_{50}=.02, and k = 8170 kN/m^3 {52 kcf}). The constant, k, and the strain corresponding to one-half the maximum principal stress difference, ε_{50}, are as recommended in the program User's Manual [4]. Results of the parametric studies are as follows:

(a) Length-Pile lengths were varied between 4.5 and 30 meters {15 and 100 ft} for lateral loads of 90 and 111 kN {20 and 25 kips}, and pile head fixity of zero (free-head) and 100 percent (fixed-head). It was found that ground line deflections will remain constant for a given lateral load for pile penetrations beyond a critical depth. This critical depth was found to be about 9 m {30 ft}for soft clay and about 7.5 m {25 ft}for loose sand with a 400 mm {16-inch}diameter pipe pile. Results of modeling a more flexible 270 mm {10.75-inch}diameter pipe pile in soft clay indicate that the critical depth is no greater than 9m {30 ft} in this case as well. Deflections increase significantly for penetrations less than the critical depth. Generally, the critical depths will be somewhat shallower for stiffer or denser soils.

(b) Pile Tip Embedment-For the soft clay case, the pile tip was embedded 0.6 m {2 ft}into soft rock (c=0.192 MPa {4,000 psf}, γ=2080 kg/m^3 {130 pcf}, ε_{50}=0.004, and k=271 000 kN/m^3 {1,000 pci}). Table1.1.3-2 contains a summary of results. Tip embedment had no effect on a 9 m {30 ft} pile, and a very minimal effect on a 4.5 m {15 ft} pile with a fixed-head under a lateral load of 90 kN {20 kips}. The only significant decrease in deflection due to tip embedment occurred with the free-head, 4.5m {15 ft} pile.

In the analysis performed for design curve development, 4.5 m {15 ft}, HP 250x62 {10x42} piles in saturated loose sand or soft clay with a 600 mm {2 ft}embedment into soft or weathered rock were considered. At a 27 kN {6 kip}lateral load, deflections were only decreased about 6 percent from the full depth saturated loose sand case. However, compared to the full-depth saturated soft clay case, deflections were decreased about 30 percent at a 27 kN {6 kip} lateral load. For a 9 m {30 ft} HP250x62 {10x42} pile in saturated soft clay, tip embedment had no effect on deflections. Based on these results, pile tip embedment will have the greatest effect on short piles in saturated, soft clay.

(c) Axial Load-As shown in Table1.1.3-3 both deflections and moments increase with increasing axial load with constant lateral force of 90 kN {20 kips}. The effect is much more significant for a free-head pile than it is for a fixed-head pile. Results of a similar analysis of a more flexible 270 mm {10.75-inch} diameter pipe pile supports these conclusions.

(d) Pile Head Fixity-For a free head pile (zero fixity) there is no moment at the top of the pile. A fixed-head pile (100 percent fixity) has a sufficient resisting moment at the top of the pile to ensure no rotation. To model fixities between these limits, a resisting moment equal to the appropriate fraction of the resisting moment developed in the 100 percent fixed condition was applied to the top of the pile. That is, for 25 percent fixity, 25 percent of the resisting moment developed for the fixed-head condition was applied to the top of the pile. The effect of pile head fixity was investigated for a pile driven into soft clay with a 600 mm {2 ft} tip embedment into soft rock. Deflections versus pile head fixity for a given pile length and lateral load are plotted in Figure1.1.3-1. For a given pile length, the greater the pile head fixity the less the deflection.

Figure 1.1.3-1 also contains a plot of the absolute value of the maximum moment versus pile head fixity. Using the sign conventions as defined for the COM624G program, positive moments in the pile increase with decreasing pile head fixity and are greatest for a free-head pile. Negative moments in the pile increase with increasing pile head fixity and are greatest for an 100 percent fixed head pile. Therefore, as shown in Figure 1.1.3-1, for fixities between 0 and 100 percent, the net moment in the pile decreases.

Although the magnitude of the differences may vary, the effect of pile head fixity on deflection and moment as illustrated on Figure 1.1.3-1 will hold true for any pile/soil combination.

(e) Groundwater - As shown in Table 1.1.3-1, the location of the groundwater table may have a significant effect on lateral deflections. The influence of the water table is most significant for the free-head pile in sand.

Table 1.1.3-1 - Parametric Studies-Groundwater

Lateral Load (kN) {kips}	Groundline Deflection (mm) {in}							
	Sand				Clay			
	Fixed		Free		Fixed		Free	
	Dry	Wet	Dry	Wet	Dry	Wet	Dry	Wet
22 {5}	1.5 {.06}	1.5 {.06}	4.25 {.17}	4.5 {.18}	0.25 {.01}	0.25 {.01}	1.0 {.04}	1.0 {.04}
44.5 {10}	3.25 {.13}	3.25 {.13}	8.75 {.35}	11.75 {.47}	1.0 {.04}	1.0 {.04}	4.0 {.16}	4.25 {.17}
89 {20}	6.5 {.26}	8.0 {.32}	19.5 {.78}	31.0 {1.24}	4.0 {.16}	4.25 {.17}	17.75 {.71}	19.0 {.76}

NOTES:

1. Pile - 400 mm {16 inch} pipe
2. Pile Length - 9 m {30 ft}

Table 1.1.3-2 - Parametric Studies-Pile Tip Embedment

Pile Length (m) {ft}	Lateral Load (kN) {kips}	Groundline Deflection (mm) {in}			
		Free Head		Fixed Head	
		w/ Embed.	w/o Embed.	w/ Embed.	w/o Embed.
4.5{15}	44.5 {10}	25.0 {.99}	47.5 {1.9}	3.0 {.12}	3.0 {.12}
	89.0 {20}	>>	>>	10.5 {.42}	11.0 {.44}
9{30}	44.5 {10}	8.75 {.35}	8.75 {.35}	2.25 {.09}	2.25 {.09}
	89.0 {20}	32.5 {1.3}	32.5 {1.3}	8.0 {.32}	8.0 {.32}

NOTES:

1. >> - Excessive groundline deflections indicate soil failure
2. Pile: 400 mm {16 inch} pipe
3. Soil Condition: Soft Clay

Table 1.1.3-3 - Parametric Studies-Axial Load

Axial Load (MPa)	Groundline Deflection (mm) {in}/Max. Moment (mm-kN) {in-kips}	
	Free Head	Fixed Head
0 {0}	31.25 {1.23}/13 334 {118}	8.0 {.32}/13 788 {-122}
41 {6}	35.5 {1.42}/15 029 {133}	8.25 {.33}/14 125 {-125}
83 {12}	41.5 {1.66}/16 950 {150}	9.5 {.38}/14 351 {-127}

NOTES:

1. Pile - 400 mm (16 inch}
2. Pile Length - 9 m {30 ft}
3. Soil Conditions - Soft Clay
4. Lateral Load - 90 kN {20 kips}

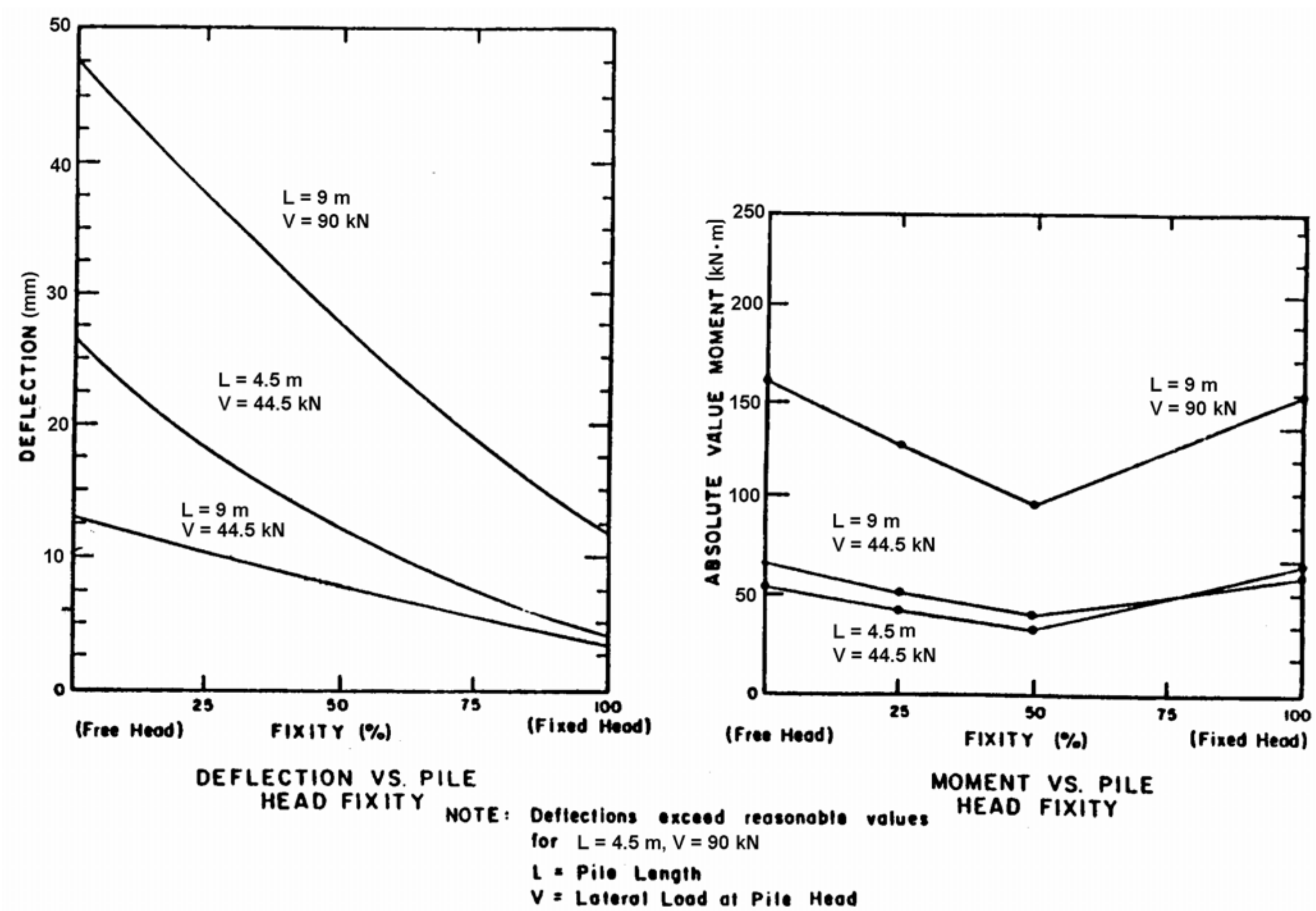


Figure 1.1.3-1 - Head Fixity

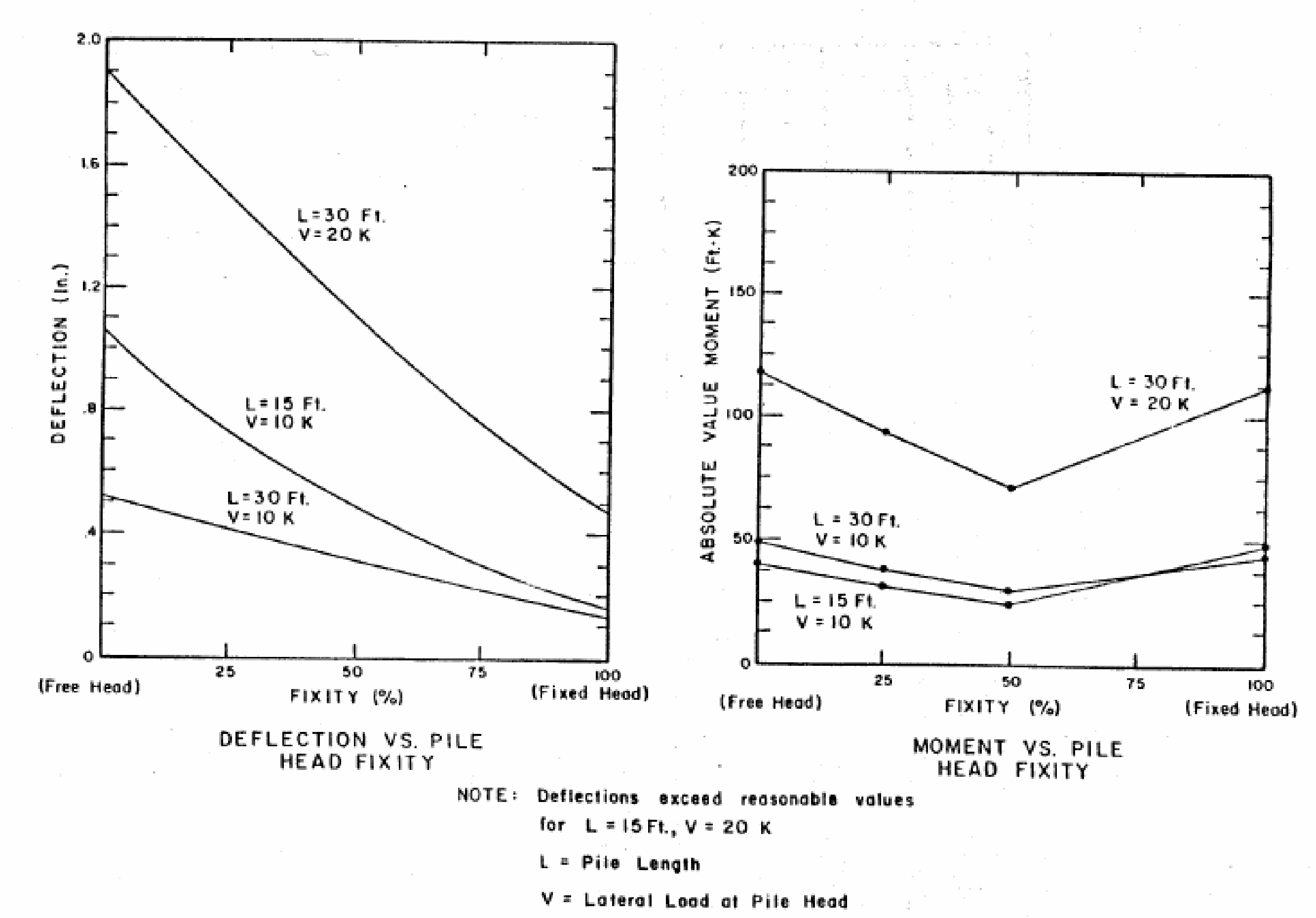


Figure 1.1.3-1 - Head Fixity

1.1.4 Comparison of Computed Results with Load Test Results

A load test program, sponsored by PennDOT, was conducted for the design of LR 1021, LR 1026, and LR 1040 in the Pittsburgh area [7]. The deflected shape, as indicated by inclinometer data, of laterally load tested piles from two sites has been compared with the deflected shape computed by the COM624G program.

Figure 1.1.4-1 shows actual and computed deflection profiles for a 18 m {60 ft}, HP 310x110 {12x74} pile. The soil profile in the vicinity of the load tested pile consisted of 3 m {10 ft}of a loose sand, gravel, slag, and rubble fill; 3 m {10 ft} of more compact fill with similar constituents; and 12 m {40 ft} of natural, medium dense sand and gravel with rock fragments. These layers were input into the COM624G program as loose sand, medium dense sand, and saturated medium dense sand. Although the computed deflected shape of the pile shows greater deflection along the length of the pile, the agreement between ground line deflections is quite good.

At the other site, the soil profile consisted of 0.9 m {3 ft} of soft silty clay, 4.5 m {15 ft} of stiff silty clay, and 2.1 m {7 ft} of weathered and broken shale. Although the materials had a significant silt content, the strata were all modeled as clays ($\varphi = 0$) and the water table was assumed to be at the top of the weathered rock. The load tested pile was an HP 250x62 {10x42}, approximately 7.5m {25 ft} in length. Figure 1.1.4-2 shows actual and computed deflection profiles for the test pile. It is evident from the plots that using the p-y curves for clay for the conditions at this site results in significantly overestimating deflections at high loads and underestimating deflections at low loads. The treatment of silty soils is addressed in Section 1.1.6.

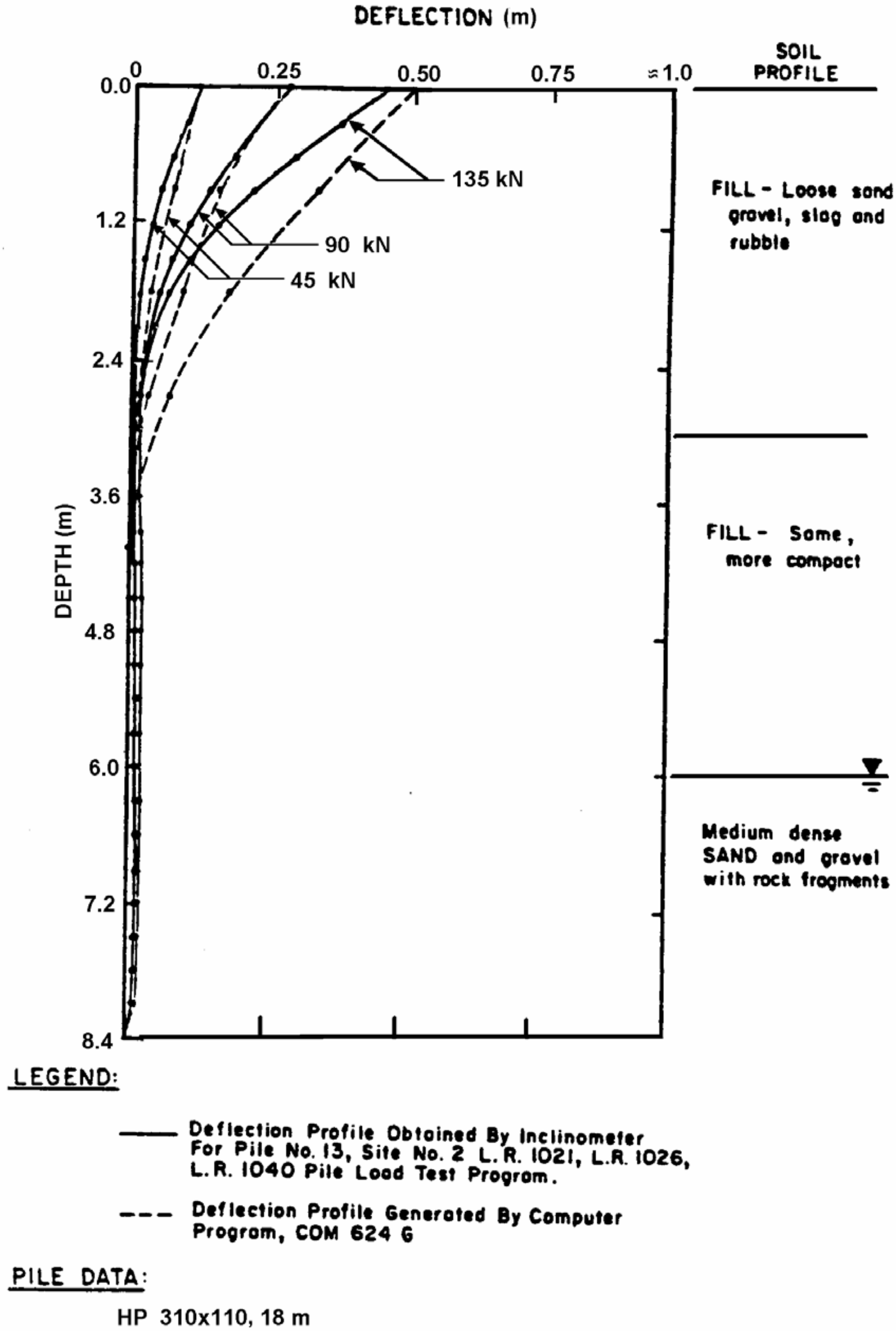


Figure 1.1.4-1 - Comparison of Load Test Results with Computer Generated Deflection Profile

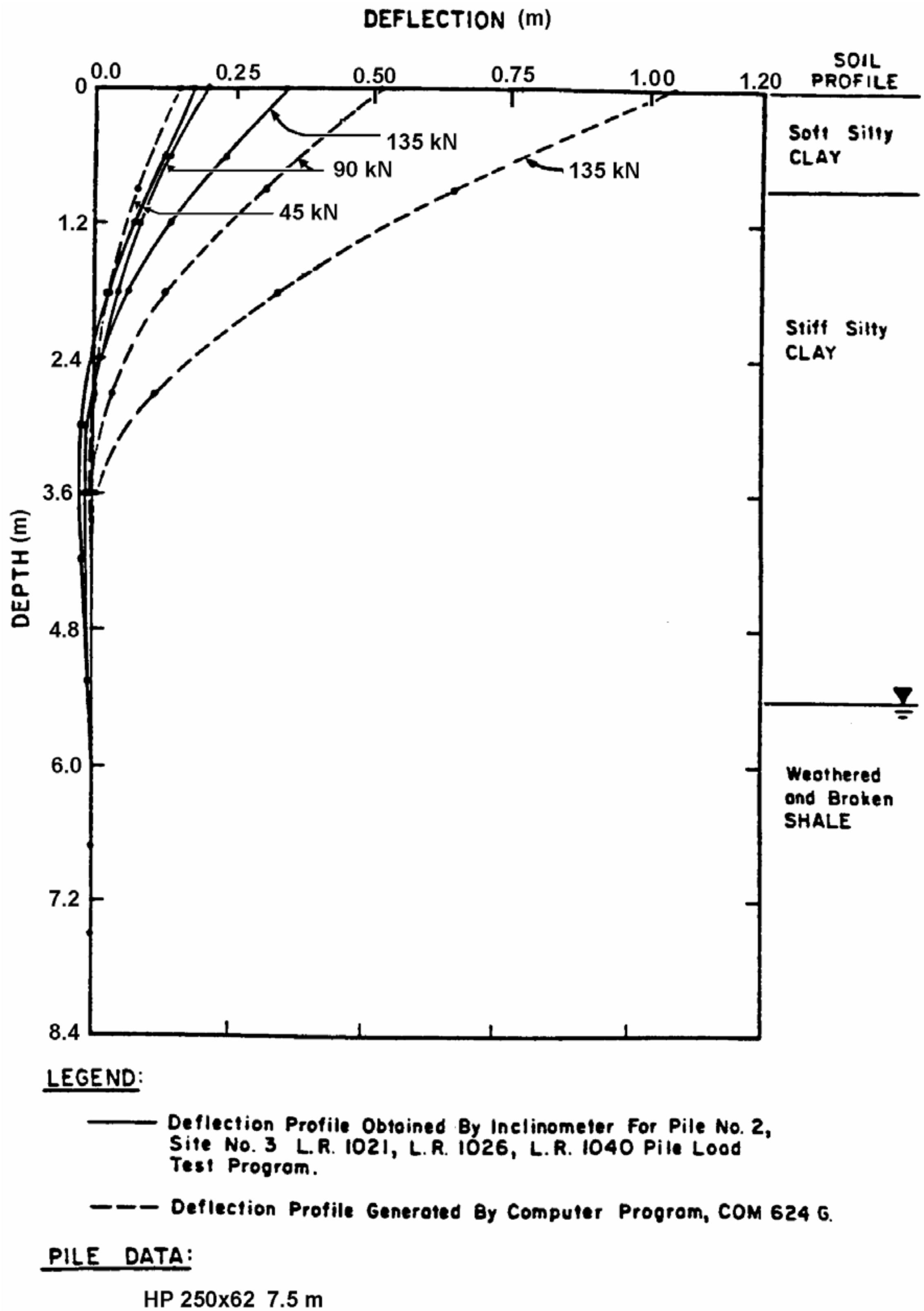


Figure 1.1.4-2 - Comparison of Load Test Results with Computer Generated Deflection Profile

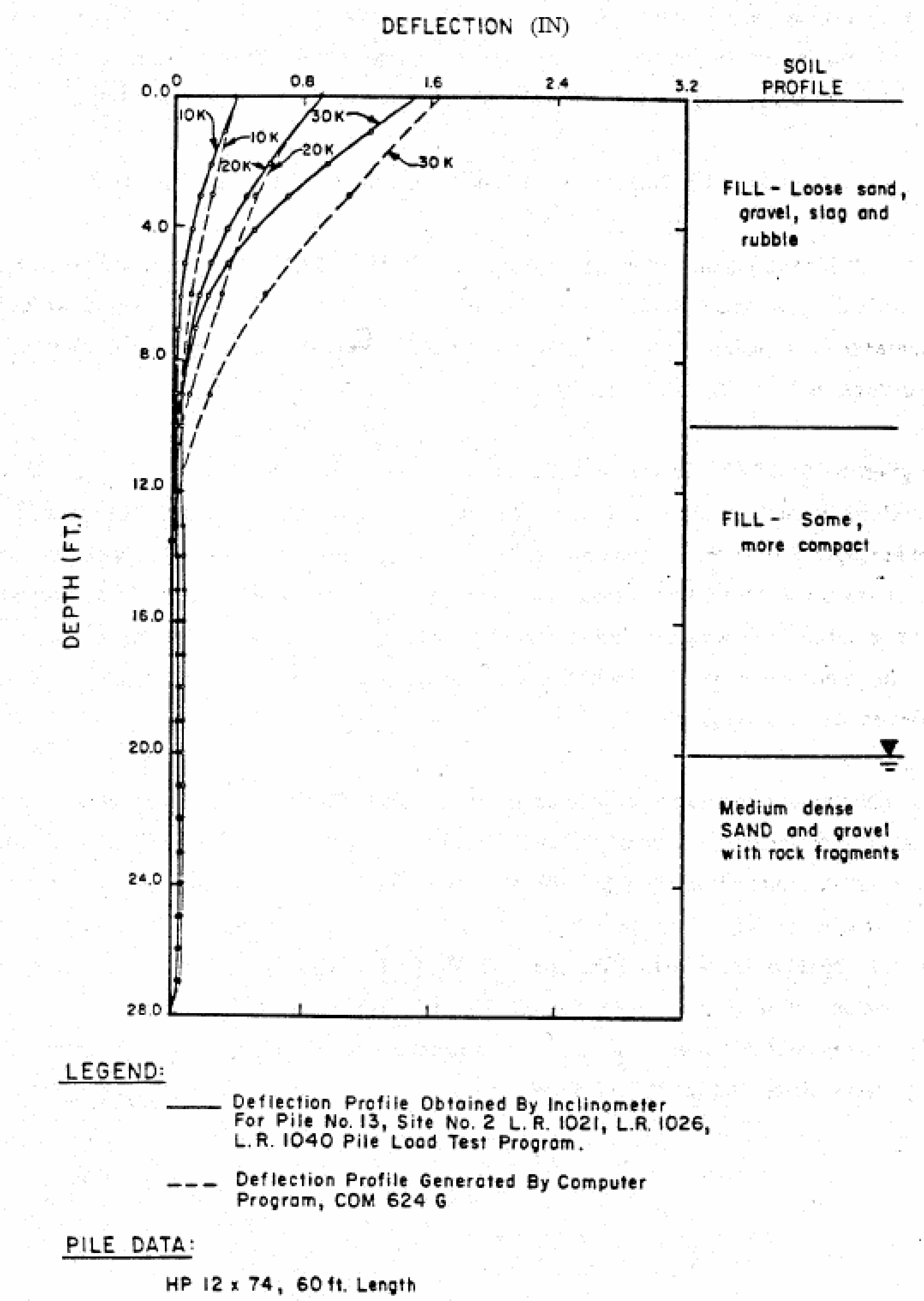


Figure 1.1.4-1 - Comparison of Load Test Results with Computer Generated Deflection Profile

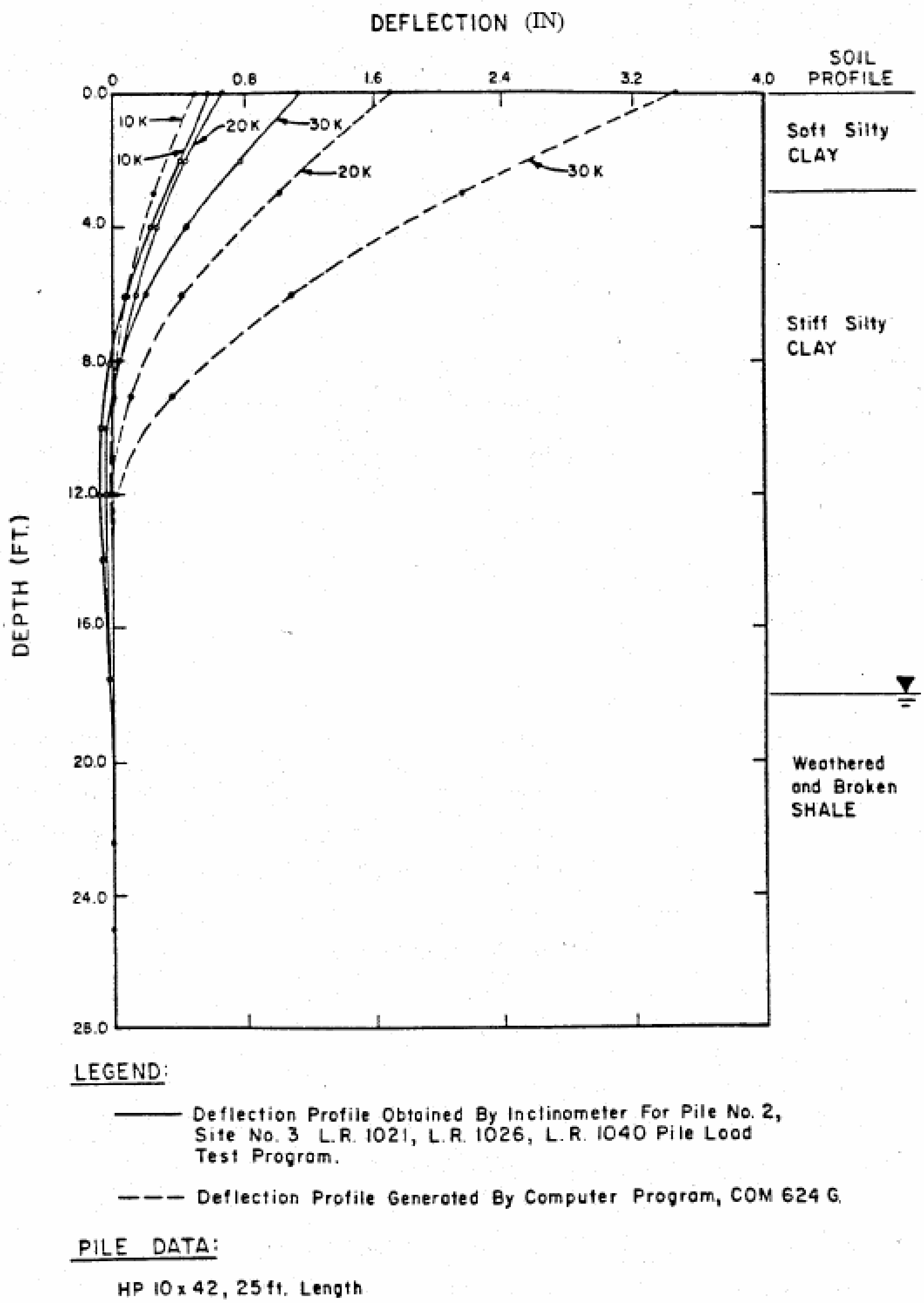


Figure 1.1.4-2 - Comparison of Load Test Results with Computer Generated Deflection Profile

1.1.5 Design Curve Development

Based on the results of analyses using COM624G, design curves presented in 1.1.8 have been developed. The curves cover five pile types for a range of lengths, soil conditions, and loads.

The pile types used for development of design curves are summarized in Table 1.1.5-1. Soil conditions considered are summarized in Figure 1.1.6-1. For a discussion of the use of the given soil profiles to model actual conditions, see 1.1.6.

The parametric studies indicate that deflections and moments will increase with increasing axial loads. A relatively high axial load has been applied to the piles studied to provide conservative results. Axial loads corresponding to the following stresses have been utilized for the various pile types:

(a) Steel H-piles and pipe piles - 83 MPa {12 ksi}
(b) Cast-in-place piles - 0.35 f'_c
(c) Concrete piles - 0.30 f'_c
(d) Timber piles - 4.1 MPa {0.60 ksi}

With axial loads higher than these, the moments will be somewhat unconservative due to the additional moment caused by the displacement of the vertical load by the deflection. Investigation of the program results showed that the actual moment increases approximately 10 percent when the pile is stressed to its ultimate axial structural capacity. However, at this level, the capacity is primarily controlled by the axial stress, and an increase of 10 percent in the moment results in an overstress of less than 2 percent in the pile. Therefore, the additional moment can be neglected in most cases. Exceptions include high strength steels and concrete, in which case the design moment shown on the curves shall be increased by an amount equal to the additional axial load times the deflection.

The cases described were analyzed for lateral loads of up to 133 kN {30 kips}. In a bridge, live loads would be considered cyclic. Therefore, the design charts have been developed for cyclic loading conditions. This is likely to be conservative, particularly for abutments, because the total lateral load would have both static and cyclic components. The methods for developing p-y curves utilized in the COM624G program are based on load tests for both static and cyclic loads. In general, effects of cyclic loading, as modeled by the COM624G program, increase with increasing load.

Pile deflections and bending moments from lateral loads are a function of the pile fixity and group effects. Pile fixity is the amount that the pile head is restrained against rotation. When the pile head is completely restrained, the pile is a fixed-head pile (100 percent fixity). When the pile head is completely free to rotate, the pile is a free-head pile (0 percent fixity). Neither condition is typically met in abutments and piers. Instead, the fixity varies due to several factors.

(a) Pile Cap Stiffness - The pile cap and the piles act as a frame when subject to lateral loads. The cap restrains the rotation of the pile head. The amount of restraint is proportional to the relative stiffness between the cap and the piles. A thicker cap or an additional row of piles would provide greater fixity. This frame action is not available when a single row of piles is used and restraint is only provided by the surrounding ground. Therefore, the fixity of a single row of piles is closer to zero.

Table 1.1.5 -1- Lateral Pile Loading Pile Types

<table>
<tr><th>Pile Type</th><th colspan="2">Size of Nominal Diameter</th><th>Moment of Inertia
(10^6 mm^4) {in^4}</th><th>Modulus of Elasticity
MPa {psi}</th><th>Remarks</th></tr>
<tr><td>Steel H-Pile</td><td colspan="2">250 X 62 {10x42}
250 X 62 {10x42}
250 X 85 {10x57}
310 X 79 {12x53}
310 X 110 {12x74}
360 X 174 {14x117}</td><td>30 {71.7}
87.5 {210}
123 {294)
163 {393}
237 {569}
508 {1,220}</td><td>200 X 10^3
{$29x10^6$}</td><td>weak axis
strong axis
strong axis
strong axis
strong axis
strong axis</td></tr>
<tr><td>Steel Pipe</td><td colspan="2">270 {10.75 in}
400 {16 in}
600 {24 in}</td><td>67 {161}
234 {562}
807 {1,940}</td><td>200 X 10^3
{$29x10^6$}</td><td>Structural capacity from steel only assumed.</td></tr>
<tr><td>Cast-in-Place</td><td colspan="2">250 mm {10 in}
300 mm {12 in}
350 mm {14 in}
450 mm {18 in}</td><td>345 {829}
522 {1,253}
879 {2,111}
3075 {7,390}</td><td>22.5 X 10^3
{$3.27x10^6$}</td><td>Structural capacity from steel shell and concrete only, no reinforcing included; Class A concrete, 28-day strength = 23 MPa. {3,300 psi}.</td></tr>
<tr><td>Precast or Prestressed Concrete</td><td colspan="2"></td><td></td><td rowspan="3">27.8 X 10^3
{$4.03x10^6$}</td><td rowspan="3">No reinforcing included in structural capacity; concrete 28-day strength = 34 MPa. {5,000 psi}.</td></tr>
<tr><td>Square</td><td colspan="2">250 mm {10 in}
300 mm {12 in}
350 mm {14 in}
450 mm {18 in}</td><td>348 {835}
719 {1,728}
1331 {3,199}
3641 {8,748}</td></tr>
<tr><td>Octagonal</td><td colspan="2">250 mm {10 in}
300 mm {12 in}
350 mm {14 in}
450 mm {18 in}</td><td>231 {155}
535 {1,285}
877 {2,107}
2394 {5,751}</td></tr>
<tr><td rowspan="4">Timber</td><td>Dia. (1)</td><td>Len.</td><td rowspan="4">Varies along length of pile due to taper.</td><td rowspan="4">7.58 X 10^3
{$1.10x10^6$}</td><td rowspan="4">Used Characteristics conservative for several species of wood (ASTM D 2555).</td></tr>
<tr><td>200 mm
{8 in}
325 mm
{13 in}</td><td>6 m
{20 ft}</td></tr>
<tr><td>250 mm
{10 in}
375 mm
{15 in}</td><td>12 m
{40 ft}</td></tr>
<tr><td>300 mm
{12 in}
450 mm
{18 in}</td><td>18 m
{60 ft}</td></tr>
</table>

NOTE: Diameter 900 mm {3 ft} below butt as per ASTM D 25

Table 1.1.5-2 - Comparison of Computer Generated Pile Capacities with Load Test Results Based on Fixity and Group Effects

Deflection (mm)	5 {0.2}	10 {0.4}	15 {0.6}	20 {0.8}	25 {1.0}
Group Efficiency-Load Test(1)	1.9	1.6	1.4	1.3	1.2
50 Percent Fixed/Free (Avg.)(2)	1.3	1.3	1.3	1.3	1.3
100 Percent Fixed/Free (Avg.)(2)	2.4	2.4	2.4	2.4	2.4

NOTES:

1. Ref. [8]; Group efficiency = Ratio of individual pile capacity in a group to the capacity of a single pile, for a given deflection; Group considered had pile spacing of 3D, with 100 mm {4 in} of soil removed below the pile cap. Data from full scale lateral load tests.
2. Average value for HP 250x62 {10x42} and HP 360x174 {14x117} piles, weak axis, soil profiles 2 and 4; pile capacities for given deflections. Data from COM624G Pile Analysis Program.

(b) Pile/Pile Cap Connection - In order for the cap to restrain the pile head, the pile shall be securely fastened to the cap. According to Davisson [9], with structural details commonly used a fixity of 50 percent is obtainable. Reinforcement located at the base of the pile cap was not in common use at the time this statement was made. Any cracking at the base of the cap would reduce the fixity of the connection. With a 300 mm {1 ft} pile embedment and cap reinforcement located at the bottom of the cap, cracking at the base of the cap will be minimized [10]. It is likely that fixities greater than 50 percent will be realized.

(c) Lateral Loads - Under small lateral loads, little rotation of the pile takes place and the fixity is approximately 100 percent. As the lateral loads increase, rotation of the cap as well as some movement of the pile in the pile/pile cap connection take place, allowing the pile head to rotate and decreasing the fixity.

Frame action in pile groups will cause a decrease in total deflection due to an increase in pile head fixity. However, given a constant pile head fixity, a group of piles would be likely to deflect more than a single pile due to disturbance of the surrounding soils by movement of adjacent piles. Full scale load test data for pile groups is minimal. In general, results of full scale and model load tests [9,11,12,13] indicate that deflection of a group is greater than deflection of a single pile. Interpretation of such data is complicated, however, by the difficulty in determining and controlling pile head fixity for single piles and pile groups.

Group effects from adjacent piles are more significant in the direction of loading. Based on model tests by Prakash [9], and confirmed by model tests by Cox [11], there will be no group effects for spacings greater than about 8D (where D = pile diameter) in the direction of loading or for spacings greater than about 2.5D normal to the direction of loading. For most abutments and piers, spacing in the direction of loading will be less than 8D.

Full scale lateral load tests in Pennsylvania as reported by Kim [14,15,16] indicate that a group of piles will be more efficient for deflections than a single pile. Kim's tests included comparing the behavior of a single vertical HP 250x62 {10x42} pile to that of two groups of vertical HP 250x62 {10x42}, one spaced at 4D, the other at 3D. Both the single pile and the pile groups were embedded 300 mm {12 in} into a 1200 mm {4 ft} thick pile cap. All pile caps had reinforcing at the base. Axial stresses of about 41 MPa {6 ksi} were applied during lateral load testing.

The results of Kim's study indicate that pile group efficiencies (the ratio of individual pile capacity in a group to the capacity of a single pile, for a given deflection) were greater than 1.0 for all groups. Group efficiencies were greater for greater pile spacings. It is probable that these trends are due to the greater fixity of the pile head for pile groups. That is, the decrease in deflection due to the increased fixity of the pile head (frame action) more than compensates for the increase in deflection due to group effects from adjacent piles.

In order to take into account fixity and group effects typical to abutments and piers, a 50 percent fixity was assumed when using the COM624G program to determine deflections. This provides deflections which are conservative for any fixity greater than 50 percent. These fixities can usually be achieved when more than one row of piles is embedded 300 mm into and connected by a concrete pile cap reinforced at the bottom. A comparison of the program output with group efficiency studies by Kim [14,15,16] was performed. Kim's pile group efficiencies were compared to the ratio of pile capacity of a free-head pile to the capacity of a pile with 50 percent fixity. As shown in Table 1.1.5-2, the results indicate that for 50 percent fixity the program produces reasonable deflections for pile groups typical to abutments and piers.

As shown in Figure 1.1.3-1, using 50 percent fixity will minimize the maximum moment in the pile in some cases. To take into account the potential for higher fixities, 80 percent of the fixed-head maximum moment is shown on the design curves. This moment will be conservative for fixities between 50 percent and 80 percent, and in many cases for fixities somewhat less than 50 percent. Fixities greater than 80 percent would be associated with small lateral loads, where moment is not critical. Fixities less than 50 percent may occur when lateral loads are very high or where there is only a single row of piles.

A pile head fixity less than 50 percent might also occur if reinforcement is not placed at the bottom of the pile cap and lateral loads are high enough to induce cracking. Where piles are closely spaced, placement of a reinforcement cage below the top of the piles may be impossible due to pile misalignment during driving. Hand work would be required to place reinforcing below the top of the piles. The data presented in Table 1.1.5-2 indicate that the ratio of the lateral resistance of a pile with 50 percent fixity to a free-head pile is 1.3 for any given deflection. Additional analysis indicates that for a given moment 1.3 is appropriate for the average ratio of the lateral resistance of a pile with 80 percent fixity to a pile with 20 percent fixity. Even with a cracked pile cap, at least 20 percent head fixity should be retained. Therefore, if the actual lateral load is less than 75 percent of the lateral resistance as determined by the design curves, reinforcing is not required to be located below the top of the piles. Alternatively, if reinforcement cannot economically be placed below the top of the piles, no more than 75 percent of the lateral resistance as determined by the design curves shall be used.

1.1.6 Soil Conditions

Design curves have been developed for the seven soil profiles shown in Figure 1.1.6-1. In the descriptions of these profiles all granular material is referred to as sand and all cohesive material is referred to as clay. Actual soil conditions may be more complex than the simplified soil profiles presented. Engineering judgment shall be employed when selecting the appropriate soils profile to model actual conditions. Soils from the ground surface to a depth equal to five to ten pile diameters have the greatest influence on laterally loaded pile behavior [5].

Soil Profiles 1 through 4 should cover most soil conditions if the soils within a depth of five to ten pile diameters are assumed to control the behavior of a laterally loaded pile. As examples, lateral load resistance can be established for the load test piles described in D10.7.3.8.

Referring to Figure 1.1.4-1, this 18 m {60 ft} long HP 310x110 {12x74} is embedded in granular material. The top 3 m {10 ft} is a loose sand above the water table. Using Soil Profile 1 should give a conservative estimate of lateral load resistance. Using Figure 1.1.8-1, for a deflection of 12.5 mm {0.5 in}, the lateral load resistance would be 53 kN {12 kips}. The results of the lateral load test are shown on Figure 1.1.4-1. The load test was performed on a free-head pile, therefore, deflections would be greater than those expected for a pile with at least 50 percent head fixity gained by pile cap embedment. The load test resulted in approximately 10 mm {0.4 in} of deflection for a 44 kN {10 kip} load. The recommended lateral load of 53 kN {12 kips}, as expected, seems slightly conservative but reasonable.

The soil profile for the other load test considered is shown on Figure 1.1.4-2. The 7.5 m {25 ft} long HP 250x62 {10x42} is embedded in predominantly cohesive soils. Surface soils consist of 0.9 m {3 ft} of soft silty clay overlying 4.5 m {15.0 ft} of stiff silty clay. Both strata are above the water table. These soil conditions fall between Soil Profile 3 and Soil Profile 4. Using Soil Profile 3 would be very conservative as it models a full-depth, soft, saturated clay. From Figure 1.1.8-1, the lateral load resistance for Soil Profile 3 is 33 kN {7.5 kips}. Soil Profile 4 is much closer to actual conditions, with the exception of the top 0.9 m {3 ft} of soft material. From Figure 1.1.8-3, the lateral load resistance for Soil Profile 4 is 73 kN {16.5 kips}. A conservative estimate for the actual soil conditions can be obtained by using a value halfway between the values obtained for Soil Profile 3 and 4. Therefore, for a lateral load resistance of 54 kN {12 kips} would be recommended. This value is reasonable when compared to the deflections obtained from the load test of a free-head pile, i.e., 14.5mm {0.58 in} for a lateral load of 44 kN {10 kips}and 16.5 mm {0.66 in}for a lateral load of 89 kN {20 kips}.

Soil Profiles 5, 6, and 7 are more specialized than the first four. Profiles 5 and 6 were developed to model very poor surface soil conditions. Any greater depth of very loose or very soft material shall be analyzed independently. Soil Profile 7 models tip embedment. Parametric studies showed that pile tip embedment into weathered or soft rock has no effect on piles 9 m {30 ft} in length. Soil profiles for full-depth sand and clay can be used for depths of 9 m {30 ft} or more. The effect on 4.5 m {15 ft} piles was most significant with saturated soft clay. Therefore, Soil Profile 7 covers this condition. Because of the minimal effect of tip embedment on piles in saturated loose sand, Soil Profile 1 can be used to adequately model this case.

Many soils in Pennsylvania exhibit a high silt content. Methods for determining p-y curves are limited to sands and clays. No model has yet been developed to simulate a soil exhibiting both a cohesion, c, and an angle of internal friction, φ, value. Therefore, a judgment must be made as to whether the soils are predominantly granular or predominantly cohesive. The comparison of computed results to load test results (D10.7.3.8) indicates that for clays, the p-y curve analysis tends to underestimate groundline deflections at low loads and overestimate groundline deflections at higher loads. Agreement of groundline deflections for sands was quite good. So that groundline deflections are not underestimated for clays at low loads for Soil Profiles 1 through 4, only the more conservative results for sand are shown on the lateral load vs. deflection curves at low loads. This curve, shown as a solid line, shall be used for both sand and clay. At higher loads the curves for clay are shown with a broken line.

An independent analysis with the COM624G program shall be performed when actual soil conditions are worse than those covered by the soil profiles. If soil conditions are better than those covered by the soil profiles, an independent analysis may be performed if a substantial cost savings can be realized by a further increase in the lateral load resistance. In many cases, where soil conditions are good, the lateral load resistance will be controlled by the available moment determined by the interactive equations rather than deflections. The effect of improved soil conditions is generally less for the load vs. maximum moment curves than it is for the load vs. deflection curves.

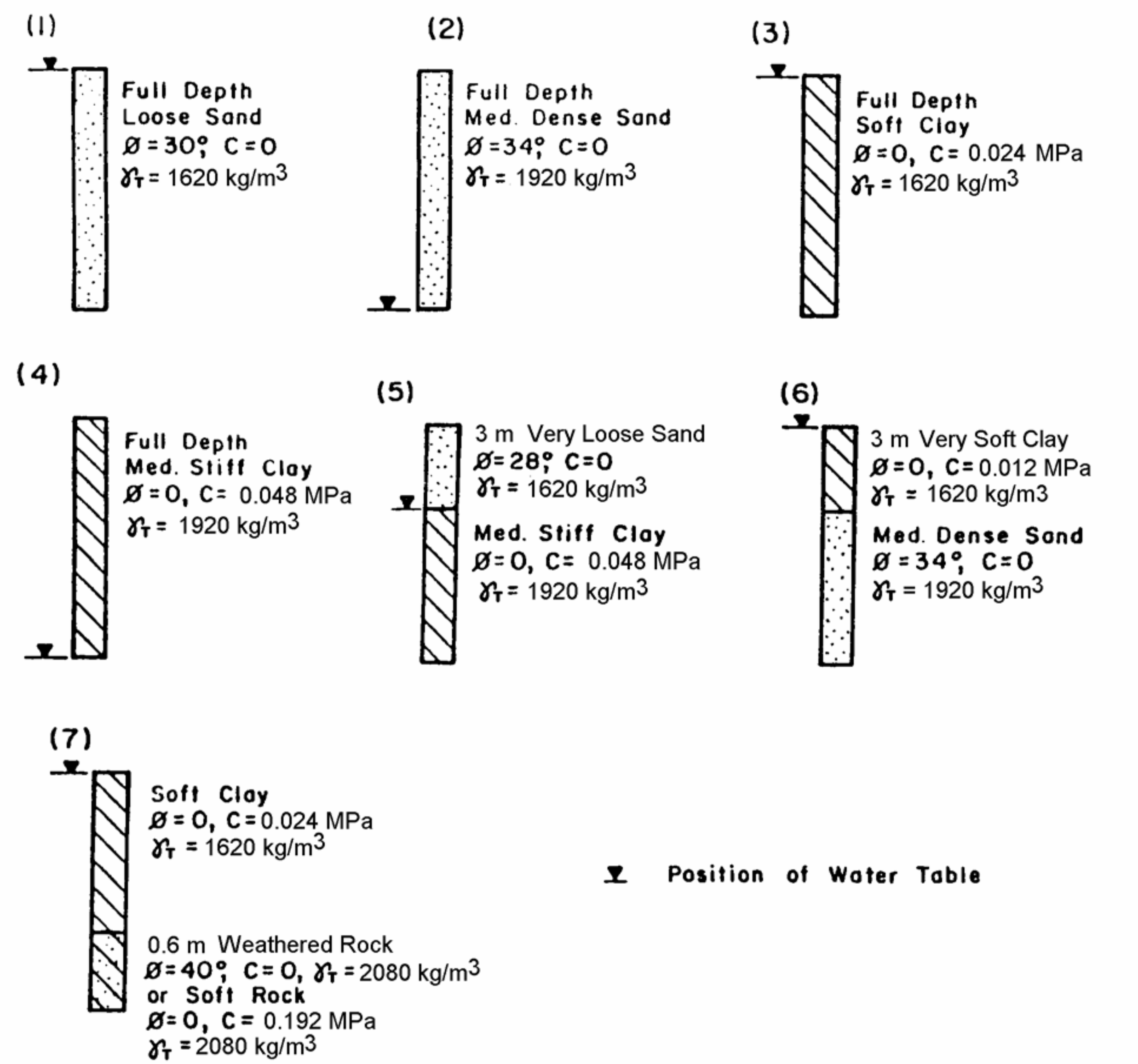


Figure 1.1.6-1 - Lateral Pile Loading Soil Profiles

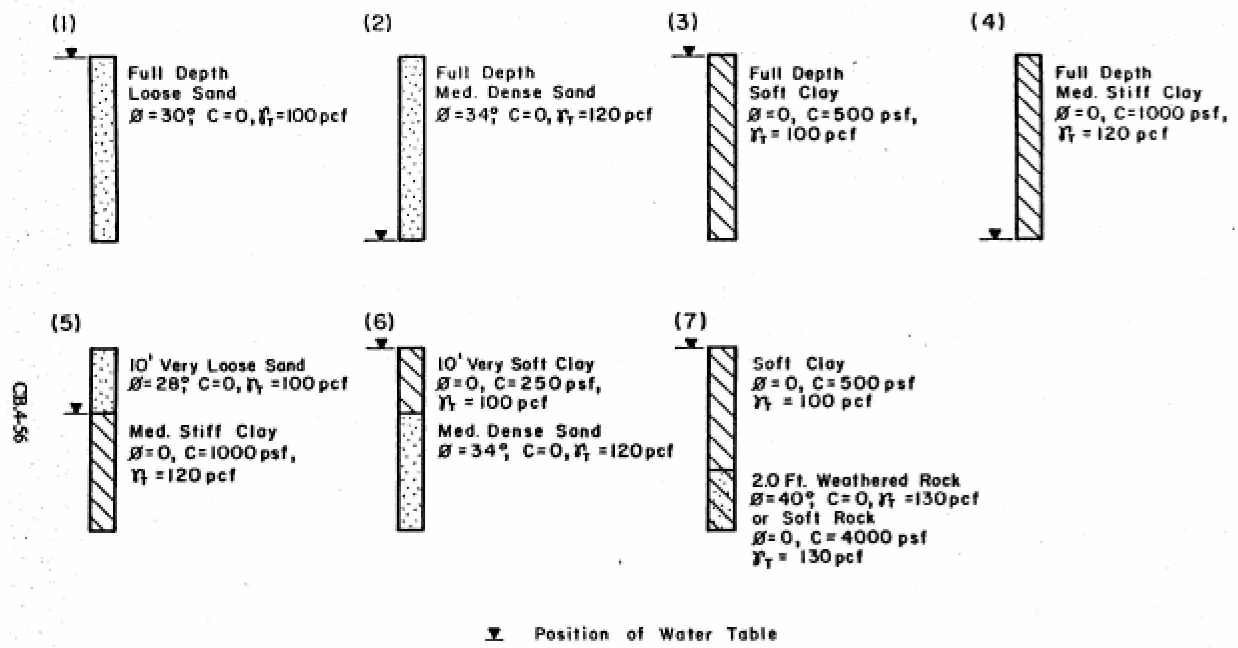


Figure 1.1.6-1 - Lateral Pile Loading Soil Profiles

1.1.7 Short Piles

The deflected shape of a short stiff pile in soft or loose soils indicates that the bottom of the pile is moving towards the load and that little bending in the pile is taking place. The stiffer the pile the more likely it is to act in this manner, given sufficient lateral load. A more flexible pile will bend with the load, and less movement will occur at the base of the pile. The lateral load versus deflection curves (see Figure 1.1.8-8) for Soil Profile 6, 4.5 m {15 ft} piles, illustrate the effect of this behavior. The stiffer HP 360x174 {14x117} (I = 508×10^6 mm^4) {1220 in^4}) shows greater deflection than the HP 310x110 {12x74} (I = 237×10^6 mm^4) {569 in4} for a lateral load of 36 kN {8 kips}. Therefore, for short piles, increasing the size of the pile may not produce greater lateral resistance based on deflections.

For very short piles, less than 4.5 m {15 ft} in depth, recommended lateral loads resistance are given in Tables 1.1.7-1, -2 and -3. Piles this short are likely to be end-bearing. Timber and precast piles are generally used as friction piles. Precast piles can be used for end-bearing if an H-pile section is cast in the tip. However, this would be uneconomical for such short piles. Therefore, the values in the above-referenced Tables apply to H-piles, pipe piles, and cast-in-place piles only.

Because of the extremely poor soil conditions in the surface 3 m {10 ft} of Soil Profiles 5 and 6, no lateral load shall be allowed for a pile less than 4.5 m {15 ft} long in these materials.

Table 1.1.7-1 - Lateral Loads Resistance for Steel H-Piles Less than 4.5 m in Length

Soil Profile	Pile Size	Lateral Load Resistance (kN)	Maximum Moment (m-kN)
1, 3, 7	250 X 62 to 310 X 110 {10x42 to 12x74}	9 {2}	12.2 {9}
1	360 X 174 {14x117}	0 {0}	0 {0}
3, 7	360 X 174 {14x117}	9 {2}	12.2 {9}
2, 4	250 X 62 {10x42}	27 {6}	20.3 {15}
2	250 X 85 to 310 X 110 360 X 174 {10x57 to 12x74} {14x117}	36 {8} 45 {10}	33.9 {25} 51.5 {38}
4	250 X 85 to 360 X 174 {10x57 to 14x117}	27 {6}	20.3 {15}

NOTES:

1. Values developed for pile length of 3 m {10 ft}, deflection at top of pile of about 6.25 mm {0.25 in}.
2. Values apply to both weak and strong axis of pile size specified.

Table 1.1.7-2 - Lateral Load Resistance for Pipe Piles Less that 4.5 m in Length

Soil Profile	Pile Diameter (mm)	Lateral Load Resistance (kN)	Maximum Moment (m-kN)
1, 3	All Sizes	0 {0}	0 {0}
2	270 {10.75} 400 {16} 600 {24}	27 {6} 27 {6} 27 {6}	19.0 {14} 24.4 {18} 38.0 {28}
4	270 {10.75} 400 {16} 600 {24}	27 {6} 36 {8} 45 {10}	16.3 {12} 27.1 {20} 39.3 {29}
7	270 {10.75}, 400, {16}, 600 {24}	18 {4} 27 {6}	16.3 {12} 36.6 {27}

NOTE: Values developed for pile length of 3 m, deflection at top of pile of about 6.25 mm {0.25 in}

Table 1.1.7-3 - Lateral Load Resistance for Cast-in-Place Piles Less than 4.5 m {15 ft} in Length

Soil Profile	Pile Diameter (mm)	Lateral Load Resistance (kN)	Maximum Moment (m-kN)
1	250, 300, 350, 450 {10, 12, 14, 18}	0 {0} 9 {2}	0 {0} 12.2 {9}
2, 4	250, 300, 350, 450 {10,12, 14, 18}	27 {6} 45 {10}	20.3 {15} 47.5 {35}
3	250, 300, 350, 450 {10, 12, 14, 18}	9 {2}	8.1 {6}
7	250, 300, 350, 450 {10, 12, 14, 18}	9 {2} 18 {4}	5.4 {4} 20.3 {15}

NOTE: Values developed for pile length of 3 m, deflection at top of pile of about 6.25 mm {0.25 in}

1.1.8 Using Design Curves

The design curves are a compilation of computed results for several pile types, sizes, and lengths for varying soil conditions. Design curves show deflection at the pile head versus lateral load and lateral load versus maximum moment for a particular soil condition and pile type, size, and length. The lateral load versus deflection curves shall be used with factored loads. The lateral load versus moment curves shall be used with factored lateral loads. In the case of multiple lateral loads for a particular group, a composite group factor shall be used. The maximum moment can be assumed to act within the top 3.6 m of the pile.

The lateral load design curves are presented as follows:

(a) Steel H-Piles – Figures 1.1.8-1 to 1.1.8-11 (Metric or U.S. Customary Units)
(b) Cast-in-Place Piles – Figures 1.1.8-12 to 1.1.8-21 (Metric or U.S. Customary Units)
(c) Pipe Piles – Figures 1.1.8-22 to 1.1.8-30 (Metric or U.S. Customary Units)
(d) Precast Concrete Piles – Figures 1.1.8-31 to 1.1.8-48 (Metric or U.S. Customary Units)
(e) Timber Piles – Figures 1.1.8-49 to 1.1.8-55 (Metric or U.S. Customary Units)

It is conservative to interpolate linearly between values of the moment of inertia (I), or pile size and pile length. The designer shall not extrapolate beyond the limits covered by the design curves, as in some cases higher lateral loads would cause soil failure. For piles less than 4.5 m in length use the lateral load resistances given in Tables 1.1.7-1, -2, and -3.

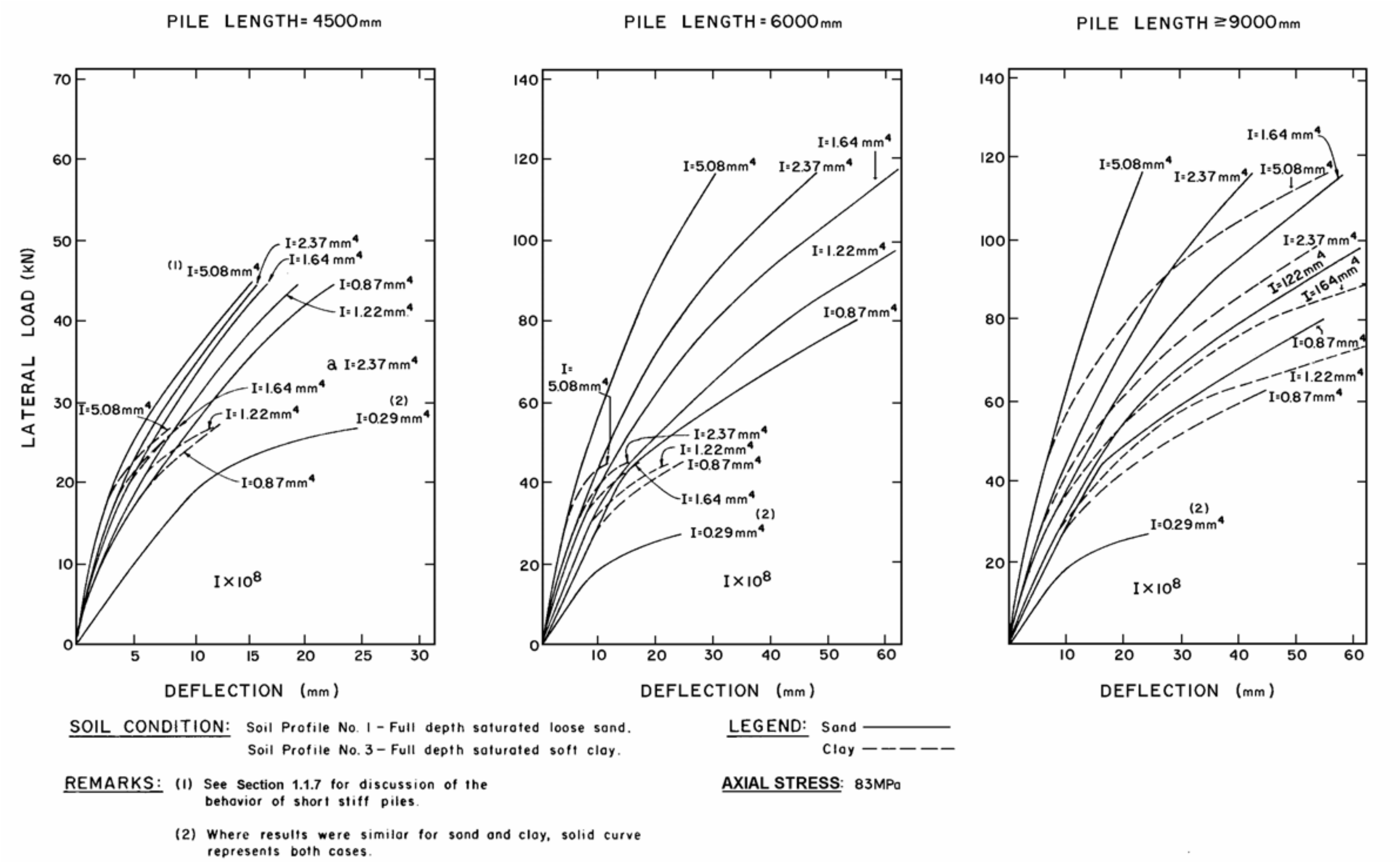
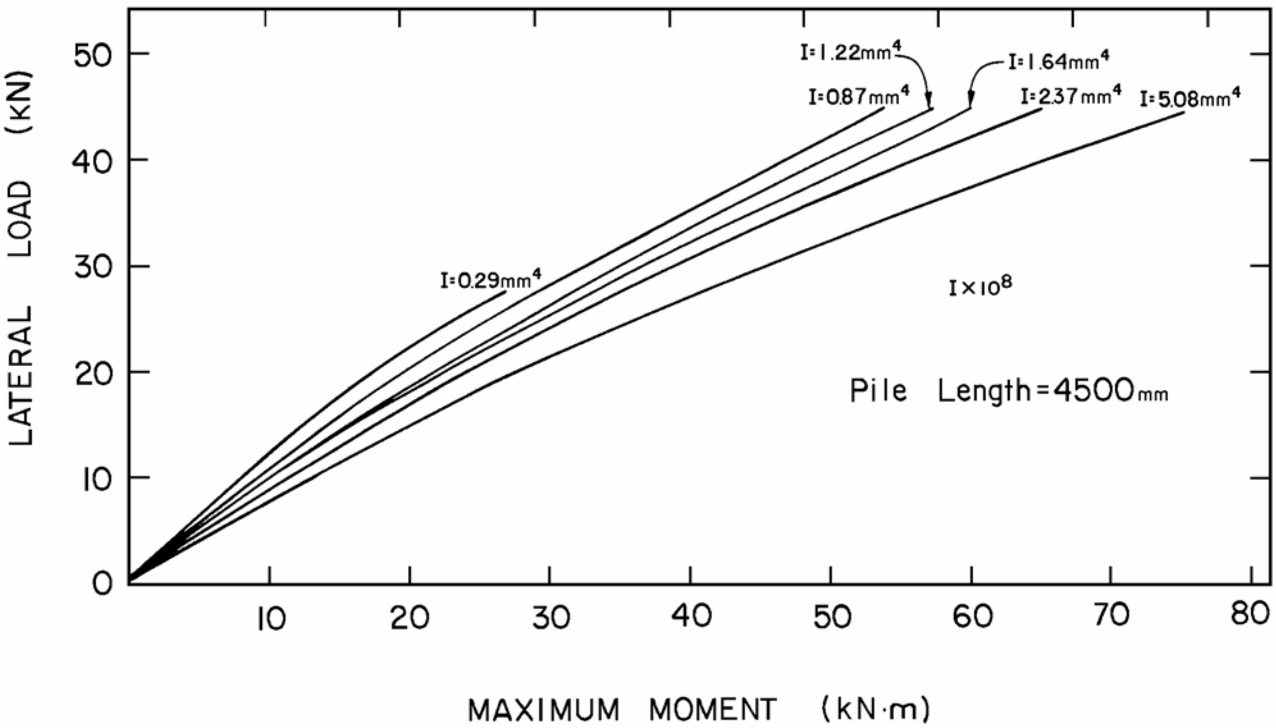


Figure 1.1.8-1 - Load Versus Deflection for Steel H-Pile, Soil Profiles 1 and 3



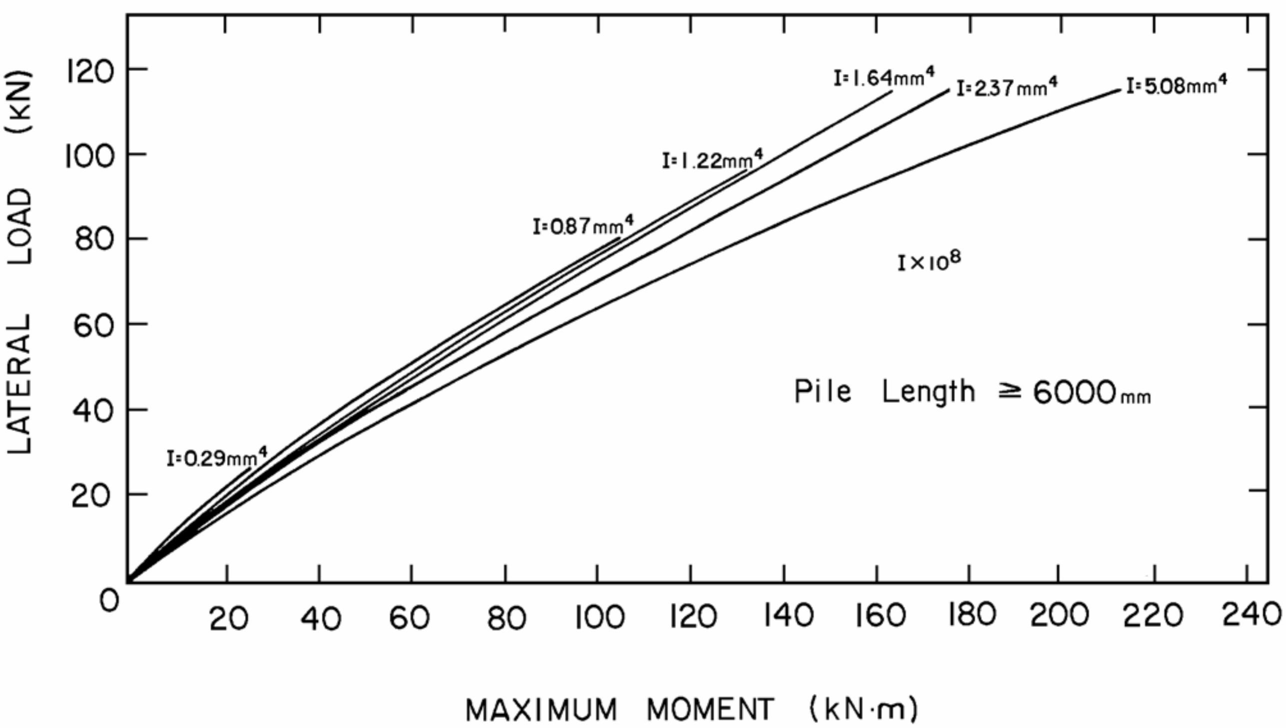
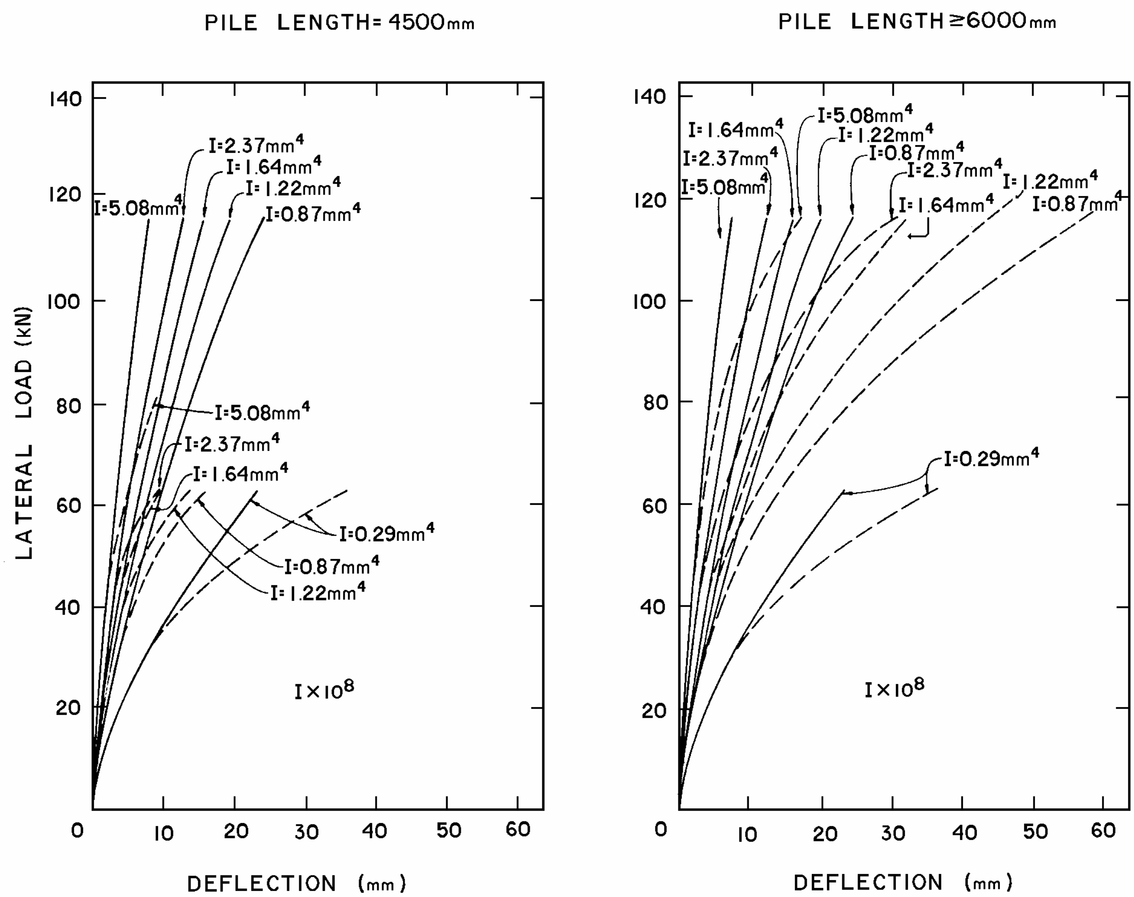


Figure 1.1.8-2 - Load Versus Maximum Moment for Steel H-Piles, Soil Profiles 1 and 3



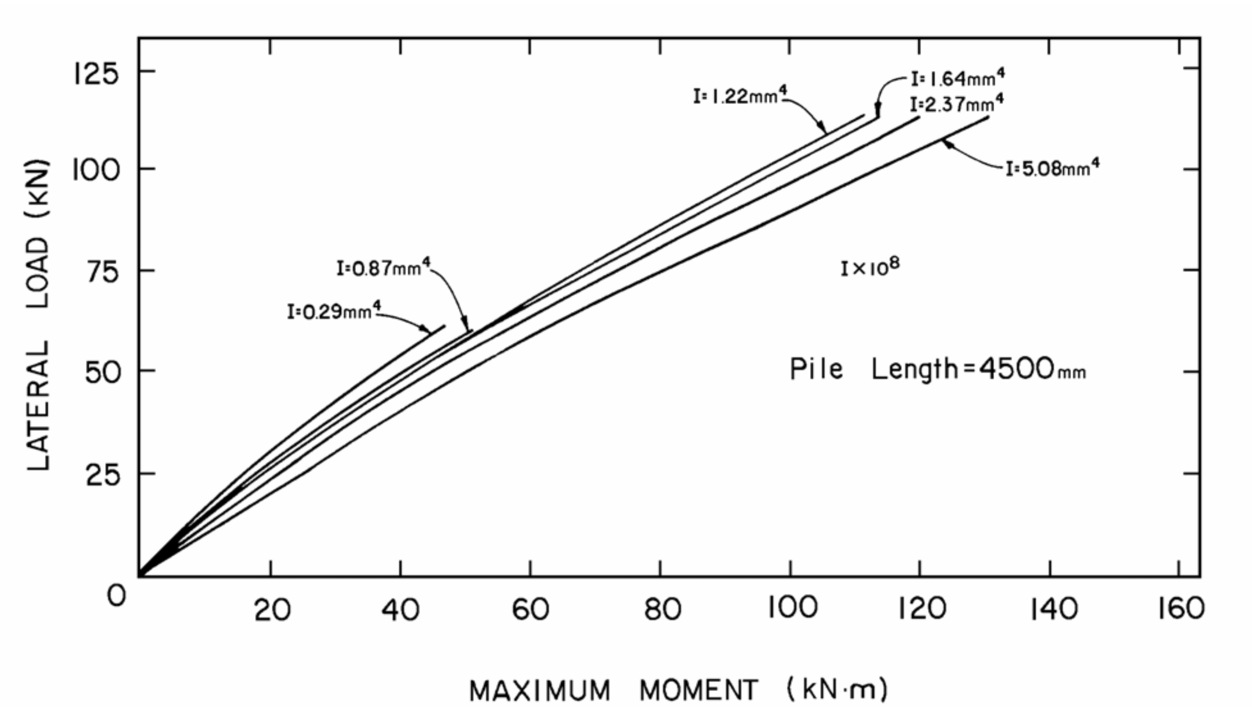
SOIL CONDITION: Soil Profile No. 2 - Full depth medium dense sand.
Soil Profile No. 4 - Full depth medium stiff clay.

LEGEND: Sand ———— Clay – – – –

REMARKS: Where results were similar for sand and clay solid curve represents both cases.

AXIAL STRESS: 83MPa

Figure 1.1.8-3 - Load Versus Deflection for Steel H-Pile, Soil Profiles 2 and 4



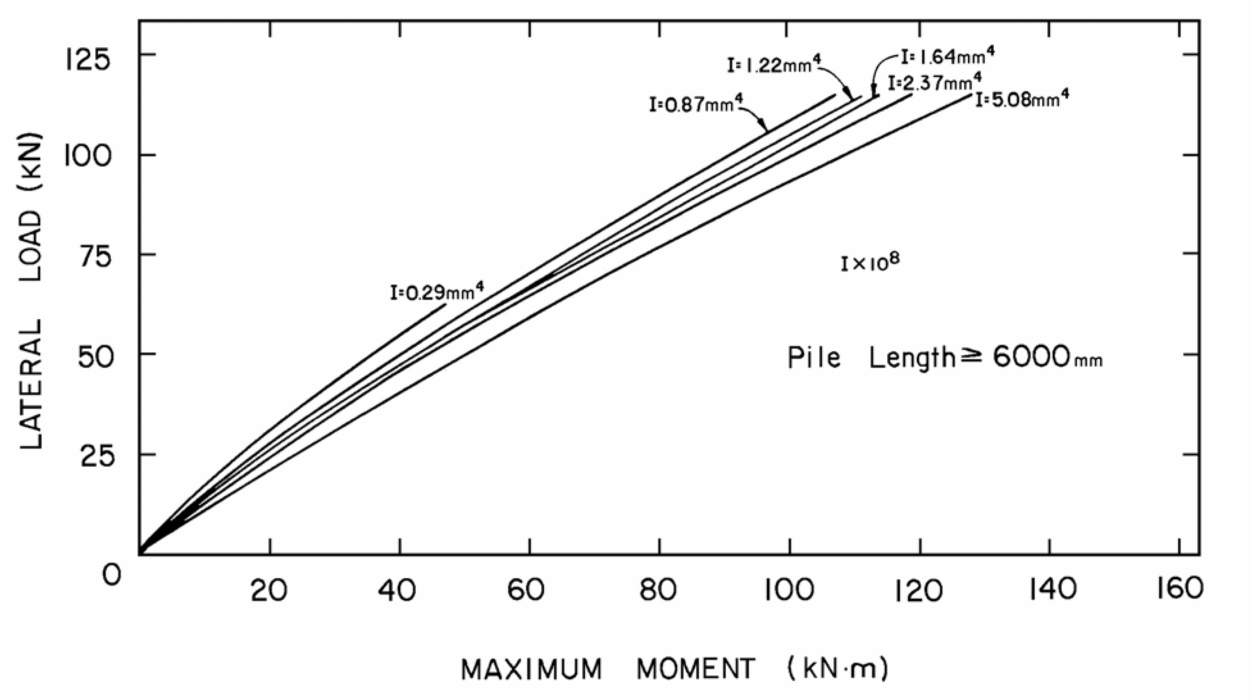
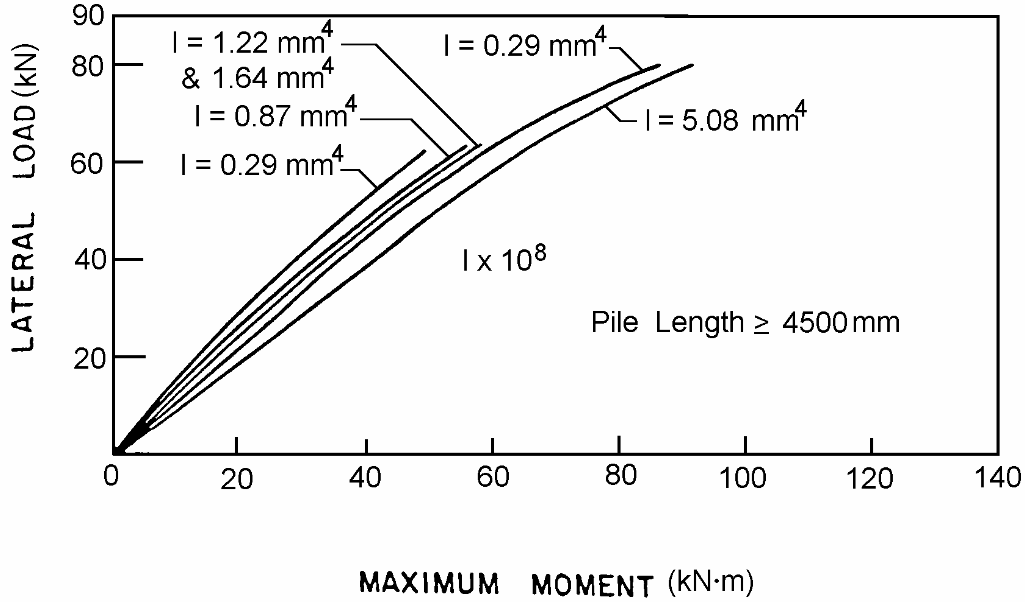


Figure 1.1.8-4 - Load Versus Maximum Moment for Steel, H-Pile, Soil Profile 2



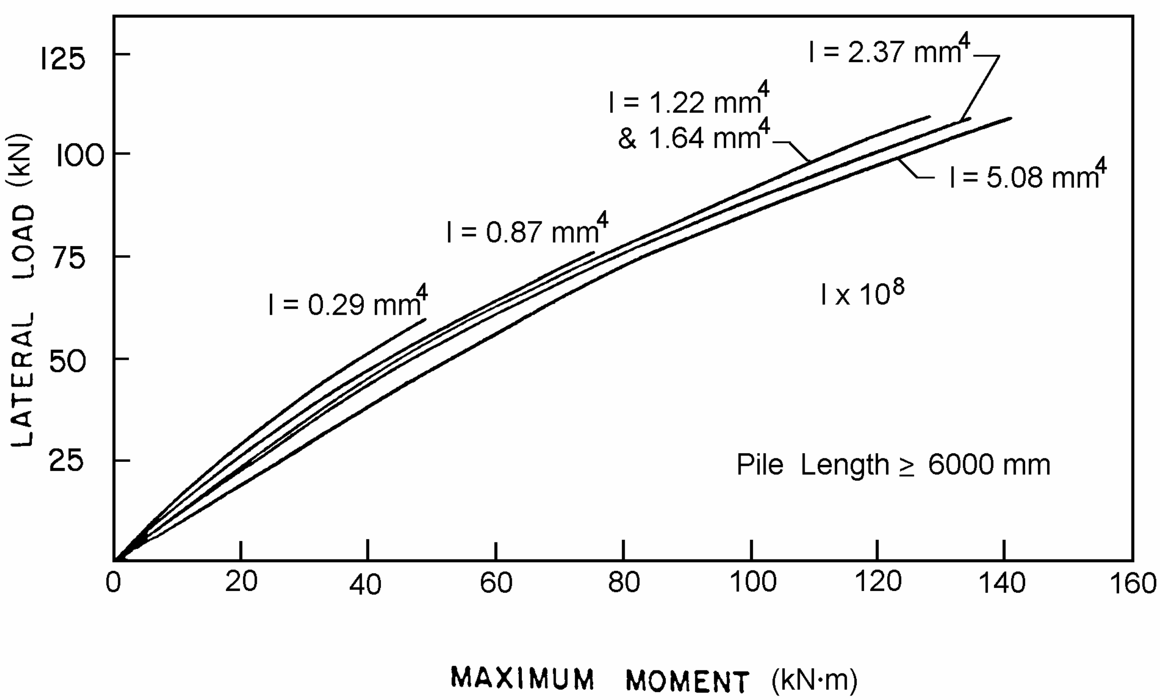


Figure 1.1.8-5 - Load Versus Maximum Moment for Steel H-Pile, Soil Profile 4

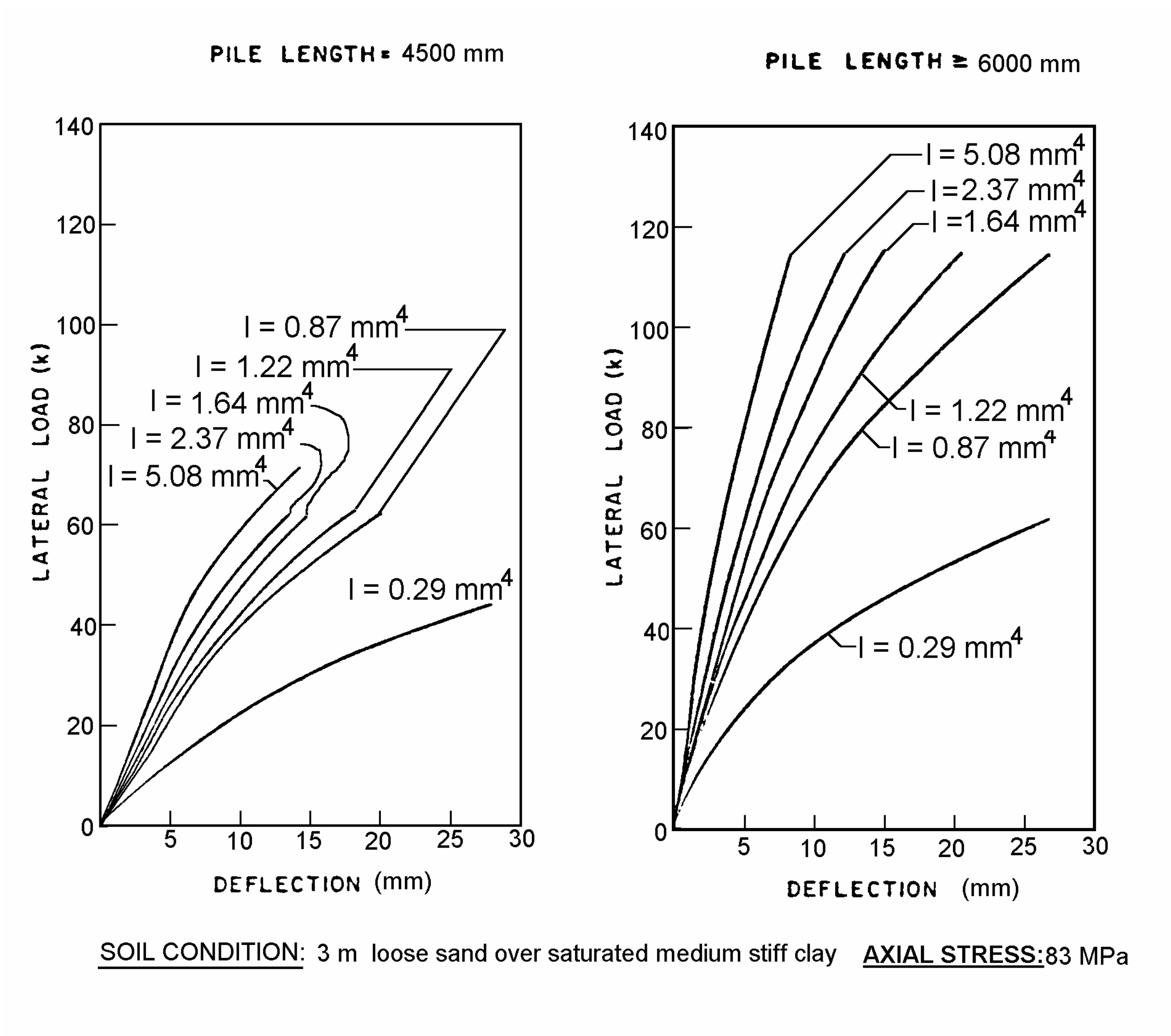


Figure 1.1.8-6 - Load Versus Deflection for Steel H-Pile, Soil Profile 5

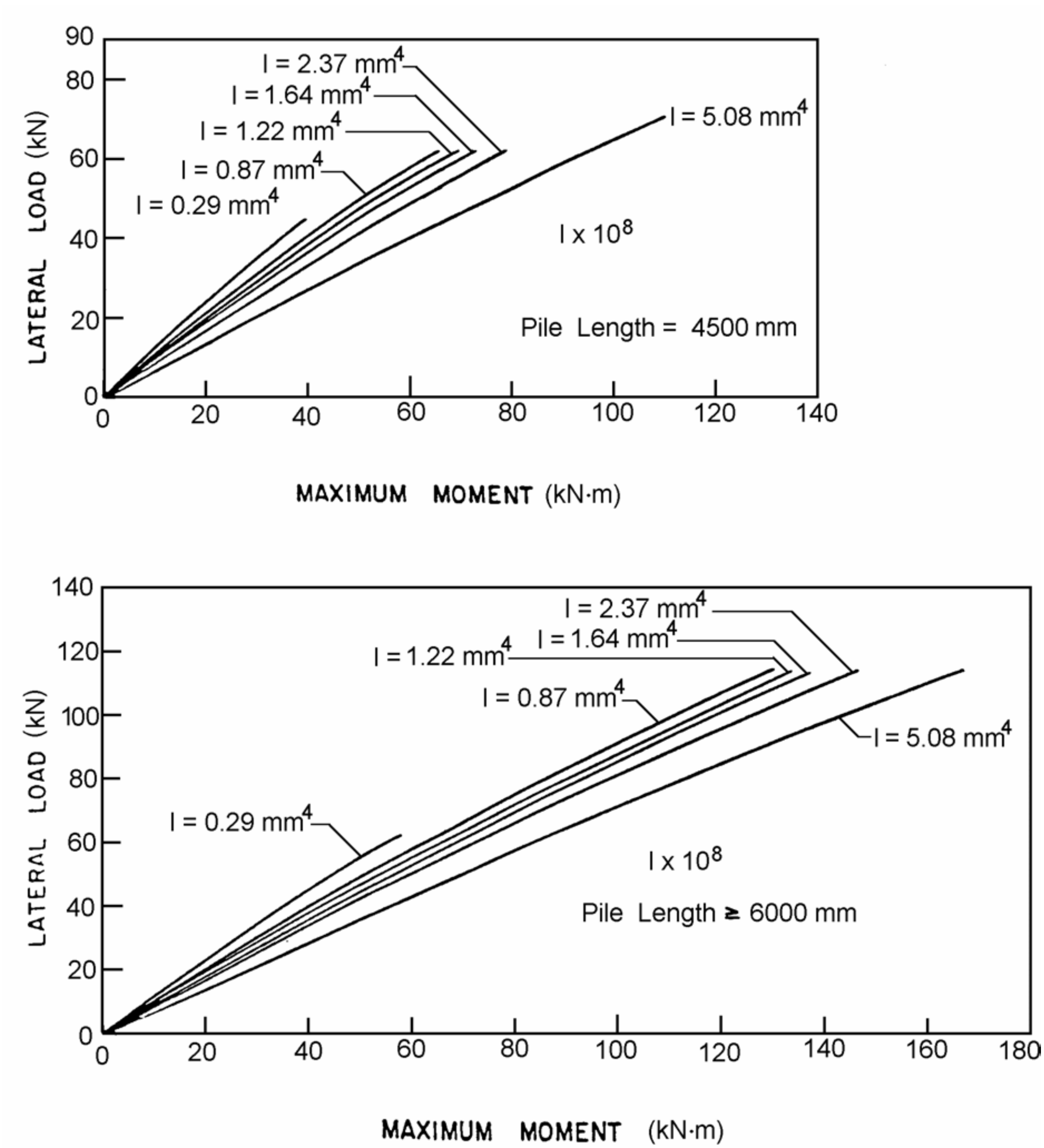


Figure 1.1.8-7 - Load Versus Maximum Moment for Steel H-Pile, Soil Profile 5

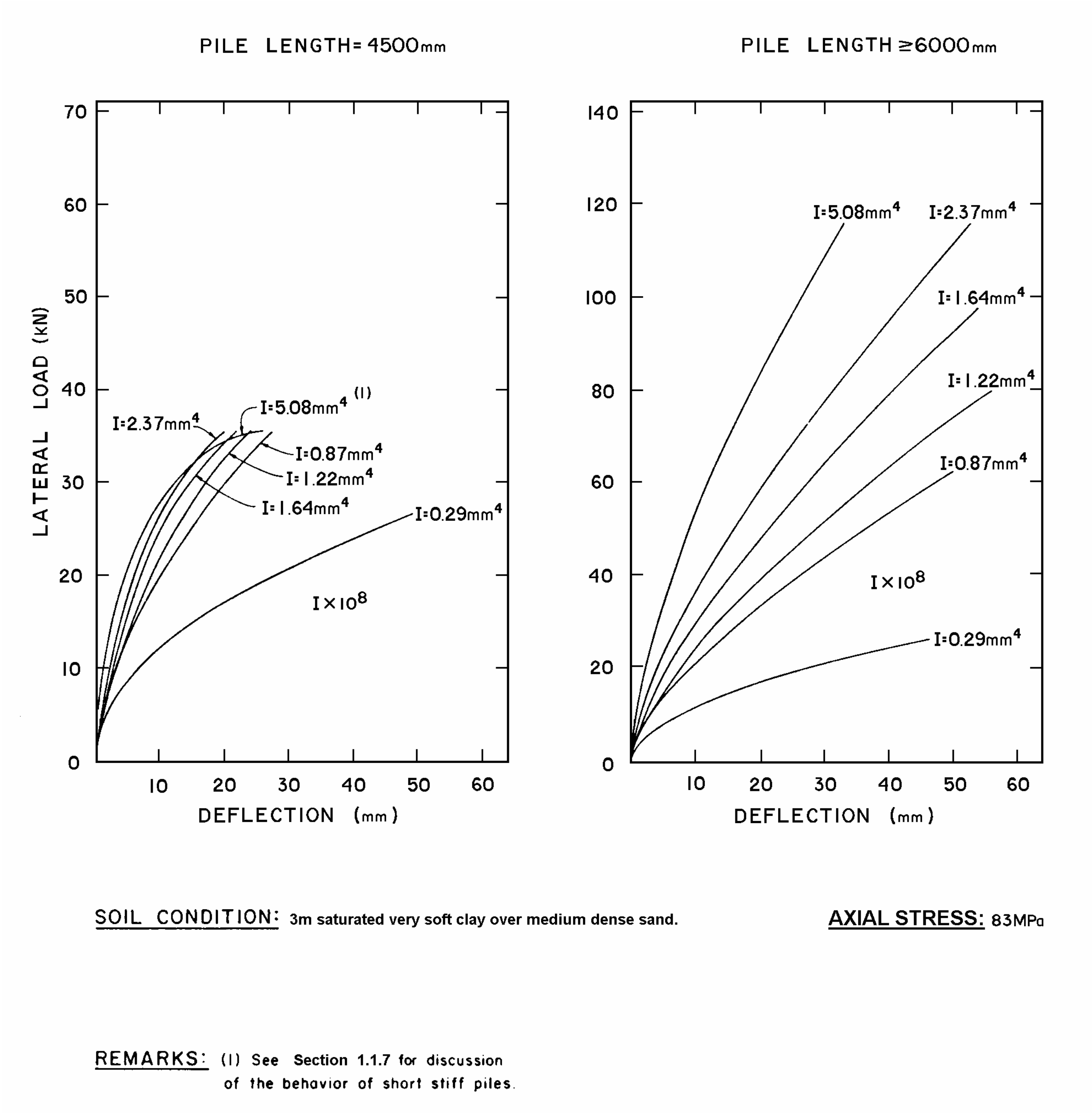
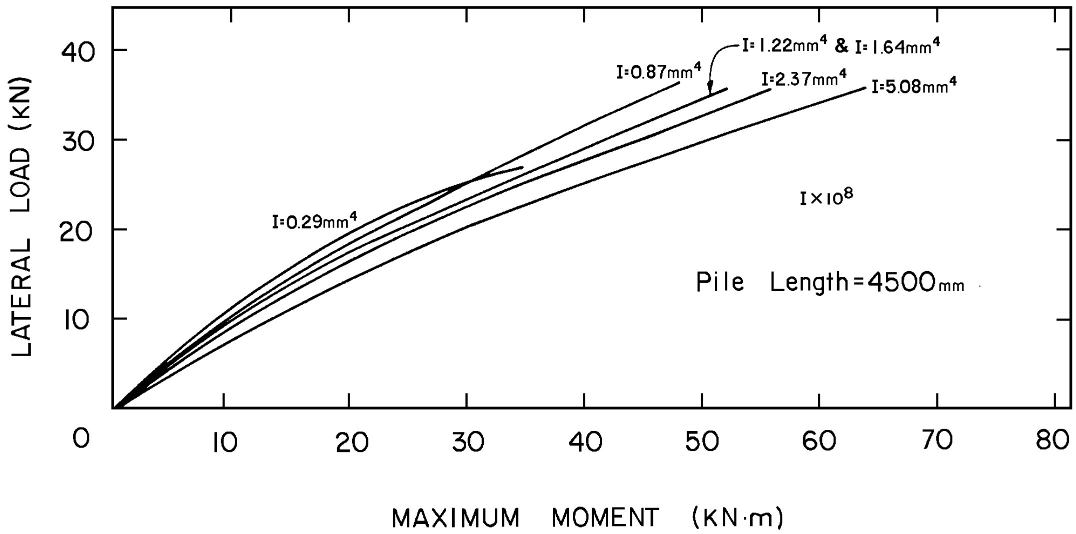


Figure 1.1.8-8 - Load Versus Deflection for Steel H-Pile, Soil Profile 6



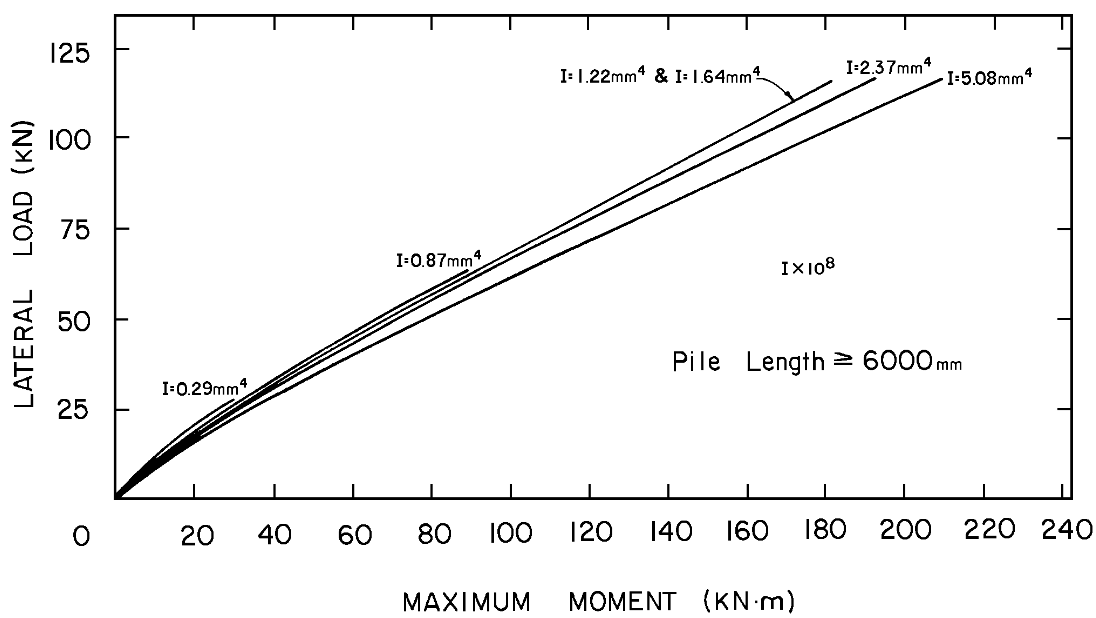
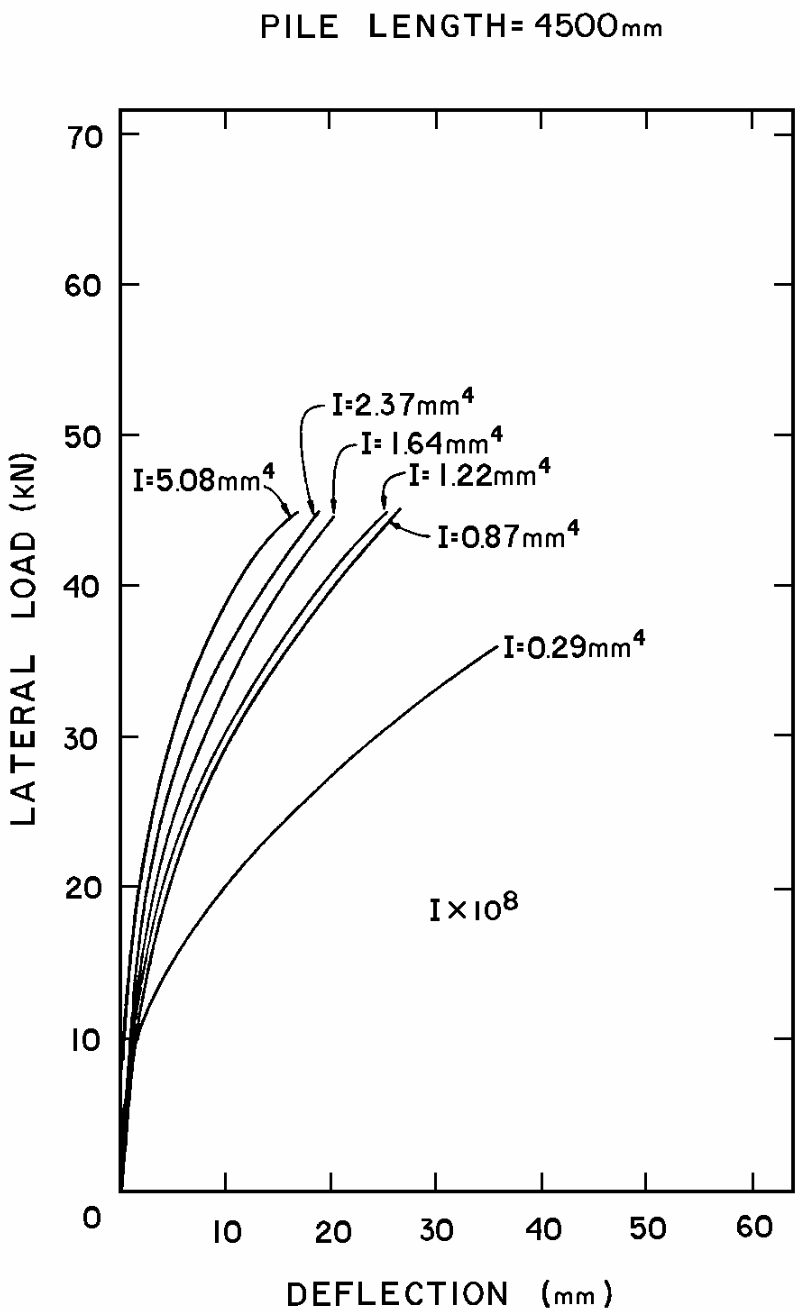


Figure 1.1.8-9 - Load Versus Maximum Moment for Steel H-Pile, Soil Profile 6



<u>SOIL CONDITION:</u> **Saturated soft clay over 600mm soft or weathered rock.**

<u>AXIAL STRESS:</u> 83MPa

Figure 1.1.8-10 - Load Versus Deflection for Steel H-Pile, Soil Profile 7

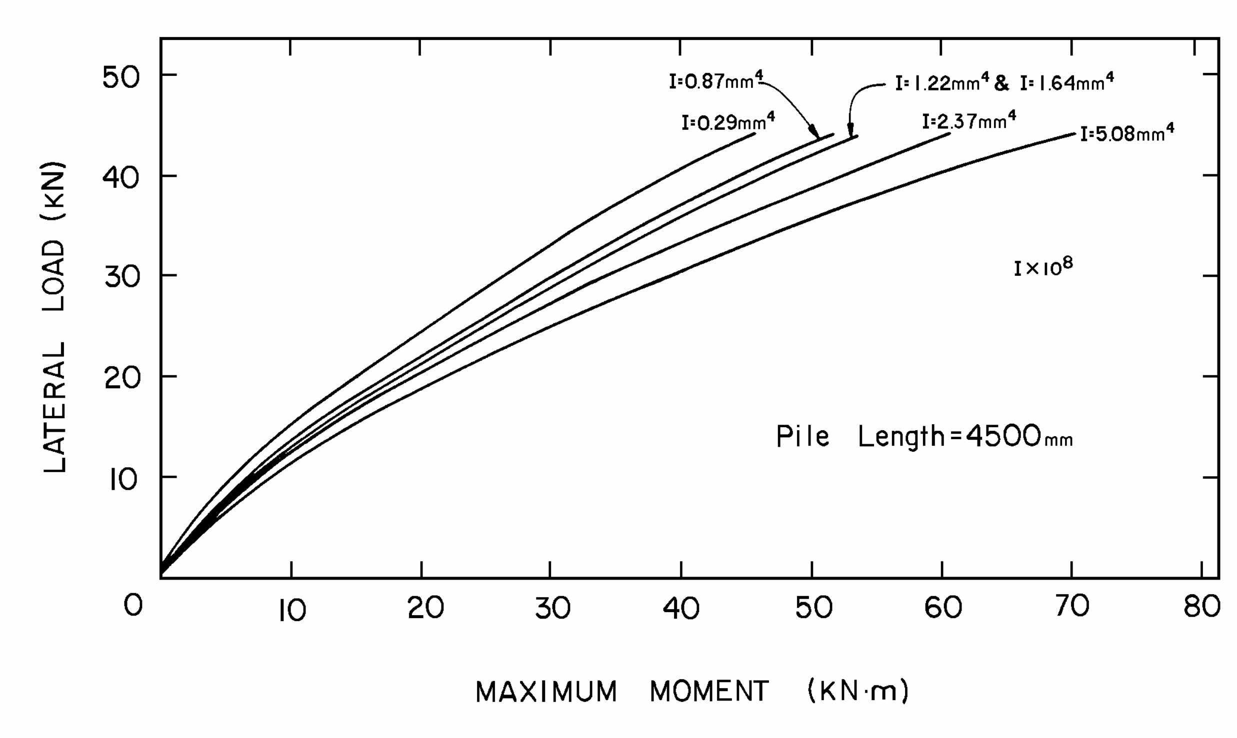


Figure 1.1.8-11 - Load Versus Maximum Moment for Steel H-Pile, Soil Profile 7

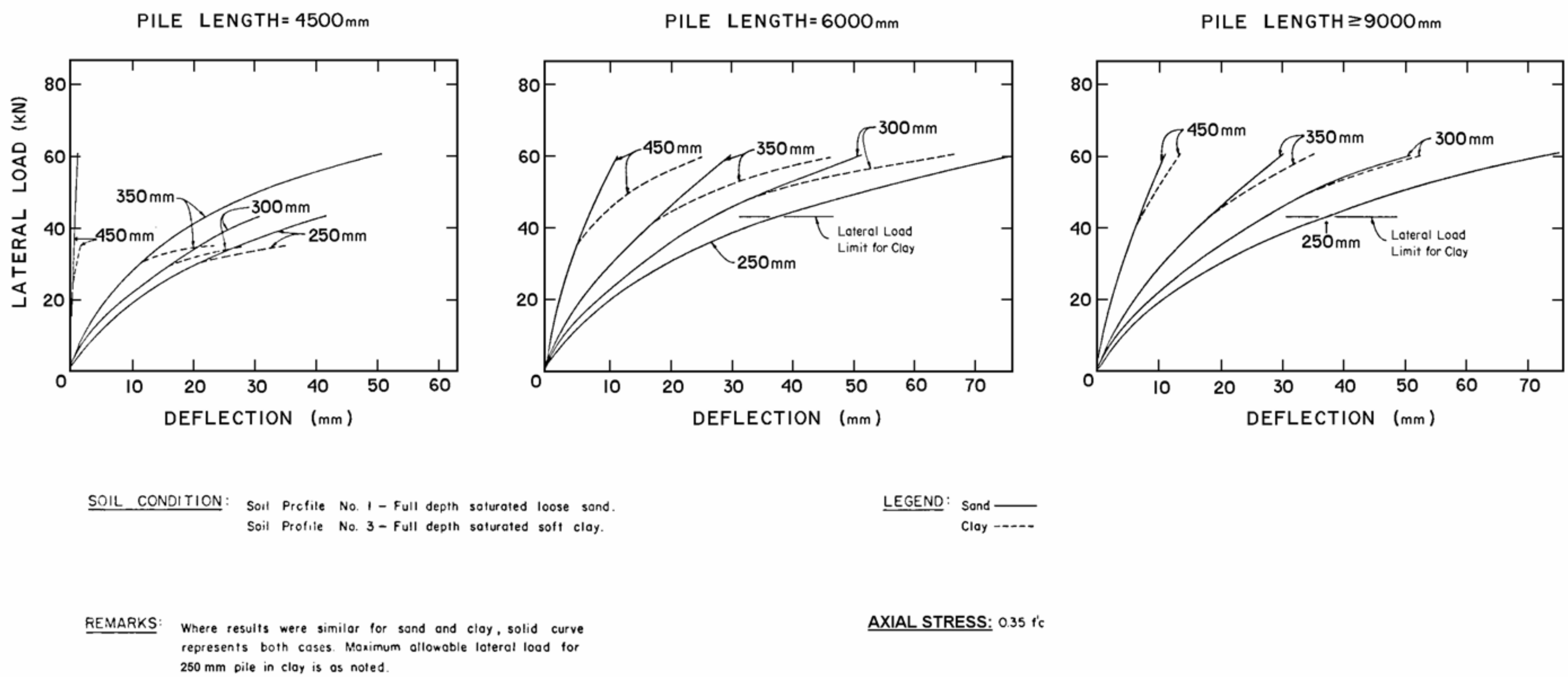


Figure 1.1.8-12 - Load Versus Deflection for Cast-in-Place Pile, Soil Profiles 1 and 3

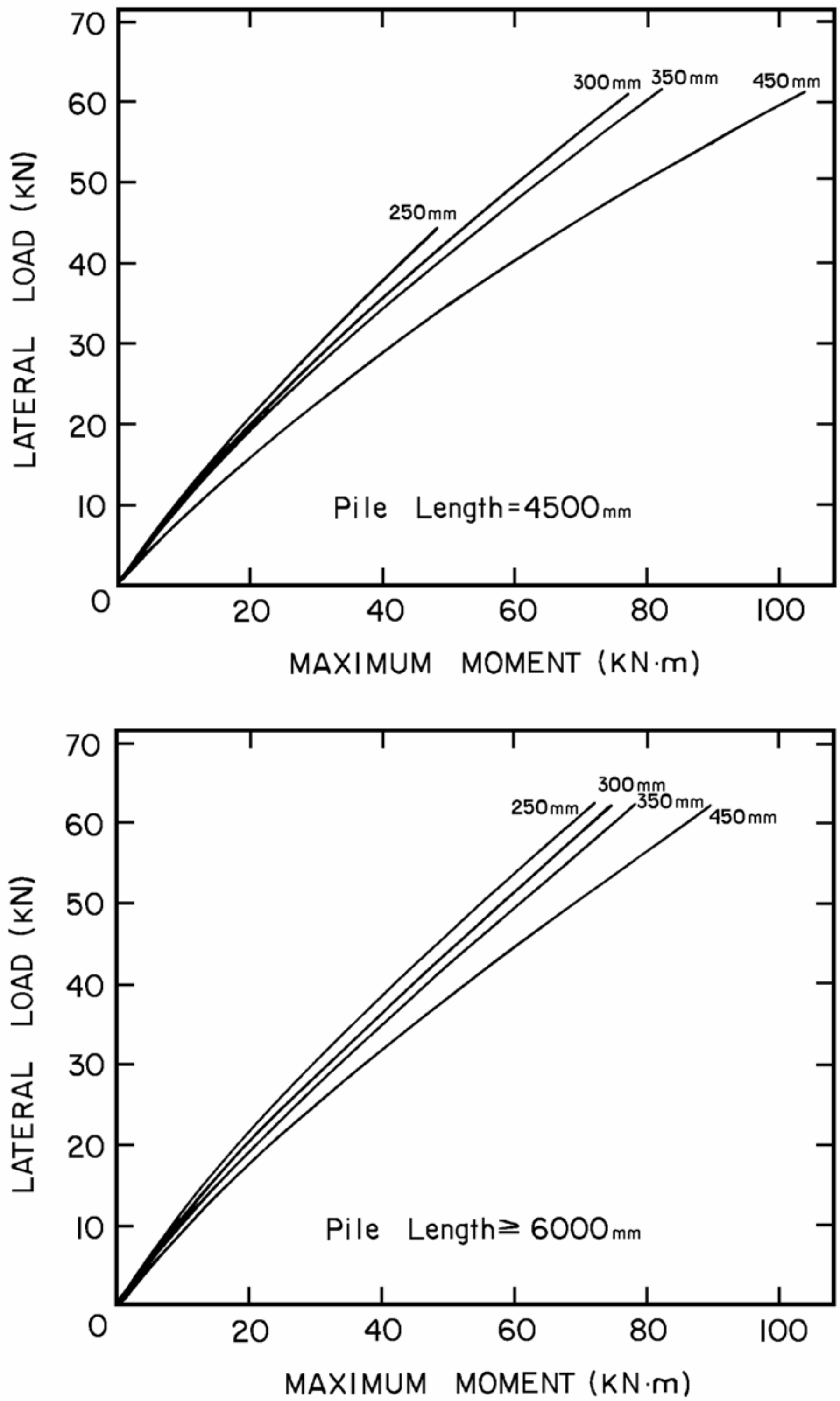


Figure 1.1.8-13 - Load Versus Maximum Moment for Cast-in-place Pile, Soil Profiles 1 and 3

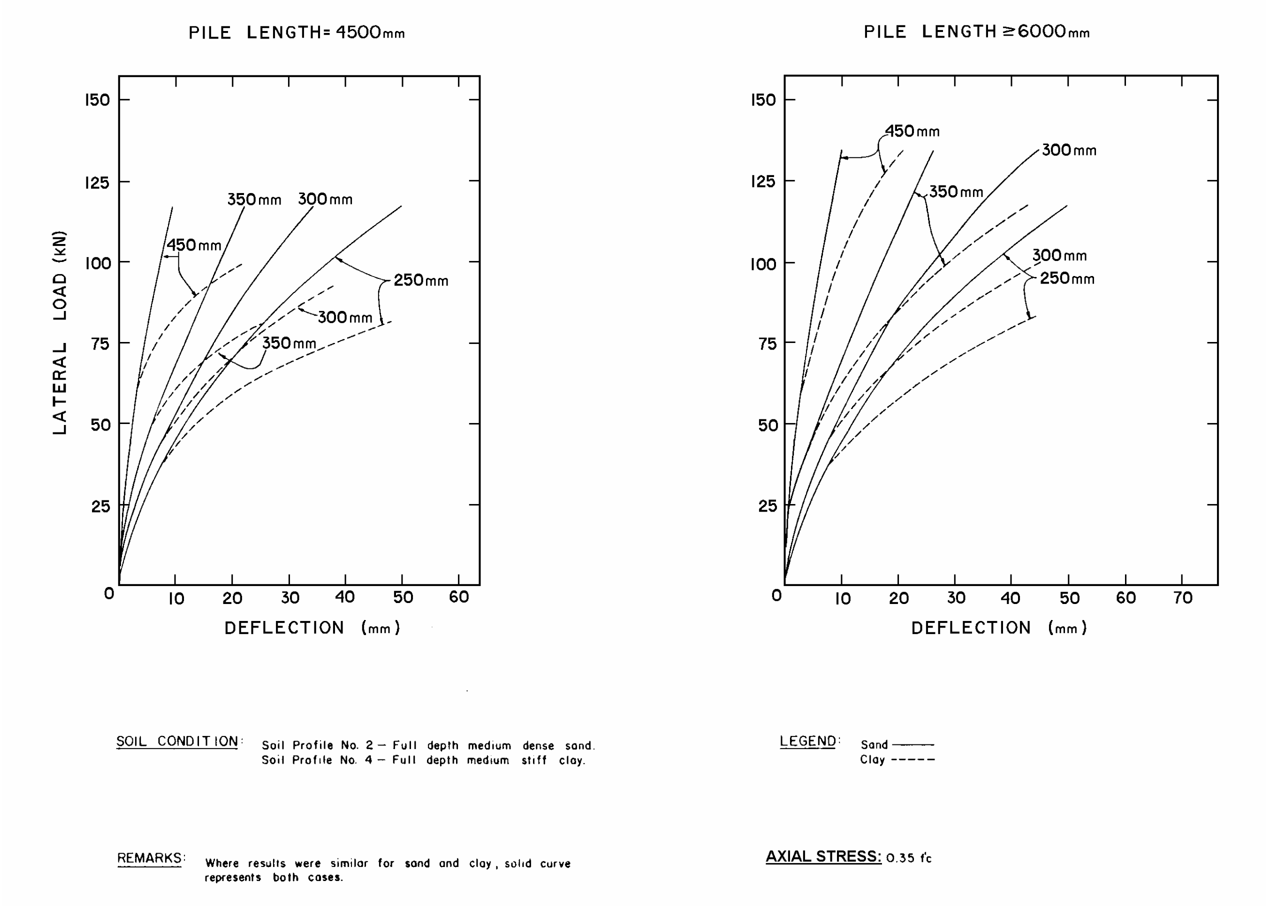


Figure 1.1.8-14 - Load Versus Deflection for Cast-in-Place Pile, Soil Profiles 2 and 4

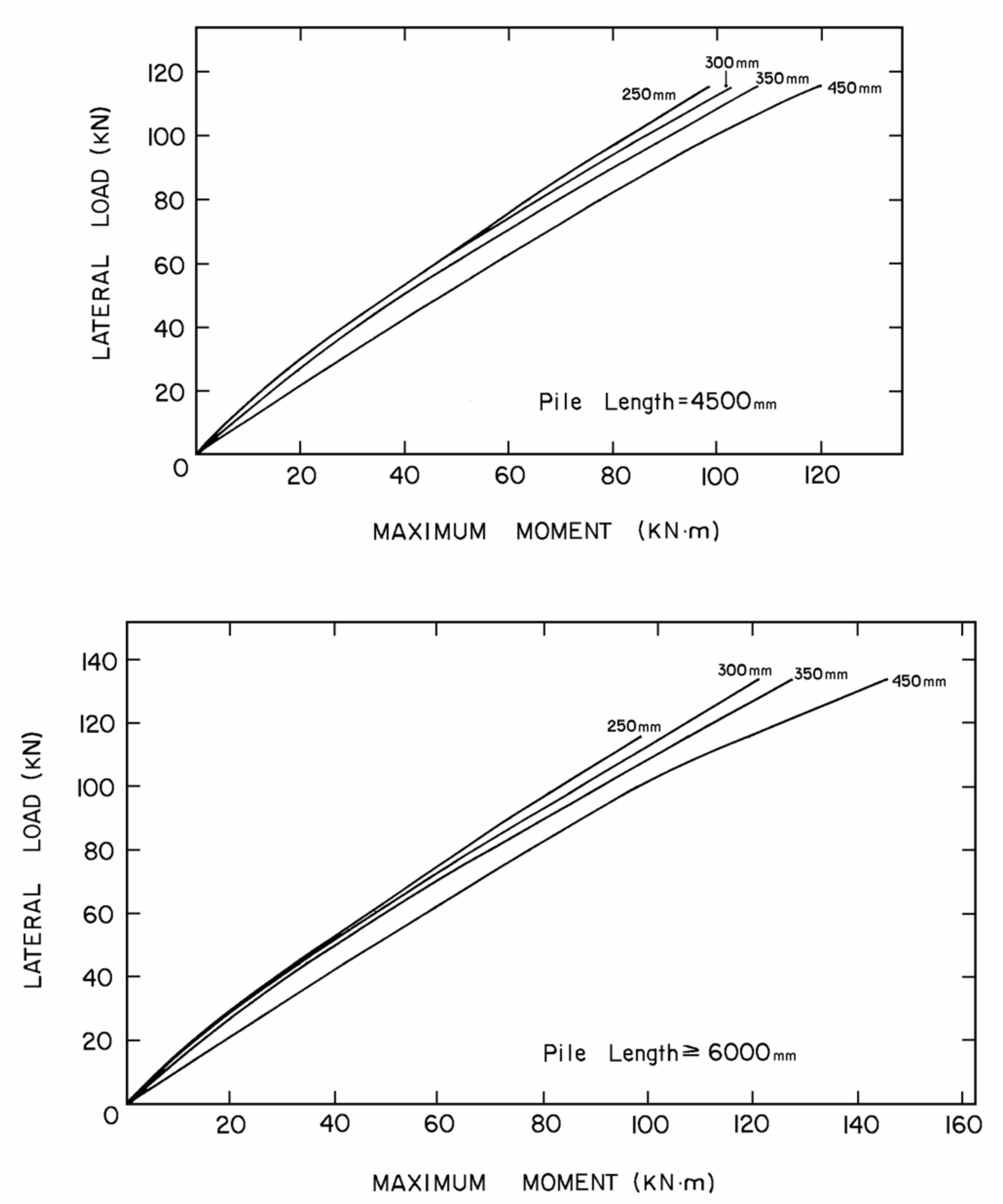
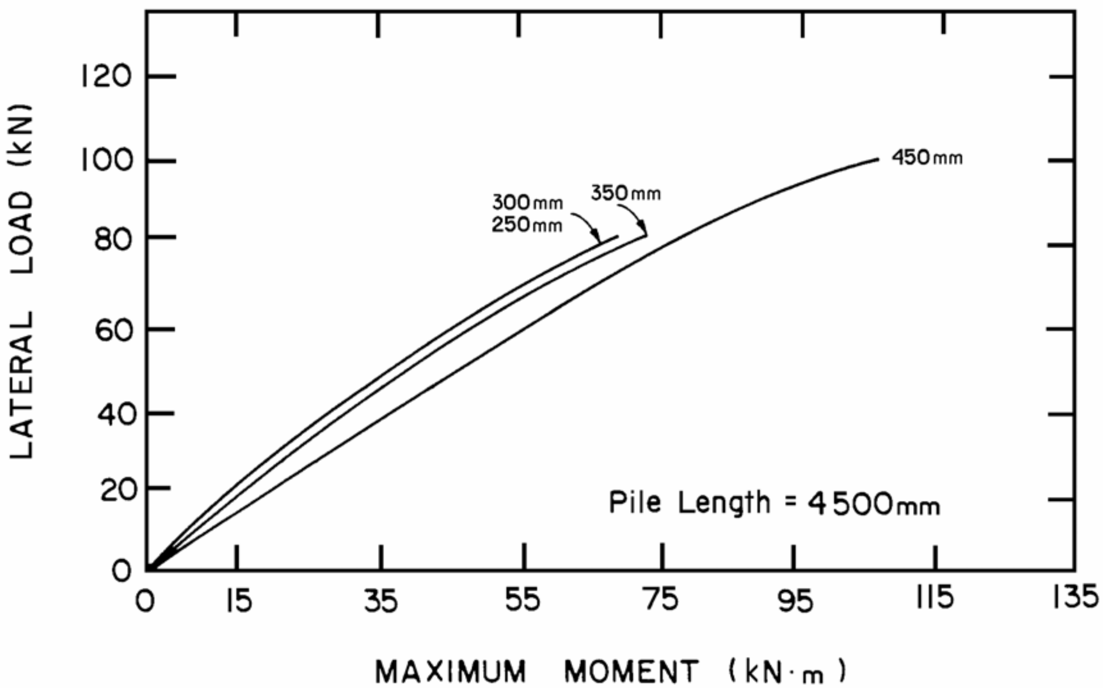


Figure 1.1.8-15 - Load Versus Maximum Moment for Cast-in-Place Pile, Soil Profile 2



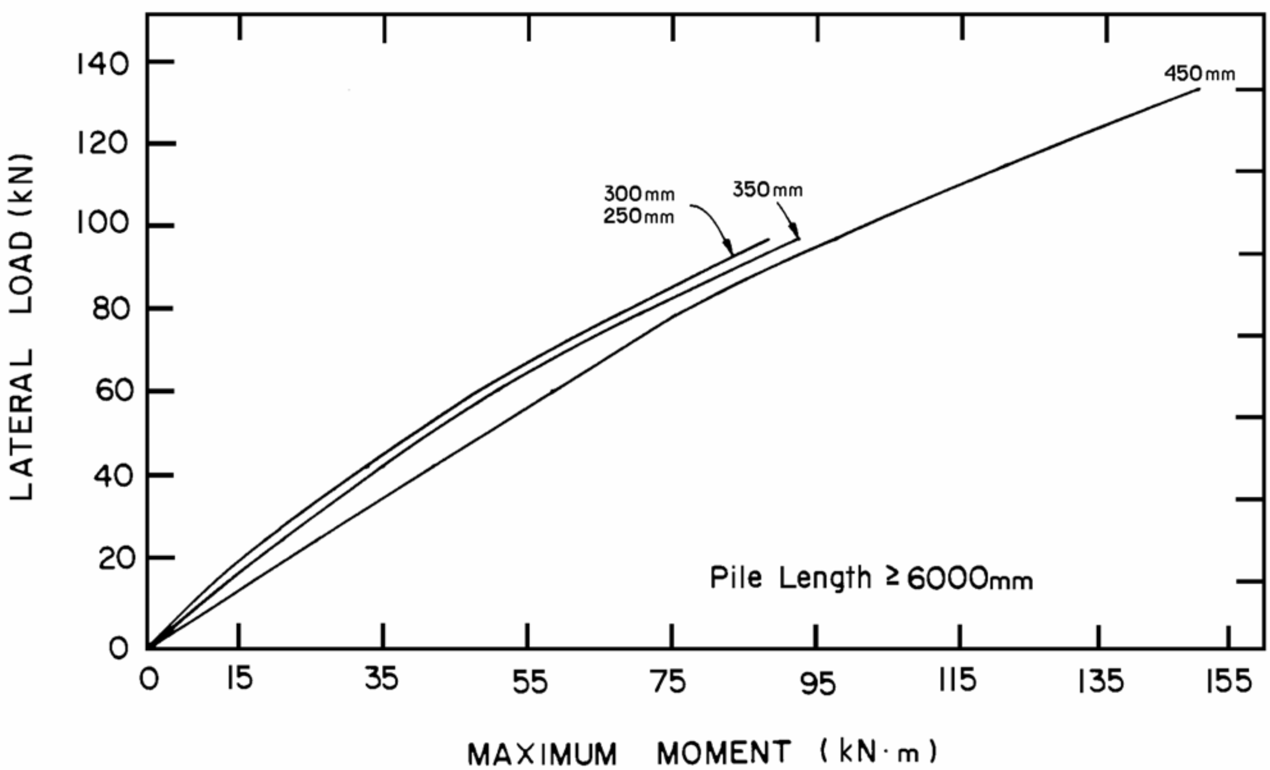


Figure 1.1.8-16 - Load Versus Maximum Moment for Cast-in-Place Pile, Soil Profile 4

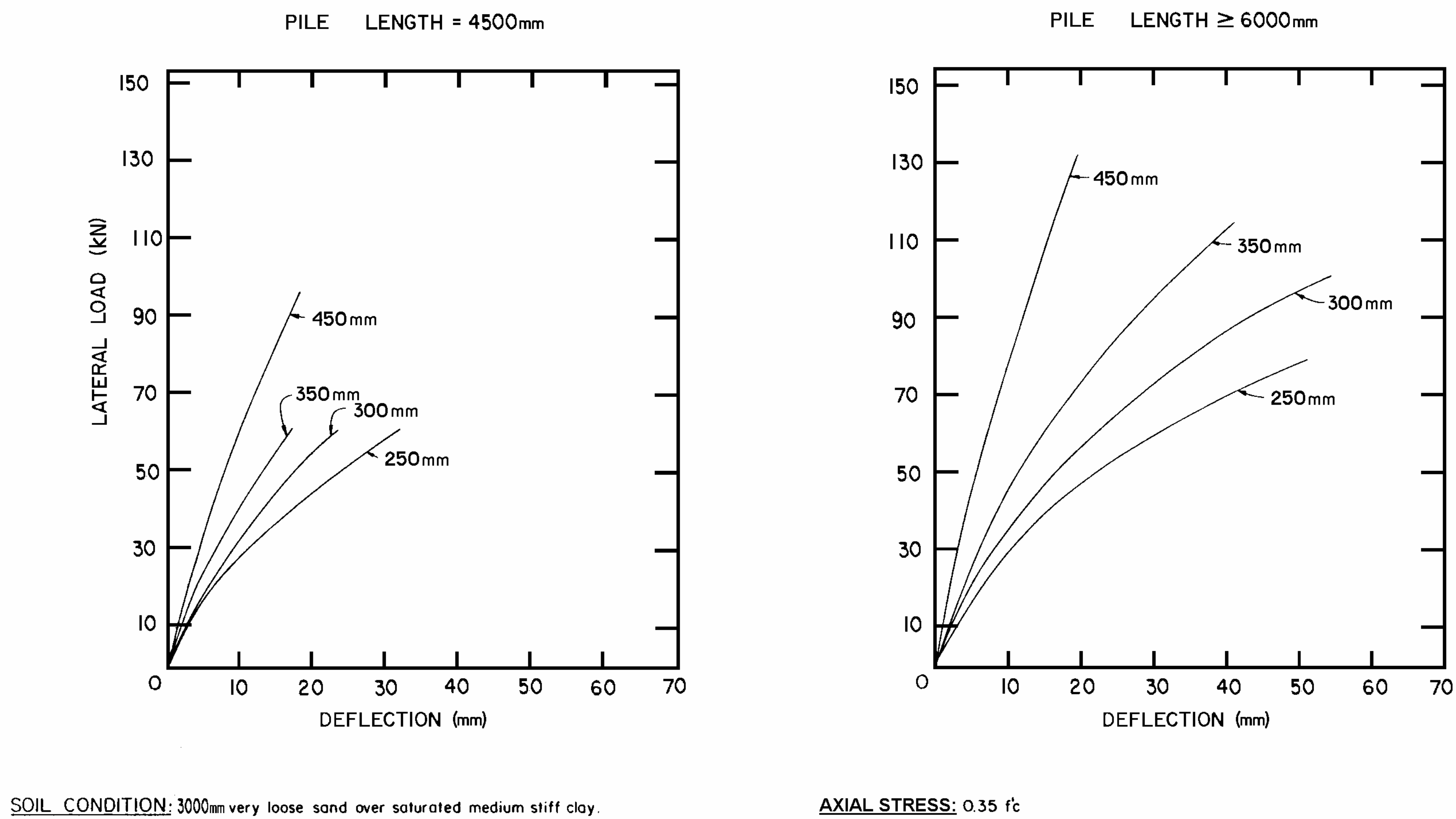
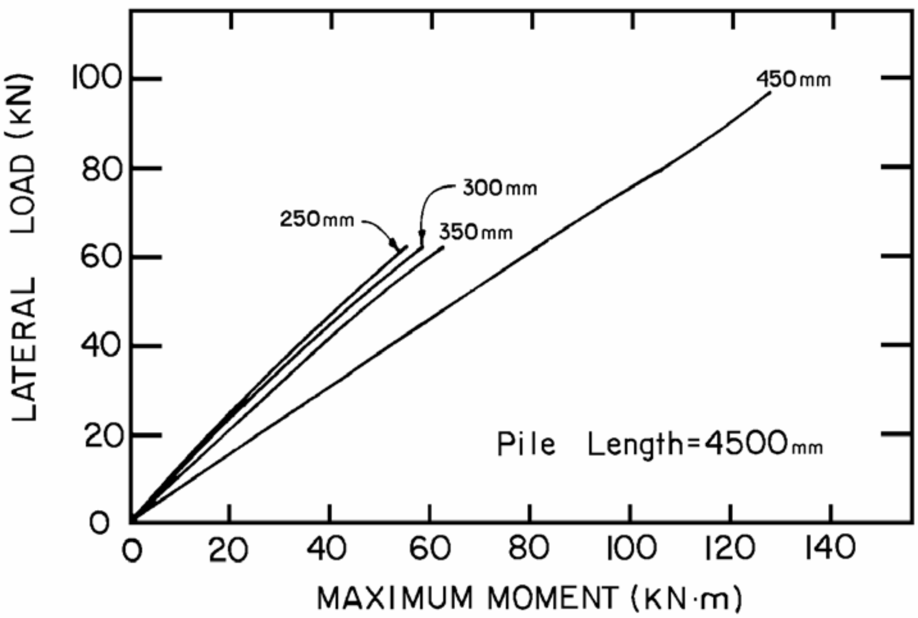


Figure 1.1.8-17 - Load Versus Deflection for Cast-in-Place Pile, Soil Profile 5



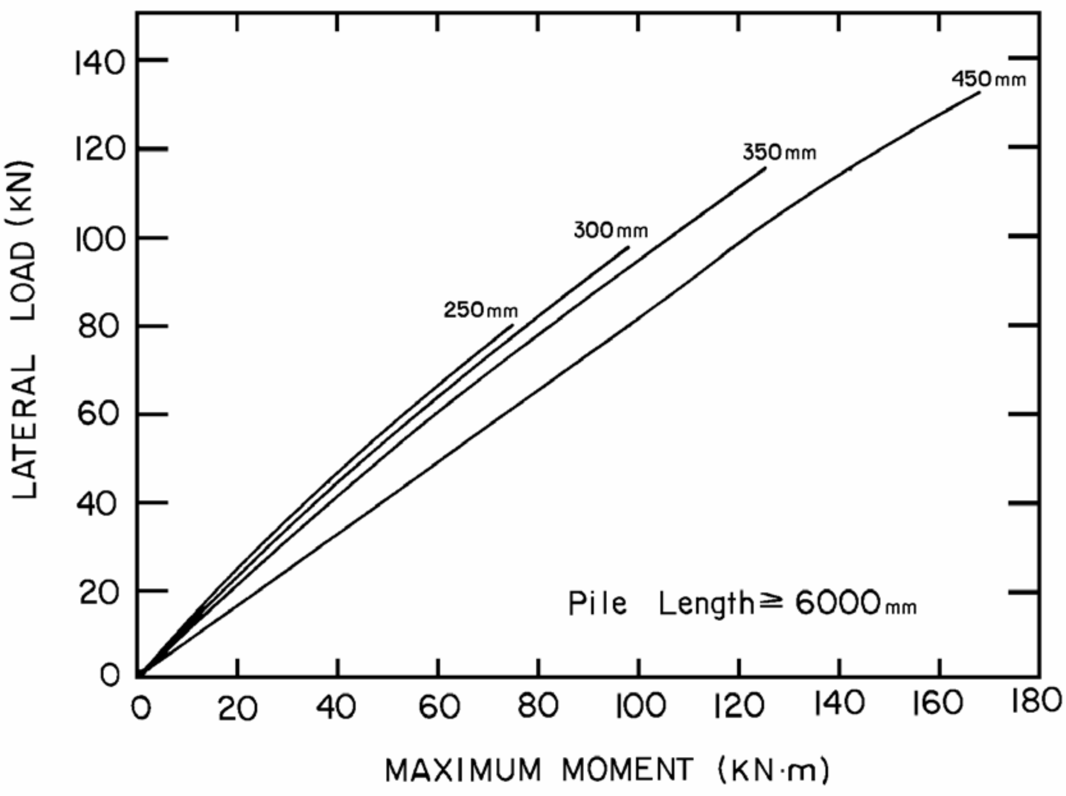
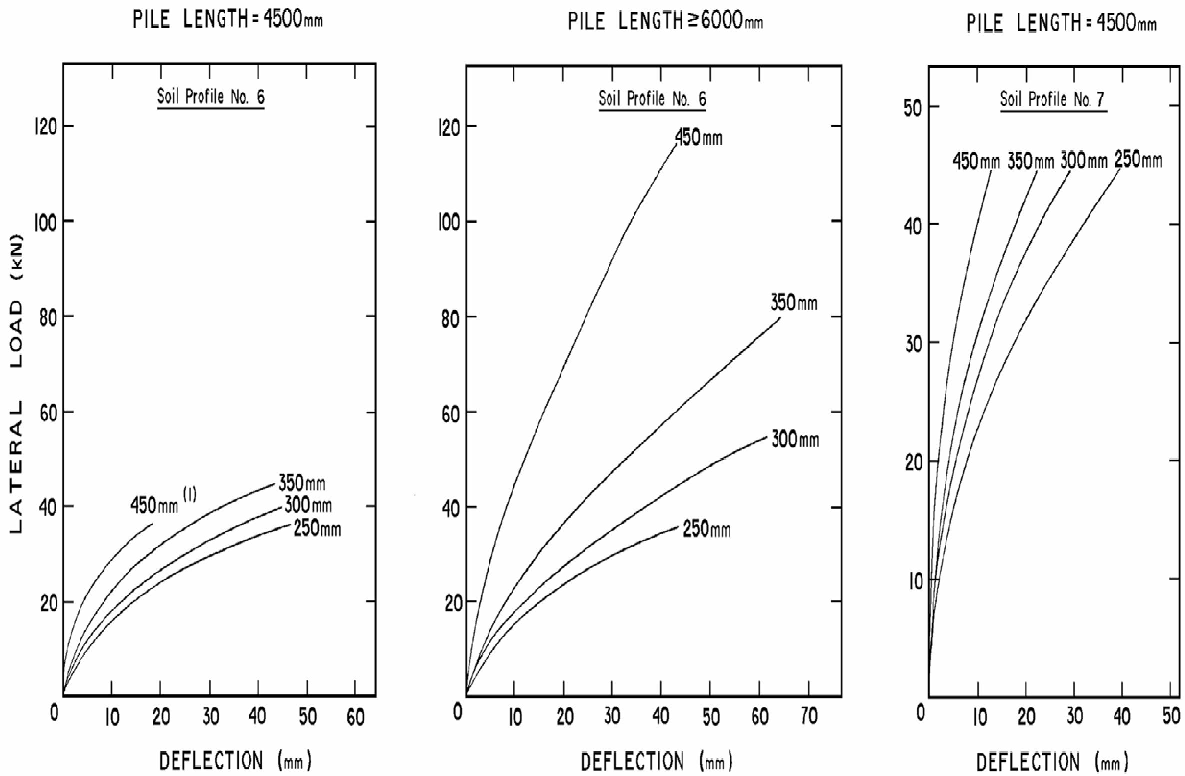


Figure 1.1.8-18 - Load Versus Maximum Moment for Cast-in-Place Pile, Soil Profile 5



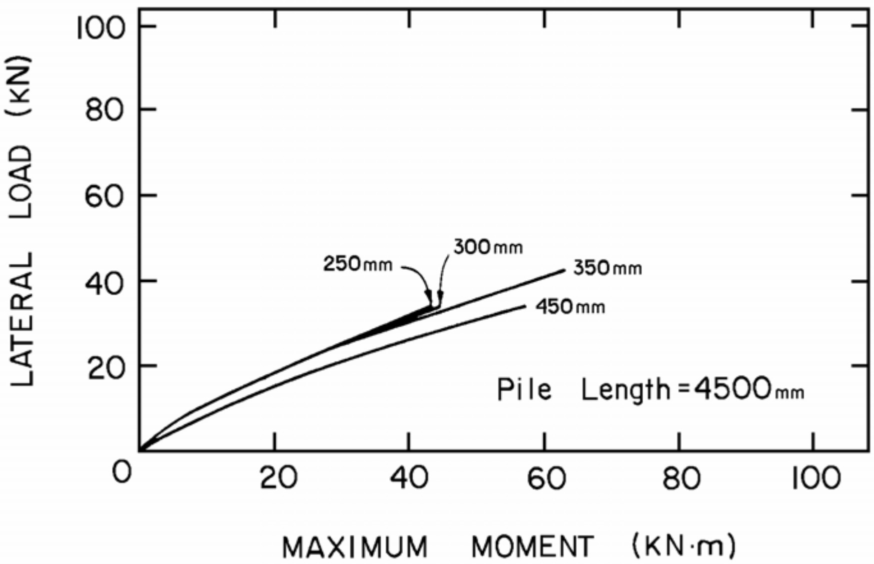
SOIL CONDITION: Soil Profile No. 6 – 3000mm saturated very soft clay over medium dense sand.

Soil Profile No. 7 – Saturated soft clay over 600mm soft or weathered rock.

AXIAL STRESS: 0.35 f'c

REMARKS: (1) See **Section 1.1.7** for discussion of the behavior of short stiff piles.

Figure 1.1.8-19 - Load Versus Deflection for Cast-in-Place Pile, Soil Profiles 6 and 7



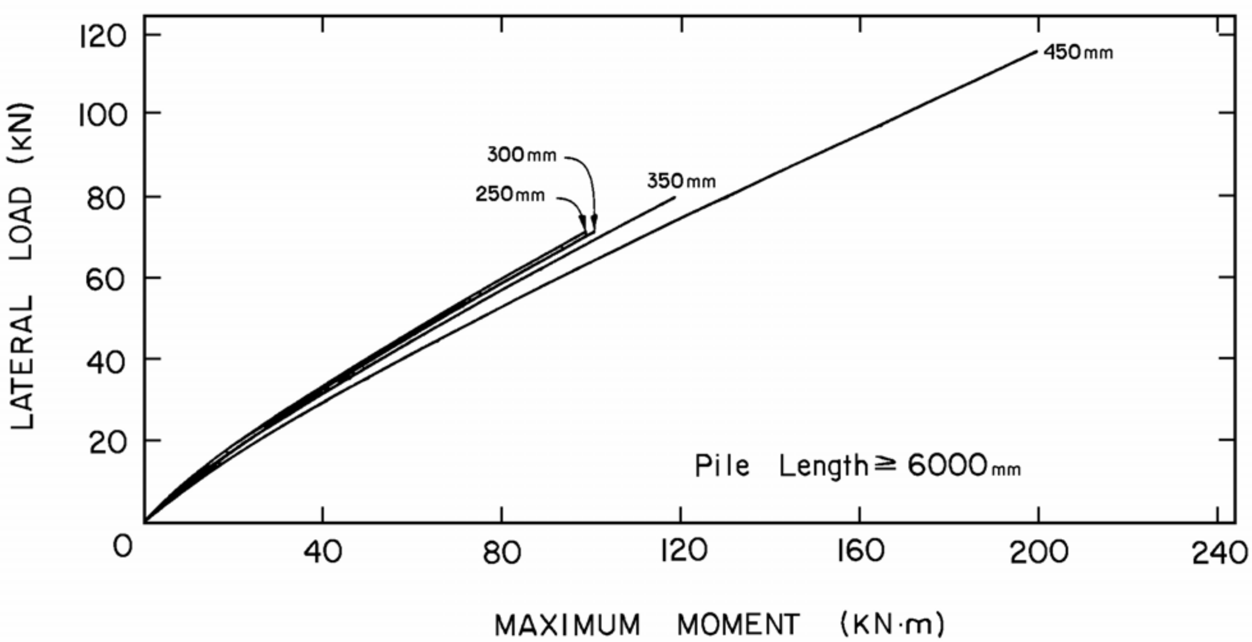


Figure 1.1.8-20 - Load Versus Maximum Moment for Cast-in-Place Pile, Soil Profile 6

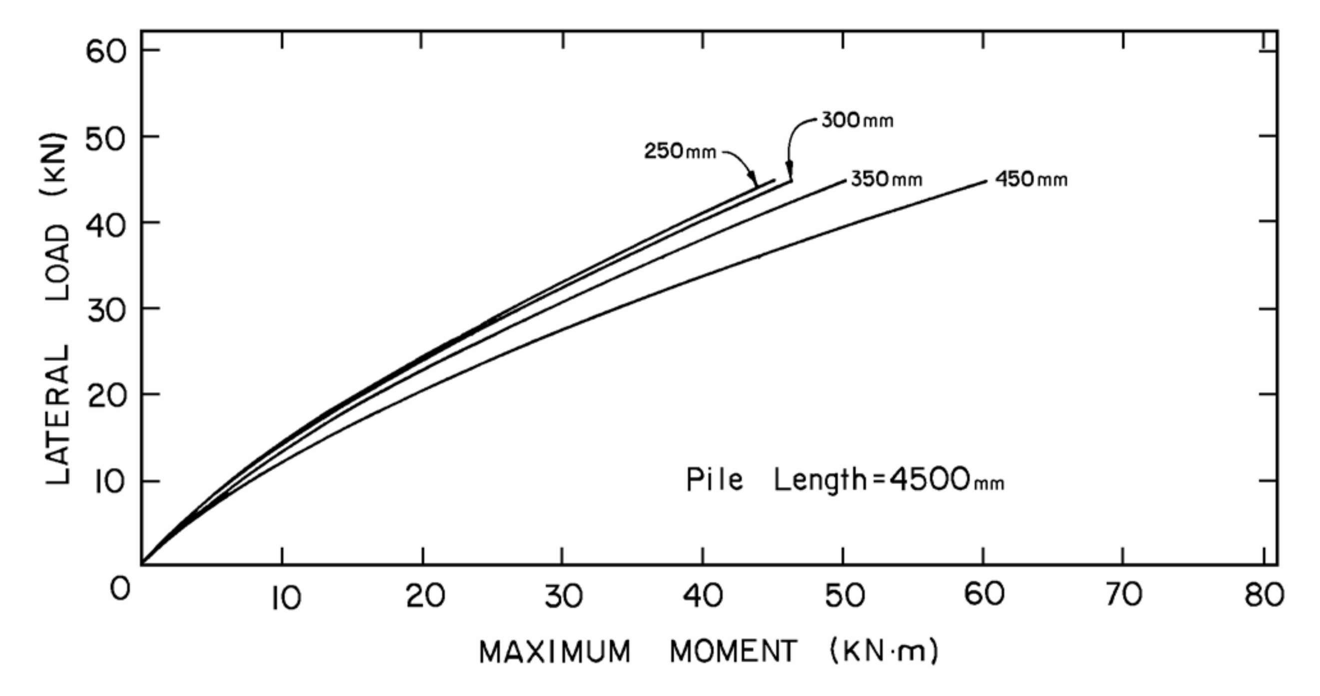
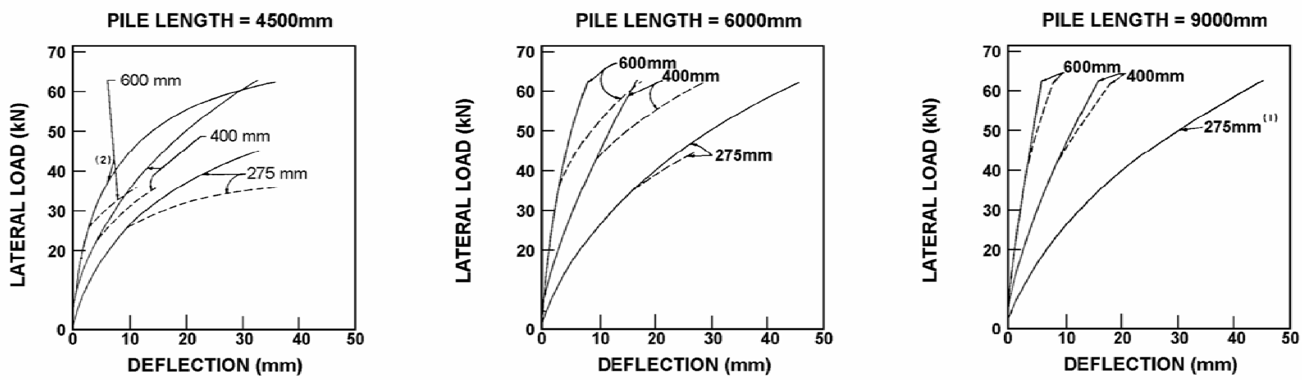


Figure 1.1.8-21 - Load Versus Maximum Moment for Cast-in-Place Pile, Soil Profile 7



SOIL CONDITION: Soil Profile No. 1 – Full depth saturated loose sand.
Soil Profile No. 2 – Full depth saturated soft clay.

LEGEND: Sand ——
Clay - - - - -

AXIAL STRESS: 83 MPa

REMARKS: (1) Where results were similar for sand and clay, solid curve represents both cases.

(2) See **Section 1.1.7** for discussion of the behavior of short stiff piles.

Figure 1.1.8-22 - Load Versus Deflection for Pipe Pile, Soil Profiles 1 and 3

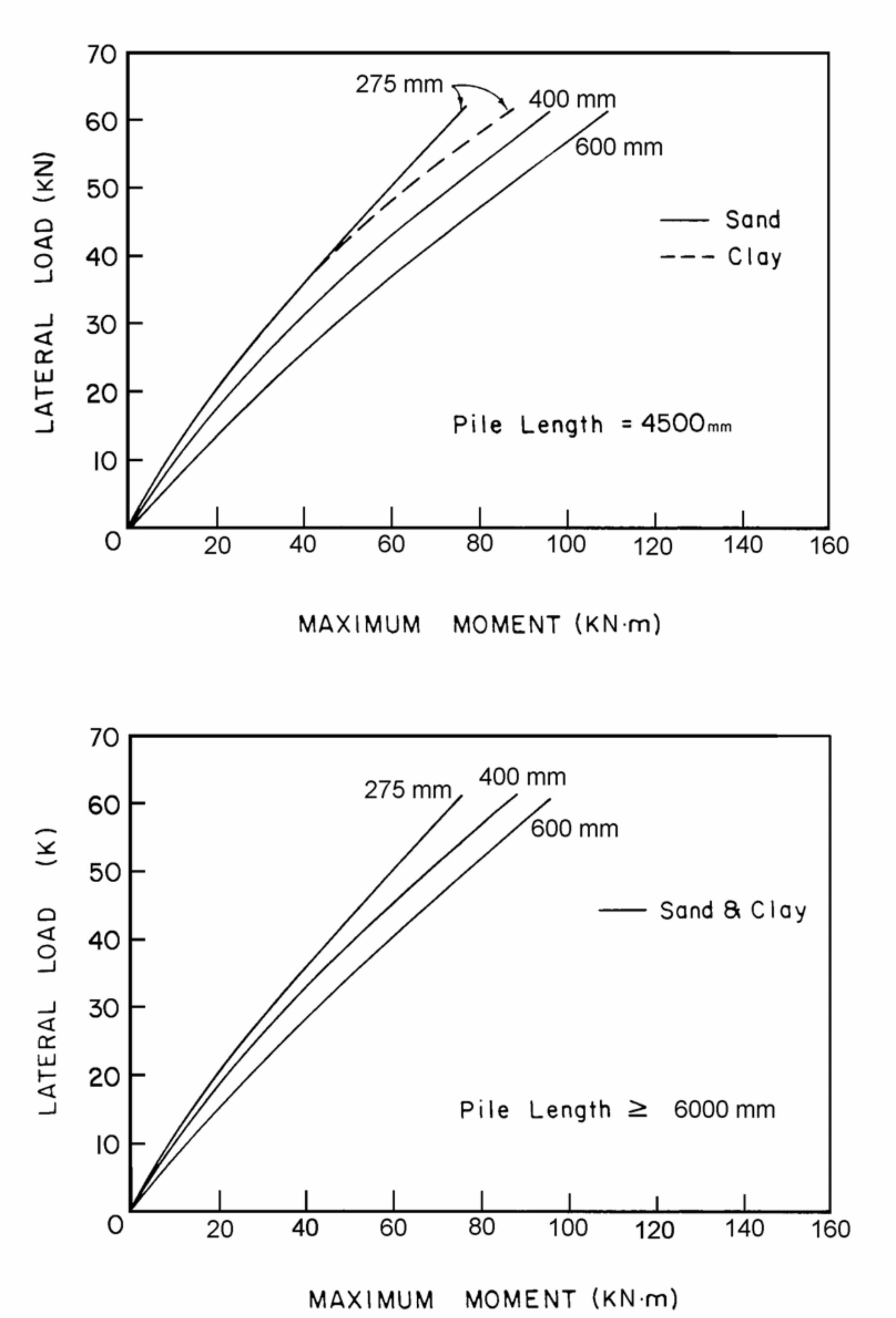


Figure 1.1.8-23 - Load Versus Maximum Moment for Pipe Pile, Soil Profiles 1 and 3

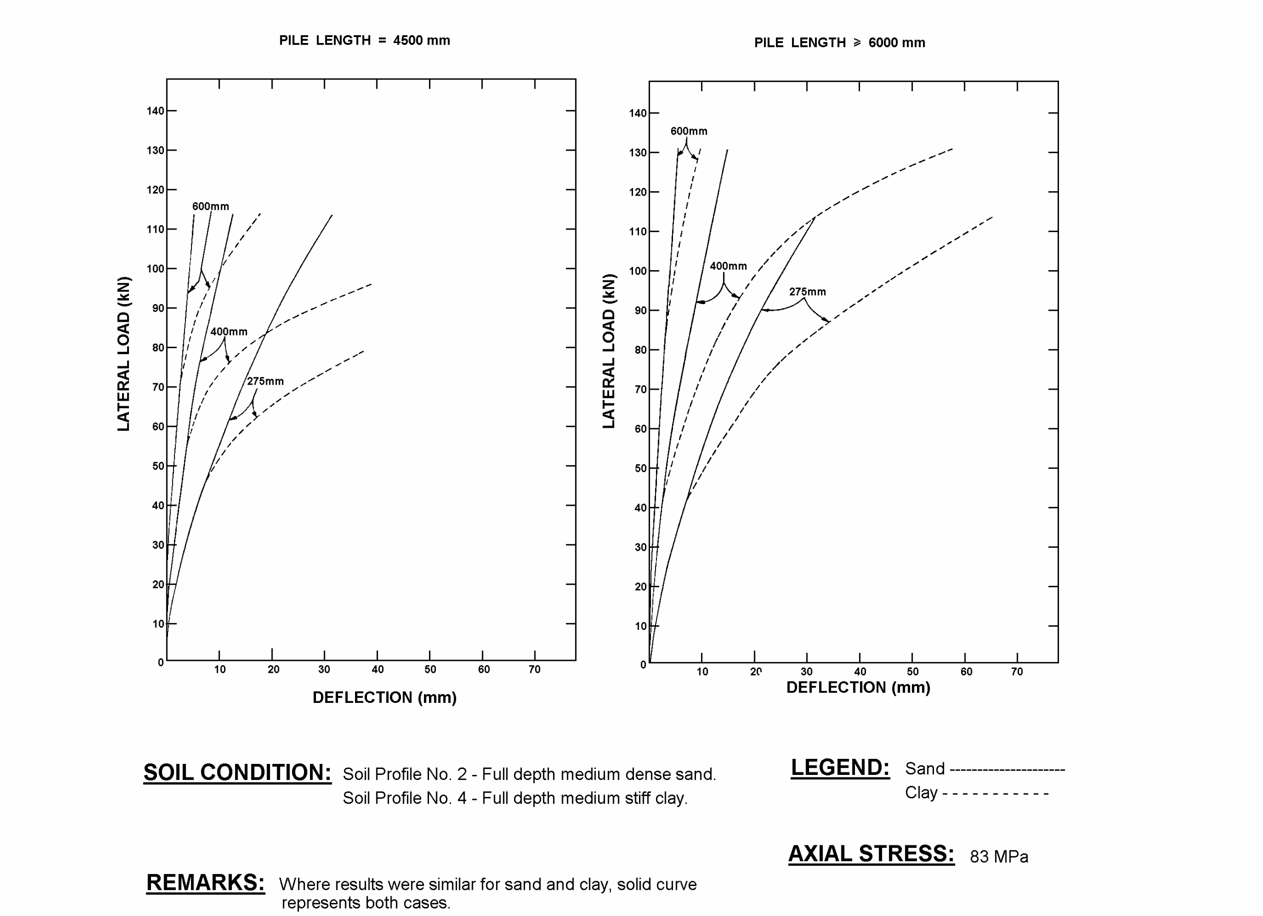
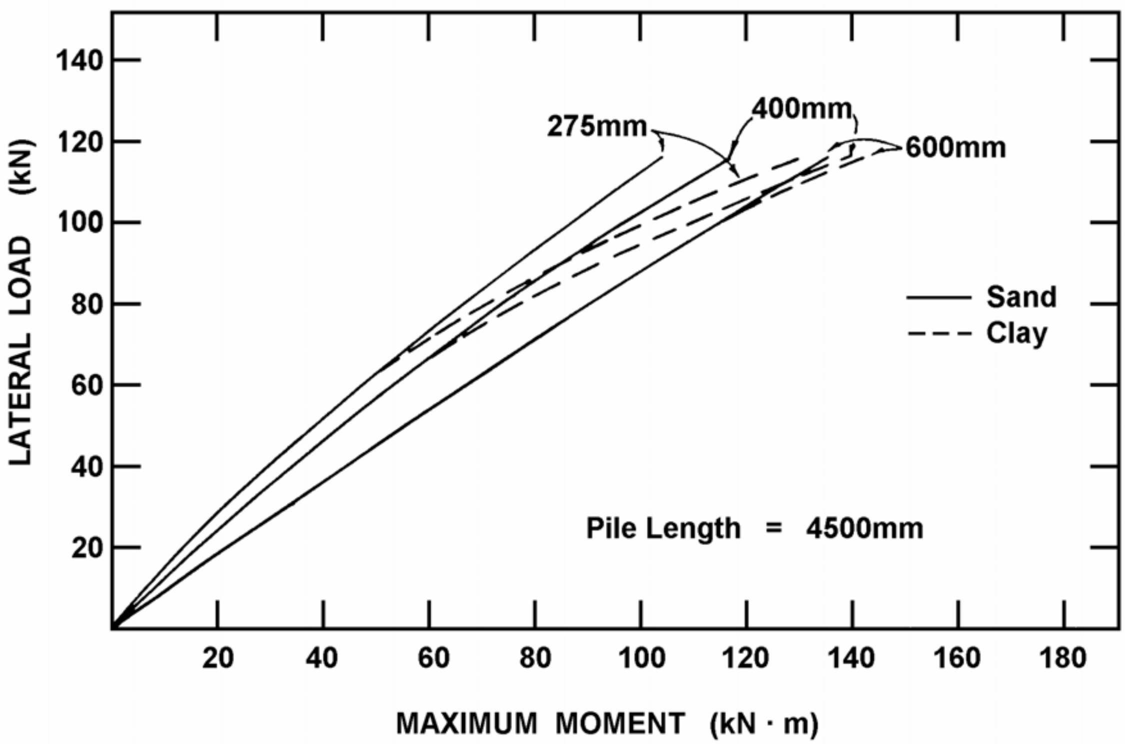


Figure 1.1.8-24 - Load Versus Deflection for Pipe Pile, Soil Profiles 2 and 4



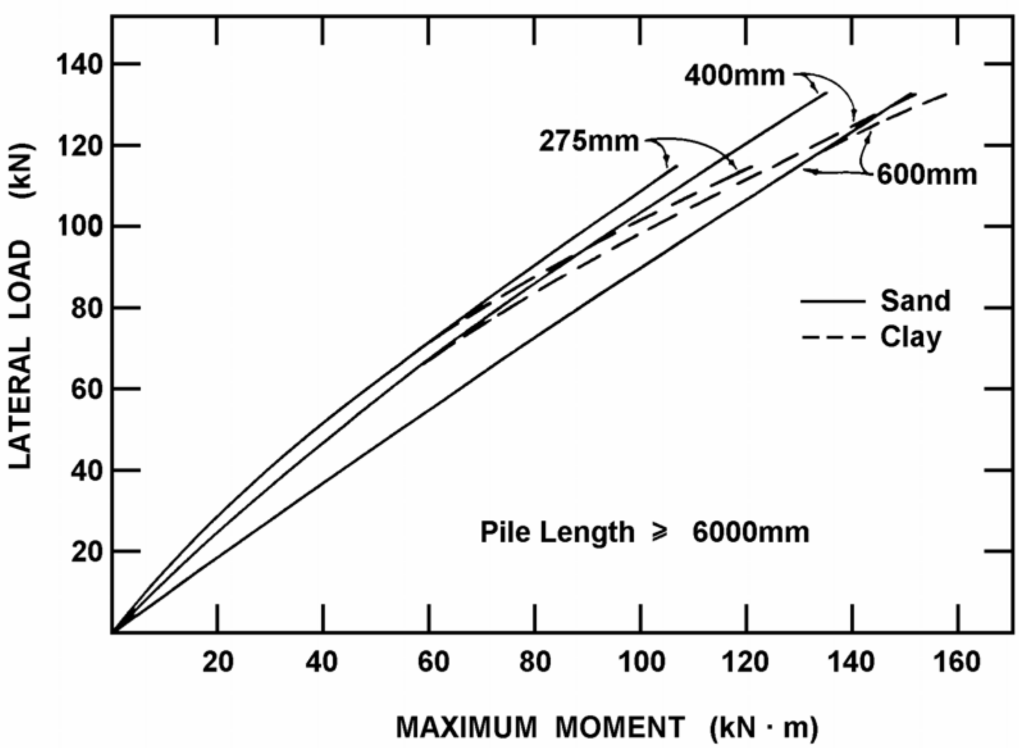


Figure 1.1.8-25 - Load Versus Maximum Moment for Pipe Pile, Soil Profiles 2 and 4

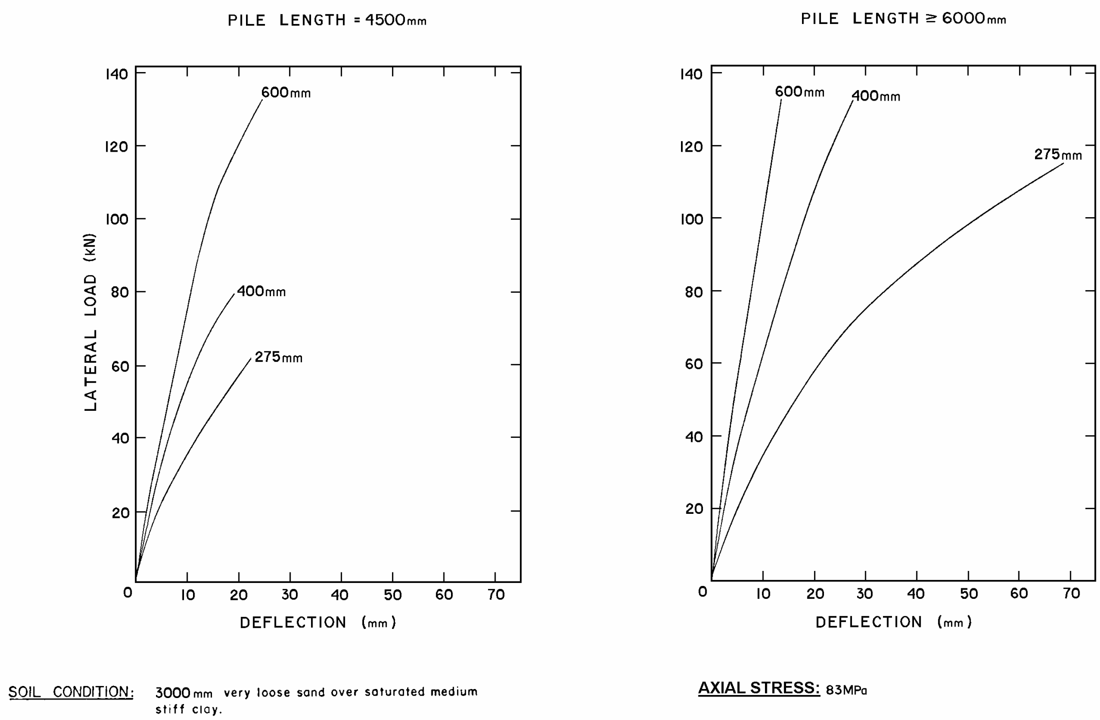


Figure 1.1.8-26 - Load Versus Deflection for Pipe Pile, Soil Profile 5

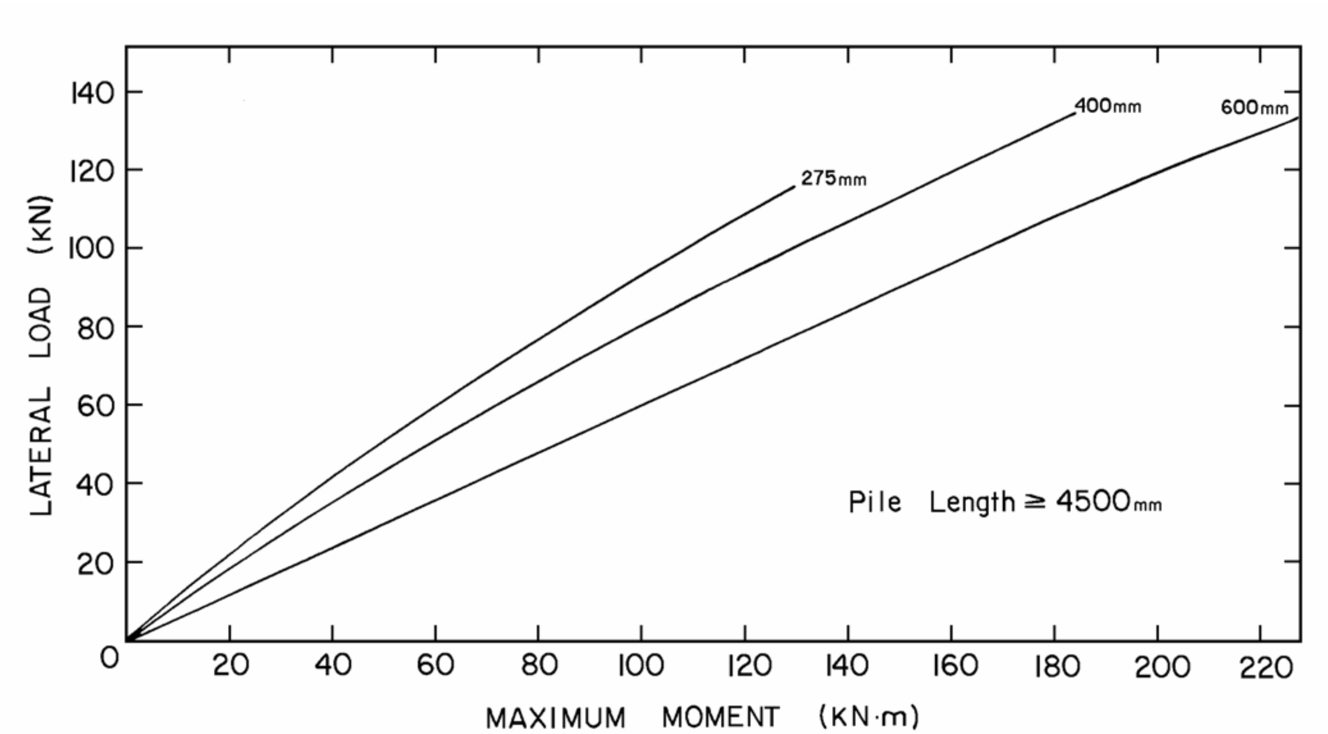


Figure 1.1.8-27 - Load Versus Maximum Moment for Pipe Pile, Soil Profile 5

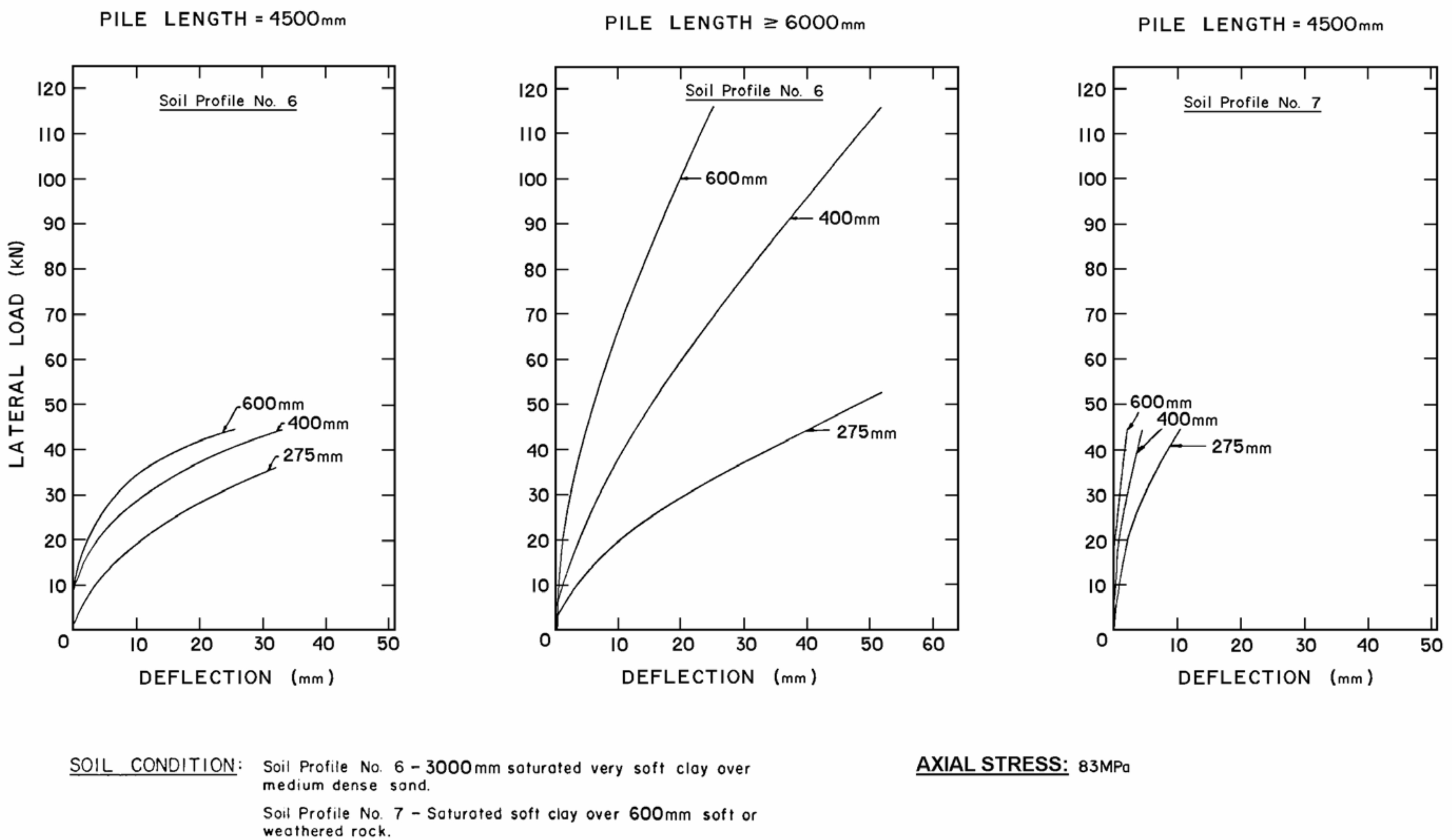
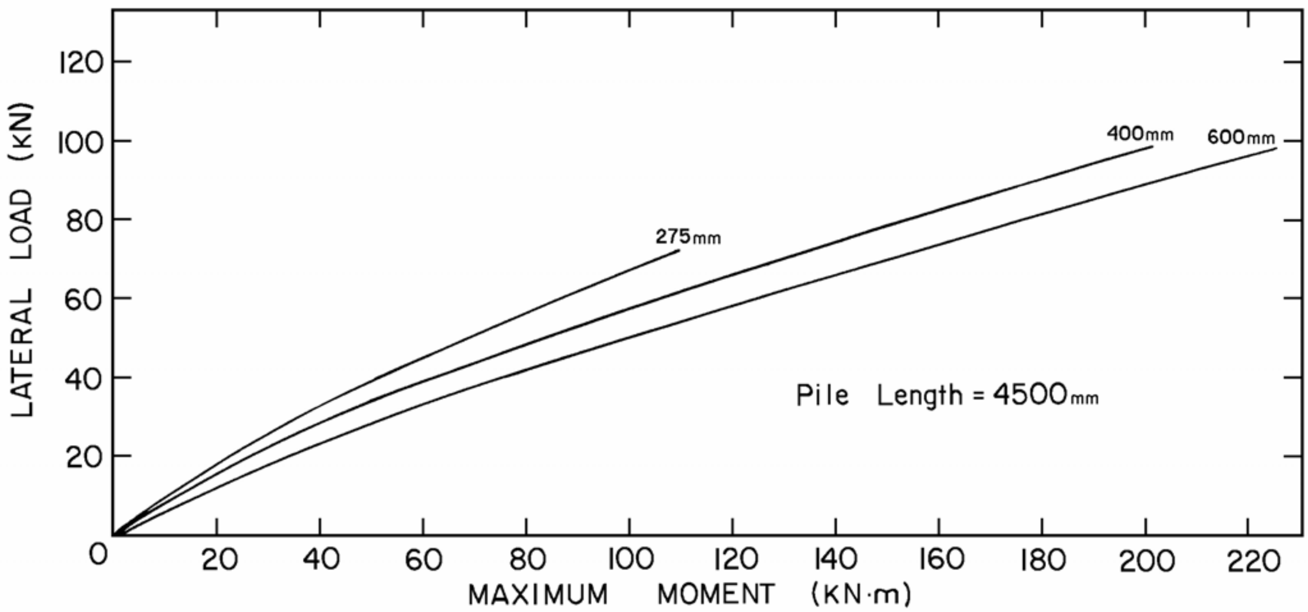


Figure 1.1.8-28 - Load Versus Deflection for Pipe Pile, Soil Profiles 6 and 7



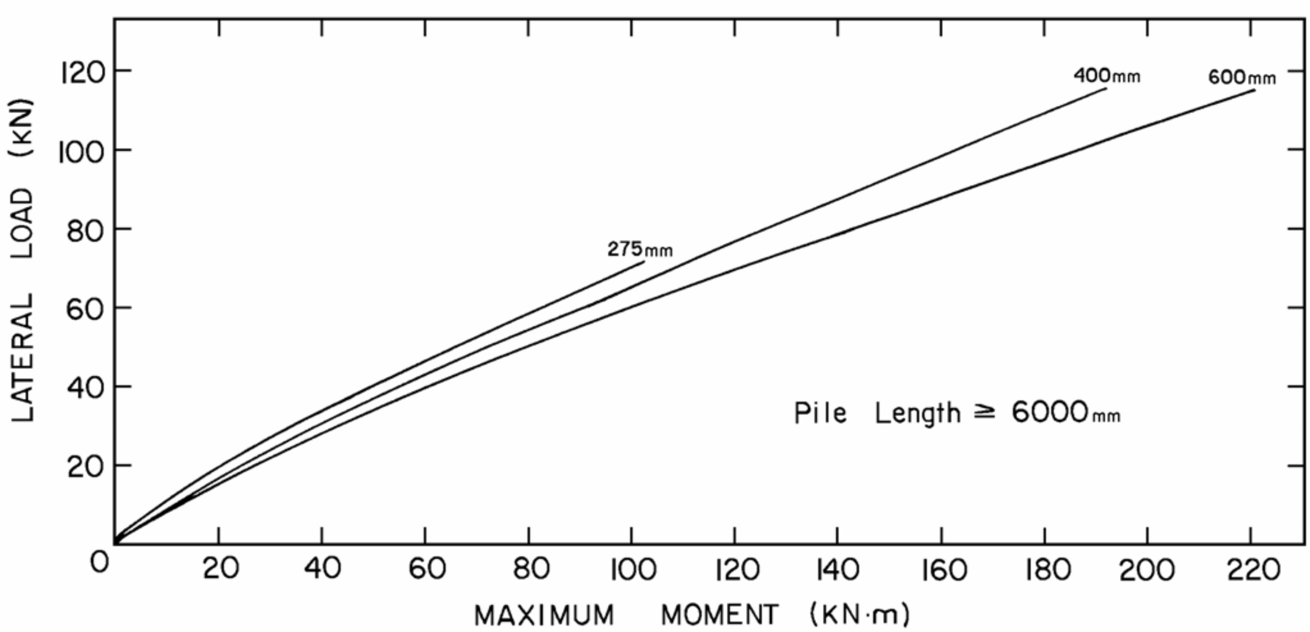


Figure 1.1.8-29 - Load Versus Maximum Moment for Pipe Pile, Soil Profile 6

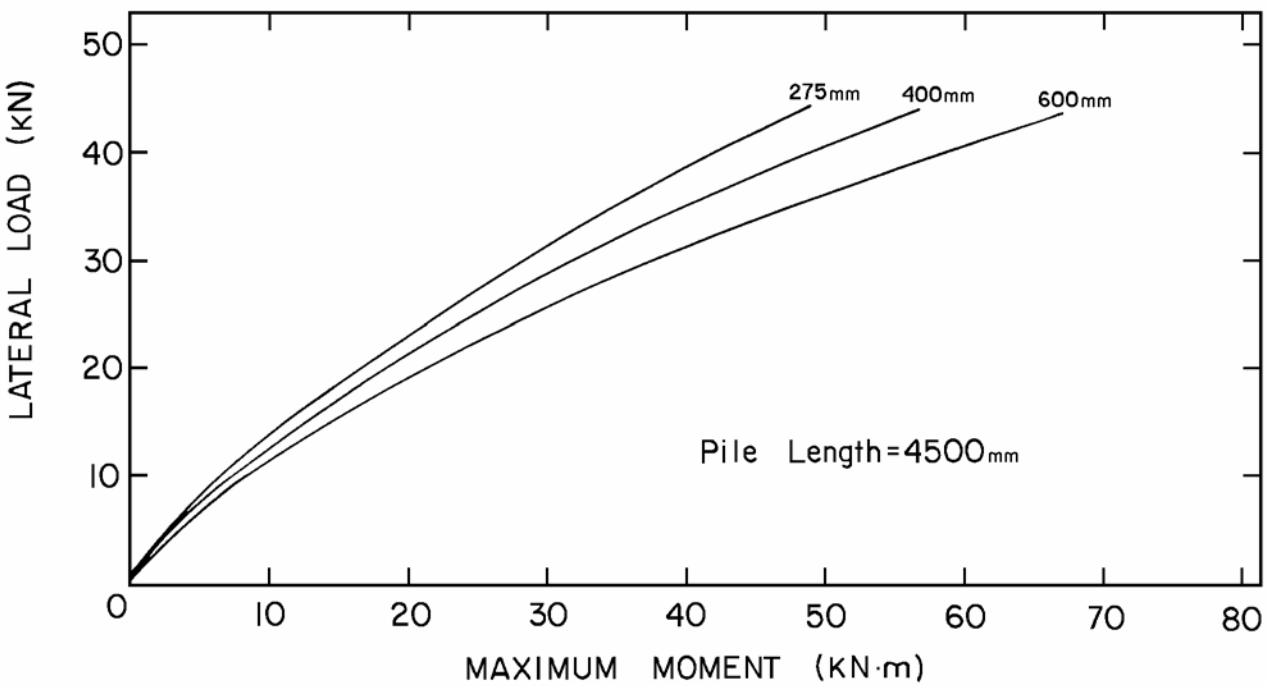
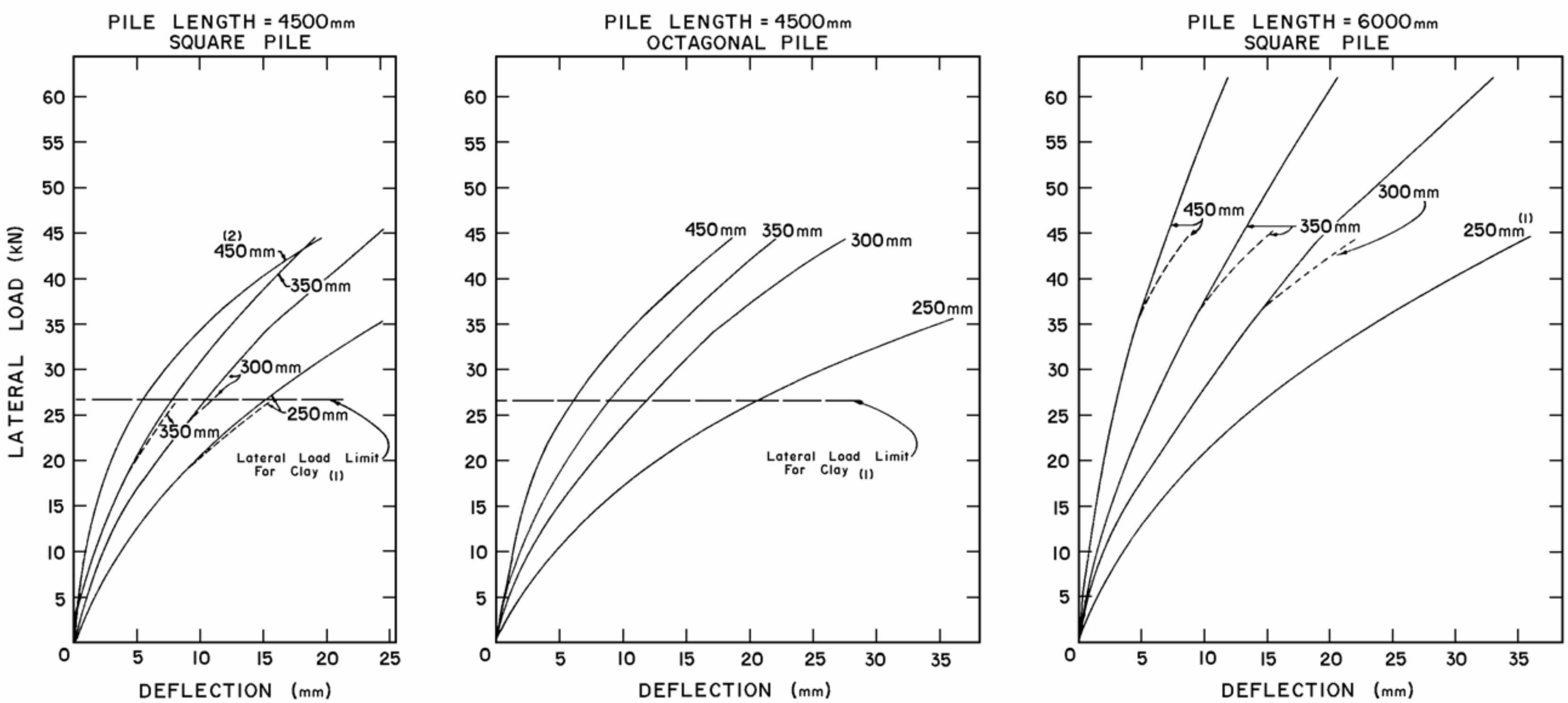


Figure 1.1.8-30 - Load Versus Maximum Moment for Pipe Pile, Soil Profile 7



SOIL CONDITION: Soil Profile No. 1 – Full depth saturated loose sand.
Soil Profile No. 3 – Full depth saturated soft clay.

LEGEND: Sand ——
Clay -----

REMARKS: (1) Where results were similar for sand and clay, solid curve represents both cases. Maximum lateral load for piles in clay is as noted.

(2) See **Section 1.1.7** for discussion of the behavior of short stiff piles.

AXIAL STRESS: 0.30 f'c

Figure 1.1.8-31 - Load Versus Deflection for Precast Concrete Pile, Soil Profiles 1 and 3

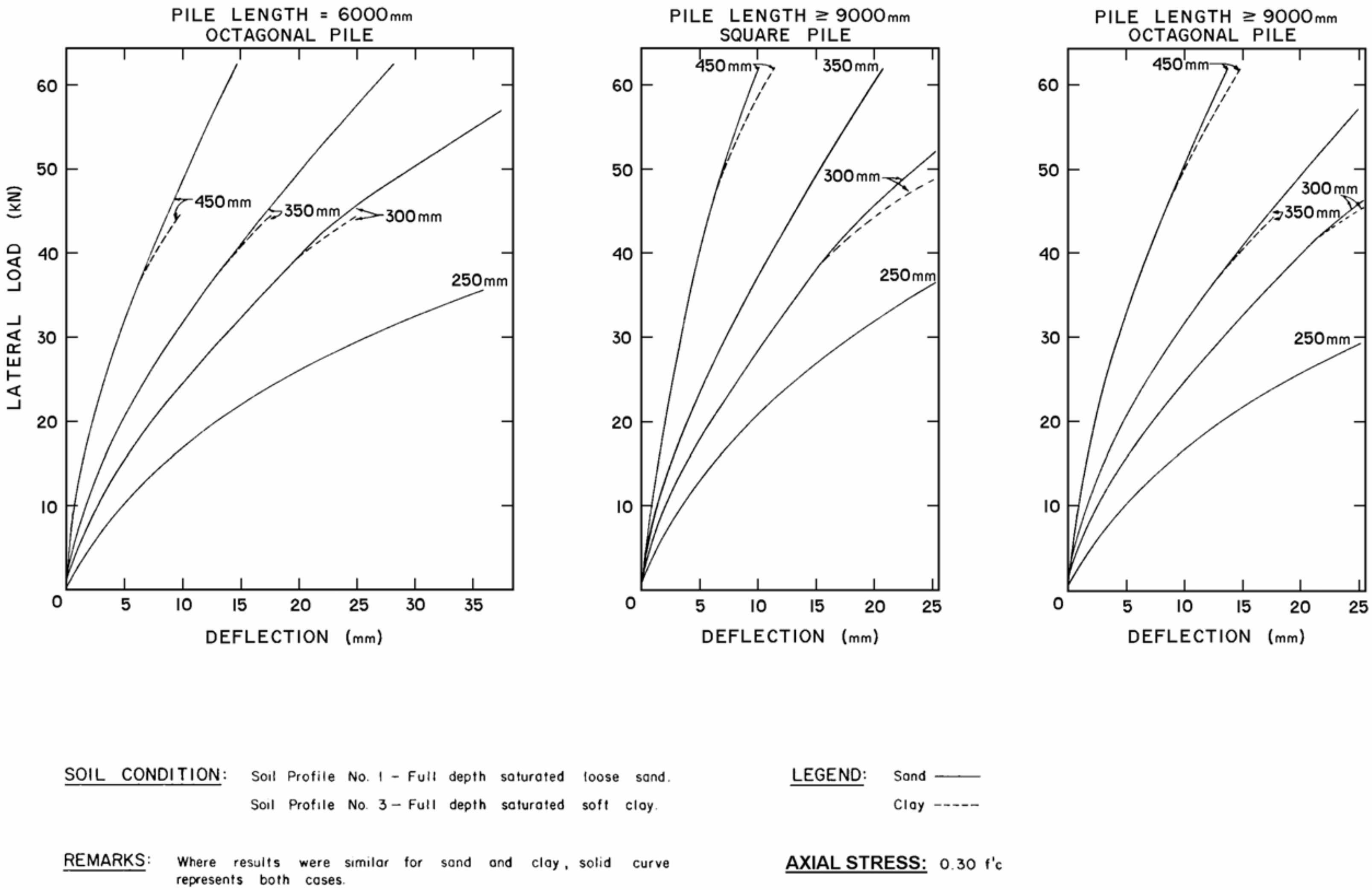
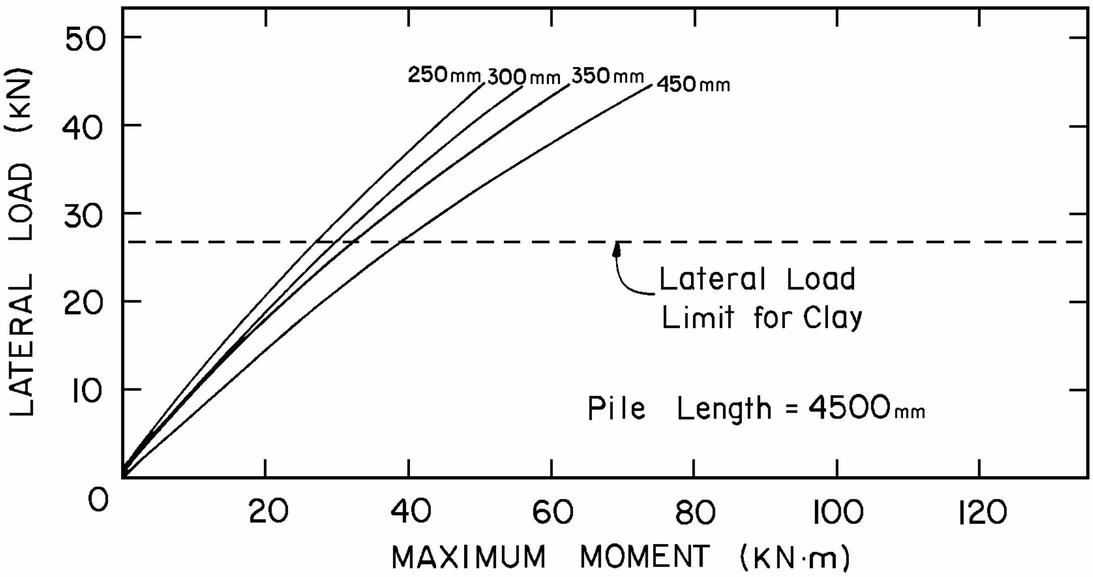
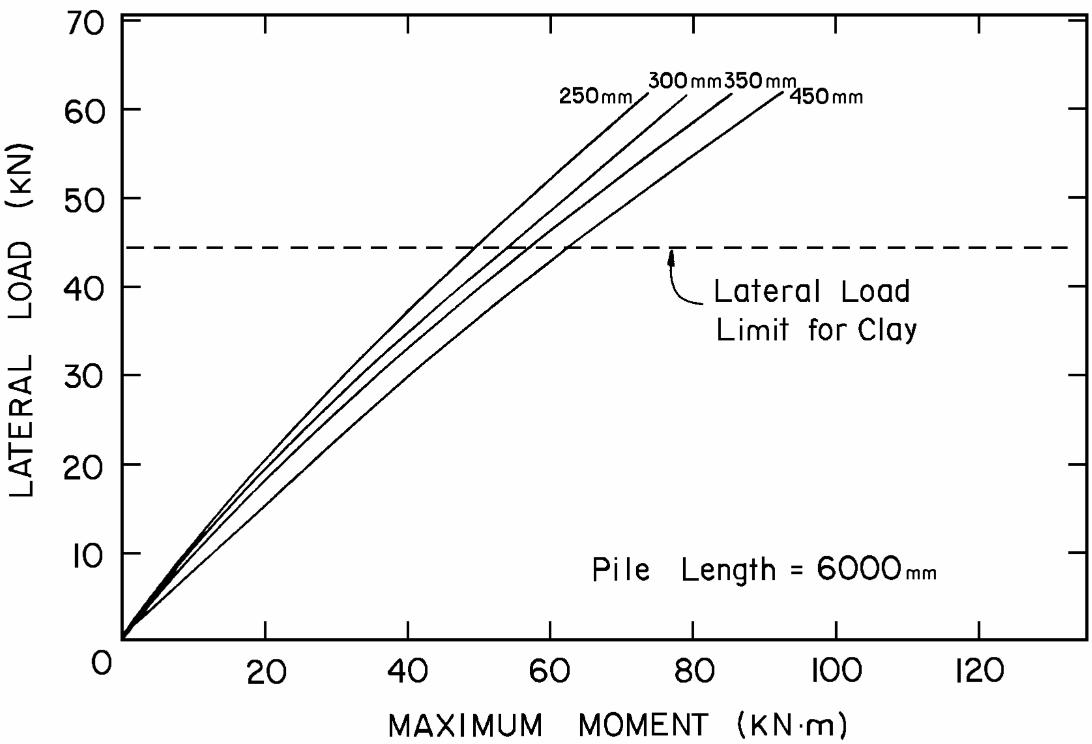


Figure 1.1.8-32 - Load Versus Deflection for Precast Concrete Pile, Soil Profiles 1 and 3





NOTE: Curves apply to both square and octagonal piles.

Figure 1.1.8-33 - Load Versus Maximum Moment for Precast Concrete Pile, Soil Profiles 1 and 3

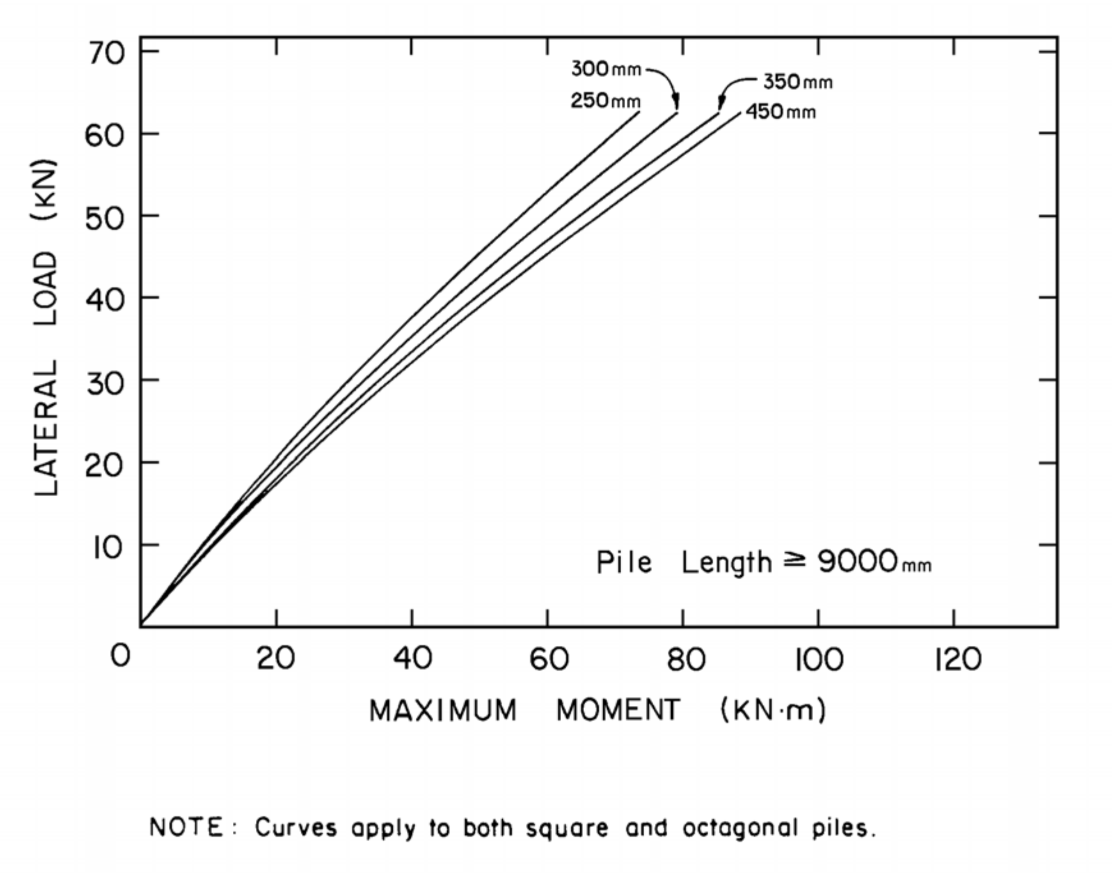


Figure 1.1.8-34 - Load Versus Maximum Moment for Precast Concrete Pile, Soil Profiles 1 and 3

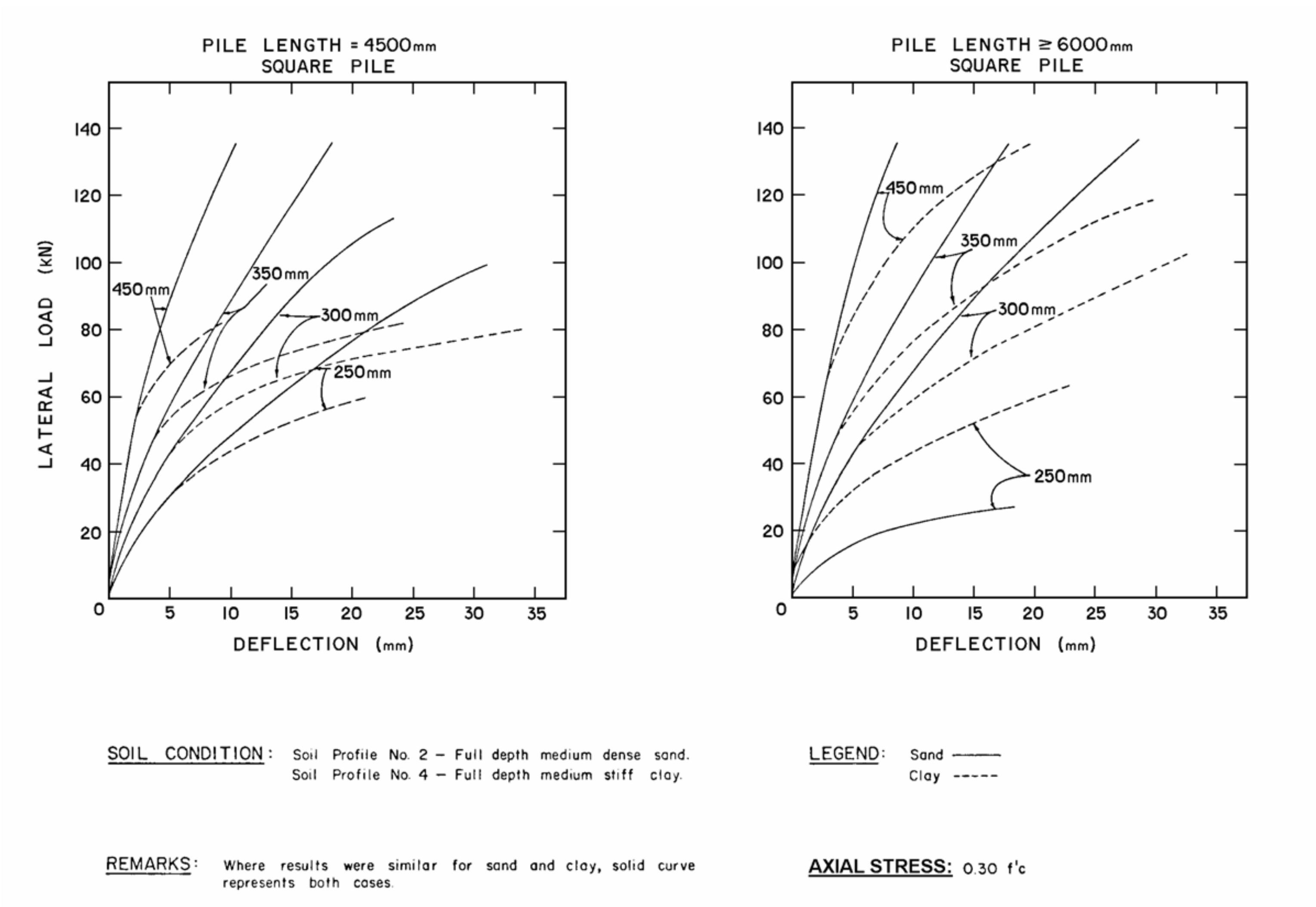


Figure 1.1.8-35 - Load Versus Deflection for Precast Concrete Pile, Soil Profiles 2 and 4

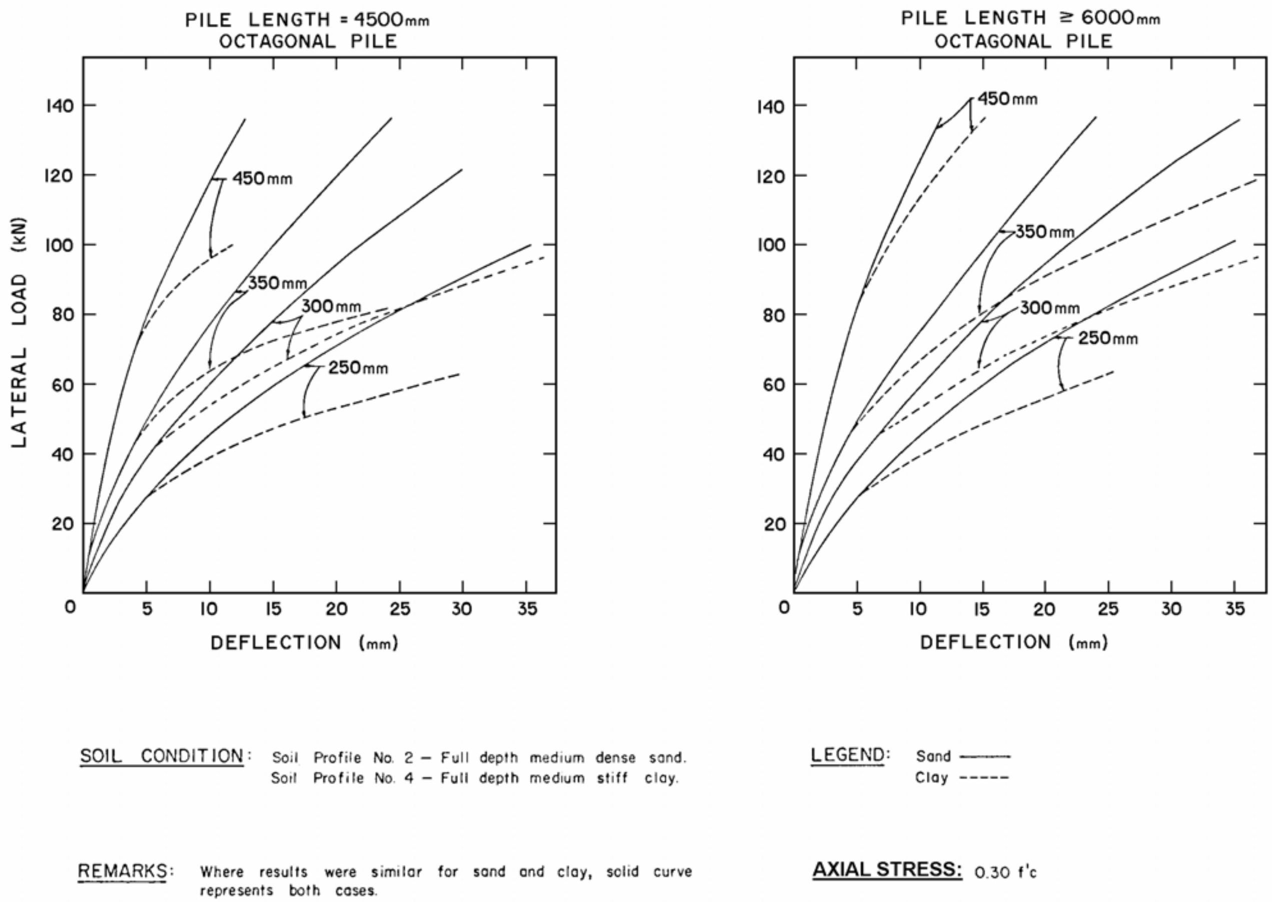


Figure 1.1.8-36 - Load Versus Deflection for Precast Concrete Pile, Soil Profiles 2 and 4

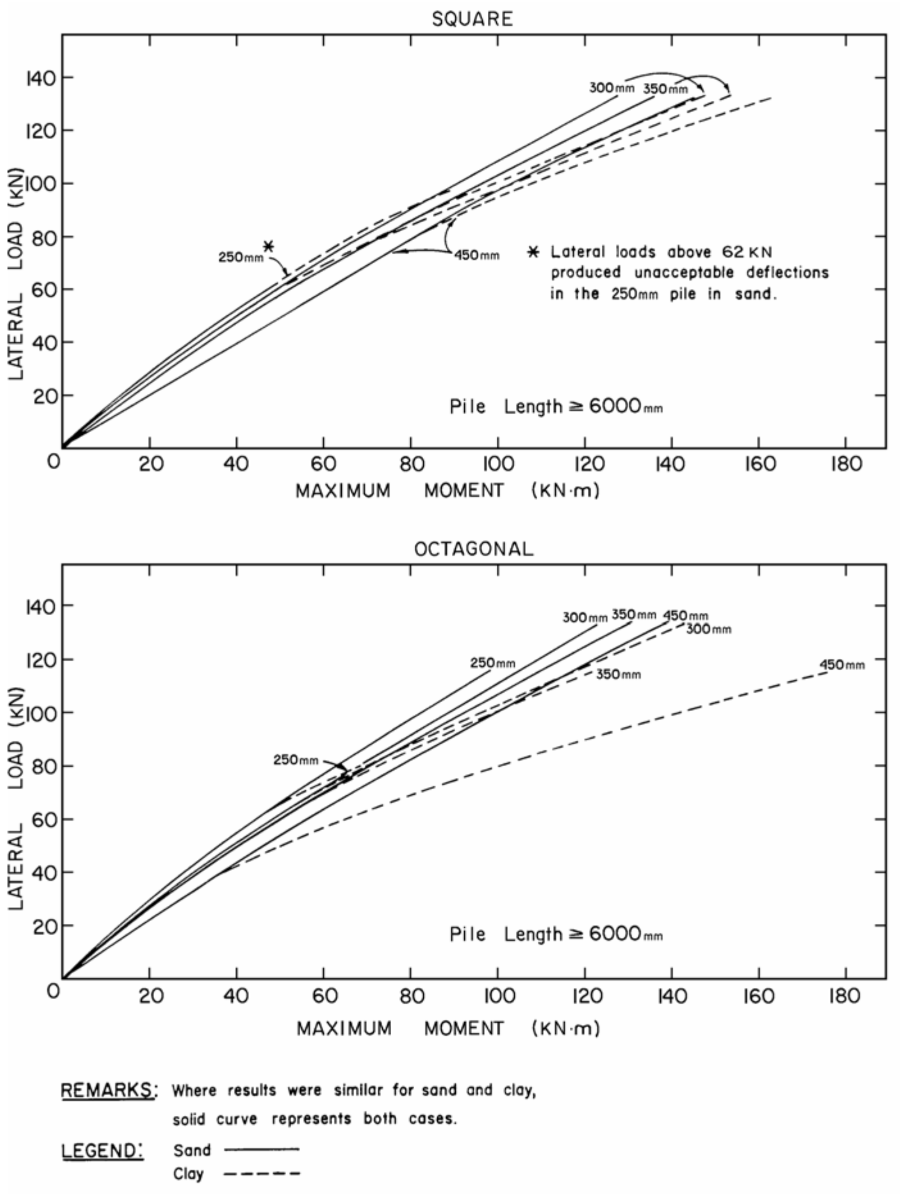


Figure 1.1.8-37 - Load Versus Maximum Moment for Precast Concrete Pile, Soil Profiles 2 and 4

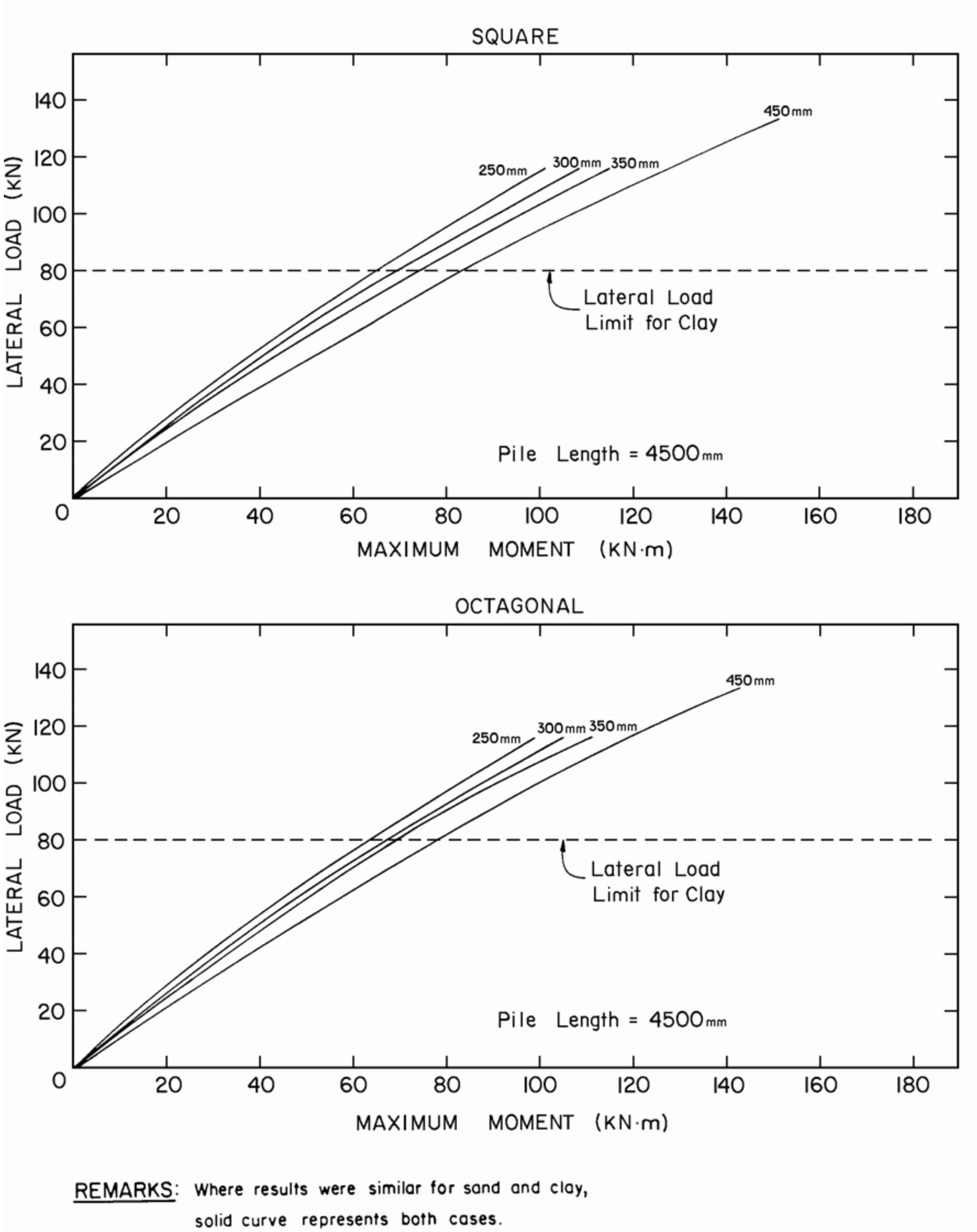


Figure 1.1.8-38 - Load Versus Maximum Moment for Precast Concrete Pile, Soil Profiles 2 and 4

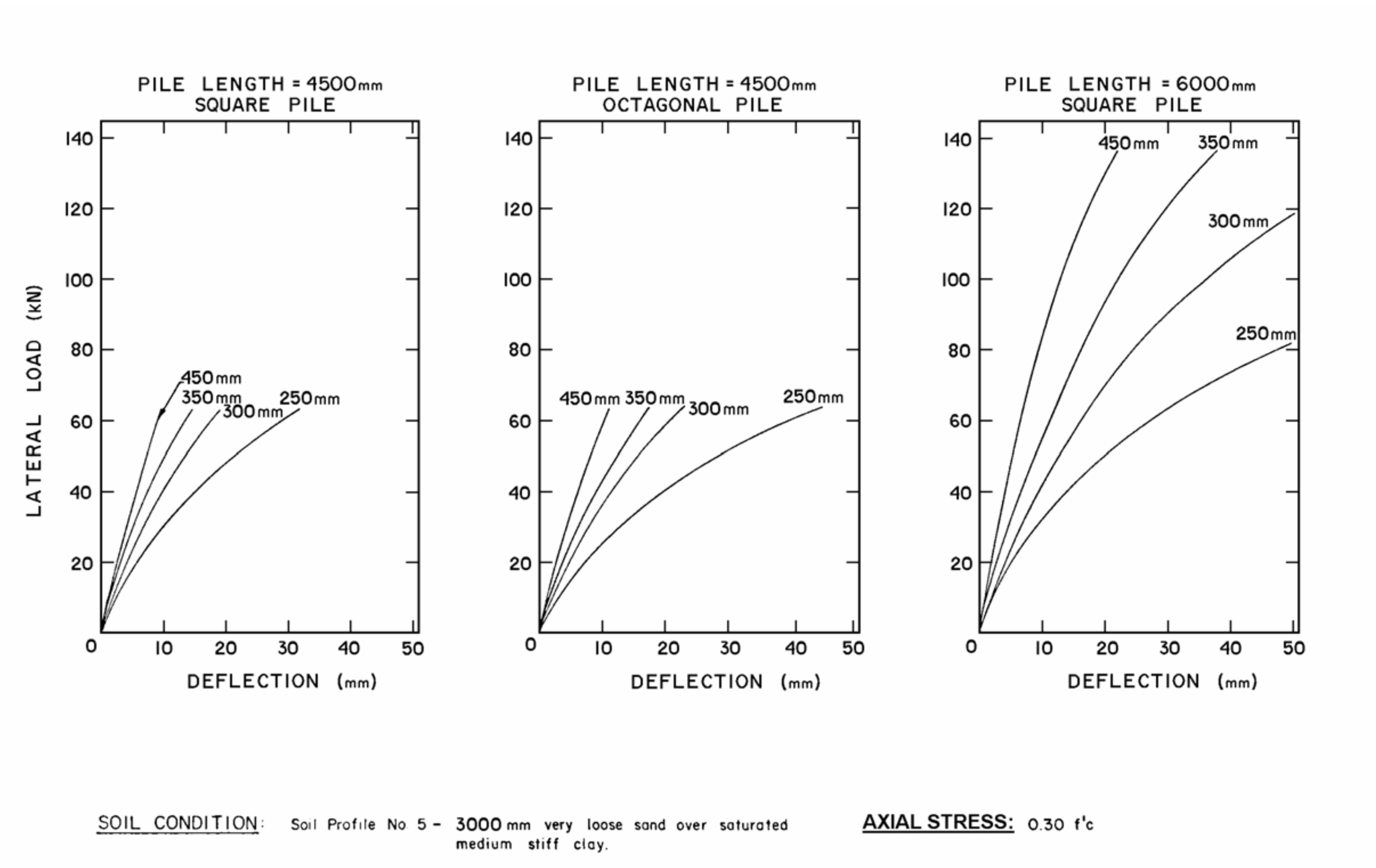


Figure 1.1.8-39 - Load Versus Deflection for Precast Concrete Pile, Soil Profile 5

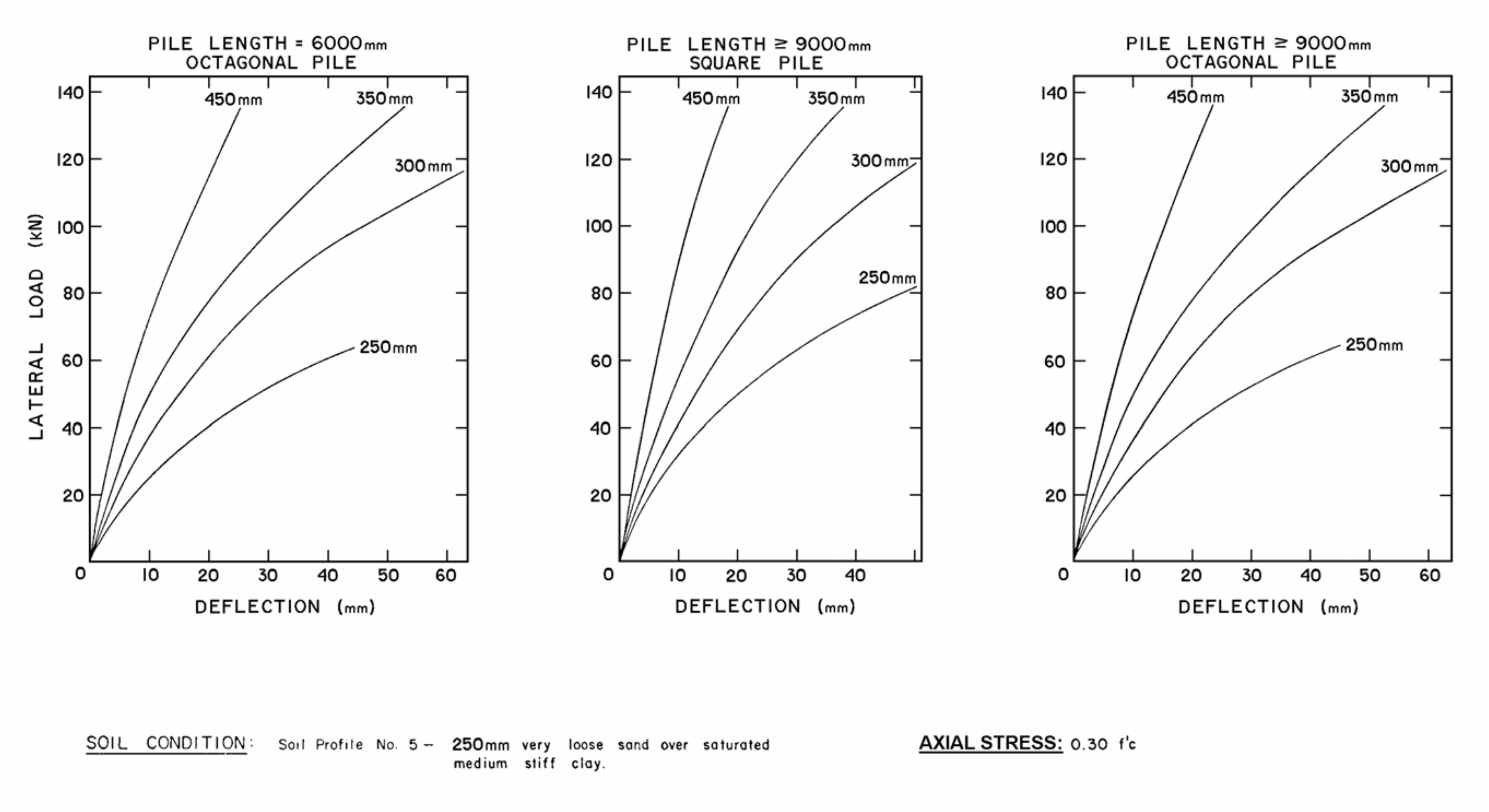


Figure 1.1.8-40 - Load Versus Deflection for Precast Concrete Pile, Soil Profile 5

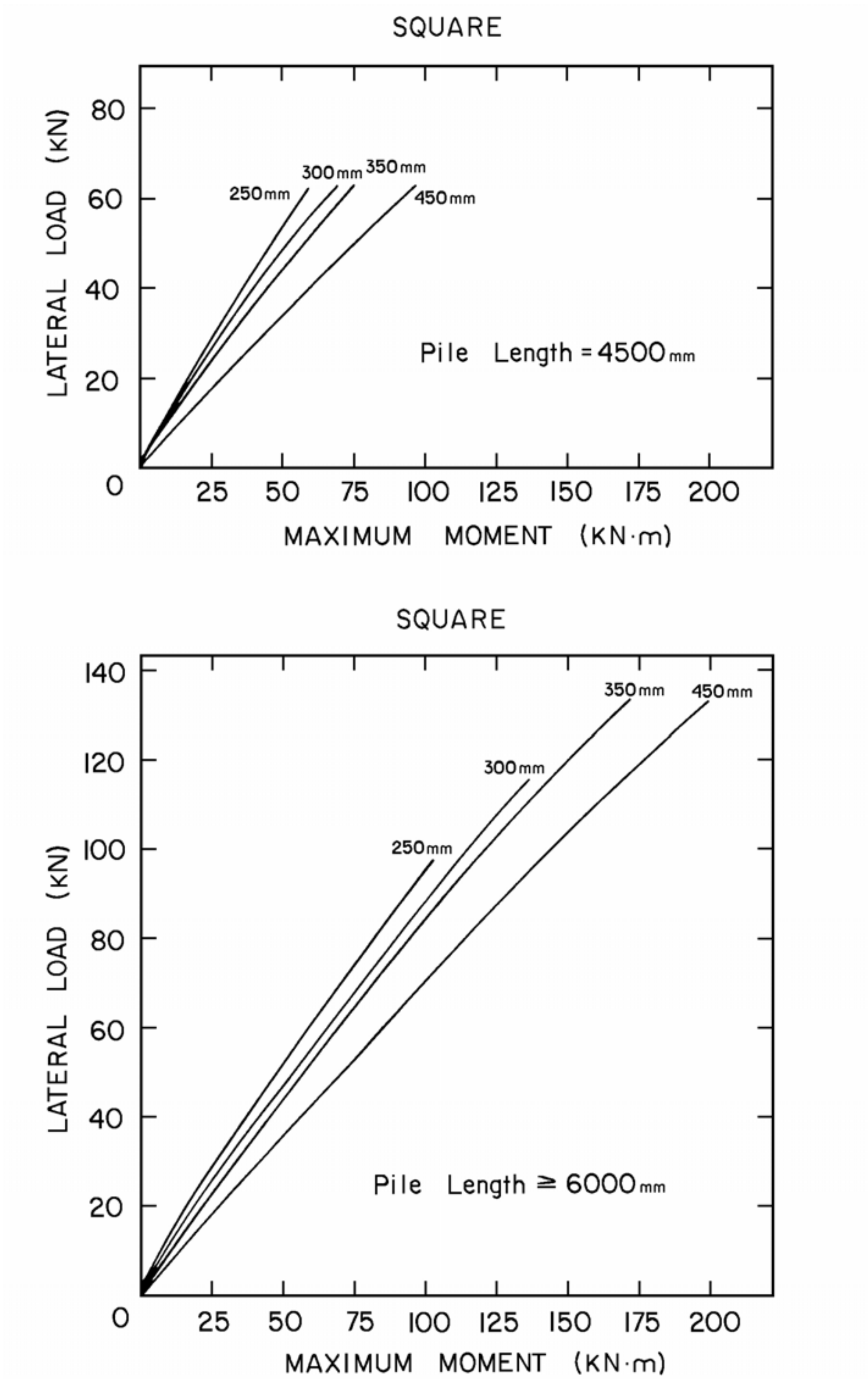


Figure 1.1.8-41 - Load Versus Maximum Moment for Precast Concrete Pile, Soil Profile 5

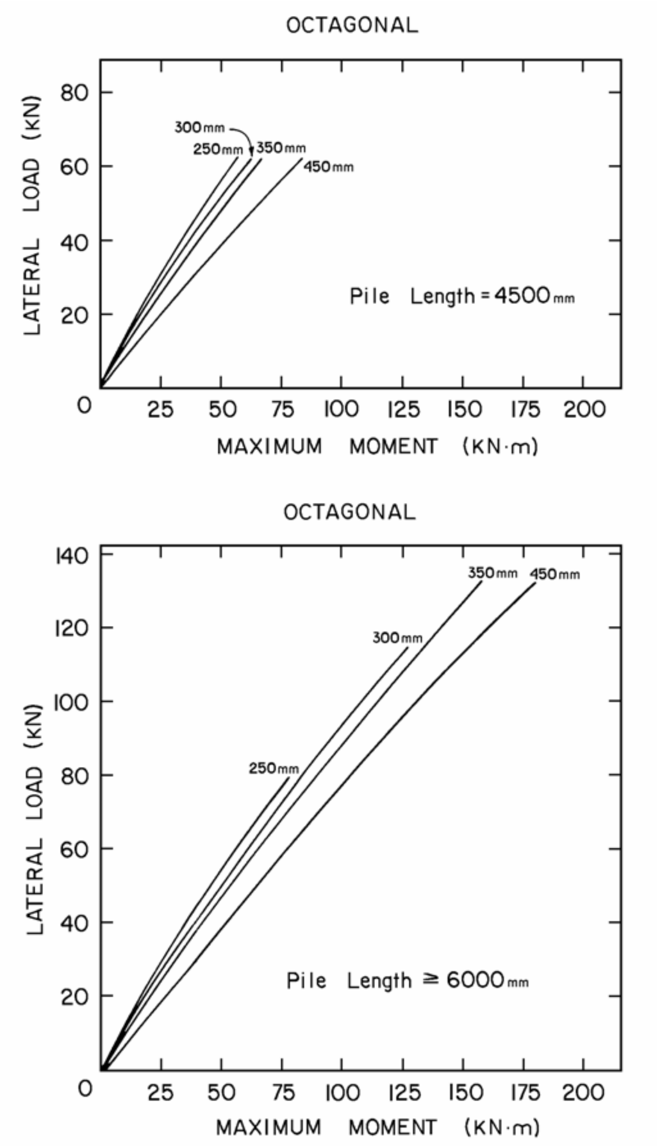
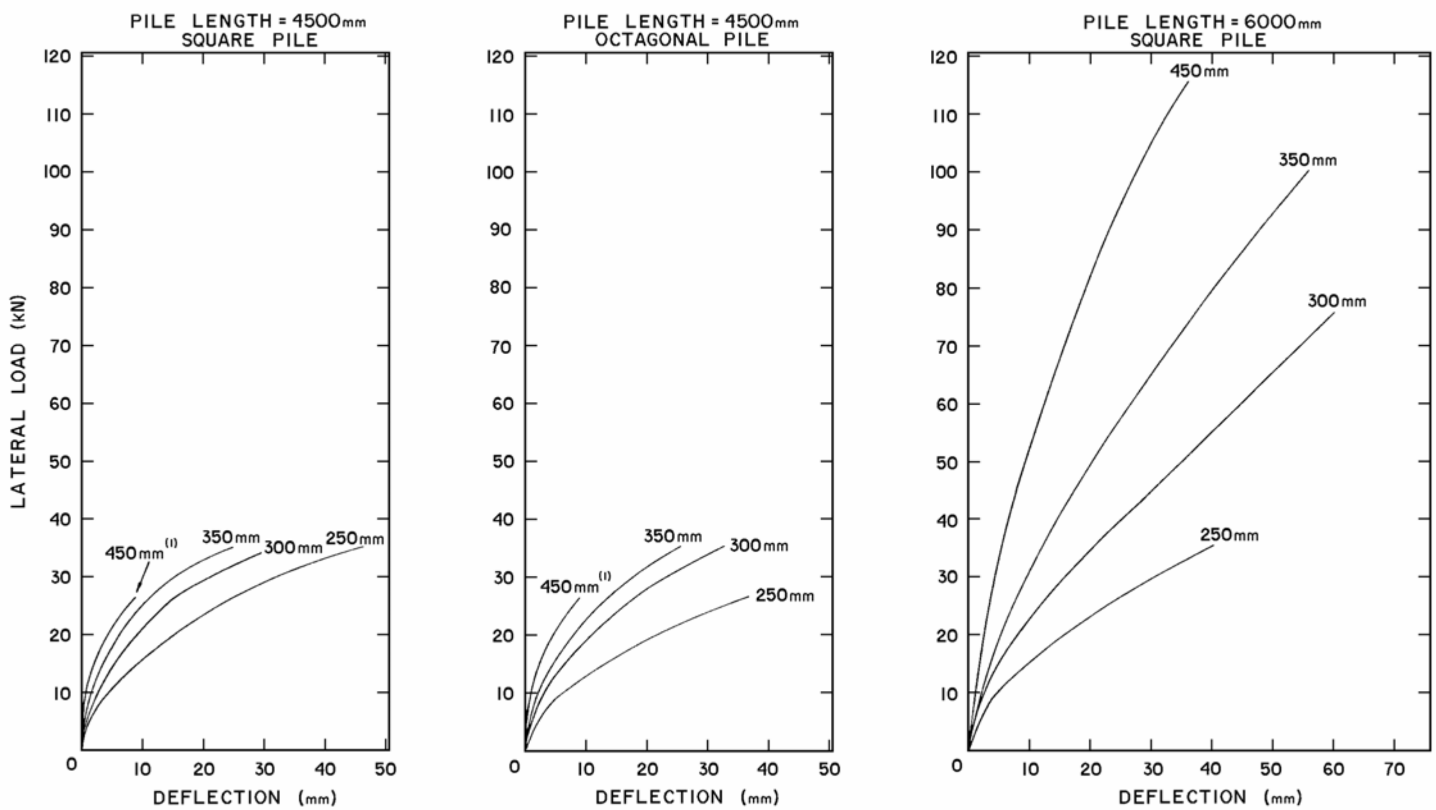


Figure 1.1.8-42 - Load Versus Maximum Moment for Precast Concrete Pile, Soil Profile 5



SOIL CONDITION: 3000 mm saturated very soft clay over medium dense sand.

AXIAL STRESS: 0.30 f'c

REMARKS: (1) see Section 1.1.7 for discussion of the behavior of short stiff piles.

Figure 1.1.8-43 - Load Versus Deflection for Precast Concrete Pile, Soil Profile 6

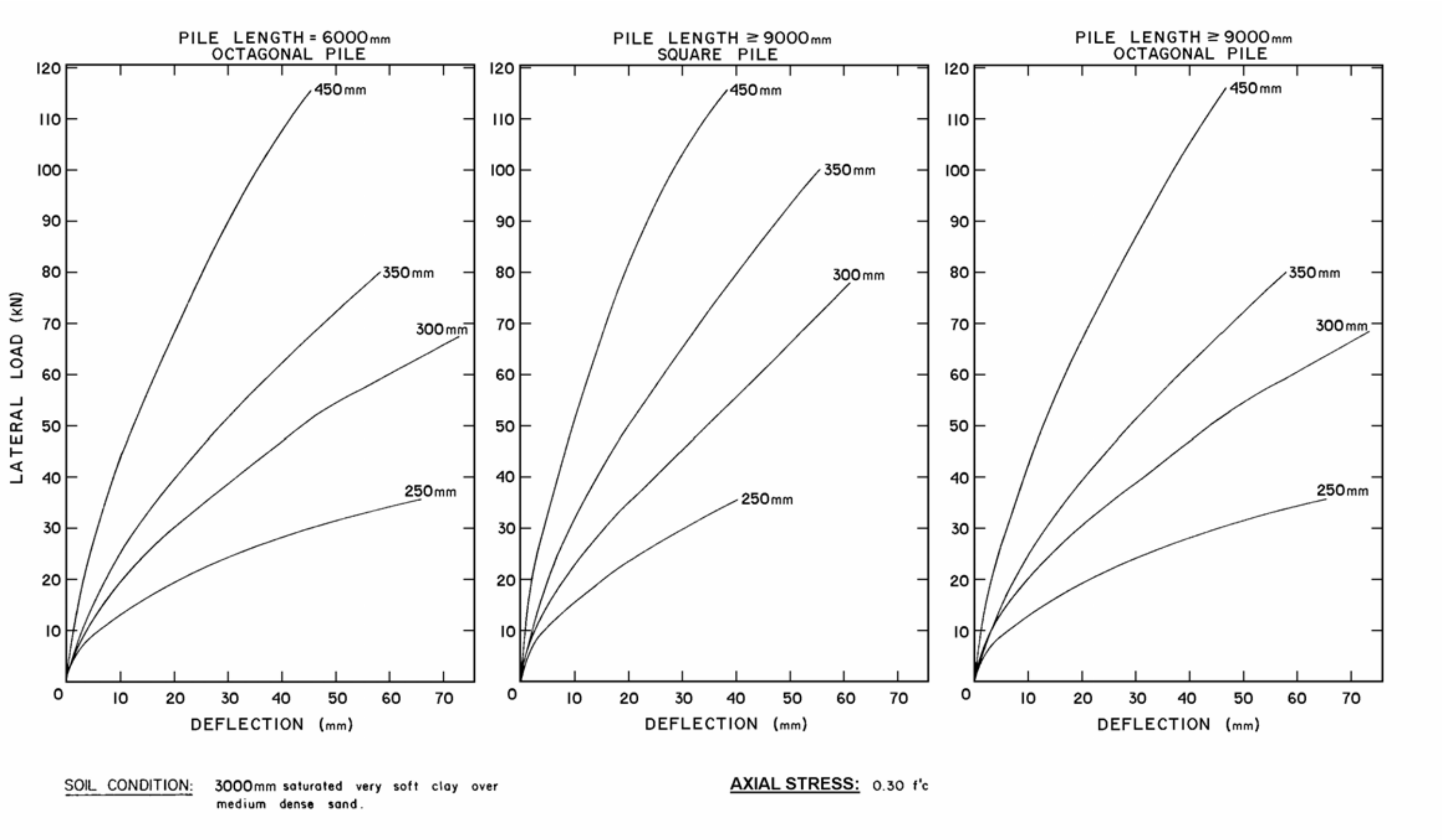


Figure 1.1.8-44 - Load Versus Deflection for Precast Concrete Pile, Soil Profile 6

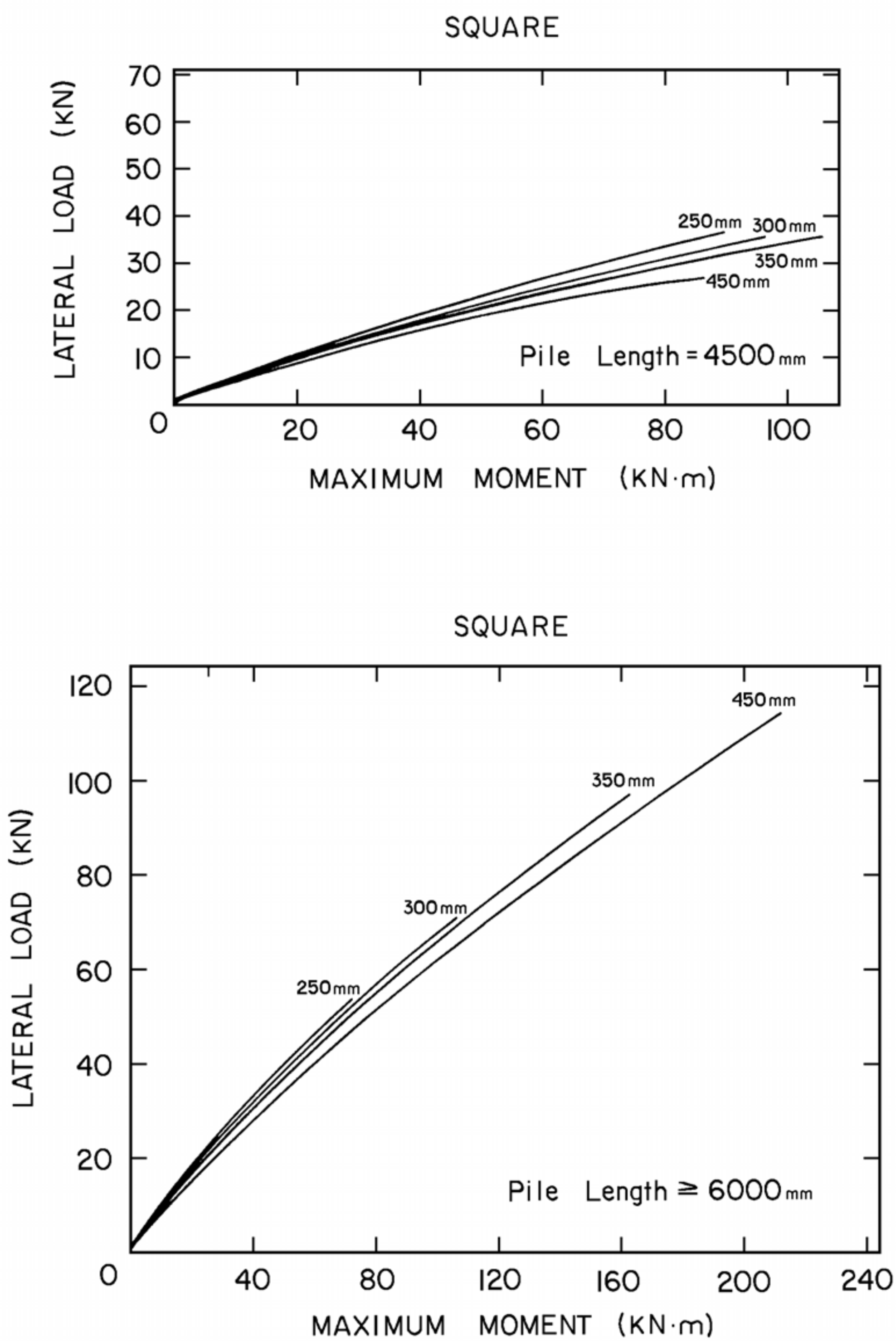


Figure 1.1.8-45 - Load Versus Maximum Moment for Precast Concrete Pile, Soil Profile 6

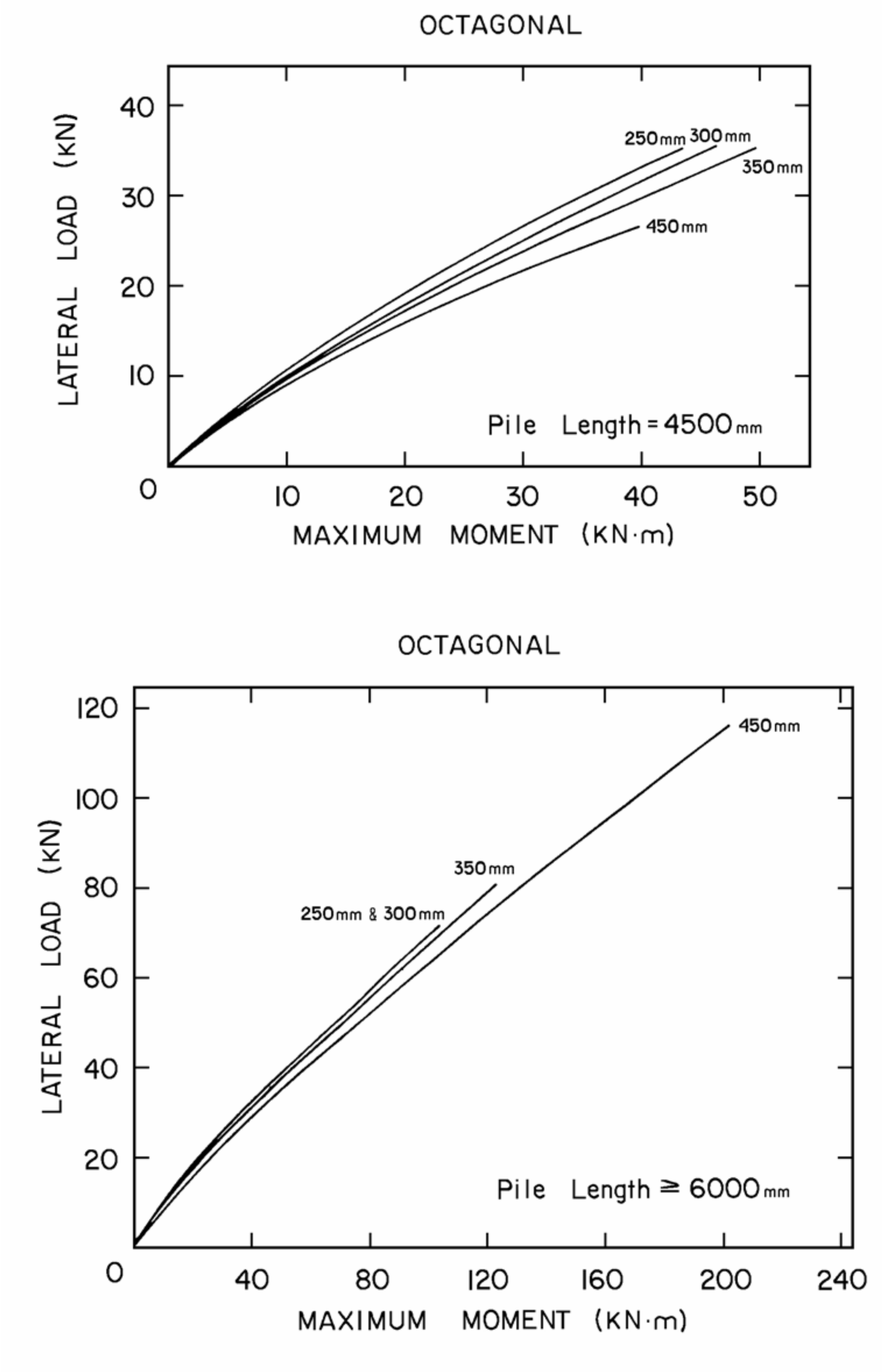
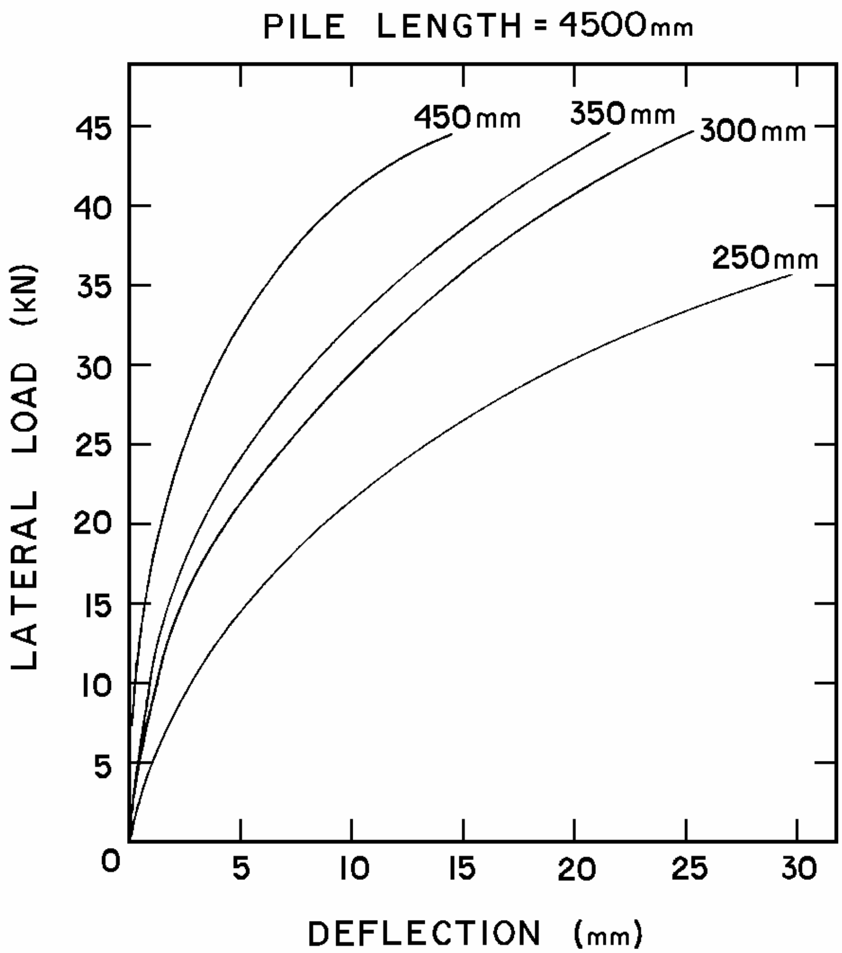


Figure 1.1.8-46 - Load Versus Maximum Moment for Precast Concrete Pile, Soil Profile 6



SOIL CONDITION: Soil Profile No. 7 – Saturated soft clay over 600mm soft or weathered rock.

REMARKS: Curves apply to both square and octagonal piles.

Figure 1.1.8-47 - Load Versus Deflection for Precast Concrete Pile, Soil Profile 7

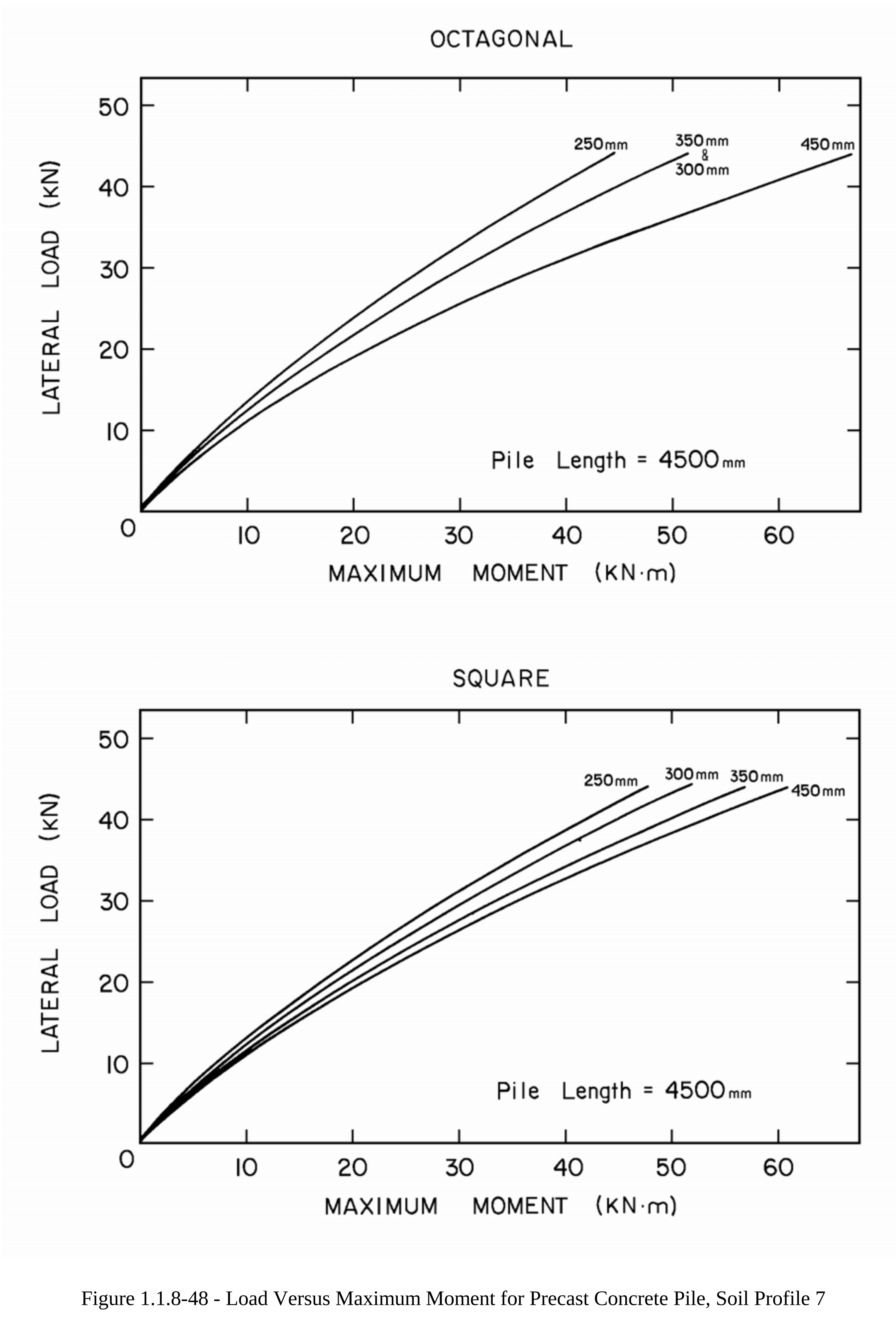


Figure 1.1.8-48 - Load Versus Maximum Moment for Precast Concrete Pile, Soil Profile 7

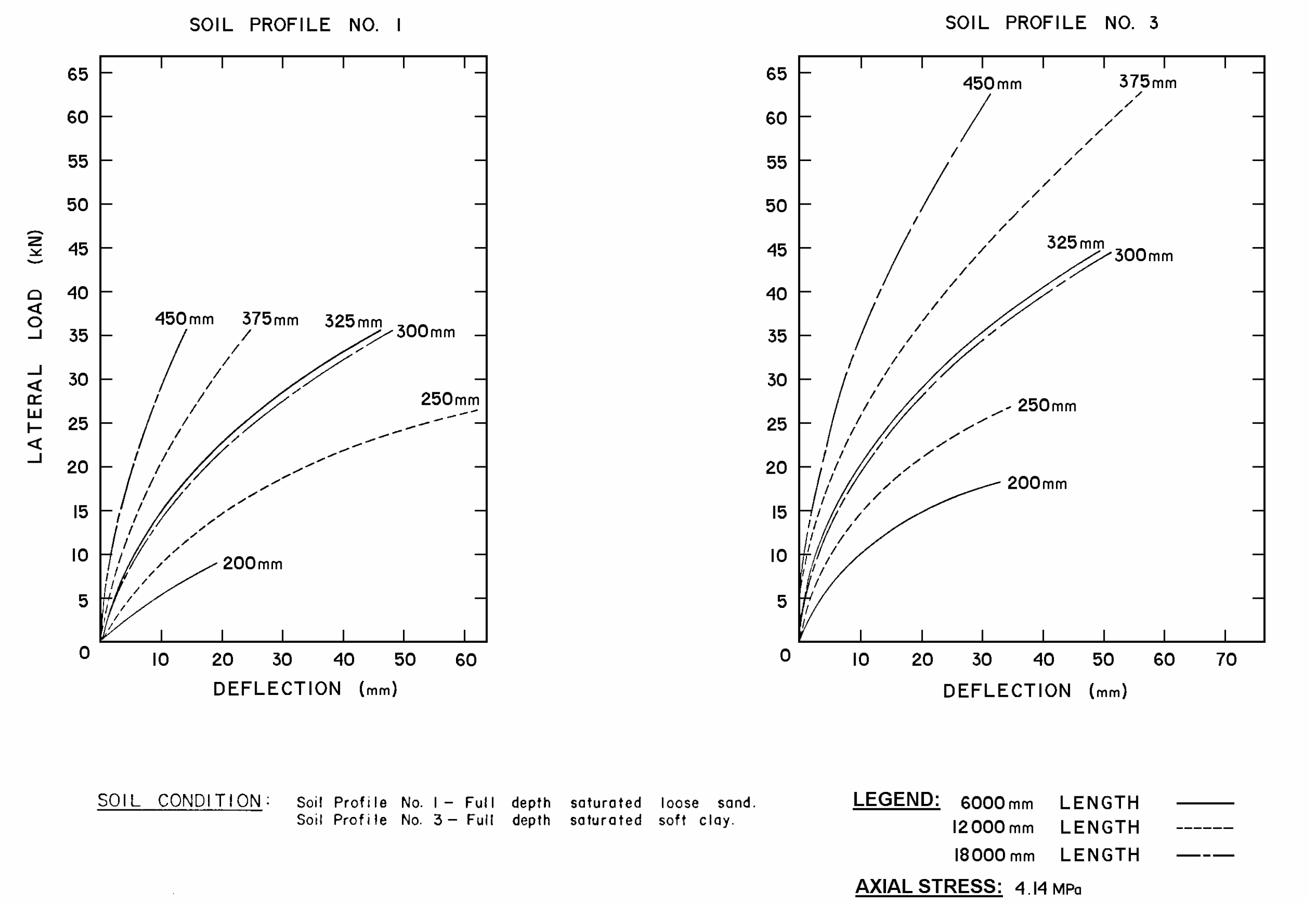
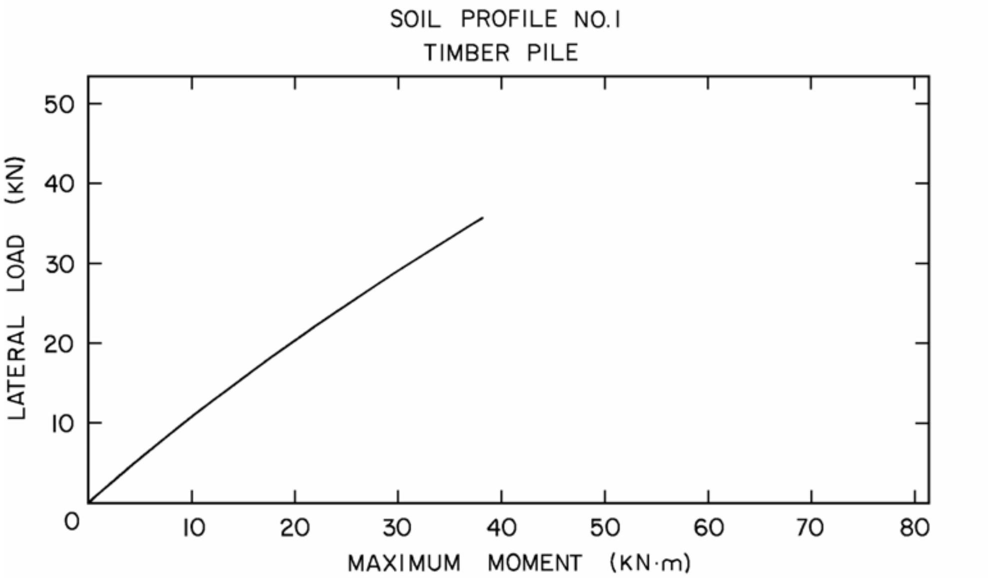


Figure 1.1.8-49 - Load Versus Deflection for Timber Pile, Soil Profile 1 and 3



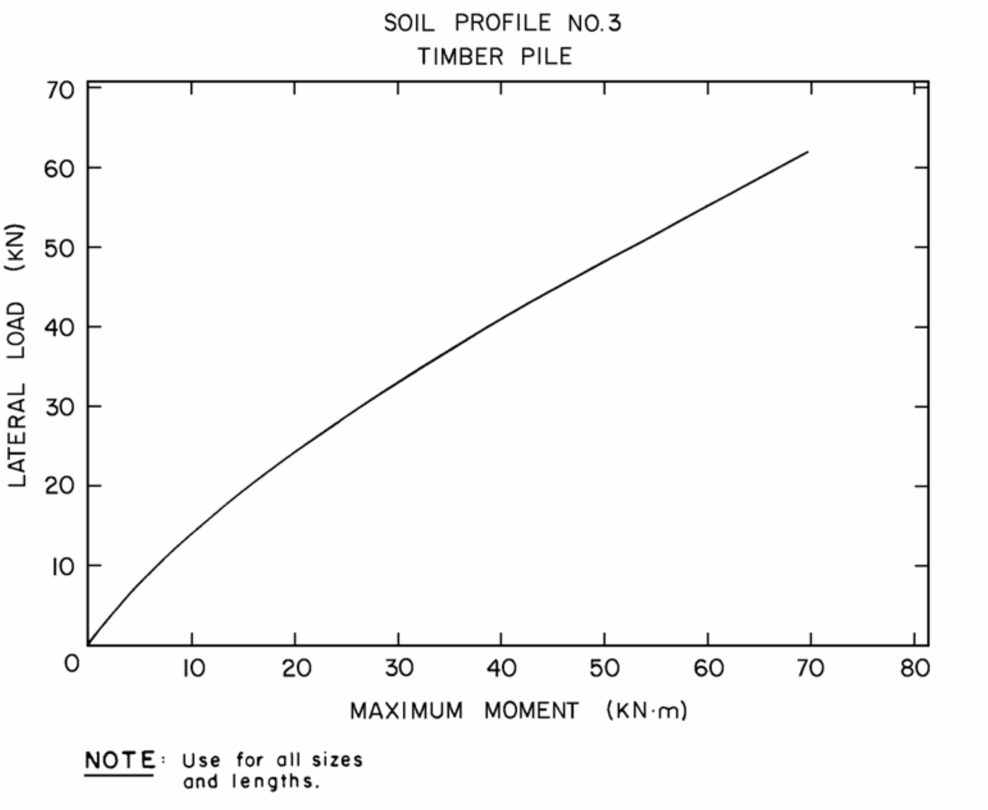


Figure 1.1.8-50 - Load Versus Maximum Moment for Timber Pile, Soil Profile 1 and 3

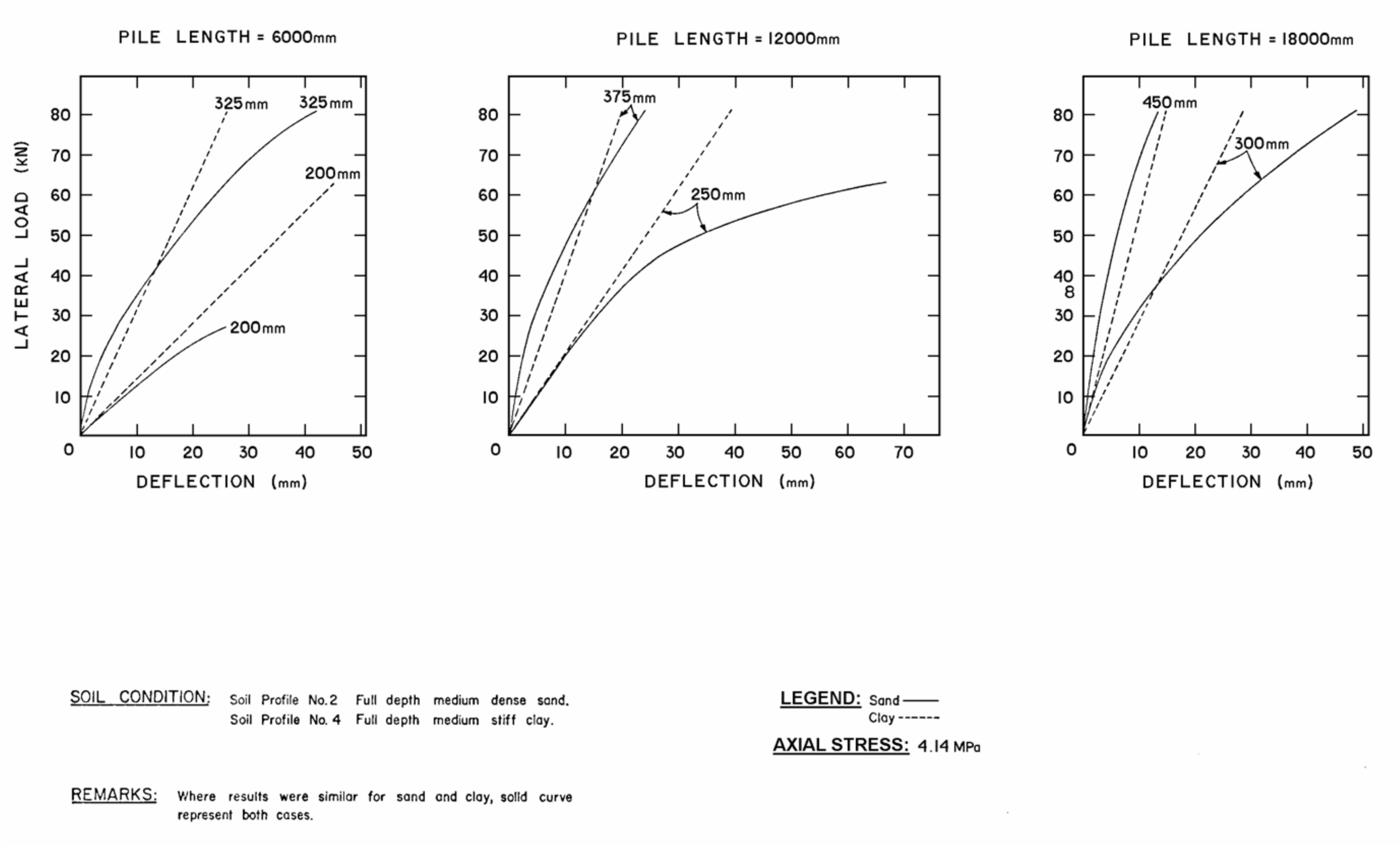


Figure 1.1.8-51 - Load Versus Deflection for Timber Pile, Soil Profile 2 and 4

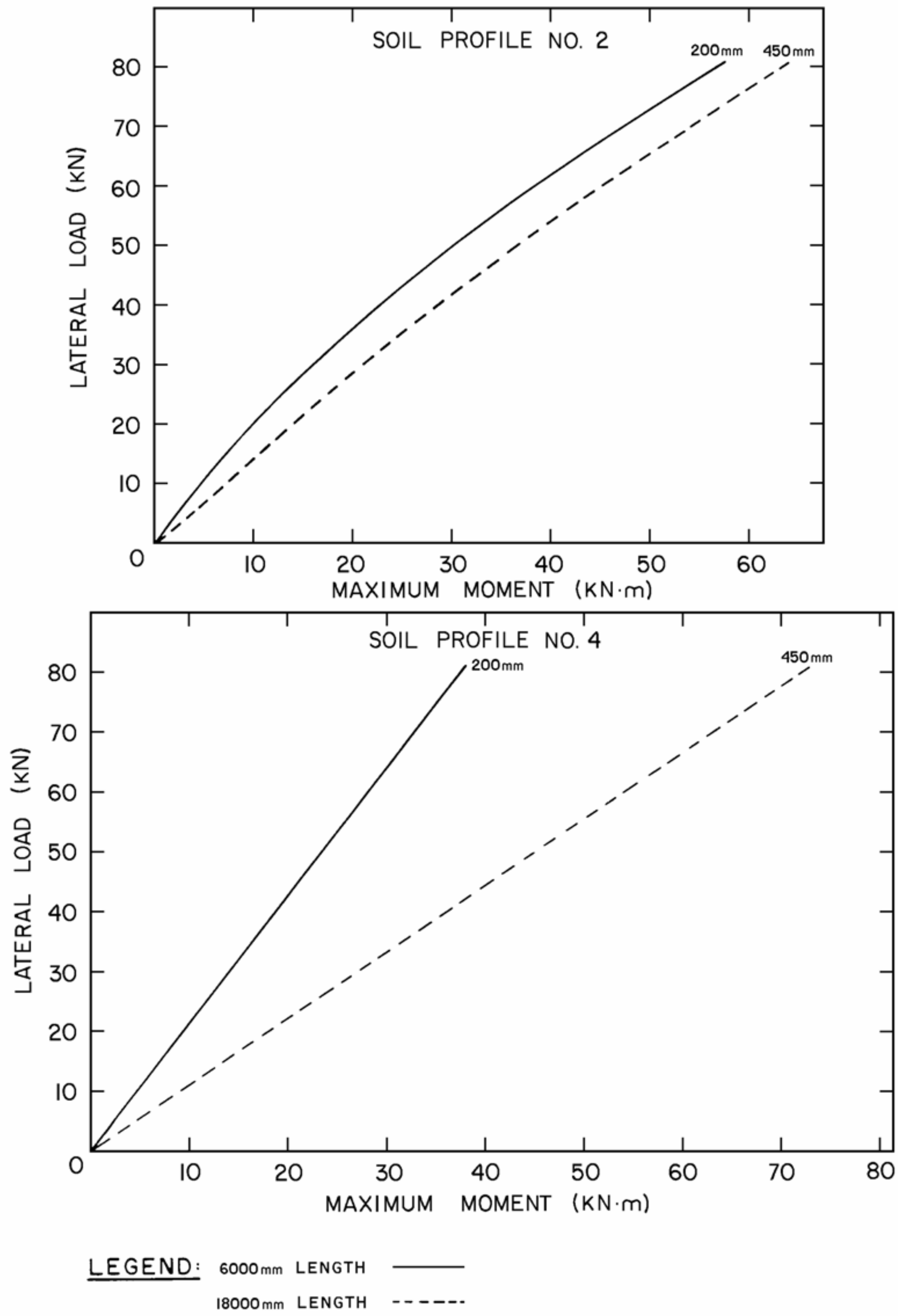


Figure 1.1.8-52 - Load Versus Maximum Moment for Timber Pile, Soil Profile 2 and 4

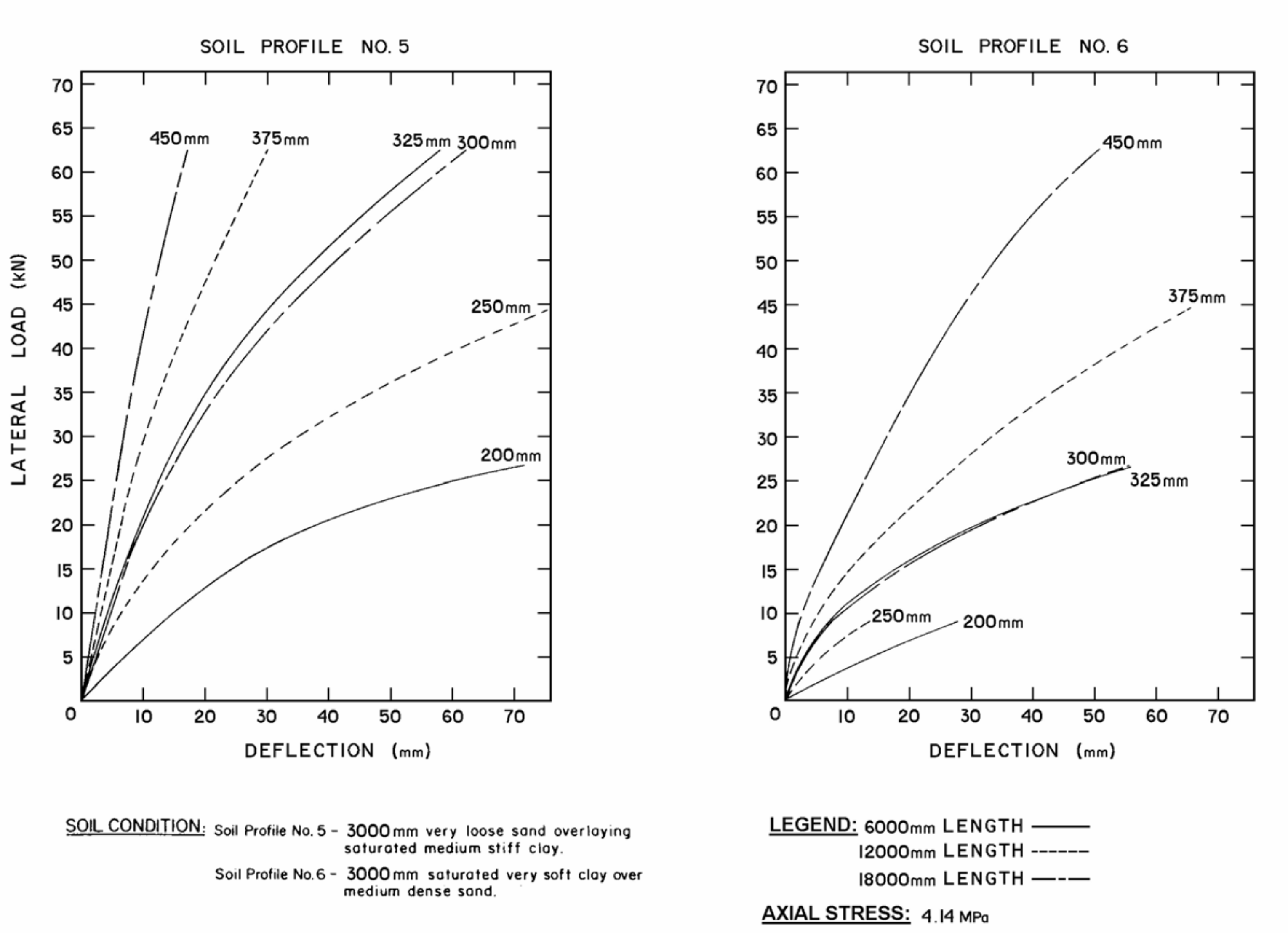


Figure 1.1.8-53 - Load Versus Deflection for Timber Pile, Soil Profile 5 and 6

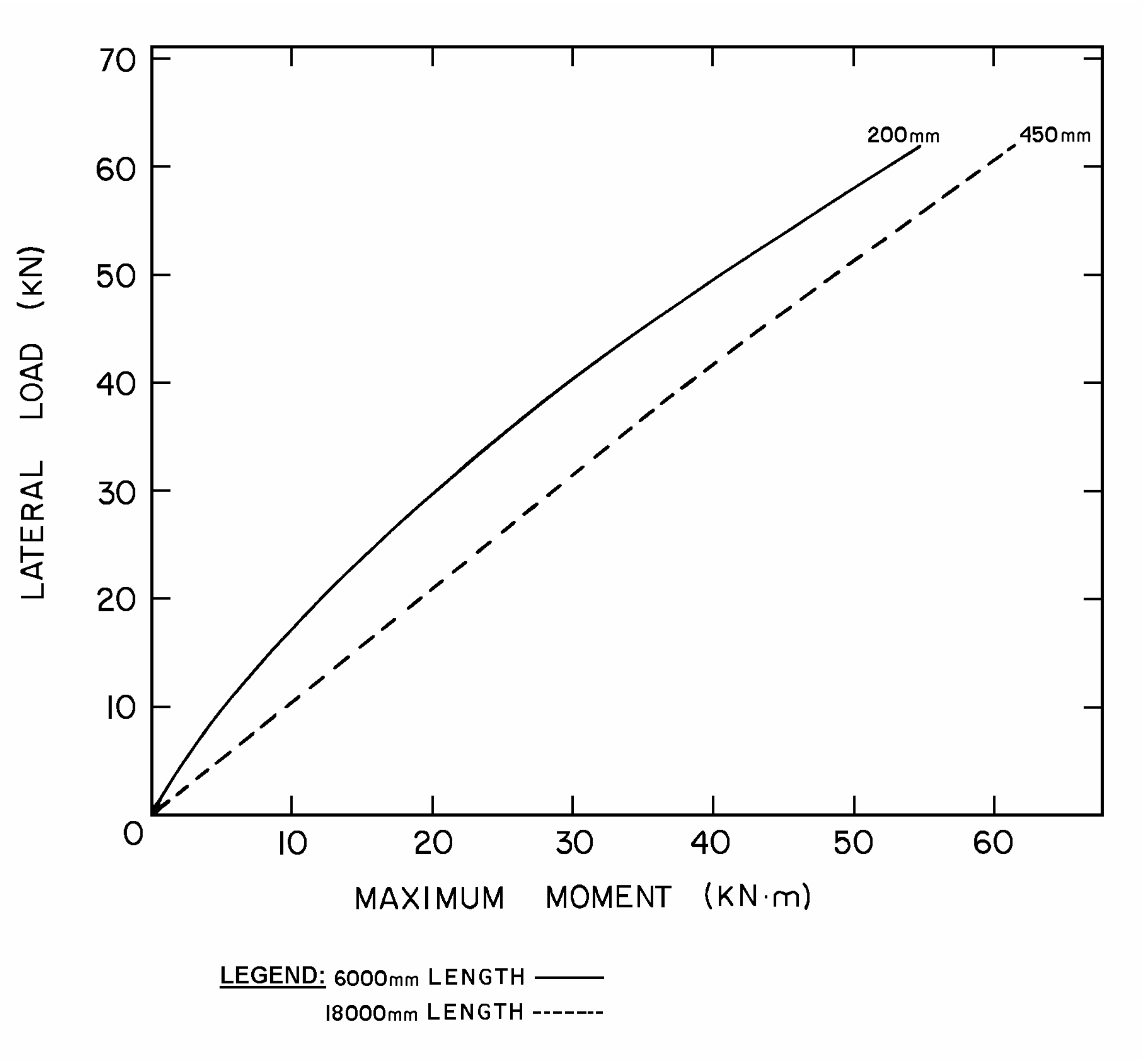


Figure 1.1.8-54 - Load Versus Maximum Moment for Timber Pile, Soil Profile 5

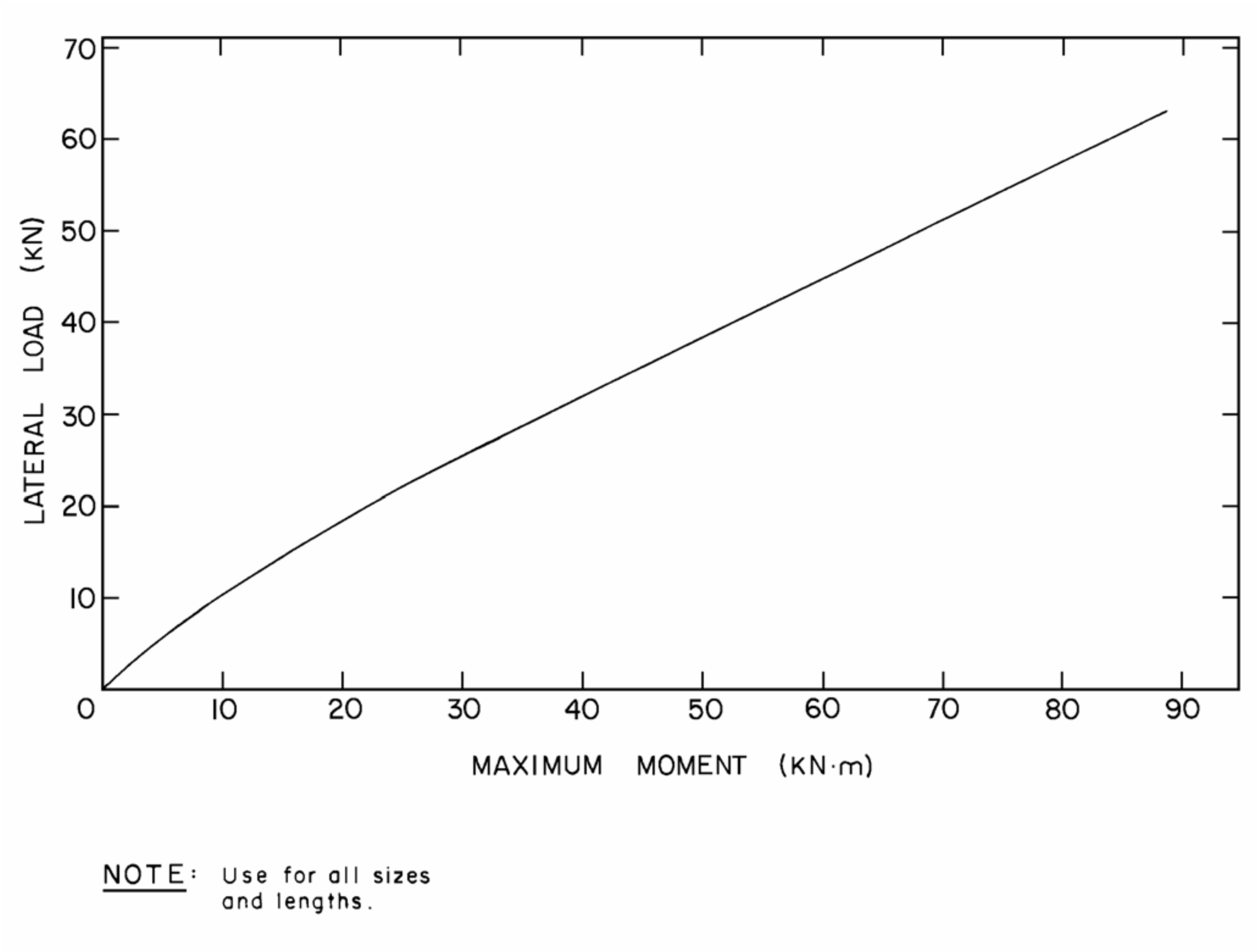


Figure 1.1.8-55 - Load Versus Maximum Moment for Timber Pile, Soil Profile 6

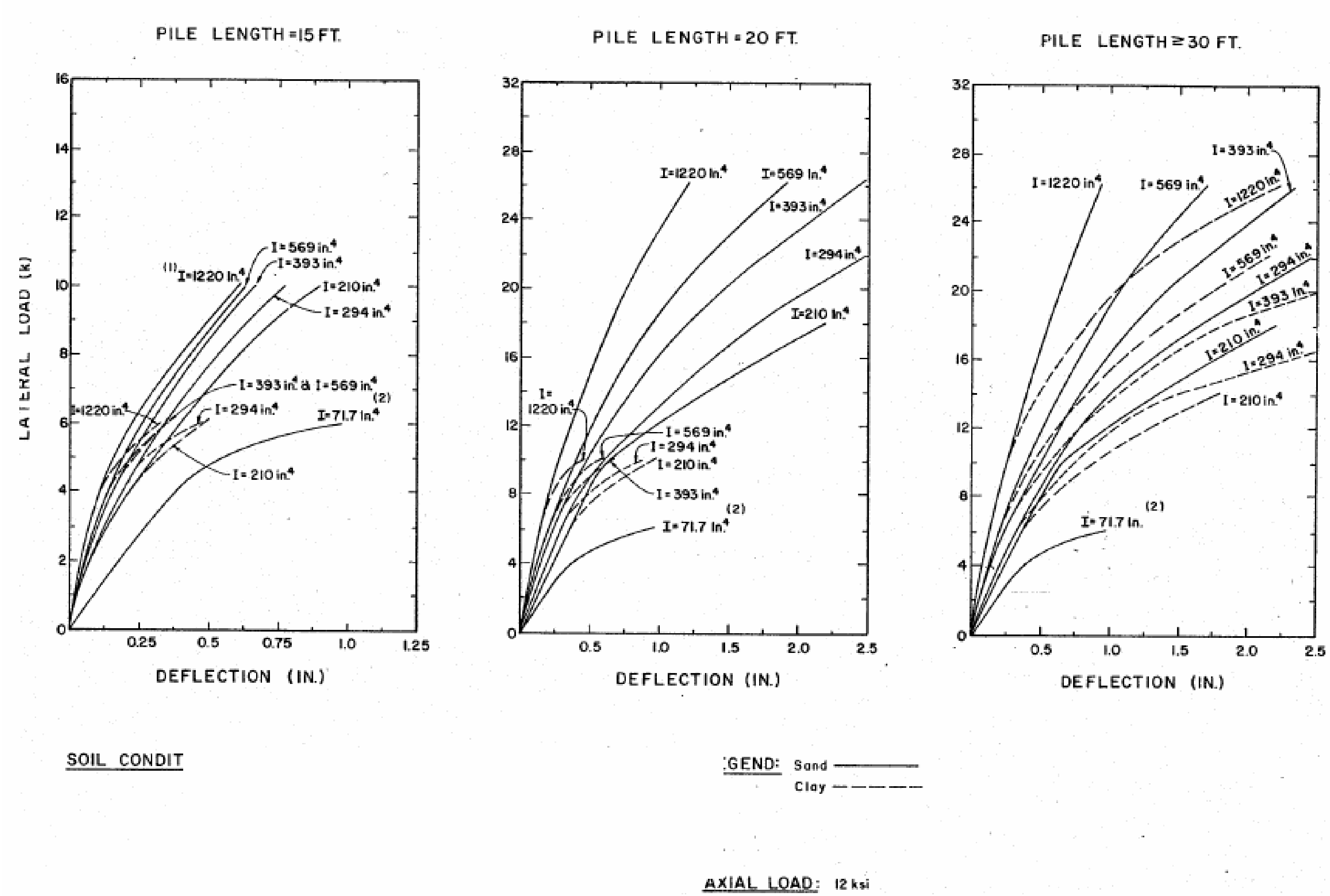


Figure 1.1.8-1 - Load Versus Deflection for Steel H-Pile, Soil Profiles 1 and 3

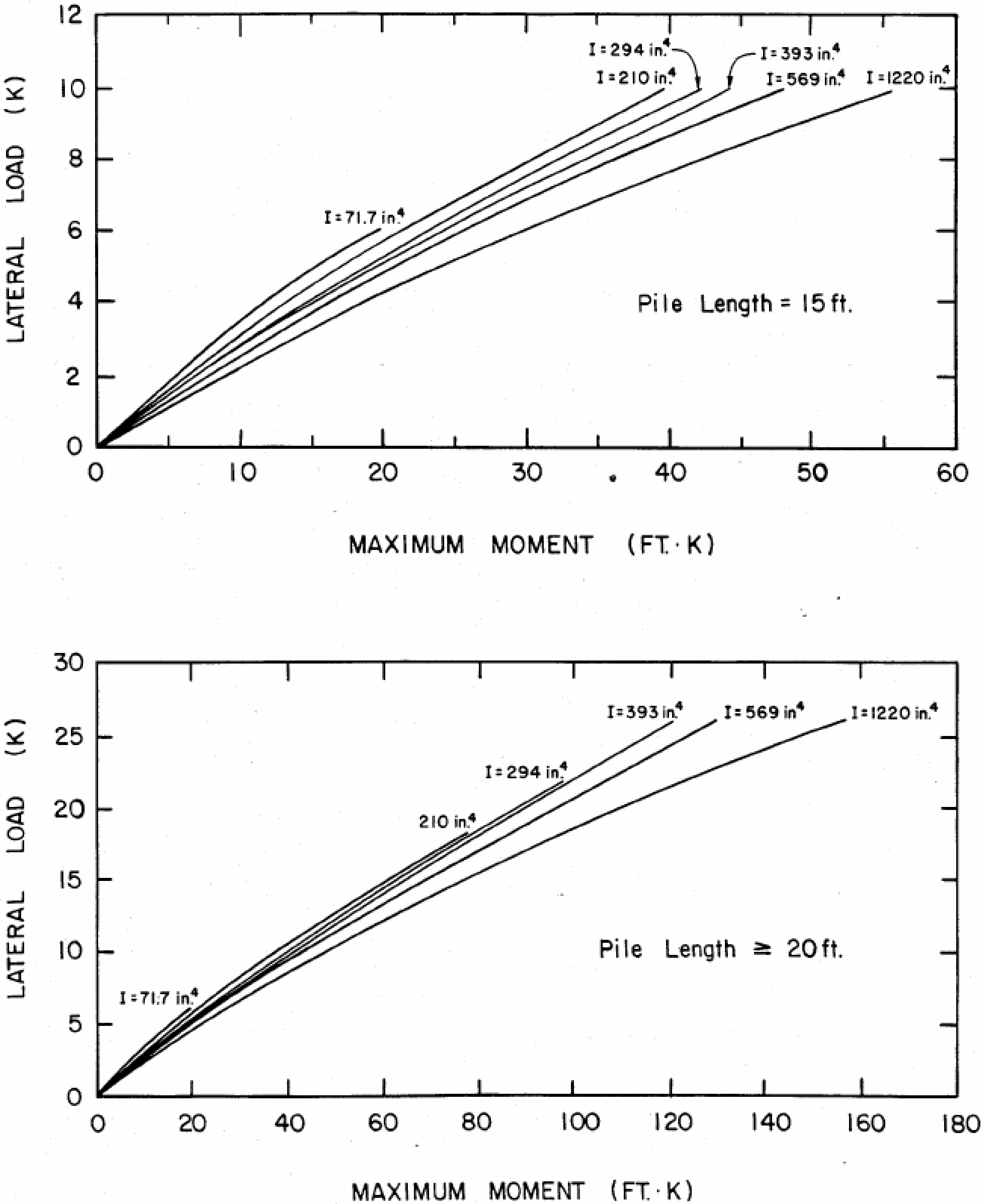


Figure 1.1.8-2 - Load Versus Maximum Moment for Steel H-Piles, Soil Profiles 1 and 3

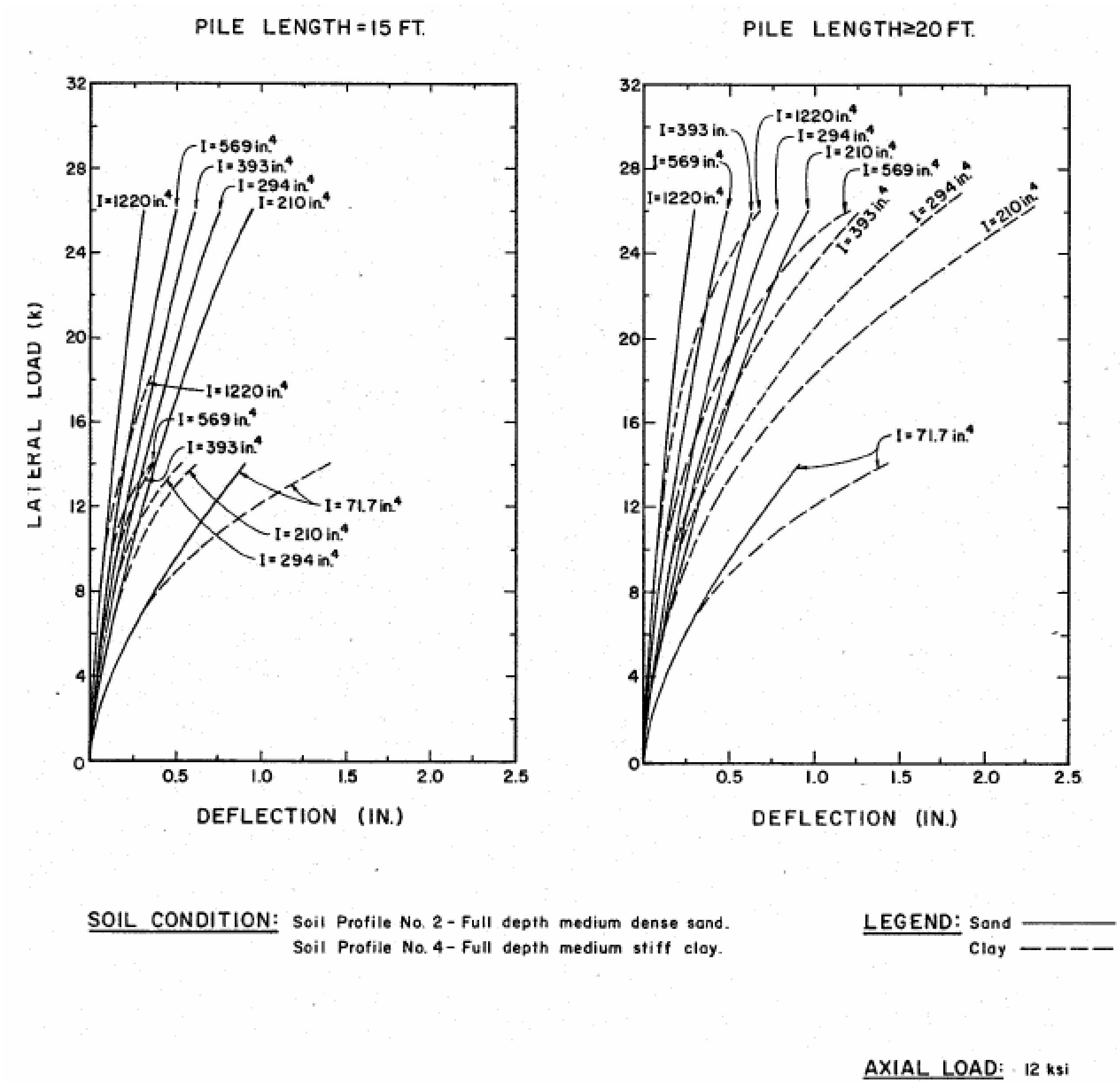
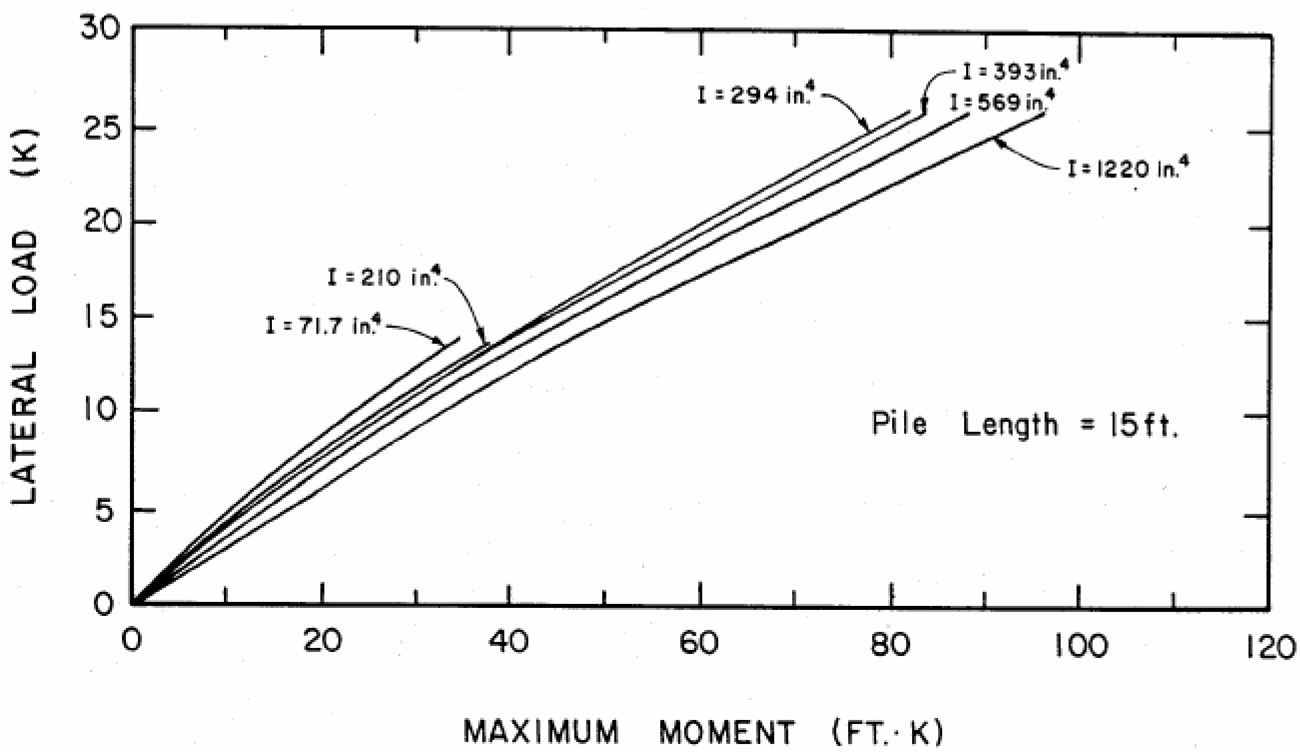


Figure 1.1.8-3 - Load Versus Deflection for Steel H-Pile, Soil Profiles 2 and 4



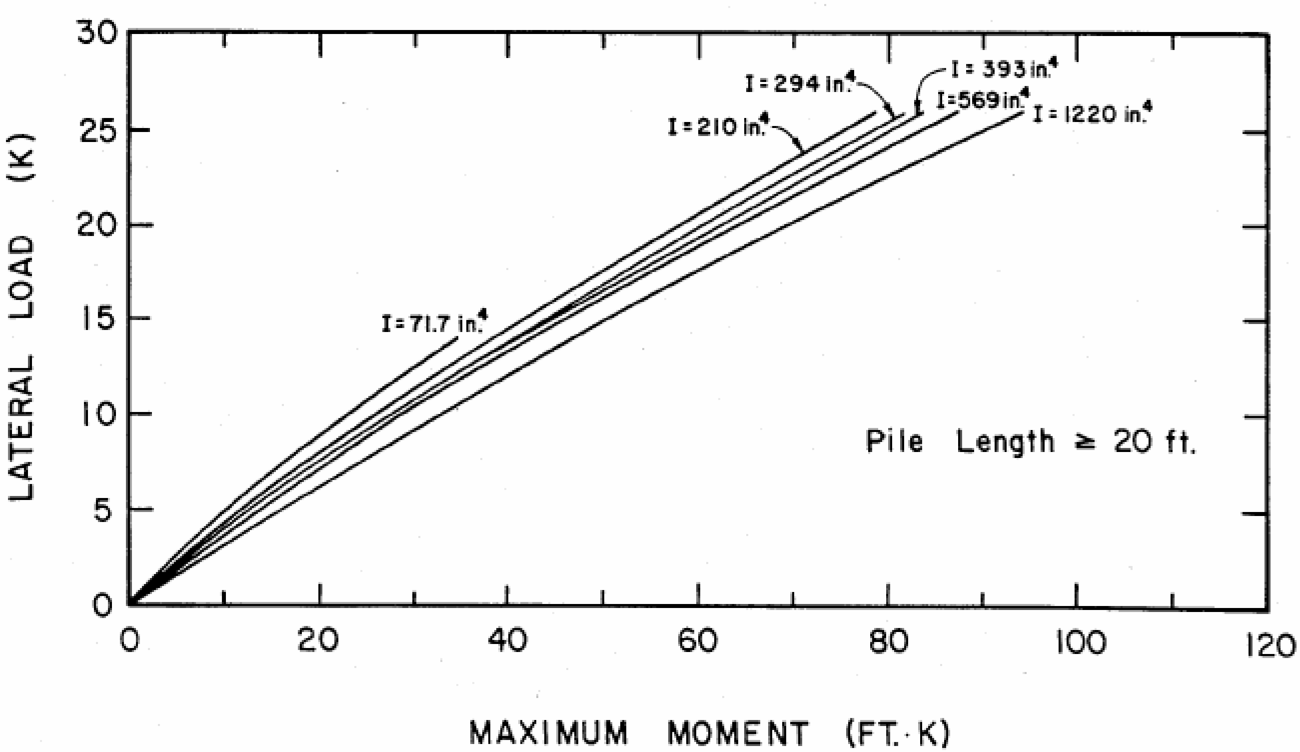
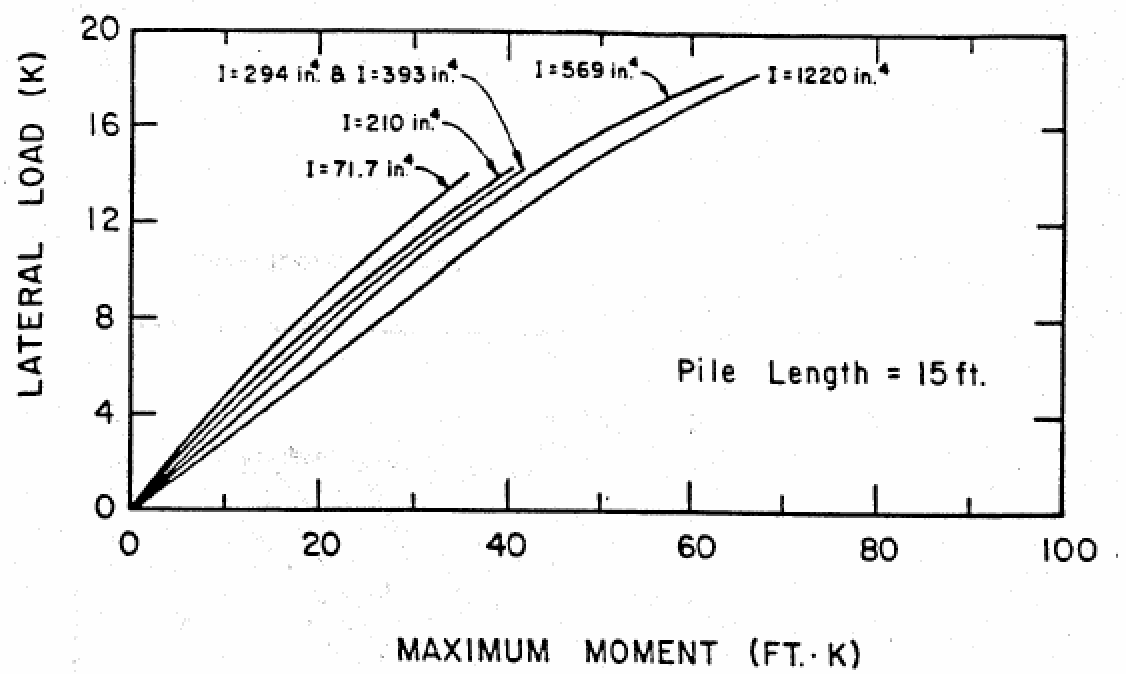


Figure 1.1.8-4 - Load Versus Maximum Moment for Steel, H-Pile, Soil Profile 2



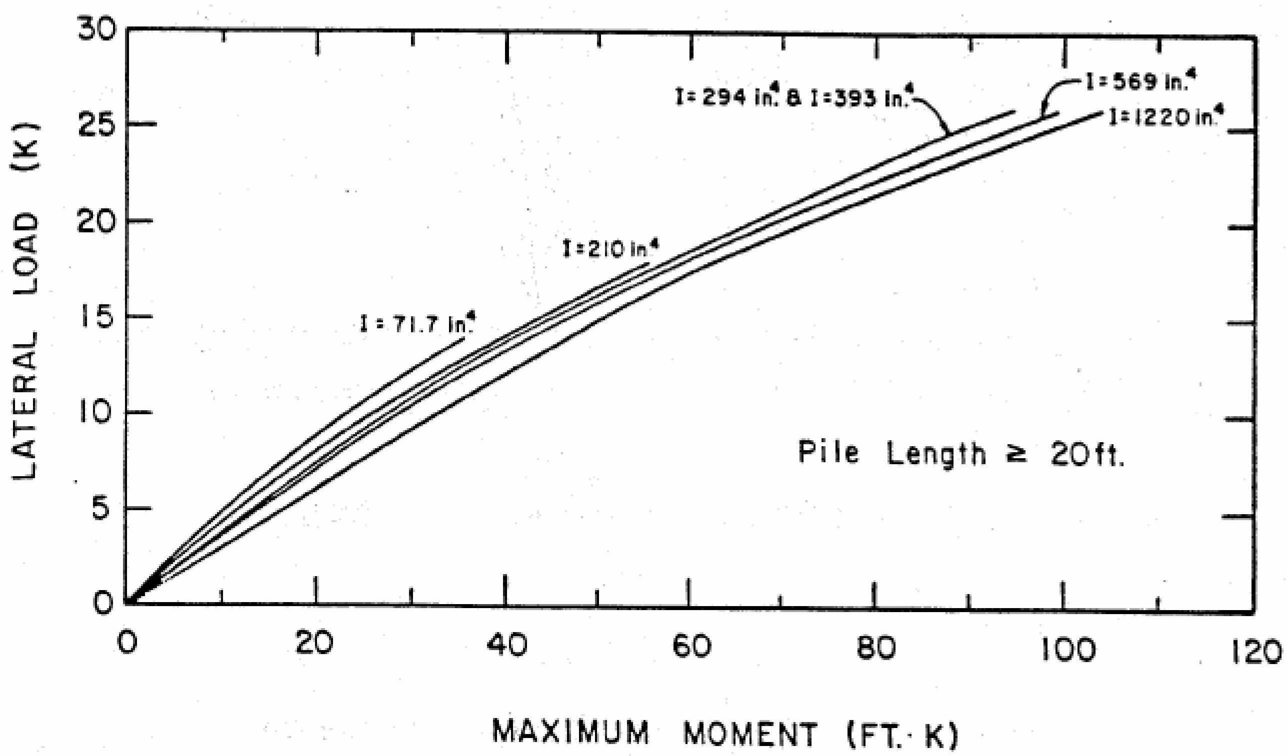


Figure 1.1.8-5 - Load Versus Maximum Moment for Steel H-Pile, Soil Profile 4

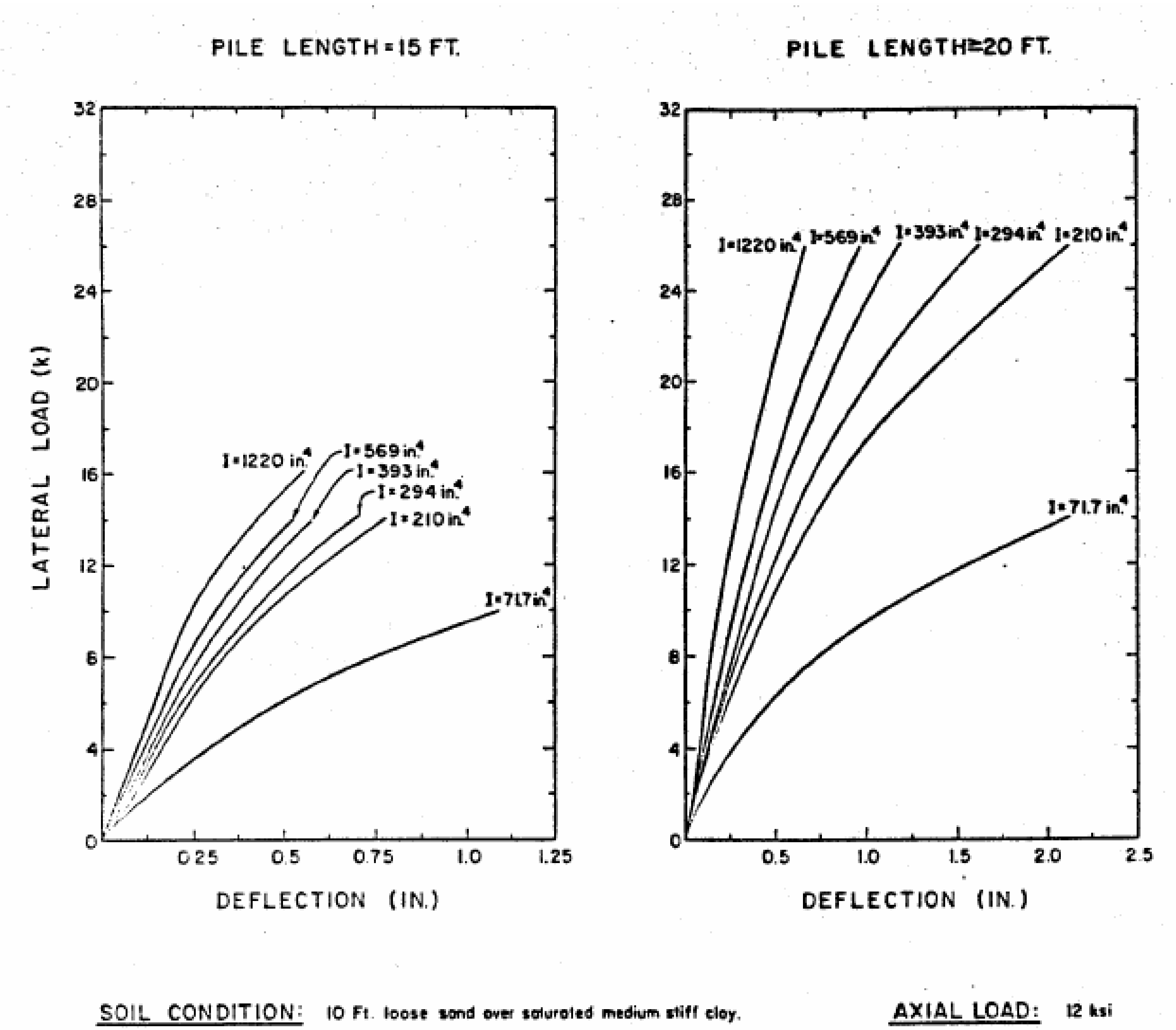


Figure 1.1.8-6 - Load Versus Deflection for Steel H-Pile, Soil Profile 5

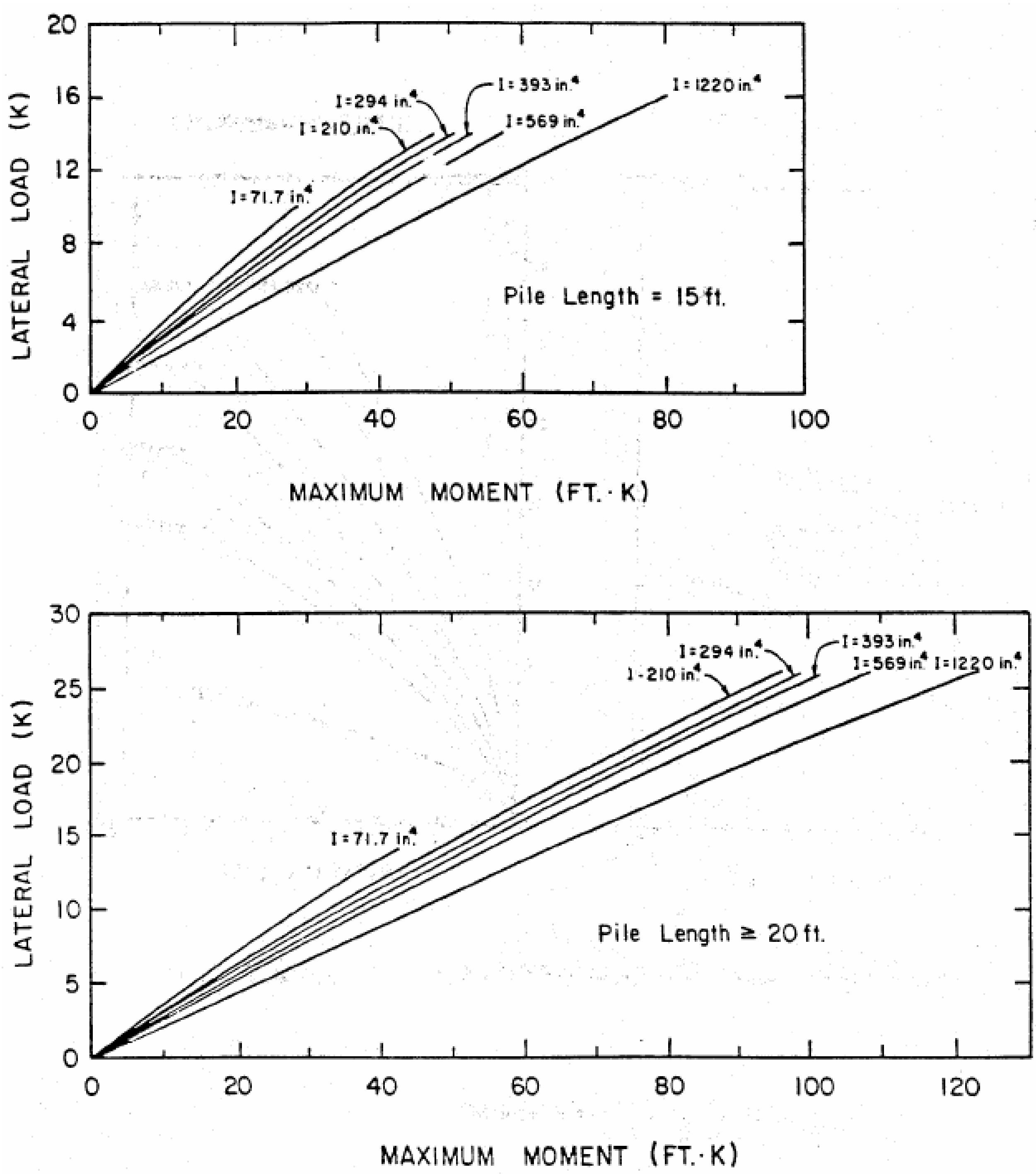


Figure 1.1.8-7 - Load Versus Maximum Moment for Steel H-Pile, Soil Profile 5

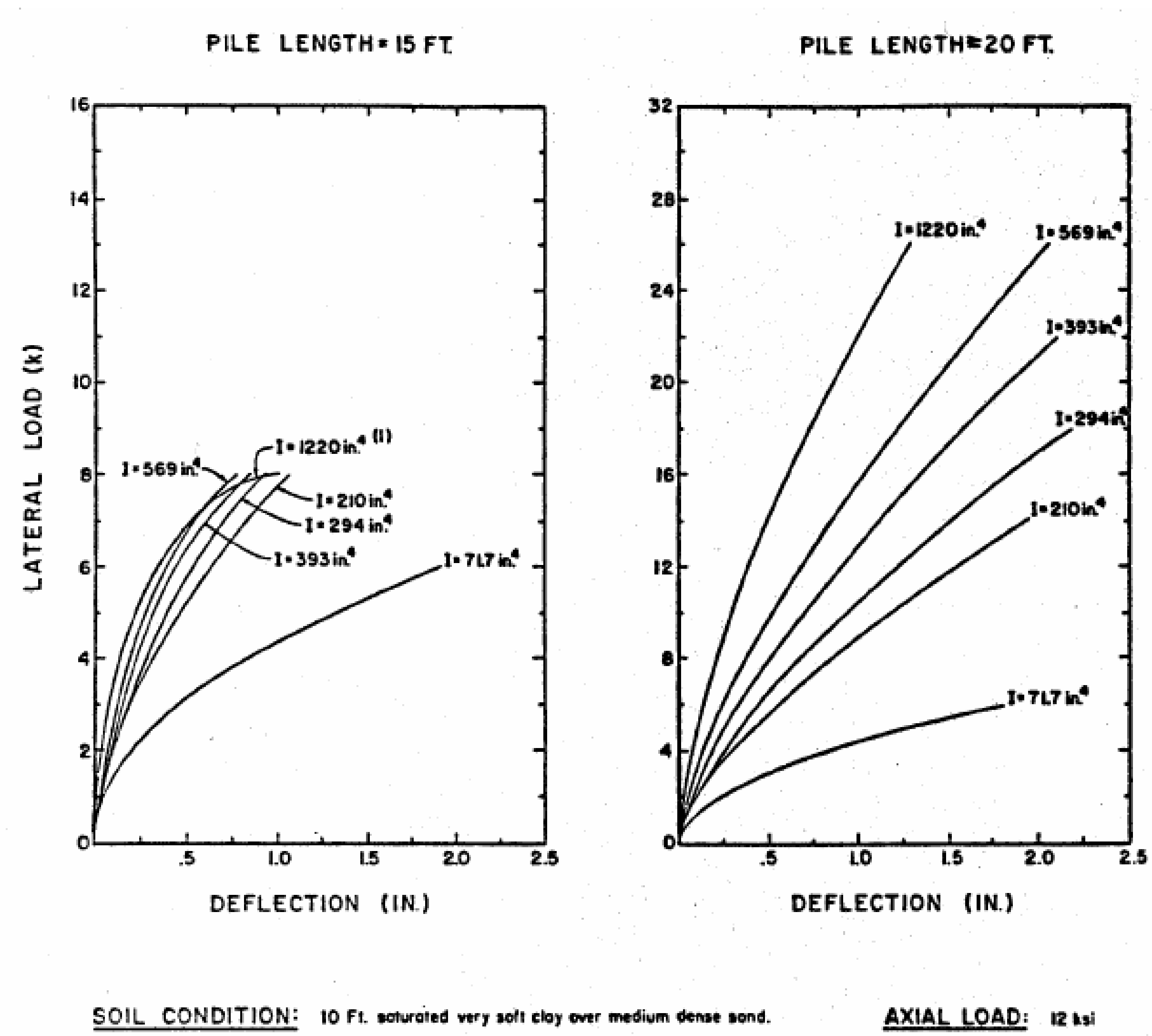
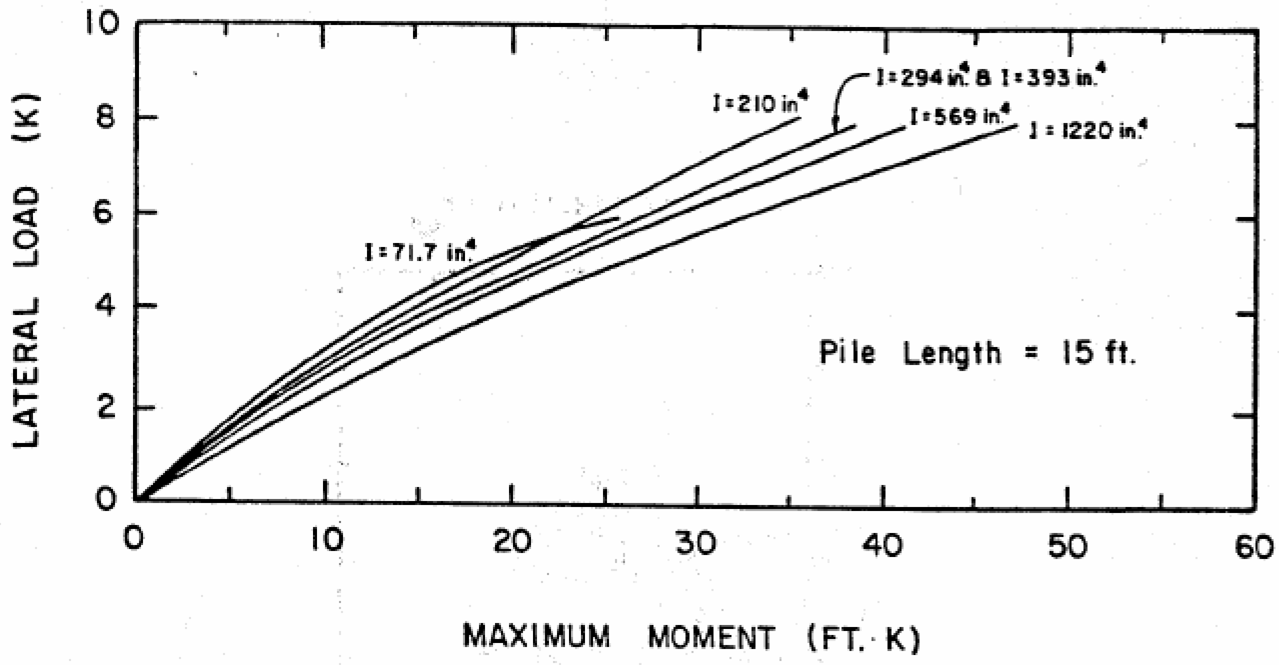


Figure 1.1.8-8 - Load Versus Deflection for Steel H-Pile, Soil Profile 6



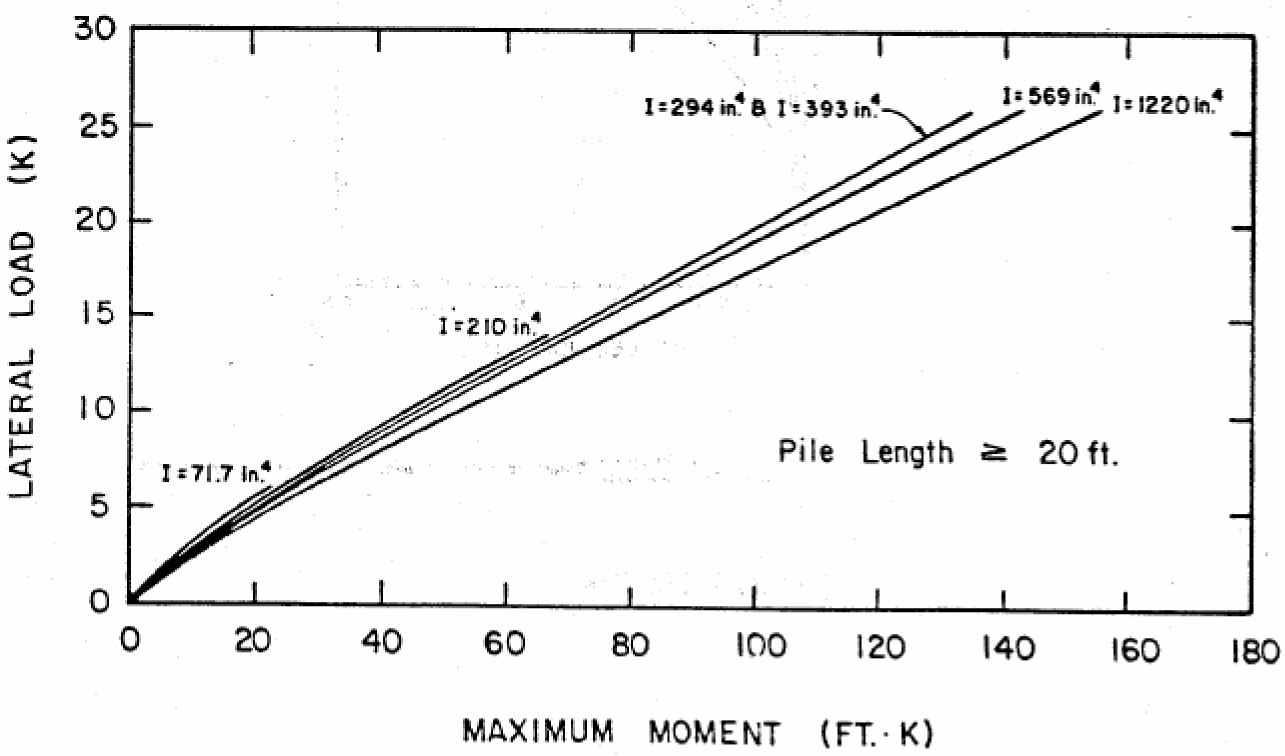
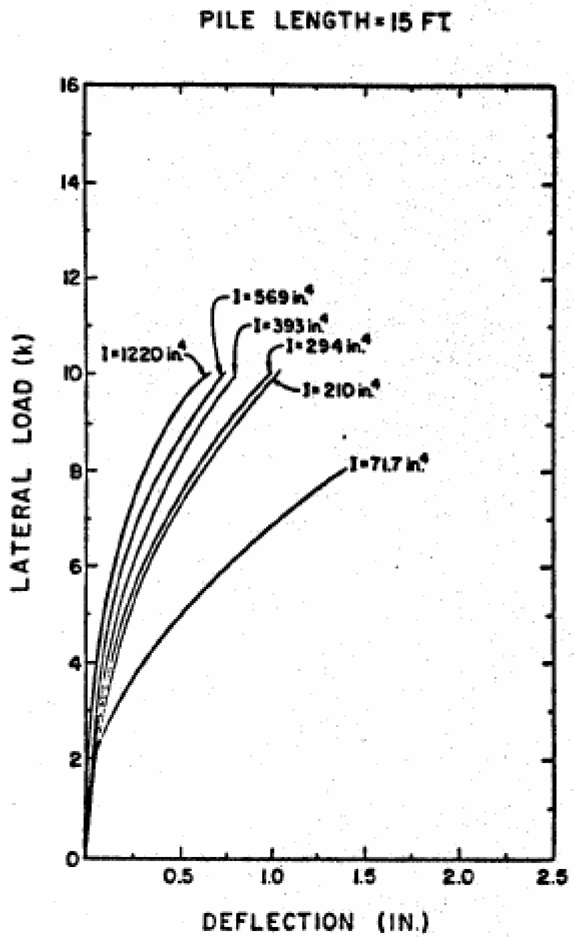


Figure 1.1.8-9 - Load Versus Maximum Moment for Steel H-Pile, Soil Profile 6



SOIL CONDITION: Saturated soft clay over 2.0 feet soft or weathered rock.

AXIAL LOAD: 12 ksi

Figure 1.1.8-10 - Load Versus Deflection for Steel H-Pile, Soil Profile 7

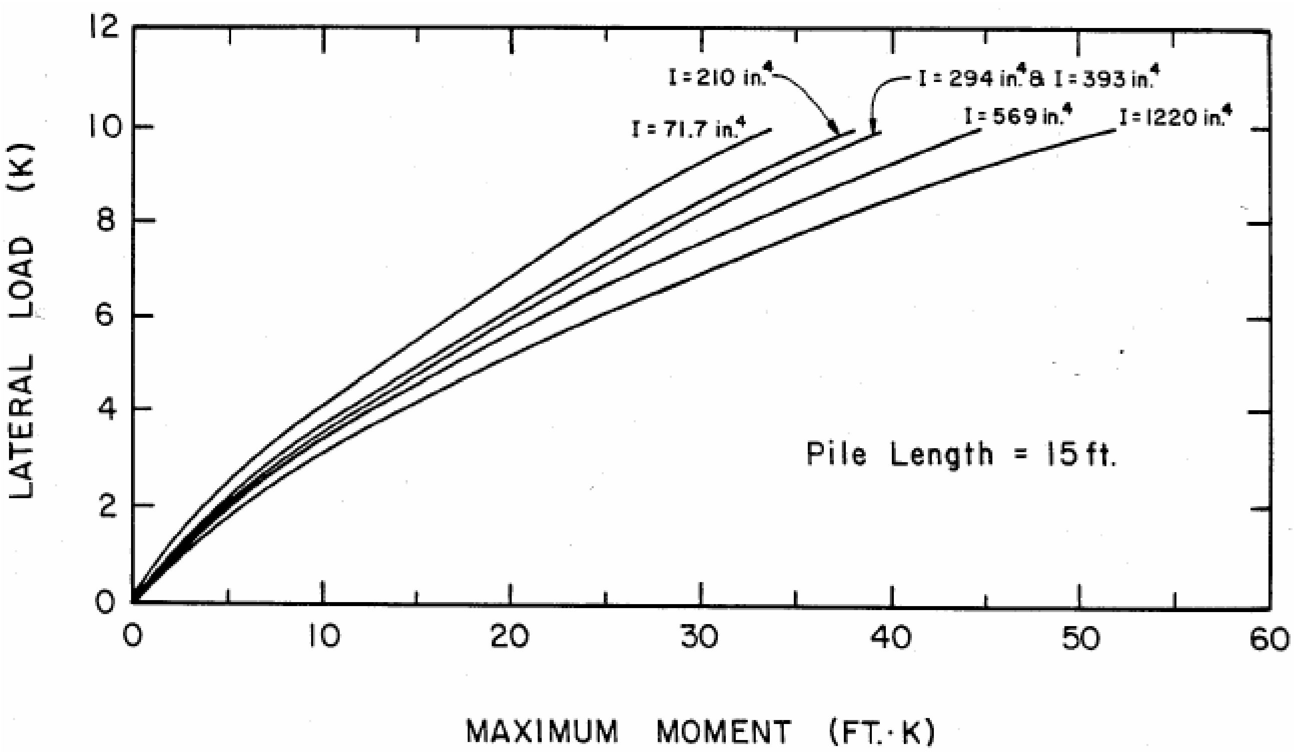
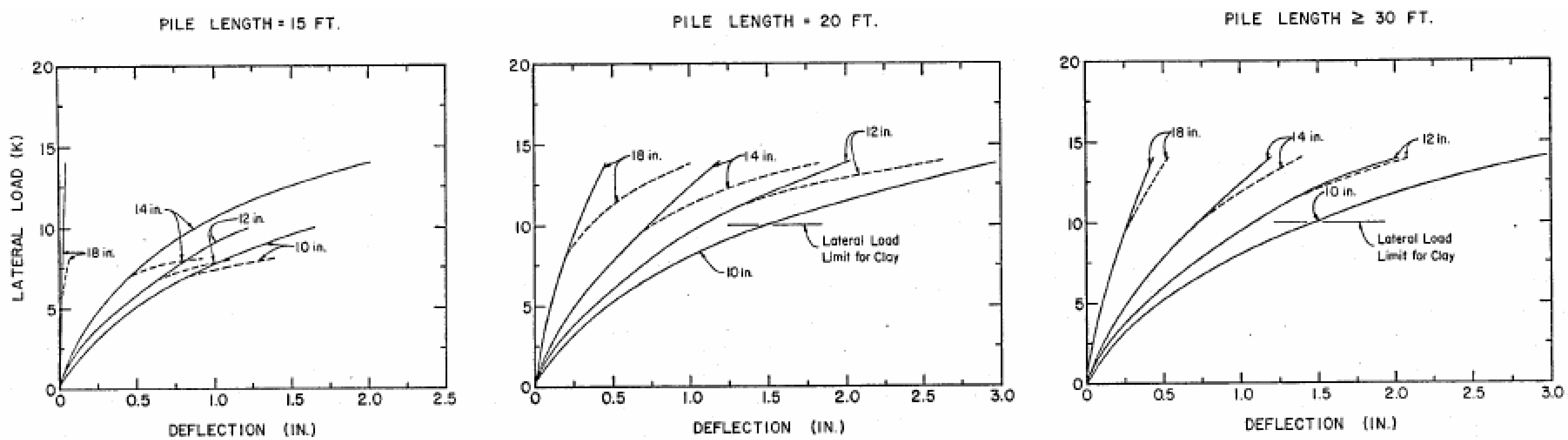


Figure 1.1.8-11 - Load Versus Maximum Moment for Steel H-Pile, Soil Profile 7



SOIL CONDITION: Soil Profile No. 1 – Full depth saturated loose sand.
Soil Profile No. 3 – Full depth saturated soft clay.

LEGEND: Sand ——— Clay - - - - -

REMARKS: Where results were similar for sand and clay, solid curve represents both cases. Maximum allowable lateral load for 10 in. pile in clay is as noted.

AXIAL LOAD: 0.35 f'c

Figure 1.1.8-12 - Load Versus Deflection for Cast-in-Place Pile, Soil Profiles 1 and 3

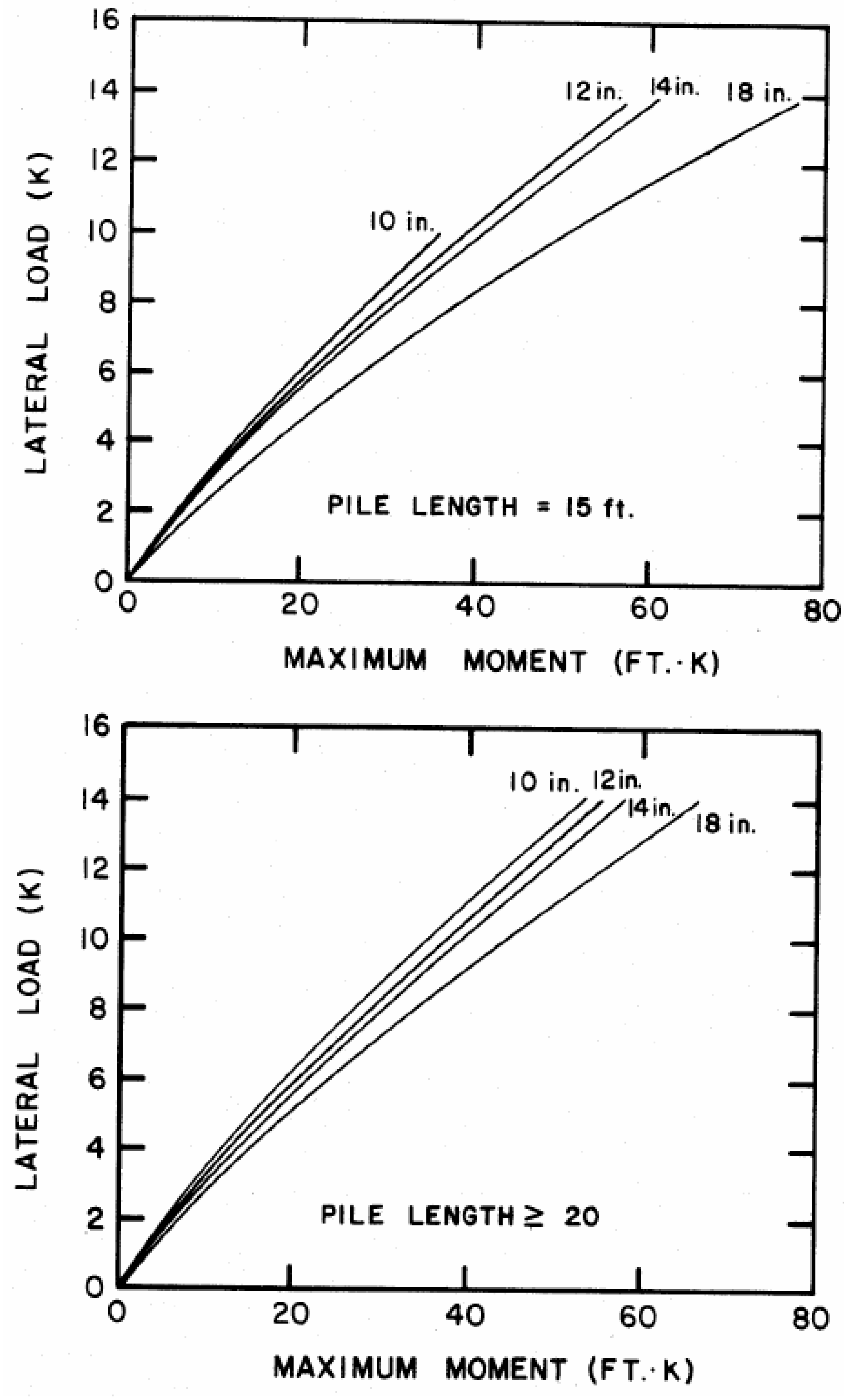


Figure 1.1.8-13 - Load Versus Maximum Moment for Cast-in-place Pile, Soil Profiles 1 and 3

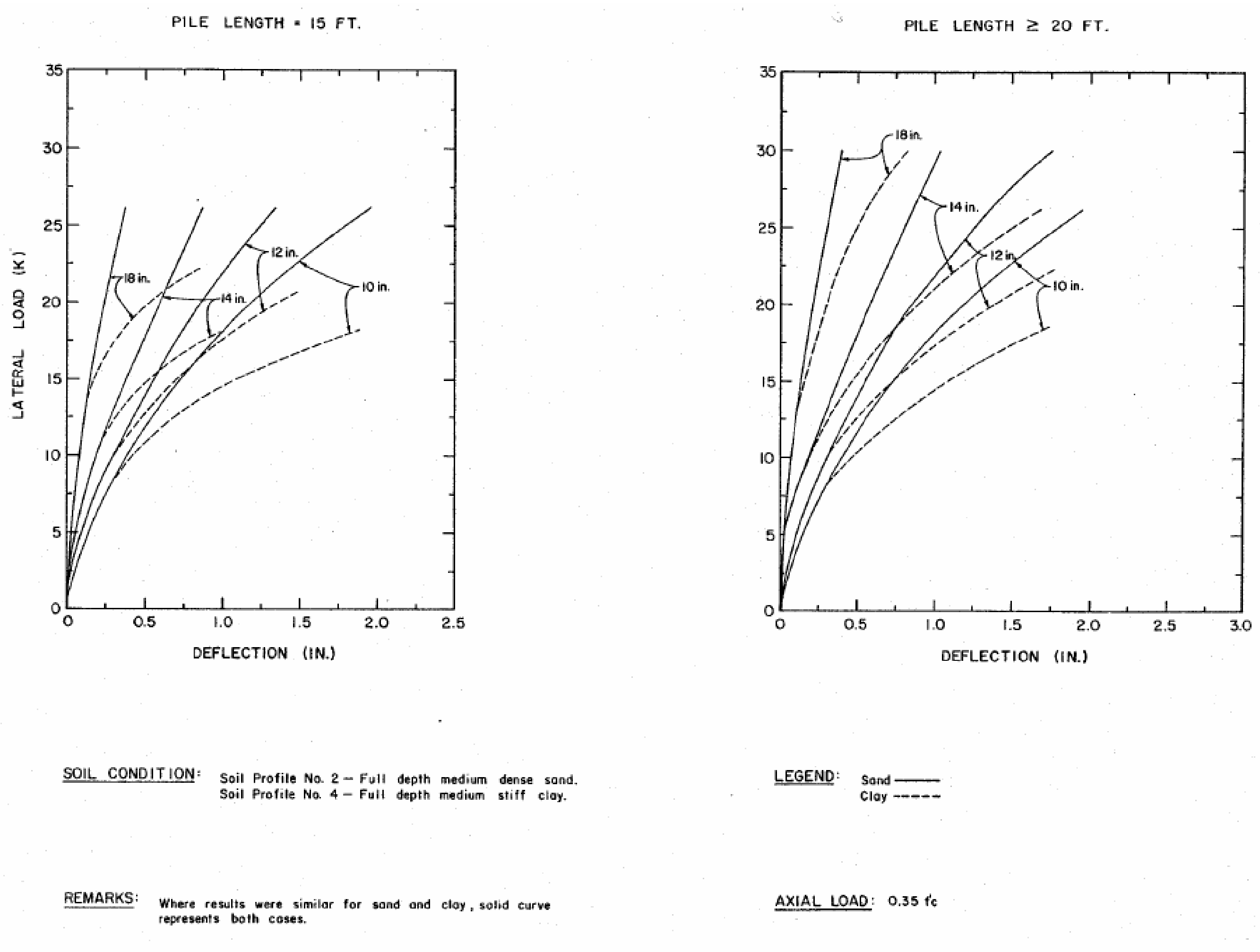


Figure 1.1.8-14 - Load Versus Deflection for Cast-in-Place Pile, Soil Profiles 2 and 4

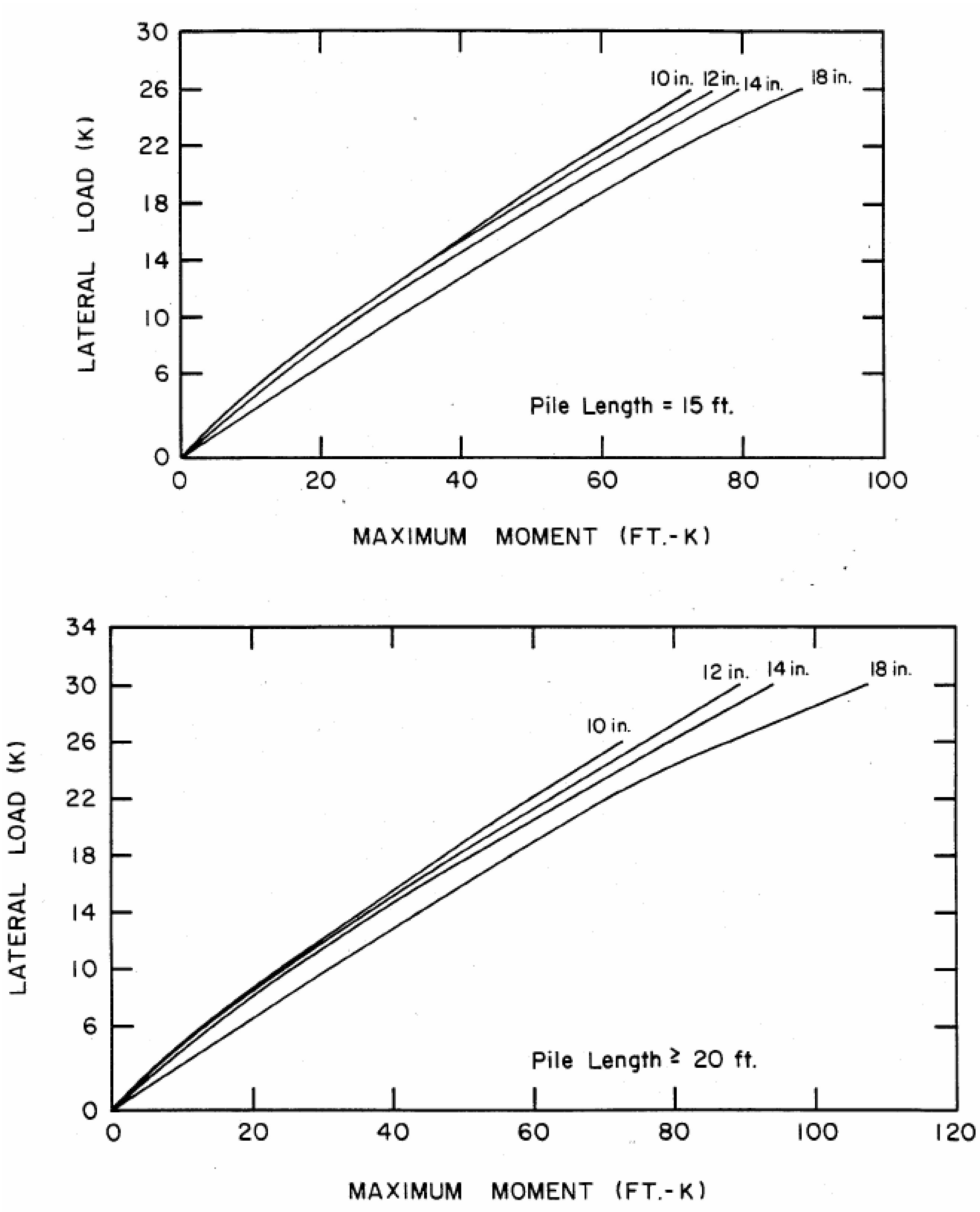
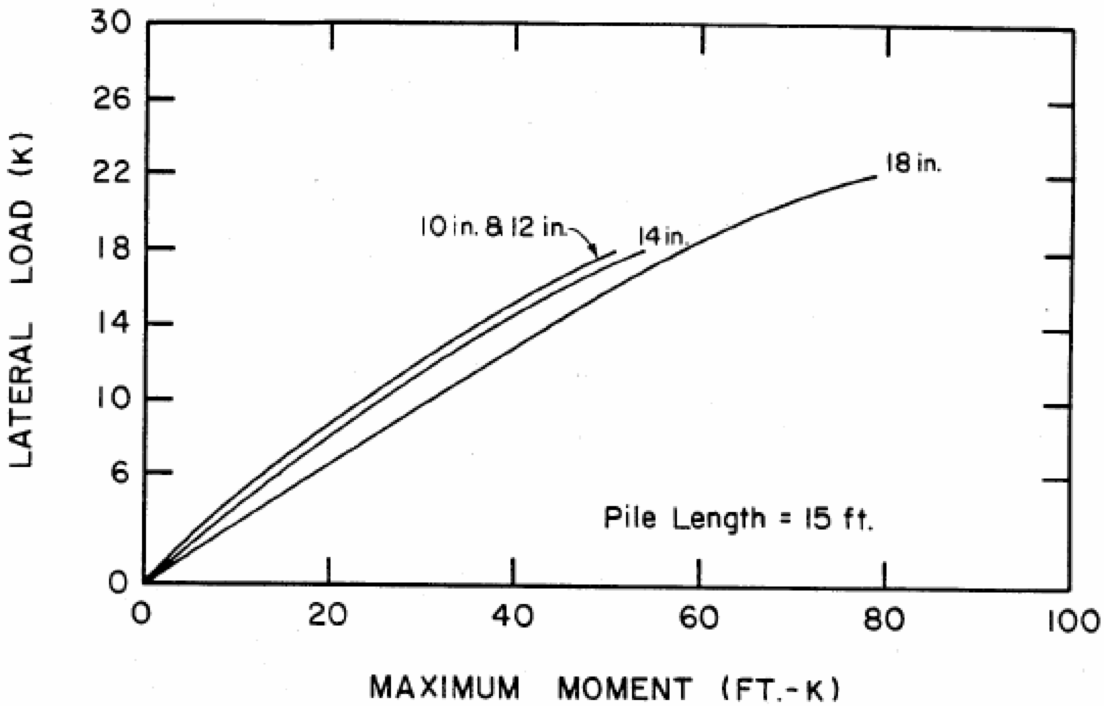


Figure 1.1.8-15 - Load Versus Maximum Moment for Cast-in-Place Pile, Soil Profile 2



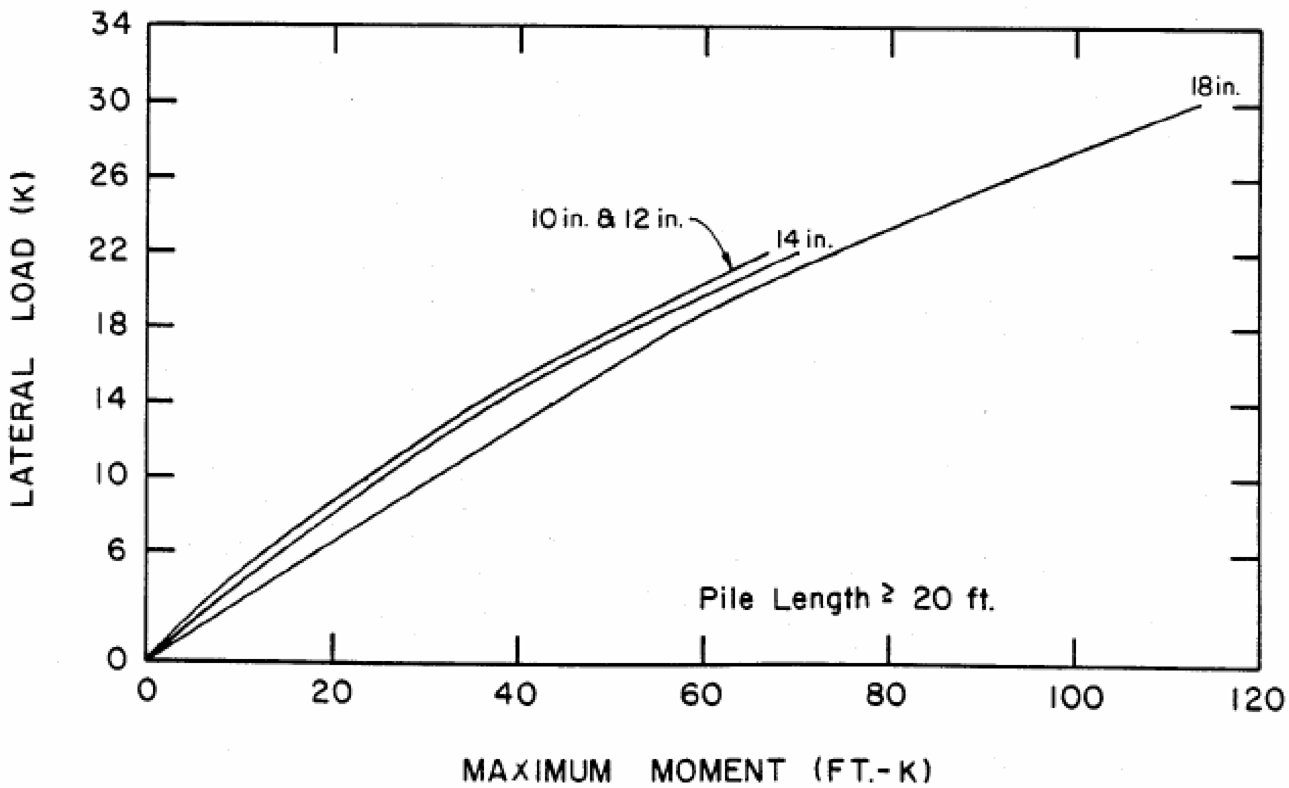


Figure 1.1.8-16 - Load Versus Maximum Moment for Cast-in-Place Pile, Soil Profile 4

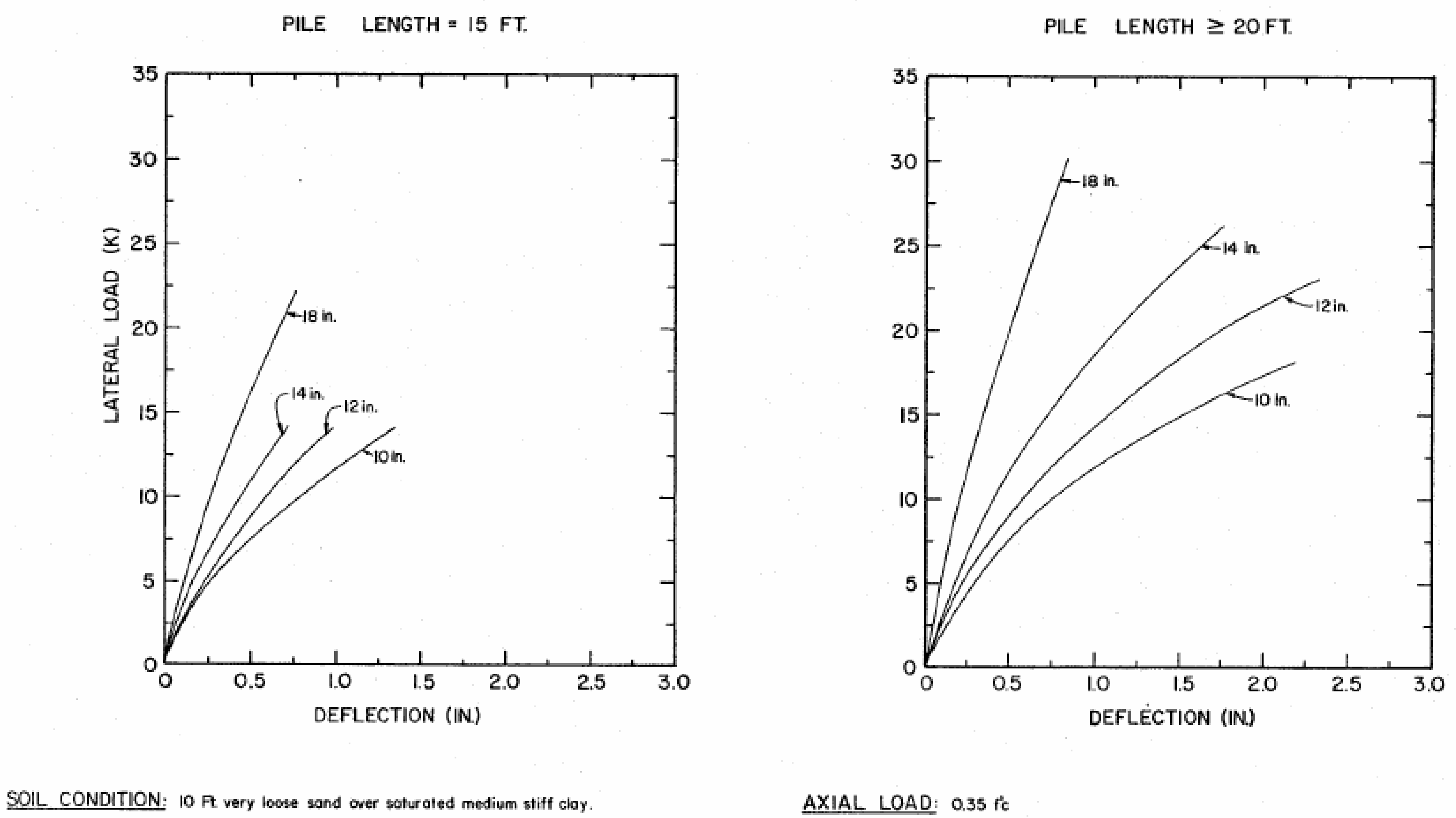


Figure 1.1.8-17 - Load Versus Deflection for Cast-in-Place Pile, Soil Profile 5

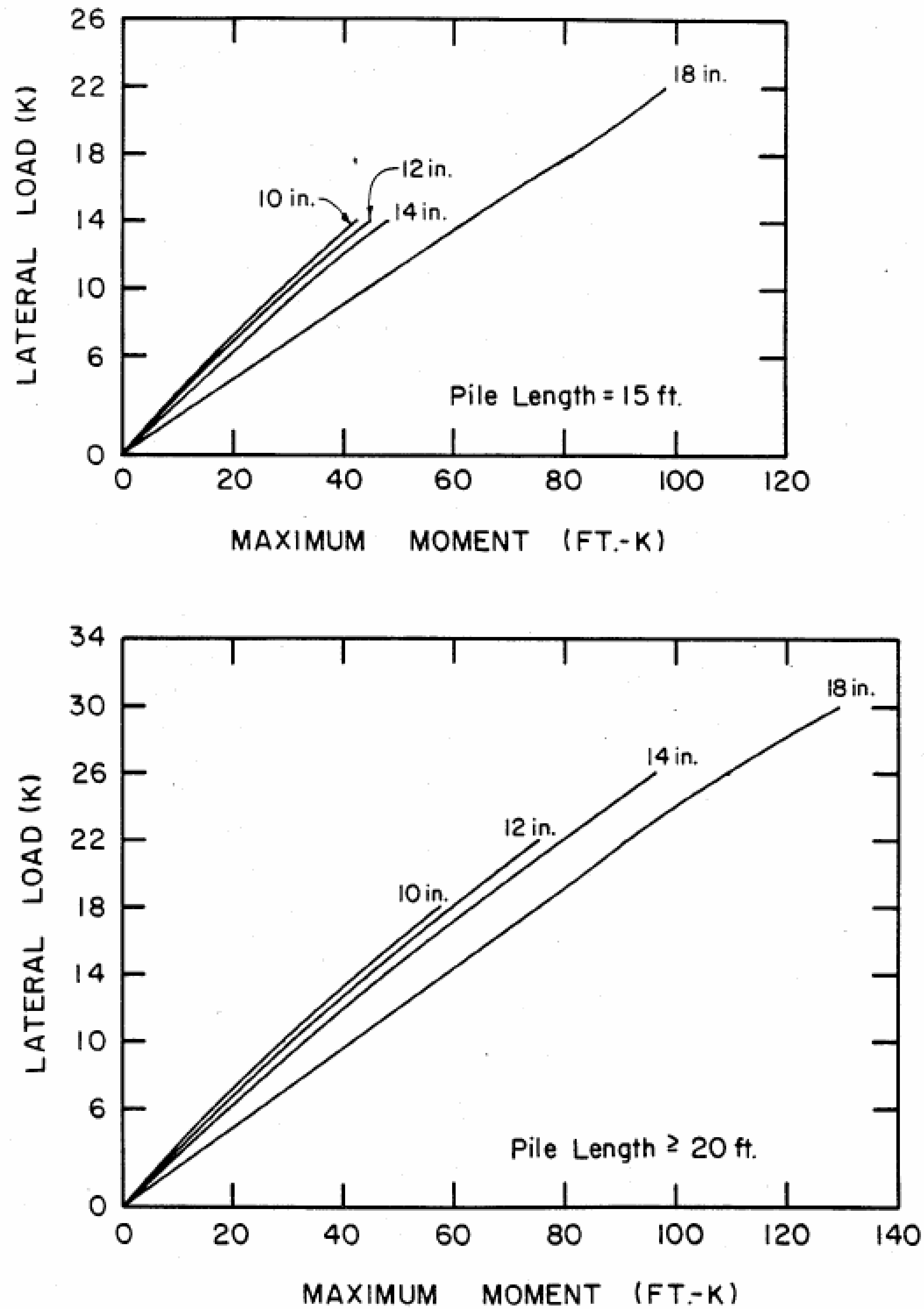


Figure 1.1.8-18 - Load Versus Maximum Moment for Cast-in-Place Pile, Soil Profile 5

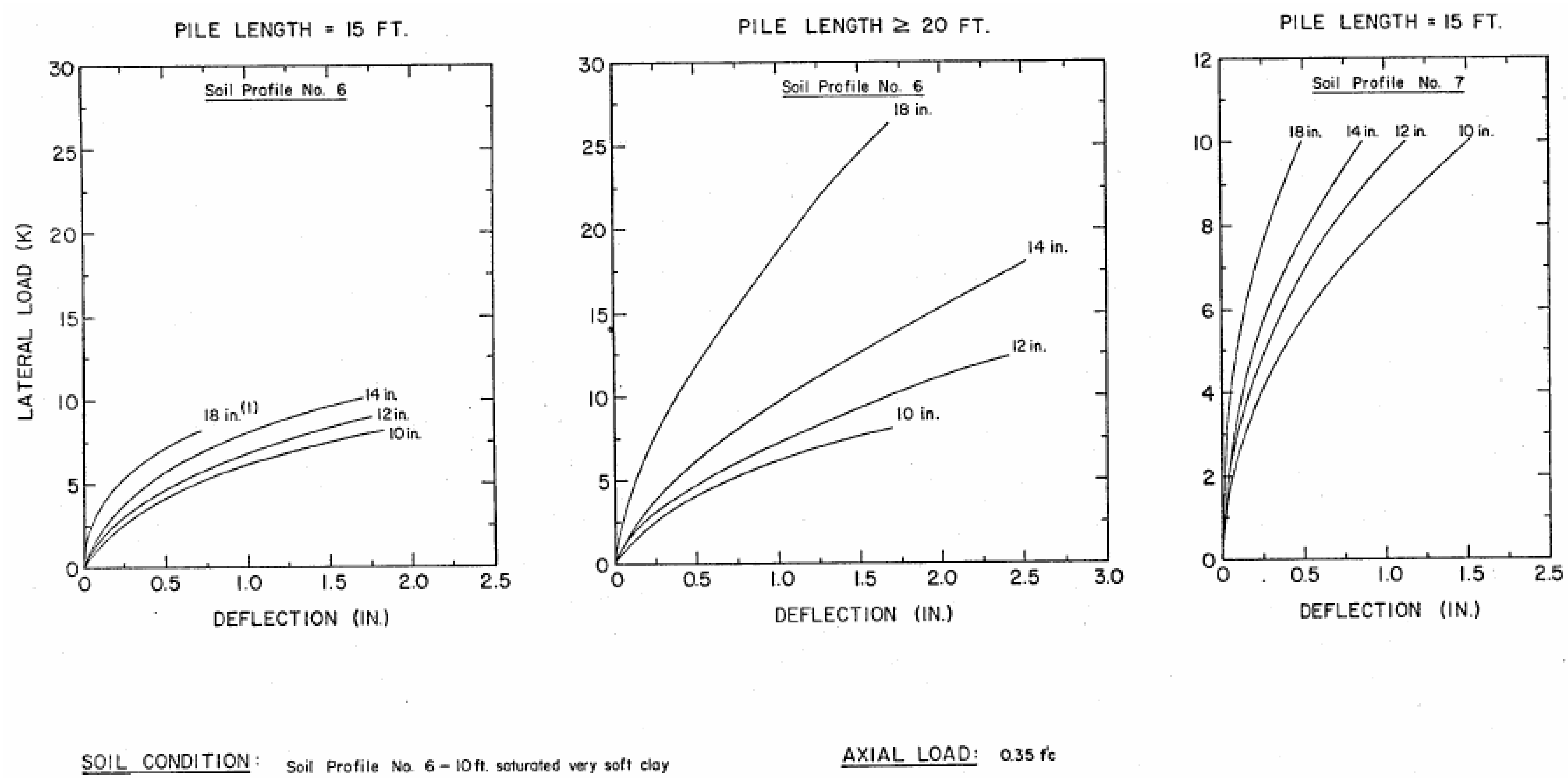
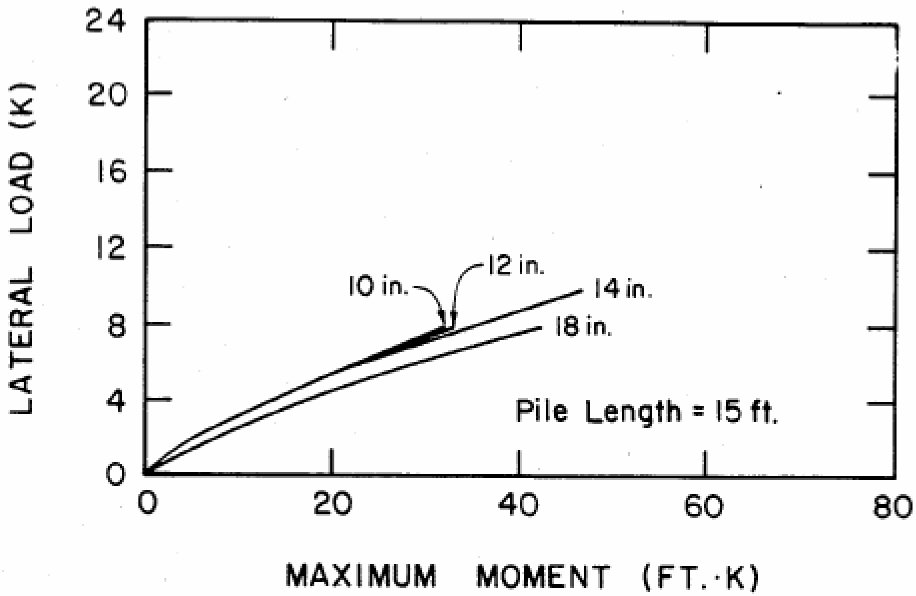


Figure 1.1.8-19 - Load Versus Deflection for Cast-in-Place Pile, Soil Profiles 6 and 7



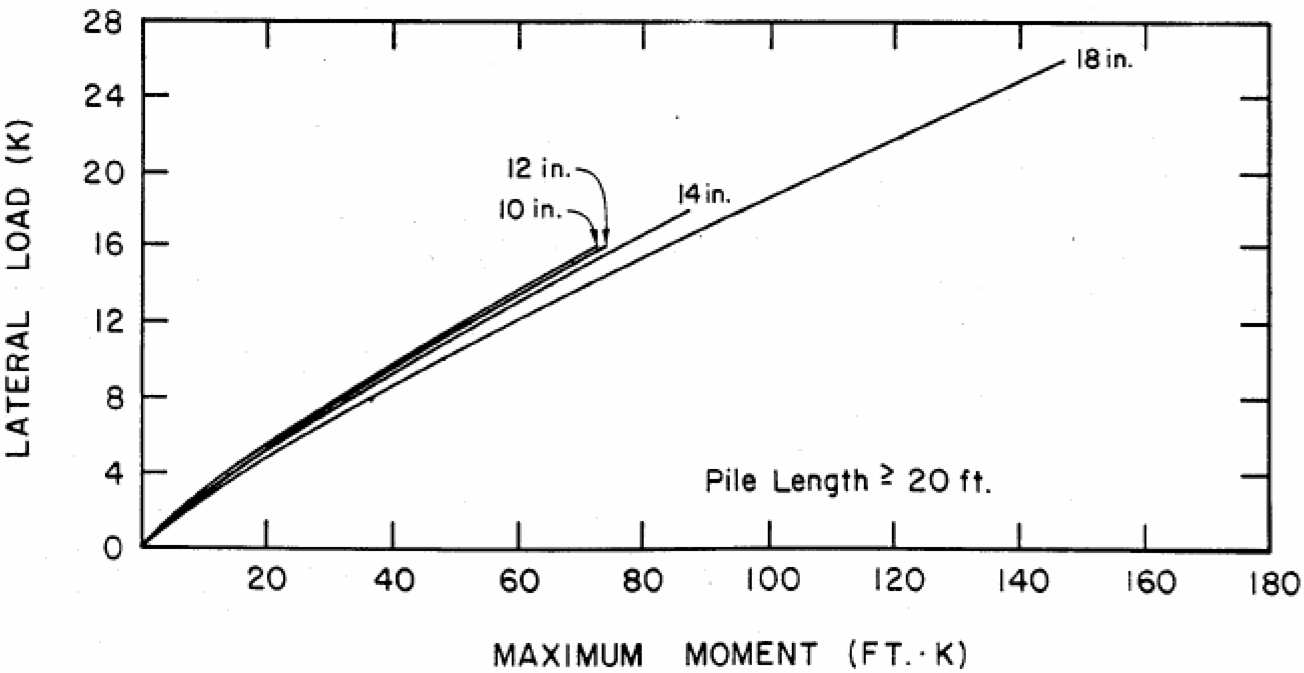


Figure 1.1.8-20 - Load Versus Maximum Moment for Cast-in-Place Pile, Soil Profile 6

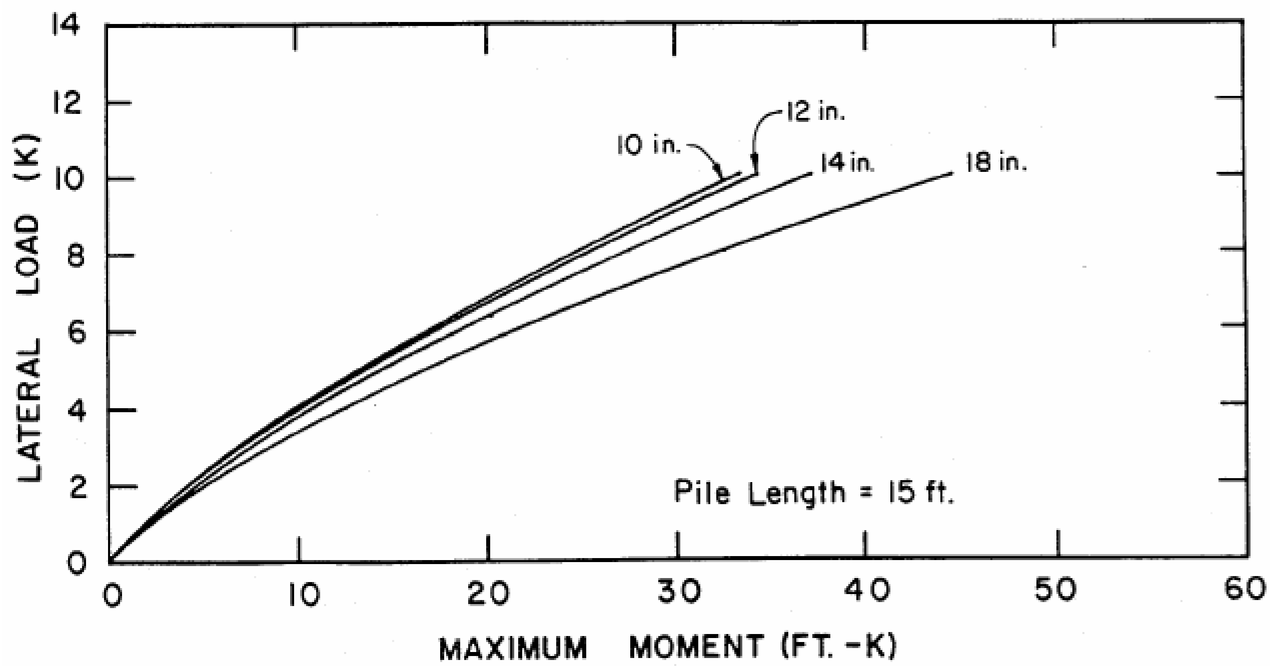


Figure 1.1.8-21 - Load Versus Maximum Moment for Cast-in-Place Pile, Soil Profile 7

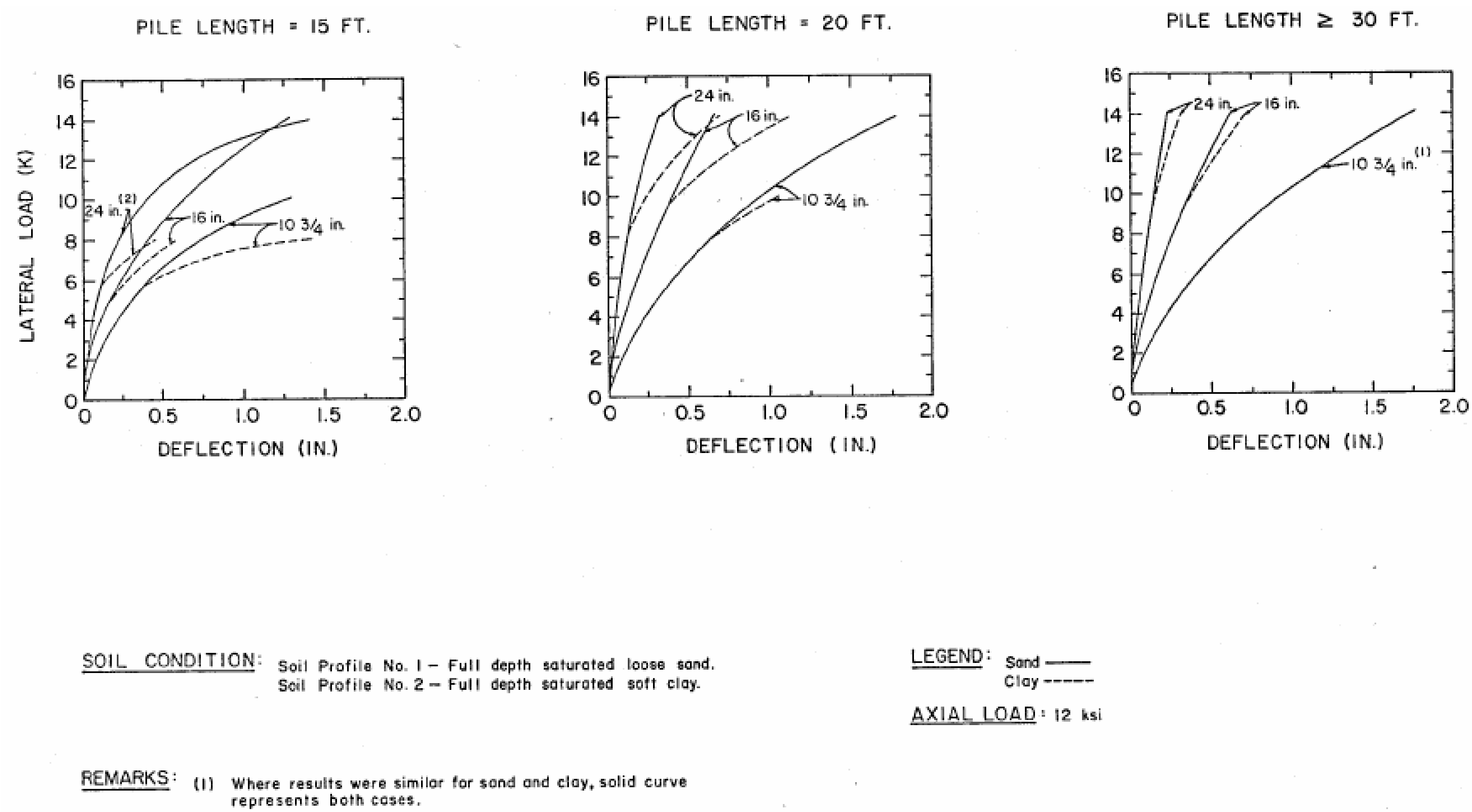


Figure 1.1.8-22 - Load Versus Deflection for Pipe Pile, Soil Profiles 1 and 3

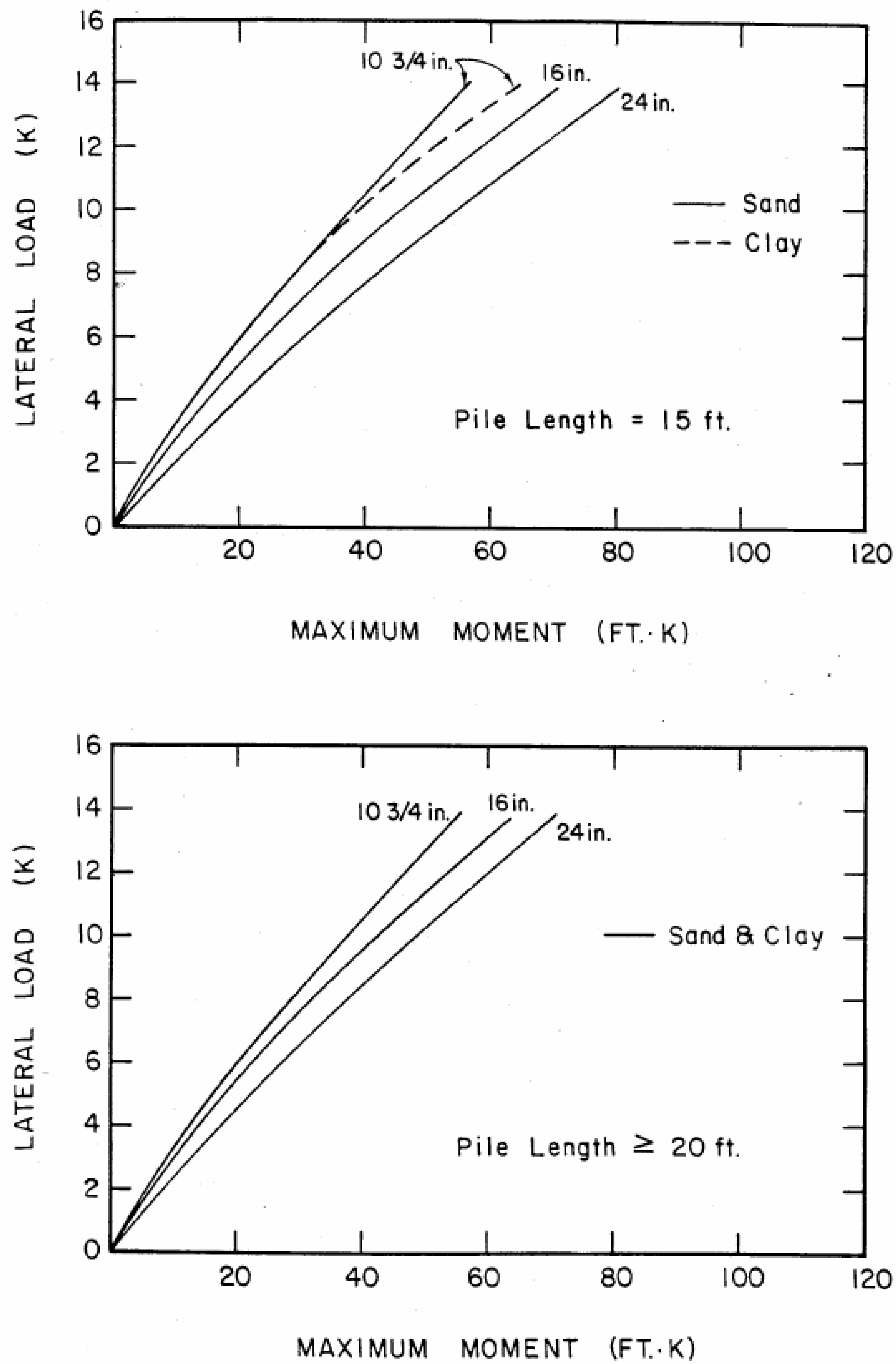


Figure 1.1.8-23 - Load Versus Maximum Moment for Pipe Pile, Soil Profiles 1 and 3

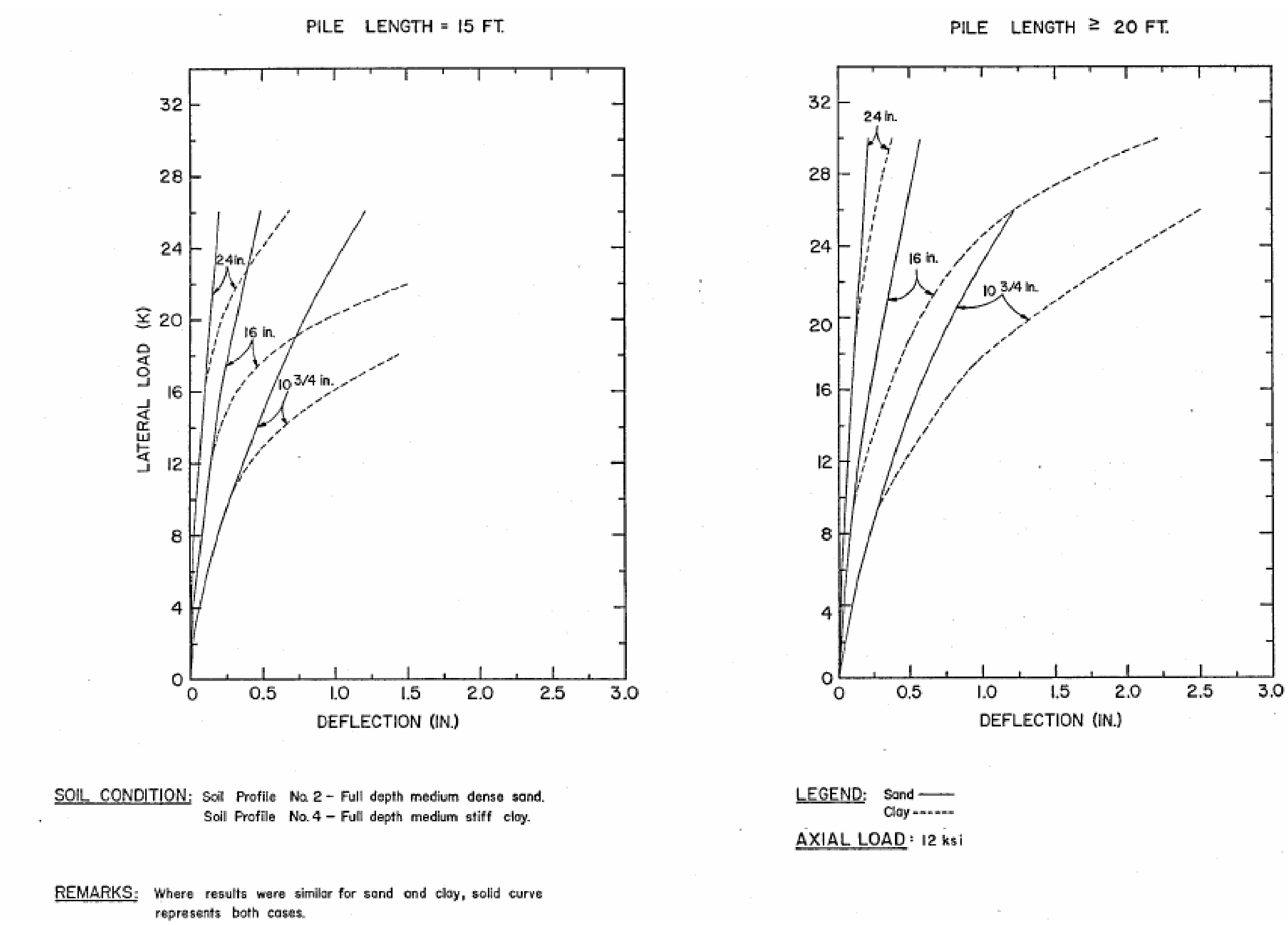


Figure 1.1.8-24 - Load Versus Deflection for Pipe Pile, Soil Profiles 2 and 4

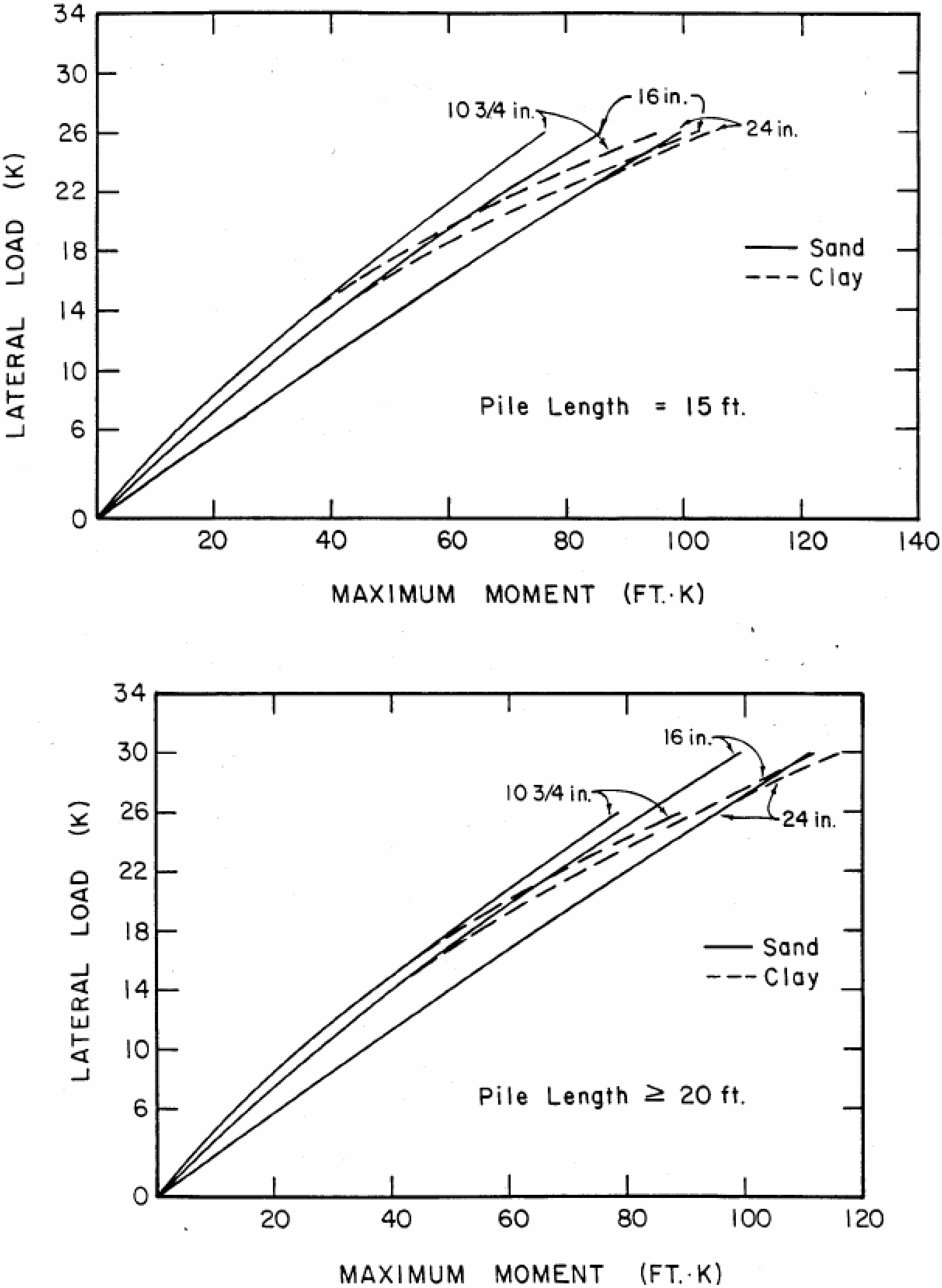


Figure 1.1.8-25 - Load Versus Maximum Moment for Pipe Pile, Soil Profiles 2 and 4

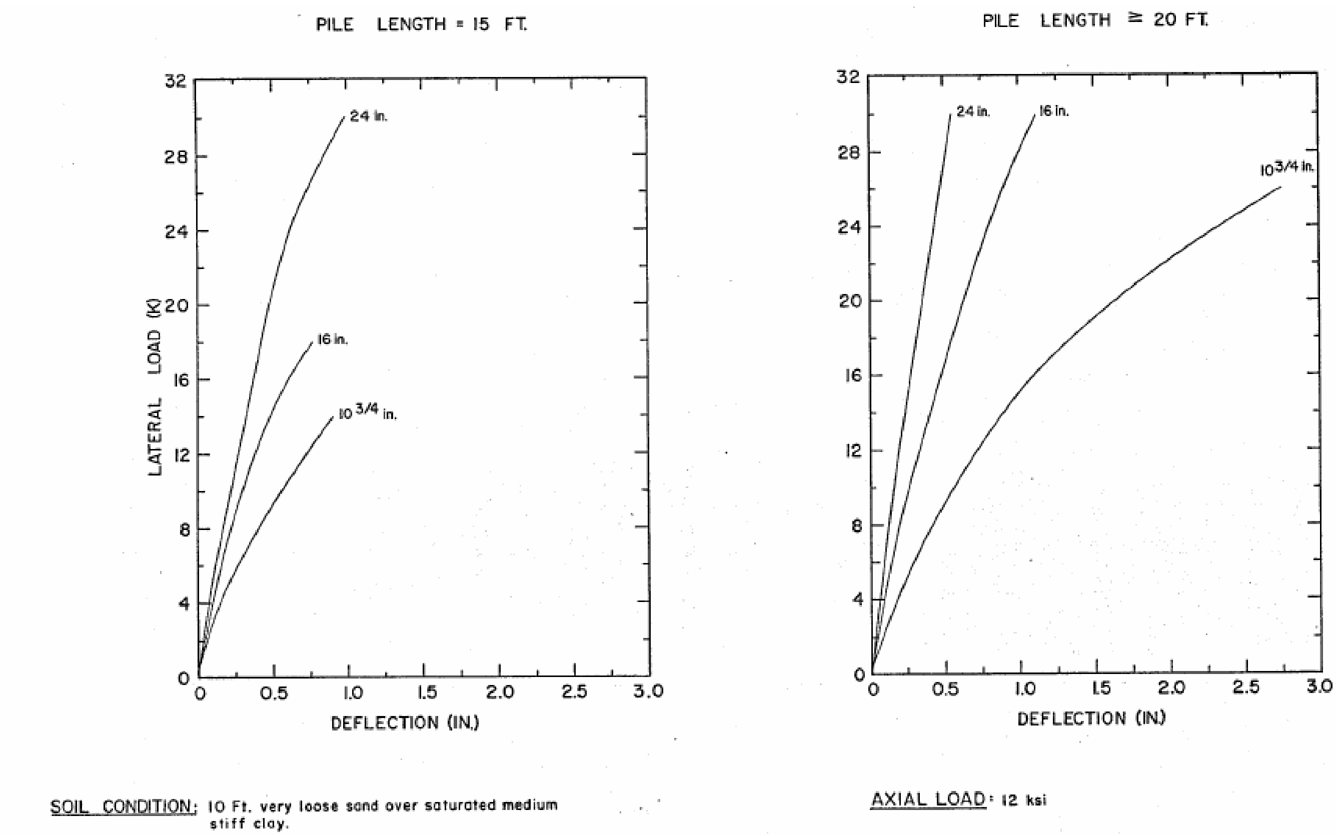


Figure 1.1.8-26 - Load Versus Deflection for Pipe Pile, Soil Profile 5

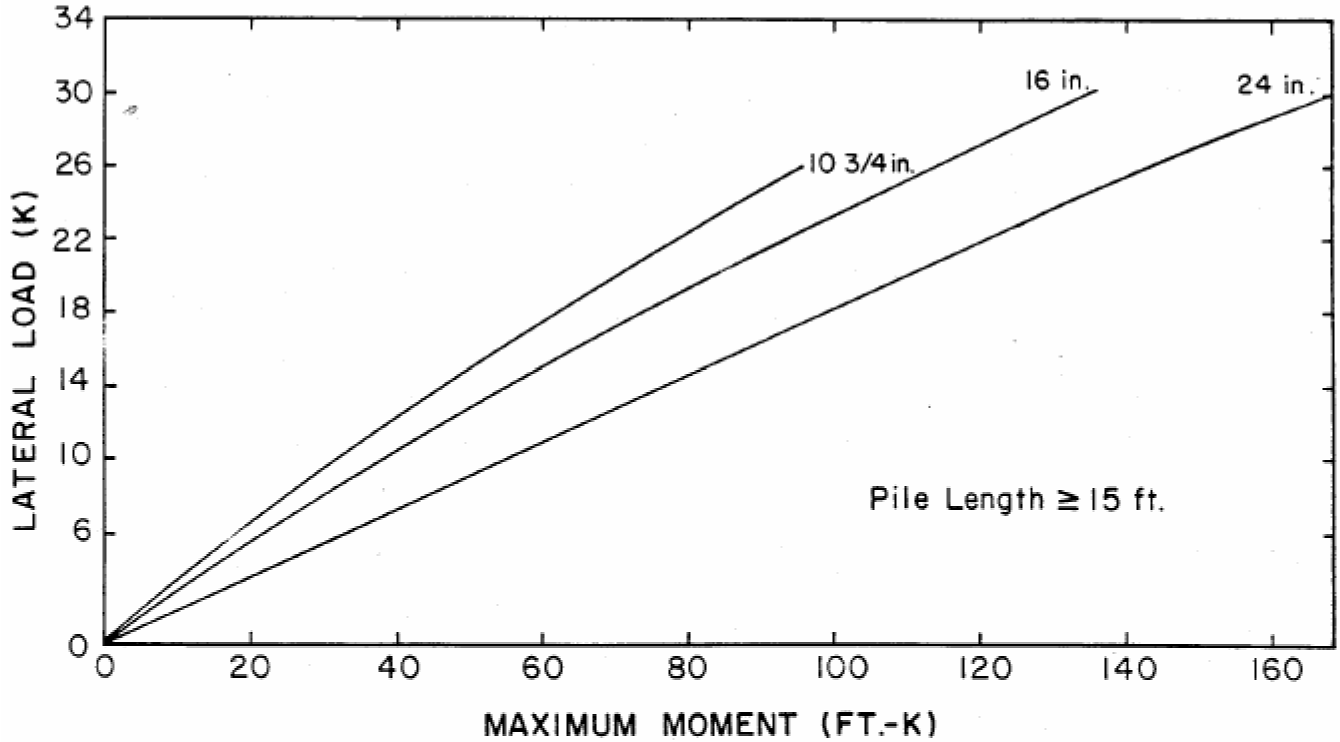


Figure 1.1.8-27 - Load Versus Maximum Moment for Pipe Pile, Soil Profile 5

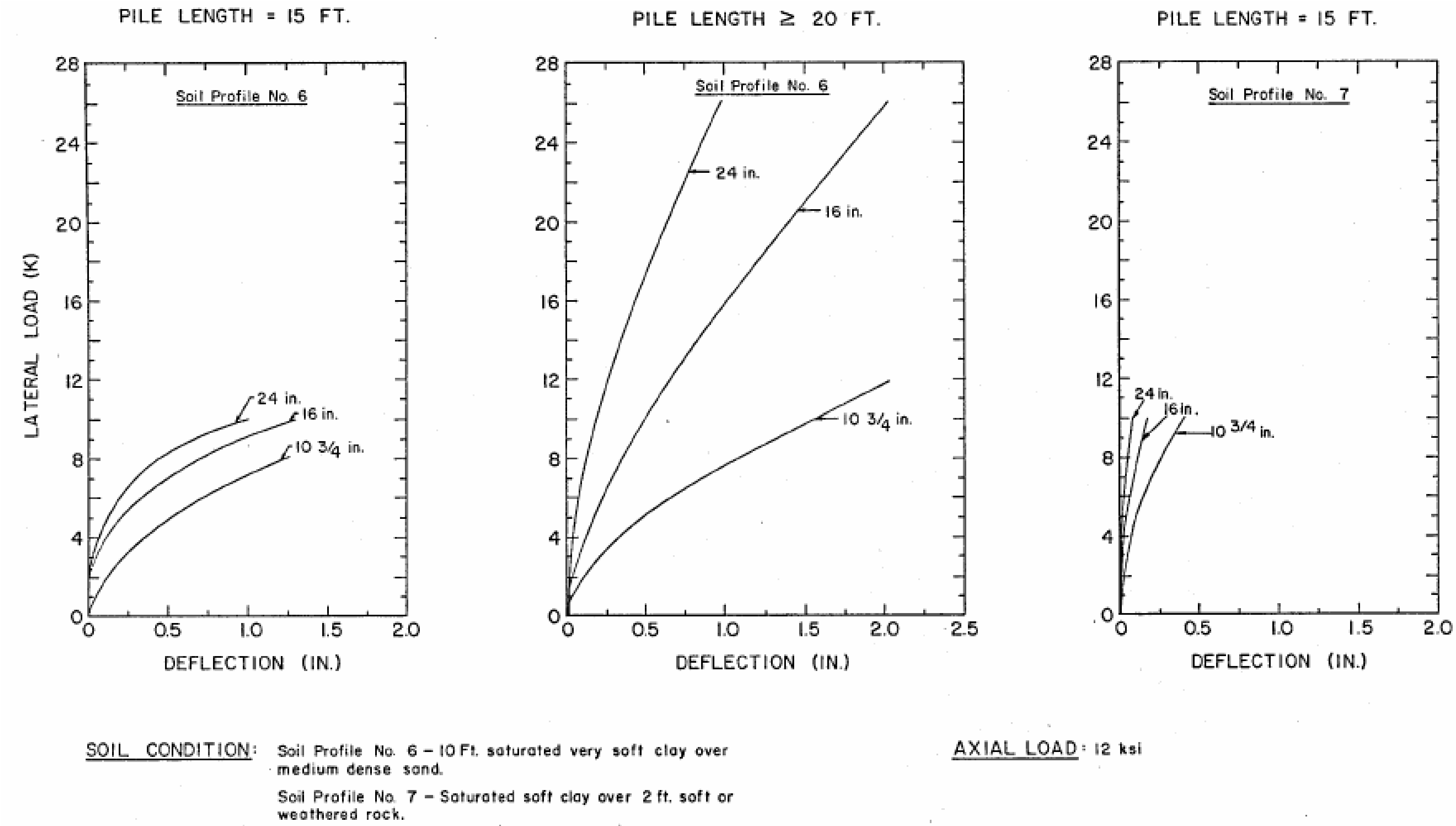


Figure 1.1.8-28 - Load Versus Deflection for Pipe Pile, Soil Profiles 6 and 7

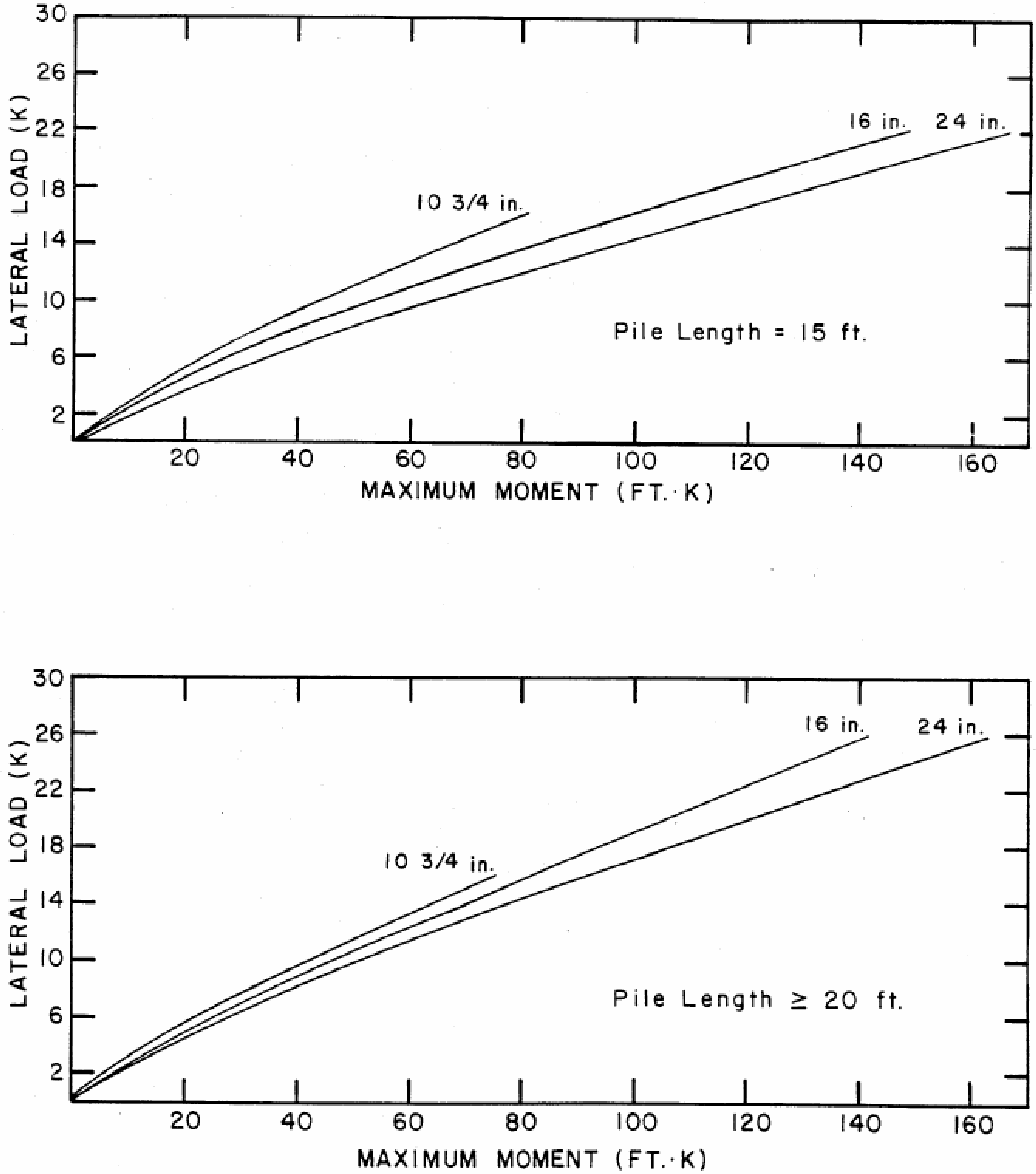


Figure 1.1.8-29 - Load Versus Maximum Moment for Pipe Pile, Soil Profile 6

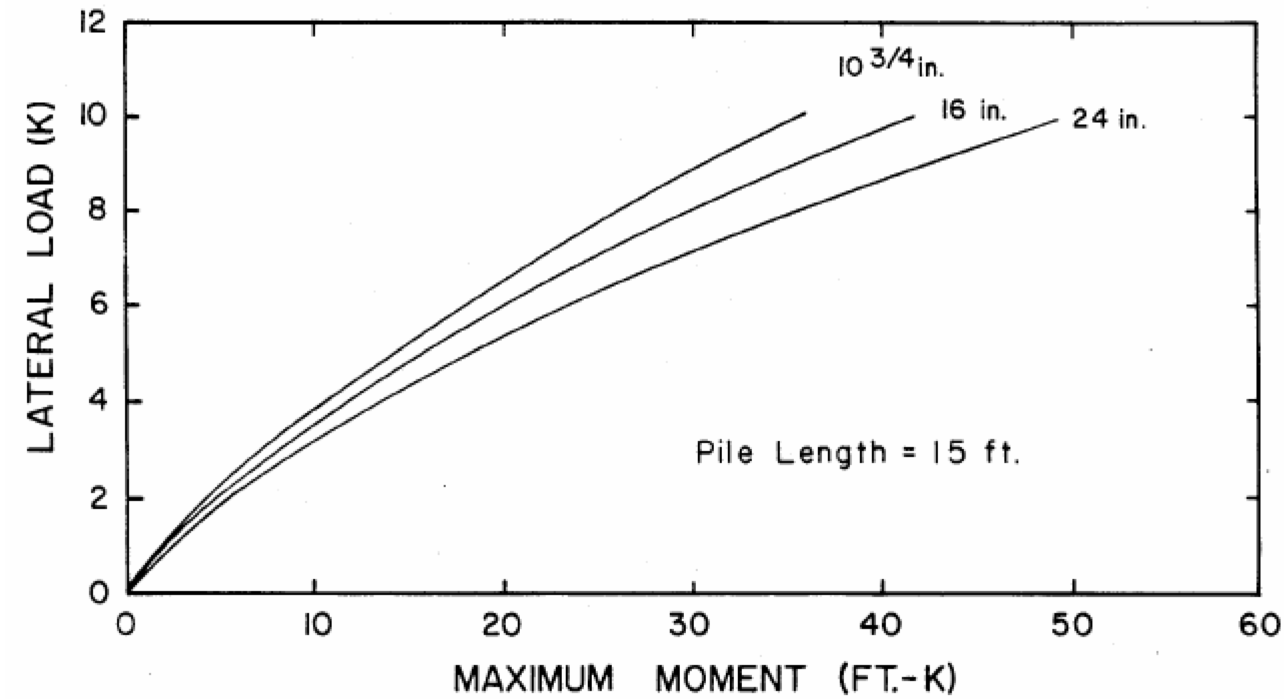
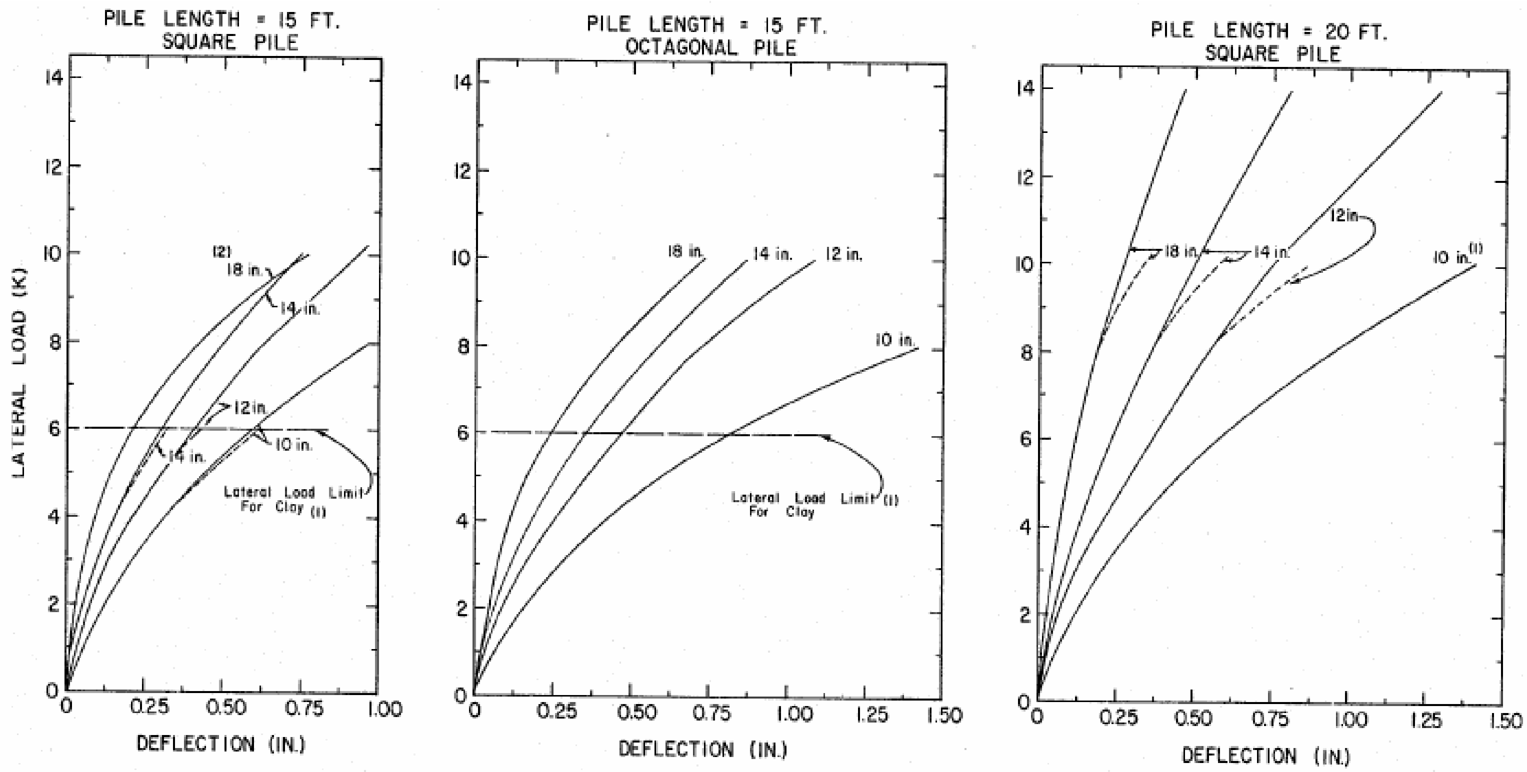


Figure 1.1.8-30 - Load Versus Maximum Moment for Pipe Pile, Soil Profile 7



SOIL CONDITION: Soil Profile No. 1 – Full depth saturated loose sand.
Soil Profile No. 3 – Full depth saturated soft clay.

LEGEND: Sand ——
Clay -----

REMARKS: (1) Where results were similar for sand and clay, solid curve represents both cases. Maximum lateral load for piles in clay is as noted.

AXIAL LOAD: 0.30 f'c

Figure 1.1.8-31 - Load Versus Deflection for Precast Concrete Pile, Soil Profiles 1 and 3

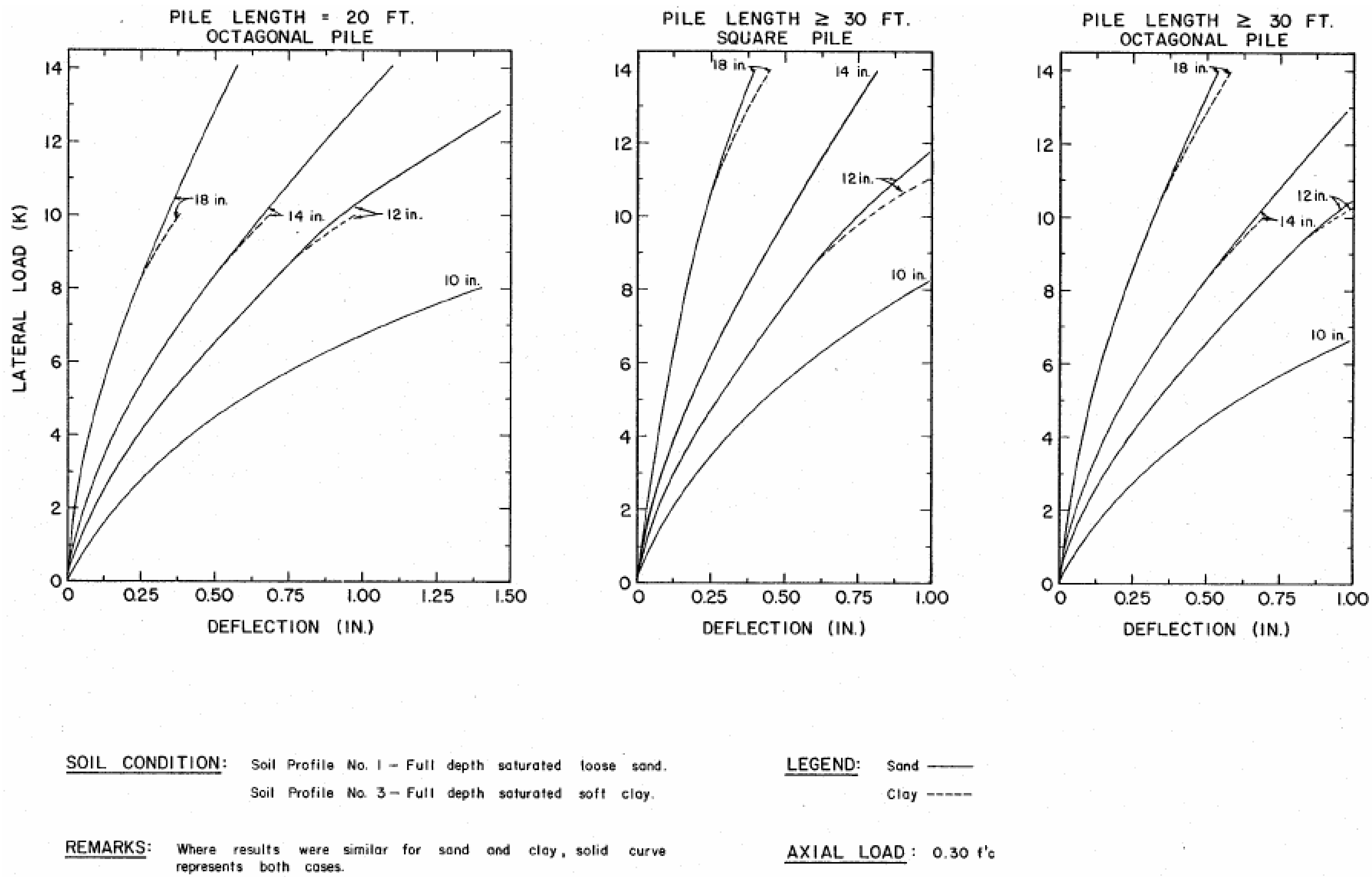


Figure 1.1.8-32 - Load Versus Deflection for Precast Concrete Pile, Soil Profiles 1 and 3

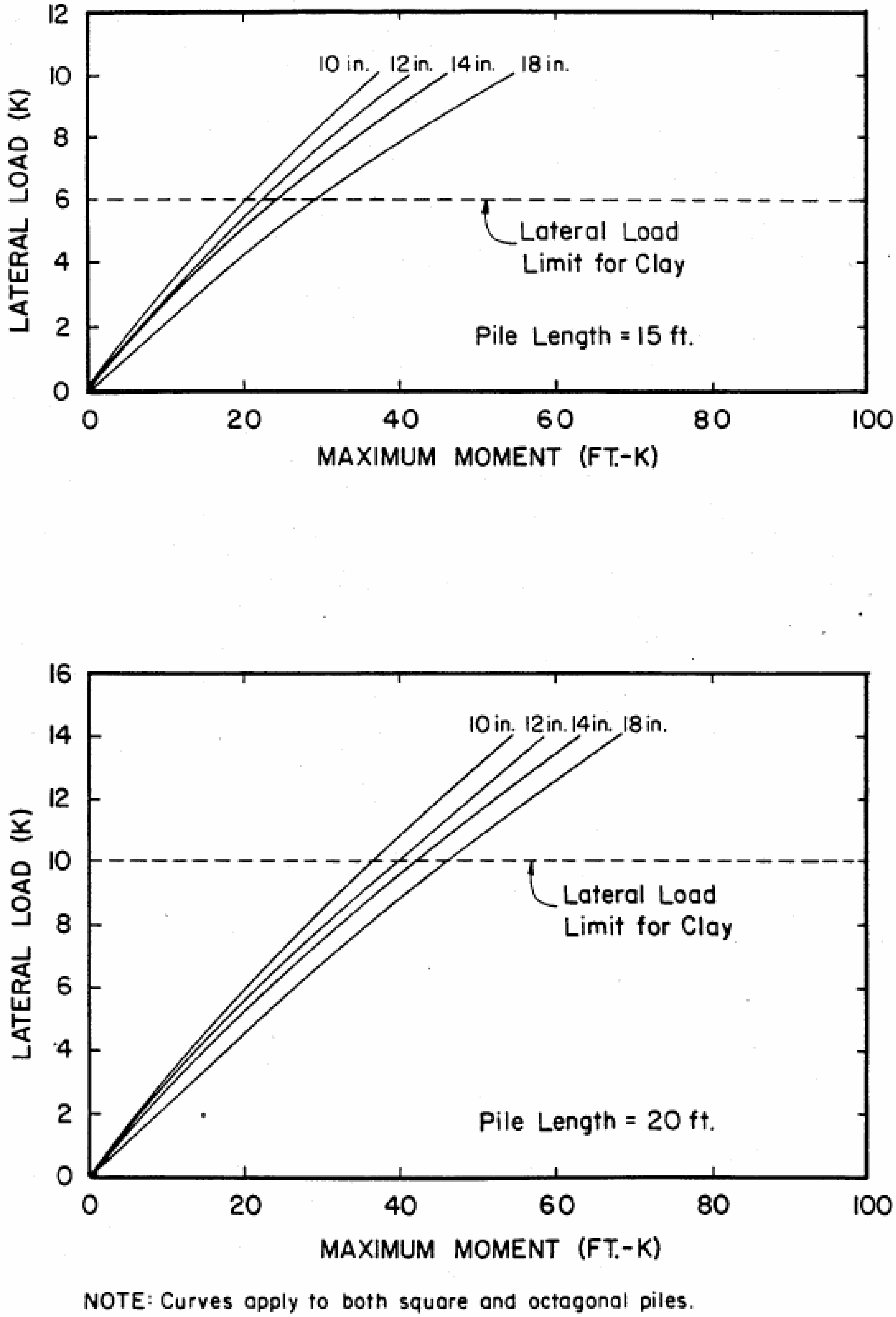


Figure 1.1.8-33 - Load Versus Maximum Moment for Precast Concrete Pile, Soil Profiles 1 and 3

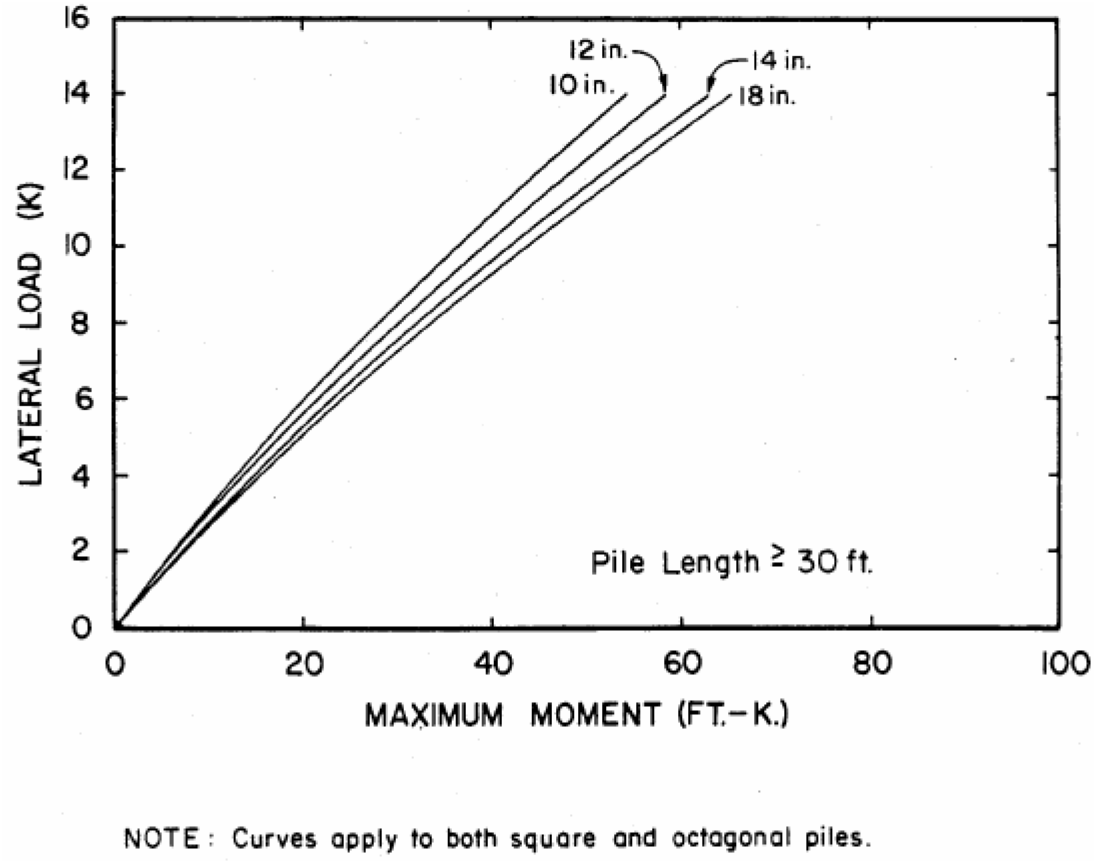


Figure 1.1.8-34 - Load Versus Maximum Moment for Precast Concrete Pile, Soil Profiles 1 and 3

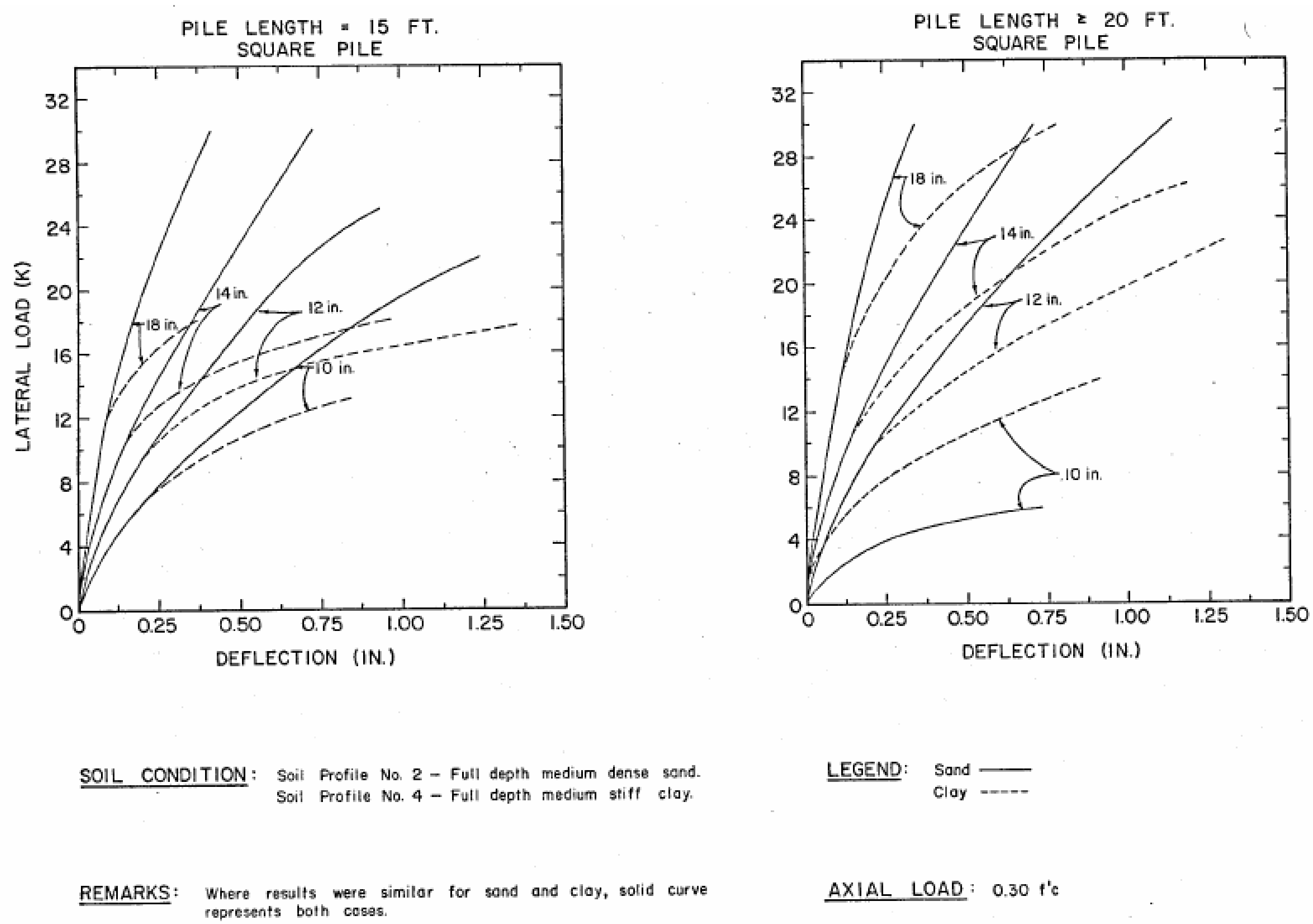
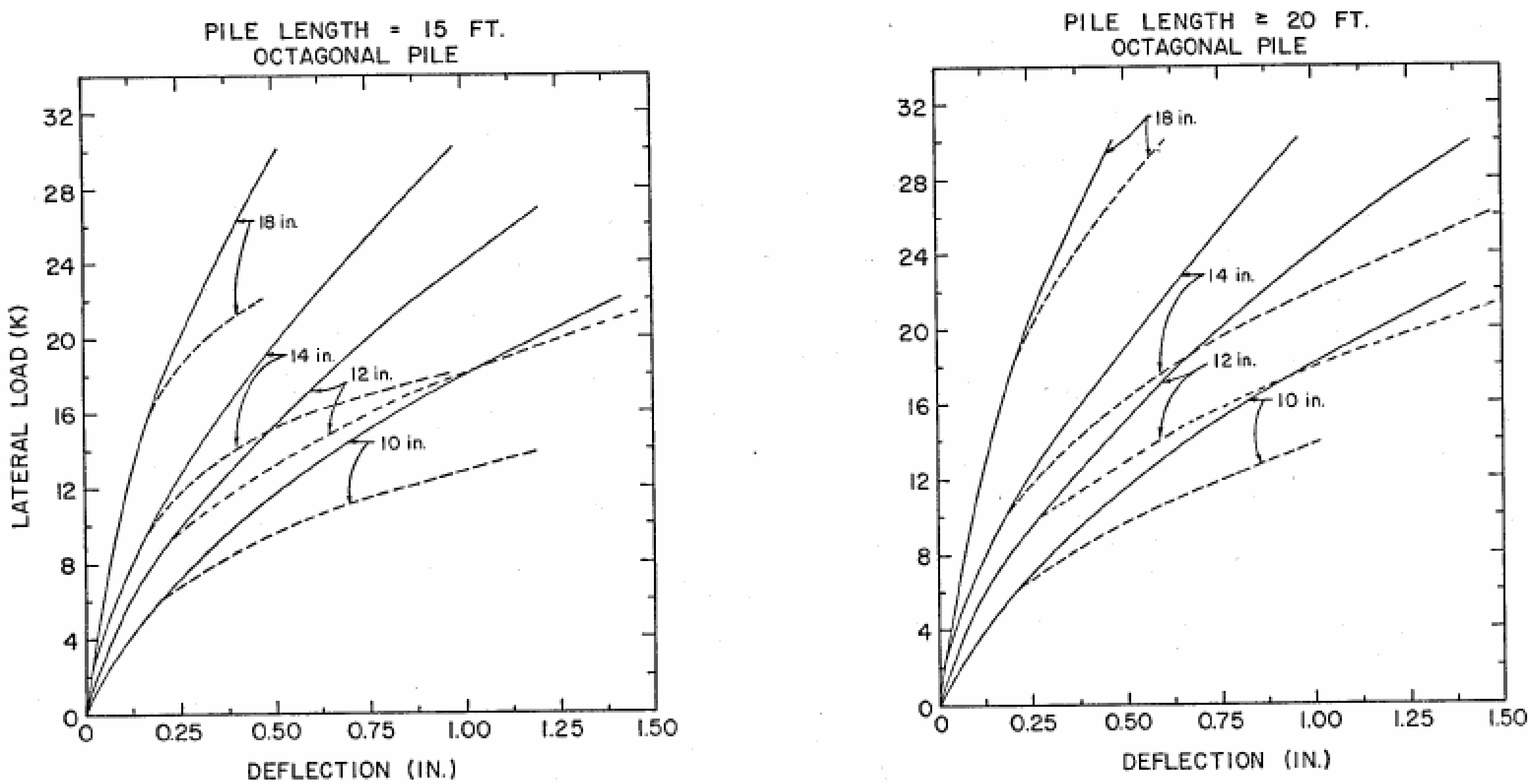


Figure 1.1.8-35 - Load Versus Deflection for Precast Concrete Pile, Soil Profiles 2 and 4



SOIL CONDITION: Soil Profile No. 2 – Full depth medium dense sand.
Soil Profile No. 4 – Full depth medium stiff clay.

LEGEND: Sand ——— Clay -----

REMARKS: Where results were similar for sand and clay, solid curve represents both cases.

AXIAL LOAL: 0.30 f'c

Figure 1.1.8-36 - Load Versus Deflection for Precast Concrete Pile, Soil Profiles 2 and 4

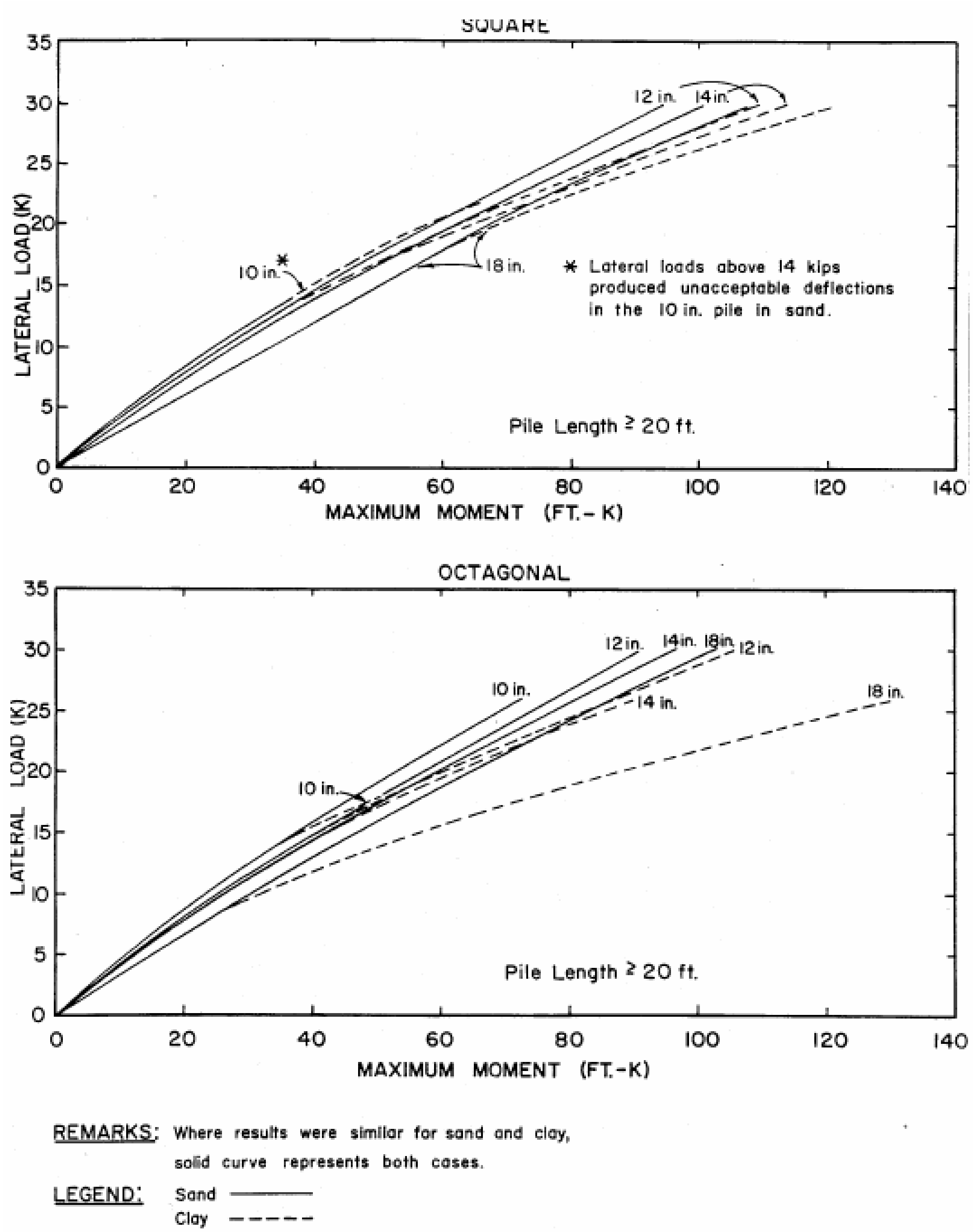


Figure 1.1.8-37 - Load Versus Maximum Moment for Precast Concrete Pile, Soil Profiles 2 and 4

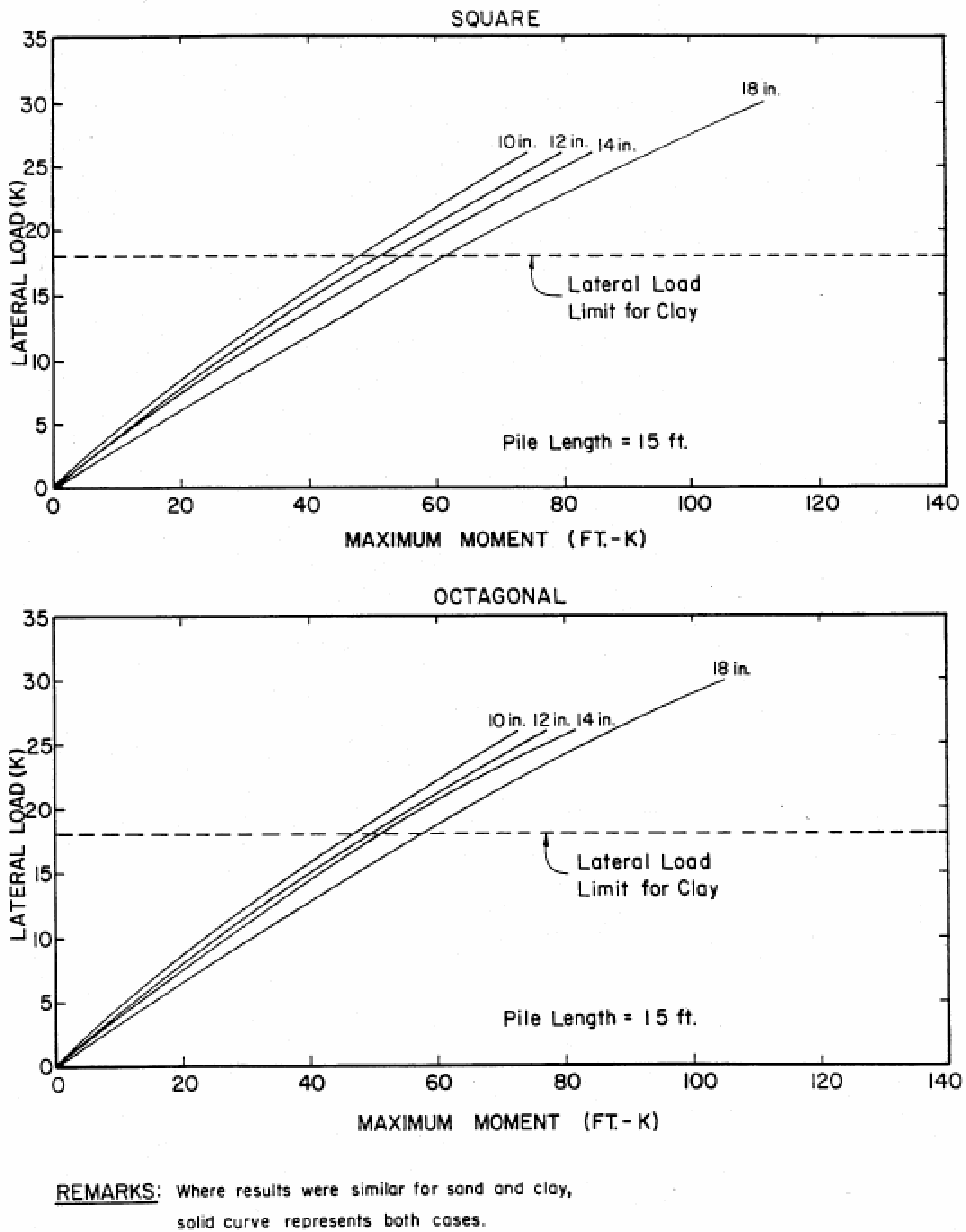


Figure 1.1.8-38 - Load Versus Maximum Moment for Precast Concrete Pile, Soil Profiles 2 and 4

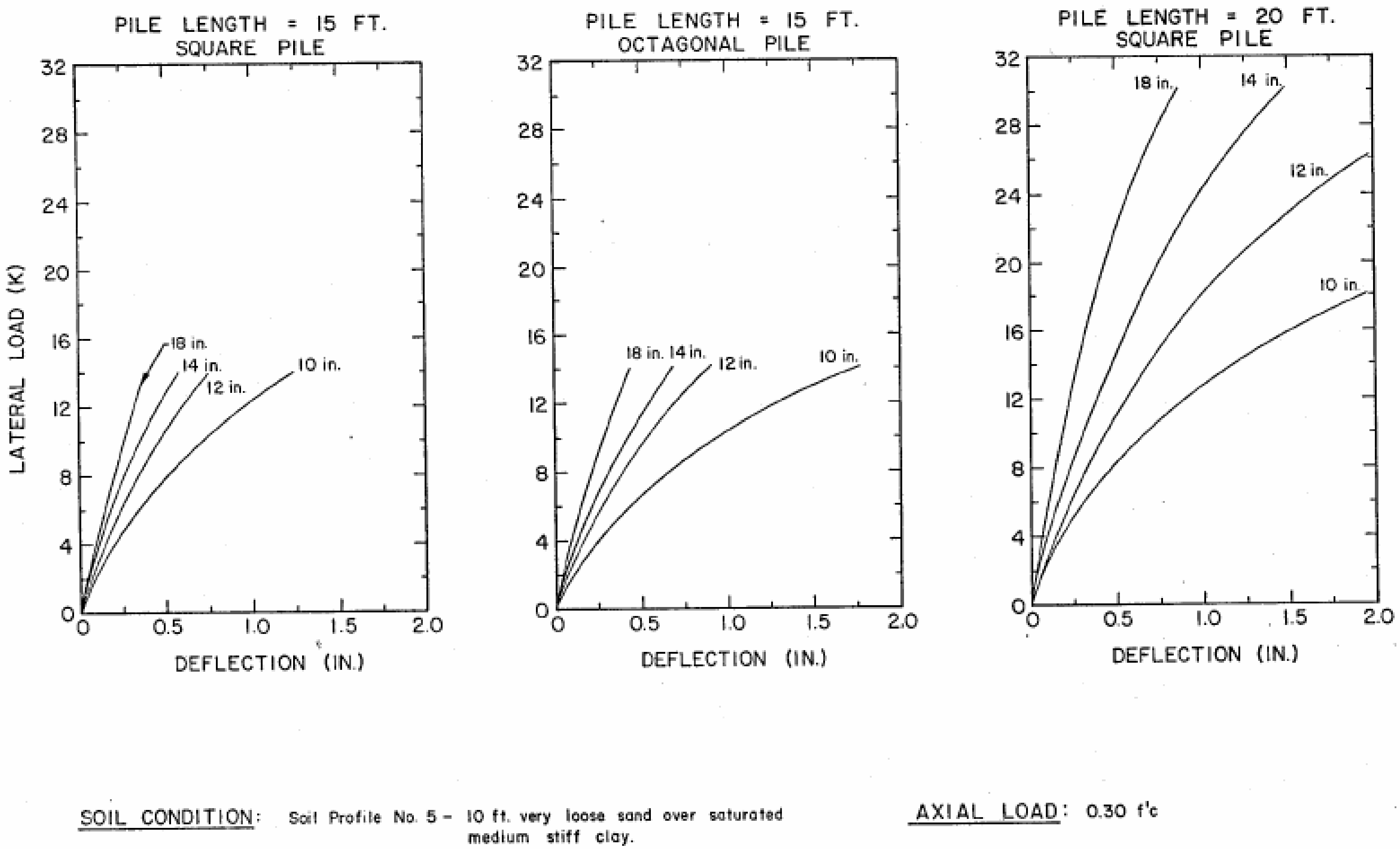


Figure 1.1.8-39 - Load Versus Deflection for Precast Concrete Pile, Soil Profile 5

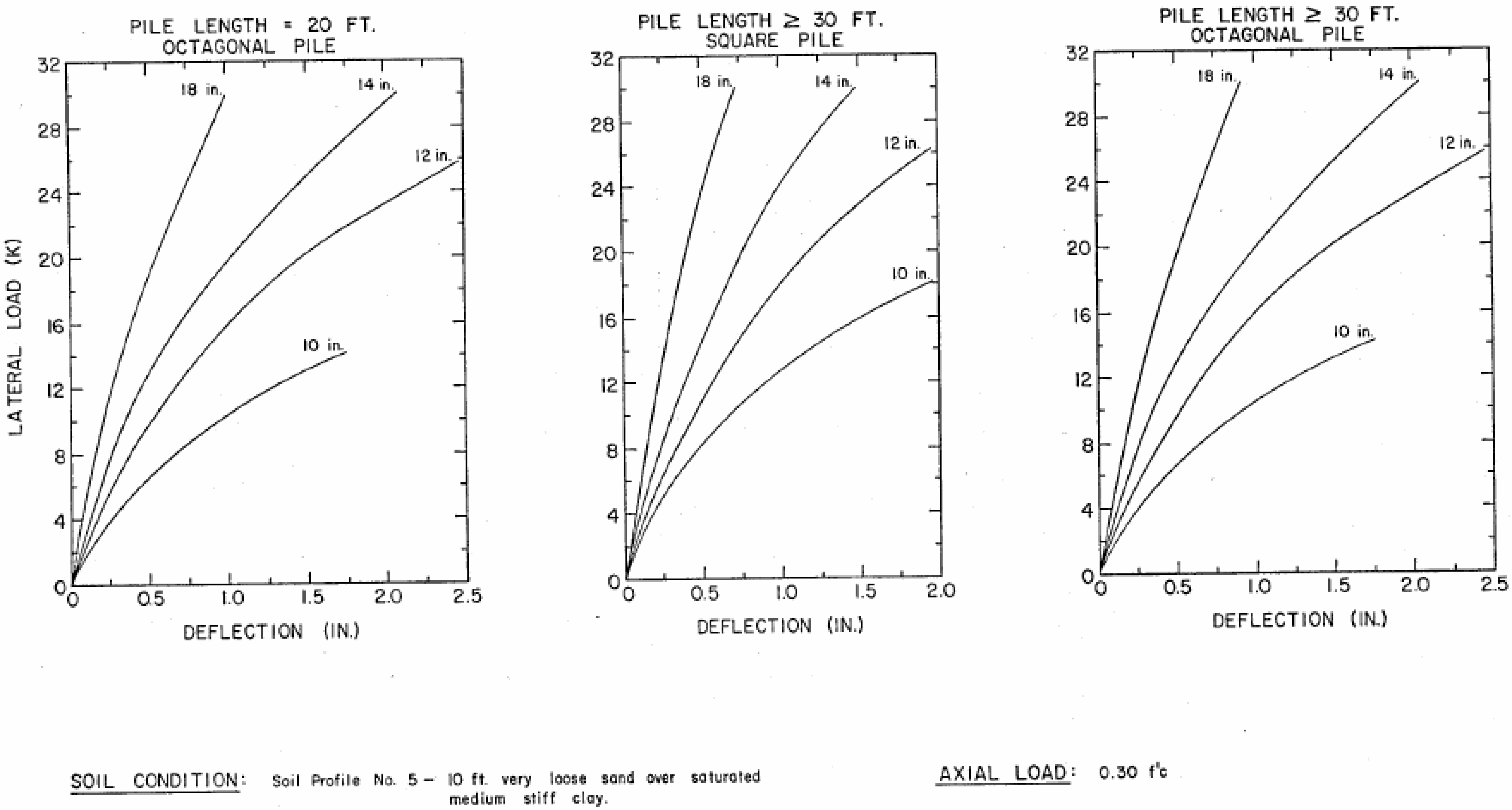


Figure 1.1.8-40 - Load Versus Deflection for Precast Concrete Pile, Soil Profile 5

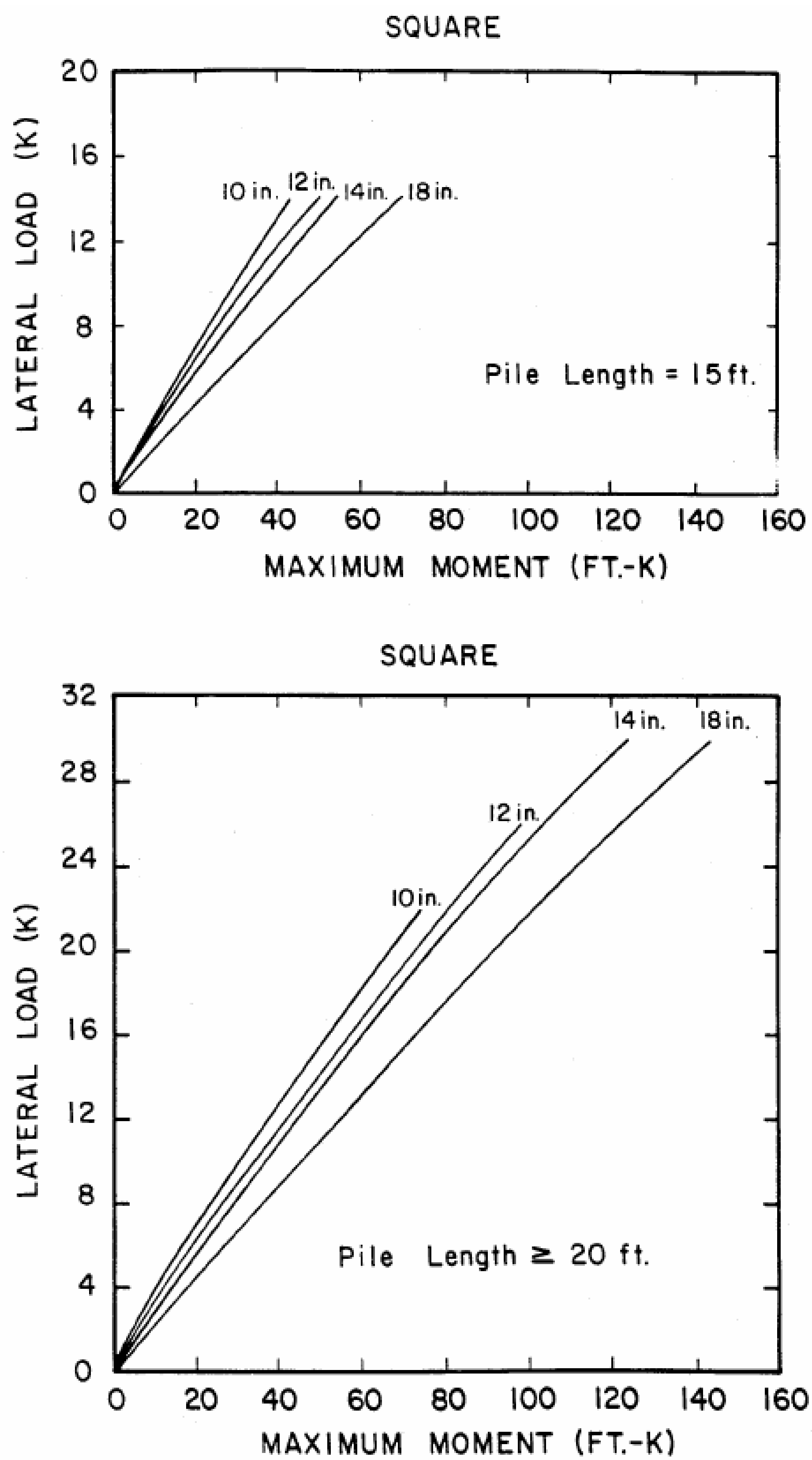
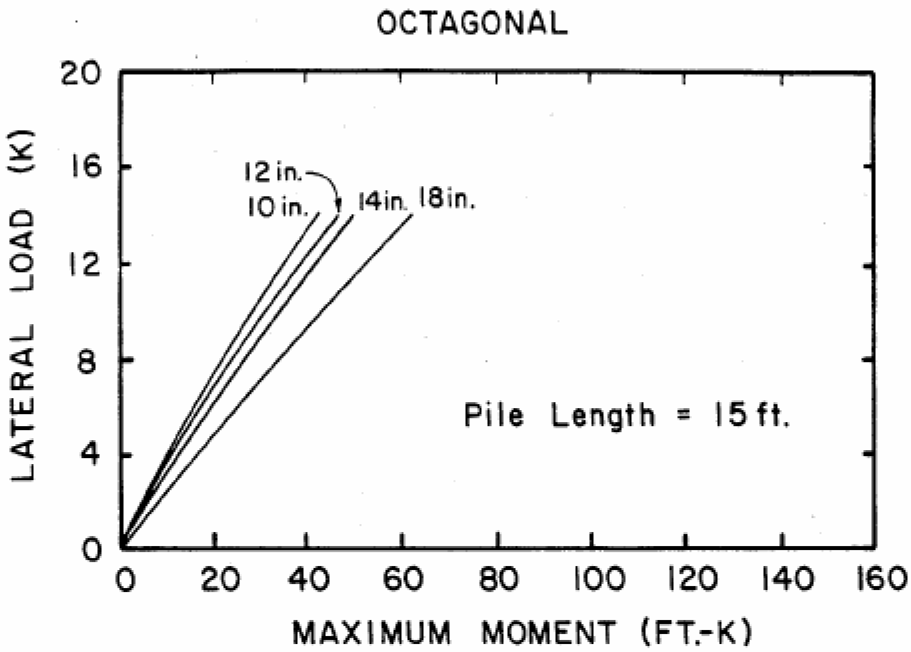


Figure 1.1.8-41 - Load Versus Maximum Moment for Precast Concrete Pile, Soil Profile 5



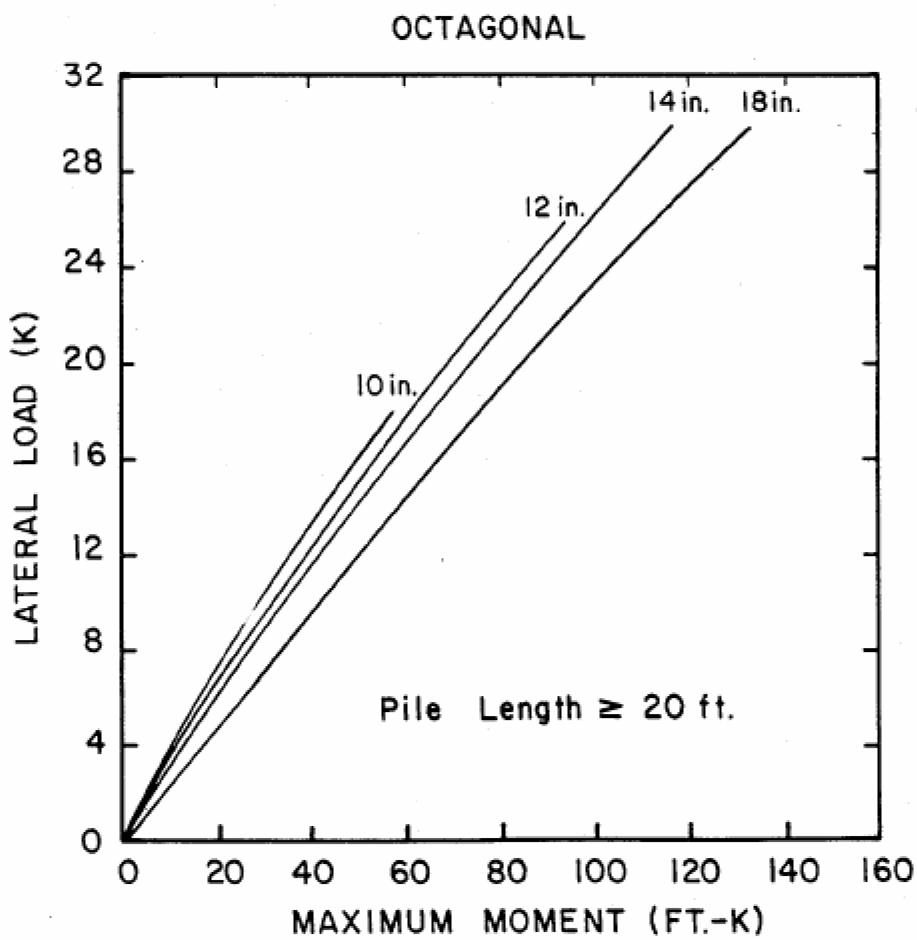


Figure 1.1.8-42 - Load Versus Maximum Moment for Precast Concrete Pile, Soil Profile 5

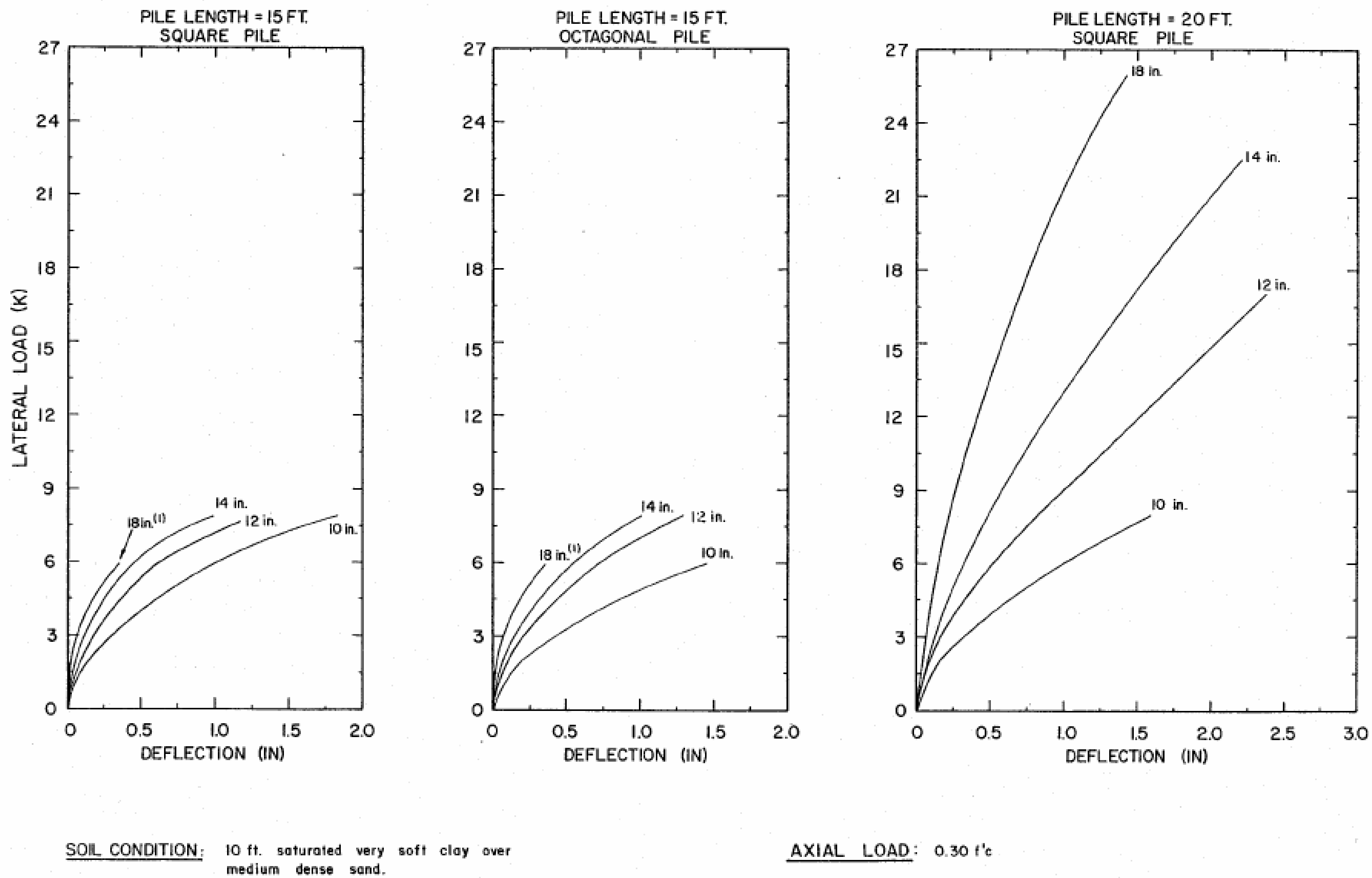


Figure 1.1.8-43 - Load Versus Deflection for Precast Concrete Pile, Soil Profile 6

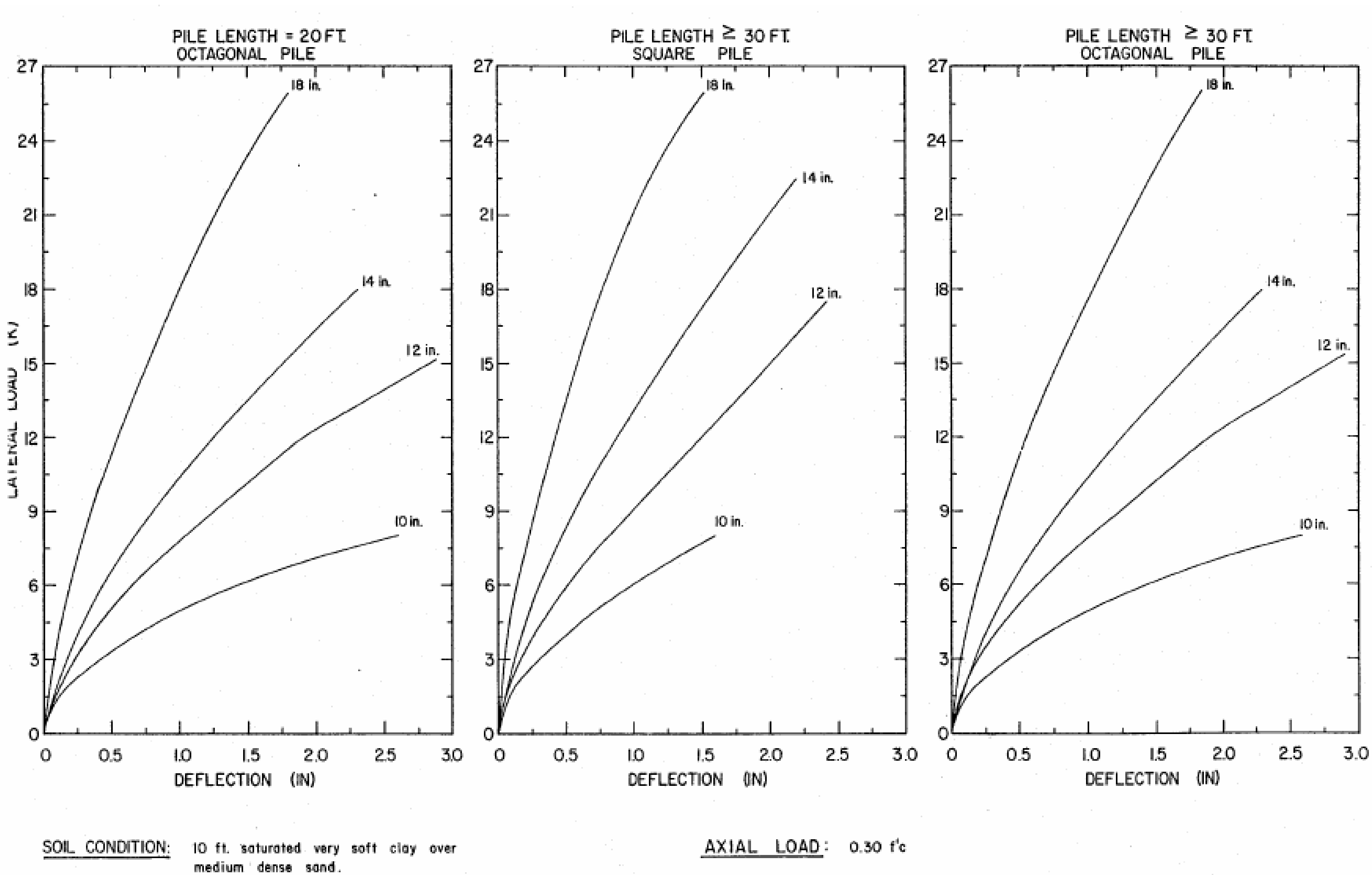
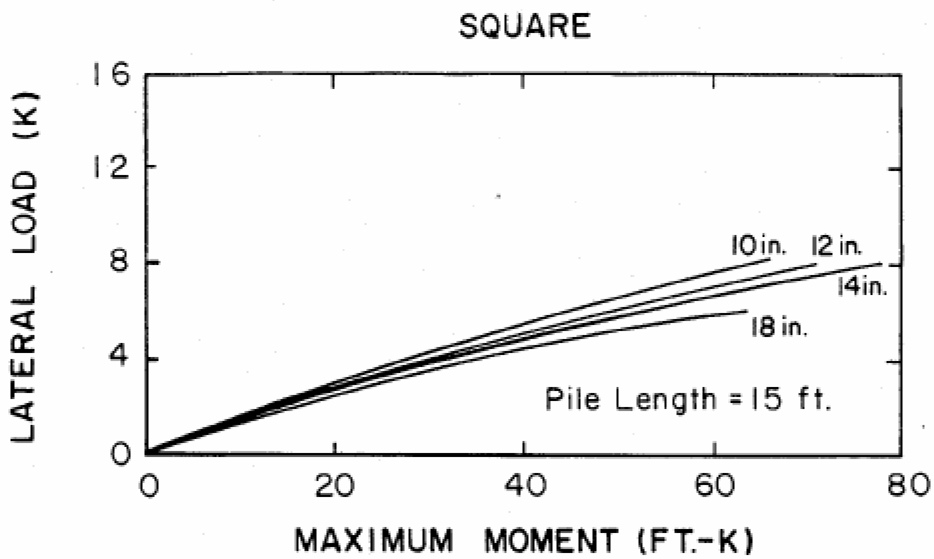


Figure 1.1.8-44 - Load Versus Deflection for Precast Concrete Pile, Soil Profile 6



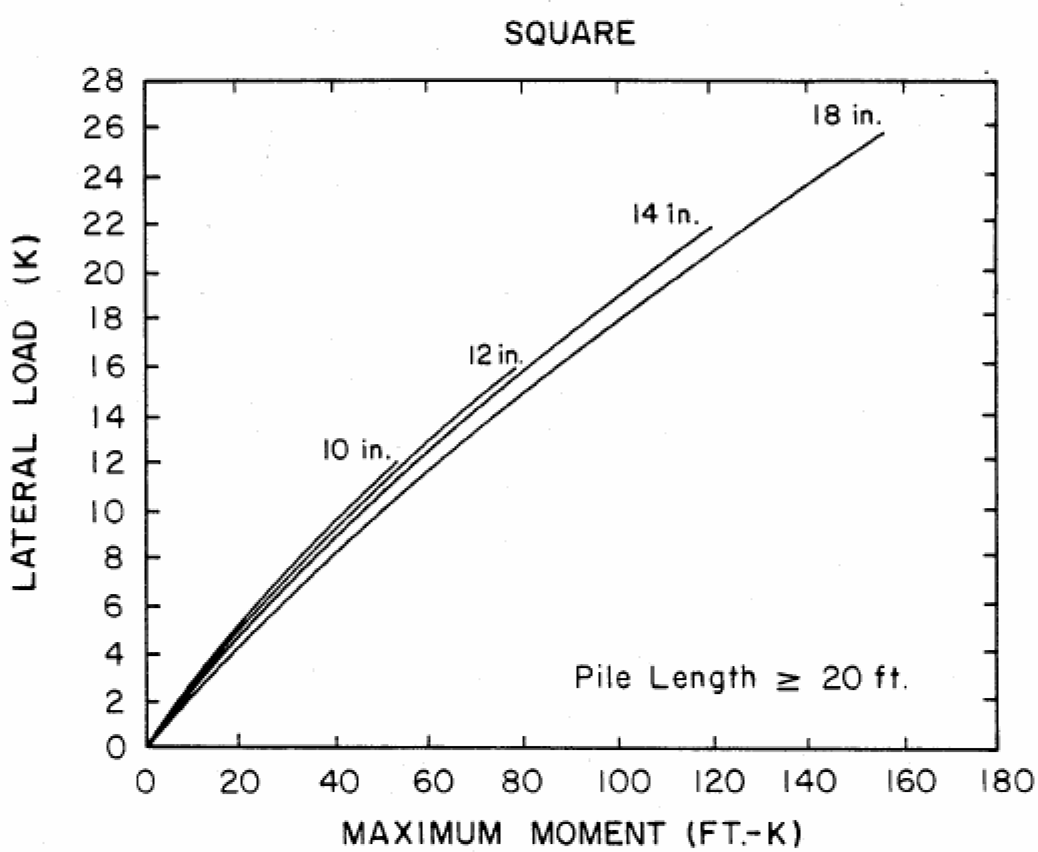


Figure 1.1.8-45 - Load Versus Maximum Moment for Precast Concrete Pile, Soil Profile 6

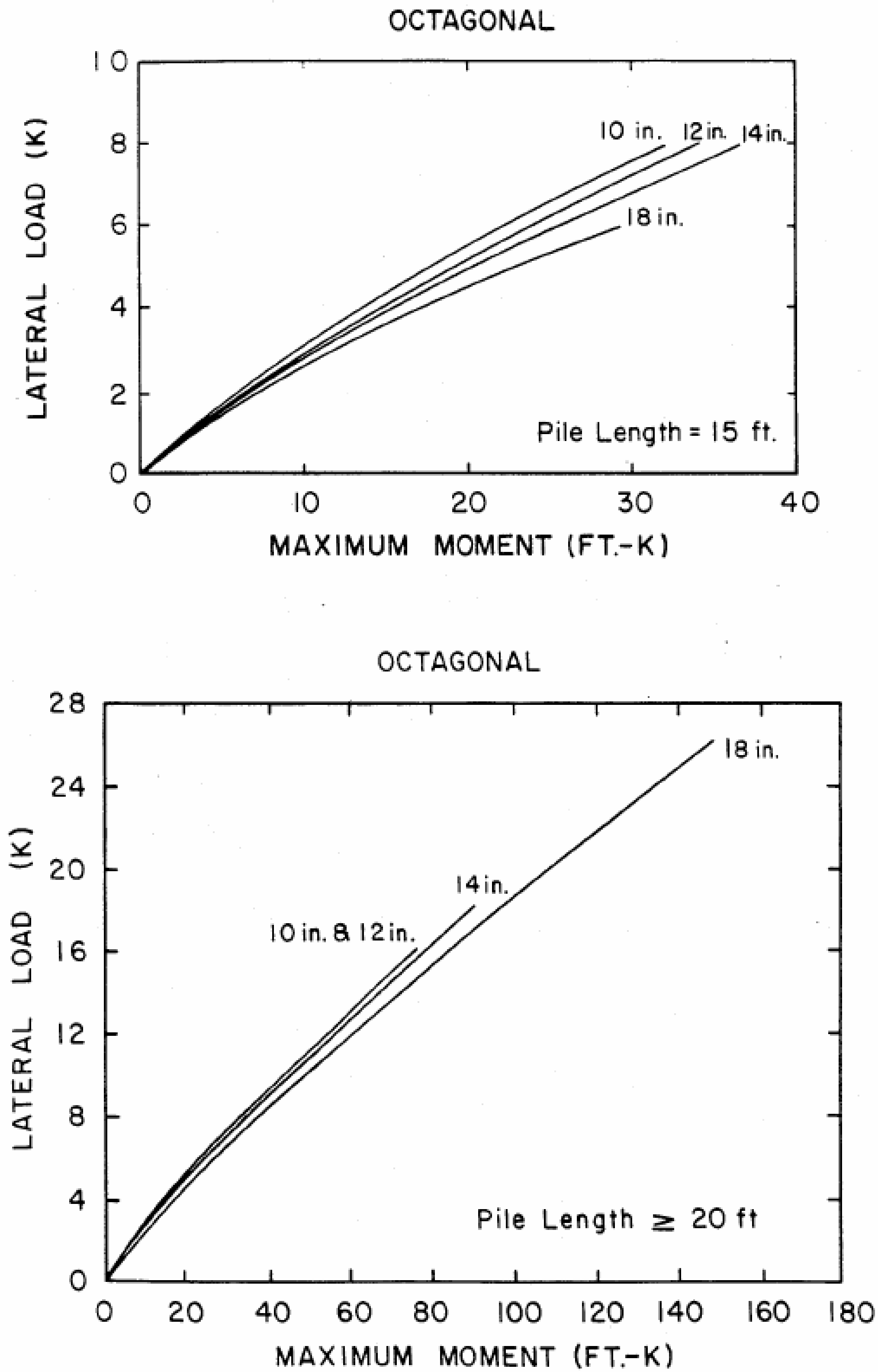
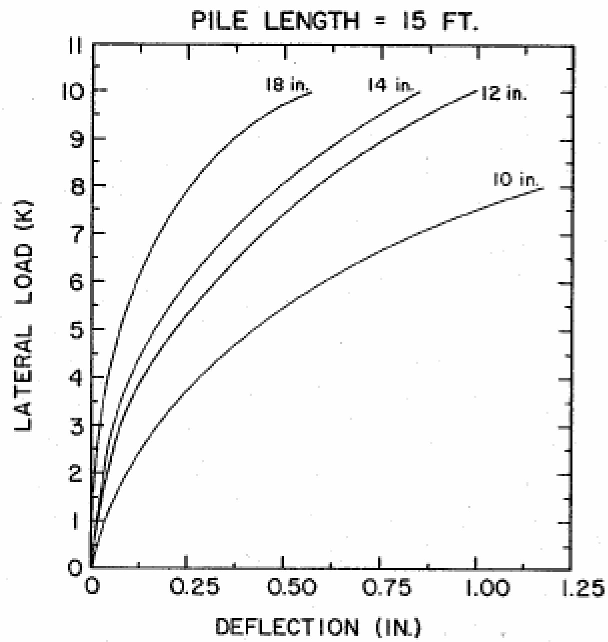


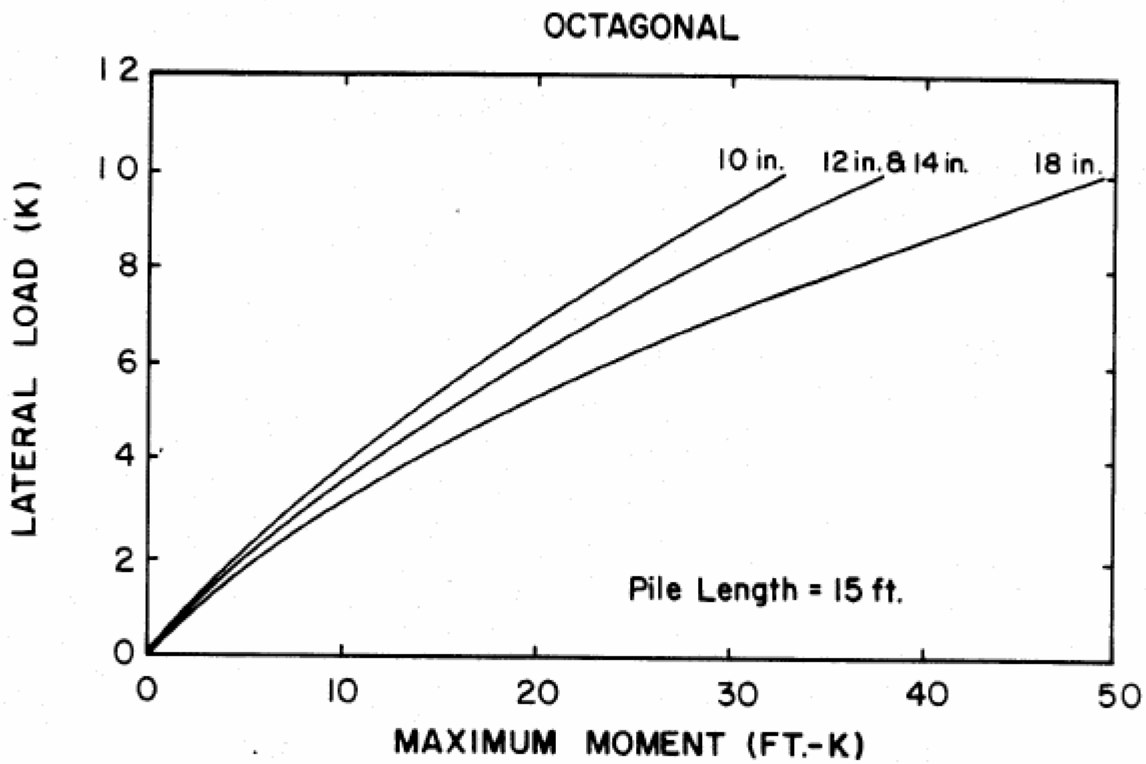
Figure 1.1.8-46 - Load Versus Maximum Moment for Precast Concrete Pile, Soil Profile 6



SOIL CONDITION: Soil Profile No. 7 — Saturated soft clay over 2 ft. soft or weathered rock.

REMARKS: Curves apply to both square and octagonal piles.

Figure 1.1.8-47 - Load Versus Deflection for Precast Concrete Pile, Soil Profile 7



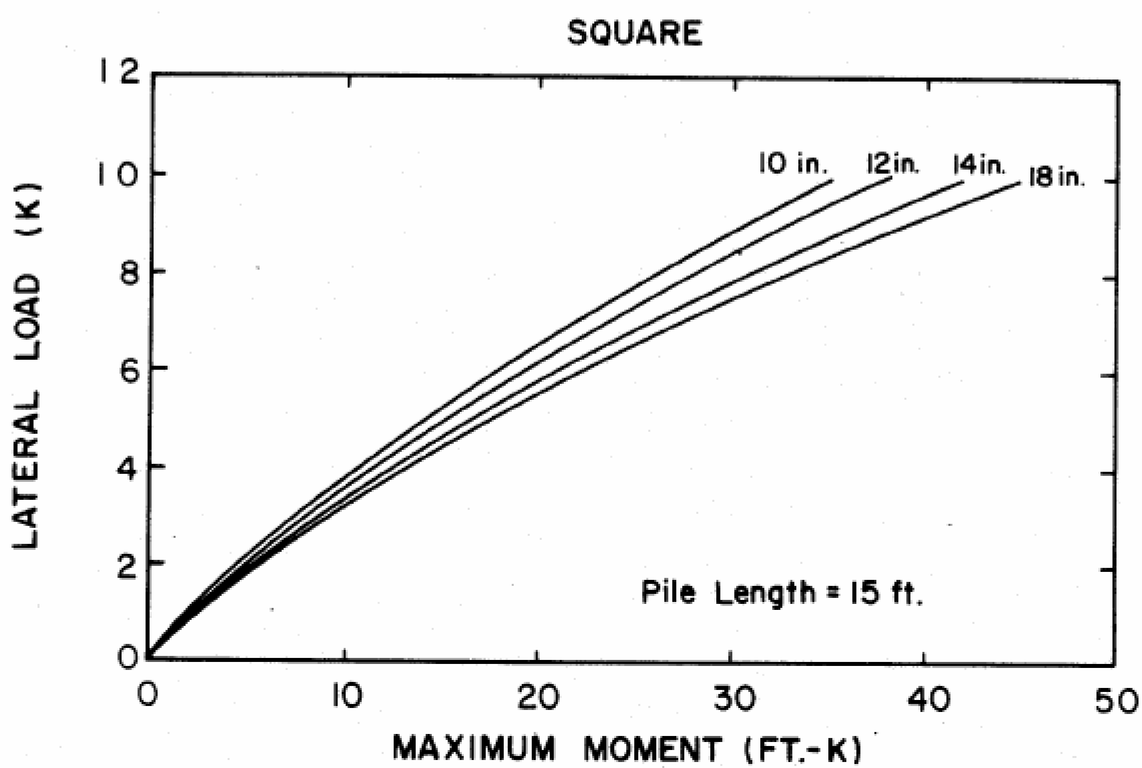


Figure 1.1.8-48 - Load Versus Maximum Moment for Precast Concrete Pile, Soil Profile 7

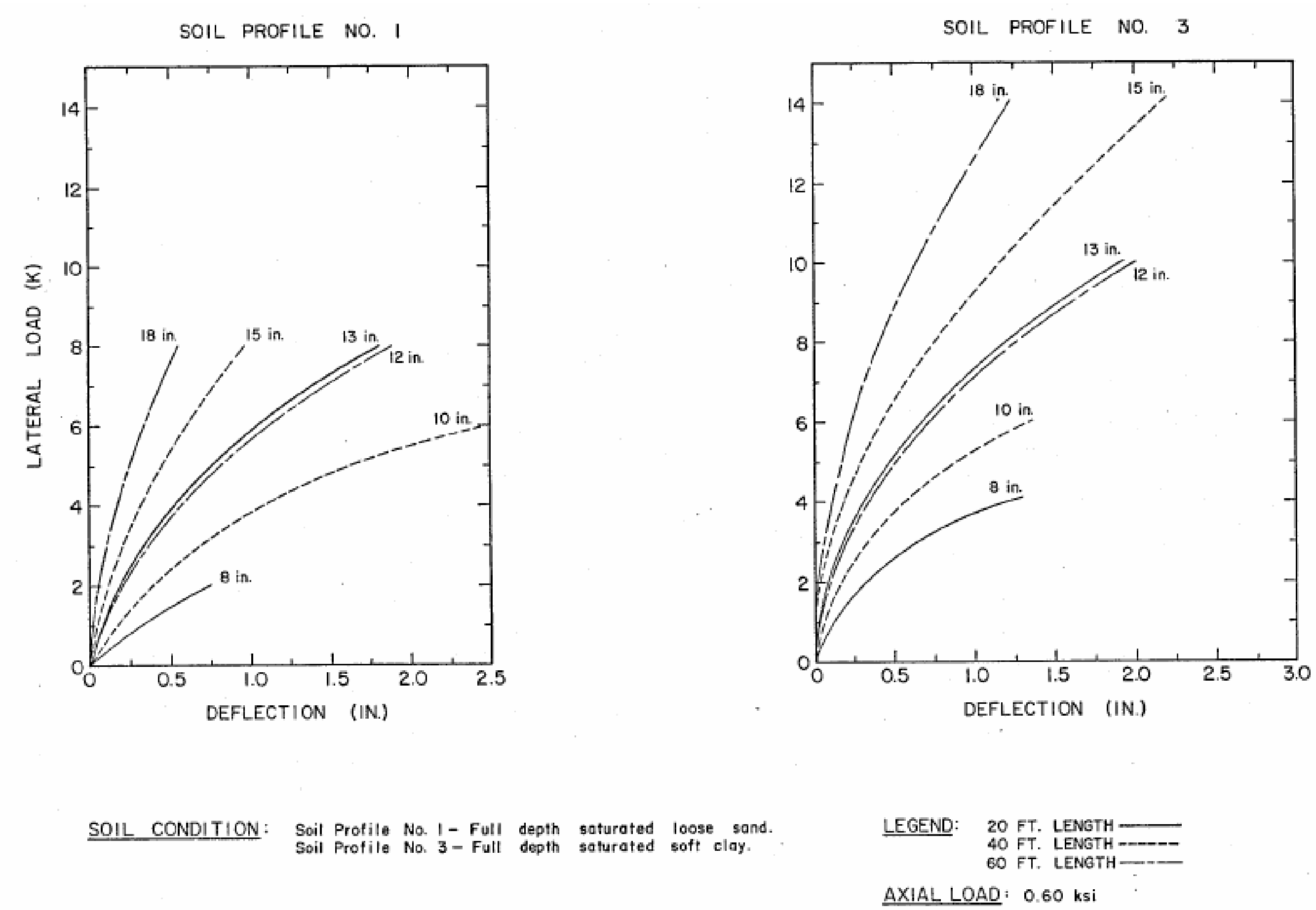


Figure 1.1.8-49 - Load Versus Deflection for Timber Pile, Soil Profile 1 and 3

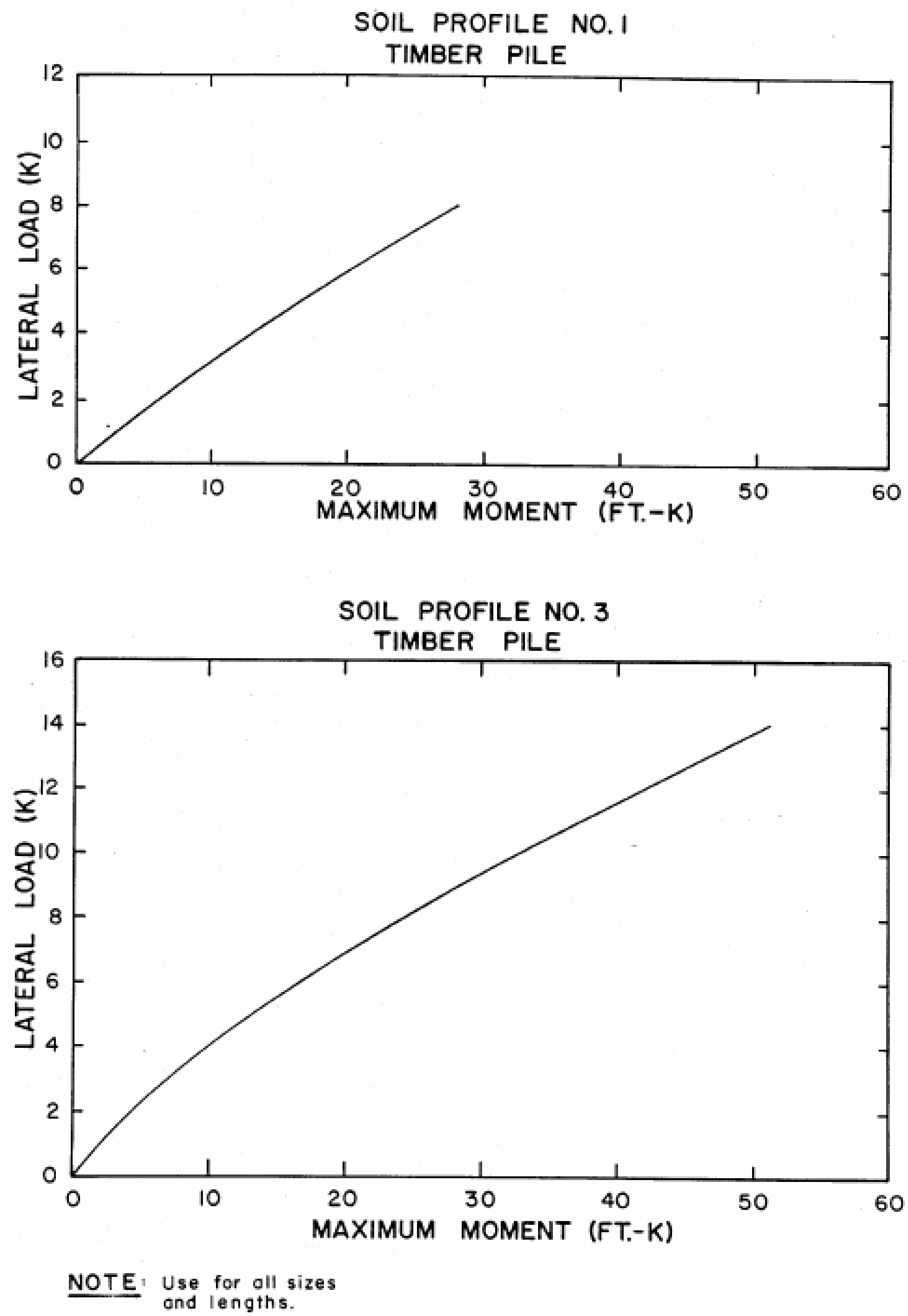


Figure 1.1.8-50 - Load Versus Maximum Moment for Timber Pile, Soil Profile 1 and 3

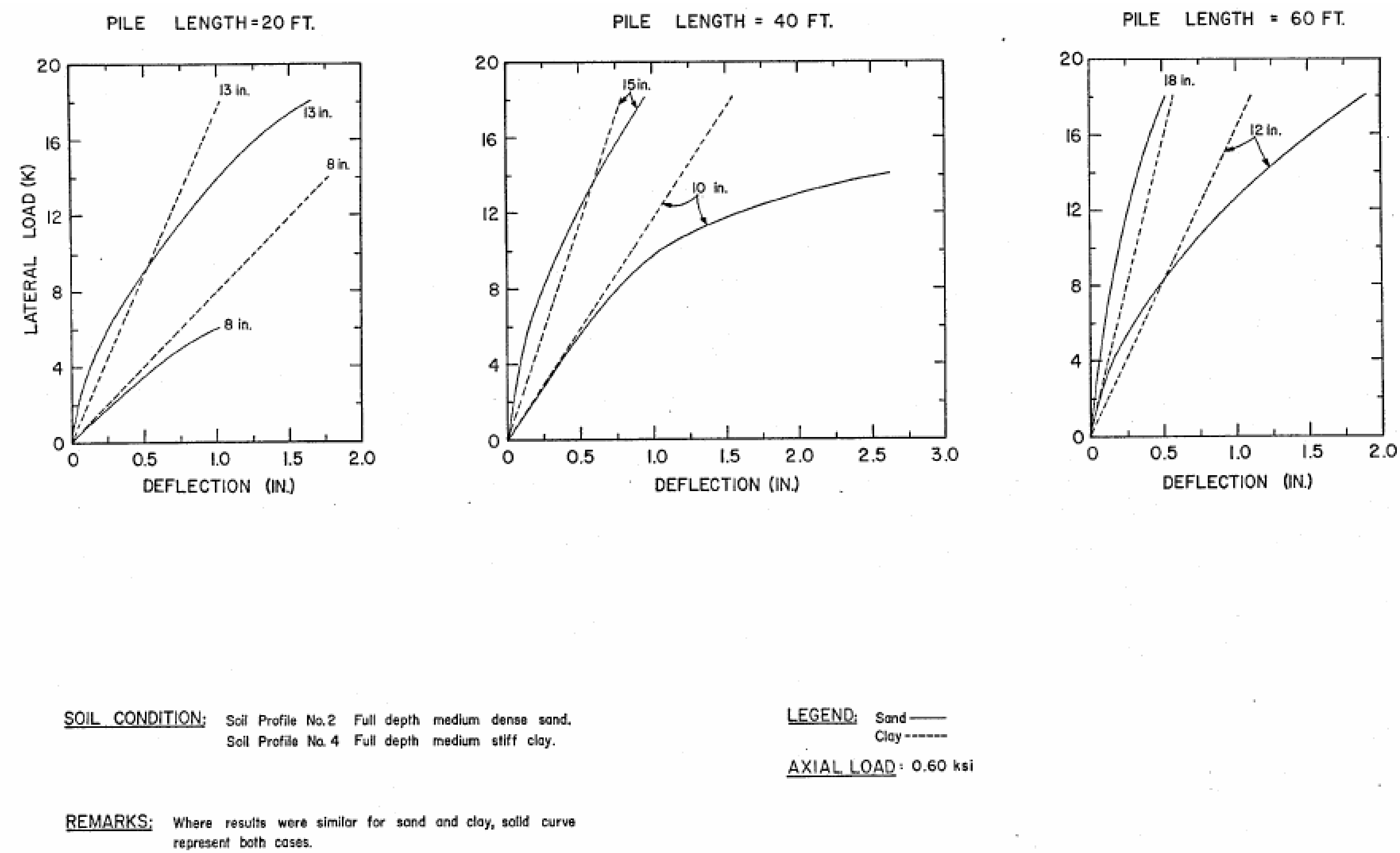


Figure 1.1.8-51 - Load Versus Deflection for Timber Pile, Soil Profile 2 and 4

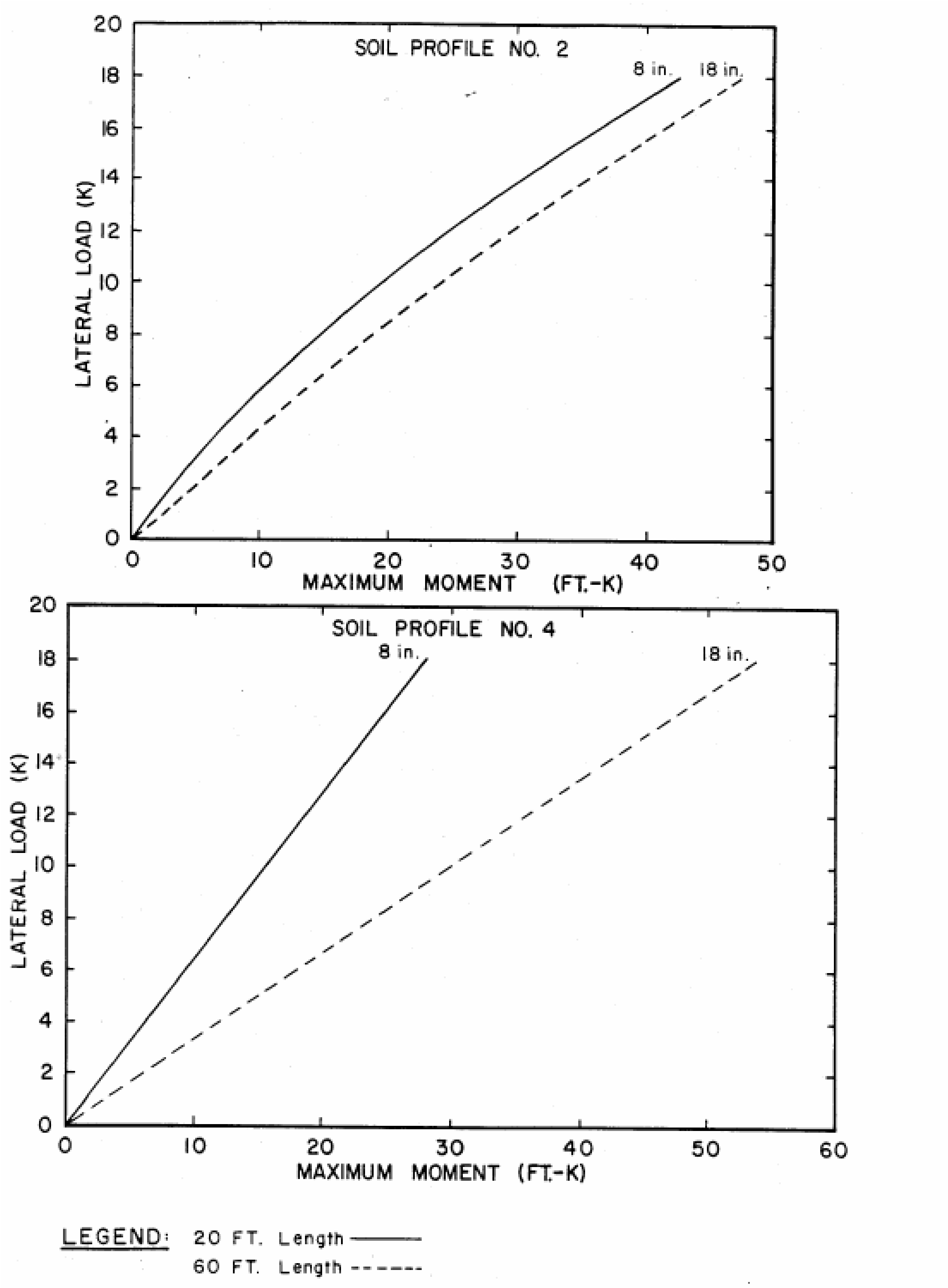


Figure 1.1.8-52 - Load Versus Maximum Moment for Timber Pile, Soil Profile 2 and 4

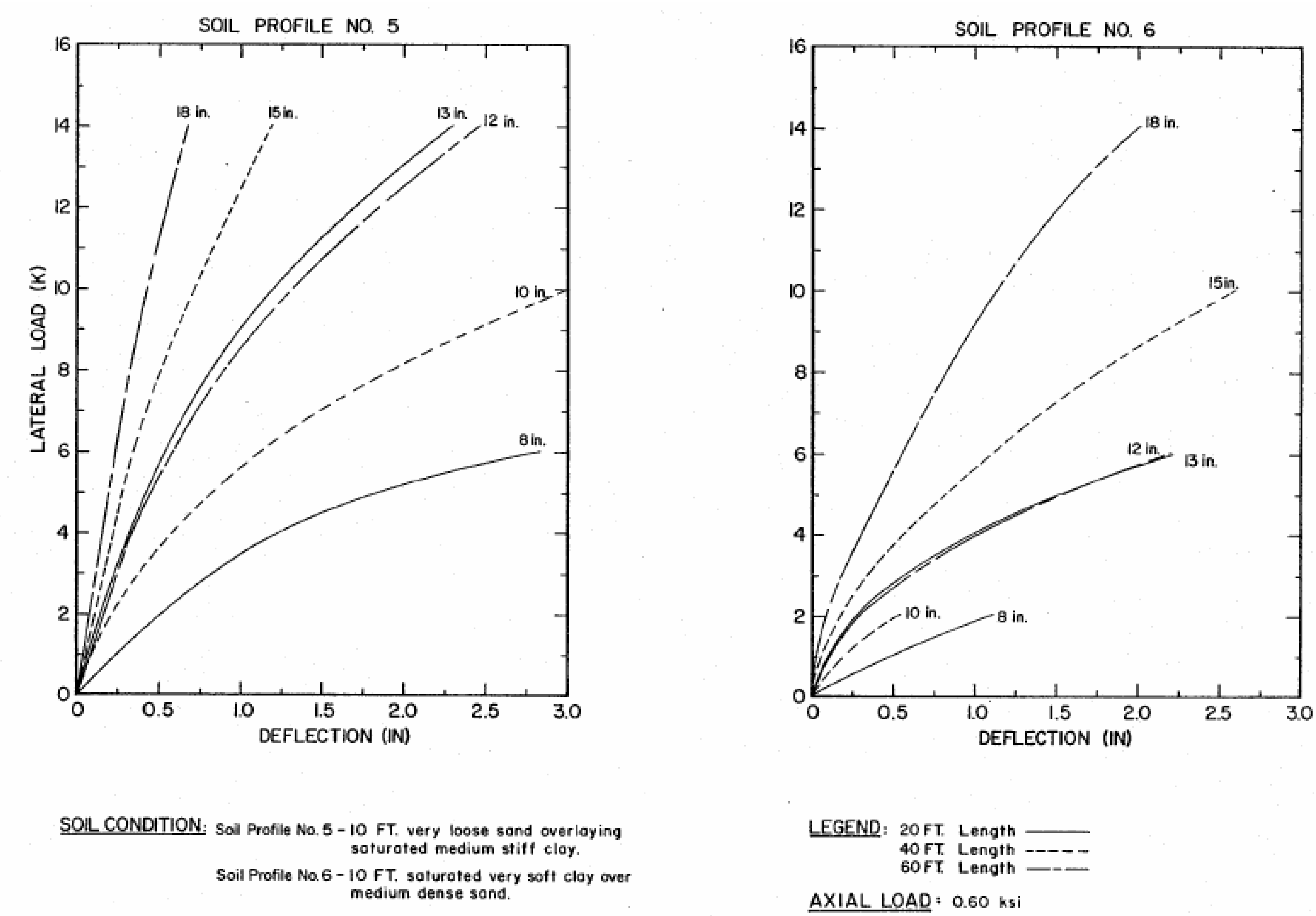


Figure 1.1.8-53 - Load Versus Deflection for Timber Pile, Soil Profile 5 and 6

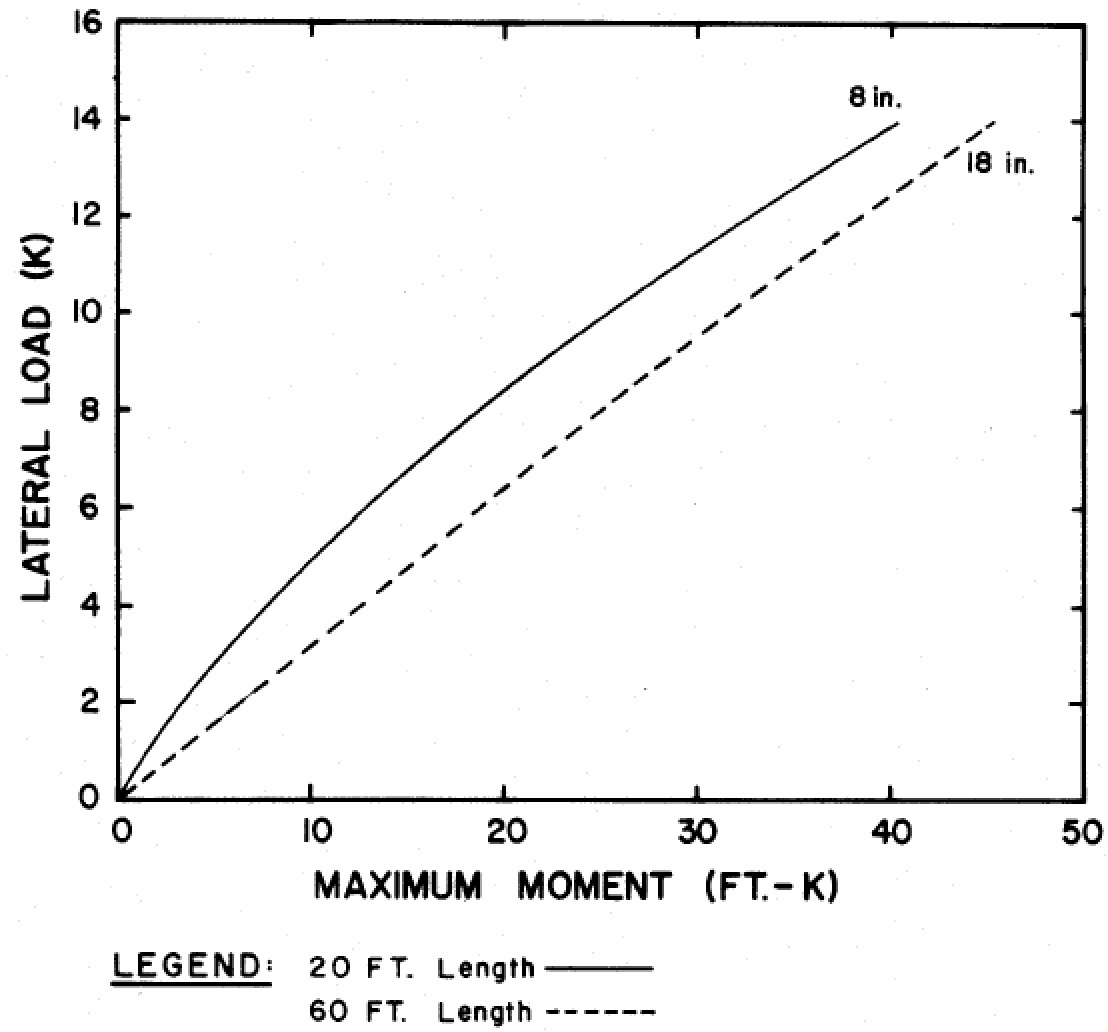


Figure 1.1.8-54 - Load Versus Maximum Moment for Timber Pile, Soil Profile 5

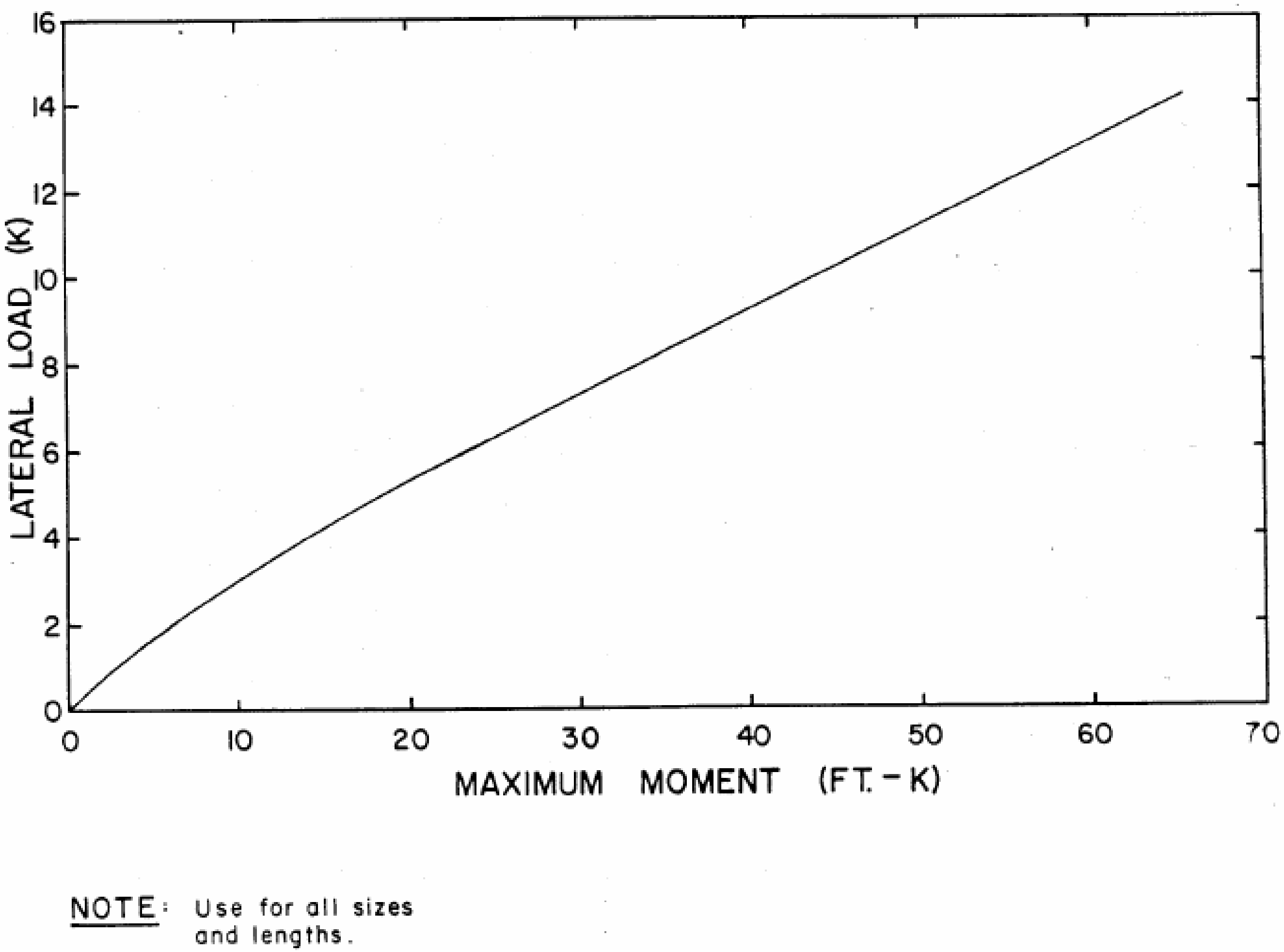


Figure 1.1.8-55 - Load Versus Maximum Moment for Timber Pile, Soil Profile 6

1.1.9 Additional Analyses

The lateral load versus deflection and lateral load versus moment curves can be used for the typical pier and abutment pile system. They provide conservative reasonable values for most design cases. Cases which shall be investigated independently include the following:

(a) Soil conditions worse than shown on the soil profiles in Figure 1.1.6-1.

(b) Single row of piles. A single row of piles performs similar to the free-head condition. Deflections taken from the curves may be considerably unconservative for a single row of piles.

(c) Piles subject to lateral loads with large (greater than 25 mm {1.0 in}) deflections. Fixities less than 50 percent may result from high lateral loads, resulting in unconservative deflections and possibly unconservative moments. Usually deflection limitations will prevent the application of such high loads.

If further increase of the lateral load resistance will allow substantial cost savings, additional analyses may also be performed for the following cases:

(a) Soil conditions better than those shown on the soil profiles in Figure 1.1.6-1 in the Manual or borderline cases where soil conditions fall between those covered by those soil profiles.

(b) Piles with small (less than 6.25 mm {0.25 in}) deflections controlling the design - The fixity of the pile subject to small lateral loads and deflections will be closer to 100 percent. Deflections found on the curves may be unnecessarily conservative in this case.

Additional analyses shall be performed with the COM624G program following the procedures outlined in the program documentation [4,11]. For known C, φ and γ_T, other input parameters can be estimated based on recommendations within Reese's work [4]. Pile head fixities other than free of 100 percent fixed can be modeled with the following procedure:

(a) For each lateral load, run the fixed-head case to obtain the fixed-head moment.

(b) Apply the appropriate percentage of the fixed-head moment to the top of the pile, and run the free-head case. For example, for 50 percent fixity, 50 percent of the fixed-head moment would be applied to the top of the pile in the direction which would restrain movement. The program would then be run for the free-head case with the appropriate lateral load.

1.1.10 Lateral Resistance Design Procedure

Incorporate the lateral resistance of vertical piles into foundation design using the following procedure:

(a) With the pile size and type selected for design and available geotechnical information, determine the moment in the pile caused by the factored lateral load by using the lateral load versus moment curves. This moment shall be included in the total factored moments used in the interactive equations for structural capacity found in AASHTO Sec. 10.4.7. A composite load factor shall be used when multiple lateral loads are used in a particular group loading.

(b) Find the estimated horizontal deflection at the top of the pile caused by the lateral loads by using the lateral load versus deflection curves with the factored lateral loads.

(c) If deflections exceed a value considered acceptable for the structure type, the following should be considered:

(1) Add batter piles to reduce design lateral load for pile.
(2) Revise pile size (preliminary design only)
(3) Add vertical piles to reduce the lateral load per pile.

2.1 DRILLED SHAFTS

2.1.1 General

The following sections list information on the development of design curves used to determine the lateral deflection and moment on drilled shafts when given the soil profile and a lateral load. The curves were originally developed in US units for service load design. Since the method used was not dependant on the design method used (LRFD or Service Load Design), the curves can be used with any design methods as long as the specified units are consistent with the loads used. Therefore, the curves were converted to SI units for use as a preliminary design tool. The design curves should be used as described in D 10.7.3.8.2P.

Section 2.3 was not converted to SI units because of the large number of empirical equations which are units dependant. All the equations and text are shown in their original form but, where applicable, the comparable SI unit value is listed in parentheses.

2.1.2 Application

Drilled shafts are deep foundations formed by boring a cylindrical hole into soil and/or rock and filling the hole with concrete. Drilled shafts are also commonly referred to as caissons, drilled caissons, bored piles or drilled piers. Similar to driven piles, drilled shafts transfer structural loads to bearing stratum well below the base of the structure. Drilled shafts are typically used to bypass soils having insufficient strength to carry the design loads.

Drilled shafts are classified according to their primary mechanism for deriving load resistance, and include floating shafts and end-bearing shafts in soil or rock. Occasionally, the base of the shaft is enlarged (i.e., belled or underreamed) to improve the load capacity of shafts bearing on less than desirable soils or to increase the uplift resistance of floating shafts.

2.1.3 Notations

The following notation shall apply for the design of drilled shaft foundations:

A = Coefficient used to define p-y curve for clay below water table by unified method (dim) (See 2.3.1.1.1.4)

A_c = Empirical coefficient for developing p-y curves for cyclic loading of stiff clay below water table (dim) (See 2.3.1.1.1.2)

$\bar{A}_c$ = Empirical coefficient for developing p-y curves for cyclic loading of sand (dim); (See 2.3.1.1.1.5)

A_s = Empirical coefficient for developing p-y curves for static loading of stiff clay below water table (dim) (See 2.3.1.1.1.2)

$\bar{A}_s$ = Empirical coefficient for developing p-y curves for static loading of sand (dim) (See 2.3.1.1.1.5)

B = Shaft diameter (ft)

B_c = Empirical coefficient for developing p-y curves for cyclic loading of sand (dim) (See 2.3.1.1.1.5)

B_s = Empirical coefficient for developing p-y curves for static loading of sand (dim) (See 2.3.1.1.1.5)

c = Undrained shear strength (ksf)

C = Empirical coefficient for developing p-y curves for stiff clay above water table (dim) (See 2.3.1.1.1.3)

$\bar{C}$ = Coefficient used in p-y curves for sand (dim) (See 2.3.1.1.1.5)

c′ = Effective stress cohesive strength (ksf)

c_a = Average undrained shear strength over depth z (ksf)

D = Shaft length (ft)

dz = Increment of shaft length (ft)

e = Distance of P above ground surface for laterally loaded shaft (ft)

E_c = Modulus of concrete shaft (ksf)

E_s = Soil modulus - secant to p-y curve (ksf)

E_{si} = Initial lateral soil modulus of subgrade reaction (kcf)

$(E_s)_{max}$ = Maximum lateral soil modulus of subgrade reaction (kcf) (See 2.3.1.1.1.4)

F = Empirical coefficient for developing p-y curves for clay below water table (dim) (See 2.3.1.1.1.4)

f'_c = Ultimate 28 day strength of concrete (psi)

i = Interval counter for load transfer analysis (dim)

I_c = Moment of inertia of shaft (ft^4)

I_g = Gross moment of inertia of shaft section (ft^4)

J = Empirical parameter for developing p-y curves for soft clay soil deposits (dim)

K = Lateral earth pressure coefficient (dim)

K_a = Active earth pressure coefficient (dim) ($[1 - \sin\varphi']/[1 + \sin\varphi']$)

k_c = Relative stiffness along soil/shaft interface due to cyclic lateral loading (kcf)

k_s = Relative stiffness along soil/shaft interface due to static loading (kcf)

K_o = At-rest lateral earth pressure coefficient equal to horizontal to vertical effective stress ratio (dim)

K_p = Passive earth pressure coefficient (dim) ($[1 + \sin\varphi']/[1 - \sin\varphi']$)

N = Number of cycles of cyclic lateral loading

NC = Normally consolidated (dim)

n_h = Coefficient of horizontal subgrade reaction (kcf)

P = Lateral load (k)

p_{cr} = Residual soil resistance for cyclic loading p-y curves by the Unified Method (kcf) (See 2.3.1.1.1.4)

p_m = Ultimate soil resistance per unit length for shaft in sand (k/ft)

p_r = Residual soil resistance for static loading p-y curves by the Unified Method (kcf) (See Table 2.3.1.1.1-4)

p_s = Ultimate soil resistance per unit length for shaft in sand (k/ft)

p_{sd} = Ultimate soil resistance at depth for shaft in sand (k/ft) (See 2.3.1.1.1.5)

p_{st} = Ultimate soil resistance near ground surface for shaft in sand (k/ft) (See 2.3.1.1.1.5)

p_u = Ultimate lateral soil resistance per unit length of shaft (k/ft)

P_{ULT} = Ultimate lateral load on shaft (k)

yc = Lateral deflection following N cycles of cyclic loading (ft); (See Table 2.3.1.1.1-3)

yk, ym,
yu = Specific deflection on p-y curve for sand (ft); (See Figure 2.3.1.1.1(J))

yp = Specific deflection on p-y curve for cyclic loading of stiff clay below water table (ft); (See Table 2.3.1.1.1-2)

ys = Lateral deflection due to short term static load (ft); (See Table 2.3.1.1.1-3)

y_{50} = Lateral deflection at one-half pu (ft)

z = Depth along shaft from ground surface (ft)

z_r = Depth to transition zone from wedge- to flow-type failure for a laterally loaded shaft (ft); (See Table 2.3.1.1.1-2)

The dimensional units provided with each notation are presented for illustration only to demonstrate a dimensionally correct combination of units for the design procedures and associated tables and figures presented herein. If other units are used, the dimensional correctness of the equations should be confirmed.

2.2 COM624 ANALYSES

Preliminary design curves have been developed for the six hypothetical subsurface profiles presented in Figure 2.2-1. Determine the static lateral capacity using the curves of lateral load versus deflection and lateral load versus absolute maximum moment for fixed and free head conditions. Use Figures 2.2-2 through 2.2-8 (Metric or U.S. Customary Units) to determine load-deflection behavior for each of the subsurface profiles. Figures 2.2-7 and 2.2-9 (Metric or U.S. Customary Units) provide relationships to determine the maximum moment for the loose sand and soft clay profiles, respectively, which represent the two profiles providing the highest level of moment development. Use linear interpolation for cases involving intermediate shaft dimensions or degrees of head fixity. For subsurface conditions not depicted in the figures, consider performing COM624 analyses incorporating the actual subsurface ground conditions.

Comments regarding important aspects of the analyses are summarized in Tables 2.2-1 and 2.2-2. The shaft sections were assumed to be unreinforced, so the concrete modulus of elasticity (E_c) and gross moment of inertia (I_g) were specified to determine the foundation stiffness (E_cI_c). The stiffness of a reinforced concrete section is dependent on the magnitude of the bending stress and amount of reinforcement; thus, the bending stiffness varies along the length of the shaft and with applied loads. However, use of the stiffness for the gross concrete section (E_cI_g), where E_c is approximated as $4800\sqrt{f'_c}$ $\{57000\sqrt{f'_c}\}$ (for E_c and f'_c in MPa), is sufficiently accurate in most instances. When more refined analysis is required, computer methods can be employed to develop interaction diagrams for the shaft and establish effective stiffness at the appropriate stress levels.

In all cases, the option to have p-y curves generated internally was selected. The reader should be aware of the techniques used to formulate the soil responses and recognize their limitations, as described in Section 2.3.1.1.1. Section 2.3.1.1.1 was not converted to SI units because of the large number of empirical equations which are units dependant.

Representative combinations of soil properties for the subsurface profiles were estimated, and values for the strain at 50 percent stress level (E_{50}) and constant coefficient of subgrade reaction (n_h in $E_s = n_h{}^z$) chosen. The values were based on recommendations in 2.3.1.1.1.1 through 2.3.1.1.1.5 for E_{50}, and n_h.

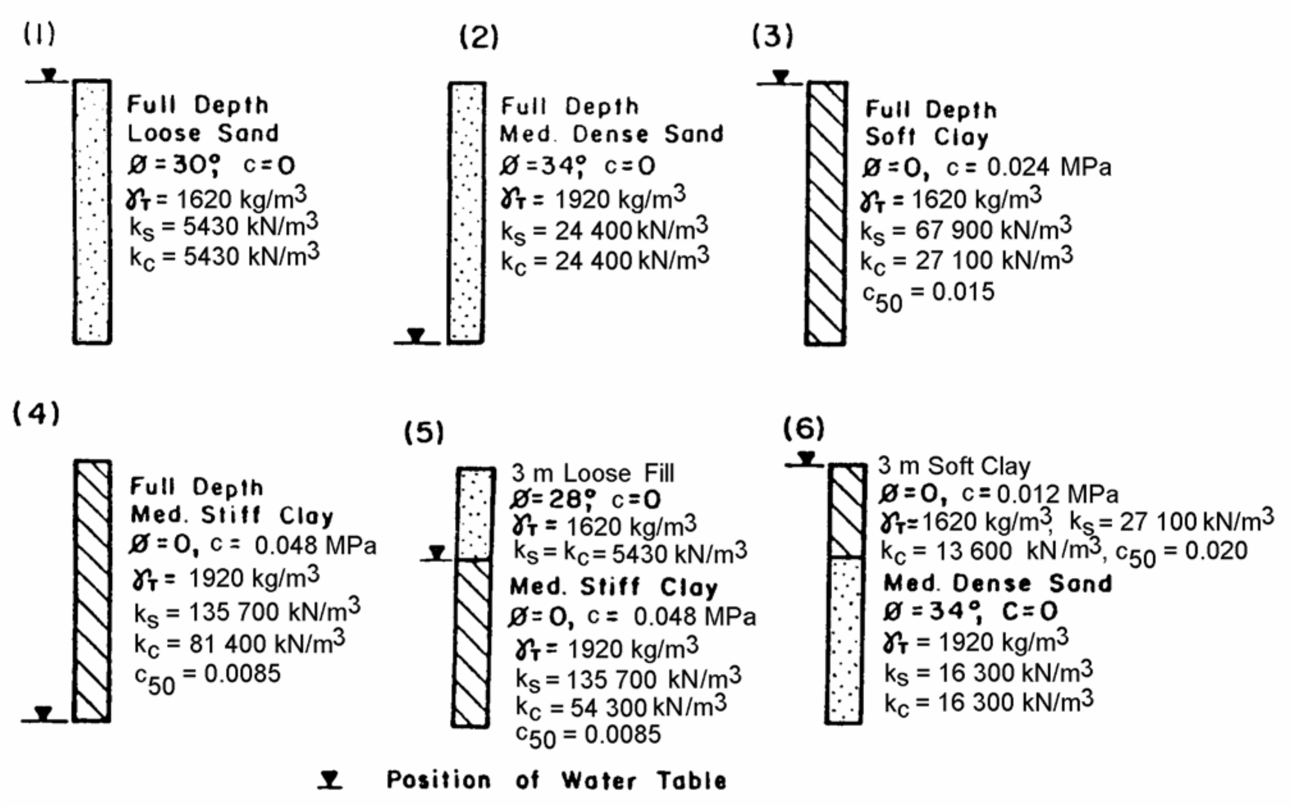


Figure 2.2-1 - Soil Profiles for COM624 Analyses

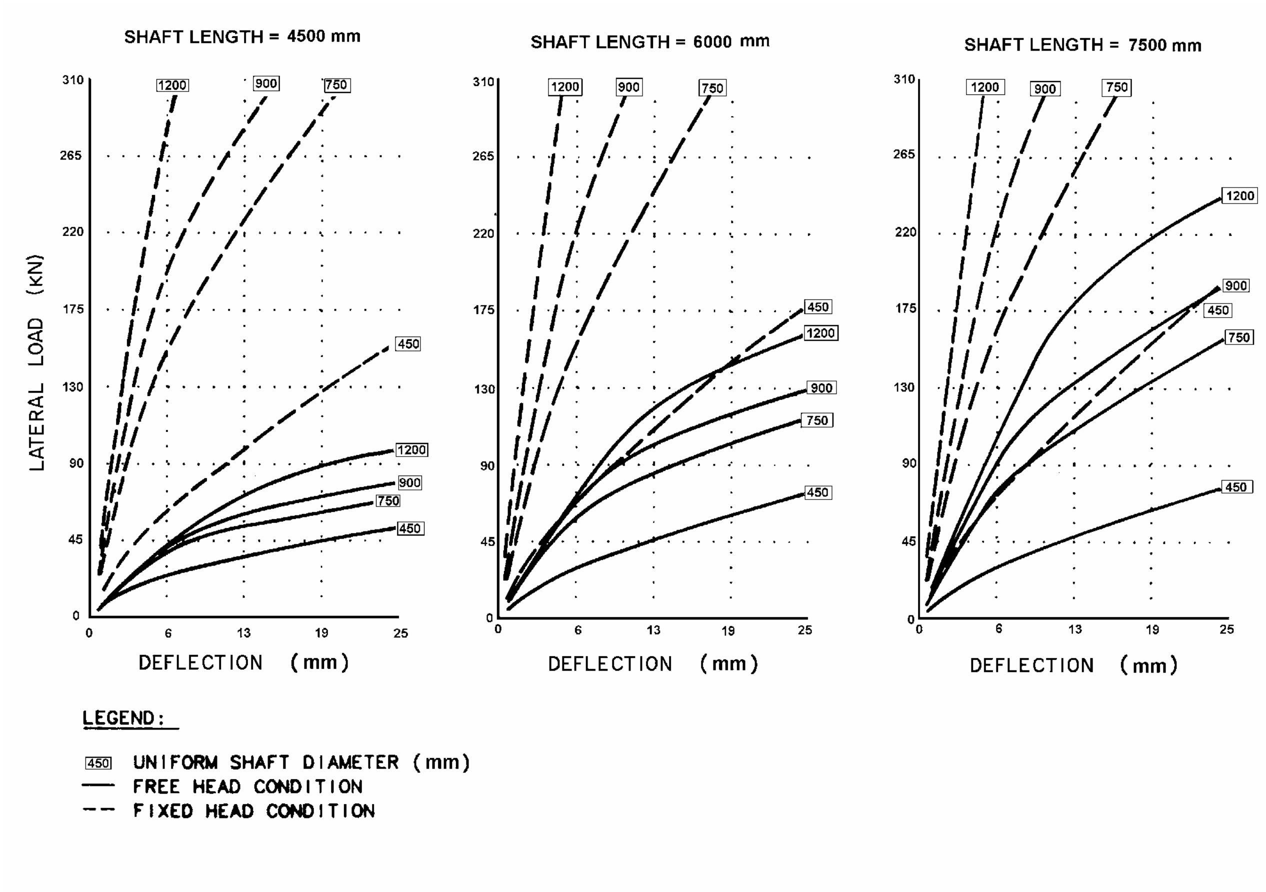


Figure 2.2-2 - COM624 Analyses for Preliminary Design
Lateral Load Versus Lateral Deflection, Static Loading, Subsurface Profile 1

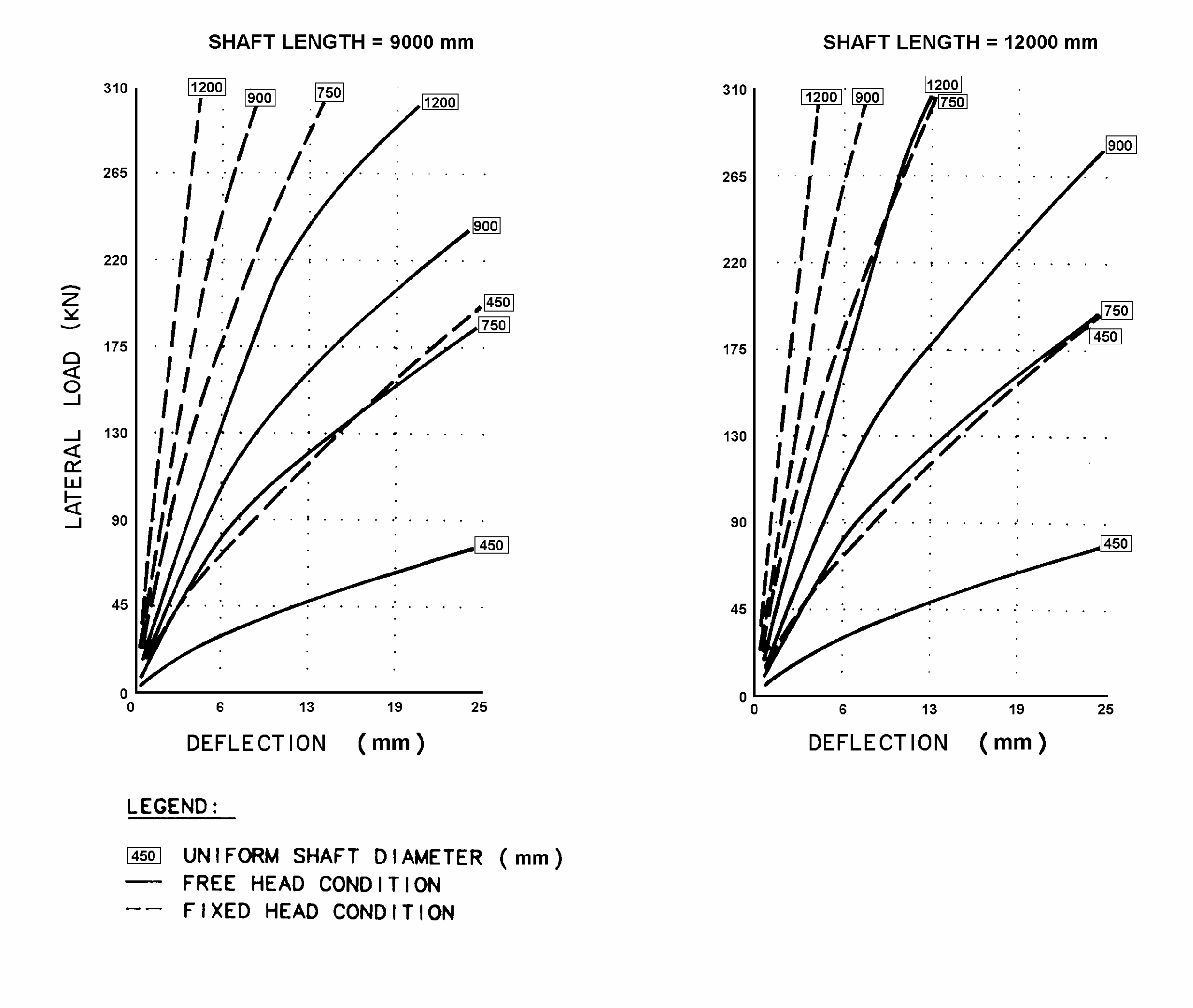


Figure 2.2-2 - COM624 Analyses for Preliminary Design
Lateral Load Versus Lateral Deflection, Static Loading, Subsurface Profile 1 (Continued)

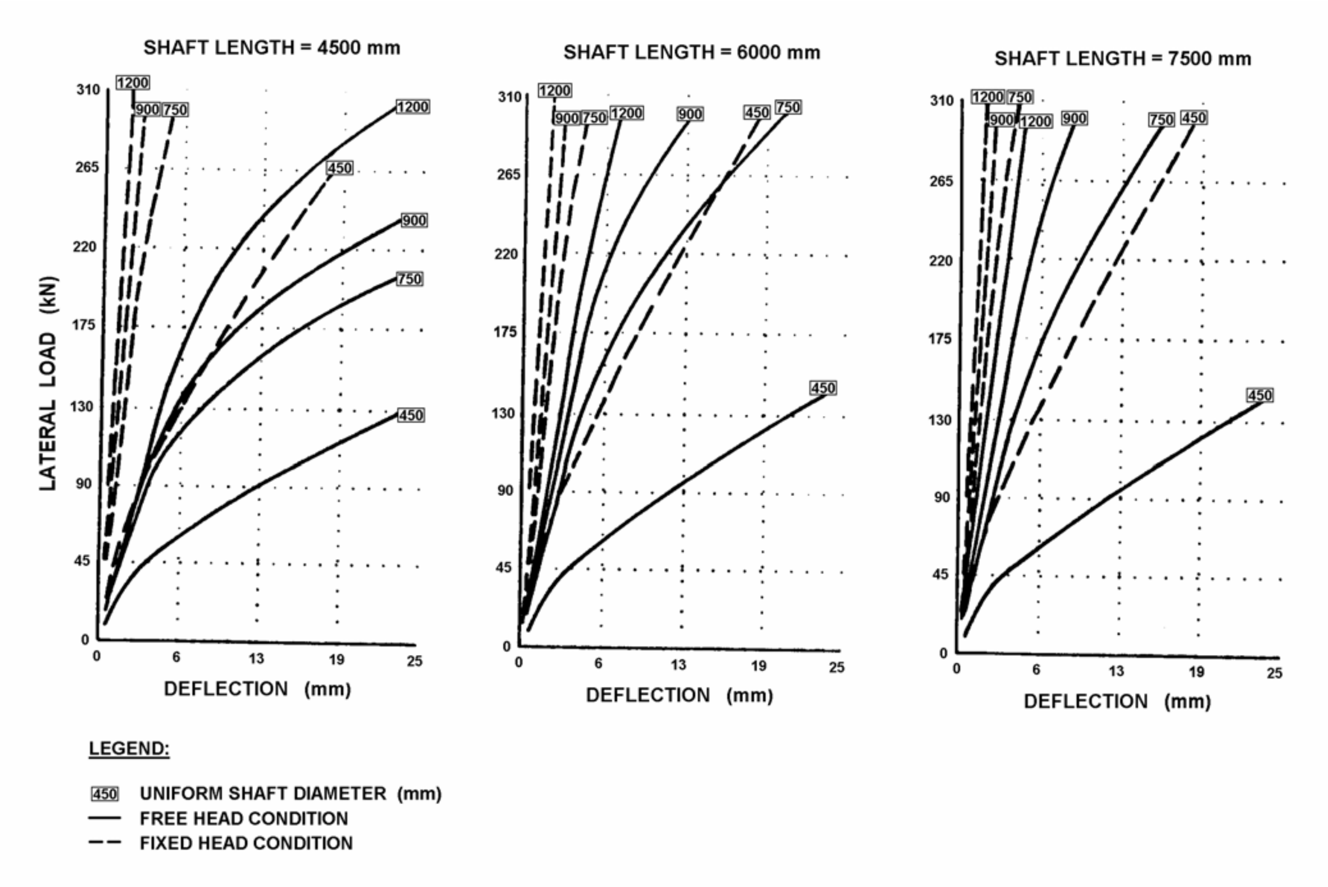


Figure 2.2-3 - COM624 Analyses for Preliminary Design
Lateral Load Versus Lateral Deflection, Static Loading, Subsurface Profile 2

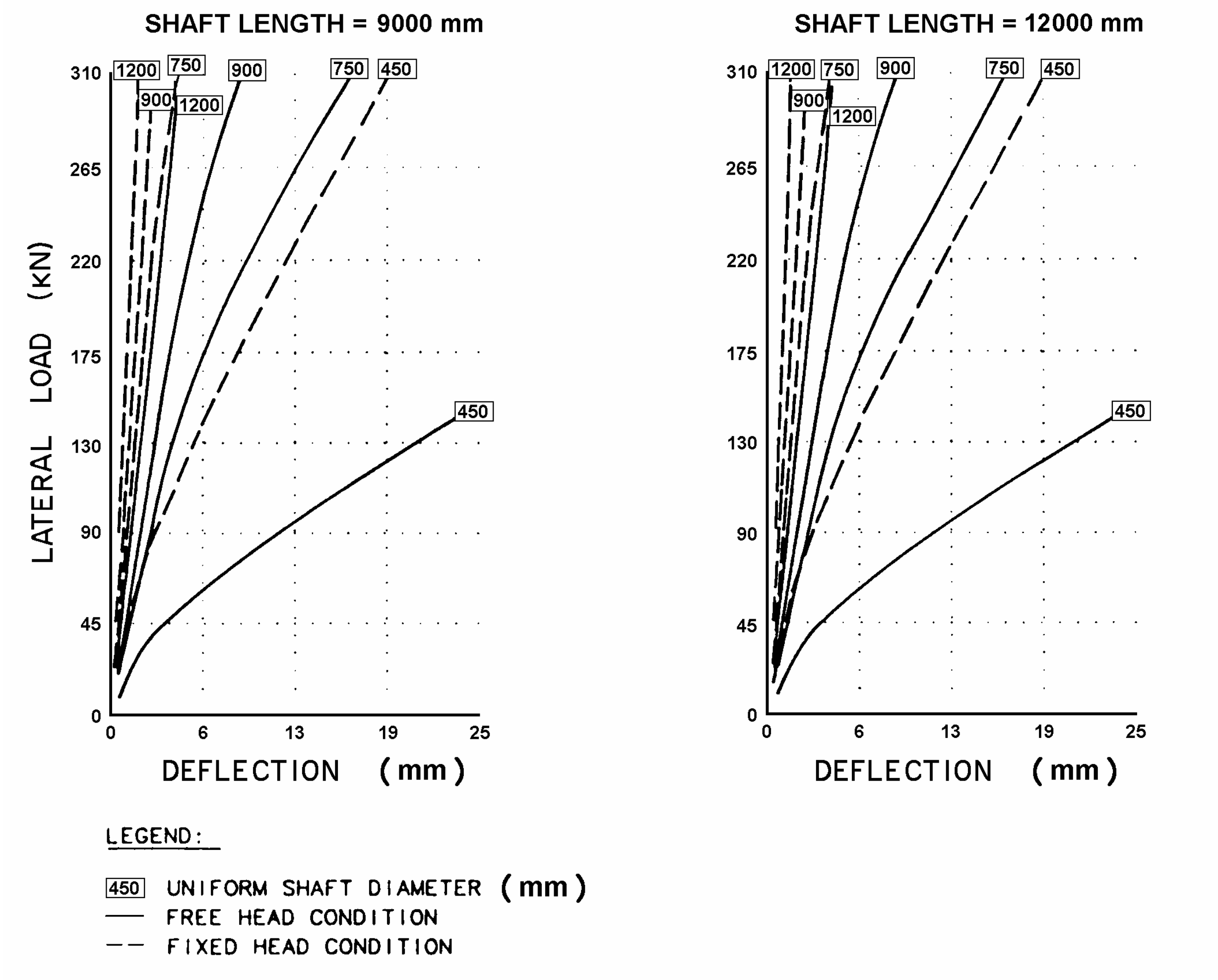


Figure 2.2-3 - COM624 Analyses for Preliminary Design
Lateral Load Versus Lateral Deflection, Static Loading, Subsurface Profile 2 (Continued)

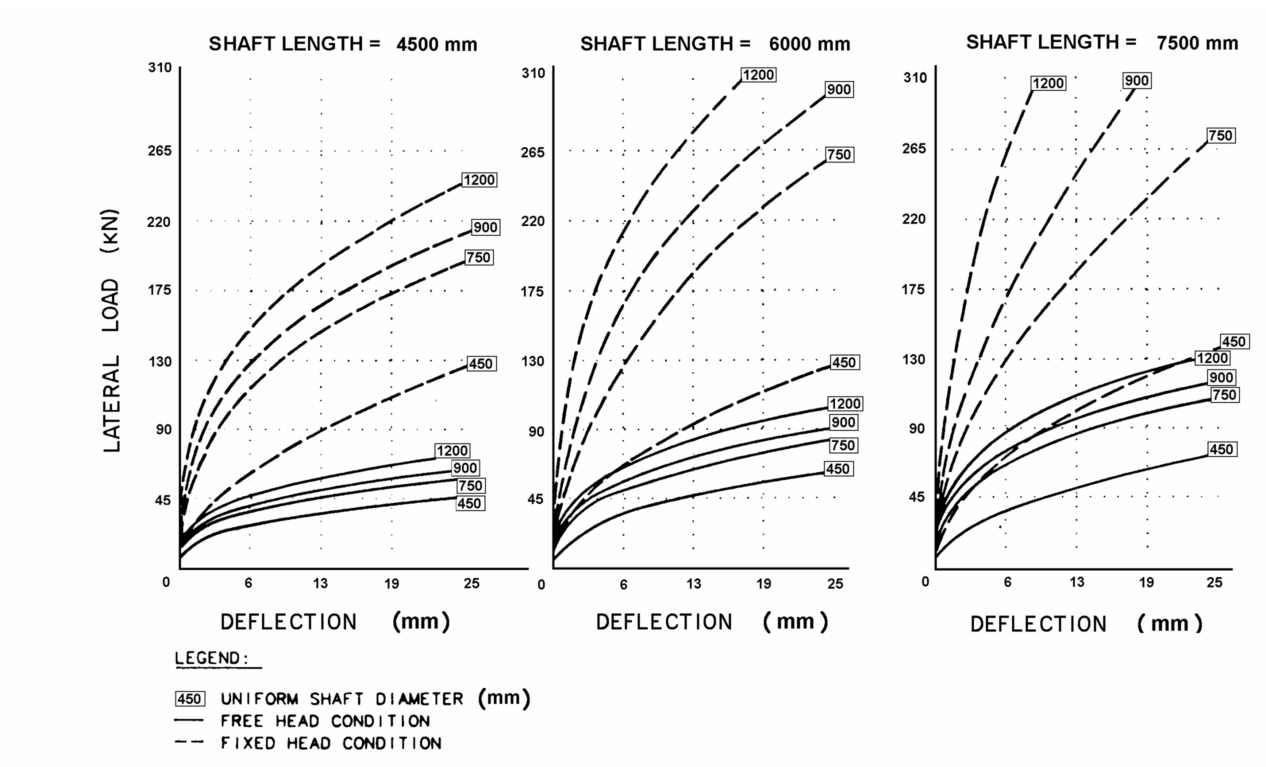


Figure 2.2-4 - COM624 Analyses for Preliminary Design
Lateral Load Versus Lateral Deflection, Static Loading, Subsurface Profile 3

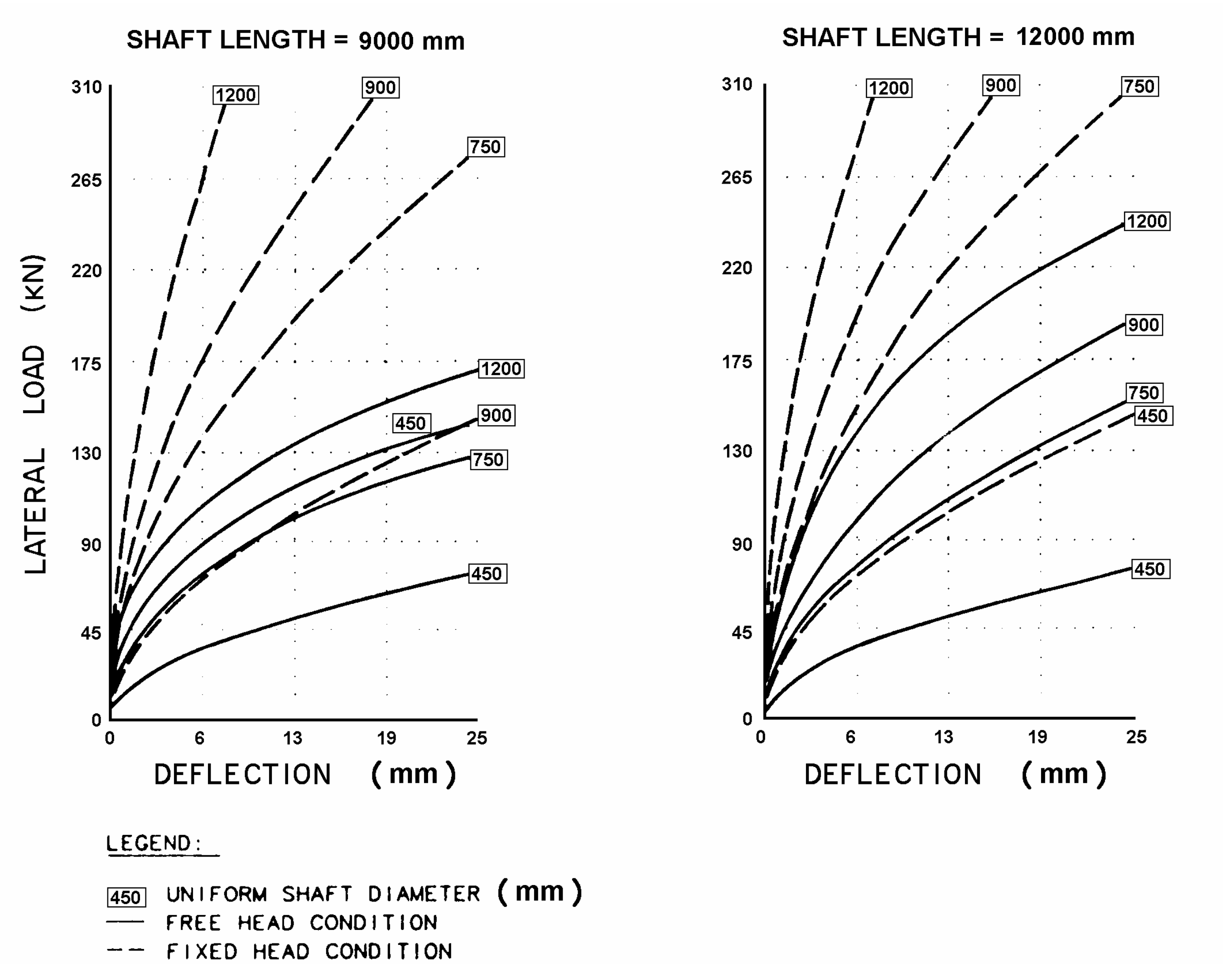


Figure 2.2-4 - COM624 Analyses for Preliminary Design
Lateral Load Versus Lateral Deflection, Static Loading, Subsurface Profile 3 (Continued)

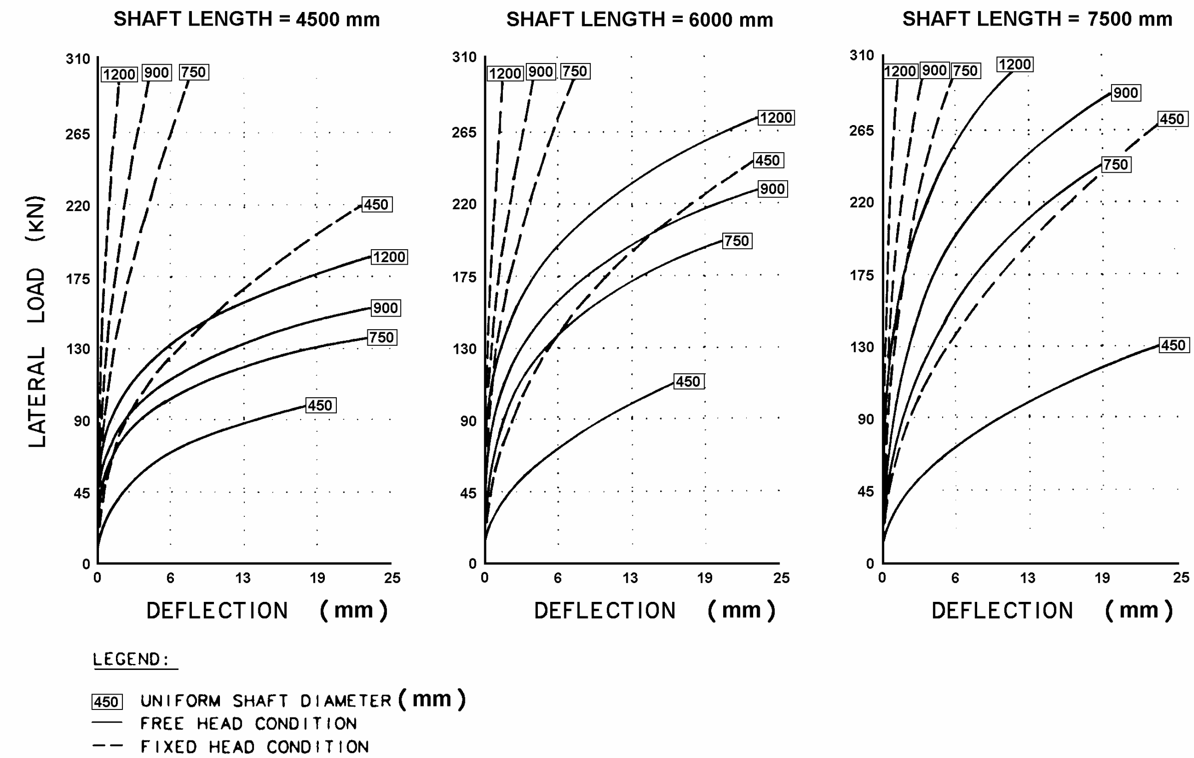


Figure 2.2-5 - COM624 Analyses for Preliminary Design
Lateral Load Versus Lateral Deflection, Static Loading, Subsurface Profile 4

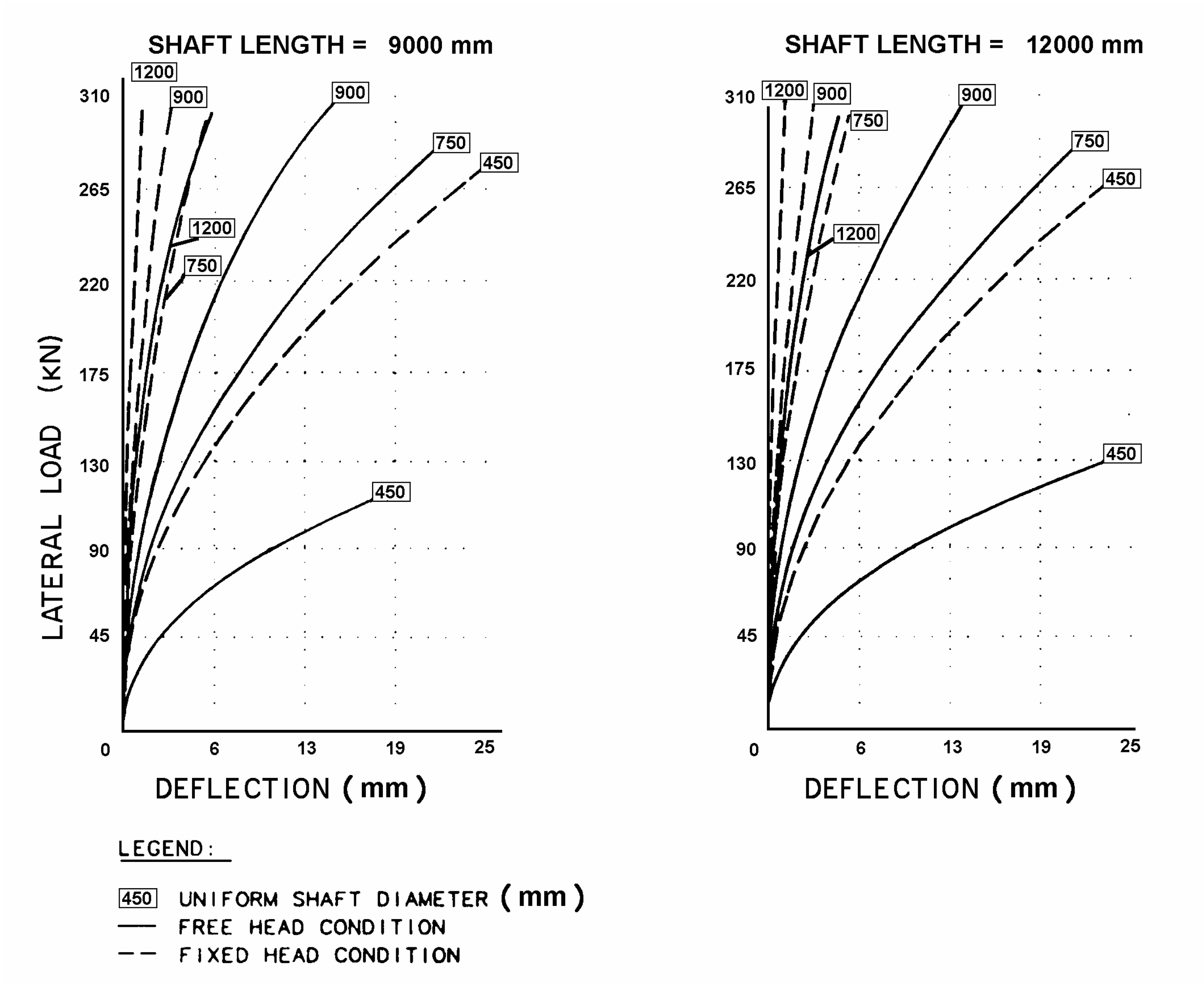


Figure 2.2-5 - COM624 Analyses for Preliminary Design
Lateral Load Versus Lateral Deflection, Static Loading, Subsurface Profile 4 (Continued)

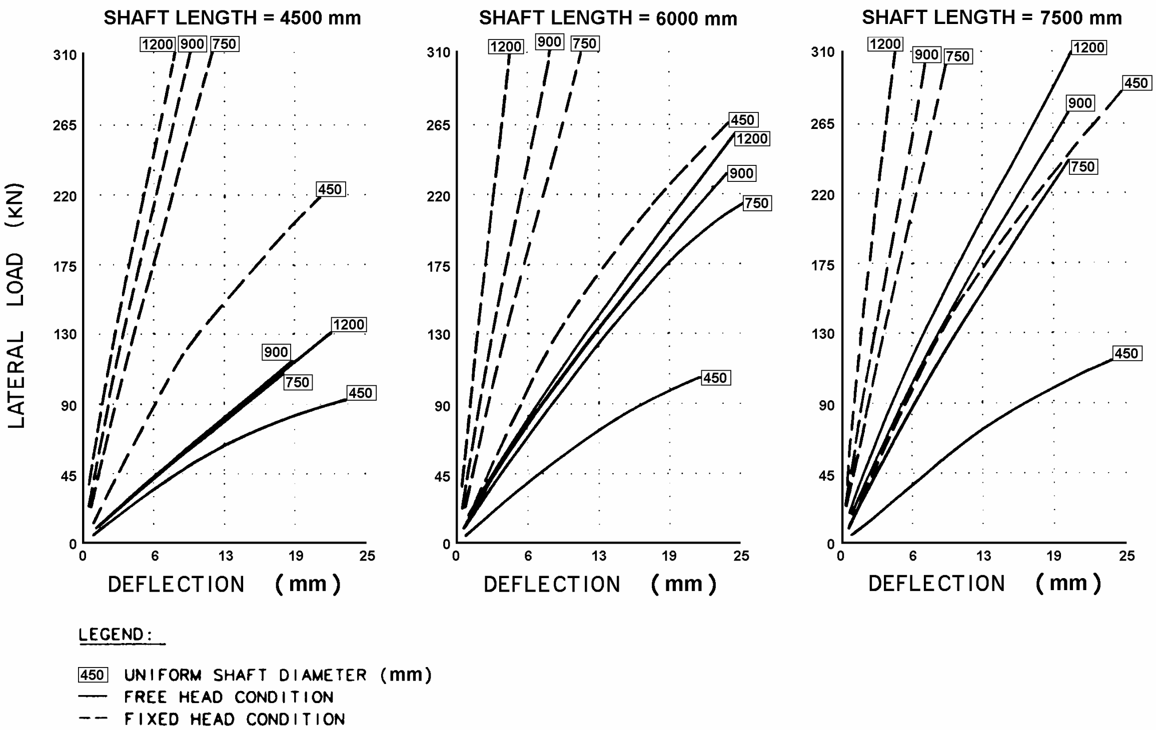


Figure 2.2-6 - COM624 Analyses for Preliminary Design
Lateral Load Versus Lateral Deflection, Static Loading, Subsurface Profile 5

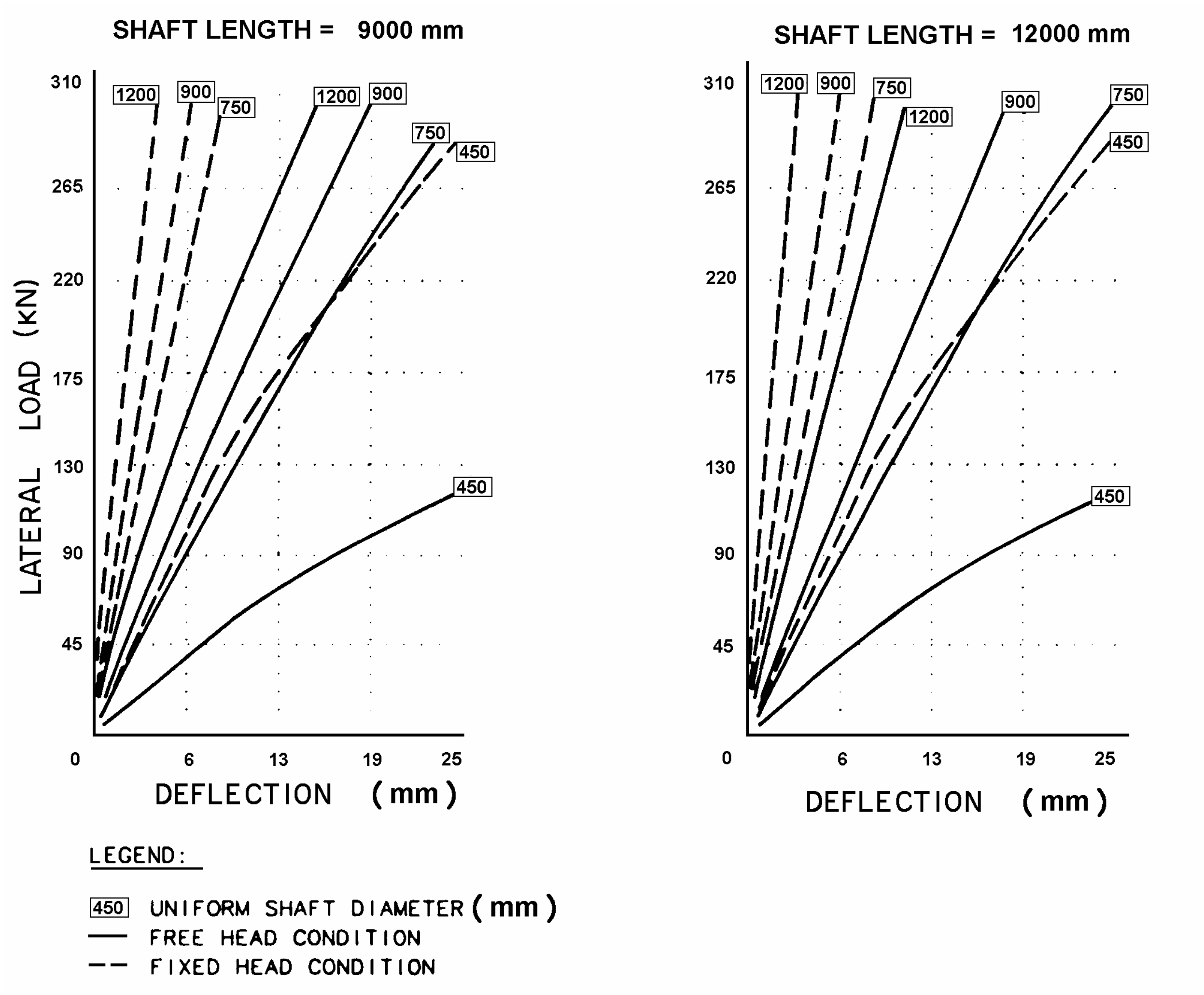


Figure 2.2-6 - COM624 Analyses for Preliminary Design
Lateral Load Versus Lateral Deflection, Static Loading, Subsurface Profile 5 (Continued)

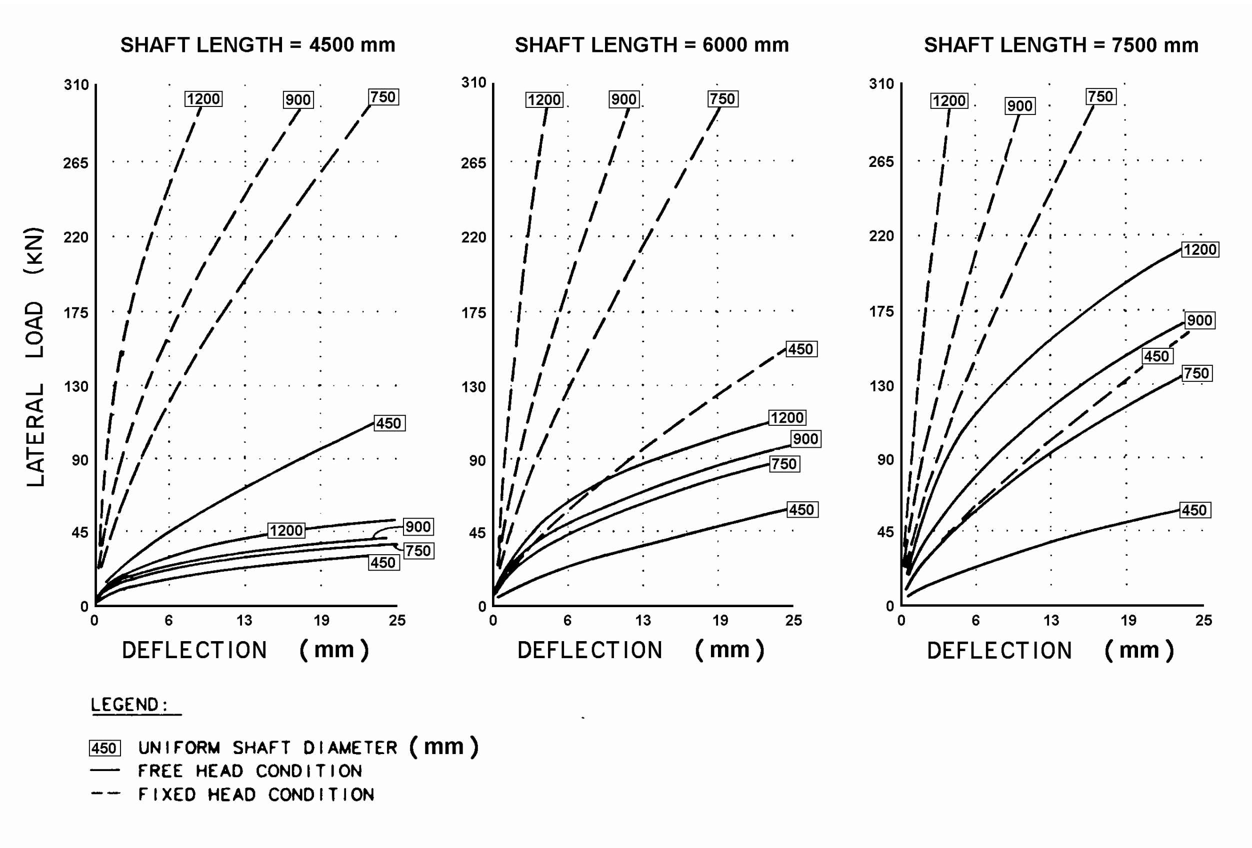


Figure 2.2-7 - COM624 Analyses for Preliminary Design.
Lateral Load versus Lateral Deflection, Static Loading, Subsurface Profile 6

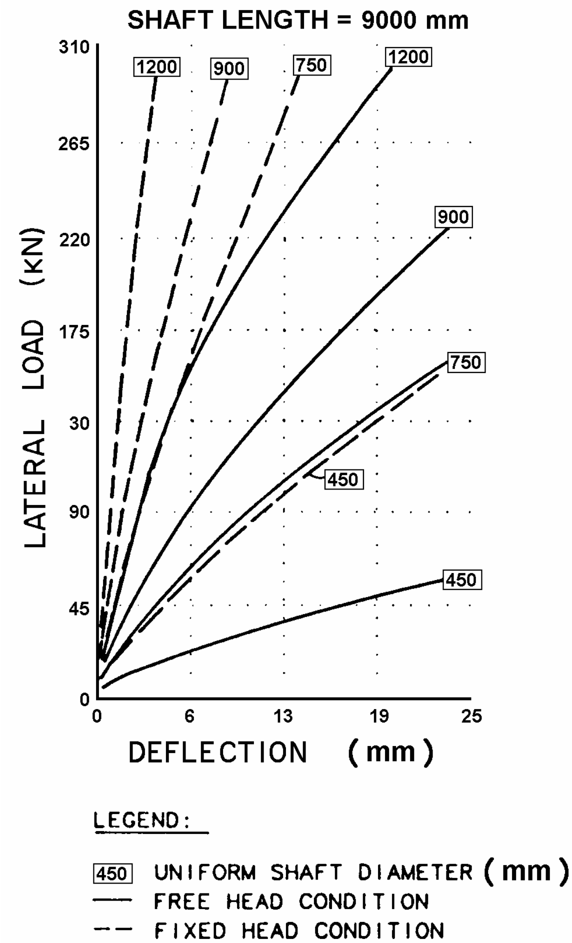


Figure 2.2-7 - COM624 Analyses for Preliminary Design.
Lateral Load versus Lateral Deflection, Static Loading, Subsurface Profile 6 (Continued)

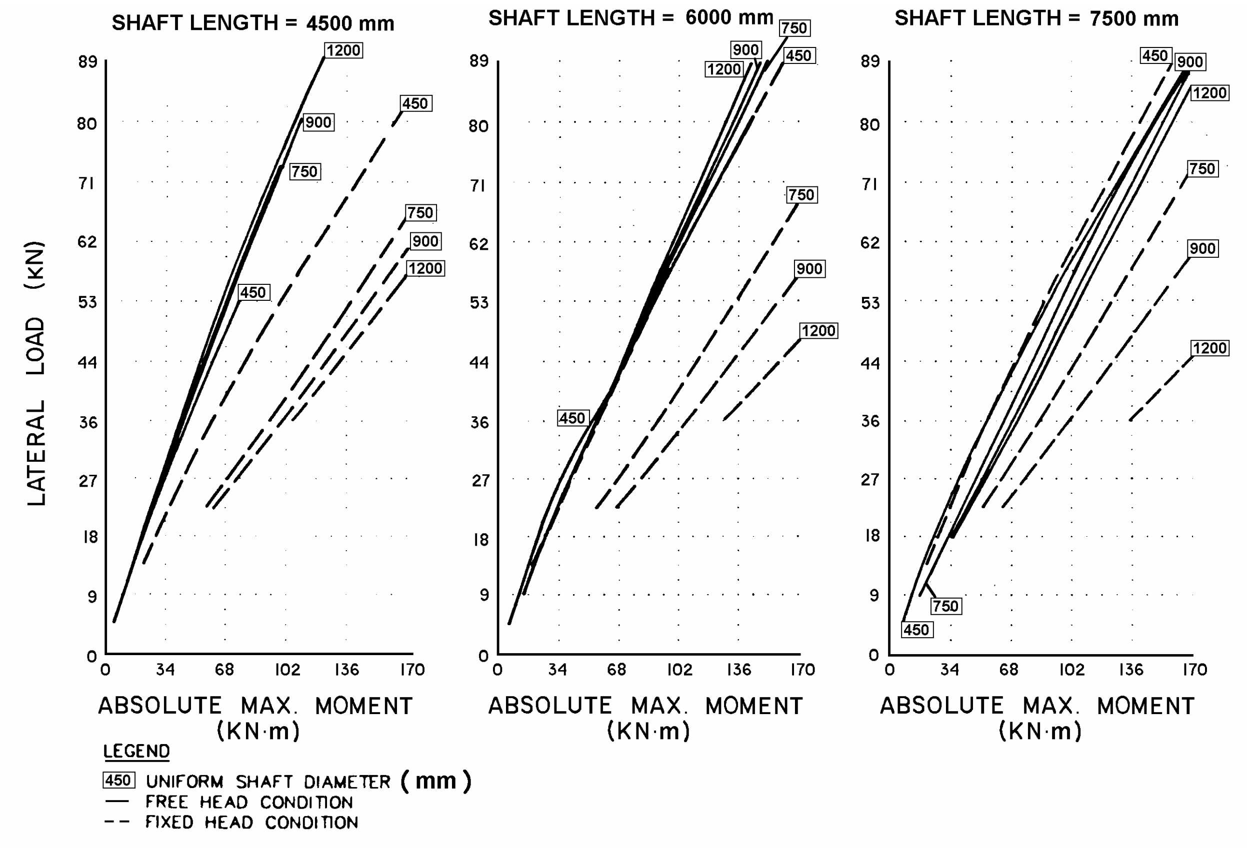


Figure 2.2-8 - COM624 Analyses for Preliminary Design
Lateral Load versus Absolute Maximum Moment, Static Loading, Subsurface Profile 1

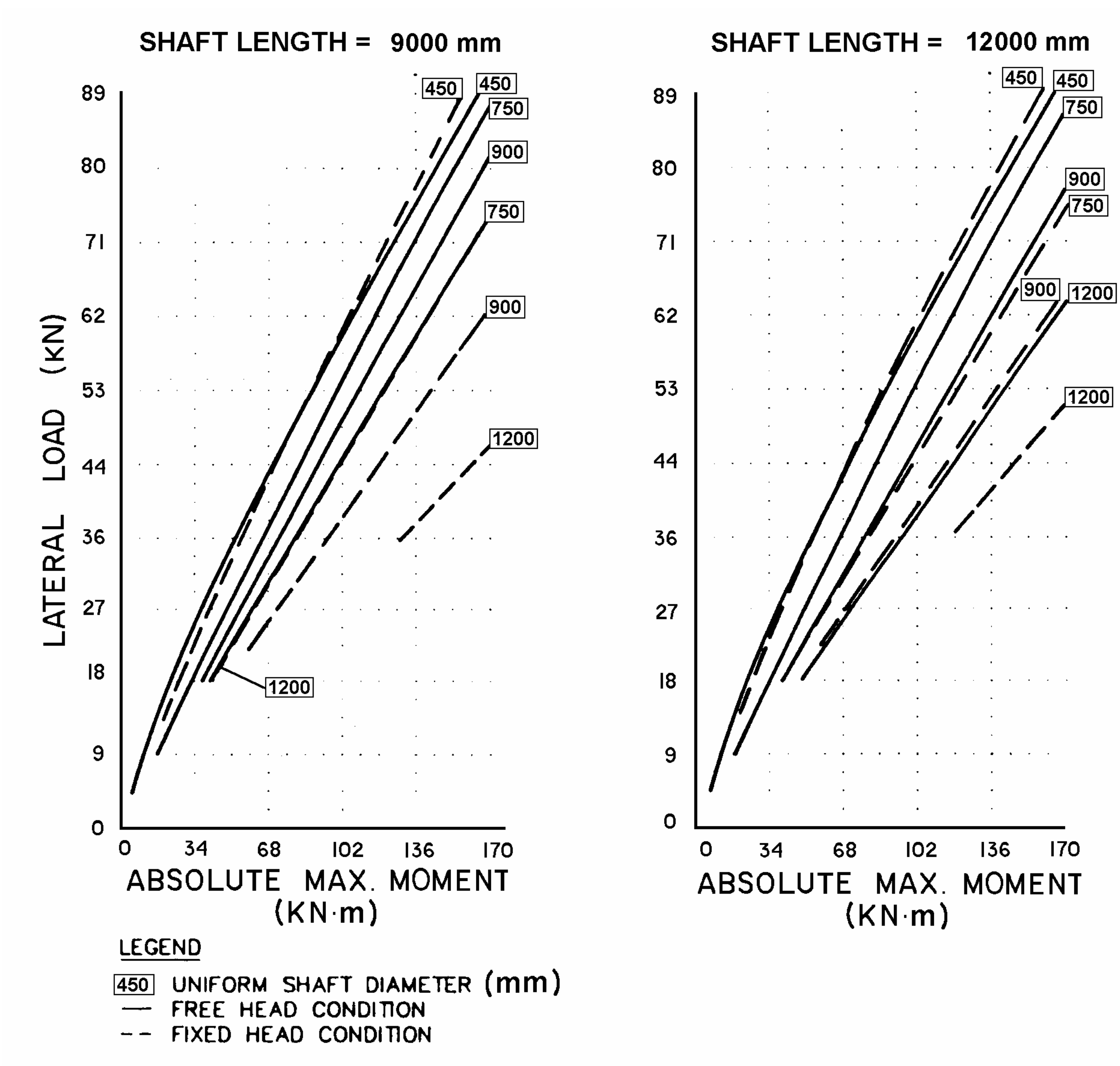


Figure 2.2-8 - COM624 Analyses for Preliminary Design
Lateral Load versus Absolute Maximum Moment, Static Loading, Subsurface Profile 1 (Continued)

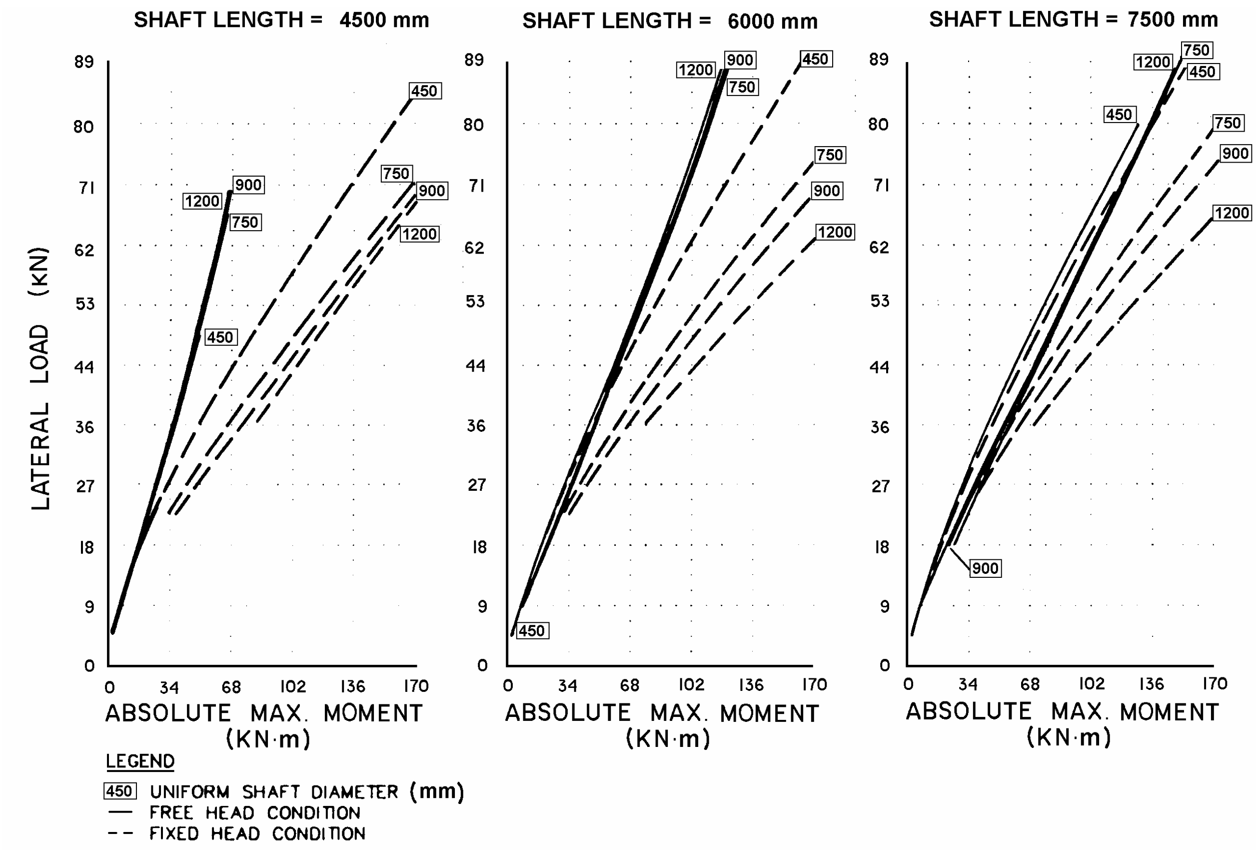


Figure 2.2-9 - COM624 Analyses for Preliminary Design
Lateral Load versus Absolute Maximum Moment, Static Loading, Subsurface Profile 3

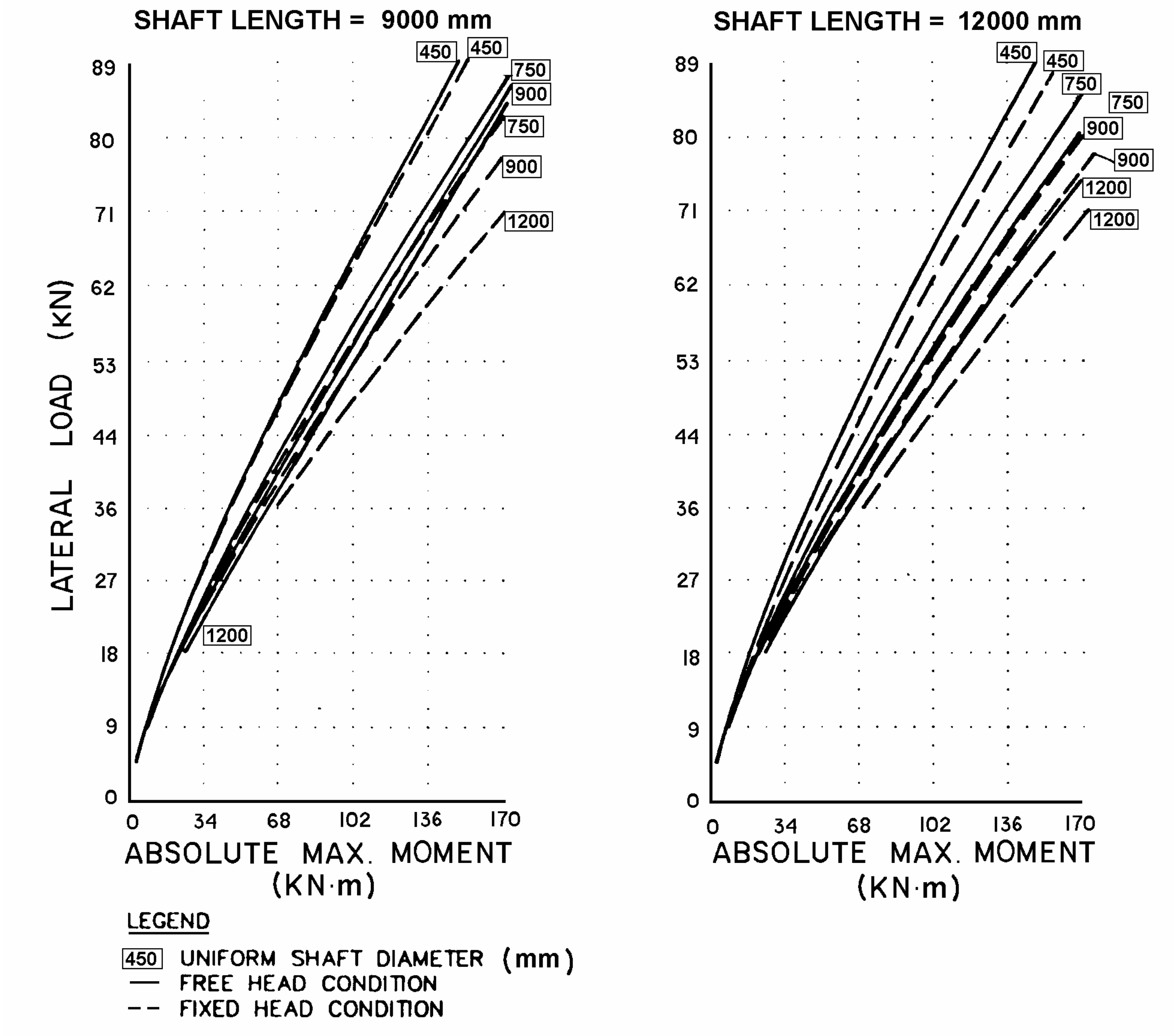


Figure 2.2-9 - COM624 Analyses for Preliminary Design
Lateral Load versus Absolute Maximum Moment, Static Loading, Subsurface Profile 3 (Continued)

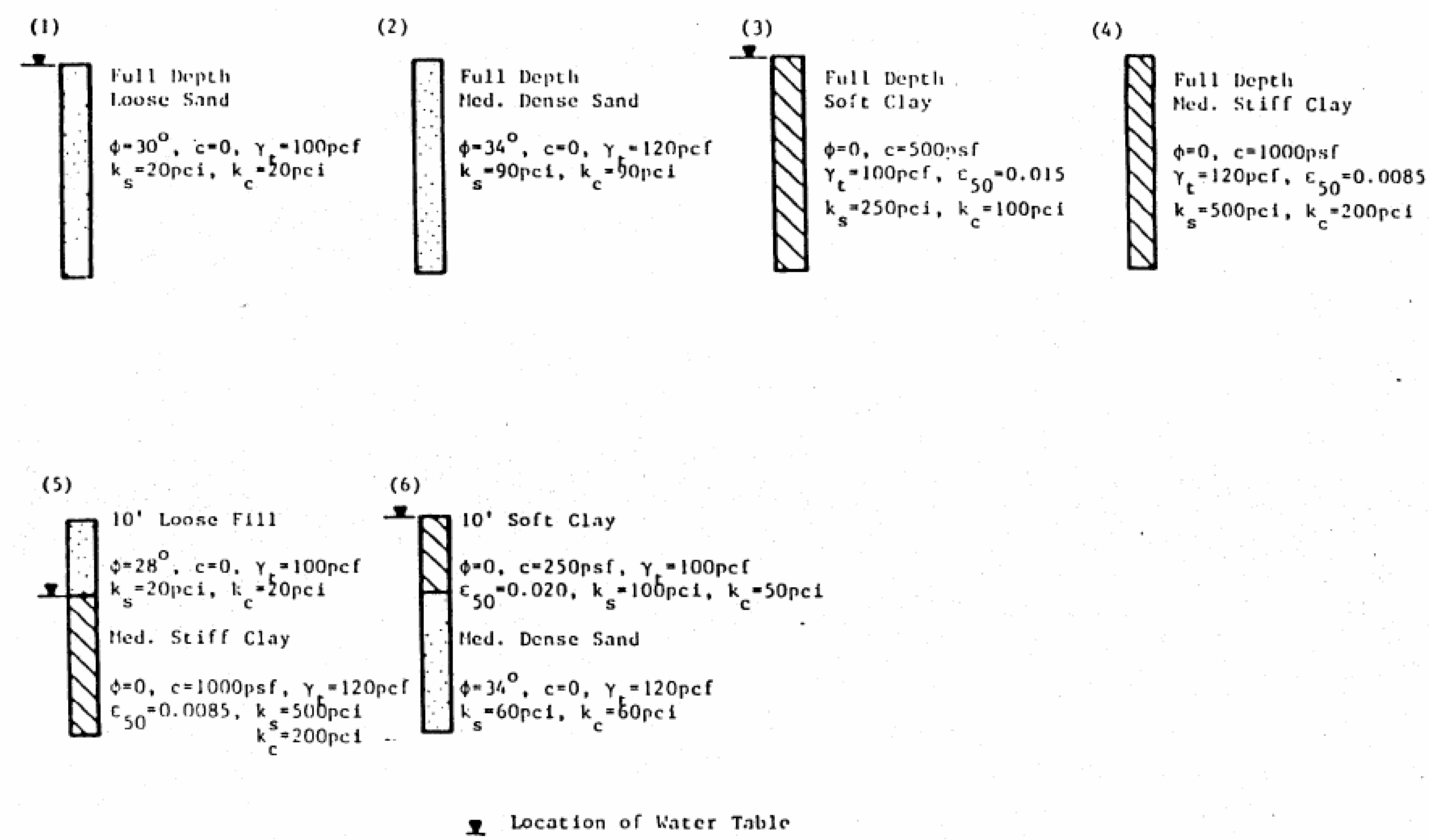


Figure 2.2-1 - Soil Profiles for COM624 Analyses

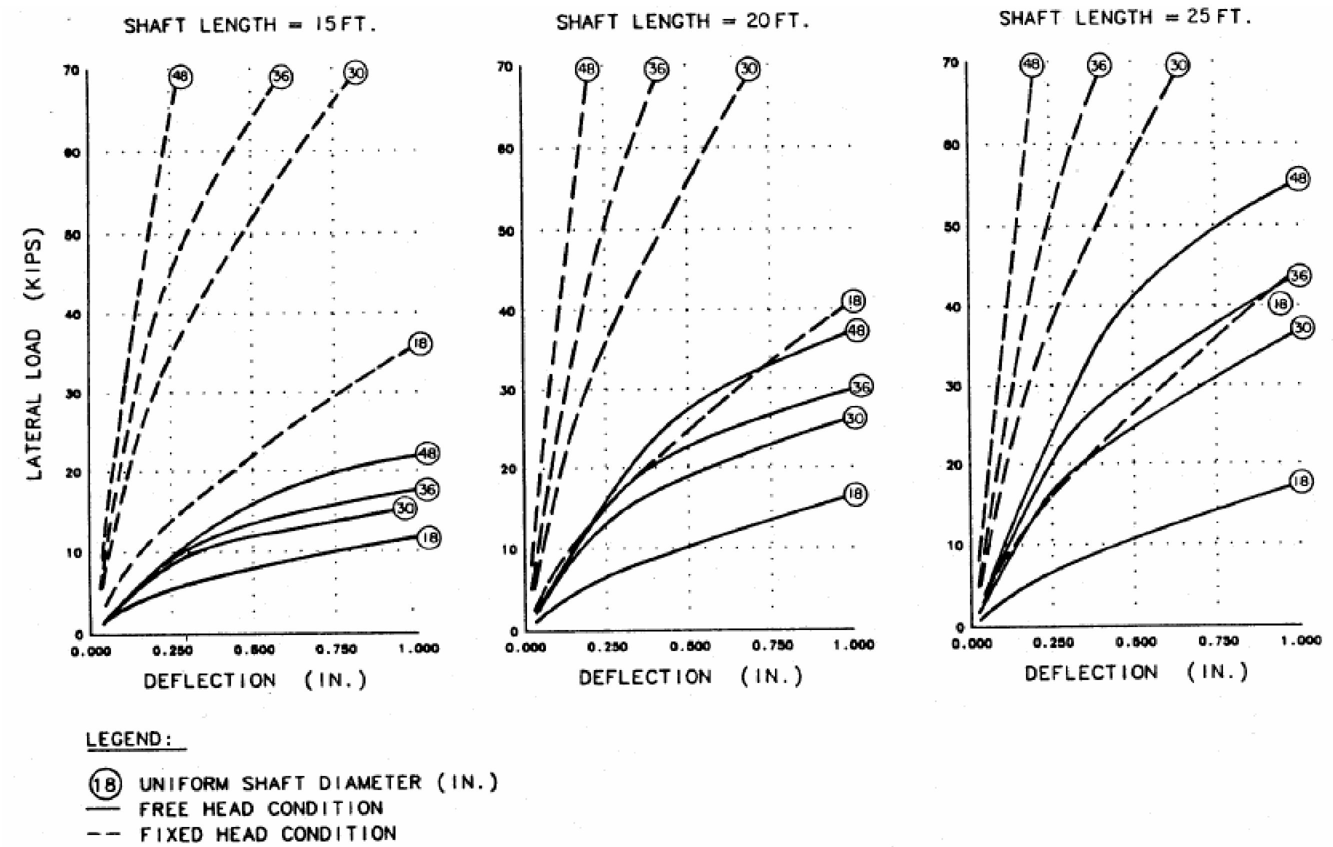


Figure 2.2-2A - COM624 Analyses for Preliminary Design
Lateral Load Versus Lateral Deflection, Static Loading, Subsurface Profile 1

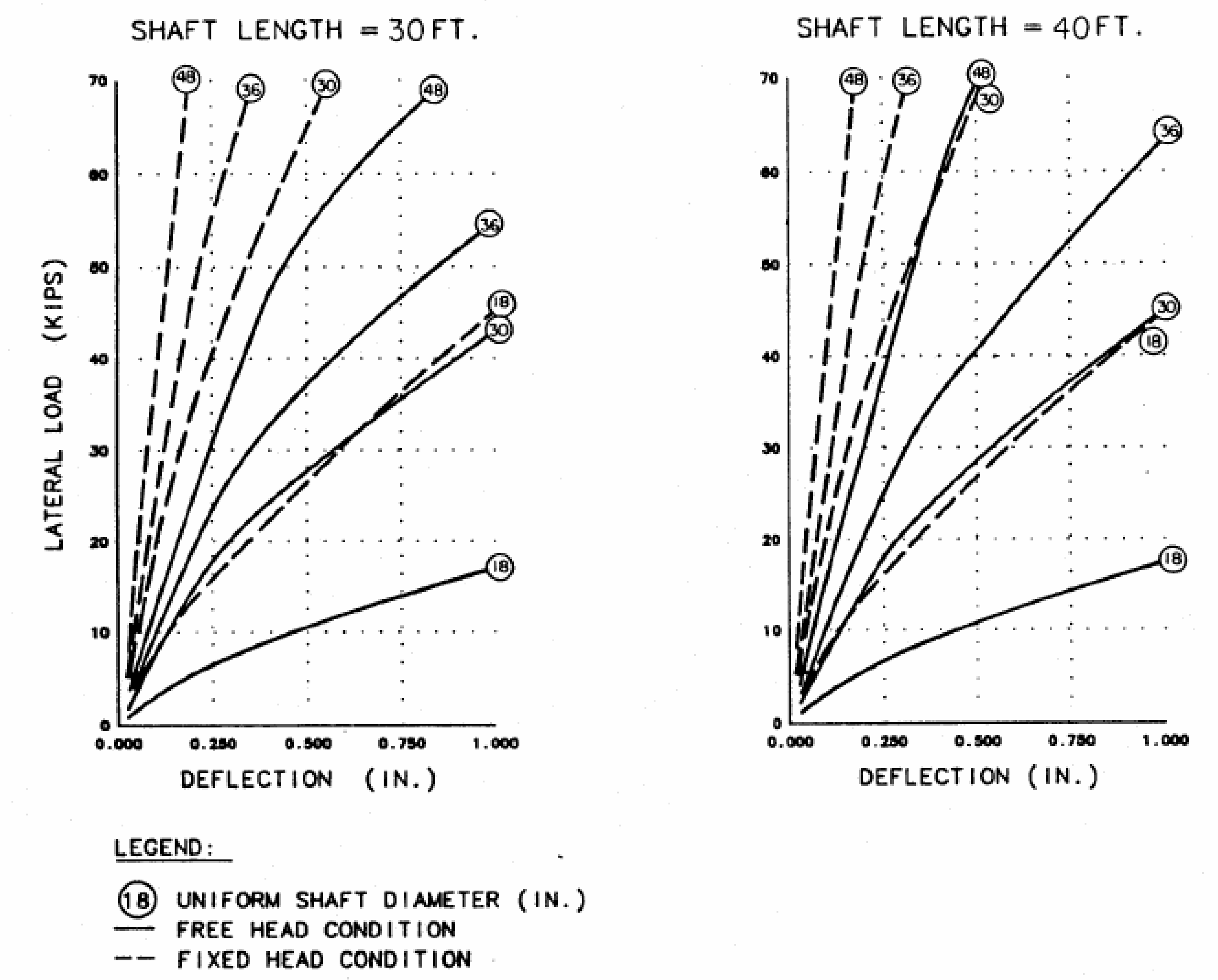


Figure 2.2-2B - COM624 Analyses for Preliminary Design
Lateral Load Versus Lateral Deflection, Static Loading, Subsurface Profile 1 (Continued)

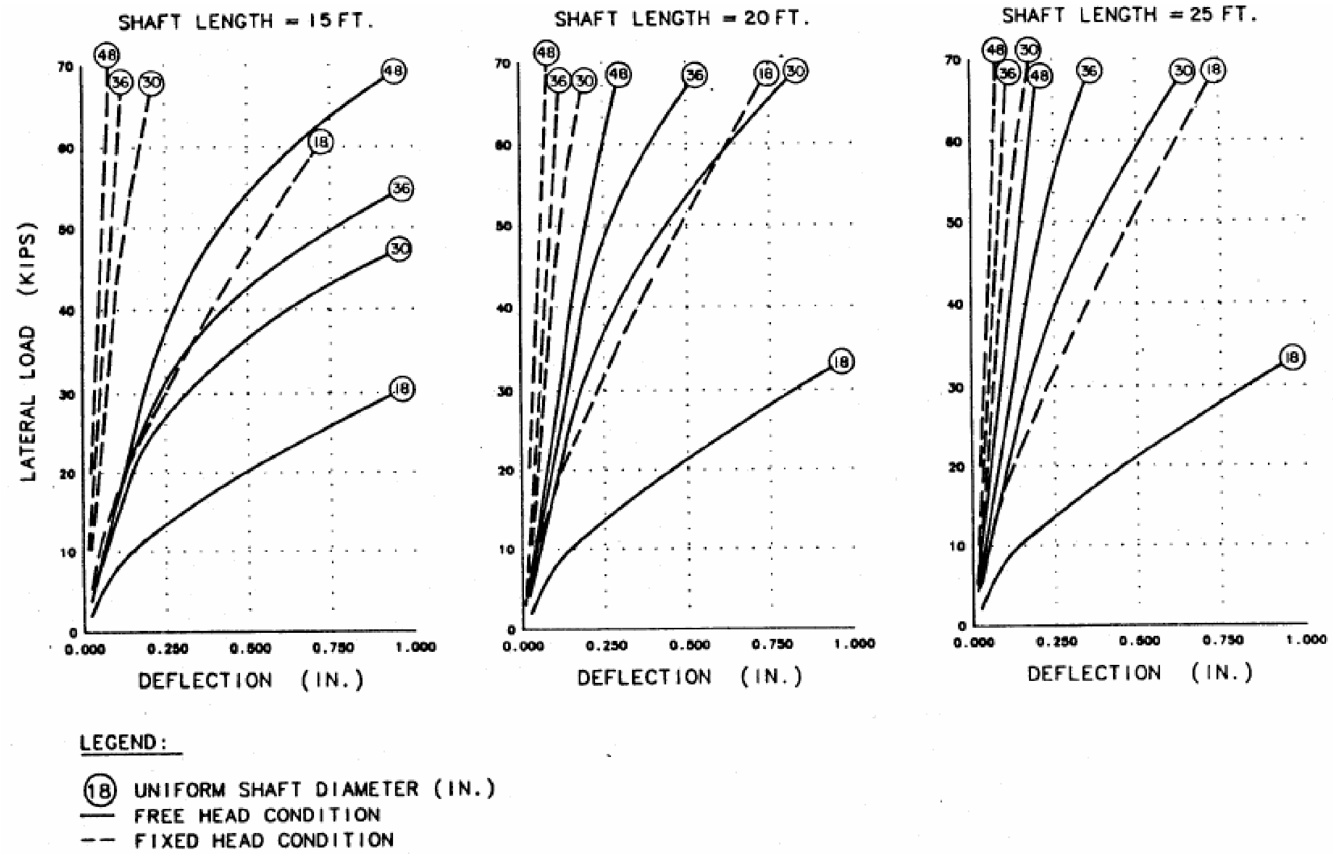


Figure 2.2-3A - COM624 Analyses for Preliminary Design
Lateral Load Versus Lateral Deflection, Static Loading, Subsurface Profile 2

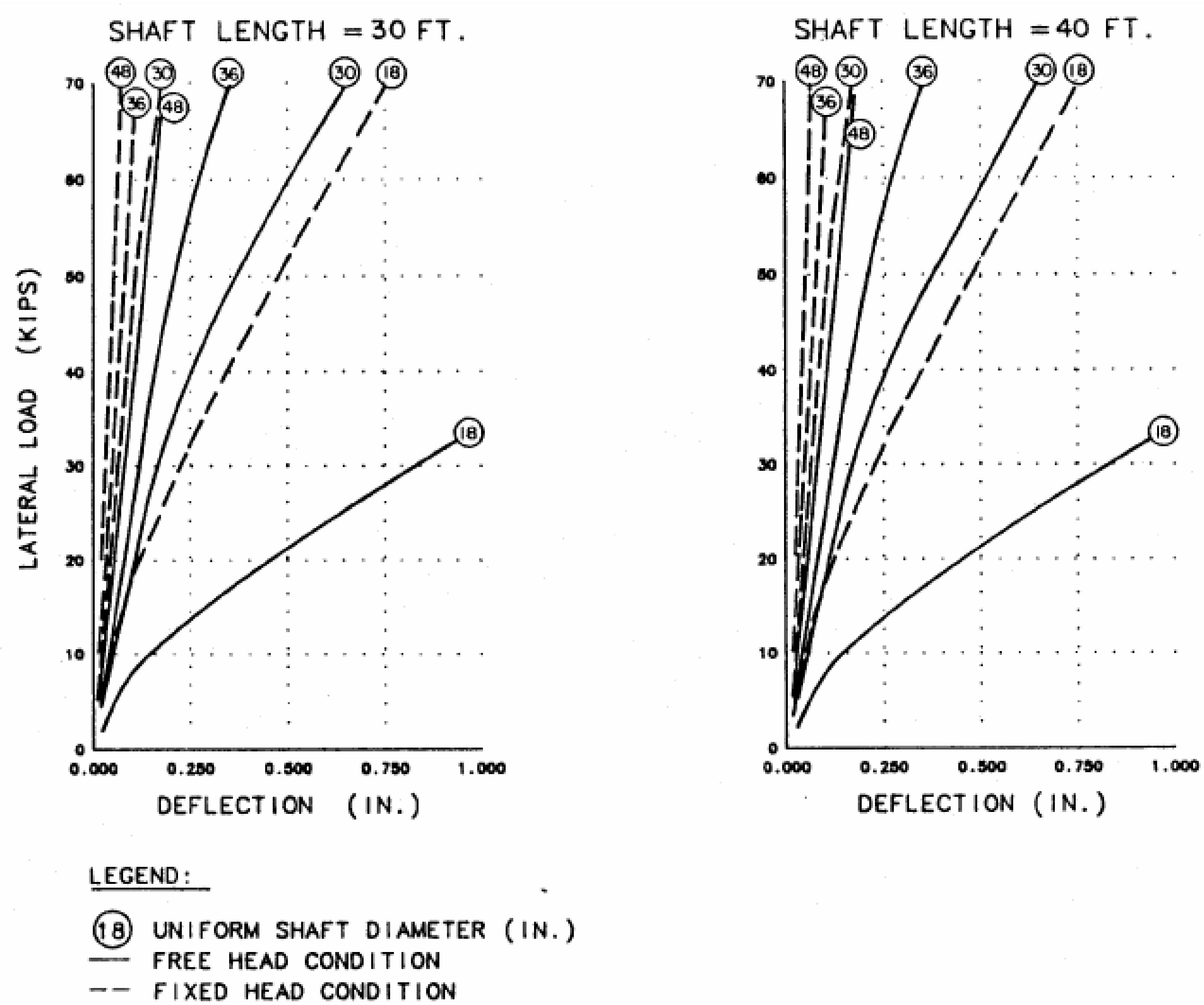


Figure 2.2-3B - COM624 Analyses for Preliminary Design
Lateral Load Versus Lateral Deflection, Static Loading, Subsurface Profile 2 (Continued)

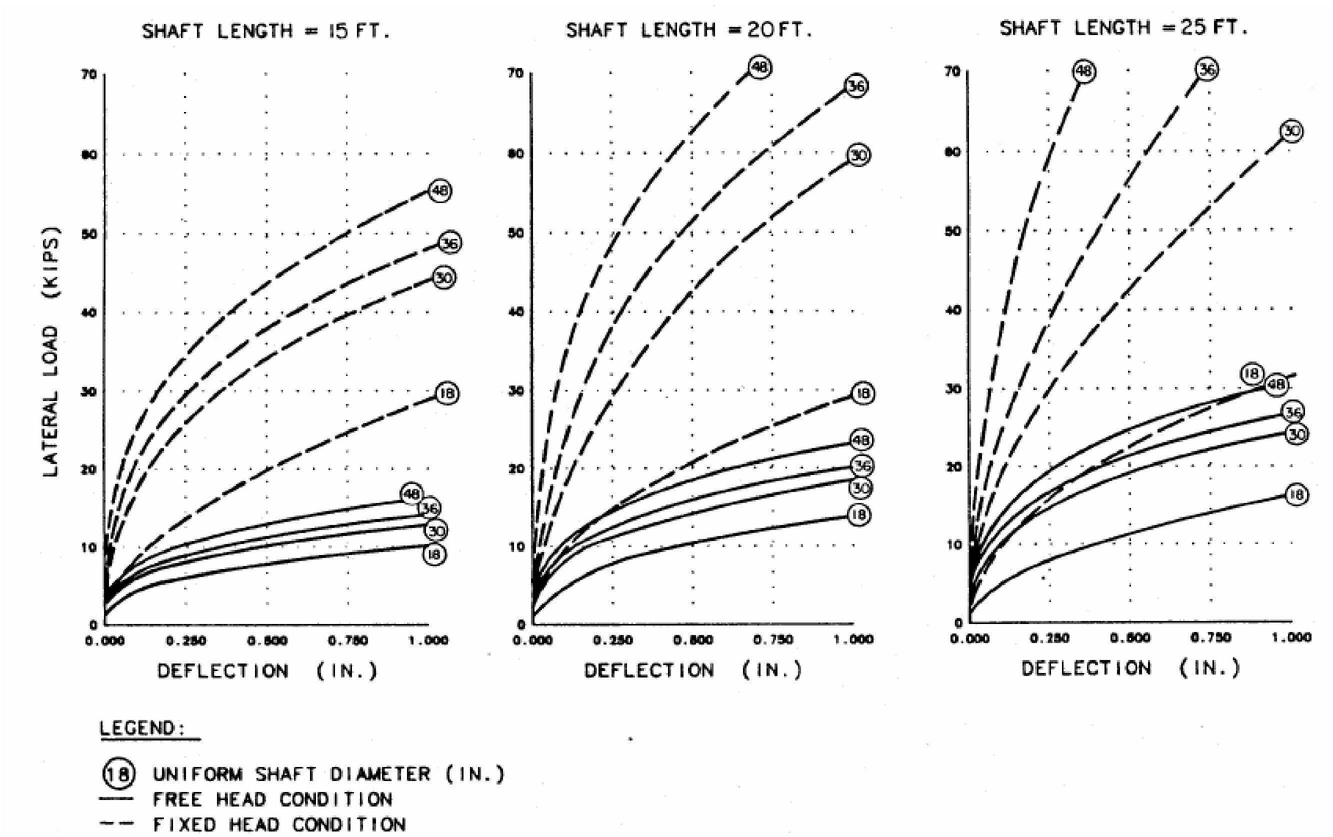


Figure 2.2-4A - COM624 Analyses for Preliminary Design
Lateral Load Versus Lateral Deflection, Static Loading, Subsurface Profile 3

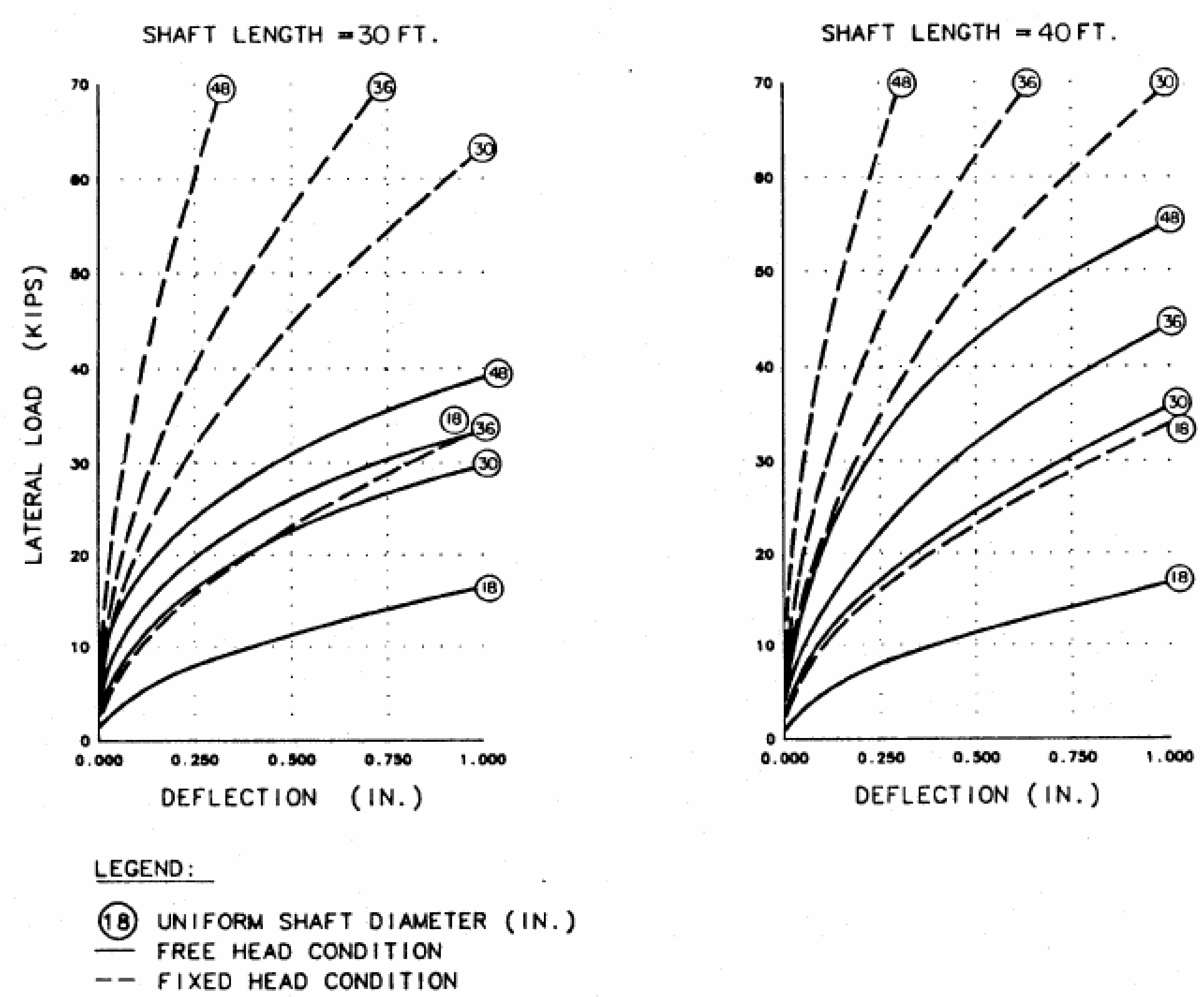


Figure 2.2-4B - COM624 Analyses for Preliminary Design
Lateral Load Versus Lateral Deflection, Static Loading, Subsurface Profile 3 (Continued)

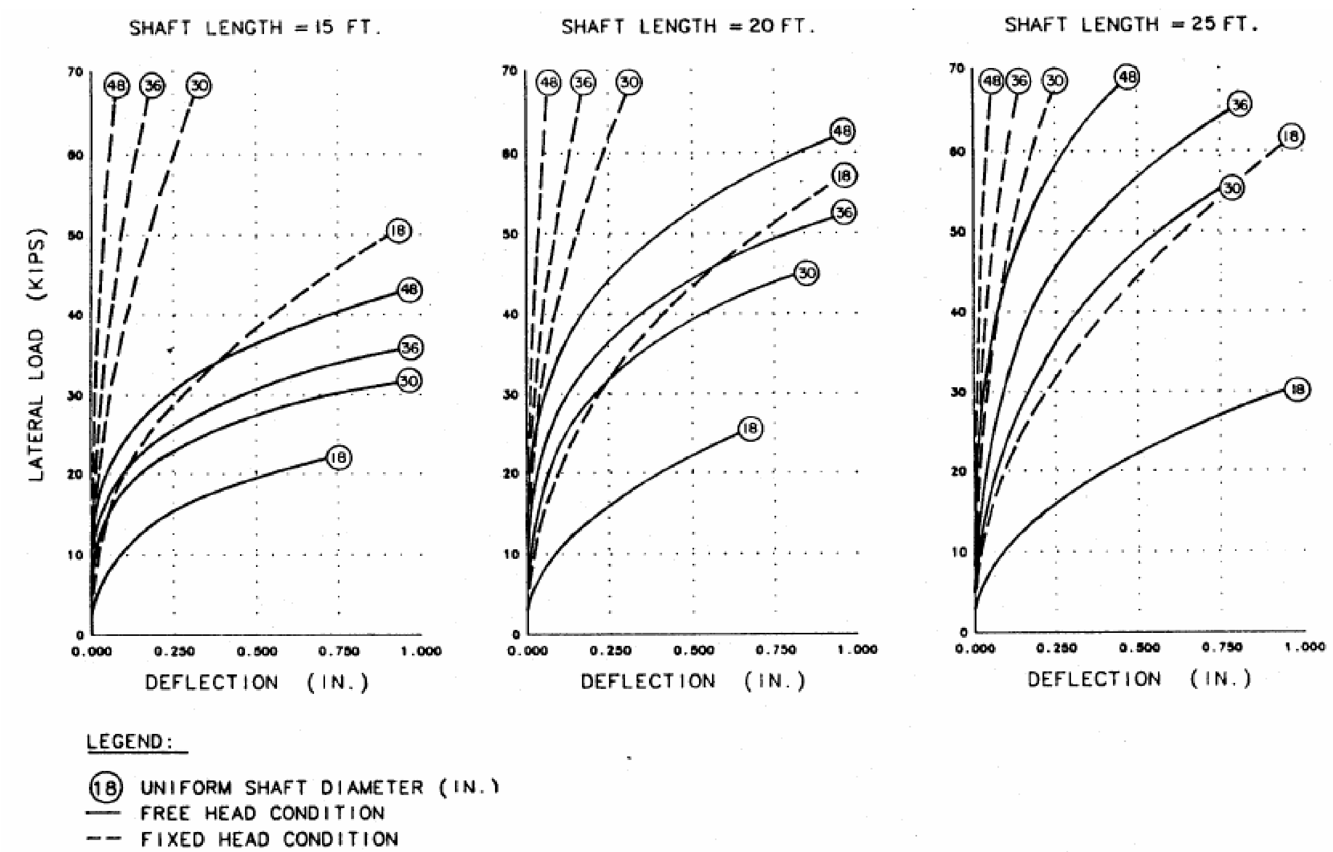


Figure 2.2-5A - COM624 Analyses for Preliminary Design
Lateral Load Versus Lateral Deflection, Static Loading, Subsurface Profile 4

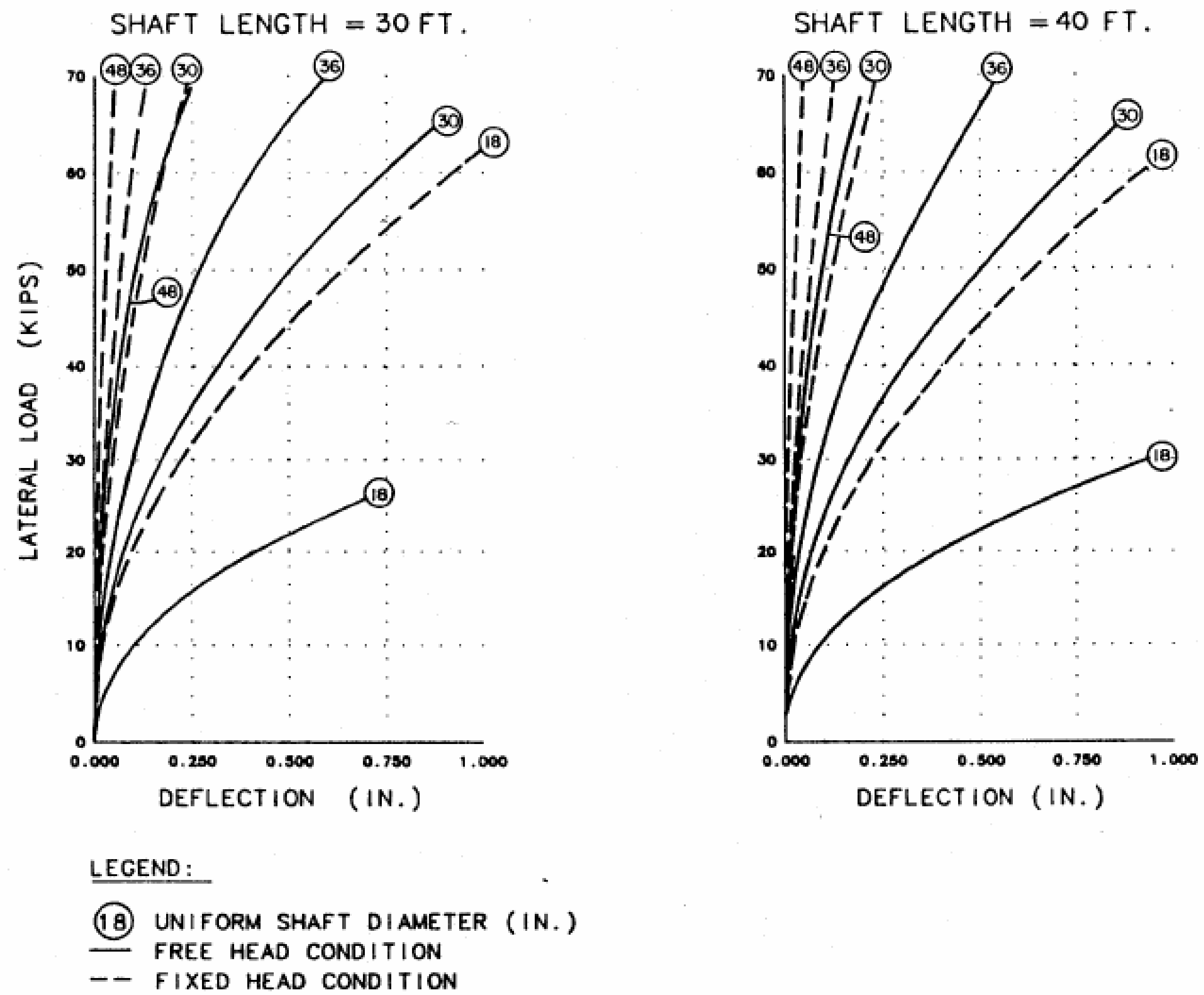


Figure 2.2-5B - COM624 Analyses for Preliminary Design
Lateral Load Versus Lateral Deflection, Static Loading, Subsurface Profile 4 (Continued)

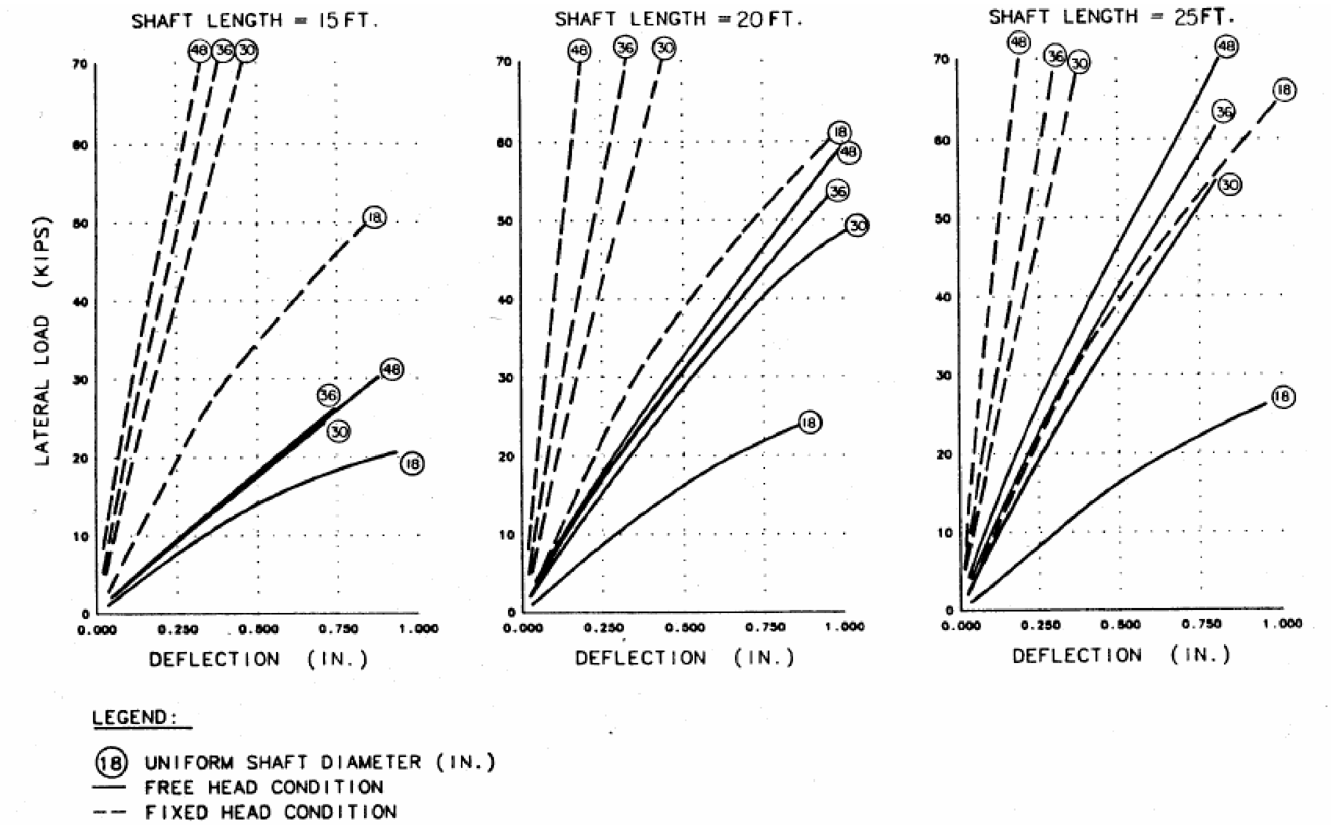


Figure 2.2-6A - COM624 Analyses for Preliminary Design
Lateral Load Versus Lateral Deflection, Static Loading, Subsurface Profile 5

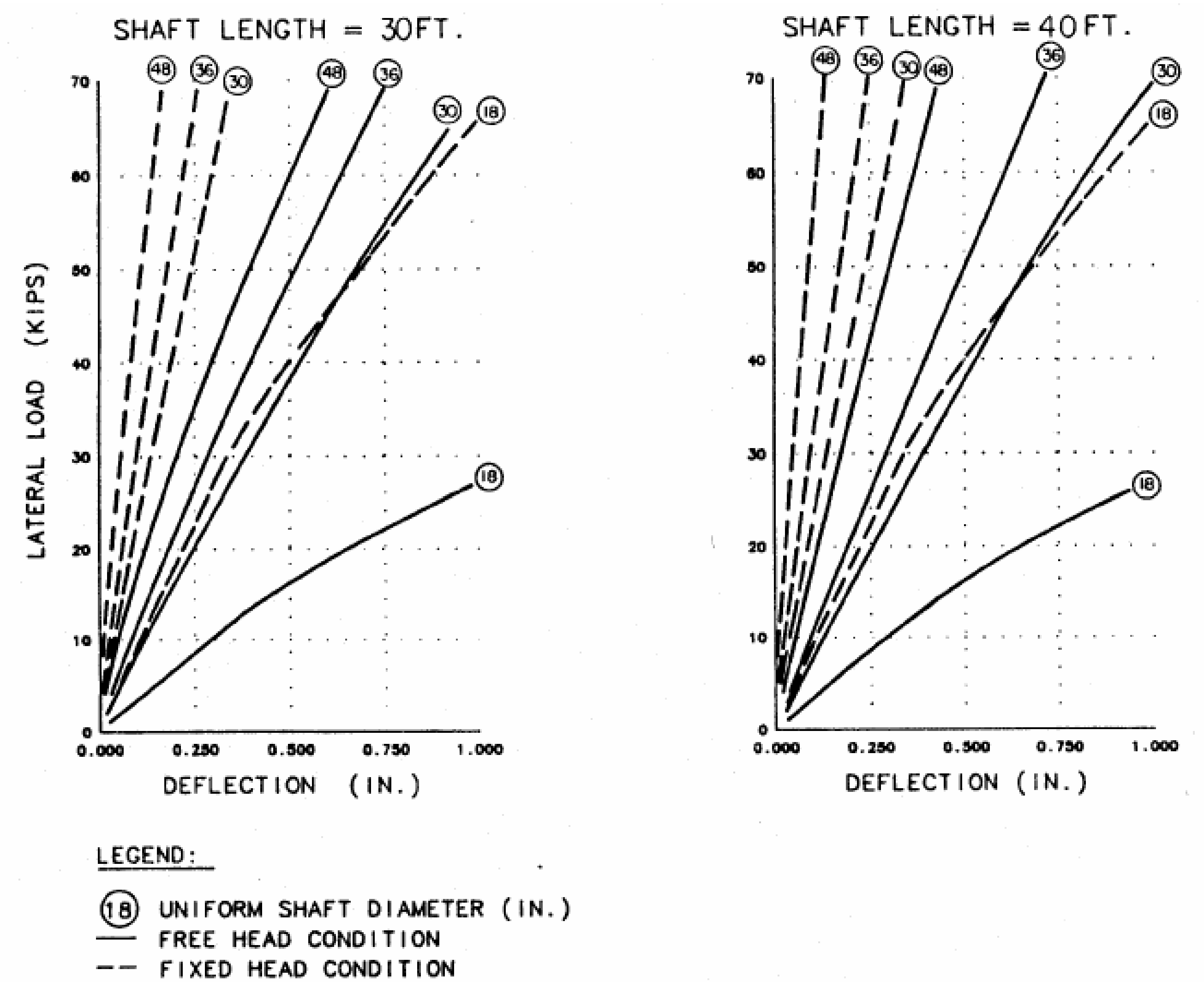


Figure 2.2-6B - COM624 Analyses for Preliminary Design
Lateral Load Versus Lateral Deflection, Static Loading, Subsurface Profile 5 (Continued)

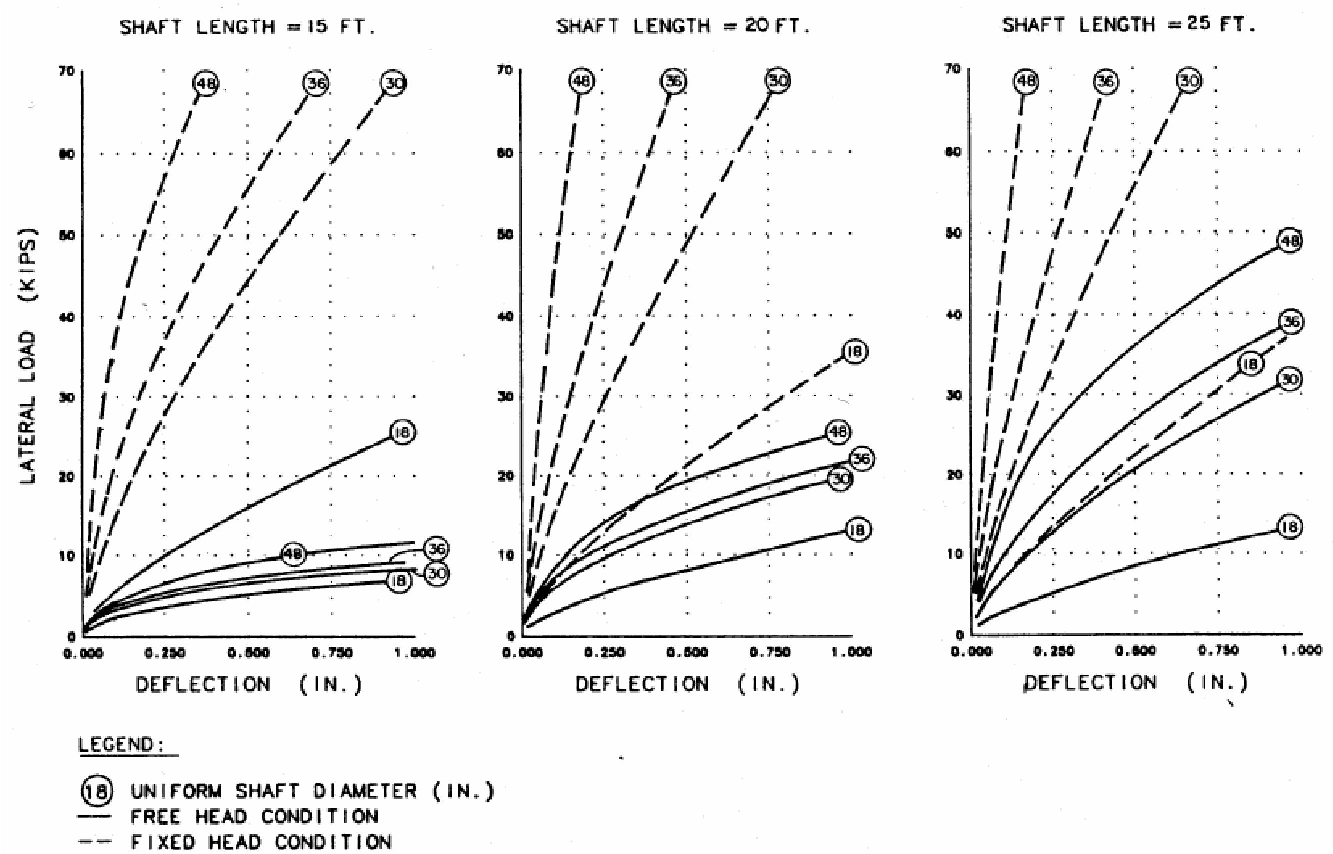


Figure 2.2-7A - COM624 Analyses for Preliminary Design.
Lateral Load versus Lateral Deflection, Static Loading, Subsurface Profile 6

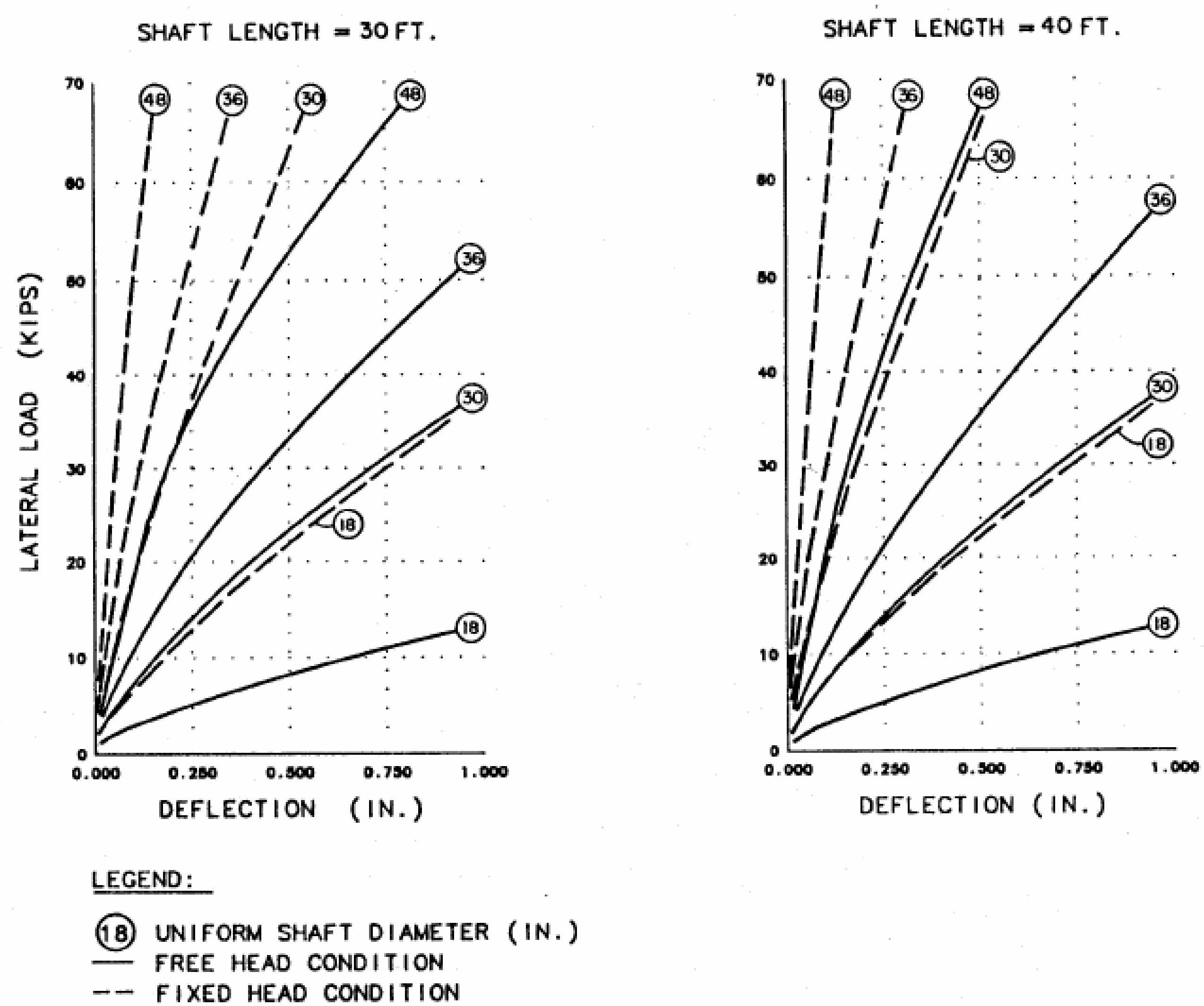


Figure 2.2-7B - COM624 Analyses for Preliminary Design.
Lateral Load versus Lateral Deflection, Static Loading, Subsurface Profile 6 (Continued)

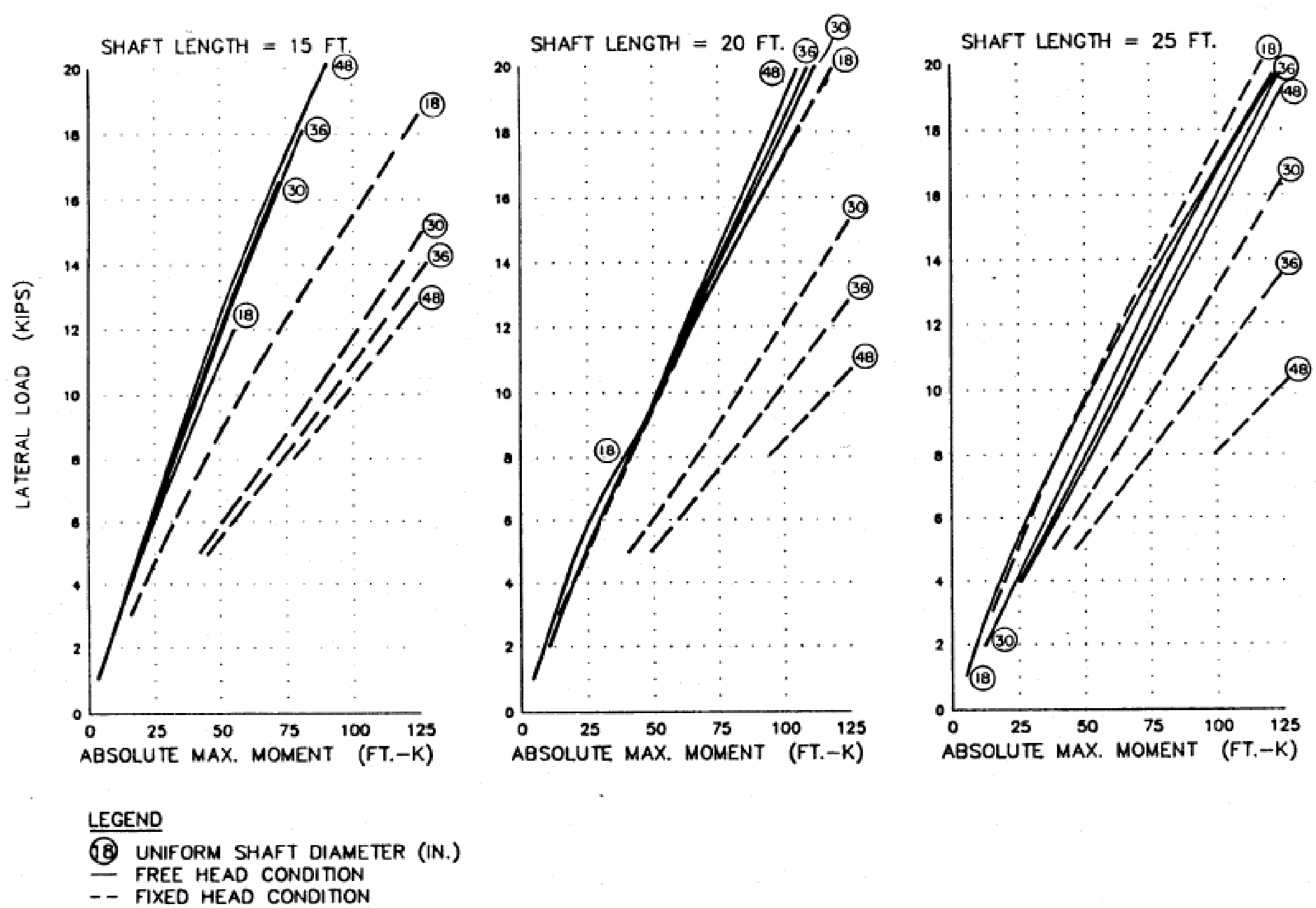


Figure 2.2-8A - COM624 Analyses for Preliminary Design
Lateral Load versus Absolute Maximum Moment, Static Loading, Subsurface Profile 1

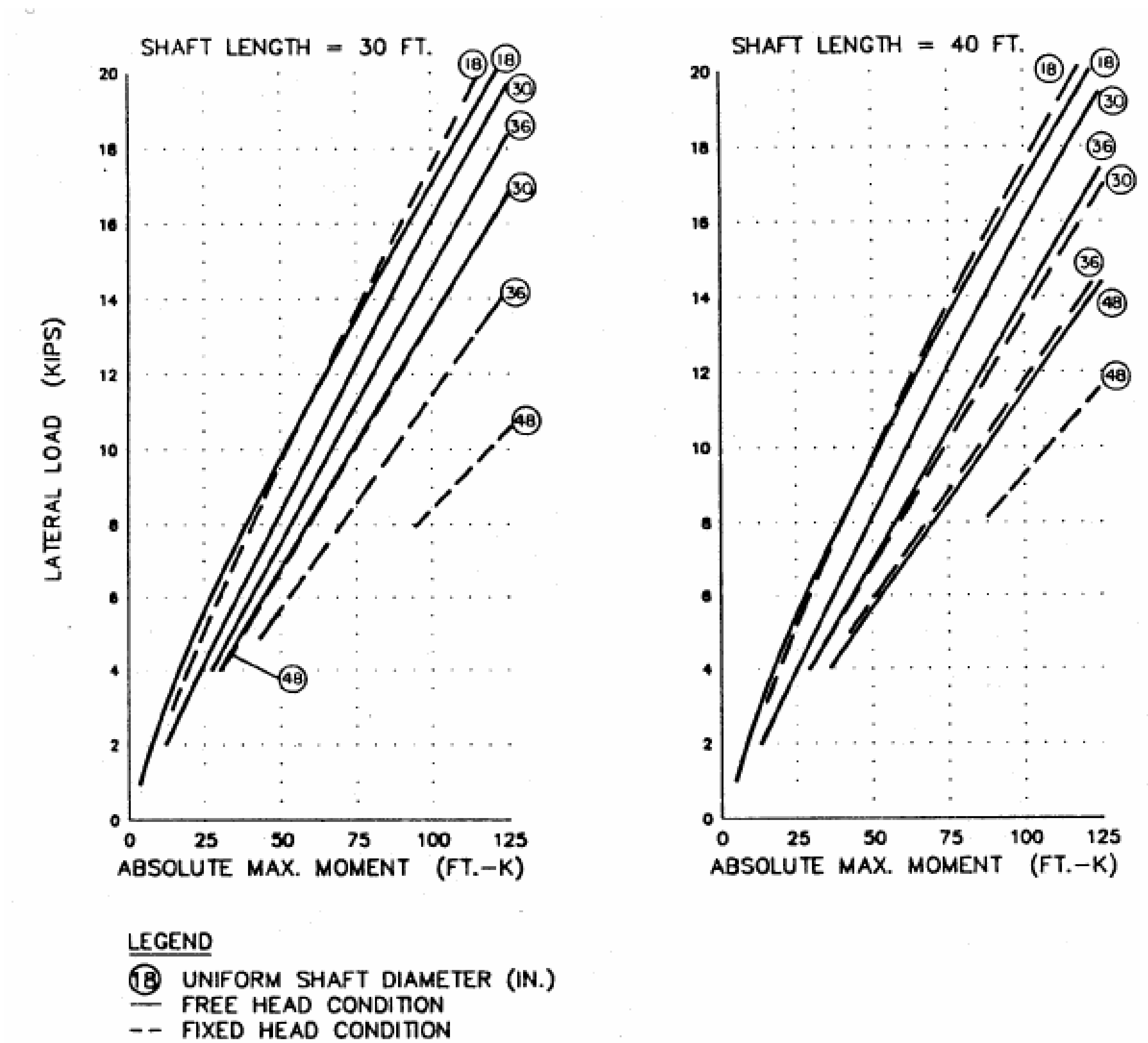


Figure 2.2-8B - COM624 Analyses for Preliminary Design
Lateral Load versus Absolute Maximum Moment, Static Loading, Subsurface Profile 1 (Continued)

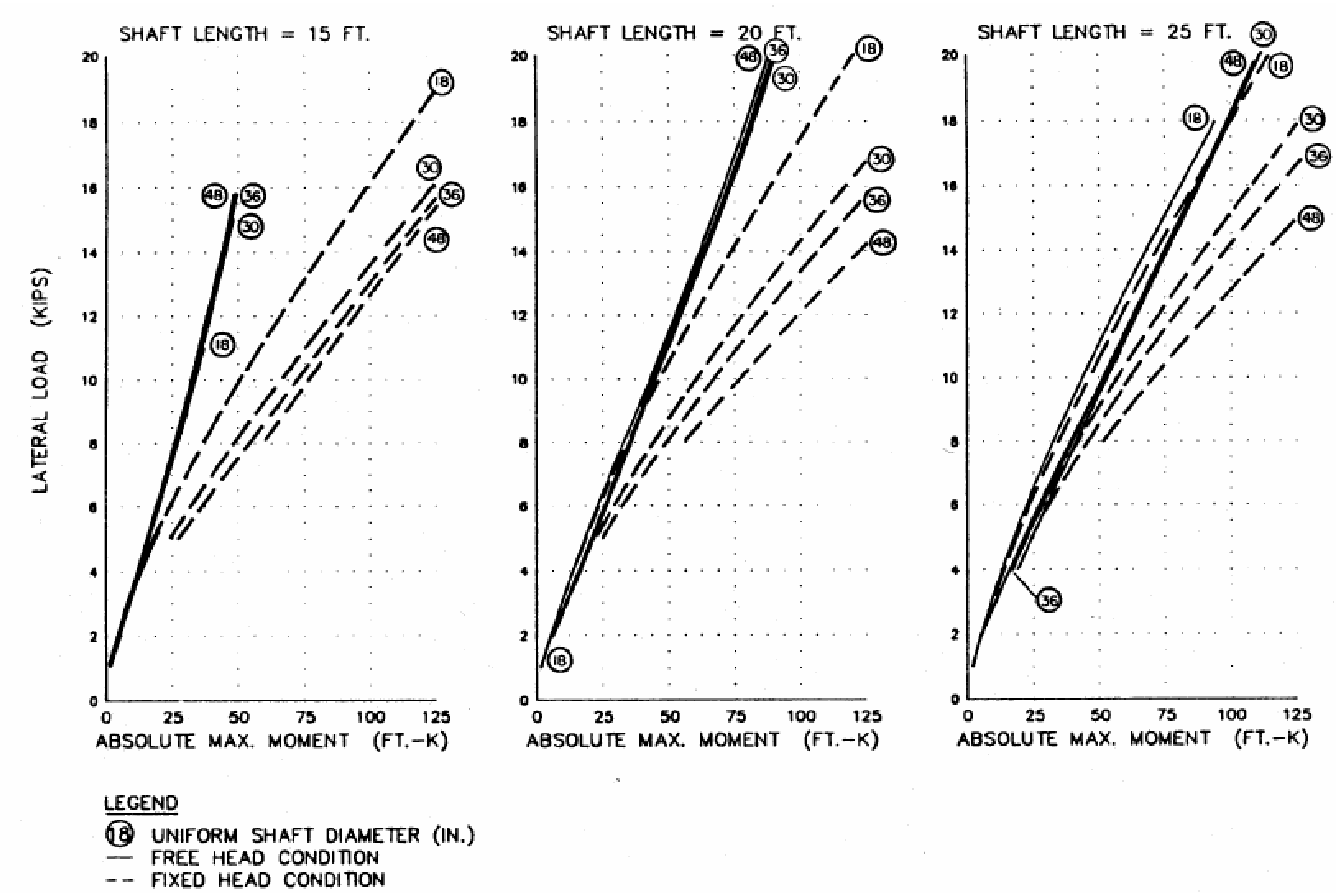


Figure 2.2-9A - COM624 Analyses for Preliminary Design
Lateral Load versus Absolute Maximum Moment, Static Loading, Subsurface Profile 3

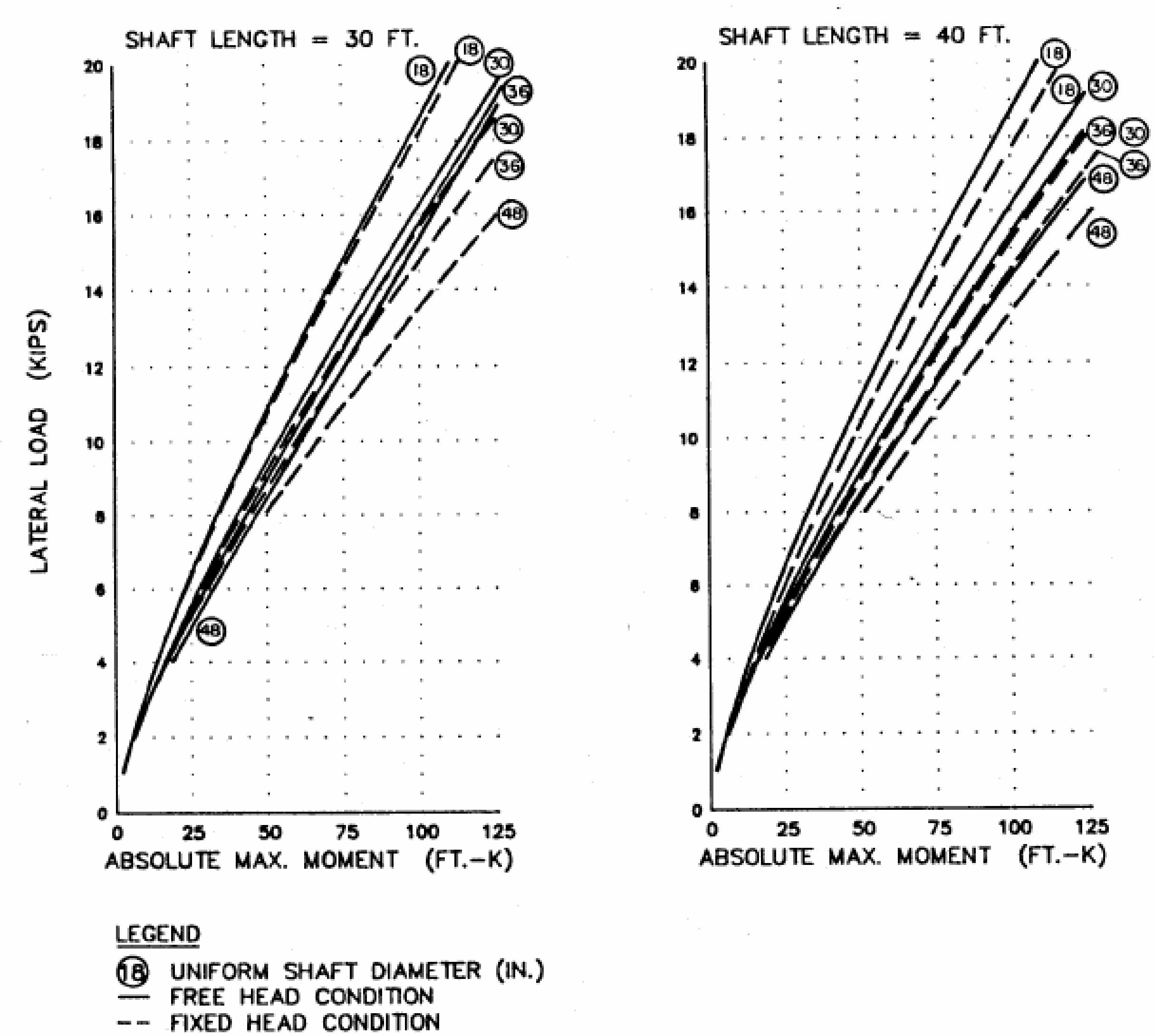


Figure 2.2-9B - COM624 Analyses for Preliminary Design
Lateral Load versus Absolute Maximum Moment, Static Loading, Subsurface Profile 3 (Continued)

Table 2.2-1 - Properties of Drilled Shafts used for COM624 Analyses

Nominal Diameter B (mm) {in}	Cross-Sectional Area A (mm^2) {in^2}	Gross Moment of Inertia I_c ($x10^9$ mm^4) {in^4}	Stiffness EcIc $N\text{-}mm^2$ {$lb\text{-}in^2$}
450 {18}	164 000 {254}	2.145 {5,153}	$60.1x10^{12}$ {$2.08x10^{10}$}
750 {30}	456 000 {707}	16.55 {39,761}	$463x10^{12}$ {$16.0x10^{10}$}
900 {36}	657 000 {1,018}	34.32 {82,448}	$961x10^{12}$ {$33.2x10^{10}$}
1200 {48}	1168 000 {1,810}	108.46 {260,576}	$3037x10^{12}$ {$105.0x10^{10}$}

NOTES:

1. Shaft lengths: D = 4, 5, 6, 7.5, 9, and 12 meters {15, 20, 25, 30, 40 ft}
2. 28-day compressive strength of concrete: f'_c = 34 MPa {5000 psi}
3. Modulus of elasticity of concrete: $E_c = 4800\sqrt{f'_c}$ {$57,000\sqrt{f'_c}$} = 28 000 MPa {$4.03x10^6$ psi}
4. No reinforcement included

Table 2.2-2 - Comments on Input for Computer Program COM624

Item	Evaluation	Comments
Number of Shaft Increments	Determined as: D/B + 15	The effects of the shaft increment were examined and the expression was found to yield values leading to accurate solutions for the ground conditions modeled.
p-y Curves	Generated internally using: • Matlock [17] criteria for soft clay • Reese, et al.[4] criteria for stiff clay below the water table • Reese and Welch [5] criteria for stiff clay above the water table • Reese, et al, [6] criteria for sand	The user should be familiar with the methods of developing the load-deformation response for different soils presented in Section 2.3.1.1.1 noting the limiting assumptions.
Location of Butt of Shaft with Respect to Ground Surface	Assumed	Butt of the shaft was assumed level with the ground surface for all cases.
Soil properties: unit weight, angle of internal friction, undrained shear strength, strain at 50% stress level constant n_h in $E_s = n_h z$	Based on assumed soil type	Combinations of soil properties for the assumed soil types (i.e., loose sand, stiff clay) were chosen on the basis of consistency between properties and representativeness of soil type.
Lateral Loads	Variable incremental increase, at least to a level inducing a ground line deflection greater than 12 mm {0.5 in}.	Lateral loads were concentrated at the butt of the shaft. No distributed lateral loads were incorporated.

2.3 STRUCTURAL DESIGN AND GENERAL SHAFT DIMENSIONS

2.3.1 General

2.3.1.1 FINAL DESIGN

2.3.1.1.1 COM624 Design Procedure

The computer program COM624 incorporates nonlinear load-deformation (p-y) curves to model the soil-foundation interaction, and adopts a finite-difference scheme to solve the problems. COM624 is capable of generating p-y curves internally using an input subsurface description with corresponding soil/rock properties, or can accept predetermined load-deformation curves for the soil/rock units. The method yields p-y data for the subsurface profile at specified increments along the shaft, if requested, and also computes the lateral deflection, rotation, moment, and shear at desired points along the foundation for a given loading. Recommendations by Reese [5] for constructing p-y curves for sand and clay are presented in 2.3.1.1.1.1 through 2.3.1.1.1.5. The recommended procedures for developing the p-y curves are derived from experimental studies, so inherent limitations exist where empirical coefficients are recommended based on particular lateral load tests.

The design of laterally loaded drilled shafts based on the deformation characteristics of the surrounding medium requires development of p-y curves for the soil and/or rock. The initial step in formulating nondimensional and computer solutions is the construction of representative p-y curves at various depths along the embedment of the shaft.

Theoretical soil mechanics and engineering experience provide the basis for developing p-y curves. Actual methods of constructing such curves arise from duplication of load-deformation behavior observed during lateral load tests. The methods are typically generalized through correlations with soil properties, shaft diameter, and depth, for broader application.

Because it is difficult to determine an exact percent of fixity and because increased shaft head fixity results in decreased ground line deflections, percent fixities somewhat less than those expected to be achieved are applied in deflection analysis of laterally-loaded drilled shafts so that the results obtained are somewhat conservative. It should be noted that the moment conditions at the two extremes of head fixity present the worst cases for consideration of the structural requirements of the shaft.

2.3.1.1.1.1 p-y Curves for Soft Clay Extending to the Ground Surface

(a) The following procedure can be used for developing p-y curves for short-term static loading as illustrated in Figure 2.3.1.1.1.1-1 [5].

(1) Obtain the best possible estimate of the variation with depth of undrained shear strength, c, and submerged unit weight, γ'.. Also obtain the values of the strain, ε_{50}, corresponding to one-half the maximum principal-stress difference. If no stress-strain curves are available, assume a value of ε_{50} equal to 0.020, 0.010, and 0.005 for soft, medium, and stiff clay, respectively.

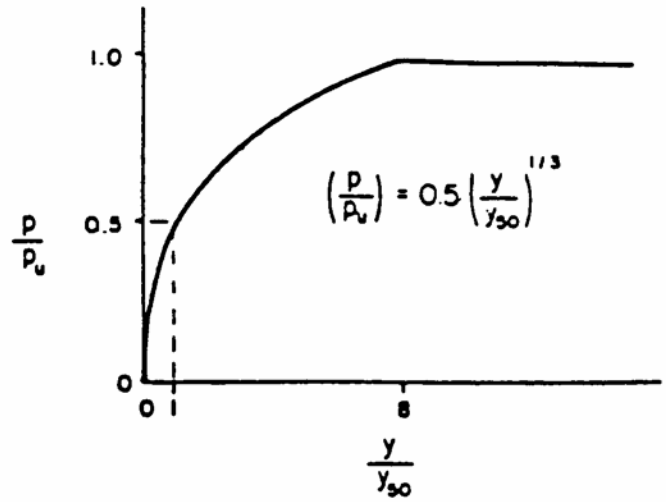
(2) Compute the ultimate soil resistance per unit length of shaft, using the smaller of the values given by the equations below.

$$P_u = [3 + \frac{\gamma'}{c} z + \frac{J}{B} z\,]\,cB$$

$$P_u = 9\,c\,B$$

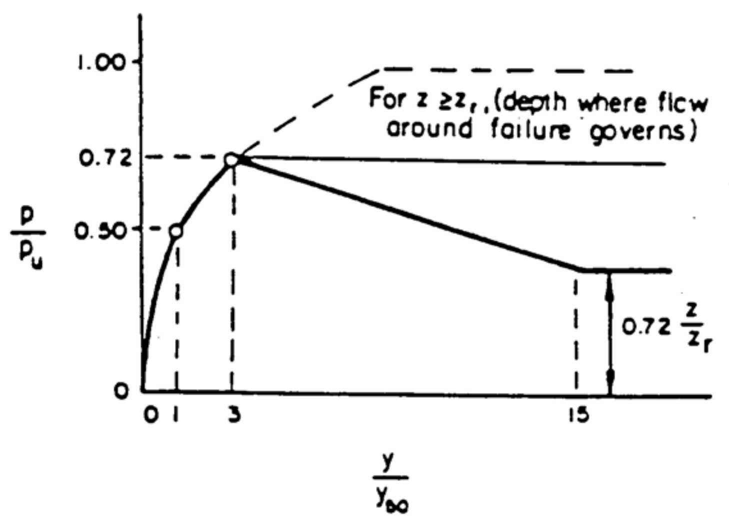
where:

z = Depth from the ground surface to the p-y curve
γ' = Average effective unit weight from the ground surface to depth, z
c = Undrained shear strength at depth z
B = Width of the shaft
J = Empirical dimensionless parameter



Source: Reference 3

Figure 2.3.1.1.1.1-1 - Characteristic Shape of p-y Curves for Static Loading of Soft Clay below Water Table



Source: Adapted from Reference 19

Figure 2.3.1.1.1.1-2 Characteristic Shape of p-y Curves for Cyclic Loading of Soft Clay below Water Table

Matlock [17] states that the value of J was determined experimentally to be 0.5 for a soft clay and about 0.25 for a medium clay. A value of 0.5 is frequently used for J. The value of p_u is computed at each depth where a p-y curve is desired, based on shear strength at that depth. COM624 develops values of y and the corresponding p values at close spacings; if hand computations are being used, p-y curves shall be computed at depths to reflect the soil profile. If the soil is homogeneous, the p-y curves shall be obtained at close spacings near the ground surface where the shaft deflection is greater.

(3) Compute the deflection, y_{50}, at one-half the ultimate soil resistance from the following equation:

$$y_{50} = 2.5\ \varepsilon_{50}\ B$$

(4) Points describing the p-y curve are computed from the following relationship:

$$\frac{P}{P_u} = 0.5\left(\frac{y}{y_{50}}\right)^{1/3}$$

The value of p remains constant beyond $y = 8\ y_{50}$.

(b) The following procedure is for cyclic loading and is illustrated in Figure 2.3.1.1.1.1-2.

(1) Construct the p-y curve in the same manner as for short-term static loading for values of p less than $0.72\ p_u$.

(2) Solve the first two equations in 2.3.1.1.1.1 simultaneously to find the depth, z_r, where the transition occurs from a wedge-type failure to a flow-around failure. If the unit weight and shear strength are constant in the upper zone, then

$$z_r = \frac{6\ c\ B}{\left(\gamma' B + J_c\right)}$$

If the unit weight and shear strength vary with depth, the value of z_r shall be computed with the soil properties at the depth where the p-y curve is desired.

(3) If the depth to the p-y curve is greater than or equal to z_r, then p is equal to $0.72\ p_u$ for all values of y greater than $3\ y_{50}$.

(4) If the depth to the p-y curve is less than z_r, then the value of p decreases from $0.72\ p_u$ at $y = 3\ y_{50}$ to the value given by the following expression at $y = 15\ y_{50}$.

$$p = 0.72\ p_u \left(\frac{z}{z_r}\right)$$

The value of p remains constant beyond $y = 15\ y_{50}$.

(c) Recommended Soil Tests

For determining the various shear strengths of the soil required in the p-y construction, Matlock [17] recommended the following tests in order of preference:

(1) In situ vane-shear tests with parallel sampling for soil identification

(2) Unconsolidated-undrained triaxial compression tests having a confining stress equal to the overburden pressure, with c being defined as half the total maximum principal stress difference

(3) Miniature vane tests of samples in tubes

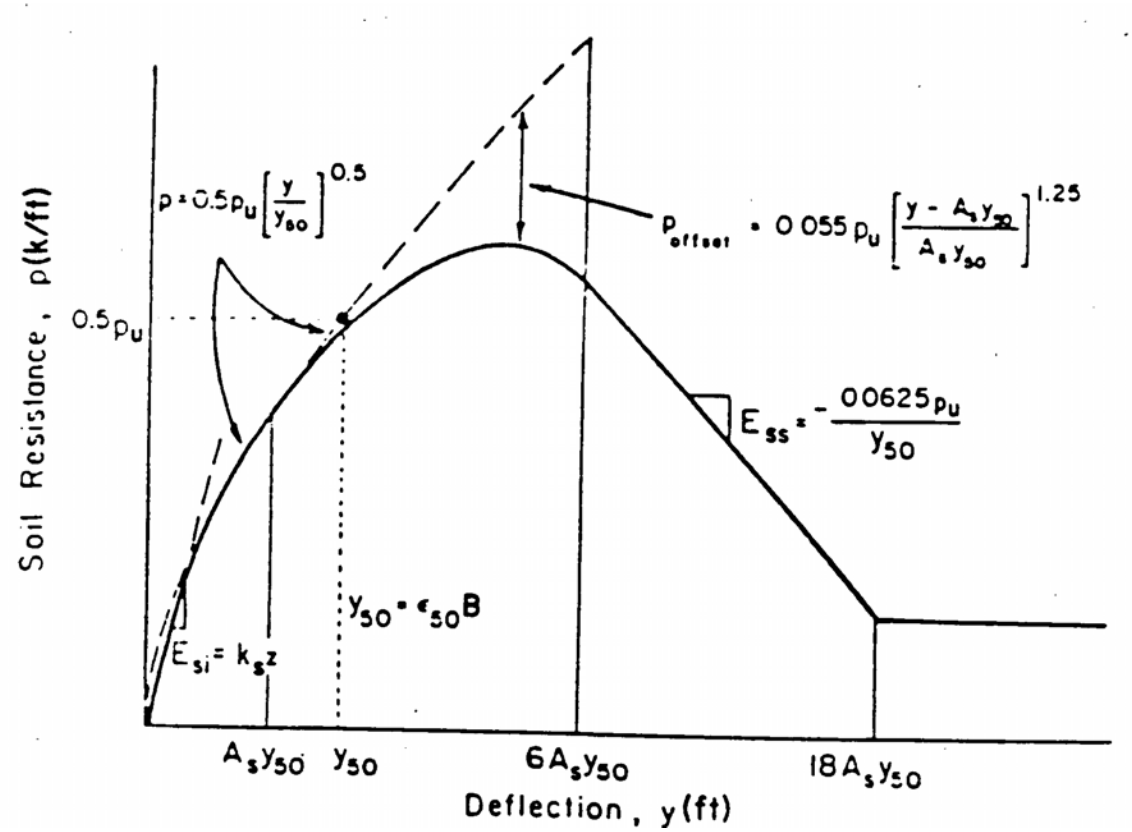
(4) Unconfined compression tests

Matlock's recommendations are related to the determination of the shear strength of the clay. If field vane tests are performed, for example, other tests shall be performed to obtain ε_{50} and the unit weight for the soil.

In the test program, emphasis shall be placed on determining soil properties at or near the ground surface. In general, the farther the soil from the ground surface, the less is its influence on shaft response. For a soil profile that is relatively homogeneous, studies have shown that the soil below 10 shaft diameters has only a small influence on shaft deflection and bending moment.

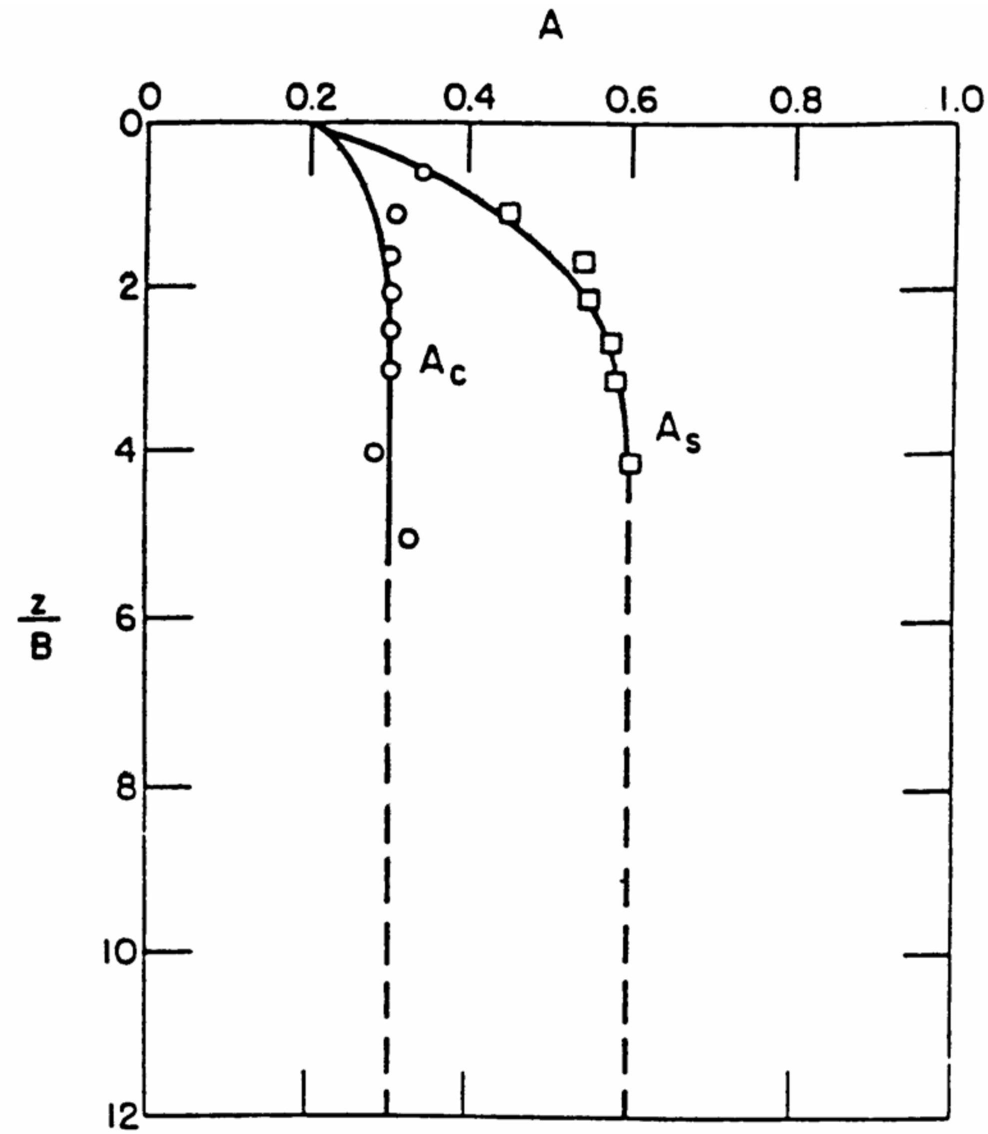
2.3.1.1.1.2 p-y Curves for Stiff Clay Below the Water Table

(a) The following procedure can be used for developing p-y curves for short-term static loading as illustrated in Figure 2.3.1.1.1.2-1. The empirical parameters, A_s and A_c, shown in Figure 2.3.1.1.1.2-2, and values of k_s and k_c are based on the results of experiments. Values of k_s for static loading shall be assumed to equal 66 800, 133 500, and 267 000 kN/m^3 {425, 850 and 1,700 kips per cubic foot (kcf)}for soft, medium, and stiff clay soils, respectively. Values of k_c for cyclic loading shall be assumed to equal 26 700, 55 000, and 110 000 kN/m^3 {170, 350 and 700 kcf} for soft, medium, and stiff clay soils, respectively.



Source: Adapted from Reference 18

Figure 2.3.1.1.1.2-1 - Characteristic Shape of p-y Curves for Static Loading in Stiff Clay below Water Table



Source: Adapted from Reference 5

Figure 2.3.1.1.1.2-2 - Values of Constants A_s and A_c

(1) Obtain the values of the undrained soil shear strength, c, soil submerged unit weight, γ', and shaft diameter, B.

(2) Compute the average undrained soil shear strength, c_a, over the depth, z.

(3) Compute the ultimate soil resistance per unit length of shaft using the smaller of the values given by the equations below:

$$p_u = 2\, c_a\, B + \gamma'\, B\, z + 2.83\, c_a\, z$$
$$p_u = 11\, c\, B$$

(4) Choose the appropriate value of A_s from Figure 2.3.1.1.1.2-2 for the particular nondimensional depth.

(5) Establish the initial straight-line portion of the p-y curve:

$$p = (kz)\, y$$

Use the appropriate value of k_s or k_c for k.

(6) Compute the following:

$$y_{50} = \varepsilon_{50}\, B$$

Use an appropriate value of ε_{50} from results of laboratory test or, in the absence of laboratory tests, assume a value of ε_{50} equal to 0.020, 0.010, or 0.005 for soft, medium, and stiff clay, respectively.

(7) Establish the first parabolic portion of the p-y curve using the following equation and obtaining p_u from the equations in (3) above.

$$p = 0.5\, p_u \sqrt{\frac{y}{y_{50}}}$$

This equation defines the portion of the p-y curve from the point of the intersection with the equation in (5) above to a point where y is equal to $A_s\, y_{50}$ (see note in step (10) below).

(8) Establish the second portion of the parabolic p-y curve using the following relationship:

$$p = 0.5\, p_u \left[\frac{y}{y_{50}}\right]^{0.5} - 0.55\, p_u \left[\frac{y - A_s\, y_{50}}{A_s\, y_{50}}\right]^{1.25}$$

This equation defines the portion of the p-y curve from the point where y is equal to $A_s\, y_{50}$ to a point where y is equal to $6\, A_s\, y_{50}$ (see note in step (10) below).

(9) Establish the next straight-line portion of the p-y curve.

$$p = 0.5\, p_u \sqrt{6\, A_s} - 0.411\, p_u - \left(\frac{0.0625}{y_{50}}\right) p_u \left(y - 6\, A_s\, y_{50}\right)$$

This equation defines the portion of the p-y curve from the point where y is equal to $6\, A_s\, y_{50}$ to a point where y is equal to $18\, A_s\, y_{50}$ (see note in step (10) below).

(10) Establish the final straight-line portion of the p-y curve.

$$p = 0.5\, p_u \sqrt{6\, A_s} - 0.411\, p_u - 0.75\, p_u\, A_s$$

or

$$p = p_u(1.225 \sqrt{A_s} - 0.75\, A_s - 0.411)$$

The above equation defines the portion of the p-y curve from the point where y is equal to 18 $A_s\, y_{50}$ and for all larger values of y (see following note).

Note: The step-by-step procedure is outlined and Figure 2.3.1.1.2-1 is drawn as if there is an intersection between the equations in (5) and (7) above. However, there may be no intersection of the former with any of the other equations defining the p-y curve. If there is no intersection, the equation shall be employed that gives the smallest value of p for any value of y.

(b) The following procedure is for cyclic loading and is illustrated in Figure 2.3.1.1.1.2-3. Steps (1), (2), (3), (5), and (6) are the same as for the static case.

(4) Choose the appropriate value of A_c from Figure 2.3.1.1.1.2-2 for the particular nondimensional depth.

Compute the following:

$$y_p = 4.1\, A_s\, y_{50}$$

(7) Establish the parabolic portion of the p-y curve.

$$p = A_c\, P_u \left(1 - \left(\frac{y - 0.45\, y_p}{0.45\, y_p}\right)^{2.5}\right)$$

This equation defines the portion of the p-y curve from the point of the intersection with the equation in step (5) to the point where y is equal to 0.6 y_p (see note in step (9) below).

(8) Establish the next straight-line portion of the p-y curve.

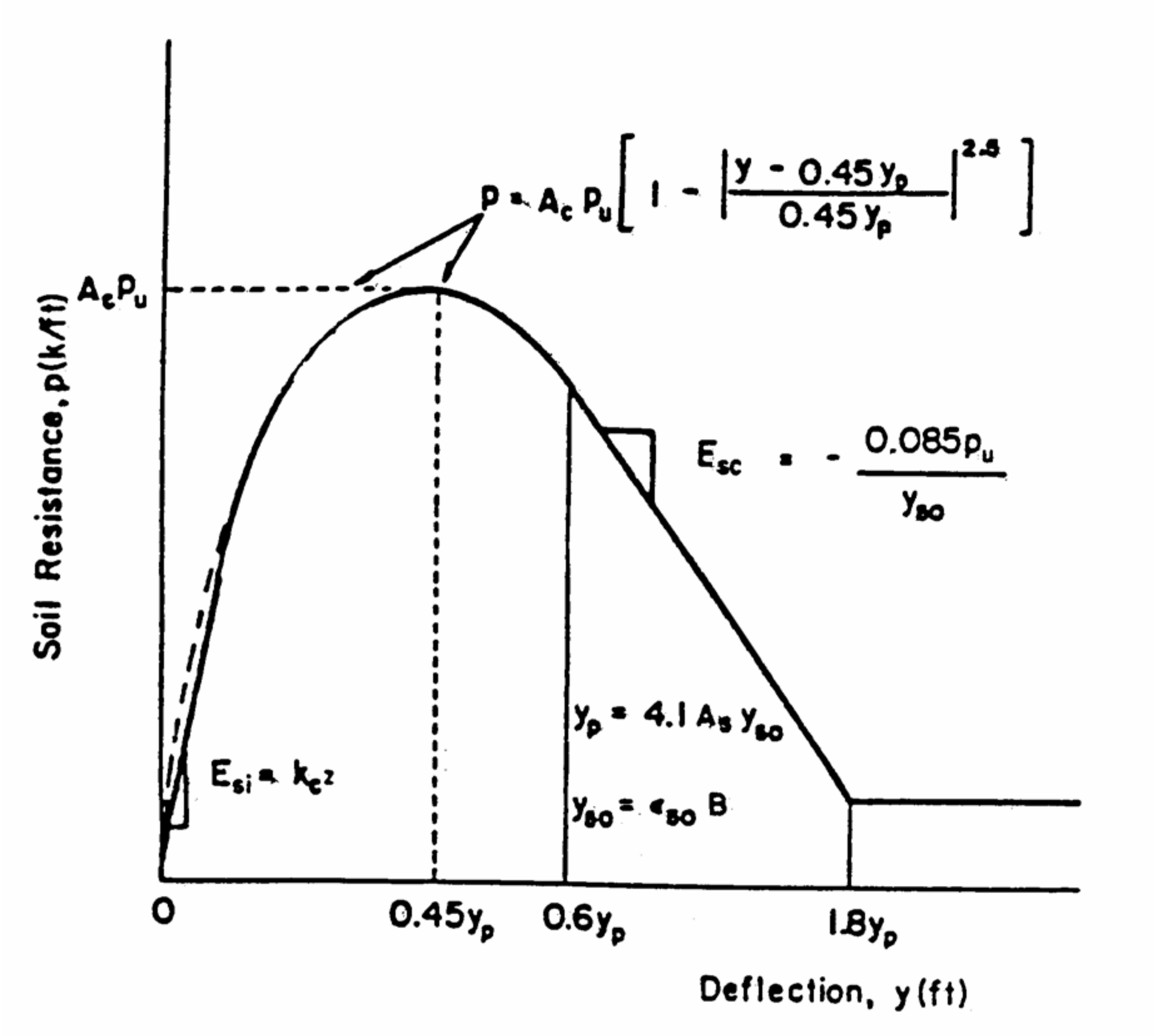
$$p = 0.936\, A_c\, p_u - \frac{0.085}{y_{50}}\, p_u\, (y - 0.6\, y_p)$$

This equation defines the portion of the p-y curve from the point where y is equal to 0.6 y_p to the point where y is equal to 1.8 y_p (see note in step (9) below).

(9) Establish the final straight-line portion of the p-y curve.

$$p = 0.936\, A_c\, p_u - \frac{0.102}{y_{50}}\, p_u\, y_p$$

This equation shall define the portion of the p-y curve from the point where y is equal to 1.8 y_p and for all larger values of y (see following note).



Source: Adapted from Reference 18

Figure 2.3.1.1.1.2-3 - Characteristic Shape of p-y Curves for Cyclic Loading in Stiff Clay below Water Table

Note: The step-by-step procedure is outlined and Figure 2.3.1.1.1.2-3 is drawn as if there is an intersection between the equations in steps (5) and (7) above. However, there may be no intersection of these two equations, and there may be no intersection of the former with any of the other equations defining the p-y curve. If there is no intersection, the equation shall be employed that gives the smallest value of p for any value of y.

(c) Recommended Soil Tests

Triaxial compression tests of the unconsolidated-undrained type with confining pressures conforming to the in situ total overburden pressures are recommended for determining the shear strength of the soil. The value of ε_{50} shall be taken as the strain during the test corresponding to the stress equal to half the maximum total-principal-stress difference. The shear strength, c, shall be interpreted as one-half of the maximum total-stress difference. Values obtained from the triaxial tests might be somewhat conservative but would represent more realistic strength values than other tests. The unit weight of the soil shall also be determined.

2.3.1.1.1.3 p-y Curves for Stiff Clay Above the Water Table

(a) The following procedure can be used for developing p-y curves for short-term static loading as illustrated in Figure 2.3.1.1.1.3-1.

(1) Obtain the values of the undrained shear strength, c, soil unit weight, γ, and shaft diameter, B. Also obtain the values of ε_{50} from stress-strain curves. If stress-strain curves are not available, use a value from ε_{50} of 0.010 or 0.005 for medium or stiff clay with the larger value being more conservative.

(2) Compute the ultimate soil resistance per unit length of shaft, p_u, using the smaller of the values given by the equations below (see 2.3.1.1.1).

$$p_u = [3 + \frac{\gamma'}{c} z + \frac{J}{B} z]\, cB$$

$$p_u = 9c\,B$$

In the use of the first equation above, the shear strength shall be taken as the average from the ground surface to the depth being considered, and J is taken as 0.5. The unit weight of the soil shall reflect the position of the water table.

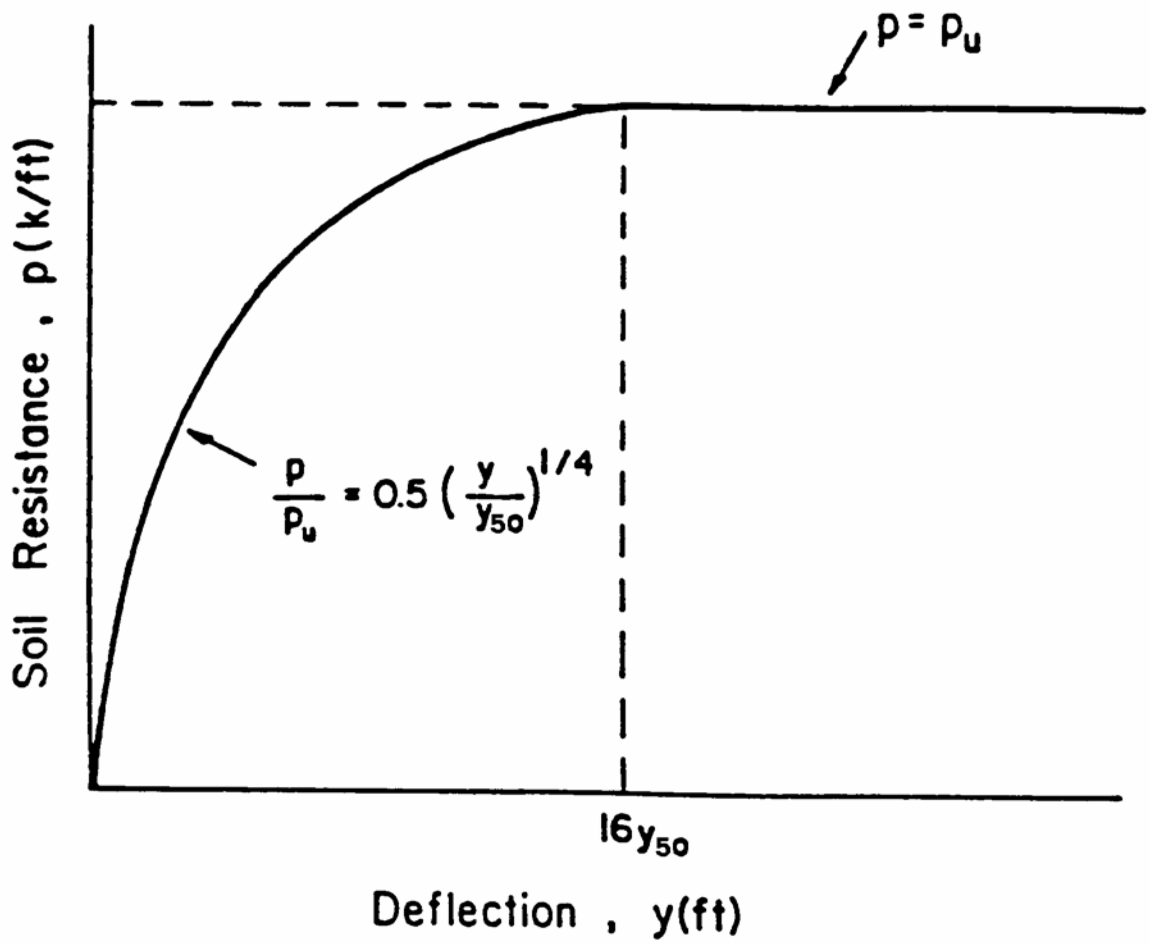
(3) Compute the deflection, y_{50}, at one-half the ultimate soil resistance from the following equation:

$$y_{50} = 2.5\,\varepsilon_{50}\,B$$

(4) Points describing the p-y curve may be computed from the relationship below.

$$\frac{p}{p_u} = 0.5\left(\frac{y}{y_{50}}\right)^{1/4}$$

(5) Beyond y = 16 y_{50}, p is equal to p_u for all values of y.



Source: Adapted from Reference 5

Figure 2.3.1.1.1.3-1 Characteristic Shape of p-y Curves for Static Loading in Stiff Clay above Water Table

(b) The following procedure is for cyclic loading and is illustrated in Figure 2.3.1.1.1.3-2.

(1) Determine the p-y curve for short-term static loading by the procedure previously given.

(2) Determine the number of times the design lateral load will be applied to the shaft.

(3) For several values of p / p_u, obtain the value of the parameter describing the effect of repeated loading on deformation, C, from a relationship developed by laboratory tests, or, in the absence of tests, from the following equation:

$$C = 9.6\left(\frac{P}{p_u}\right)^4$$

(4) At the value of p corresponding to the values of p/ p_u selected in step (3), compute new values of y for cyclic loading from the following equation:

$$y_c = y_s + y_{50} C \log N$$

where y_s is the deflection under the short-term static load, p.

(5) The p-y curve defines the soil response after N cycles of load.

(c) Recommended Soil Tests

Triaxial compression tests of the unconsolidated-undrained type with confining stresses equal to the overburden pressures at the elevations from which the samples were taken are recommended to determine the shear strength. The value of ε_{50} shall be taken as the strain during the test corresponding to the stress equal to one-half the maximum total principal stress difference. The undrained shear strength, c, shall be defined as one-half the maximum total principal stress difference. The unit weight of the soil shall also be determined.

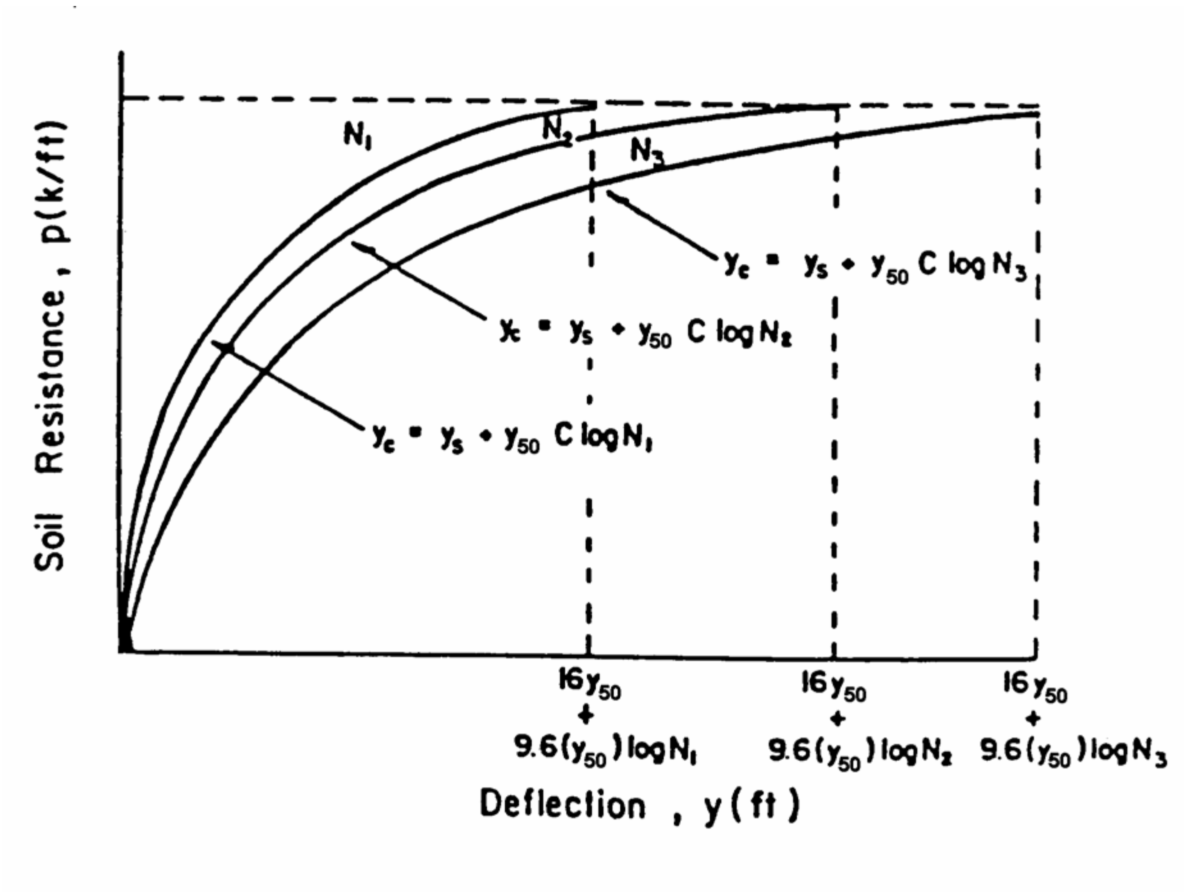


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Figure 2.3.1.1.1.3-2 Characteristic Shape of p-y Curves for Cyclic Loading in Stiff Clay above Water Table

2.3.1.1.1.4 p-y Curves for Clay Below Water Table by Unified Method

(a) The following procedure can be used for developing p-y curves for short-term static loading as illustrated in Figure 2.3.1.1.1.4-1.

(1) Obtain the values of the undrained shear strength, c, the submerged unit weight, γ′, and the shaft diameter, B. Also obtain values of ε_{50} from stress-strain curves. If stress-strain curves are not available, assume a value of ε_{50} equal to 0.020 for very soft clay, 0.010 for soft clay, 0.007 for medium clay, 0.005 for stiff clay, and 0.004 for very stiff clay.

(2) Compute c_a and σ'_v, for $z < 12$ B, where c_a equals the average undrained shear strength, σ'_v equals average effective stress, and z is the depth.

(3) Compute the variation of p_u with depth using the equations below.

For $z < 12$ B, p_u is the smaller of the values computed from the two equations below.

$$p_u = (2 + \frac{\sigma'_v}{c_a} + 0.833\frac{z}{B})\, c_a\, B$$

$$p_u = (3 + 0.5\frac{z}{B})cB$$

For $z > 12$ B

$p_u = 9$ c B

The steps below are for a particular depth, z.

(4) Select the coefficients A and F as indicated below. The coefficients A and F are determined empirically based on the results of load tests. For normally consolidated inorganic clay, values of A and F are of the order of 2.5 and 1.0, respectively. For heavily overconsolidated inorganic clays, values of A and F are of the order of 0.35 and 0.5, respectively. The recommended procedure for estimating A and F for other clays is given below.

- Determine as many of the following properties of the clay as possible, including c, ε_{50}, overconsolidation ratio, sensitivity, degree of fissuring, and Atterberg limits.
- Compare the properties of the soil in question to the properties of typical normally and heavily overconsolidated, inorganic clays. If the properties are similar to either clay type, use A and F for the similar clay.
- If the properties are not similar to either, estimate A and F using engineering judgment.

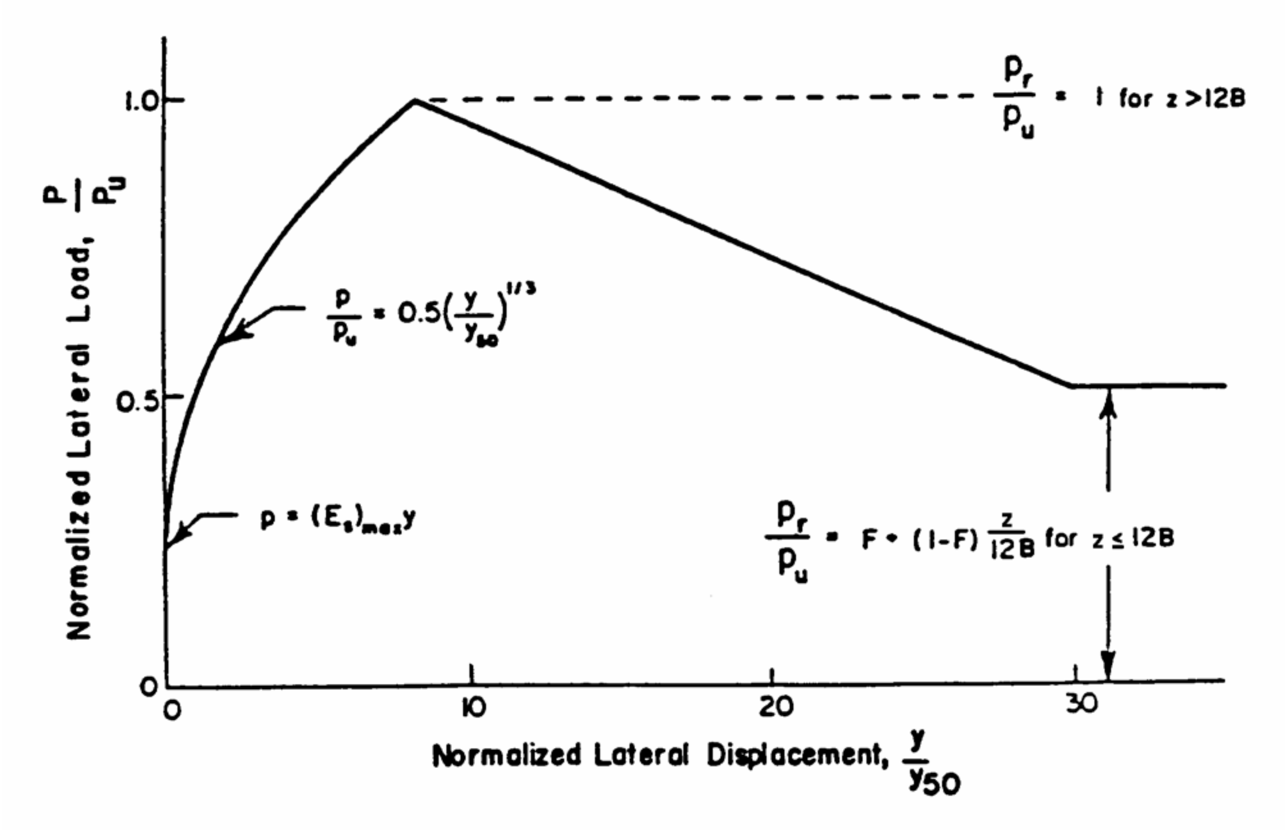

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Figure 2.3.1.1.1.4-1 Characteristic Shape of p-y Curves for Unified Clay Criteria, Static Loading

(5) Compute: $y_{50} = A\ \varepsilon_{50}\ B$

(6) Determine $E_{s\ max}$. When no other method is available, use the equation below.

$E_{s\ max} = n_h\ z$

Assume values of n_h equal to 3930 kN/m^3 {25 kips per cubic foot (kcf)} for very soft clay, 13 350 kN/m^3 {85 kcf} for soft clay, 39 270 kN/m^3 {250 kcf} for medium clay, 133 500 kN/m^3 {850 kcf} for stiff clay, and 392 700 kN/m^3 {2,500 kcf} for very stiff clay.

(7) Use the following equation to compute the deflection at the intersection between the initial linear portion and curved portion.

$$y_g = \left[\frac{0.5\, p_u}{(E_s)_{max}}\right]^{1.5} \left[y_{50}\right]^{-0.5}$$

y_g can be no larger than 8 y_{50}.

(8) For $0 > y < y_g$,

$p = E_{s\ max}\ y$

For $y_g < y < 8\,y_{50}$

$$p = 0.5\,p_u \left(\frac{y}{y_{50}} \right)^{1/3}$$

For $8y_{50} < y < 30y_{50}$

$$p = p_u + \frac{p_r - p_u}{22\,y_{50}}(y - 8y_{50})$$

where:

$$p_r = p_u \left[F + (1 - F)\frac{z}{12B} \right]$$

p_r will be equal to or less than p_u.

For $y > 30\,y_{50}$

$p = p_r$

(b) The following procedure is for cyclic loading and is illustrated in Figure 2.3.1.1.1.4-2.

(1) Repeat steps (1) through (8) for static loading.

(2) Compute:

$$p_{cr} = 0.5\,p_u \left[\frac{z}{12B} \right] \leq 0.5\,p_u$$

For $y_g < y < y_{50}$

$$p = 0.5\,p_u \left[\frac{y}{y_{50}} \right]^{1/3}$$

For $y_{50} < y < 20y_{50}$

$$p = 0.5\,p_u + \left[\frac{p_{cr} - 0.5\,p_u}{19\,y_{50}} \right](y - y_{50})$$

For $y > 20\,y_{50}$

$p = p_{cr}$

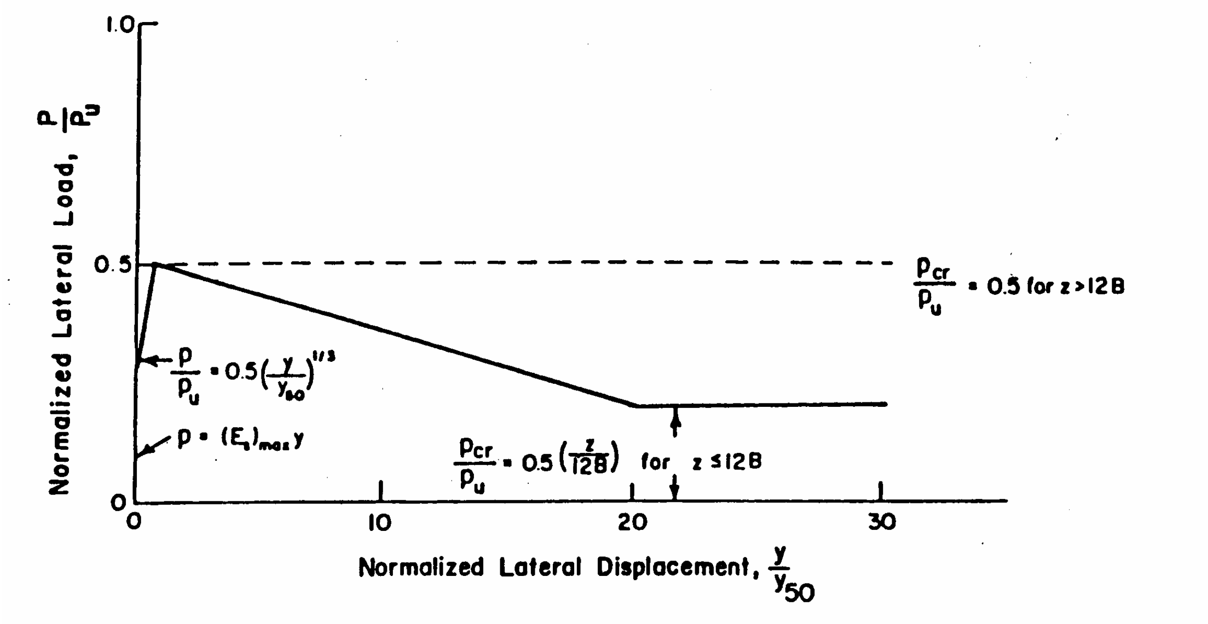

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Figure 2.3.1.1.1.4-2 - Characteristic Shape of p-y Curves for Unified Clay Criteria, Cyclic Loading

2.3.1.1.1.5 p-y Curves for Sand

(a) The following procedure can be used for developing p-y curves for short-term static loading and for cyclic loading of sand as illustrated in Figure 2.3.1.1.1.5-1.

(1) Obtain values for the angle of internal friction, φ', the effective soil unit weight, γ', and shaft diameter, B.

(2) Make the following preliminary computations:

$\alpha = \varphi/2$

$\beta = 45 + \varphi/2$

$K_o = 0.4$

$K_a = \tan^2 (45 - \varphi/2)$

$K_p = \tan^2(45 + \varphi/2)$

(3) Compute the ultimate soil resistance per unit length of shaft using the smaller of the values given by the equations below, where z is equal to the depth below the ground surface.

$p_{st} = \gamma B^2[S_1 (z/B) + S_2 (z/B)^2]$

$p_{sd} = \gamma B^2[S_3 (z/B)]$

where:

$S_1 = (K_p - K_a)$

$S_2 = \tan \beta(K_p \tan \alpha + K_o[\tan \varphi \sin \beta (\sec \alpha + 1) - \tan \alpha])$

$S_3 = K_p^2 (K_p + K_p \tan \varphi) - K_a$

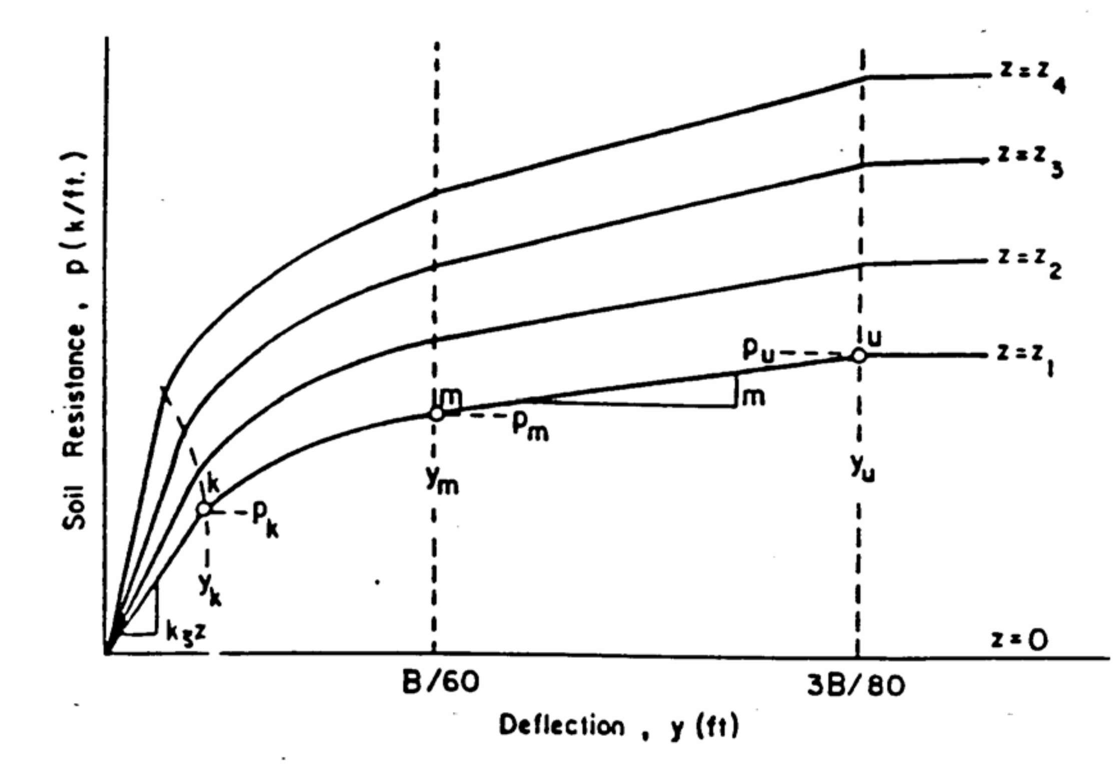

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Figure 2.3.1.1.1.5-1 - Characteristic Shape of a Family of p-y Curves for Static and Cyclic Loading in Sand

(4) The depth of transition, z_t, can be found by equating the expressions in the first two equations in (3) above as follows:

$z_t / B = (S_3 - S_1) / S_2$

The appropriate γ for the position of the water table shall be employed. Above z_t, use the first equation in (3). Below z_t, use the second equation in (3). S_1, S_2, S_3, and z_t/B are functions only of φ, therefore, the values shown in Table 2.3.1.1.1.5-1 can be computed.

(5) Select a depth at which a p-y curve is desired.

(6) Establish y_u as 3 B/80. Compute p_u using one of the following equations:

$p_u = \overline{A}_s \, p_s$

$p_u = \overline{A}_c \, p_s$

Use the appropriate value of As or A_c from Figure 2.3.1.1.1.5-2 for the particular nondimensional depth, and for either the static or cyclic case. Refer to the computation in Step (4) for the appropriate equation to use for p_s.

(7) Establish y_m as B/60. Compute p_m using one of the following equations:

$p_m = B_s \, p_s$

$p_m = B_c p_s$

Use the appropriate value of B_s or B_c from Figure 2.3.1.1.1.5-3 for the particular nondimensional depth, and for either the static or cyclic case. Use the appropriate equation for p_s. The two straight-line portions of the p-y curve beyond the point where y is equal to B/60 can now be established.

(8) Establish the initial straight-line portion of the p-y curve.

$p = n_h\, z\, y$

Use Figure 2.3.1.1.1.5-4 and Table 2.3.1.1.1.5-2 to select an appropriate value of n_h.

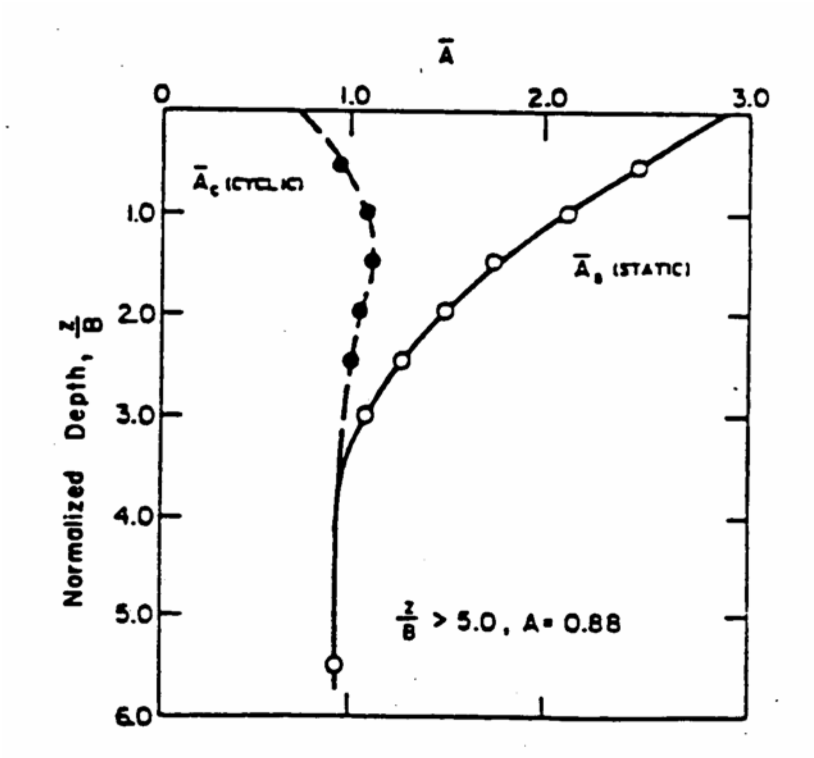

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Figure 2.3.1.1.1.5-2 - Nondimensional Coefficients A_c and A_s

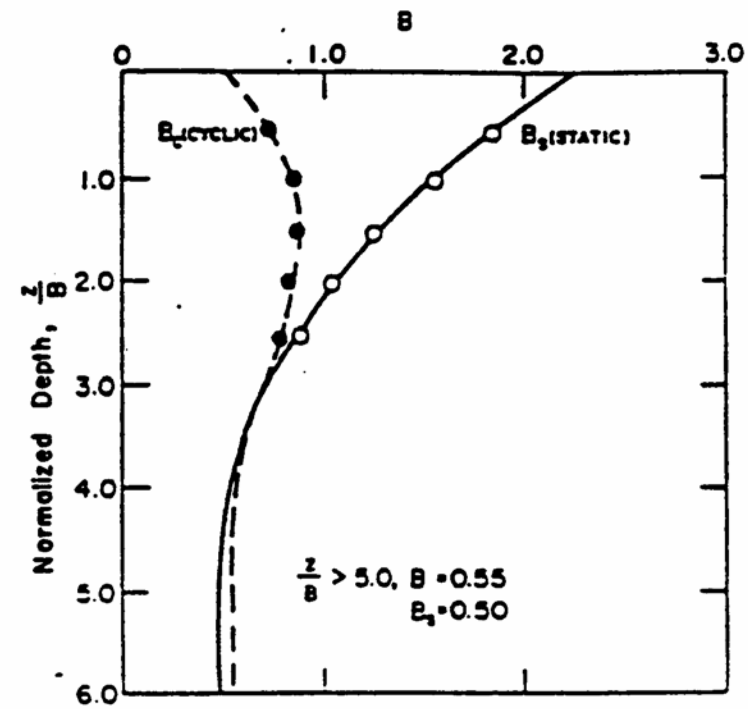

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Figure 2.3.1.1.1.5-3 - Nondimensional Coefficient B

Table C2.3.1.1.1.5-1 - Nondimensional Coefficients for p-y Curves for Sand

Angle of Friction φ	S_1[1]	S_2[2]	S_3[3]	z_t/B
(deg)	(dim)	(dim)	(dim)	(dim)
25	2.05805	1.21808	15.68459	11.18690
26	2.17061	1.33495	17.68745	11.62351
27	2.28742	1.46177	19.95332	12.08526
28	2.40879	1.59947	22.52060	12.57407
29	2.53509	1.74906	25.43390	13.09204
30	2.66667	1.91170	28.74513	13.64147
31	2.80394	2.08866	32.51489	14.22489
32	2.94733	2.28134	36.81400	14.84507
33	3.09732	2.49133	41.72552	15.50508
34	3.25442	2.72037	47.34702	16.20830
35	3.41918	2.97045	53.79347	16.95848
36	3.59222	3.24376	61.20067	17.75976
37	3.77421	3.54280	69.72952	18.61673
38	3.96586	3.87034	79.57113	19.53452
39	4.16799	4.22954	90.95327	20.51883
40	4.38147	4.62396	104.14818	21.56704

Notes:

(1) $S_1 = (K_p - K_a)$

(2) $S_2 = \tan\beta(K_p \tan\alpha \; + \; K_o[\tan\varphi \sin\beta \,(\sec[\alpha + 1]) \; - \; \tan\alpha])$

(3) $S_3 = K_p^{\,2}\,(K_p + K_p \tan\varphi) \; - \; K_a$

Source: Adapted from Reference 5

Table 2.3.1.1.1.5-2 - Representative Values of n_h for Sand

Relative Density D_r (%)	n_h (kcf)	
	Sand Below Water Table	Sand Above Water Table
< 35	35 (5500 kN/m^3)	45 (7070 kN/m^3)
35 - 65	100 (15 700 kN/m^3)	155 (24 300 kN/m^3)
> 65	215 (33 800 kN/m^3)	390 (61 300 kN/m^3)

Source: Adapted from Reference 5.

(9) Establish the parabolic section of the p-y curve.

$$p = \overline{C} y^{1/n}$$

Fit the parabola between points k and m as follows:

- Determine the slope of line between points u and m as follows:

$$\mathrm{m} = \frac{\mathrm{p_u - p_m}}{\mathrm{y_u - y_m}}$$

- Obtain the power of the parabolic section by

$$\mathrm{n} = \frac{\mathrm{p_m}}{\mathrm{m\ y_m}}$$

- Obtain the coefficient $\overline{\mathrm{C}}$ as follows:

$$\overline{\mathrm{C}} = \frac{\mathrm{p_m}}{\mathrm{y_m^{1/n}}}$$

- Determine point k as

$$y_k = \left(\frac{\overline{C}}{N_h z} \right)^{n/(n-1)}$$

- Compute appropriate number of points on the parabola by using the first equation in (9) above.

Note: The step-by-step procedure is outlined and Figure 2.3.1.1.1.5-1 is drawn as if there is an intersection between the initial straight-line portion of the p-y curve and the parabolic portion of the curve at point k. However, in some instances there may be no intersection with the parabola. The equation in (8) above defines the p-y curve until there is an intersection with another branch of the p-y curve; or, if no intersection occurs, this equation defines the complete p-y curve. The soil-response curves for other depths can be found by repeating the above steps for each desired depth.

(b) Recommended Soil Tests

Triaxial compression tests are recommended for obtaining the angle of internal friction of the sand. Confining pressures shall be used which are close or equal to those at the depths being considered in the analysis. Tests shall be performed to determine the unit weight of the sand. In many instances, however, undisturbed samples of sand cannot be obtained, and the value of φ must be obtained from correlations with static cone penetration tests or from dynamic penetration tests.

Theoretical soil mechanics and engineering experience provide the basis for developing p-y curves. Actual methods of constructing such curves arise from duplication of load-deformation behavior observed during lateral load tests. The methods are typically generalized through correlations width soil properties, shaft diameter, and depth for broader application.

Because it is difficult to determine an exact percent of fixity, and because increased shaft head fixity results in decreased ground deflections, percent fixities somewhat less than those expected to be achieved are applied in deflection analysis of laterally-loaded drilled shafts so that the results obtained are somewhat conservative. It should be noted that the moment conditions at the two extremes of head fixity present the worst cases for consideration of the structural requirements of the shaft.

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PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 2

APPENDIX G - INTEGRAL ABUTMENTS

TABLE OF CONTENTS

SPECIFICATIONS

1.1 PURPOSE

These guidelines establish design criteria for Integral Abutments. Construction details are presented in the BD-667M Standard Drawing.

COMMENTARY

C1.1

Integral abutment bridges (Jointless Bridges) serve to accomplish the following desirable objectives:

- Long-term serviceability of the structure
- Minimal maintenance requirements
- Economical construction
- Improved aesthetics and safety considerations

A jointless bridge concept is defined as any design procedure that attempts to achieve the goals listed above by eliminating as many expansion joints as possible from the structure. The ideal jointless bridge, for example, contains no expansion joints in the superstructure, substructure or deck.

Integral abutment bridges are generally founded on one row of flexible piles made of steel or concrete. This permits the elimination of expansion joints, bearings, piles for horizontal earth loads and other uneconomical details.

When expansion joints are completely eliminated from a bridge, thermal stresses must be relieved or accounted for in some manner. The integral abutment bridge concept is based on the assumption that due to the flexibility of the piles, thermal stresses are transferred to the substructure by way of a rigid connection, i.e. the uniform temperature change causes the abutment to translate without rotation. The concrete abutment contains sufficient bulk to be considered a rigid mass. A positive connection to the girder ends is generally provided by encasement in the reinforced concrete backwall. This provides for full transfer of forces due to thermal movements and live load rotational displacement to the abutment piles.

1.2 DESIGN CRITERIA

1.2.1 Bridge Length

Maximum allowed bridge total length, for 90° skews, measured between the centerlines of end bearings shall be taken as:

Concrete Structures = 180 m {590 ft.}

Steel Structures = 120 m {390 ft.}

Use of integral abutments on structures with lengths over the above limits shall be considered on case-by-case basis and shall require the written approval of the Chief Bridge Engineer at the Type, Size and Location stage.

Expansion bearings shall be eliminated wherever possible.

C1.2.1

Length limits for integral abutment bridges are a function of local soil and weather conditions. At the present, comprehensive design guidelines that take all design parameters into account are nonexistent in most states. Such guidelines would include design criteria concerning piles, approach slabs, wingwalls, backfill, drainage provisions, and the safe length limits of integral abutment bridges based on soil and weather profiles.

Based on past experience with jointless bridges in Pennsylvania, bridges within the specified range did not have any serious problems. Some other states have successfully constructed longer jointless bridges. When economically feasible, the use of jointless bridges with lengths exceeding the limits specified herein should be investigated.

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Span arrangement and interior bearing selection shall be such that approximately equal movements take place at both abutments.

The ratio between the span lengths in the bridge shall be chosen such that no net negative reaction is produced at the abutment at any limit state.

COMMENTARY

The live load negative reaction at the abutments of bridges with short end spans may be relatively high. Transverse and uplift wind forces also produce negative forces acting on some of the piles. Due to the relatively light weight of integral abutments, it is possible to have a negative factored net force on some piles of bridges with short end span. To avoid any problems with the pile-abutment connection, a negative net force is not allowed on any pile.

1.2.2 Skew

Minimum allowed skew angle is:

- 70º for single spans in excess of 40 000mm {130 ft.} and for multiple spans.
- 60º for single spans in excess of 27 000 mm {90 ft.} but not longer than 40 000 mm {130 ft.}.
- 45º for single spans 27 000 mm {90 ft.} or less.

C1.2.2

Earth pressure acts in a direction perpendicular to the abutments. For skewed bridges, the earth pressure forces on the two abutments produce a torque that causes the bridge to twist in plan. Limiting the skew angle reduces this effect. For continuous bridges, the mild skew angles reduce the forces acting on intermediate bents. For simple spans, limiting the length reduces the torque and the resulting twist of the bridge.

1.2.3 Horizontal Alignment and Bridge Plan Geometry

Only straight beams may be used. Curved superstructures utilizing straight beams may be used, if approved by the Chief Bridge Engineer. All beams in each span of a curved bridge shall be parallel to each other. Integral abutments shall be allowed for straight bridges with splayed girders when the difference between the length of the two abutments does not exceed 10%.

C1.2. 3

Curved beams are not allowed to guard against the possibility of flange buckling caused by the beams trying to expand between the restraining abutments.

1.2.4 Grade

The maximum vertical grade between abutments shall be 5%.

1.2. 5 Scour

For structures over stream crossings, conduct an investigation for scour in accordance with guidelines given in PP7. Utilize rip rap, geotextile and casing details per BD-667.

C1.2. 5

Even when the piles are properly designed to withstand the loss of lateral support caused by scour, there is some concern for the integrity of bridge approaches. In light of this concern, the effect of scour on the stability of bridge approach embankment should be investigated.

1.2. 6 Drainage

The area behind the abutments shall be backfilled in accordance to Ap.G.1.2.9 and a proper drainage system shall be provided (see BC-751M for drainage details).

C1.2. 6

Providing a proper granular backfill and drainage system behind the abutment beneath the approach slab eliminates the build up of hydrostatic pressure and controls erosion of the abutment embankment.

1.2.7 Loads and Load Combinations

1.2.7.1 PERMANENT LOADS

Permanent loads on the abutments include the weight of

C1.2.7.1

The dead loads on the abutments are distributed fairly

SPECIFICATIONS

the girders, deck and approach slab, attached wingwalls, intermediate diaphragms and the abutment backwall. Other permanent loads on the first span and the approach slab such as wearing surface, barriers, utilities, sign structures, lighting systems and sound barriers shall also be considered. The dead loads on the first span of the bridge shall be determined using the dead load reaction of the girders calculated during the design of the superstructure. The dead loads on the abutments shall be distributed equally to all piles.

1.2.7.2 LIVE LOADS

For the design of the abutment and the piles, live loads will be assumed equally distributed to all girders in the cross section. No multiple presence factors will be applied.

The total live load on the abutment shall be determined assuming the largest number of traffic lanes that may be allowed by the total bridge width as specified in A3.6.1.1.1. For bridges with side walks, the following two load cases shall be investigated:

1. The sidewalk is assumed to be eliminated. The number of traffic lanes is calculated based on the total width of the bridge including the width of the shoulders and sidewalks.

2. The number of traffic lanes is calculated based on the

COMMENTARY

uniformly across the width of the bridge. It is sufficiently accurate to assume equal distribution of dead loads to all piles.

C1.2.7.2

Due to the high rigidity of the abutment beam, the loads are expected to be distributed to more piles than those directly under the load. The loads are likely to be distributed equally to all piles, particularly in the case of narrow bridges. For wide bridges, where the total length-to-depth ratio of the abutment beam is relatively high, loads applied near the edge of the bridge will not be distributed equally to all piles. However, the critical load case for the piles is expected to be the case of maximum load on the bridge, i.e. all traffic lanes and sidewalks are loaded. In this case, the loads are distributed across the width of the bridge and is expected to produce approximately equal loads on all piles. The elimination of the multiple presence factor will eliminate the possibility of underestimating the maximum pile load for wide bridges.

For the design of integral abutments, the live load reaction of any girder may be calculated as follows:

$$LLR_{IA} = \frac{LLR_{DP} * NL}{DF_{DP} * NG}$$

Where:

- LLR_{IA}: LL girder reaction for integral abutment design (N) {kips}
- LLR_{DP}: LL reaction for interior girders from the girder design program (N) {kips}
- DF_{DP}: LL shear distribution factor from the girder design program
- NG: Number of girders in the cross sections
- NL: Maximum number of traffic lanes allowed by bridge clear width

Equation 1 assumes that both the end span and approach

SPECIFICATIONS

total width of the bridge excluding the sidewalks (i.e. the number of traffic lanes is calculated based on the clear width between curbs). For this load case, pedestrian loads acting on the abutment shall be calculated using Equation 1:

$$SWL = \frac{PL * W_{SK} * (L_{ES} + L_{AS})}{2} \qquad (1.2.7.2\text{-}1)$$

SWL: Sidewalk reaction at the abutment (N) {kips}
PL: Pedestrian load per unit area as specified in A3.6.1.6 (MPa) {ksf}
W_{SK}: Width of sidewalk (mm) {ft.}
L_{ES}: Length of end span of the bridge (mm) {ft.}
L_{AS}: Length of the approach slab (mm) {ft.}

Centrifugal force shall be applied to the abutments of curved bridge. The magnitude and point of application of the centrifugal force shall conform to the requirements of A3.6.3.

Braking forces shall not be considered in the design of integral abutments.

Dynamic load allowance shall be considered in the design of both the abutment wall and the supporting piles.

In determining vehicular loads on the approach slabs, all possible traffic lanes shall be assumed to have the uniform load part of the PHL-93 loading. The approach slab shall be assumed to act as a simple beam supported at one end of the abutment and supported at the other end on the sleeper slabs.

1.2.7.3 WIND LOADS

Wind uplift on the superstructure and transverse wind loads on the superstructure and on live load shall be considered in the design. The magnitude and point of application of these forces shall be calculated according to A3.8 and D3.8.

The direction of transverse wind forces shall be taken perpendicular to the longitudinal axis of the bridge.

1.2.7.4 THERMAL MOVEMENTS

The change in the bridge length due to uniform temperature change, Δt, shall be calculated as:

$$\Delta L = \alpha \, \Delta t \, L \qquad (1.2.7.4\text{-}1)$$

Where:

L: Total bridge length between the centerlines of the abutments (mm) {in.}
α: Coefficient of thermal expansion of the bridge material as specified in D5.4.2.2 and A6.4.1 for concrete and steel bridges, respectively (mm/mm/º C) {in/in/º F}

COMMENTARY

slab are simply supported at both ends. The sidewalk load is generally small compared to traffic loads. Ignoring the continuity of the end span of continuous bridges is an acceptable approximation for the design of integral abutment

Braking forces are not considered in determining the lateral earth pressure on the abutment. Braking forces will conceivably push the abutment against the backfill causing the earth pressure to increase. However, the maximum earth pressure will never exceed the full passive pressure. The design of abutment components is based on the full passive pressure and, therefore, braking forces are not included.

Since the piles are attached directly to the structure, the dynamic load allowance is required to be considered in pile design.

The truck load in each lane is assumed to be on the bridge and is considered in the girder reaction.

C1.2.7.3

The magnitude of wind forces due to wind pressure on the substructure is often small, relative to wind on the superstructure and on live loads, and may be neglected in the analysis.

In case of curved bridges, subject to the limitations of Ap.G.1.2.3, the direction of longitudinal and transverse wind forces on the end span of the bridge shall be taken parallel and perpendicular to the girders of the end spans, respectively.

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Δt: Uniform temperature change (º C) {º F}
ΔL: Change in bridge length due to uniform temperature change Δt (mm) {in.}

For the purpose of determining the maximum thermal movement at the abutments in one direction, Δ_{max} , after the time the connection between the abutment and the superstructure is made, the following uniform temperature changes, Δt_{max}, shall substitute Δt in Equation 1.

- For steel bridges:
  Δt_{max} = 56º C {100º F} temperature rise or fall
- For concrete bridges:
  Δt_{max} = 45º C {80º F} temperature rise or fall

Where:

Δt_{max}: The design uniform temperature rise or fall, equal to the maximum possible difference between the construction temperature and the extreme temperature of the bridge during its useful life (ºC) {ºF}

For the purpose of determining the total change in the bridge length between the extreme high and low temperatures during the life span of the bridge, the uniform temperature change, Δt_{total}, shall substitute Δt in Equation 1.

Where:

Δt_{total}: the sum of the temperature rise and temperature fall specified in D3.12.2.1for the bridge material.

For simple spans with constant width, and for continuous spans having constant width and with both superstructure and substructure symmetric in the bridge elevation, the thermal movements at each abutment of bridges with integral abutments at both ends shall be taken as half the change in bridge length due to uniform temperature change.

For other bridges, the thermal movements of the abutments due to uniform temperature change shall be determined taking into account the type of bearings at intermediate supports and the stiffness of the substructure

COMMENTARY

The design uniform temperature rise or fall, Δt_{max}, used in determining the maximum movement in one direction (after making the connection between the abutment and the superstructure) is meant to be an upper-bound for the following two cases:

- The difference between the extreme high temperature during the life span of the bridge and the lowest allowable construction temperature
- The difference between the extreme low temperature during the life span of the bridge and the highest allowable construction temperature

Using the specified design uniform temperature rise or fall, Δt_{max}, will make the actual thermal movement always less than or equal to the design thermal movement regardless of the construction temperature.

The total range of thermal movement is used in the pile ductility check. This total range of thermal movement is not a function of the construction temperature. It is a function of the difference between the extreme low and high temperatures of the structure during its useful life.

In some cases, the bed rock may be too close to the surface at one end of the bridge that the use of piles may not be feasible. If approved by the Chief Bridge Engineer, a concrete footing on rock with a semi-integral abutment may be used at one end of the bridge and an integral abutment at the other end. The smaller depth of the semi-integral abutment relative to the integral abutment will produce smaller earth pressure on the semi-integral end of the bridge. This will cause most of the thermal movements to take place at the semi-integral end. For design purposes, the full change in bridge length will be assumed to take place at the semi-integral end of bridges with one integral abutment and one semi-integral abutment. Bridges with one integral abutment and one semi-integral abutment will usually be short bridges due to the limitations of maximum movements of semi-integral abutments.

Similarly, if the bridge has one or more expansion joints, the entire thermal change in length of the bridge unit starting at an integral abutment and ending at an expansion joint will be assumed to take place at the expansion joint.

The stiffness of the abutment is a function of the depth of the abutment, the thermal movement and the backfill type and compaction. Due to the nonlinear change of the coefficient of earth pressure, the stiffness of the abutment varies nonlinearly

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including the piers, intermediate bents and the abutments.

The following approximate procedure should be used to determine the stiffness and estimated maximum thermal movement of the abutments:

1. Estimate the thermal movement of one abutment, Δ_{max} (mm) {in.}, based on the total length of the bridge and the design uniform temperature rise or fall, Δt_{max}, for the bridge material. As a first step, this value may be assumed half the total change in the bridge length due to uniform temperature rise or fall, Δt_{max}.

2. Determine the relative displacement ratio Δ_{max}/H, where H is the total height of the abutment, i.e. the sum of the heights of the pile cap, bearing pad, beams, haunch and deck slab (mm) {in.}.

3. Determine the coefficient of passive earth pressure, k_p, corresponding to the calculated Δ_{max}/H as:

   Metric units: $k_p = 0.5 + 125\dfrac{\Delta_{max}}{H} \le 3.0$ (1.2.7.4-2)

   U.S. Customary Units: $k_p = 0.5 + 10.4\dfrac{\Delta_{max}}{H} \le 3.0$

4. Determine the component of the total earth pressure force on the abutment parallel to the longitudinal axis of the bridge, F, using Equation 1.

   Metric units: $F = \frac{1}{2}\gamma g k_p H^2 L \sin\theta$ (1.2.7.4-3)

   U.S. Customary units: $F = 1/2\ \gamma k_p H^2 L sin\theta$

   Where:

   γ: the soil density (kg/mm^3) {kcf}
   θ: Skew angle (DEG)
   g: the acceleration of gravity = 9.81 m/sec^2
   H: Abutment height (mm) {ft.}
   L: Length of the abutment along the skew (mm) {ft.}

5. Determine the stiffness of the abutment k = $F/\Delta max$ (N/mm) {k/in}

6. Similarly, calculate the stiffness of the second abutment. The sum of the movements of the two abutments should equal the change in bridge length calculated for a uniform temperature change, Δt_{max}, using Equation 3

7. Use a structural analysis computer program acceptable to the Department to determine a more accurate value of the abutment displacement, Δ_{max}, for the specified design uniform temperature rise or

COMMENTARY

depending on the magnitude of the displacement.

Equation 2 is based on a two-line approximation of the passive pressure coefficient curve for loose sand in Figure C1. It assumes that the coefficient of earth pressure changes linearly from the at rest coefficient of 0.5 for no displacement to 3.0 when $\Delta t/H$ is equal to or greater than 0.02

Figure C1 is from Barker (1997).

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fall, Δt_{max}. The abutment stiffness shall be modeled as a horizontal spring with a stiffness equal to that calculated for the abutment. The end of the bridge at the abutment shall be modeled as being supported on a vertical movable support. Alternatively, the abutment may be modeled as a single vertical column fixed at the bottom and pinned to the superstructure. The column shall have a stiffness equal to the stiffness of the abutment calculated in step 5 above. The length, L_c, the modulus of elasticity, E_c, and the moment of inertia, I_c, of the column shall be chosen to satisfy:

$$k = \frac{3E_c I_c}{L_c^3} \qquad (1.2.7.4\text{-}4)$$

8. Compare the estimated displacement used in Step 1 to the displacement calculated in Step 7. If the difference is within 10%, there is no need for further iterations. If larger difference is obtained, go back to Step 2 and use the calculated displacement to determine a new value of Δ_{max}/H.

The total range of movement of an integral abutment during the bridge life span, Δ_{total}, (mm) {in}shall be determined as:

$$\Delta_{total} = \Delta_{max}\,(\Delta t_{total}/\Delta t_{max}) \qquad (1.2.7.4\text{-}5)$$

Where:

Δ_{max}: The maximum design thermal movement in one direction as calculated in Step 8 above (mm) {in.}

COMMENTARY

Due to the approximate nature of the process, a 10% difference between the estimated displacement (Step 1) and the calculated displacement (Step 8) is considered to be an acceptable difference.

The total range of thermal movement, Δ_{total}, is used in the pile ductility check. As shown in Steps 1 through 8, the movement of each abutment as a percentage of the total thermal change of length is a function of the stiffness of the abutment. Equation 1 assumes that the stiffness of the abutment used to calculate the maximum thermal movement in one direction will remain constant as the bridge moves from the extreme high temperature to the extreme low temperature. Ignoring the change in the abutment stiffness is an approximation used to simplify the design. The effect of this approximation is expected to be minor due to the small difference between Δt_{total} and Δt_{max}.

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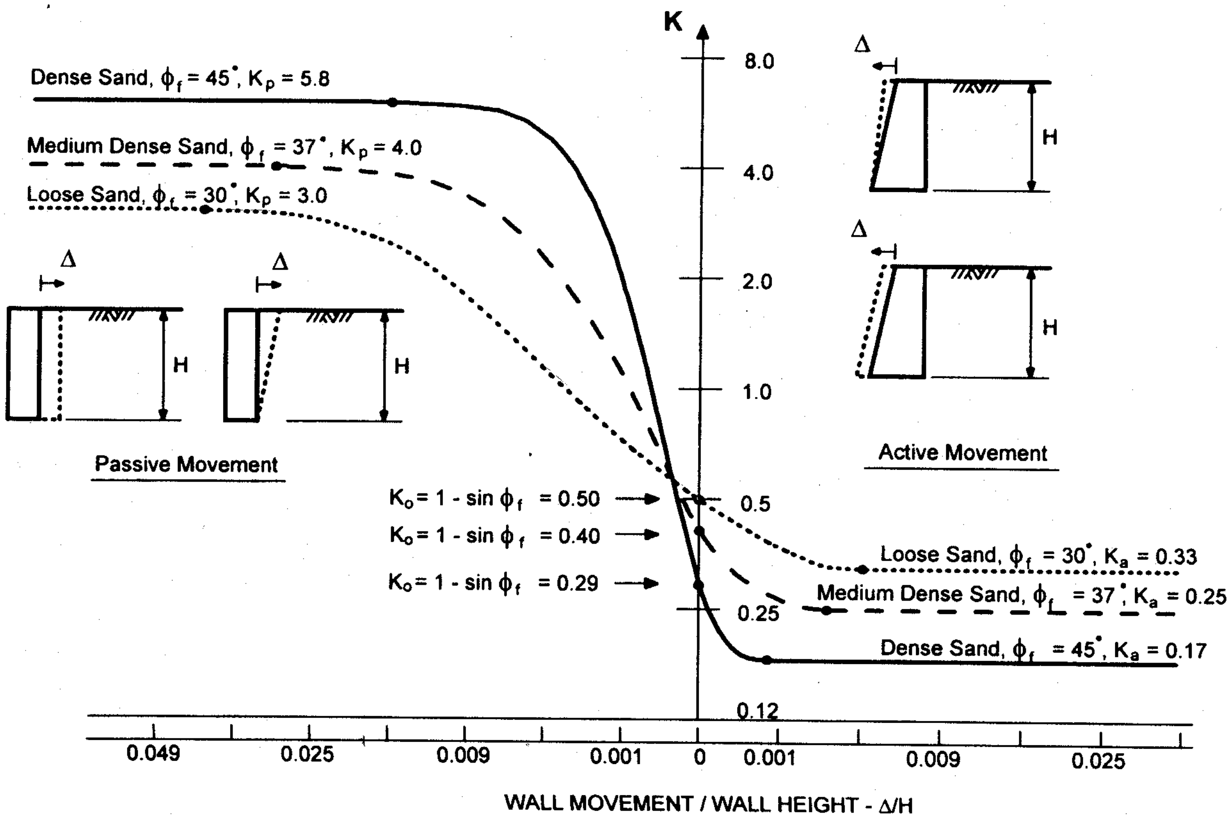

Figure C1.2.7.4-1 Passive Pressure Coefficient Curves

1.2.7.5 SECONDARY LOADS

Except for the effect of creep and shrinkage on the vertical reactions of simple prestressed spans made continuous for live loads, abutment loads caused by creep, shrinkage, thermal gradient and differential settlements need only be considered for bridges longer than those specified in Ap.G.1.2.1, which are approved by the Chief Bridge Engineer. The method of applying secondary loads requires the approval of the Chief Bridge Engineer.

1.2.7.6 LOAD COMBINATIONS

Load combinations for integral abutments shall be taken as shown in Table 1.

C1.2.7.5

The states contacted during the development of these specifications, which utilize integral abutment bridges, do not consider secondary loads in the design of integral abutment bridges with total length within the limits stated in Ap.G.1.2.1. For longer bridges, the effect of secondary loads may be significant and is required to be considered in the design.

Table 1.2.7.6-1 – Load Combinations for Integral Abutments

Load Factors for Integral Abutments														
Load Case	SERV I		STR I		STR IP		STR II		STR III		STR V		Extreme II	
	max	min	max	min	max	min	max	min	max	min	max	min	max	min
γ_{DC}	1.00	1.00	1.25	0.90	1.25	0.90	1.25	0.90	1.25	0.90	1.25	0.90	1.25	0.90
γ_{DW}	1.00	1.00	1.50	0*	1.50	0*	1.50	0*	1.50	0*	1.50	0*	1.50	0*
γ_{EV}	1.00	1.00	1.35	1.00	1.35	1.00	1.35	1.00	1.35	1.00	1.35	1.00	1.35	1.00
γ_{EH}^{**}	1.00	1.00	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50
γ_{ES}	1.00	1.00	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50
γ_{LS}	1.00	1.00	1.75	1.75	1.35	1.35	1.35	1.35	0.00	0.00	1.35	1.35	0.50	0.50
γ_{LLIM}	1.00	1.00	1.75	1.75	1.35	1.35	1.35	1.35	0.00	0.00	1.35	1.35	0.50^{+}	0.50^{+}
γ_{PL}	0.00	0.00	0.00	0.00	1.75	1.75	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
γ_{WS}	0.30	0.30	0.00	0.00	0.00	0.00	0.00	0.00	1.40	1.40	0.40	0.40	0.00	0.00
γ_{WL}	0.30	0.30	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	1.00	0.00	0.00
γ_{CE}	1.00	1.00	1.75	1.75	1.35	1.35	1.35	1.35	0.00	0.00	1.35	1.35	0.50	0.50
γ_{TU}^{++}	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Design Vehicle	PHL – 93		PHL-93		PHL-93		P-82		---		PHL-93		PHL-93	

* For existing bridges where the future wearing surface has been already applied, use 0.65 for the minimum γ_{DW}

** Use the specified load factors for active pressure. Use a load factor of 1.25 for passive pressure

\+ Extreme Event II Limit State is used to check the piles after scour takes place. A reduced live load is assumed on the bridge during the time between scour taking place and the time the bridge is closed to traffic.

++ Thermal movements is a major source of loads on the abutment and abutment piles. Both the passive earth pressure on the abutment and the stresses in steel piles due to thermal movements are not reduced by the plastic flow of the concrete expected due to the seasonal nature of the thermal movements. Therefore, no reduction in the load factor for uniform temperature is allowed and a load factor of 1.0 is used all the time. For concrete-filled pipe piles, the same load factor is applied. The effect of the plastic flow of the concrete filling is considered in the analysis by applying a reduction factor of 0.4 to the transformed area and moment of inertia of the concrete filling.

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1.2.8 Girder Types, Maximum Depth and Placement

Only steel I-beams, Concrete I-beams and concrete spread box beams shall be allowed for use with integral abutments.

Maximum girder depth for use with integral abutments shall be 1825 mm {72 in.}. Deeper girders may be used in integral abutment bridges if approved by the Chief Bridge Engineer.

All girders in integral abutment bridges, including box beams, shall be placed with their webs vertical. The roadway cross slopes and superelevation shall be accommodated by changing the depth of the concrete haunch across the width of the girders.

1.2.9 Backfill Material and Sequence

The area behind the abutments shall be backfilled with granular material such as O.G.S. (See RC-12M for backfill limits).

Flowable fill and large stone fill shall not be allowed in conjunction with integral abutments

A 25 mm {1 in.} thick sheet of Styrofoam shall be placed against the entire area of the back face of the abutment below the bottom of the approach slab.

The fill within a distance 600 mm {2 ft.} wide directly behind both the abutment and the attached wingwalls shall be nominally compacted using two passes of a walk-behind vibratory-plate soil compactor. The fill in this area shall be compacted in 100 mm {4 in.} thick lifts. The fill behind both abutments shall be compacted simultaneously. The difference in the fill depth between the two ends shall not exceed 300 mm {1 ft.} at any point of time.

1.3 SUPERSTRUCTURE DESIGN

The superstructure shall be designed similar to conventional superstructures with expansion joints. The fixity developed as a result of rigidly connecting the superstructure to the abutments and the piers shall not be considered in the design of the superstructure.

Compressive axial load equal to the passive earth pressure on the abutment shall be considered in the design of the superstructure.

When flexibility for locating substructures is available, it is desirable to provide span ratios (interior/exterior) that produce near equal total design moments for all spans.

COMMENTARY

Deeper abutments are subjected to larger earth pressure force and, therefore, less flexible. Girder depth limit is based on past successful practices by other states and is meant to ensure a reasonable level of flexibility of the abutment. Deeper girders may be allowed when the soil conditions are favorable and the total length of the bridge is shorter than the length limits in Ap.G1.2.1.

Placing the girders with their webs vertical will allow the beam seat area to be horizontal in all cases and will facilitate using thin neoprene bearing pads.

C1.2.9

Flowable fill and large stone fill are self compacting. They will reduce the desired flexibility of the abutment.

The specified compaction is meant to reduce the passive earth pressure acting on the abutment and the wingwalls when moving toward the soil.

Filling behind both abutments simultaneously will keep the earth pressure on both abutments approximately equal at all times during construction. This will minimize unanticipated movements of the bridge due to imbalanced earth pressure forces.

C1.3

This provision will only be used to check the compressive stress at the ends of deep, widely spaced prestressed beams (beam depth greater than 1825 mm {6 ft.} and spacing greater than 3600 mm {12 ft.}. The estimated additional compressive stress is in the order of 1.5 MPa {30 ksf}. For sections not close to the end of the span, the design under all loads is usually controlled by tension in the concrete and the additional compressive stress will not be a critical factor.

This preferred arrangement of span lengths results in more economical structures. The resulting length of end spans is approximately 80% of the adjacent interior spans. This length ratio produces relatively small negative live load reaction, thus, reduces the possibility of negative net reaction at the abutments.

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1.4 INTEGRAL ABUTMENT DESIGN

1.4.1 General

The portion of the deck slab within 1200 mm {4 ft.} from the front face of the abutment shall be poured after the remaining of the deck is poured. The end portion of the deck and the backwall of the abutment will be poured at the same time.

Integral abutments shall be constructed in two stages:

Stage 1:

A pile cap supported on one row of vertical piles shall be constructed. The top of the pile cap shall reach the bottom of the bearing pads under the girders. The top of the pile cap shall be smooth in the area directly under the girders and a strip 50 mm {2 in.} wide around this area. Other areas shall be intentionally roughened (rake finished).

Stage 2:

After pouring the entire deck slab, except for the portions of the deck within 1200 mm {4 ft.} from the front face of the abutments, an end diaphragm (backwall) encasing the ends of the bridge girders shall be poured. The end 1200 mm {4 ft.} of the deck shall be poured simultaneously with the end diaphragm for beams ≤ 900 mm {36 in.}. For beams deeper than 900 mm {36 in.}, wait a minimum of 2 hours after casting the end diaphragm before casting the deck slab portion over the diaphragm. This ensures that settlement of the poured concrete has occurred in the end diaphragm placement before beginning placement of the deck slab portion over the end diaphragm. The concrete diaphragm and the end portion of the deck shall be poured when the surface temperature is between 2E C {35º F} and 32E C {89º F}. The expected air temperature during the six hours following pouring this concrete should be within the same range. The end diaphragm shall have the same width as the pile cap constructed in Stage 1. The end diaphragm shall extend from the top of the pile cap to the top surface of the deck slab.

The width of the pile cap shall be 1200 mm {4 ft.}.

The height of the pile cap shall be 1000 mm {3'-3"} at its shallowest point.

The maximum difference between the minimum and maximum depth of the pile cap shall not exceed 300 mm{1 ft.} for skews less than 80E and 450 mm {1'-6"} for skews 80Eor larger.

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C1.4.1

Pouring most of the deck slab before pouring the connection between the integral abutment and the superstructure minimizes the dead load rotations imposed on the piles. The dead load rotations at the ends of the girders will not be transferred to the piles. The end portions of the deck and the abutment wall (end diaphragm) may be poured immediately after the deck.

When a median barrier exists on the bridge, split the abutment at the center of the barrier and leave a 25 mm {1 in.} gap between the two halves. Place a closed cell neoprene sponge in the gap. Glue the neoprene sponge to both sides. Place an approved membrane waterproofing along the rear face of the abutment along the vertical joint.

The 1200 mm {4 ft.} width can accommodate four bars of longitudinal reinforcement of the pile cap, concrete cover, stirrups, HP360 {HP14} piles at 45E skew and allows for 75 mm {3 in.} of pile placement tolerance in any direction.

The height of the pile cap will vary along the length of the abutment due to differences in the beam seat elevations. These differences are due to the cross slopes of the roadway on the bridge and due to superelevation.

The bottom of the pile cap will generally be horizontal. If the limit on the maximum difference in pile cap depth could not be satisfied, the bottom of the abutment may be made parallel to the roadway surface resulting in a constant total height of the abutment.

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1.4.2 Piles

1.4.2.1 GENERAL

Integral abutments may be supported on end bearing piles or friction piles. Steel-encased concrete piles or steel H-piles may be used.

Monotube piles shall not be allowed in integral abutments.

The minimum distance between the piles and the end of the abutment, measured along the skew, shall be 450 mm {1'-6"} and the maximum distance shall be 750 mm {2'-6"}. The piles shall be embedded 450 mm {1'-6"} into the abutment. Maximum pile spacing for integral abutment piles shall be 3000 mm {10 ft.}. The minimum pile spacing requirements of A10.7.1.5P shall apply.

Steel H-piles shall be driven with their weak axis perpendicular to the centerline of the beams of the end span regardless of the skew.

For structures less than or equal to 30 000 mm {100 ft.} in length, pre-augering is not required except for situations where shallow rock, rock embankments, or obstructions are encountered within 3000 mm {10 ft.} of the bottom of pile cap, or unless otherwise required due to the geology of the location. For structures over 30 000 mm {100 ft.} in length, oversize pre-augered holes shall be drilled at pile locations prior to driving the piles. The minimum depth of the pre-augered holes shall be 3000 mm {10 ft.}. The piles shall then be placed in the pre-augered holes. The holes shall be filled with dry sand or pea gravel after the piles are placed in the holes but before driving the piles. Following, the piles shall be driven to the required tip elevation, bearing stratum and driving resistance.

The minimum diameter of the pre-augered holes shall be the larger of:

- 600 mm {2 ft.}
- The largest dimension of the pile, i.e., pipe pile diameter or H-pile diagonal, plus 250 mm {10 in.}.

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C1.4.2.1

Steel-encased concrete piles (pipe piles) are less ductile than H-piles and may be damaged if subjected to relatively large lateral displacements and rotations. Thus, the analysis may prove they are inadequate for use in longer bridges, in particular when the girders in the end span are flexible. The ends of such girders will be subjected to a relatively high rotation due to live loads and composite dead loads. .

There is no available data on the behavior of monotube piles under plastic deformations. The thin walls, the taper and the wall corrugations of the monotube piles are expected to make their inelastic behavior different from that of smooth pipes. These factors make it likely that these piles will not have the ductility required to resist the movements of integral abutments without local buckling of the pile sections.

Thermal movements of integral abutments are parallel to the longitudinal axis of the girders of square bridges. Orienting the H-piles with their weaker axis perpendicular to the longitudinal axis of the bridge reduces the restraint forces developed at the pile-to-abutment connection due to thermal movements. In case of skewed and curved bridges, thermal movements are affected by the geometry of the bridge. However, especially for mild skews, the thermal movements are still close to be parallel to the girders of the end span. Therefore, H-piles are still required to be driven with the weaker axis perpendicular to the longitudinal axis of the girders of the end span.

Installing the piles in pre-augered oversize holes is a common practice used in the design of integral abutments. Horizontal soil forces act on the piles of integral abutments when subjected to thermal movements. The magnitude of the horizontal soil forces is a function of the type of soil and the magnitude of thermal movements. The use of pre-augered holes reduces the magnitude of the horizontal soil forces and moves the point of fixity lower in the soil. The deeper the point of fixity, the larger the radius of curvature of the piles when deformed by the effect of thermal movement. The larger radius of curvature reduces the ductility demand on the piles. Increasing the depth of the pre-augered holes should be considered when higher ductility of the piles is required; particularly when the top layers of the soil are stiff.

Pre-augered oversize holes may also be used to minimize downdrag forces when piles are driven in compressive soils. Excessive downdrag forces, if not prevented, may lead to a premature failure of the piles. The use of oversized pre-augered holes also tends to minimize the effects of concrete

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shortening due to creep effects when used with prestressed concrete beam superstructures.

Two conflicting concerns are to be considered when refilling pre-augered holes. First, pile buckling is to be prevented by providing lateral support along the full length of the pile and, second, the fill should be flexible enough to allow the pile to move laterally without being subjected to high lateral loads and moments. Sand or pea gravel fills provide the required characteristics.

Integral abutment piles shall be anchored to the abutment as follows:

- For pipe piles: A reinforcement cage shall be placed inside the top portion of the pile. The longitudinal bars shall be 4-#25 {4-#8} for 305 mm {1'-0"} diameter piles and 5-#25 {5-#8} for 356 mm {1'-2"} and 406 mm {1'-4"} diameter piles.
- For H-piles: One #16 {#5} reinforcement bar, 900 mm {3 ft.} long shall be placed through a hole in the web of the pile. The hole shall be 25 mm {1 in.} in diameter and shall be located 150 mm {6 in.} from the top of the pile.

The details of the pile anchorage are shown on the BD-667M Standard Drawings.

Integral abutment piles are anchored in the abutment to guard against any unanticipated uplift on the piles. In addition, for the stiffer pipe piles, they ensure adequate moment transfer between the pile and the abutment.

All piles shall have a penetration length into the original soil at least equal to the larger of the following two values:

(1) For piles through embankment fills, the minimum penetration into original ground specified in A10.7.1.2.1P, and,

(2) The depth to point of fixity specified in D6.15.3.3P plus 1500 mm {5 ft.}.

This minimum length is required to provide fixity and adequate capacity to the piles. Piles satisfying the required minimum length are expected to adequately resist vertical and lateral loads and avoid a stilt-type effect.

Article D6.15.3.3P requires COM624P or LPILE 5.0 analysis for the determination of the point to fixity.

The minimum acceptable pile length, measured from the bottom of the abutment to the pile tip, shall be 4500 mm {15 ft.}.

When scour is anticipated, the minimum pile length shall be provided beyond the depth of computed maximum scour.

In case the bedrock is close to the surface, the pre-augered holes shall be continued into the bedrock to provide the minimum required pile length.

The minimum pile length is provided below the design scour depth to ensure the safety of the structure.

1.4.2.2 PILE DESIGN

Piles shall be designed for normal vertical loads and lateral loads. Piles shall also be analyzed for both the abutment thermal movements and the superstructure live load and composite dead load rotations. The thermal movements shall be calculated in accordance with Ap.G.1.2.7.4.

C1.4.2.2

Larger pile reactions may be obtained by maximizing the transverse moments on the abutments. These transverse moments are produced by (1) transverse wind load on the structure and on live load, and (2) centrifugal force on curved bridges.

Overturning moments due to wind load on the superstructure and on live load, wind uplift and centrifugal force transmitted to the abutment are assumed to be resisted by axial forces on the piles. The pile loads are assumed to increase linearly from zero at the center of the abutment to maximum for the piles near the end of the abutment as shown in Figure C1. For clarity, only centrifugal force and wind on the superstructure are shown in this figure. Wind uplift and wind on live load are applied similarly.

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The design of the pile is controlled by the minimum

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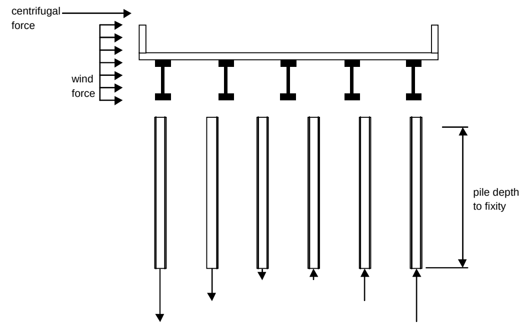

Figure C1.4.2.2-1. Pile Vertical Loads Due to Overturning Moments

Earth pressure on the abutments of continuous spans will produce additional vertical loads on the piles. The value of the additional vertical load per pile may be approximately calculated using Figure C2 and Equation C1. This approximate calculation assumes that the end span is separated from the remaining of the bridge and that the end of the span at the first pier is simply supported.

$$P_T = \frac{\left(\frac{2FH}{3N}\right) + H_T H + M_T}{L} \quad \text{(C1.4.2.2-1)}$$

Where:

- P_T: The additional vertical load per pile due to earth pressure on the abutment (N) {kips}
- N: Number of piles in the abutment
- F : Passive pressure force on the abutment (N) {kips}
- H: The height of the abutment (mm) {ft.}
- H_T: The horizontal force on the pile, may be calculated as twice the plastic moment of the pile divided by the distance from the bottom of the abutment to the point of fixity of the piles (N) {kips}
- M_T: The moment at the top of the pile, may be conservatively taken as the plastic moment of the pile (N mm) {k ft}

- L: Length of end span (mm) {ft.}

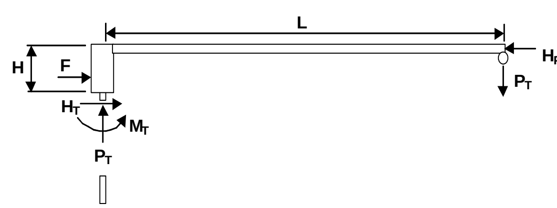

Figure C1.4.2.2-2 Pile Vertical Load Due to Earth Pressure on the Abutment

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capacity as determined for the following cases:

Case A: Capacity of the pile as a structural member according to the procedures outlined in D10.7.4.1 and in Ap.G.1.4.2.4. The design for combined moment and axial force shall be based on an analysis that takes the effect of the soil into account.
Case B: Capacity of the pile to transfer load to the ground
Case C: Capacity of the ground to support the load as specified in A10.7.2.5 and D10.7.3.

The top portion of friction piles where the lateral deflection exceeds 0.02 of the pile width perpendicular to the longitudinal axis of the bridge shall be ignored in determining the friction capacity of the piles.

1.4.2.3 PILE ANALYSIS

When the piles in an abutment vary in length such that some or all of the piles have one point of inflection (zero moment), the analysis shall be conducted for both of the end piles in the abutment. For other cases, the analysis shall be conducted on the longer of the two end piles.

Moments, shears and deflections along the length of the piles shall be determined using the computer program COM624P or LPILE 5.0. The following procedure shall be followed in analyzing the piles:

Apply the maximum pile vertical load to the top of the pile simultaneously with the abutment maximum thermal movement, Δ_{max}, calculated using Ap.G.1.2.7.4. Apply a moment at the head of the pile and increase this moment incrementally. As the moment increases, the rotation at the head of the pile changes. The final positions of the pile will be obtained when one of the following two situations, whichever occurs first, is reached:

1. The applied moment is less than the plastic moment of the pile and the pile head rotation reaches the superstructure end rotation due to live loads and composite dead loads.

2. The moment at the head of the pile reaches the plastic moment of the pile section.

The soil properties within the depth of the pre-augered holes shall be assumed to be the weaker of loose sand and the surrounding soil.

In determining the plastic moment of concrete-filled pipe piles, the concrete inside the pipe shall be ignored.

For concrete-filled pipe piles, M_y used in the analysis

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Maximum load capacity of the pile based on the soil conditions will be determined using the soil properties provided by the geotechnical engineers.

The movement of the pile reduces the effectiveness of its contact with the soil at its top portions. This effect diminishes when the pile movement is less than 0.02 of the pile width (Fleming et. al. 1985). The pile width should be taken as the H-pile depth or the diameter of round piles.

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Extending the length of the piles below the second point of zero moment has a very little effect on the results in the upper, more critical, portions of the pile. In some cases, the bedrock elevation, and the length of piles, vary significantly and some or all of the piles will have only one inflection point. The behavior in theses cases may vary and it is required to perform the analysis for both of the end piles. The choice of the end piles is based on them having the highest load and that they are likely to be the longest and shortest piles in the abutment (assuming bedrock slope is approximately linear across the width of the bridge.)

The computer program COM624P or LPILE 5.0 is based on assuming elastic behavior. For small movements, the pile will remain elastic. Step 1 assumes elastic behavior as long as the maximum moment is below the plastic moment. Once the pile reaches its plastic moment, the moment cannot be increased further.

Once the moment at the head of the pile reaches the plastic moment and a plastic hinge forms, the rotations of the pile head become independent of the rotations of the abutment. The difference between the rotation of the pile head obtained from COM624P or LPILE 5.0 analysis and the rotation of the superstructure.

There is no simple way to use COM624P or LPILE 5.0 to analyze piles with maximum moment between the yield moment and the plastic moment. As a simplification, piles with maximum moments exceeding the yield moment but below the plastic moment are assumed to remain elastic. This will be a conservative solution since that COM624P or LPILE 5.0 results overestimate the moment in the piles when the moment exceeds the yield moment. This is because the program does not consider the redistribution of forces associated with inelastic behavior.

The plastic rotations required at plastic hinge locations will cause the concrete filling to crack and become ineffective in resisting the applied plastic moments. At lower moments, the concrete will still be effective and should be considered in determining the area and moment of inertia of the section.

When the applied moment is less than the yield moment,

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shall not exceed M_p.

The moment of inertia of the concrete-filled pipes shall be taken as that of the transformed section (transformed to steel). The modulus of elasticity shall be taken as that of steel. A reduction factor of 0.4 shall be applied to the concrete area and moment of inertia when calculating the transformed section properties.

Under normal conditions, a COM624P or LPILE 5.0 analysis shall be conducted assuming that the entire pile is below ground surface. When scour is expected, a separate analysis shall be conducted assuming that the length of the pile from the bottom of the abutment to the ground surface after scour takes place is unsupported.

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the concrete filling is assumed effective. In some cases, M_y (calculated using Equation C4 below) may exceed M_p (calculated using Equation C1). This is not allowed as the plastic moment is assumed to be the upper bound of the section resistance.

Thermal movements of integral abutments are seasonal in nature. The relatively long time over which these movements take place allows for plastic flow of the concrete. The effective stiffness of the concrete in these cases is less than its elastic stiffness. The reduction factor of 0.4 applied to the concrete area and moment of inertia when calculated the transformed section properties is meant to account for this nonlinear behavior. The value of 0.4 is chosen to match that required by A6.9.5.1 when analyzing concrete-filled pipe columns.

The plastic moment, transformed area, transformed moment of inertia and yield moment of concrete-filled pipe can be calculated as follows:

$$M_P = 4 f_y R^2 t \qquad \text{(C1.4.2.3-1)}$$

$$A_t = 2\pi\, Rt + \frac{0.4\pi R_i^{\,2}}{n} \qquad \text{(C1.4.2.3-2)}$$

$$I_t = \pi R^3 t + \frac{0.1\pi R_i^{\,4}}{n} \qquad \text{(C1.4.2.3-3)}$$

$$M_y = \frac{f_y I_t}{R_o} \qquad \text{(C1.4.2.3-4)}$$

Where:

R_i: The inside radius of the steel pipe (mm) {in.}
R_o: The outside radius of the steel pipe (mm) {in.}
R : the average radius of the steel pipe (mm) {in.}
t : the thickness of the pipe (mm) {in.}
f_y: the specified minimum yield stress for the pipe material (MPa) {ksi}
n: The modular ratio between the steel and the concrete filling
M_p: The plastic moment of the steel pipe (N·mm) {k·in}
M_y: The yield moment of the composite section (N·mm) {k·in}
A_t: The transformed area of the steel and concrete section (mm^2) {in^2}
I_t: The transformed moment of inertia of the steel and concrete (mm^4) {in^4}

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1.4.2.4 PILE DESIGN AS A STRUCTURAL MEMBER

The portion of the pile between the bottom of the abutment and the closest point of zero moment shall be checked for ductility as specified in Ap.G.1.4.2.5.

The piles shall be checked as a structural member subject to axial load and flexure. The axial force shall be taken as the full pile load and the moment shall be taken as the maximum moment within the length under consideration. The moments shall be obtained from the COM624P or LPILE 5.0 computer program output. The length used in determining the axial load capacity of the pile as a compression member shall be taken as:

1. The portion of the pile between the two points of zero moment closest to the bottom of the abutment. This length shall be assumed pinned at both ends.
2. For shorter piles that have only one point of zero moment, the length between the point of zero moment and the tip of the pile. This length shall be assumed pinned at both ends.
3. When scour is expected to expose the top portion of the piles, the length of the pile from the bottom of the abutment to the pile point of fixity. This length shall be assumed fixed at both ends. This case shall only be checked if the length from the bottom of the abutment to the first point of fixity is larger than 1.54 times the length used in checking the pile before scour (case 1 or 2 above, whichever was applicable). This case is applied at the extreme event limit state.

1.4.2.5 PILE DUCTILITY REQUIREMENTS

Integral abutment piles shall satisfy the following equations:

For steel H-piles:

$$2\left[\frac{\Delta_{total}}{2L} - \frac{M_p L}{6EI}\right] + \theta_w \leq \frac{3C_i M_p L}{4EI} \qquad (1.4.2.5\text{-}1)$$

$$C_i = \frac{19}{6} - 5.68\sqrt{\frac{f_y}{E}}\,\frac{b_f}{2t_f} \qquad (1.4.2.5\text{-}2)$$

For concrete-filled pipe piles:

$$2\left[\frac{\Delta_{total}}{2L} - \frac{M_p L}{6EI}\right] + \theta_w \leq \frac{C_i M_p L}{2.08EI} \qquad (1.4.2.5\text{-}3)$$

$$C_i = 3.5 - 1.25\sqrt{\frac{f_y}{E}}\,\frac{D}{t} \qquad (1.4.2.5\text{-}4)$$

Where:

Δ_{total}: Total thermal movement of the abutments calculated

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The stresses in the pile near its connection to the abutment need not be checked. Yielding and redistribution of forces is allowed in this area.

For friction piles, the axial force decreases along the length of the pile. However, the first point of zero deflection is usually a short distance below the bottom of the abutment and the axial force at this point will essentially be equal to the pile load.

The scour case shall control when the exposed portion of the piles is long. The 1.54 factor is the ratio between the effective length factor for a member pinned at both ends, i.e. 1.0, and that of a member fixed at both ends, i.e., 0.65. Beyond the specified length ratio, the fixed-fixed compression member shall have a smaller capacity than the pinned-pinned member.

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The bases for the ductility requirements for H-piles may be found in the Transportation Research Record Report No. 1223 and Greimann, et. al (1987). The ductility requirements for pipe piles were driven using the same principals.

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by Equation 1.2.7.4-5 (mm) {in.}

L: Twice the length from the bottom of the abutment to the first point of zero moment in the pile (mm) {in.}

M_p: Plastic moment of the H-pile in weak axis bending or the plastic moment of the steel pipe without considering the concrete filling (N mm) {k in}

E: Modulus of elasticity of the steel (MPa) {ksi}

I: H-pile moment of inertia about the weak axis or moment of inertia of the filled pipe considering both the steel pipe and the concrete filling calculated using Equation C1.4.2.3-3 (mm^4) {in^4}

θ_w: Maximum range (Positive + $|$negative$|$) of factored angle of rotation of the superstructure at the abutment calculated assuming the structure is simply supported on the abutment, always taken as a positive quantity. This rotation is the sum of the rotations due to composite dead loads and live loads. Live load rotations shall be calculated assuming all traffic lanes on the bridge are loaded and the live loads are distributed equally to all girders (RAD).

C_i: A ductility reduction factor for piles, $0.0 < C_i < 1.0$

f_y: The specified minimum yield stress for pipe material (MPa) {ksi}

b_f: width of pile flange (mm) {in.}

t_f: pile flange thickness (mm) {in.}

D: Outer diameter of the concrete-filled pipe pile (mm) {in.}

t: Thickness of concrete-filled pipe pile (mm) {in.}

1.4.3 Abutment/Backwall/Pile Cap Design

The thickness of the abutment backwall shall be 1200 mm {4 ft.}.

The effect of cushion material, placed behind the backwall of the abutment, on the earth pressure shall not be considered in the design.

The longitudinal bars in the deck slab shall be extended as far as practical into the backwall. When the transverse bars in the deck are placed parallel to the abutment, these bars need not be placed past the front face of the backwall. In case of skewed bridges with deck transverse bars placed perpendicular to the girders, the transverse bars of the slab shall be extended inside the backwall and be terminated as close to the back face of the backwall as practical.

The longitudinal reinforcement of the abutment shall be designed assuming that all vertical loads on the abutment are resisted by the bottom 1000 mm {3'-4"} of the abutment; i.e. resisted by the pile cap. Abutment walls shall be designed for the following two cases:

Case 1: Vertical loads assuming the abutment wall to act as a continuous beam supported on the piles. The effect of the lateral loads transmitted to the abutment shall be considered in determining the maximum girder vertical reactions.

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The earth pressure forces acting on the abutment due to thermal expansion of the bridge will be reduced by the styrofoam sheet placed behind the abutment before backfilling, as specified in Ap.G.1.2.9. The styrofoam will yield under pressure and thus help in reducing the restraining forces.

The reinforcement details shown on the BD-667M Standard Drawings are intended to satisfy these requirements.

In case of skewed bridges with deck transverse bars perpendicular to the girders, place the top reinforcement bars of the backwall below the top reinforcement layer of the deck. This arrangement eliminates the interference between the backwall reinforcement and the transverse deck slab bars extending inside the backwall.

Loads acting on the abutments are listed in Ap.G. 1.2.7

Simplified approaches may be used to calculate shears and moments in the abutment walls. Continuous beam moments may be taken as 80% of simple span moments. Shears may be taken equal to simple span shears. Due to the relatively large depth and short spans of the abutment walls, minimum reinforcement is usually sufficient to satisfy the strength requirement.

In determining the maximum girder reaction, overturning moments due to wind load on the superstructure and on live load, wind uplift and centrifugal force are assumed to be

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transmitted to the abutment through vertical girder reactions. The girder vertical reactions due to lateral loads are assumed to increase linearly from zero at the center of the abutment to maximum for the exterior girders as shown in Figure C1.

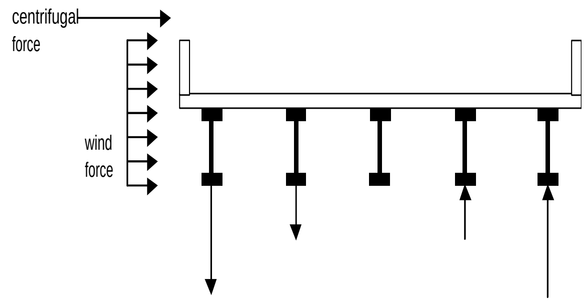

Figure C1.4.3-1 - Girder Vertical Reactions Due to Overturning Moments

Dead loads applied before the top part of the abutment hardens are resisted by the pile cap. Loads applied after that are resisted by the full abutment wall. For simplicity, all loads are assumed to be applied to the pile cap. Due to the large size of the pile cap, the increase in the reinforcement due to this conservative assumption is minor. Figure C2 gives the longitudinal reinforcement of the pile cap. The shown reinforcement represents an upper-bound for the required reinforcement assuming the girders are located at the positions that produce maximum effects on the pile cap and assuming a conservative value of other dead loads on the abutment wall.

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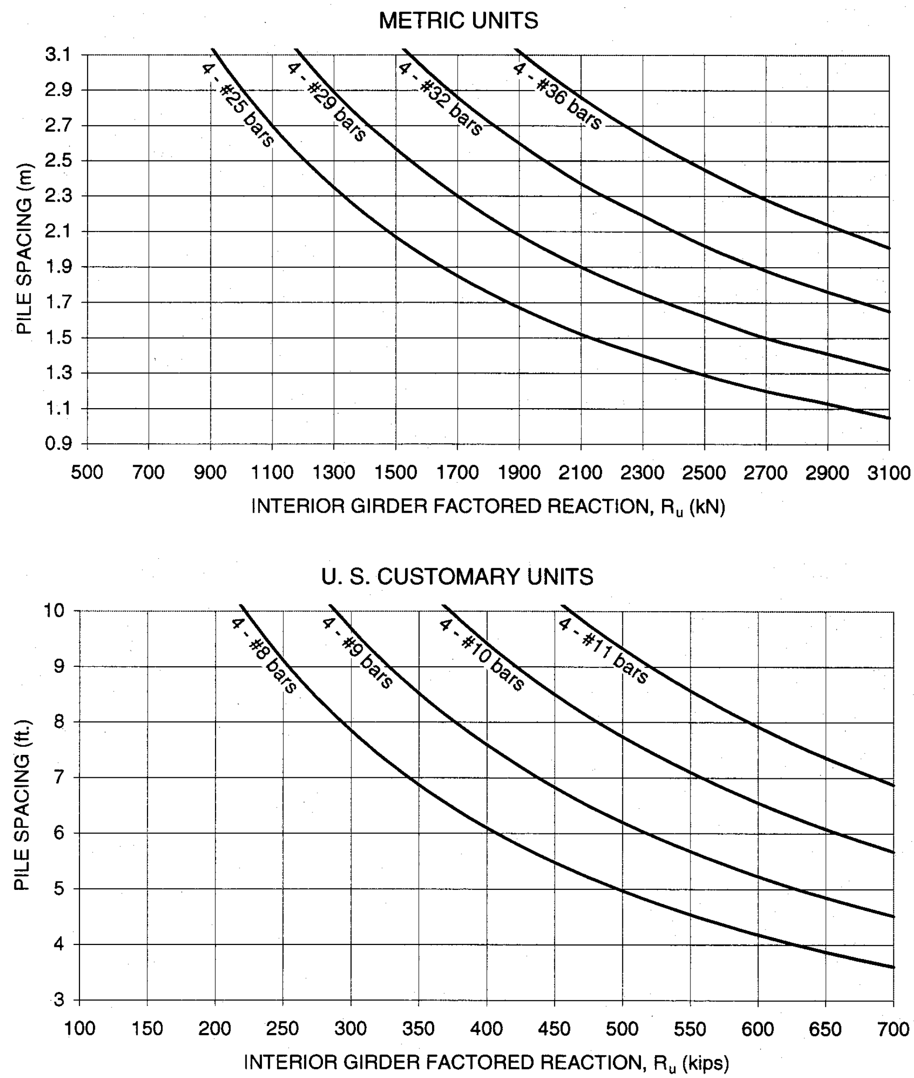

Figure C1.4.3-2 - Pile Cap Top and Bottom Reinforcement

Case 2: Horizontal earth pressure loads assuming the abutment to act as a continuous beam supported on the girders.

Stirrups designed to resist vertical shear forces acting on the abutment shall be provided.

L-shaped reinforcement bars shall be provided to transfer the connection moment between the abutment and the superstructure. The vertical leg of these connection bars shall be placed as close as practical to the back face of the abutment. The horizontal leg of these bars shall be extended into the deck at the elevation of the deck top longitudinal

The maximum possible earth pressure force on the abutment is the passive earth pressure. For abutments with relatively small thermal movement, the full passive pressure may not develop. The reinforcement shown on the BD-667M Standard Drawings will be sufficient to resist the full passive pressure for all cases of integral abutments. Due to the large thickness of the abutment, this reinforcement is controlled by the minimum reinforcement requirements for most cases.

In some cases in the past, transverse cracks in the deck were observed parallel to the integral abutments. This was attributed to the premature termination of the bars connecting the superstructure to the abutment. The provided reinforcement (#19 @ 250 mm) {#6 @ 10 in.} and their specified embedment in the deck are intended to satisfy the

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reinforcement for a minimum of 900 mm {3 ft.} beyond the front face of the integral abutment. The connection bars shall be #19 @ 250 mm {#6 @ 10 in.} for girders up to 2440 mm {8 ft.} deep. For deeper girders, if such girders were approved by the Chief Bridge Engineer, the L-shaped bars shall be designed to transfer the maximum expected connection moment between the abutment and the superstructure.

1.4.4 Wingwalls

Only U wingwalls (wingwalls parallel to the longitudinal axis of the bridge) shall be used in conjunction with integral abutments.

A 300 mm {1 ft.} chamfer shall be used between the abutment and all wingwalls. The 300 mm {1 ft.} shall be measured from the point of intersection of the back face of the abutment and the wingwall.

Depending on the situation, one of the following three wingwall configurations shall be used:

1. Attached rectangular wingwall: This is the preferred wingwall configuration and shall be used where possible. The maximum length of the rectangular wingwall measured from the back face of the abutment shall be 2660 mm {8'-8"}.
2. Tapered attached wingwalls: Tapered wingwalls shall be used when the length of the wingwall exceeds the maximum length allowed for rectangular wings. The maximum length of the tapered wingwalls measured from the back face of the abutment shall be 5070 mm {16’-7”}. The depth of the wingwall at its free end shall be 600 mm {2 ft.}.
3. Detached wingwalls: Detached wingwalls shall be used when the required length of the wingwall exceeds that allowed for attached wingwalls. A 900 mm {3 ft.} long attached rectangular wing wall shall be used and a compression seal expansion joint will separate the attached portion of the wingwall from the detached portion. The detached portion of the wingwall shall be designed as an independent retaining wall.

Wingwall reinforcement shall be taken as shown on the BD-667M Standard Drawings

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connection requirements for girders up to 2440 mm {8 ft.} deep spaced at up to 4200 mm {13'-9"} girder spacing.

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Wingwalls parallel to the abutment are subject to significant passive pressure and are not allowed to be integral with the abutment.

Rectangular wingwalls simplify construction but they are subject to high earth pressure. As the length of the wingwall increases, tapering the wings will reduce the earth pressure force and will allow the use of longer wings.

Skewed integral abutment bridges tend to twist in the horizontal plane. This results in some wingwalls moving toward the backfill. These movements are not expected to be large enough to produce passive pressure. However, due to the uncertainty of the magnitude of the movements, the wingwalls are checked for the full passive pressure.

Wingwall reinforcement shown on the BD-667M Standard Drawings was checked for the following cases:

1. Passive earth pressure acting on the wingwall
2. Collision force at Performance Level 2 (PL-2) acting near the free end of the wingwall plus active earth pressure.

The soil angle of internal friction used in designing the

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wingwall reinforcement is 30°.

1.5 APPROACH SLAB

Approach slabs shall be provided at each abutment. Approach slabs shall be cast on two (2) layers of 0.1 mm {1/256"} thick polyethylene sheets. The width of the approach slab perpendicular to the direction of traffic shall run from bridge gutter line to bridge gutter line. The orientation of the edge of the approach slab away from the bridge and the length of the approach slab shall be as follows:

- For bridges with skews less than 80.5 degrees, the end of the approach slab away from the bridge shall be parallel to the abutment. The length of the approach slab parallel to traffic shall be 7500 mm {24'-7"}.
- For bridges with skews between greater than 80.5 degrees, the approach slab end away from the bridge shall be at a 1:6 slope to the perpendicular to the direction of traffic, i.e. parallel to typical rigid pavement joints. The length of the shorter of the two edges of the approach slab parallel to the direction of traffic shall be 7500 mm {24'-7"}.

The approach slab shall be designed as a 7500 mm {24'-7"} long simple span slab bridge subjected to all applicable loads. The soil support under the approach slab shall be ignored in the design. The thickness of the approach slab shall be 450 mm {1'-6"}.

A contraction joint shall be located along the edge of the approach slab at the abutment. Form the joint and seal with an approved sealer. The vertical interface between the abutment and the approach slab shall be coated with an approved bonding adhesive prior to pouring the approach slab.

Use bidwell to finish the approach slab.

The approach slab shall be connected to the abutment using epoxy coated dowels extending from the backwall through the bottom of the approach slab near the back face of the backwall. These bars shall be #19 @ 250 mm {#6 @ 10 in.} and will be detailed as shown on the BD-667M Standard drawings.

The approach slab shall rest on the abutment at one end and on a sleeper slab at the other end. The reinforcement and dimensions of the approach and sleeper slabs are shown on the RC-23 Standard Drawings.

C1.5

Providing a reinforced concrete approach slab tied to the bridge deck moves the expansion joint away from the end of the bridge. In addition, the approach slab eliminates settlement due to traffic compaction and backfill settling into the void left when the bridge contracts. It also prevents undermining of the abutments due to drainage at the bridge ends. Approach slabs are required despite the type of roadway pavement (rigid or flexible pavement).

Typical rigid pavement joints are placed at a slope 1:6 relative to the perpendicular to the direction of traffic. This orientation reduces the impact of vehicle tires on the joints by not allowing the two tires in an axle to impact the joint at the same time. A skew of 80.5 degrees will cause the far end of the approach slab to have a slope of 1:6. For skews greater than 80.5 degrees, the end of the approach slab away from the bridge will not be parallel to the abutment resulting in unequal length of the two longitudinal edges of the approach slab.

The 7500 mm {24'-7"} approach slab length is used in the analysis in all cases. The soil support under the approach slab was ignored to account for the possibility of soil settlement and erosion expected to take place directly behind the abutment. When one of the longitudinal edges of the approach slab exceeds 7500 mm {24'-7"}, i.e. for skews greater than 80.5 degrees, the soil is assumed to provide some support under the far end of the approach slab. This support is assumed to reduce the effective span length to 7500 mm {24'-7"}.

Contraction joints at bridge abutments provide a controlled crack location rather than allowing a random crack pattern to develop.

The specified location of the dowels is intended to eliminate the development of negative moments in the approach slab along its connection to the abutment. This allows the approach slab to deflect without causing tension cracking at its top surface.

1.6 EXPANSION JOINTS

For total bridge lengths of 45 000 mm {150 ft.} or less, no provisions for expansion at the end of the approach slab shall be provided when the roadway has flexible pavement. For longer bridges when the roadway has flexible pavement and for all bridges when the roadway has concrete pavement, provisions shall be made for expansion, in the form of a strip seal expansion dam at the end of the approach slab. A short

C1.6

Expansion joint at the end of the approach slab placed over a sleeper slab is a working joint. It opens and closes due to thermal expansion and contraction. The sleeper slab is provided to ensure proper movement of the structure and facilitate proper functioning of the dam. It also provides protection against permanent migration.

SPECIFICATIONS

sleeper slab shall be provided beneath the expansion dam at the end of the approach slab and beginning of the rigid pavement. Details of the expansion joint are shown on the RC-23M Standard Drawing.

When a detached wingwall is used, provide a compression seal expansion joint between the abutment and the detached wingwall. Details of the expansion joint are shown on the BD-667M Standard Drawing.

When possible, the expansion devices at the end of the approach slab and adjacent to detached wingwalls shall have a total range of movement equal to twice the abutment thermal movement, Δ_{max}, calculated using Ap.G.1.2.7.4, but not less than 50 mm {2 in.}. The gap between the two sides of the expansion dam at the time of construction shall be such that it allows the expansion device to be subjected to a displacement Δ_{max} in expansion or in contraction safely.

In the case of a relatively large abutment design movements, expansion devices having a total range of movement twice the abutment thermal movement, Δ_{max}, may not be available. In these cases, the largest available expansion device of the type specified above for each location shall be used. The opening of the expansion dam at the time of construction shall be adjusted based on the expected total abutment movement, Δ_{total}, calculated using Ap.G.1.2.7.4, and the actual construction temperature. The excess capacity of the expansion device beyond Δ_{total} shall be divided equally between expansion and contraction.

COMMENTARY

The calculations of the abutment movements are subject to a relatively high level of uncertainty due to the variation in soil properties and compaction conditions. The specified range of movements for expansion devices is intended to account for these uncertainties.

1.7 BEARING PADS

Plain, 50 durometer neoprene bearing pads shall be placed under all girders. The bearing pads shall be 20 mm {3/4"} thick and 300 mm {1 ft.} wide. The length of the bearing pads shall depend on the width of the bottom flange of the girders. The pads shall be placed as shown on the BD-653M standard drawing.

Block the areas under the girders not in contact with the bearing pads using backer rods.

C1.7

Blocking the areas under the girders not in contact with the bearing pads is intended to prevent honeycombing of the surrounding concrete. Honeycombing will take place when the cement paste enters the 20 mm {3/4"} gap between the bottom of girders and the top of the pile cap in the areas under the girders not in contact with the bearing pads.

1.8 DESIGN DETAILS

Integral abutment standard details are shown on the BD-667M Standard drawings.

REFERENCES

1. W. G. Fleming, A. J. Weltman, M. F. Randolph and W. K. Elson. Piling Engineering. Halsted Press, New York, 1985.

2. L. F. Greimann, R. E. Abendroth, D. E. Johnson and P. B. Ebner, "Pile Design and Tests for Integral Abutment Bridges". Iowa Department of Transportation Report HR-273, 1987.

3. Richard M. Barker and Jay A. Puckett, Design of Highway Bridges, John Wiley and Sons, Inc., 1997.

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 2

APPENDIX H - PENNSYLVANIA INSTALLATION DIRECT DESIGN (PAIDD) FOR CONCRETE PIPES

TABLE OF CONTENTS

1.0 SCOPE

1.1 Appendix H covers the direct design of buried precast concrete pipe using Standard Installations manufactured in accordance with Publication 408, Section 601, and the design and construction of the soil/pipe interaction system, intended for the conveyance of sewage, industrial wastes, storm water and drainage.

1.2 When buried, concrete pipe is part of a composite system comprised of the pipe and the surrounding soil envelope, which interact and contribute to the strength and structural behavior of the system.

1.3 Sections 6 through 14 of Appendix H present the PAIDD method for buried precast concrete pipe. This is a design and analysis method which accounts for the interaction between the pipe and soil envelope in determining loads, pressure distributions, moment, thrust and shear in the pipe, and which includes a procedure for calculating the required reinforcement.

1.4 Section 10 of Appendix H presents construction requirements for precast concrete pipe designed by the PAIDD method.

1.5 Appendix H shall be used by the designer working on Pennsylvania projects in preparing project specifications based on the PAIDD method.

1.6 The design procedures given in Appendix H are intended for use by engineers who are familiar with the installation and pipe characteristics that affect the structural behavior of buried concrete pipe installations and the significance of the installation requirements associated with each Standard Installation Type. Before applying the design procedures given in Sections 6 through 14, the engineer should review the guidance and requirements given in other sections of Appendix H and its reference documents.

2.0 APPLICABLE DOCUMENTS

2.1 ASTM (AMERICAN SOCIETY FOR TESTING AND MATERIALS)

2.1.1 C 822 Terminology Relating to Concrete Pipe and Related Products

2.1.2 D 2487 Classification of Soils for Engineering Purposes

2.1.3 D 2488 Practice for Description and Identification of Soils (Visual-Manual Procedure)

2.1.4 D 698 Test Method for Laboratory Compaction Characteristics of Soil using Standard Effort (12,400 ft-lbf/ft^3 (600 kN-m/m^3))

2.1.5 D 1557 Test Method for Laboratory Compaction Characteristics of Soil using Modified Effort (56,000 ft-lbf/ft^3 (2700 kN-m/m^3))

2.2 AASHTO (AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS)

2.2.1 T 99 The Moisture-Density Relations of Soils using a 2.5 kg (5.5-lb) Rammer and a 305 mm (12-in.) Drop

2.2.2 T 180 The Moisture-Density Relations of Soils using a 4.54 kg [10-lb] Rammer and an 457 mm [18-in.] Drop

2.3 PENNDOT (PENNSYLVANIA DEPARTMENT OF TRANSPORTATION)

2.3.1 Design Manual, Part 4, Structures

3.0 DEFINITIONS

3.1 For definitions of terms relating to concrete pipe, see ASTM Definitions C 822.

3.2 For terminology related to soil classifications, see ASTM Classification D 2487 and ASTM Practice D 2488.

3.3 For terminology and definition of terms relating to structural design, see PennDOT Design Manual, Part 4 Structures.

3.4 Orientation Angle - An angular tolerance assumed for the position of the top of the pipe during the design of a pipe requiring a specific installation orientation because of the reinforcement cage configuration or the positioning of stirrups.

3.5 Prism Load - Weight of column of earth above the outside diameter of pipe.

3.6 Figure 1 illustrates the definitions and limits of the terms foundation, subgrade, bedding, haunch, lower side, sidefill, backfill or overfill, invert, crown, springline, top of pipe and bottom of pipe as used in this Standard Practice.

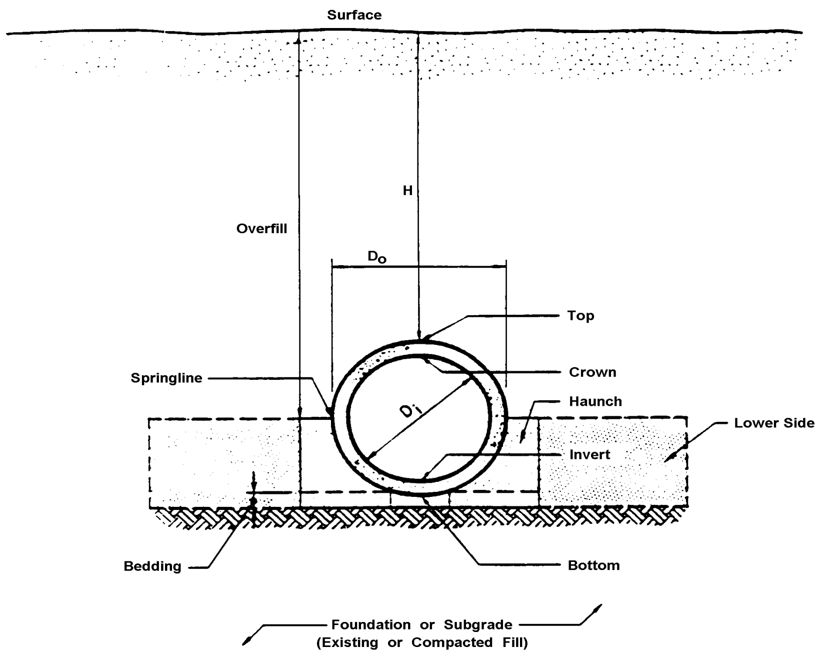

Figure 3.6-1 - Pipe/Installation Terminology

4.0 NOTATIONS

A_s = required tension reinforcement area (mm^2/m) {in^2/ft.}
A_{si} = required total inner cage reinforcement area (mm^2/m) {in^2/ft}
A_{so} = required total outer cage reinforcement area (mm^2/m) {in^2/ft}
A_{vr} = required stirrup reinforcement area to resist radial tension forces (mm^2/m) {in^2/ft} in each line of stirrups at circumferential spacing s_v
A_{vs} = required stirrup reinforcement area to resist shear (mm^2/m) {in^2/ft} in each line of stirrups at circumferential spacing, s_v
B_1 = crack control coefficient for effect of spacing and number of layers of reinforcement
b = width of section which resists stress, 1000 mm/m {12 in/ft}
C_1 = crack control coefficient for type of reinforcement
d = distance from compression face to centroid of tension reinforcement (mm) {in.}
D_i = inside diameter of pipe (mm) {in.}
D_o = outside diameter of pipe (mm) {in.}
f_s = maximum service load strength of reinforcing steel for crack control (MPa) {ksi}
f_v = maximum allowable strength of stirrup material (MPa) {ksi}
f_y = design yield strength of reinforcement (MPa) {ksi}
F_c = factor for effect of curvature on diagonal tension (shear) strength in curved components
F_d = factor for crack depth effect resulting in increase in diagonal tension (shear) strength with decreasing d

F_{cr}	=	crack width control factor for adjusting crack control relative to an average maximum crack width of 0.25 mm {0.01 in.} at 25 mm {1 in.} from the tension reinforcement when $F_{cr} = 1.0$
F_{rp}	=	factor for process and materials that affect the radial tension strength of pipe
F_v	=	factor for crack depth effect resulting in increase in diagonal tension (shear) strength with decreasing d
F_{vp}	=	factor for process and materials that affect the shear strength of pipe
F_{rt}	=	factor for pipe size effect on radial tension strength
F_N	=	coefficient for effect of thrust on shear strength
f'_c=		design compressive strength of concrete (MPa) {ksi}
g_c	=	gravitational acceleration (m/s^2) {ft/sec^2}
h	=	overall thickness of member (wall thickness) (mm) {in.}
H	=	design height of earth above top of pipe (mm) {in.}
i	=	coefficient for effect of axial force at service load stress, f_r
l_θ	=	total additional arc length beyond calculated arc lengths requiring stirrups (mm) {in.}
M_s	=	service load bending moment (N mm/m) {K in/ft}
M_u	=	factored moment acting on cross-section (N mm/m) {K in/ft}
M_{nu}	=	factored moment as modified for effects of compressive or tensile thrust (N mm/m) {K in/ft}
n	=	number of layers of reinforcement in a cage, 1 or 2
N_s	=	axial thrust acting on cross-section, service load condition (+ when compressive; - when tensile) (N/m) {kip/ft}
N_u	=	factored axial thrust acting on cross-section (+ when compressive, - when tensile) (N/m) {kip/ft}
PL	=	PL denotes the prism load (weight of the column of earth) over the pipe's outside diameter and is calculated as:

Metric Units: $PL = \dfrac{g_c w}{1x10^6}\left[H + .12D_o\right]$

U.S. Customary Units: $PL = \dfrac{wD_o}{12}\left[H + 0.01D_o\right]$

r	=	radius to centerline of pipe wall (mm) {in.}
r_s	=	radius of the inside reinforcement (mm) {in.}
s_v	=	circumferential spacing of stirrups (mm) {in.}
s_l	=	spacing of circumferential reinforcement (mm) {in.}
t_b	=	clear cover over reinforcement (mm) {in.}
V_b	=	basic shear strength of critical section, at $M_{nu}/(V_u d) = 3.0$
V_c	=	nominal shear strength provided by concrete cross-section (N/m) {kip/ft}
V_u	=	factored shear force acting on cross-section (N/m) {kip/ft}
V_{uc}	=	factored shear force at critical section, where $M_{nu}/(V_u d) = 3.0$
w	=	unit weight of soil (kg/m^3) {kcf}
ρ	=	ratio of reinforcement area to concrete area
φ_f	=	strength reduction factor for flexure
φ_v	=	strength reduction factor for shear
φ_r	=	strength reduction factor for radial tension
θ	=	orientation angle ( )

5.0 SUMMARY OF PRACTICE

5.1 The PAIDD method is a direct design method that accounts for the interaction between the pipe and soil envelope in determining loads and distribution of earth pressure on a buried pipe.

5.2 The two Standard Installations, embankment and shoring/trench box, that are a part of this Standard Practice are based on the results of research on pipe/soil interaction, together with evaluation of current construction practice, equipment, procedures and experience.

5.3 Earth load effects are determined from the pressure distribution coefficients for the two Standard Installations (see Standard Drawing BD-636M, Sheet 1 for Soil Pressure Distribution Model and Sheet 2 for Tables B and C).

5.4 The structural design of concrete pipe is based on a limits state design procedure that accounts for strength and serviceability criteria. The design criteria include: structural aspects, such as flexure, thrust, shear and radial tension strengths; handling and installation; and crack width control.

5.5 The design of a concrete pipe for these particular Standard Installations is based on the assumption that the specified design bedding and fill requirements will be achieved during construction of the installation.

6.0 GENERAL

6.1 Design procedures and criteria shall conform to applicable sections of this Standard Practice.

6.2 The manufacturer shall submit to the owner for approval a detailed design of the pipe based on the Owner's Design Requirements.

7.0 DESIGN REQUIREMENTS

7.1 The designer shall establish the following criteria and requirements:

7.1.1 Intended use of pipeline.

7.1.2 Pipe inside diameter, D_i.

7.1.3 Pipeline plan and profile drawings with installation cross-sections as required.

7.1.4 Design earth cover height above the top of the pipe, H (see Figure 3.6-1).

7.1.5 Allowable Standard Installation Types. Embankment or shoring/trench box installations.

7.1.6 Performance requirements for pipe joints, for sanitary installation.

7.1.7 Design live and surcharge loading, if heavier than standard PennDOT design loads.

7.1.8 Design intermittent internal hydrostatic pressures, if required.

7.1.9 Crack width control criteria, other than specified in Section 7.2, if permitted by the Chief Bridge Engineer.

7.2 The following design requirements shall apply:

7.2.1 Load Factors

Dead and Earth Load Factor (Shear and Moment)		1.3
Dead and Earth Load Factor (Thrust)		
Reinforcement Design		1.0
Concrete Compression		1.3
Live Load Factor (Shear and Moment)		2.17
Impact Factors	Metric Units: IM = 40 (1.0-0.0004H)	
	U.S. Customary Units: IM = 40 (1.0-0.125H)	

7.2.2 Strength Reduction (φ) Factors

	Heavy Duty	Standard Duty
Flexure	$\phi_f = 0.9$	$\phi_f = 0.95$
Radial Tension	$\phi_v = 0.85$	$\phi_v = 0.9$
Diagonal Tension	$\phi_r = 0.85$	$\phi_r = 0.9$

7.2.3 Crack Control Factor 0.7
(unless modified by the owner)

7.2.4 Orientation Angle $\theta = \pm 10^\circ$

7.2.5 Process and Material Factors

Radial Tension	1.0
Diagonal Tension	1.0

7.3 The manufacturer shall submit the following manufacturing design data to the owner for approval.

7.3.1 Pipe Wall Thickness

7.3.2 Concrete Strength

7.3.3 Reinforcement specification, reinforcement Type 2 or 3 as shown in Table 12.4.1-1, design yield strength, placement and design concrete cover, cross-sectional diameters, spacing, cross-sectional area, description of longitudinal members, and if used, developable stirrup design stress, stirrup shape, placement and anchorage details.

7.3.3.1 Crack Control Coefficient, C_1, as indicated in Table 12.4.1-1. Note that Standard Drawing BD-636M and PAIDD software have values built-in for Type 2 reinforcement only.

7.3.3.2 The minimum design concrete cover over the reinforcement shall be 25 mm {1 in.} in pipe having a wall thickness of 64 mm {2 1/2 in.} or greater, and 19 mm {3/4 in.} in pipe having a wall thickness of less than 64 mm {2 1/2 in.}. For stirrup design provide 32 mm {1 1/4 in.} of concrete cover to the circumferential reinforcement.

7.3.4 Pipe laying length and joint information.

7.3.5 The yield strength and ultimate strength of the tension reinforcement used for design shall be as specified in 8.2.1 or 8.2.2.

8.0 MATERIALS

8.1 CONCRETE

8.1.1 Concrete shall conform to the requirements of Publication 408, Section 601.

8.2 REINFORCEMENT

8.2.1 Reinforcement shall consist of cold-drawn steel wire conforming to ASTM Specification A 82, or ASTM Specification A 496, or of cold-drawn steel welded wire fabric conforming to ASTM Specification A 185, or ASTM Specification A 497, or of hot-rolled steel bars conforming to ASTM Specification A 615/A 615M.

8.2.2 The use of cold drawn steel or cold drawn steel welded wire fabric with strengths exceeding ASTM Specification values may be approved by the owner when the reinforcing manufacturer's mill test report certifies a higher minimum yield and ultimate strength steel is being provided. The other requirements of the appropriate ASTM Specifications listed in 8.2.1 (A 82, A 496, A 185 or A 497) shall be met by the higher minimum strength steels.

The yield strengths shall not be taken greater than 86% of the ultimate strength, or 552 MPa {80 ksi}, whichever is lower. Note that Standard Drawing BD-636M and PAIDD use 448 MPa {65 ksi} yield strength.

8.2.2.1 Section 8.2.2 does not apply to wire sizes having a nominal diameter of less than 2.03 mm {0.088 in.} or nominal cross-sectional area of less than 3.23 mm^2 {0.005 in^2}. Section 8.2.2 does not apply to (any size of) hot-rolled steel manufactured in accordance with ASTM A 615/A 615M.

9.0 LOADS

9.1 DEAD LOADS

9.1.1 The dead load of the pipe weight shall be considered in the design and based on a reinforced concrete density of 2400 kg/m^3 {0.150 kcf}, unless otherwise specified.

9.1.2 The earth load from the fill over the pipe shall be based on the design soil unit density of <u>2250 kg/m^3 {0.140 kcf}</u>.

9.1.3 For unpaved and flexible pavement areas, the minimum fill, including flexible pavement thickness, over top outside of the pipe shall be 300 mm {1 ft.}, or 1/8 of the inside diameter, whichever is greater. Under rigid pavements, the distance between the top of the pipe and the bottom of the pavement slab shall be a minimum of 230 mm {9 in.} of compacted granular fill. However, to avoid conflict with other design and field constraints refer to Standard Drawing RC-30M for vertically locating the pipe.

9.1.4 The dead load of fluid in the pipe shall be based on a unit density of 1000 kg/m^3 {0.0624 kcf}, unless otherwise specified.

9.2 LIVE LOADS

9.2.1 Truck loads shall be either the AASHTO {<u>HS25</u>} or the increased AASHTO Interstate Design Load. An impact factor shall be:

Metric Units: $IM = 40(1.0 - 0.0004H)$ (9.2.1-1)

U.S. Customary Units: $IM = 40(1.0 - 0.125H)$

9.2.2 Railroad loads shall be the AREA designated Cooper E-series.

9.2.3 Aircraft or other live loads shall be as specified by the owner.

9.3 INTERMITTENT INTERNAL HYDROSTATIC PRESSURE

9.3.1 Internal hydrostatic pressure caused by hydraulic surcharges or other temporary hydraulic conditions shall be as specified by the owner.

10.0 STANDARD INSTALLATIONS

10.1 Soil classifications are in accordance with ASTM Classification D 2487 and Practice D 2488. For step-by-step installation refer to Standard Drawing RC-30M.

10.2 The soil-type and compaction requirements for the Standard Installations are presented in Standard Drawings RC-30M and BD-636M. Table 1 relates the Standard Installations designated soils to the AASHTO and Unified Soil System Classifications categories.

Table 10.2-1 - Equivalent USCS and AASHTO Soil Classifications for SIDD Soil Designations

SIDD Soil	Representative Soil Types		Percent Compaction	
	USCS	AASHTO	Standard Proctor	Modified Proctor
Gravelly Sand (SW)	SW, SP GW, GP	A1, A3	100 95 90 85 80 61	95 90 85 80 75 59
Sandy Silt (ML)	GM, SM, ML Also GC, SC with less than 20% passing #200 sieve	A2, A4	100 95 90 85 80 49	95 90 85 80 75 46
Silty Clay (CL)	CL, MH, GC, SC	A5, A6	100 95 90 85 80 45	90 85 80 75 70 40
	CH	A7	100 95 90 45	90 85 80 40

10.3 The soil types and compaction requirements for the Standard Installations are defined as follows:

10.3.1 Soil Materials

SW, Well-graded sands, gravelly sands, little or no fines (also includes GW - well graded gravels, gravel-sand mixtures little or no fines, and crushed stone).

ML, Inorganic silts, fine sands or clayey silts with slight plasticity. (May be used for native granular soils.)

CL, Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays.

10.3.2 Compaction Specifications

SP, Standard Proctor Density - AASHTO T 99 or ASTM D 698

MP, Modified Proctor Density - AASHTO T 180 or ASTM D 1557

L, Loose - Uncompacted

10.3.3 In-Situ Materials (INS)

R - Rock
F - Firm to hard
M - Medium
A - Average
S - Soft
SCR - Loosened by scarifying

11.0 PRESSURE DISTRIBUTION

11.1 The pressure distributions on the pipe from applied loads and bedding reaction as determined from soil-structure analyses for the Standard Installations are presented in Standard Drawing BD-636M. Also listed are the relative dimensions and coefficients, based on a unit prism load on the pipe. The use of the computer program "PAIDD" simplifies the analysis of a pipe for the moments, thrusts and shears caused by these pressure distributions.

12.0 REINFORCEMENT

12.1 Reinforcement for Flexural Strength shall be not less than A_s, where:

Metric Units: $A_s f_y = g\phi_f d - N_u - \sqrt{g\left[g\left(\phi_f d\right)^2 - N_u\left(2\phi_f d - h\right) - 2M_u\right]}$ (12.1.1)

U.S. Customary Units: $A_s f_y = g\phi_f d - N_u - \sqrt{g\left[g\left(\phi_f d\right)^2 - N_u\left(2\phi_f d - h\right) - 2M_u\right]}$

Metric Units: $g = bf_c'\left[0.85 - 0.05\dfrac{\left(f_c' - 28\right)}{7}\right]$ (12.1-2)

U.S. Customary Units: $g - bf_c'\left[0.85 - 0.05\left(f_c' - 4\right)\right]$

$g_{\max} = 0.85bf'_c$

$g_{\max} = 0.65bf'_c$

b=1000 mm/m {12 in/ft}

12.2 MINIMUM REINFORCEMENT

12.2.1 The following equation applies only to circular pipe larger than 838 mm {33 in.} in diameter with circular reinforcement. For inside face of pipe, reinforcement shall not be less than A_{si}, where:

Metric Units: $A_{si} = \dfrac{\left(D_i + h\right)^2}{40 f_y}$ (12.2.1-1)

U.S. Customary Units: $A_{si} = 1.1\dfrac{\left(D_i + h\right)^2}{1000 f_y}$

to account for transportation stresses.

12.2.2 For outside face of pipe, reinforcement shall not be less than A_{so}, where:

Metric Units: $$A_{so} = \frac{(D_i + h)^2}{68 f_y} \quad (12.2.2\text{-}1)$$

U.S. Customary Units: $$A_{so} = 0.65 \frac{(D_i + h)^2}{1000 f_y}$$

12.2.3 For pipe 838 mm {33 in.} diameter and smaller, with a single cage reinforcement in the middle third of the pipe wall, reinforcement shall not be less than A_s, where:

Metric Units: $$A_s = \frac{(D_i + h)^2}{20 f_y} \quad (12.2.3\text{-}1)$$

U.S. Customary Units: $$A_s = 2.2 \frac{(D_i + h)^2}{1{,}000 f_y}$$

12.3 MAXIMUM FLEXURAL REINFORCEMENT WITHOUT STIRRUPS OR TIES

12.3.1 When stirrups are not used, the wall shall be designed so that the flexural reinforcement required by 12.1 does not exceed "A_s max" as determined by 12.3.2 or 12.3.3.

12.3.2 Limited by Radial Tension

Metric Units: $$A_{s\max} f_y = 110.68 r_s F_{rp} \sqrt{f_c'} \left(\frac{\phi_r}{\phi f} \right) F_{rt} \quad (12.3.2\text{-}1)$$

U.S. Customary Units: $$A_{s\max} f_y = 0.506 r_s F_{rp} \sqrt{f_c'} \left(\frac{\phi_r}{\phi_f} \right) F_{rt}$$

Metric Units: $F_{rt} = 1 + 0.000327\,(1800 - D_i)$ for 300 mm < $D_i \leq$ 1800 mm (12.3.2-2)

U.S. Customary Units: $F_{rt} = 1 + 0.00833\,(72 - D_i)$ for 12 in. < $D_i \leq$ 72 in.

Metric Units: $$F_{rt} = \frac{(3660 - D_i)^2}{16\,774\,160} + 0.80 \text{ for } 1800\,mm < D_i \leq 3660\,mm \quad (12.3.2\text{-}3)$$

U.S. Customary Units: $$F_{rt} = \frac{(144 - D_i)^2}{26{,}000} + 0.80 \text{ for } 72\ in < D_i \leq 144\,in$$

Metric Units: $F_{rt} = 0.8$ *for* $D_i > 3660\,mm$ (12.3.2-4)
U.S. Customary Units: $F_{rt} = 0.8$ *for* $D_i > 144\,in.$

12.3.3 **Limited by Concrete Compression**

Metric Units: $$A_{s\max} f_y = \left[\frac{380 g' \phi_f d}{600 + f_y} \right] - 0.75 N_u \quad (12.3.3\text{-}1)$$

U.S. Customary Units: $A_{s\max} f_y = \left[\frac{55g' \phi_f d}{87 + f_y}\right] - 0.75N_u$

Metric Units: $g' = bf_c' \left[0.85 - 0.05\frac{(f_c' - 28)}{7}\right]$ (12.2.2-2)

U.S. Customary Units: $g' = bf_c' \left[0.85 - 0.05\frac{(f_c' - 4)}{1}\right]$

$g'_{max} = 0.85\, bf'_c$

$g'_{min} = 0.65\, bf'_c$

b = 1000 mm/m {12 in/ft}

12.4 CRACK CONTROL

12.4.1 Crack control is assumed to be at 25 mm {1 in.} from the tension reinforcement. The crack control factor, F_{cr}, shall not exceed that specified, where:

Metric Units: $F_{cr} = \frac{B_1}{206.83\phi_f dA_s}\left[\frac{M_s + N_s\left(d - \frac{h}{2}\right)}{ij} - 0.083C_1 bh^2\sqrt{f_c'}\right]$ (12.4.1-1)

U.S. Customary Units: $F_{cr} = \frac{B_1}{30{,}000\phi_f dA_s}\left[\frac{M_s + N_s\left(d - \frac{h}{2}\right)}{ij} - 0.0316C_1 bh^2\sqrt{f_c'}\right]$

C_1 – See Table 1

b = 1000 mm/m {12 in/ft}

$j = 0.74 + 0.1\frac{e}{d}$

$j_{\max} = 0.9$

$i = \frac{1}{1 - \frac{jd}{e}}$ (12.4.1-3)

$e = \frac{M_s}{N_s} + d - \frac{h}{2}, mm\ \{in.\}$ (12.4.1-4)

if $e/d_{min} < 1.15$ crack control will not govern

Metric Units: $B_1 = \sqrt[3]{\frac{t_b s_l}{2n(25.4)^2}}$ (12.4.1-5)

U.S. Customary Units: $B_1 = \sqrt[3]{\frac{t_b s_l}{2n}}$

s_l = spacing of circumferential reinforcement, mm {in.}

n = 1, when tension reinforcement is a single layer

n = 2, when tension reinforcement is made of multiple layers

Table 12.4.1-1 – Crack Control Coefficients

Type of Reinforcement	C_1
Smooth wire or plain bars	1.0
Welded smooth wire fabric, 200 mm {8 in.} maximum spacing of longitudinals	1.5
Welded deformed wire fabric, deformed wire, deformed bars or any reinforcement with stirrups anchored thereto.	1.9

12.4.2 If the service load thrust, N_s is tensile rather than compressive (this may occur in pipes subject to intermittent hydrostatic pressure), the quantity (1.1 M_s-0.6N_sd) shall be used (with tensile N_s taken negative) in place of the quantity [M_s + N_s (d - h/2)]/ij in Equation 1.

12.5 SHEAR STRENGTH

12.5.1 The area of reinforcement, A_s, determined in Section 12.1 and 12.4, shall be checked for shear strength adequacy, so that the basic shear strength, V_b, is greater than the factored shear force, V_{uc}, at the critical section located where $M_{nu}/V_u d$ = 3.0. See Equation 7 for M_{nu}.

Metric Units: $$V_b = 0.083 b \phi_v d F_{vp} \sqrt{f_c'}(1.1 + 63\rho)\left[\frac{F_d F_N}{F_c}\right] \qquad (12.5.1\text{-}1)$$

U.S. Customary Units: $$V_b = 0.0316 b \phi_v d F_{vp} \sqrt{f_c'}(1.1 + 63\rho)\left[\frac{F_d F_N}{F_c}\right]$$

b = 1000 mm/m {12 in/ft}

$f'_{c\ max}$ = 48 MPa {7 ksi}

$$\rho = \frac{A_s}{bd}$$
$$\rho_{max} = 0.02 \qquad (12.5.1\text{-}2)$$

Metric Units: $$F_d = 0.8 + \frac{40.64}{d} \qquad (12.5.1\text{-}3)$$

U.S. Customary Units: $$F_d = 0.8 + \frac{1.6}{d}$$

max F_d = 1.3, for pipe with two cages

max F_d = 1.4, for pipe through 914 mm {36 in.} diameter with a single circular cage

$$F_c = 1 \pm \frac{d}{2r} \qquad (12.5.1\text{-}4)$$

(+) tension on the inside of the pipe
(-) tension on the outside of the pipe

For compressive thrust (+N_u):

Metric Units: $F_N = 1 + \frac{N_u}{13789h}$ (12.5.1-5)

U.S. Customary Units: $F_N = 1 + \frac{N_u}{24h}$

For tensile thrust (-N_u):

Metric Units: $F_N = 1 + \frac{N_u}{3447h}$ (12.5.1-6)

U.S. Customary Units: $F_N = 1 + \frac{N_u}{6h}$

$$M_{nu} = M_u - N_u \frac{(4h - d)}{8} \quad (12.5.1\text{-}7)$$

12.5.2 If V_b is less than V_{uc} stirrups shall be provided. See Section 12.6.

12.6 STIRRUPS

12.6.1 If stirrups are required for radial tension by 12.3.2 or for shear by 12.5, they shall meet the following requirements.

12.6.2 Radial Tension Stirrups

$$A_{vr} = \frac{1.1 s_v (M_u - 0.45 N_u \phi_r d)}{f_v r_s \phi_r d} \quad (12.6.2\text{-}1)$$

$f_{v\,max} = f_y$ or anchorage strength, whichever is less

$s_{v\,max} = 0.75\phi_r d$ (12.6.2-2)

12.6.3 Shear Stirrups

$$A_{vs} = \frac{1.1 s_v}{f_v \phi_v d}\left[V_u F_c - V_c\right] + A_{vr} \quad (12.6.3\text{-}1)$$

$$V_c = \frac{4V_b}{\frac{M_{nu}}{V_u d} + 1} \quad (12.6.3\text{-}2)$$

Metric Units: $V_{c\,\max} = 0.166\phi_v bd\sqrt{f'_c}$ (12.6.3-3)

U.S. Customary Units: $V_{c\,\max} = 0.0632\phi_v bd\sqrt{f'_c}$

b = 1000 mm/m {12 in/ft}

12.6.4 Extent of Stirrups

12.6.4.1 When stirrups are required at the invert or crown regions for shear strength, or for shear and radial tension, they shall be spaced at s_v along the inner reinforcing and extend over a basic length on each side of the invert or crown where V_u is greater than V_c, plus an additional minimum arc length 0.5 l_θ from each end of the basic arc length to allow for installation variations up to the orientation angle, θ, where

$$l_\theta = \frac{\pi\theta}{180}\left(D_i + 2t_b\right) + h \qquad (12.6.4.1\text{-}1)$$

If stirrups are also required at the springline region (may occur in very high loading conditions), they shall be spaced at s_v and shall extend around the entire pipe circumference.

12.6.4.2 When stirrups are required for radial tension only, they shall be spaced at s_v along the inner reinforcing and extend over a basic length to the locations on each side of the invert or crown where the required A_s f_y is less than the limiting value given by Equation 12.1-1, plus an additional minimum arc length 0.5 l_θ from each end of the basic arc length to allow for installation variations up to the orientation angle, θ.

12.6.5 Stirrup Anchorage

12.6.5.1 Anchorage of both ends of the stirrup shall be sufficient to develop the factored stress in the stirrup. The maximum factored tensile stress in the stirrup shall be the yield strength or the stress that can be developed by anchorage shall be considered to be 448 MPa {65 ksi} for continuous stirrups, prefabricated stirrup mats and bent wire shear steel as shown in PAIDD Manufacturing Standard (Publication 280).

12.6.5.2 Anchorage systems that differ from those shown in PAIDD Manufacturing Standard (Publication 280) and corresponding anchorage design stresses may be submitted to the owner for approval.

12.7 LAPS, SPLICES AND DEVELOPMENT OF REINFORCEMENT

13.0 PIPE DESIGNATION

13.1 The pipe designation shall be in accordance with Publication 408, Section 601.

14.0 MULTIPLE PIPE INSTALLATIONS

Multiple pipe installations are typically used at locations where limited cover prevents the use of a single pipe of larger cross-sectional area. A minimum of 600 mm {2 ft.} clearance between adjoining pipes shall be provided. See Figures 1 and 2.

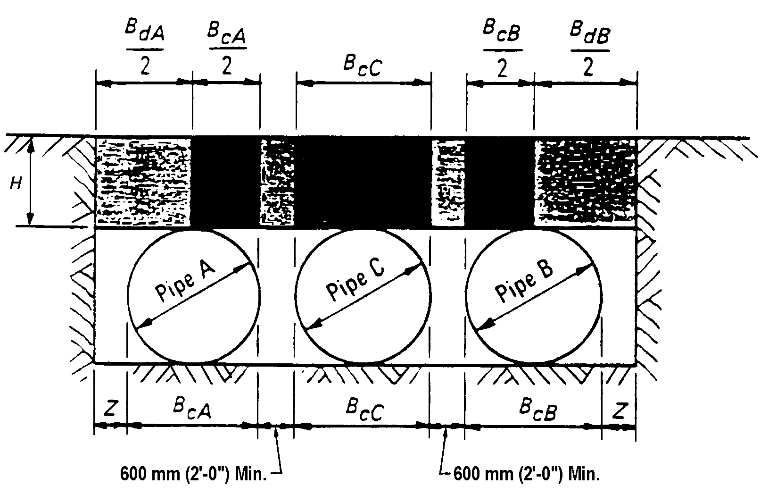

Figure 14.0-1 - Flat Trench Multiple Pipe Installation

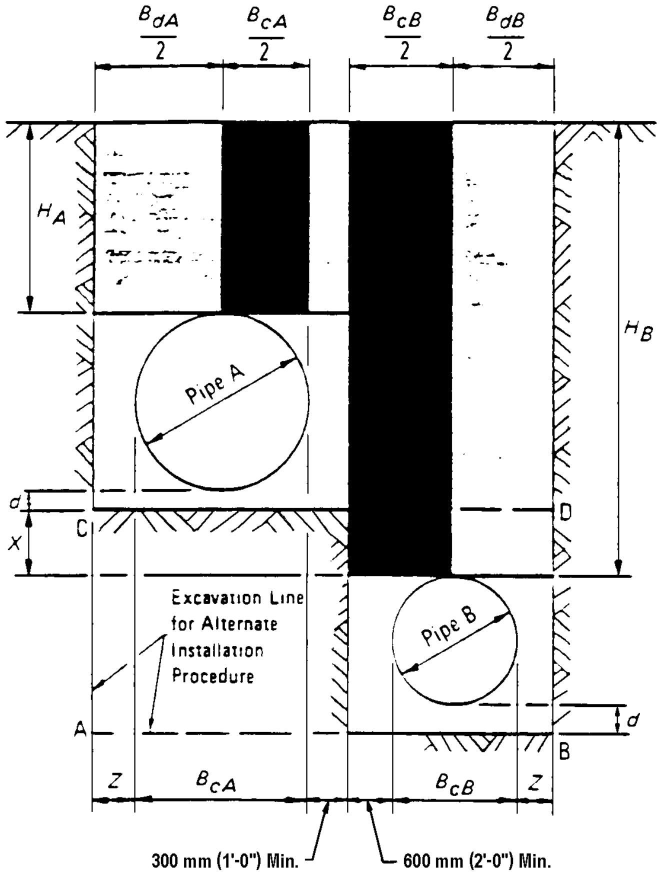

Figure 14.0-2 - Benched Trench Installation

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APPENDIX I - EXTENSIBLE REINFORCEMENTS

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SPECIFICATIONS

1.1 EXTENSIBLE REINFORCEMENT

Extensible reinforcement may be used for MSE walls if their performance has been substantiated either by tests or experience and approval by the Department. Reinforcements may be manufactured from long chain polymers, such as polypropylene, HDPE and polyester. The design of MSE walls with extensible reinforcements shall comply with provisions of A11.10 and D11.10, except as modified in this Appendix.

COMMENTARY

C1.1

The most commonly used extensible reinforcement materials are geotextiles and geogrids made of polyethylene, polyester and polypropylene. The three main types of geotextiles are wovens, nonwovens and knitted. Only woven and nonwoven geotextiles are used for soil reinforcement applications. There are two main types of geogrids, those manufactured by drawing a perforated polymer sheet in one or two perpendicular directions and those manufactured by overlapping perpendicular or nearly perpendicular polymer strands, which are then bonded at their junctions.

Extensible reinforcements are characterized by reinforcement tensile moduli that are relatively low and result in working reinforcement strains greater than those associated with full mobilization of active lateral earth pressures. This property significantly affects the state of stress in the reinforced zone and the location of the internal failure zone and determines the outward deflection of the structure. Extensible reinforcement tensile moduli are significantly lower by one or two orders of magnitude than the tensile moduli of inextensible reinforcements such as steel.

Polymers at present do not have sufficient in-ground use to predict behavior based on observed performance. Experience based on over 20 years with plastic pipe performance strongly suggests that time, temperature, mechanical damage, molecular structure, as affected by radiation (UV) and chemical exposure, microbiological attack, environmental stress cracking hydrolysis, plasticization and synergistic effects from all these variables can significantly affect stress strain behavior and durability for in-ground service.

To enhance durability, additives are incorporated in the manufacturing process to prevent high temperature degradation during the extrusion process: to protect against UV attack by the addition of finely divided Carbon Black which screens the polymer or hindered amine light stabilizers (HALS) which function by interrupting the radical chain degradation mechanism, and with the inclusion of long-term anti-oxidants to further enhance the inherent resistance of polymers to the aging process. The degree of protection or total resistance achieved is based on the selection of proper additives which will vary from product-to-product.

To protect against UV exposure polymers generally contain for maximum protection 2.0% to 3.0% of finely divided carbon black well dispersed in the polymer. Alternately, the addition of hindered amine light stabilizers (HALS) has been shown to be effective in providing UV resistance.

Environmental Stress Cracking (ESC) is the process under which premature failure can occur without chemical change in polymers subjected to concurrent stress in a particular environment. Uniaxially stretched geogrids may be prone to environmental stress cracking in the rib portion which is generally non-oriented.

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COMMENTARY

For polyester fibers in certain forms, degradation has been reported as a result of hydrolysis generated by absorption of water by the fibers in alkaline environments, by thermal degradation or due to UV exposure. For maximum resistance to strength losses due to hydrolysis, polyesters should be formulated to high molecular weights (>25,000) and with low Carboxyl End Group numbers (CEG < 25) and should not be used in highly acidic or alkaline environments (i.e., if the pH is less than 3.0 or more than 9.0).

Polypropylenes are only slightly affected by hydrolysis as their absorption characteristics of water are considerably smaller than those of polyesters, but lose strength due to oxidation, especially in the presence of transition metals (F_e, C_u, M_n, etc. compounds) in the backfill.

The inclusion of effective primary, secondary and synergistic antioxidants are essential in extending the lifetimes of all polyoleofin products. New anti-oxidants are under development to extend survivability. Other aging losses have not been systematically evaluated.

Significant research is presently underway to quantify potential strength losses as a function of polymer characteristics and environment.

1.1.1 Technical Considerations, Restrictions

Extensible reinforcement may be used in lieu of steel inextensible reinforcements. It is particularly well suited where the soil environment is aggressive and corrosion is anticipated or where stray current sources are present, or where the fill is expected to significantly settle differently both in the longitudinal or transverse direction.

Restrictions outlined in A11.10.1 and D11.10.1 shall apply, except as follows:

(a) No impervious membrane is necessary where wall supporting roadways are deiced with chemical additives.

(b) Walls shall be limited to 10 m {35 ft.} heights.

C1.1.1

MSE walls with extensible reinforcements are not generally economical at greater heights.

1.1.2 Structure Dimensions

Requirements for structure dimensioning, embedment depth, bearing capacity of foundations, external stability considerations and drainage shall be as per the applicable sections of A11.10 and D11.10.

An additional requirement is the determination of the factor of safety against sliding of the reinforced backfill directly on a layer of reinforcement. For this calculation, the soil reinforcement interface friction shall be determined.

For geotextiles or geogrids, the angle of friction associated with direct sliding of the reinforced backfill on the geotextile or geogrid, φsg, shall be obtained from direct shear tests carried out in accordance with ASTM D 5321 for Soil-Geosynthetic Friction.

In the absence of specific test data, the angle of soil

C1.1.2

FHWA DP82-1 provides further details of the use of

SPECIFICATIONS

reinforcement friction, ϕsg, shall be taken as 2/3 of the internal angle of friction of the backfill used in the reinforced zone.

1.1.3 Loading

The requirements of A11.10.3 and D11.10.3 shall apply. (the referenced sections will be verified and updated to reflect the new DM 4 Section 11 when it is finalized)

1.1.4 Internal Stability

Internal stability for structures constructed with polymeric reinforcements shall be analyzed using a tie-back wedge method approach.

It is assumed that the full shear strength of the reinforced fill is mobilized and active lateral earth pressures are developed. The assumed failure plane is defined by the Rankine active earth pressure zone defined by a straight line passing through the wall toe and oriented at an angle of $45^\circ + \phi/2$ from the horizontal for both horizontal and sloping backfill conditions, and where ϕ is the effective friction angle of the soil in the reinforced zone.

The tensile force in the reinforcement is a function of the vertical stress due to self weight, uniform normal surcharge and active thrust multiplied by K_a. Reinforcement tensions induced by vertical or horizontal line loads, or by point loads shall be added by superposition to the tensile forces induced by the reinforced wall fill soil and the retained backfill. The method of computation shall assume an unyielding rigid wall rotating about its toe.

The value of K_a in the reinforced soil mass is assumed to be independent of all external loads, except sloping fills. The maximum friction angle used for the computation of horizontal stress within the reinforced soil mass, composed of select backfill, shall be 34° in the absence of backfill specific tests. Where site-specific tests are performed, the soil strength shall be evaluated at residual stress levels.

Minimum grid or geotextile length in the resistant zone shall be 1 m {3 ft.}. The total reinforcement length at any level shall be equal to the sum of the lengths in the active and resistant zone, but shall be in no case less than 2.4 m {8 ft.}.

1.1.5 Movements Under Service Limit State

The provisions of A11.10.4 shall apply.

Horizontal wall movements may approach H/75, mainly during construction or roughly three times the movements anticipated with extensible reinforcement.

1.1.6 Safety Against Soil Failure

The provisions of A11.10.5 and D11.10.5 shall apply.

1.1.7 Safety Against Structural Failure

COMMENTARY

ϕsg taken as 2/3 of the internal angle of friction of the backfill.

C1.1.4

The tie-back wedge procedure is a limit equilibrium method of design that assumes full mobilization of the reinforced fill shear resistance and generation of active lateral earth pressures. It is assumed that the full shear strength of the reinforced fill is mobilized so that the soil mass bounded by the failure plane moves as a rigid body and active lateral earth pressures are developed. Consequently, Rankine theory is applicable to the design of MSE walls with extensible reinforcements. Applicability of Rankine theory to the design of MSE walls with extensible, i.e. polymeric, reinforcements has been documented by Bonaparte, et al, (1986) and Christopher, et al, (1990).

The residual stress level is determined as the stress after failure at which strain continues to increase with no significant change in stress.

C1.1.7

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Polymeric materials exhibit creep (time and temperature dependent) behavior. Long-term stress-strain-time behavior of the reinforcement shall be determined from results of controlled laboratory creep tests conducted for a minimum duration of 10,000 hours for a range of load levels on samples of the finished product in accordance with ASTM D 5262. Samples shall be tested in the direction in which the load will be applied in use in either a confined or unconfined mode. Results shall be extrapolated to the required design life using procedures outlined in ASTM D 2837.

From these tests, the following shall be determined:

- The highest load level at which the log time creep-strain rate continues to decrease with time within the required lifetime and no failure either brittle or ductile can occur. This value shall be termed the Limit State Tensile Load, designated as T_ℓ.

- The tension level at which total strain is not expected to exceed 5% within the design lifetime. This value of load shall be designated T_5, the Serviceability State Tensile Load.

The effects of aging, chemical and biological exposure, environmental stress cracking, stress relaxation, hydrolysis and variations in the manufacturing process, as well as the effects of construction damage shall be evaluated and extrapolated to the required design life.

For these evaluations, the specified design life shall be a minimum of 75 years and the assumed in ground temperature shall be 21° C {70° F}. Permanent structures shall have a design life of 100 years.

COMMENTARY

The methodology for determining tensile strength has been adopted from the recommended practice proposed by Task Force 27 of AASHTO-AGC-ARTBA.

Where polymeric reinforcements are expected to permanently resist tensile loads, creep derived tensile strength will govern the determination of useable tensile stresses.

In lieu of full-scale construction damage tests and substantial experimental data regarding the long-term effects of environmental conditions and aging, default resistance factors outlined in C1.1.8 should be used.

1.1.8 Resistance Factors

The following resistance factors shall be evaluated when using extensible polymeric reinforcements. For strength and extreme event load conditions, the limit state tensile load T_ℓ is applicable. For service load conditions, the serviceability tensile load T_5 is applicable.

C1.1.8

The resistance factors for tensile strength of polymeric reinforcements must reflect creep reduction factors as well as reduction factors for aging and environmental losses and construction damage.

Creep derived limiting stresses at the expected service temperatures and projected life of the structure are obtained from laboratory creep tests conducted for minimum durations of 10,000 hours in general accordance with procedures outlined by McGowan (1986) or ASTM D 5262, which are modifications to creep test procedures (ASTM D 2837) commonly used to determine pressure ratings on thermoplastic pipe.

In the absence of such data, NCHRP 290 (Mitchell and Villet, 1987), and the FHWA Geotextile Engineering Manual (Christopher and Holtz, 1985) suggest certain reduction factors to the tensile resistance obtained from ASTM D 4595 (ASTM 1994) for various types of polymers to determine a creep-derived limiting tensile resistance.

Polymer Type	Creep Reduction Coefficients
Polyester	0.40

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Polypropylene	0.20
Polyethylene	0.20

The effects of aging, oxidation, hydrolysis, radiation and chemical effects are considered in a additional resistance factor for each polymer type. At present, very little data is available from the manufacturers to determine quantitatively the effects of these mechanisms for a 75-year minimum design life. Therefore, default resistance factors should be adopted to the creep-derived limit state load, unless substantial experimental data for each specific polymer is developed by the manufacturer and presented. Short-term EPA 9090 test data is not considered sufficient to quantify an appropriate long-term strength reduction factor. For polyoleofin products (PP, HDPE) oxidation strength losses may be determined using oven aging tests in accordance with methods outlined by Wisse (1990). For polyester products, hydrolysis strength losses may be determined in accordance with methods outlined by McMahon (1959). In the absence of product specific data, a resistance factor for all aging effects, FD shall be taken as 2.0. In no case shall FD be less than 1.1.

The construction damage reduced resistance factor shall be determined from the results of full-scale construction damage tests using representative fill materials and construction procedures. In the absence of product-specific construction damage test data, the resistance factor for construction damage, FC, shall be taken as 3.0. For fills specified in Publication 408, this reduction factor can vary from 1.05 to 1.4 for sand fills containing no gravels and from 1.4 to 3.5 for fills containing significant gravel percentages. The lower values are generally associated with geogrids and the higher values with lightweight geotextiles.

Table 1.1.8 – Resistance Factors for Extensible Reinforcement

CONDITION	TENSILE LOAD EVALUATION FACTORS	RESISTANCE FACTORS
Strength, Extreme Events	T_ℓ from laboratory creep tests of 10,000 hours minimum duration FC and FD for each specific product and backfill	$0.8\left[\dfrac{1}{FCxFD}\right]$
Service	T_5 from laboratory creep tests of 10,000 hours minimum duration at a maximum strain of 5% FC and FD for each specific product and backfill	$0.8\left[\dfrac{1}{FCxFD}\right]$

1.1.9 Pullout Design Parameters

SPECIFICATIONS

COMMENTARY

The applicable provisions for extensible polymeric reinforcements in A11.10.6.3.2 and D11.10.6.3.2 shall apply. *(this section will be verified and updated to reflect the new DM 4 Section 11 when it is finalized)* The coefficient f_d, an interaction coefficient, for geogrids shall be determined using GRI Test Method GG-5. It shall be evaluated under both short-term and long-term conditions. In the latter case, the pullout load shall be increased systematically up to 1.5 times the geogrid design load and kept constant for 1000 hours. The pullout load shall then be increased until failure occurs. Only the long-term coefficient shall be used for design. The test shall be performed using site-specific soil. The ultimate pullout load shall be based on a maximum elongation of the embedded geogrid of 20 mm {3/4 in.} as measured at the leading edge of the compressive zone and not the ultimate pullout capacity.

1.1.10 Connection Design

Extensible reinforcement connections to the facing shall be designed to carry the maximum factored load at each reinforcement level. A representative connection shall be load tested in order to determine the allowable connection working load.

Polymeric reinforcement strength is temperature dependent, generally decreasing at higher temperatures. Resistance factors derived under 1.1.8 are for in-ground service assumed at 21° C {70° F}. Connection service temperatures can be considerably higher and, therefore, for creep reduction and aging, relevant higher service temperatures shall be determined.

C1.1.10

Service temperatures at connections have been measured to be nearly equal to ambient maximum temperatures.

1.1.11 Construction Specifications

Construction specifications (Special Provisions) for MSE walls with concrete panels and polymeric reinforcements shall be the same as for Mechanically Stabilized Earth walls, with the following exceptions:

(a) Section I. Description -- Substitute "sheet or grid polymeric reinforcements" for "strip or grid metallic reinforcements".

(b) Section III. Materials -- Eliminate (b) Reinforcement and substitute the following:

(1) Polymeric grid reinforcement -- Provide a polymeric geogrid with a junction strength greater than as specified hereinafter. The summation of the shear strengths of joints occurring in a 300 mm {12 in.} length of grid sample shall be greater than the ultimate tensile strength of the grid element to which they are attached. The ultimate tensile strength shall be determined from wide width tensile tests (ASTM D 4595 (1994)). Junction strength shall be measured in accordance with

SPECIFICATIONS COMMENTARY

GRI Method, G.G. 2-87. The grid shall have been prequalified by the engineer for in-ground reinforcement use, with design values established for long-term resistance factors based on creep, aging and construction damage testing as shown on the plans.

(2) Polymeric sheet reinforcement -- Provide a polymeric sheet reinforcement manufactured from virgin polymers meeting the requirements of Publication 408, Section 735, Class 4.

The sheet reinforcement shall have been prequalified by the engineer for in-ground reinforcement use with design values established for long-term resistance factors based on creep, aging and construction damage testing as shown on the plans.

(3) Fasteners -- Provide 12 mm {1/2 in.} (nominal)-diameter PVC Pipe, Schedule 80, or approved equal for pinned geogrid connection when used.

(c) Modify (e) Granular Fill Materials as follows:

The electrochemical criteria with respect to resistivity, chlorides and sulfates is waived.

(d) Section IV. Construction -- Add the following after the second paragraph of (e) Backfilling.

Place the geogrid or geotextile flat on the compacted soil and attach it to the facing according to the plans. Pretension the reinforcement such that it is free of wrinkles or any slack at the connection with the facing. Maintain the pretensioning until sufficient backfill is placed to keep the reinforcement secure. Cover the reinforcement with a minimum lift of 150 mm {6 in.} before compaction equipment is allowed to operate. Avoid the development of wrinkles and deformations of the reinforcement during the placement and compaction of backfill soils.

REFERENCES

Bonaparte, R., Holts, R. D. and Giroud, J. P., "Soil Reinforcement Design Using Geotextiles and Geogrids", GEOTEXTILE TESTING AND THE DESIGN ENGINEER, ASTM STP 952, J. E. Fluet, Jr., ed., Philadelphia, Pennsylvania, pp. 69-116, 1986

Christopher, B. R., Gill, S. A., Giroud, J., Juran, I., Mitchell, J. K., Schlosser, F. and Dunnicliff, J., "Reinforced Soil Structures, Volume I, Design and Construction Guidelines", FHWA RD-89-043, Federal Highway Administration, U. S. Department of Transportation, U. S. Government Printing Office, Washington, D. C., 301p., 1990

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APPENDIX J - APPROVED COMMERCIALLY AVAILABLE OR CONSULTANT - DEVELOPED SOFTWARE

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1.0 COMPUTER PROGRAM ACCEPTANCE - GIRDER BRIDGES

1.1 REFINED METHODS OF ANALYSIS

Programs approved for use on LFD design projects are listed in Table 1.1-1 (LRFD listed in Table 1.1-2). The approval of these programs is subject to the following conditions and limitations:

1. While certain software packages provide design optimization and/or code compliance checks, these aspects were not included in the review and approval process. Acceptance has been based solely upon the review of generalized design forces (moments, shears, reactions, etc.) as calculated by the software.

2. Acceptance of a software package by the Department does not affect the responsibility of the user for the proper application of the software and interpretation of its results. The acceptance of a software package does not constitute an endorsement nor does it relieve the vendor and the designer from their responsibility for accurate, technically correct, and sound engineering results and services to the Department.

3. The Department's acceptance does not constitute any form of implied warranty, including warranty of merchantability and fitness for a particular purpose. The Commonwealth makes no warranty or representation, either expressed or implied, with respect to this software or accompanying documentation, including their quality performance, merchantability, or fitness for a particular purpose. In addition, the Commonwealth will not be liable for any direct, indirect, special, incidental, or consequential damages arising out of the use, inability to use, or any defect in the software or any accompanying documentation.

Only the version of a program listed in the tables below has been tested and approved. If any changes and/or modifications have been made to a program since its approval date, then re-approval of the program is required.

Table 1.1-1 - The List of Approved Computer Programs for the "3D or Refined" Analysis of Girder Bridges (for LFD design)

CONSULTANT	PROGRAM NAME	DATE SUBMITTED	DATE APPROVED
BSDI Coopersburg, PA	3D System	10/14/88	11/10/88
Modjeski and Masters, Inc. Harrisburg, PA	CURVBRG plus CGIS V4.2	10/17/88	11/10/88
SAI Consulting Engineers, Inc. Pittsburgh, PA	LOAD3D and SUPERSAP	12/12/88	12/27/88
Telos Technologies, Inc. Syracuse, NY	CBRIDGE	5/16/90	6/11/90

Table 1.1-2 - The List of Approved Computer Programs for the "3D or Refined" Analysis of Girder Bridges (for LRFD design)

CONSULTANT	PROGRAM NAME	DATE SUBMITTED	DATE APPROVED
SAI Consulting Engineers, Inc. Pittsburgh, PA	LOAD3DLRFD	2/27/08	11/12/08

2.0 COMPUTER PROGRAM ACCEPTANCE – GENERAL/SPECIFIC ANALYSIS

a). STAAD/Pro 2007 has been tested and approved by the Department for general purpose structural analysis.

However, the following features of STAAD have not yet proven satisfactory to the Department and shall not be used until approved by the Chief Bridge Engineer:

1. The moving load generator.
2. The response spectrum and forced vibration analysis capabilities.
3. All design code check capabilities.
4. Analyses of live loads on 3-dimensional structural models.
5. Seismic design and analyses.

b). VBent V2.3.0 has been tested and approved by the Department for Pier analysis (in addition to PAPIER).

3.0 COMPUTER PROGRAM ACCEPTANCE - SEISMIC ANALYSIS

SEISAB V4.2, having been tested and approved by the Department, shall be used for seismic design and analysis of all girder bridges.

4.0 COMPUTER PROGRAM ACCEPTANCE - GEOTECHNICAL

The following programs have been tested and approved for use on department projects:

1. GRLWEAP 2003 - Pile hammer analysis (refer to section D10.7.3.8.4a)
2. Driven V1.2 - Ultimate Vertical Static Pile Capacity (refer to FHWA Report SA-98-074 and D10.7.3.8.6)
3. GSTABL7 with STEDwin - Slope stability analysis (refer to Pub 293 and D10.6.2.5)
4. COM624P and/or LPILE 5.0 (refer to D10.7.3.12.2)

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APPENDIX K - APPROVED PREFABRICATED RETAINING WALLS

The following companies produce PennDOT approved prefabricated structural retaining walls:

The Reinforced Earth Company
8614 Westwood Center Drive
Suite 1100
Vienna, VA 22182

Prefabricated T-Wall Retaining Wall System
The Neel Company
8328-D Traford Lane
Springfield, VA 22152

Everwall Abutment and Retaining Wall Systems
Schuylkill Products Inc.
121River Street
Cressona, PA 17929-1133

Dura-Hold Dryeast Retaining Wall*
Dura-Sales
2481 Bull Creek Road
Tarentum, Pennsylvania 15084

Tricon Retained Soil Wall Panels
Tricon Precast, Ltd.
15055 Henry Road
Houston, Texas 77060

*This is approved on a provisional basis. The associated special provision is a Series P special provision indicating that the product is not permanently approved for use in Pennsylvania and must be monitored for at least five years. Refer to Publication 408 for more information.

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APPENDIX L - SOIL-CORRUGATED METAL STRUCTURE INTERACTION SYSTEMS
SOIL-CULVERT INTERACTION (SCI) DESIGN PROCEDURE

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1.0 GENERAL

1.1P SCOPE

The Soil-Culvert Interaction (SCI) Design Procedure[1] includes provisions for the design of flexible corrugated metal structure installations with concrete relieving slabs and/or tension struts. The SCI procedure may only be used with the prior approval of the Chief Bridge Engineer.

1.2P ASSUMPTIONS

The SCI design procedure is based on the following important assumptions which shall be addressed by the designer:

(a) The structure shall be backfilled symmetrically.

(b) No significant live loads shall be applied before the backfill reaches the crown of the structure.

(c) No separate check of the buckling resistance is required if the structure is designed to meet all other requirements of the SCI procedure. Tests indicate that the buckling criterion will be satisfied provided the backfill is at least of the minimum quality provided by an ML or CL soil, in accordance with the USCS, compacted in lifts to 90% of the maximum dry unit weight as specified in ASTM D 1557 or AASHTO T 180.

(d) The structural backfill shall extend for a minimum distance of 900 mm {3 ft.}or two-thirds of the span (2/3 S) on either side of the structure, whichever is greater. The structural backfill shall extend to the surface of the fill above the structure, or to a depth of cover of 2/3 S, whichever is less.

The culvert section shall be designed for the maximum ring compression force, P_{max}, and the selected section shall be checked for structural adequacy against the following failure modes:

(a) Seam strength

(b) Handling stiffness (factory assembled pipe)

(c) Moment capacity

1.3P NOTATION

The following notation shall apply for the design of buried metal structures by the SCI procedure:

AL = Axle load (N) {kips}

B = Footing width (mm) {ft.}

C = Vertical distance from invert to crown (mm) {ft.}

C_1 = Bearing constant dependent on soil type and strength (MPa) {ksf} (Table 2.1.9-1)

C_2 = Bearing constant (MPa/mm) {ksf/ft} (Table 2.1.9P-1)

C_3 = Bearing constant (MPa) {ksf} (Table 2.1.9P-1)

C_4 = Bearing constant for soil beneath footing (MPa) {ksf} (Table 2.1.10P-1)

C_5 = Bearing constant (MPa/mm) {kcf} (Table 2.1.10P-1)

C_6 = Bearing constant (MPa/mm) {kcf} (Table 2.1.10P-1)

D_f = Footing depth (mm) {ft.}

D_s = Depth at inside of footing (mm) {ft.}

E = Young's modulus of the culvert (MPa) {ksf}

E_s = Secant modulus of the soil backfill (MPa) {ksf} (Table 2.1.4P-1)

F_p = Factor of safety

F_{pmin} = 1.65 for the fill at the crown and no live load

F_s = Factor of safety against seam or wall compression failure

g = Acceleration of gravity = 9.81 m/s^2 (N/kg) {32.2 ft/sec^2}

H = Depth of cover (mm) {ft.}

H_{RS} = Depth of cover measured to top of relieving slab (mm) {ft.}

I = Moment of inertia of the culvert (mm^4/mm) {in^4/in} (See Appendix to A12 and D12)

K_4 = Line load factor (mm) {ft.} (Table 2.1.1P-1)

K_{T1} = Moment coefficient for culvert with struts

K_{m1} = Moment coefficient with zero cover (2.1.5P)

K_{m2} = Moment coefficient due to cover (2.1.5P)

K_{m3} = Moment coefficient due to live load (2.1.5P)

K_{m4} = Moment coefficient due to depth of soil cover and anticipated live load, or moment coefficient due to live load acting on concrete relieving slab (2.2.4P)

K_{mm} = Minimum moment coefficient based on experience with factory-fabricated pipe equal to 1.33 N/mm {0.091 kip/ft}

K_{p1}, K_{p2}
K_{p3} = Ring compression force coefficients

$K_{p\ min}$ = Minimum acceptable plastic moment capacity for factory assembled pipe (N·mm/mm) {k·ft/ft}

LL = Line load (AL/K_4) or line load equivalents for vehicles supported by concrete slab (N/mm) {kip/ft} (Table 2.2.4P-1)

M_1 = Maximum bending moment for
H = 0 (N·mm/mm) {k·ft/ft} (2.1.6P)

M_{1T} = Maximum moment for H = 0 (occurs at quarter-point and crown) with tension struts (N·mm/mm) {k·ft/ft} (2.3.2P)

M_B = Maximum moment at quarter-point with final cover (N·mm/mm) {k·ft/ft} (2.1.6P)

M_{B+L} = Maximum moment at quarter-point with live load over quarter-point (N·mm/mm) {k·ft/ft) (2.1.6P) (Positive sign indicates compression for extreme fiber.)

M_{BT} = Maximum quarter-point moment at final cover with tension struts (N·mm/mm) {k·ft/ft} (2.3.5P)

M_S = Quarter-point bending moment with slab in place (N·mm/mm) {k·ft/ft}

M_{S+L} = Quarter-point moment with live load on slab (N·mm/mm) {k·ft/ft}

M_p = Plastic moment capacity (N·mm/mm) {k·ft/ft} (Table 2.1.3P(A))

$M_{p\ min}$ = Minimum acceptable plastic moment for factory fabricated pipe (N·mm/mm) {k·ft/ft}

N_f = Flexibility number for zero cover

P_1 = Quarter-point ring compression force for H = 0 (N/mm) {kip/ft}

P_{1T} = Quarter-point ring compression force for H = 0 with tension struts (N/mm) {kip/ft}

P_2 = Quarter-point ring compression force with cover depth equal to H (N/mm) {kip/ft}

P_H = Horizontal load on footing (N/mm) {kip/ft}

P_{S+L} = Quarter-point ring compression force with live load on slab (N/mm) {kip/ft}

P_V = Vertical load on footing (N/mm) {kip/ft}

P_{all} = Allowable soil bearing pressure beneath footing (MPa) {ksf}

P_{max} = Maximum ring compression force (N/mm) {kip/ft}

P_p = Plastic load capacity (N/mm) {kip/ft}

P_{ult} = Seam strength or wall strength (N/mm) {kip/ft}

p = Vertical bearing pressure (MPa) {ksf}

q = Horizontal bearing pressure (MPa) {ksf}

R = Top rise or vertical distance from haunch level to crown of structure (mm) {ft.} (Figure 2.1.1P-1)

R_B = Reduction factor for moment due to backfill (2.1.5P)

R_L = Reduction factor for moment due to live load (2.1.5P)

R/S = Rise span ratio

r_a = Radius of adjacent section above or below corner (mm) {ft.} (k = 0.8 for r_a below corner; k = 0.9 for r_a above corner)

r_c = Corner radius (mm) {ft.}

S = Span (mm) {ft.} (Figure 2.1.1P-1)

S_u = Undrained shear strength (MPa) {ksf}

T = Maximum tension strut force (N/mm) {kip/ft}

t = Slab thickness (mm) {ft.}

α = Angle between culvert wall and vertical at footing (deg) (Figure 2.1.10P-1)

γ = Unit mass of backfill (kg/m^3) {kcf}

γ_s = Unit mass of relieving slab (kg/m^3) {kcf} (assumed to be 2400 kg/m^3 {0.150 kcf})

Δq = Difference between soil bearing at the corner and adjacent to the corner (MPa) {ksf}

Δq_{all} = Allowable difference in soil bearing pressure (MPa) {ksf}

The dimensional units provided with each notation are presented for illustration only, to demonstrate a dimensionally correct combination of units for the design of flexible corrugated metal buried structures using the SCI procedure. If other units are used, the dimensional correctness of the equations should be confirmed. Base dimensions for the SCI procedure are presented in terms of newton {kips} and millimeter {feet} units. Service load and load factor procedures are presented in terms of newton {pound} and millimeter {inch} units.

2.0 DESIGN

2.1P CORRUGATED METAL CULVERTS

Corrugated metal culverts constructed without relieving slabs and tension struts shall be designed by the SCI procedure using the following steps.

2.1.1P Maximum Ring Compression Force

The maximum ring compression force shall be determined using the following relationship:

Metric Units: $P_{max} = \left(K_{p1}\gamma\, gS^2 + K_{p2}\gamma\, gHS\right)\left(10^{-9}\, m^3 / mm^3\right) + K_{p3}LL$

U.S. Customary Units: $P_{max} = K_{p1}\gamma\, S^2 + K_{p2}\gamma\, HS + K_{p3}LL$

where:

K_{pl} = 0.2 (R/S)

K_{p2} = 0.9 – 0.5 (R/S)

K_{p3} = 1.25 – (H/S) for 0.25 < H/S < 0.75)
= 1.00 for H/S ≤ 0.25
= 0.50 for H/S ≥ 0.75

For tandem axles spaced at less than one-third the span of the culvert, the value of AL used to calculate the equivalent line load shall equal the sum of the loads carried on both axles. Table 1 shows value of K_4. For definitions of R and S for various culvert types, see Figure 1.

Table 2.1.1P-1 – Line Load Factors, K_4

Metric Units				
Depth (mm)	No Relieving Slab			Relieving Slab Over Crown (mm)
	2 Wheels per axle (mm)	4 Wheels per axle (mm)	8 Wheels per axle (mm)	
300	1310	1520	2590	3900
600	1620	1950	2800	4080
900	2410	2650	3230	4270
1500	3750	3810	4110	4450
2100	4390	4420	4450	4660
3000	4880	4880	4880	4880
4500	6710	6710	6710	---
6000	8530	8530	8530	---
9000	12 190	12 190	12 190	---

U.S. Customary Units				
Depth (ft.)	No Relieving Slab			Relieving Slab Over Crown (ft.)
	2 Wheels per axle (ft.)	4 Wheels per axle (ft.)	8 Wheels per axle (ft.)	
1	4.3	5.0	8.5	12.8
2	5.3	6.4	9.2	13.4
3	7.9	8.7	10.6	14.0
5	12.3	12.5	13.5	14.6
7	14.4	14.5	14.6	15.3
10	16.0	16.0	16.0	16.0
15	22.0	22.0	22.0	---
20	28.0	28.0	28.0	---
30	40.0	40.0	40.0	---

Source: Reference 1

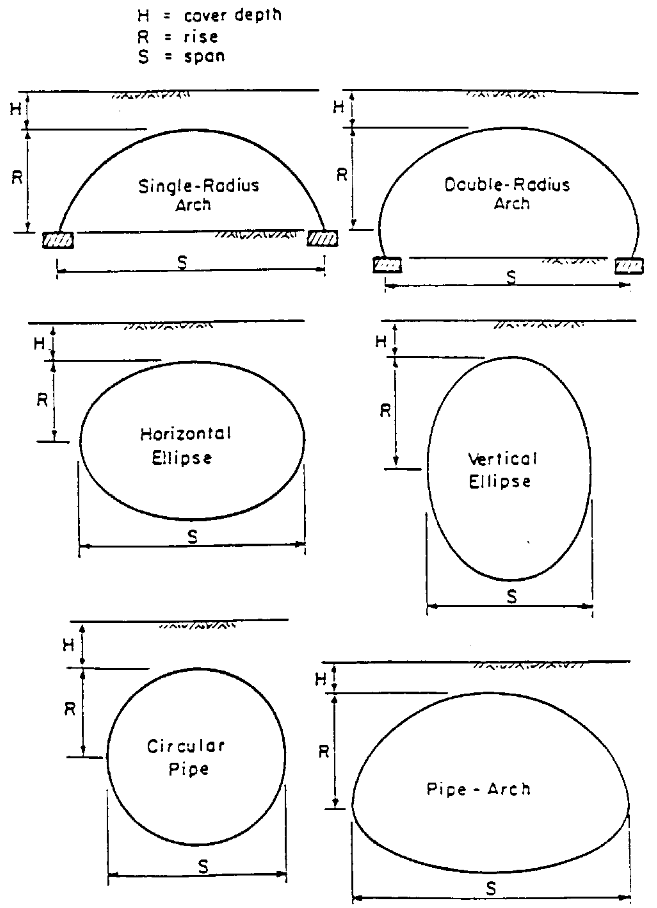


Figure 2.1.1P-1 - Typical Culvert Shapes

2.1.2P Seam Strength

The seam strength of the section shall be checked using the following relationship:

$$F_s = \frac{P_{ult}}{P_{\max}} > 2.0$$

See AASHTO Standard Specification for Highway Bridges, Table 12.4.2, for values of P_{ult}. (Please note that values from AASHTO Standard Specification for Highway Bridges are in U.S. Customary Units.) For the computation of P_{max}, see 2.1.1P.

2.1.3P Handling Stiffness (Plastic Moment Capacity)

The culvert section shall be checked for handling and installation rigidity using the following relationship to calculate the minimum plastic moment capacity:

Metric Units: $M_{p\,\min} = K_{mm}\, S\left(10^{-3}\, kN/N\right)$

U.S. Customary Units: $M_{p\,\min} = K_{mm} S$

For values of M_p for corrugated aluminum and steel plate, see Table 1 and the value of K_{mm} = 1.33 N/mm {K_{mm} =0.091 kip/ft} based on experience for factory fabricated pipes.

Table 2.1.3P-1 - Properties of Corrugated Aluminum and Steel Plate

Metric Units				
Plate Corrugation	Uncoated Thickness (mm)	Moment of Inertia mm^4/mm	Plastic Moment Capacity, Mp (kN mm/mm)	Plastic Load Capacity, Pp (kN/mm)
Corrugated Aluminum: 67.7 mm x 12.7 mm	1.52	31.1	1.02	0.271
	1.91	39.6	1.29	0.339
	2.67	56.6	1.87	0.475
	3.43	73.6	2.40	0.611
	4.17	93.4	2.98	0.747
76.2 mm x 25.4 mm	1.52	141.6	2.40	0.312
	1.91	178.4	2.98	0.390
	2.67	252.0	4.23	0.546
	3.43	331.3	5.52	0.704
	4.17	410.6	6.76	0.861
152.4 mm x 25.4 mm	1.52	138.8	2.27	0.271
	1.91	175.6	2.85	0.339
	2.67	243.5	3.96	0.475
	3.43	317.1	5.12	0.611
	4.17	390.8	6.27	0.747
Corrugated Steel: 67.7 mm x 12.7 mm	1.52	31.1	1.73	0.452
	1.91	39.6	2.18	0.565
	2.67	56.6	3.07	0.792
	3.43	73.6	4.00	1.019
	4.17	93.4	4.94	1.245
76.2 mm x 25.4 mm	1.52	141.6	4.00	0.520
	1.91	178.4	4.98	0.649
	2.67	252.0	7.07	0.911
	3.43	331.3	9.16	1.173
	4.17	410.6	11.30	1.435

Source: Reference 1

Table 2.1.3P-1 - Properties of Corrugated Aluminum and Steel Plate (continued)

U.S. Customary Units				
Plate Corrugation	Uncoated Thickness (in.)	Moment of Inertia 1×10^{-4}(ft^4/ft)	Plastic Moment Capacity, M_p (k-ft/ft)	Plastic Load Capacity, P_p (k/ft)
Corrugated Aluminum: 2 2/3 in. x 1/2 in.	0.060	0.011	0.23	18.59
	0.075	0.014	0.29	23.22
	0.105	0.020	0.42	32.56
	0.135	0.026	0.54	41.85
	0.164	0.033	0.67	51.18
3 in. x 1 in.	0.060	0.050	0.54	21.36
	0.075	0.063	0.67	26.71
	0.105	0.089	0.95	37.43
	0.135	0.117	1.24	48.21
	0.164	0.145	1.52	58.99
6 in. x 1 in.	0.060	0.049	0.51	18.59
	0.075	0.062	0.64	23.22
	0.105	0.086	0.89	32.56
	0.135	0.112	1.15	41.85
	0.164	0.138	1.41	51.18
Corrugated Steel: 2 2/3 in. x 1/2 in.	0.060	0.011	0.39	31.0
	0.075	0.014	0.49	38.7
	0.105	0.020	0.69	54.3
	0.135	0.026	0.90	69.8
	0.164	0.033	1.11	85.3
3 in. x 1 in.	0.060	0.050	0.90	35.6
	0.075	0.063	1.12	44.5
	0.105	0.089	1.59	62.4
	0.135	0.117	2.06	80.4
	0.164	0.145	2.54	98.3

Source: Reference 1

2.1.4P Flexibility Number

The flexibility number accounts for the effects of soil-structure interaction. It shall be determined using the following relationship:

$$N_f = \frac{E_s S^3}{EI}$$

See Table 1 for values of E_s. Use values of E equal to 70 000 MPa {1,468,800 ksf} for aluminum and 200 000 MPa {4,176,000 ksf} for steel. For section properties of steel and aluminum conduit, see AASHTO Standard Specifications for Highway Bridges, Articles 12.4.3.1 and 12.4.3.2, respectively. (Please note that values from AASHTO Standard Specifications for Highway Bridges are in U. S. Customary Units.)

Using the equation above, calculate N_f for each cover depth and live load condition which may result in peak stress conditions. It should be noted than N_f varies with the value of E_s. Values of E_s are presented in Table 1. Load conditions, both during and after construction, shall be considered for each combination of cover depth and live load that may control the design.

Table 2.1.4P-1 - Secant Modulus of Backfill, E_s (MPa) {ksf}

Metric Units													
Backfill Material *	R/C ** (%)	Depth of Cover Over Quarter-Point, (H+R)/2 (mm)***											
		600	1200	1800	2400	3000	3600	4200	4800	5400	6000	6600	7200
ML & CL	90	1.48	1.58	1.58	1.58	1.58	1.58	1.58	1.58	1.58	1.58	1.58	1.58
	95	2.16	2.49	2.54	2.54	2.54	2.54	2.54	2.54	2.54	2.54	2.54	2.54
	100	2.87	3.40	3.54	3.59	3.59	3.59	3.59	3.59	3.59	3.59	3.59	3.59
SM-SC	90	1.68	2.16	2.40	2.54	2.59	2.63	2.68	2.68	2.73	2.73	2.73	2.73
	95	2.40	3.16	3.54	3.83	3.98	4.17	4.22	4.22	4.22	4.26	4.26	4.31
	100	4.98	6.75	7.76	8.38	8.81	9.10	9.29	9.29	9.53	9.63	9.68	9.72
GW, GP & SW, SP	90	1.20	1.39	1.53	1.58	1.68	1.72	1.77	1.77	1.82	1.87	1.87	1.87
	95	3.21	3.78	4.12	4.36	4.50	4.69	4.79	4.89	4.98	5.03	5.13	5.17
	100	6.75	8.05	8.81	9.39	9.82	10.15	10.44	10.68	10.92	11.11	11.26	11.40

U.S. Customary Units													
Backfill Material*	R/C ** (%)	Depth of Cover Over Quarter-Point, (H+R)/2 (ft)***											
		2	4	6	8	10	12	14	16	18	20	22	24
ML & CL	90	31	33	33	33	33	33	33	33	33	33	33	33
	95	45	52	53	53	53	53	53	53	53	53	53	53
	100	60	71	74	75	75	75	75	75	75	75	75	75
SM-SC	90	35	45	50	53	54	55	56	56	57	57	57	57
	95	50	66	74	80	83	87	88	88	88	89	89	90
	100	104	141	162	175	184	190	194	197	199	201	202	203
GW, GP & SW, SP	90	25	29	32	33	35	36	37	37	38	39	39	39
	95	67	79	86	91	94	98	100	102	104	105	107	108
	100	141	168	184	196	205	212	218	223	228	232	235	238

*Unified Soil Classification System per ASTM
**R/C = Percent of the maximum dry unit weight in accordance with ASTM D 1557 or AASHTO T 180
***R = Rise of corrugated buried structure (mm) {ft.}

Value of H_{RS} is substituted for value of H when concrete relieving slab is used.

Source: Reference 1

2.1.5P Moment Coefficients

Moment coefficients are calculated for each appropriate loading condition. For the calculated value of N_f for the case of zero cover over the culvert, the moment coefficients K_{m1} and R_B shall be determined using the following relationships:

K_{m1} = $0.0046 - 0.0010 \log_{10} N_f$ for $N_f < 5000$
= 0.0009 for $N_f \geq 5000$

R_B = $0.67+0.87 ((R/S)-0.2)$ for $0.2 \leq R/S < 0.35$
= $0.80+1.33 ((R/S)-0.35)$ for $0.35 \leq R/S < 0.50$
= $2 R/S$ for $0.50 \leq R/S \leq 0.60$

For each of the anticipated loading conditions other than H = 0, the moment coefficients shall be determined using the following relationships:

K_{m2} = 0.018 - 0.004 $\log_{10} N_f$ for $N_f < 5000$
= 0.0032 for $N_f \geq 5000$

K_{m3} = 0.120 - 0.018 $\log_{10} N_f$ for $N_f < 100\,000$
= 0.03 for $N_f \geq 100\,000$

$$R_L = \frac{0.265 - 0.053 \log_{10} N_f}{(H/S)^{0.75}} \leq 1.0$$

2.1.6P Maximum Moment

The maximum moment will occur at both the quarter- point and the crown of the structure for the condition of zero cover and no live load. The maximum moment shall be determined using the following relationship:

Metric Units: $M_1 = K_{m1} R_B \gamma\, g S^3 \left(10^{-9} m^3 / mm^3\right)$

U.S. Customary Units: $M_1 = K_{m1} R_B \gamma\, S^3$

The maximum moment for conditions of fill over the crown with live load occurs at the quarter-point. For each loading condition, determine the quarter-point moment using the following relationship:

$$M_{B+L} = M_B + K_{m3} R_L SLL$$

where:

Metric Units: $M_B = M_1 - K_{m2} R_B \gamma\, g S^2 H \left(10^{-9} m^3 / mm^3\right)$

U.S. Customary Units: $M_B = M_1 - K_{m2} R_B \gamma\, S^2 H$

For normal traffic loads up to H20 (i.e., axle loads to 145 kN{32 kips}), moments induced by live loads need only be considered where the cover depth is less than 20% of the span (i.e., H/S less than 0.20). Moments induced by axle loads up to 145 {32 kips} kN can be ignored for greater cover depths. For traffic or construction axle loads exceeding 145 kN {32 kips}, moments induced by live loads shall be considered for all heights of cover.

In no case shall any buried structure be designed with a depth of cover over the crown of less than 300 mm {1 ft.}.

2.1.7P Quarter-Point Ring Compression Force

The quarter-point ring compression force for the condition of zero cover shall be determined using the following relationship:

Metric Units: $P_1 = 0.95 K_{p1} \gamma\, g S^2 \left(10^{-9} m^3 / mm^3\right)$

U.S. Customary Units: $P_1 = 0.95 K_{p1} \gamma\, S^2$

The ring compression force for each additional anticipated loading condition shall be determined using the following relationship:

Metric Units: $P_2 = P_1 + 0.95\left(K_{p2}\gamma\, gHS\left(10^{-9} m^3 / mm^3\right) + K_{p3}LL \right)$

U.S. Customary Units: $P_2 = P_1 + 0.95\left(K_{p2}\gamma\, HS + K_{p3}LL\right)$

2.1.8P Factor of Safety Against Development of Plastic Hinge

The development of a plastic hinge corresponds to mobilization of the yield stress throughout the section. For each loading condition including H = 0, the factor of safety against development of a plastic hinge shall be computed using the following relationship:

$$F_P = \left(0.5\frac{P_p}{P}\right)\left(\sqrt{\left(\frac{M}{M_p}\right)^2\left(\frac{P_p}{P}\right)^2 + 4} - \left(\frac{M}{M_p}\right)\left(\frac{P_p}{P}\right)\right)$$

The minimum factor of safety, $F_{p\,min}$, against development of a plastic hinge is 1.65.

As an alternative to solving the above equation, the value of F_p can be determined from Figure 1 using $M = M_1$ and $P = P_1$. When H is greater than 0, F_p can be determined using the above equation or Figure 1 with $M = M_{B+L}$ and $P = P_2$.

See Table 2.1.3P-1 for values of M_p and P_p for corrugated aluminum and steel pipe.

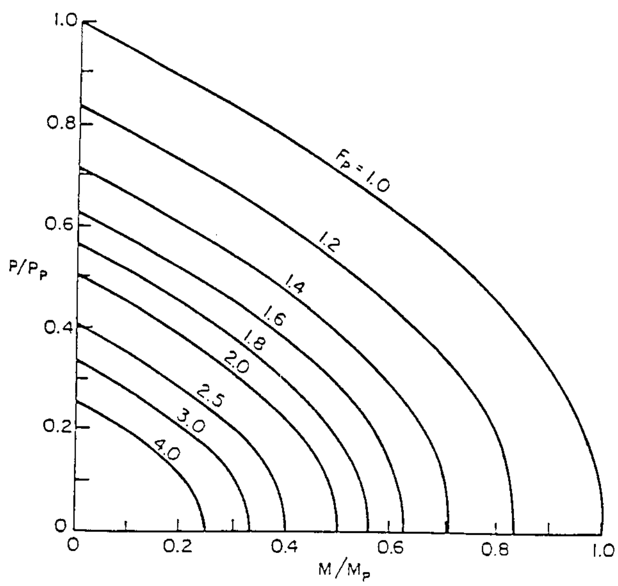


Figure 2.1.8P-1 - Factor of Safety Against Development of a Plastic Hinge

2.1.9P Difference in Soil Bearing Pressures (Pipe Arches Only)

$$\Delta q = P_{\max}\left[\left(\frac{1}{r_c}\right) - \left(\frac{1}{r_a}\right)\right]k$$

The value of Δ q calculated using the above equation shall not exceed the value of q_{all}, as determined by the following relationships:

$$q_{all} < C_1 + C_2\left(H + R\right)$$

$$q_{all} < C_3$$

See Table 1 for values of the bearing constants C_1, C_2 and C_3.

Table 2.1.9P-1 - Lateral Bearing Constants, Pipe Arches and Arch Footings

Metric Units				
Saturated Plastic Silts and Clays, Short-Term Undrained Conditions				
Soil Consistency	Undrained Shear Strength (MPa)	Bearing Constants		
		C_1 (MPa)	C_2 (MPa/mm x 10^{-4})	C_3 (MPa)
Soft	0.019	0.048	---	0.048
Medium	0.038	0.096	---	0.096
Stiff	0.077	0.192	---	0.192
Very Stiff	0.153	0.383	---	0.383

U.S. Customary Units				
Saturated Plastic Silts and Clays, Short-Term Undrained Conditions				
Soil Consistency	Undrained Shear Strength (ksf)	Bearing Constants		
		C_1 (ksf)	C_2 (ksf/ft)	C_3 (ksf)
Soft	0.4	1.0	---	1.0
Medium	0.8	2.0	---	2.0
Stiff	1.6	4.0	---	4.0
Very Stiff	3.2	8.0	---	8.0

Source: Reference 1

Table 2.1.9P-1 - Lateral Bearing Constants, Pipe Arches and Arch Footings (continued)

Metric Units				
Compact Soils				
Soil Type*	Relative Compaction (%)	Bearing Constants		
		C_1 (MPa)	C_2 (MPa/mm x 10^{-4})	C_3 (MPa)
SW, SP, GW, GP	90 95	0.048 0.072	0.911 1.21	1.44 1.92
SM, SC, GM, GC	90 95	0.105 0.115	0.717 0.710	1.20 1.20
ML, CL, MH, CH	90 95	0.086 0.096	0.572 0.629	0.95 1.20

U.S. Customary Units				
Compact Soils				
Soil Type*	Relative Compaction (%)	Bearing Constants		
		C_1 (ksf)	C_2 (ksf/ft)	C_3 (ksf)
SW, SP, GW, GP	90 95	1.0 1.5	0.580 0.770	30 40
SM, SC, GM, GC	90 95	2.2 2.4	0.456 0.452	25 25
ML, CL, MH, CH	90 95	1.8 2.0	0.364 0.400	20 25

*Unified Soil Classification System per ASTM

Source: Reference 1

2.1.10P Footing Bearing Pressure

Due to the behavior and nature of the connection between the corrugated metal plates and the footing, both lateral and horizontal stresses may be transferred to the soil backfill and subgrade (Figure 1). The following relationships shall be used to determine the horizontal and vertical bearing pressures imposed by the culvert:

Metric Units:

$$q = \left(\frac{P_H}{D_f}\right) - 0.5\gamma\, g\left(H + R\right)\left(10^{-9}\, m^3 / mm^3\right)$$

$$p = \frac{P_v}{B}$$

where:

P_H = $[0.5\ \gamma\ g\ HS\ (10^{-9}\ m^3/mm^3) + (AL/1.6\ H)] \sin \alpha$

P_v = $[0.5\ \gamma\ g\ HS\ (10^{-9}\ m^3/mm^3) + (AL/1.6\ H)] \cos \alpha$

U.S. Customary Units:

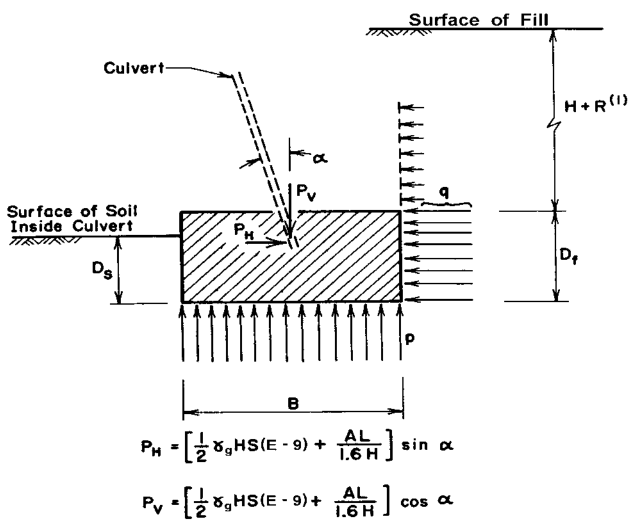
$$q = \left(\frac{P_H}{D_f}\right) - 0.5\gamma\left(H + R\right)$$

$$p = \frac{P_v}{B}$$

where:

P_H = $[0.5\gamma HS + (AL/1.6H)] \sin \alpha$

P_v = $[0.5\gamma HS + (AL/1.6H)] \cos \alpha$



(1) H + R + Height from foundations to springline for low- and high-profile arches

Figure 2.1.10P-1 - Loads and Pressures on Arch Footings

The value of q calculated above shall not exceed the allowable value for the type of soil adjacent to the footing as calculated according to 2.1.9P.

The value of p calculated above shall not exceed the allowable value calculated using the following relationship:

$$p_{all} = C_4 + C_5 B + C_6 D_s$$

See Table 1 for bearing constants C_4, C_5, and C_6. For definitions of P_H, P_v, H + R, D_f, B, D_s and α, see Figure 1.

Table 2.1.10P-1 - Allowable Vertical Bearing Constants Arch Footings

Metric Units					
Naturally Occurring Soil					
Soil Type*	Relative Density	Blow Count N (Blow/300 mm)	Bearing Constants		
			C_4 (MPa)	C_5 (MPa/mm x 10^{-4})	C_6 (MPa/mm x 10^{-4})
SW, SP, GW, CP	Loose Medium Dense	5 12 30	--- --- ---	0.314 0.786 1.89	0.786 1.26 3.14
SM, SC, GM, GC	Loose Medium Dense	5 12 30	0.096 0.144 0.192	0.314 0.786 0.786	0.786 1.26 1.26
ML, CL, MH, CH Saturated	---	---	$P_{all} = 2S_u + \frac{D_s}{62630}$		
ML, CL, MH, CH Unsaturated	--- --- ---	5 12 30	0.048 0.096 0.144	0.314 0.314 0.314	0.786 0.786 0.786

U.S. Customary Units					
Naturally Occurring Soil					
Soil Type*	Relative Density	Blow Count N (Blow/ft)	Bearing Constants		
			C_4 (ksf)	C_5 (ksf/ft)	C_6 (ksf/ft)
SW, SP, GW, CP	Loose Medium Dense	5 12 30	- - -	0.2 0.5 1.2	0.5 0.8 2.0
SM, SC, GM, GC	Loose Medium Dense	5 12 30	2.0 3.0 4.0	0.2 0.5 0.5	0.5 0.8 0.8
ML, CL, MH, CH Saturated	---	---	$P_{all} = 2S_u + \frac{D_s}{10}$		
ML, CL, MH, CH Unsaturated	--- --- ---	5 12 30	1.0 2.0 3.0	0.2 0.2 0.2	0.5 0.5 0.5

Unified Soil Classification System

Source: Reference 1

Table 2.1.10P-1 - Allowable Vertical Bearing Constants Arch Footings (continued)

Metric Units				
Compact Soils				
Soil Type*	Relative Compaction (%)	Bearing Constants		
		C_4 (MPa)	C_5 (MPa/mm x 10^{-4})	C_6 (MPa/mm x 10^{-4})
SW, SP, GW, GP	90 95	--- ---	0.786 1.89	1.26 3.14
SM, SC, GM, GC	90 95	0.144 0.192	0.786 0.786	1.26 1.26
ML, CL, MH, CH	90 95	0.096 0.144	0.314 0.314	0.786 0.786

U.S. Customary Units				
Compact Soils				
Soil Type*	Relative Compaction (%)	Bearing Constants		
		C_4 (ksf)	C_5 (ksf/ft)	C_6 (ksf/ft)
SW, SP, GW, GP	90 95	--- ---	0.5 1.2	0.8 2.0
SM, SC, GM, GC	90 95	3.0 4.0	0.5 0.5	0.8 0.8
ML, CL, MH, CH	90 95	2.0 3.0	0.2 0.2	0.5 0.5

*Unified Soil Classification System per ASTM

Source: Reference 1

2.2P CORRUGATED METAL CULVERTS WITH CONCRETE RELIEVING SLABS

Portland cement concrete relieving slabs (rigid pavements), as illustrated in Figure 1, may be constructed over the top of corrugated metal culverts to reduce the moments induced in the structure by live loads. The design procedures described herein are a modified form of the SCI procedures presented in 2.1P to account for the effects of relieving slabs[1]. The relieving slab shall extend a minimum of 1500 mm {5 ft.} beyond the haunch of the culverts on each side, and shall cover all areas over the top of the culvert where traffic may travel, including trafficable shoulders.

The adequacy of the culvert design during construction (i.e., backfill up to the crown level and no live loading) shall be determined in accordance with the SCI procedures presented in 2.1P. The effects of live loads on the structure before the relieving slab is constructed shall be determined in accordance with the SCI procedure. The effects of live loads on the structure after the relieving slab is constructed shall be determined as presented below.

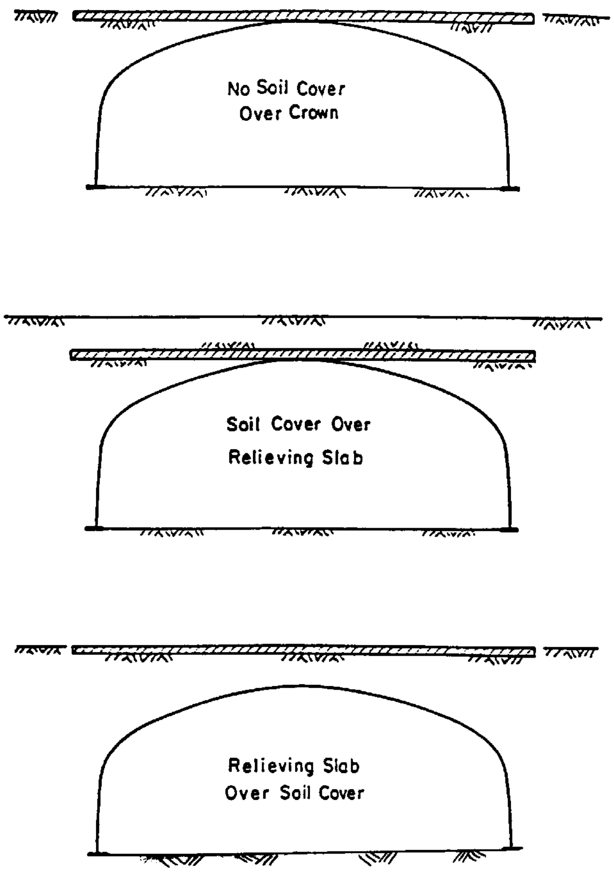


Figure 2.2P-1 - Metal Box Culverts with Cement Concrete Relieving Slabs

2.2.1P Flexibility Number

The flexibility number, N_f, shall be determined according to 2.1.4P. However, the depth of soil cover used to determine the secant modulus, E_s, of the soil backfill in Table 2.1.4P-1 shall be based on depth of soil cover, H_{RS}, measured from the top of the relieving slab.

2.2.2P Moment Coefficients

The moment coefficient for the condition of zero cover shall be determined using the following relationships:

K_{m2} = 0.018 - 0.004 $\log_{10} N_f$ for $N_f < 5000$
= 0.0032 for $N_f \geq 5000$

The moment coefficient for live loading shall be determined using one of the following relationships:

K_{m4} = 0.024 - 0.005 $\log_{10} N_f$ for $N_f < 25\,000$
= 0.002 for $N_f \geq 25\,000$

2.2.3P Bending Moment

The bending moment for the condition of zero soil cover shall be determined using the following relationship:

Metric Units: $M_s = M_1 - R_B K_{m2} S^2 \left[\gamma_s g t + \gamma_g \left(H - t\right)\right]\left(10^{-9} m^3 / mm^3\right)$

U.S. Customary Units: $M_s = M_1 - R_B K_{m2} S^2 \left[\gamma_s t + \gamma\left(H - t\right)\right]$

The moments and moment coefficients shall be determined in accordance with the equations in 2.1.5P and 2.1.6P.

2.2.4P Quarter-Point Moment

The quarter-point moment, M_{S+L}, for the dead and anticipated live load shall be computed using the following relationship:

$$M_{S+L} = M_s + K_{m4} SLL$$

The moment coefficient for live loading on a relieving slab shall be determined using one of the following relationships:

K_{m4} =0.024 - 0.005 $\log_{10} N_f$ for $N_f < 25\ 000$
=0.002 for $N_f \geq 25\ 000$

Table 2.2.4P-1 shows values of LL.

Table 2.2.4-1 - Line Load Equivalents for Vehicles Supported by Concrete Slabs

Metric Units		U.S. Customary Units	
Axle Load of Design Vehicle (kN)*	Line Load, LL (N/mm)**	Axle Load of Design Vehicle (kips)*	Line Load, LL (kip/ft)**
45kN	11	10	0.8
90kN	23	20	1.6
135kN	34	30	2.3
145kN	36	32	2.5
180kN	46	40	3.1
200kN	51	45	3.5

*Total load on single axle or on both axles of a closely spaced tandem pair

**Produces same peak stress beneath slab as design vehicle

Note: For other axle loads, use linear interpolation.

Source: Reference 1

When the slab is constructed directly on top of the culvert, H is equal to the slab thickness, t. If there is a layer of fill above the crown and beneath the relieving slab, H will be greater than t.

2.2.5P Quarter-Point Ring Compression Force

The quarter-point ring compression force shall be determined using the following relationship:

Metric Units: $P_{S+L} = P_1 + 0.95\{K_{p2} S[\gamma_s gt + \gamma g(H-t)](10^{-9} m^3/mm^3) + LL\}$

U.S. Customary Units: $P_{S+L} = P_1 + 0.95\{K_{p2} S[\gamma_s t + \gamma(H-t)] + LL\}$

For the procedures to determine P_1 and K_{p2}, see 2.1.7P and 2.1.1P, respectively.

2.2.6P Factor of Safety Against Development of Plastic Hinge

The factor of safety, F_p, against development of a plastic hinge shall be determined in accordance with 2.1.8P or in accordance with Figure 2.1.8P-1 using $P = P_{S+L}$ (see 2.2.5P) and $M = M_{S+L}$ (see 2.2.4P). The calculated factor of safety against development of a plastic hinge shall satisfy the criterion presented in Figure 2.1.8P-1.

2.3P CORRUGATED METAL CULVERTS WITH TEMPORARY VERTICAL TENSION STRUTS

For cases in which bending moments from soil backfill will result in unacceptable deformation or stresses in the culvert during construction, tension strut may be used between the invert and crown to reduce bending moments and control deformations during backfilling, as illustrated in Figure 1.

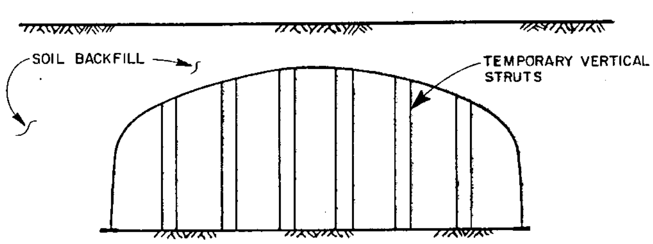


Figure 2.3P-1 - Corrugated Metal Culvert with Temporary Vertical Tension Struts

2.3.1P Maximum Tension Strut Force

The maximum tension strut force may be estimated using the following equation:

Metric Units: $T = K_{T1}\gamma\, gSC\left(10^{-9} m^3 / mm^3\right)$

U.S. Customary Units: $T = K_{T1}\gamma\, SC$

where:

K_{T1} = 0.125 - 0.025 $\log_{10} N_f$ for $N_f < 10\,000$
= 0.025 for $N_f \geq 10\,000$

The value of N_f shall be determined in accordance with 2.1.4P.

Due to the possibility of the progressive failure of all tension rods if any individual rod should fail, it is recommended that the axial stress in the tension rods shall not exceed one-half the yield stress.

The spacing between tension rods shall not exceed 900 mm {3 ft.} to prevent excessive local deformation of the crown and invert.

2.3.2P Bending Moment

The maximum bending moment for soil backfill at the crown shall be determined using the following relationship:

$$M_{1T} = 0.67 M_1$$

See 2.1.6P for the procedure to determine M_1.

2.3.3P Quarter-Point Ring Compression Force

The quarter-point ring compression force shall be determined using the following relationship:

$$P_{1T} = P_1 + 0.5T$$

The values of P_1 and T shall be determined in accordance with 2.1.7P and 2.3.1P, respectively.

2.3.4P Factor of Safety Against Development of Plastic Hinge

The factor of safety, F_p, against development of a plastic hinge shall be determined in accordance with 2.1.8P using P = P_{1T}, M = M_{1T} and $F_p \geq 1.65$.

2.3.5P Moment Due to Backfill

The maximum moment due to the final fill cover with tension struts in place shall be determined using the following relationship:

$$M_{BT} = R_T M_B$$

where:

R_T = 0.67 + 2.2 H/S for H/S < 0.15
= 1.00 for H/S $\geq$ 0.15

The value of M_B shall be determined in accordance with 2.1.6P.

REFERENCES

1. Duncan, J. M., and Drawsky, R. H., Design Procedures for Flexible Metal Culvert Structures, Report No. UCB/GT/83-02, Department of Civil Engineering, University of California at Berkeley, in cooperation with Kaiser Aluminum and Chemical Sales, Inc., 2nd ed., 1983

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 2

APPENDIX M - SOIL-CORRUGATED METAL BOX STRUCTURE INTERACTION SYSTEMS
SOIL-CULVERT INTERACTION (SCI) DESIGN PROCEDURE

APPENDIX M - TABLE OF CONTENTS

SPECIFICATIONS

1.0 GENERAL

Use of SCI method requires prior approval of the Chief Bridge Engineer.

1.1P SCOPE

Corrugated aluminum and steel box structures may be designed in accordance with the following procedures.

1.2P ASSUMPTIONS

See Appendix L, 1.2P.

1.3P NOTATION

In addition to the notation presented in Appendix L for the design of corrugated metal structures by the SCI method, the following notation shall apply for the design of corrugated metal box structures by the SCI procedure:

AL = Maximum single-axle load of design vehicle or, if larger, the maximum load carried by two load-sharing (tandem) axles (N) {kips} (Table 2.1.1P-1)

F_B = Footing load due to dead load (N/mm) {kip/ft}

F_L = Footing load due to live load (N/mm) {kip/ft}

H = Height of cover over box as measured to the inside crown (mm) {ft.} (greater than H_{min})

H_{min} = Minimum cover depth equal to 420 mm {1.4 ft.} for all spans

K = Live load coefficient (1/mm) {1/ft} (Table 2.1.4.1P-1)

K_{1B}, K_{2B}
K_{3B} = Box culvert moment coefficients

K_4 = Line load factor (mm) {ft.} (Appendix L, Table 2.1.1P-1)

M_{CD} = Crown design moment (N·mm/mm) {k·ft/ft}

M_{HD} = Haunch design moment (N·mm/mm) {k·ft/ft}

M_{TB} = Sum of crown and haunch moments due to

COMMENTARY

C1.1P

Corrugated metal box structures are used for applications where large flow areas are required with minimum available heights. The addition of circumferential, or lateral, reinforcing ribs, similar to the stiffening elements discussed in AASHTO Standard Specification for Highway Bridges, Article 12.7.2.2, allows the support of relatively large spans without the full benefit of the soil-structure interaction realized by the lateral restraint the backfill provides to the circular, elliptical and arch shapes.

SPECIFICATIONS

backfill (N·mm/mm) {k·ft/ft} (2.1.2P)

P_B = Crown moment coefficient

R = Live load footing reaction reduction factor (Table 2.1.4.1P-1)

R_{DL} = Dead load footing reaction (N/mm) {kip/ft} (2.1.4.1P)

R_{HB} = Haunch moment coefficient (Figure 2.1.2P-1)

R_{LL} = Live load footing reaction (N/mm) {kip/ft} (2.1.4.1P)

S = Span (mm) {ft}

γ = Unit mass of soil backfill (kg/m^3) {kcf}

ΔM_{TL} = Change of crown and haunch moments due to live load (N·mm/mm) {k·ft/ft} (2.1.2P)

The dimensional units provided with each notation are presented for illustration only, to demonstrate a dimensionally correct combination of units for the design of corrugated metal box structures. If other units are used, the dimensional correctness of the equations shall be confirmed.

2.0 DESIGN

2.1P LOAD FACTORS

The following load factors shall be applied to the dead and live loads:

(a) Dead loads: 1.5

(b) Live loads: 2.0

2.1.1P Live Loads

Vehicle loads are represented by equivalent line loads, LL, which, at the level of the crown of the culvert, produce

COMMENTARY

C2.1P

The load and impact factors included in the design moments are appropriate for these flexible buried structures, which differ from conventional bridge structures in the effects produced by soil-structure interaction. The key values used in designs by this specification are the crown and haunch moments.

The specification utilizes calculated moments from computer analysis of conditions known to produce more conservative moments than the actual live loads and soils. Allowance for impact is incorporated in the values of line load used to develop the live load moments the specification requires. Calculated moments based on these equivalent line loads are longer than actual measured moments caused by moving vehicle loads. This is true, even in cases which the vehicles were made to jump as they passed over the structures by placing 15 mm {5/8 in.} thick wooden planks directly above the structures crowns. The equivalent line loads used in developing the design live load moments in the specification, thus, contain an allowance for impact effects that have been shown to be conservative by comparison with actual field measurements.

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the same peak stress as the wheel or axle loading. The value of LL shall be determined using the following relationship:

$LL = AL/K_4$

For tandem axles spaced at less than one-third the span of the culvert, AL is the sum of the loads carried on both axles. Table 1 shows values of the axial loads for various vehicles. For values of K_4, see Appendix L, Table 2.1.1P-1.

Table 2.1.1P-1 - Values of AL for Various Vehicle Types and Loading Conditions

Vehicle Type	AL	
HS20	142 000 N {32 kips}	
HS25	178 000 N {40 kips}	
15 m^3 {20 yd^3} scraper	Empty: 191 000 N {43 kips}	Fully Loaded 267 000 N {60 kips}

Note: Axle load, AL (kN) {kips}, of loaded wheel tractor scraper may be estimated by multiplying the rated carrying capacity (m^3) {yd^3} by 17 500 for metric units {by 3 for U.S. Customary Units}. Axle load when empty is approximately 70% of the fully-loaded axle load.

Source: Reference 1.

2.1.2P Bending Moments

The sum of the crown and haunch bending moments due to loading from the backfill shall be determined using the following relationship:

Metric Units:

$$M_{TB} = \left(K_{1B}\gamma g S^3 + K_{2B}\gamma g\left(H - H_{\min}\right)S^2\right)\left(10^{-9}\, m^3/mm^3\right)$$

where:

K_{1B} = 0.0053-0.787 (10^{-6})(S-3658) for spans between 2400 and 8000 mm.

K_{2B} = 0.053

U.S. Customary Units:

$$M_{TB} = \left(K_{1B}\gamma S^3 + K_{2B}\gamma\left(H - H_{\min}\right)S^2\right)$$

where:

K_{1B} = 0.0053-0.00024 (S-12) for spans between 8 and 26

SPECIFICATIONS COMMENTARY

ft.

K_{2B} = 0.053

The change of the crown and haunch bending moments due to live loads shall be determined using the following relationship:

$$\Delta M_{TL} = K_{3B} LLS$$

where:

for spans under 6000 mm (20 ft.}

$$K_{3B} = \frac{0.08}{\left(\frac{H}{S}\right)^{0.2}}$$

Metric Units: $K_{3B} = \frac{0.08 - 6x10^{-6}(S - 6000)}{\left(\frac{H}{S}\right)^{0.2}}$

U.S. Cusomary Units: $K_{3B} = \frac{0.08 - 0.002(S - 20)}{\left(\frac{H}{S}\right)^{0.2}}$

The equations above are valid only for spans between 2400and 8000mm {8 and 26 ft.}.

The design crown moment shall be determined using the following relationship:

$$M_{CD} = P_B(1.5M_{TB} + 2.0\Delta M_{TL})$$

where:

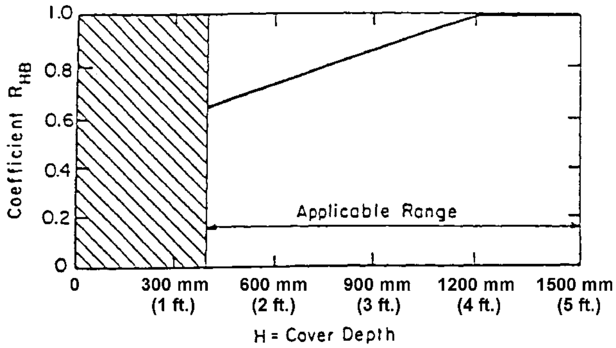
P_B	=	0.55 to 0.70 for spans	under 3000 mm {10 ft.}
	=	0.50 to 0.70 for spans	$\geq$ 3000 mm {10 ft.}, < 4500 mm {15 ft.}
	=	0.45 to 0.70 for spans	$\geq$ 4500 mm {15 ft.}, < 6000 mm {20 ft.}
	=	0.45 to 0.60 for spans	$\geq$ 6000 mm {15 ft.}, < 8000 mm {26 ft.}

The design haunch moment shall be determined using the following relationship:

$$M_{HD} = (1 - P_B)(1.5M_{TB} + 2.0R_{HB}\Delta M_{TL})$$

See Figure 1 for values of R_{HB} as a function of the cover depth, H.

SPECIFICATIONS COMMENTARY



Source: Reference 2

Figure 2.1.2P-1 - Haunch Moment Coefficient

2.1.3P Footing Load

The footing load due to the weight of backfill shall be determined using the following relationship:

Metric Units:

$$F_B = \left(0.025\gamma g S^2 + 0.5\gamma g S H\right)\left(10^{-9}\, m^3 / mm^3\right)$$

U.S. Customary Units:

$$F_B = \left(0.025\gamma S^2 + 0.5\gamma SH\right)$$

The footing load due to the live load shall be determined using the following relationship:

$$F_L = R_{HB} LL$$

The design footing load is the summation of the two equations above. Allowable footing pressures are given in Appendix L, 2.1.10P.

2.1.4P Footing Design

Footings for long-span corrugated metal structures shall be designed to support the loads imposed by the side walls of the structure and reduce the differential settlements which could cause load redistribution and excessive structural distortion. The footings for long-span structures shall be designed to preclude the possibility of a bearing failure into the foundation soils while allowing settlement relative to the adjacent backfill. The design of footings for long-span structures other than corrugated metal box structures shall be in accordance with 1993 DM-4, Section 12, Soil-Corrugated Metal Structure Interaction Systems.

2.1.4.1P FOOTING REACTIONS

The vertical component of the footing reaction for dead and live loads for long-span corrugated metal structures

SPECIFICATIONS COMMENTARY

shall be determined using the following relationships:

(a) Dead load:

Metric Units:

$R_{DL} = (0.025\ \gamma g\ S^2 + 0.5\ \gamma g\ S\ H)\ (10^{-9}\ m^3/mm^3)$

U.S. Customary Units:

$R_{DL} = (0.025\ \gamma\ S^2 + 0.5\ \gamma\ S\ H)$

(b) Live load:

$R_{LL} = AL\ K\ R$

Table 1 shows values of K and R. See Table 2.1.1P-1 for values of AL for various vehicle types and loading conditions.

SPECIFICATIONS COMMENTARY

Table 2.1.4.1P-1 - Live Load Coefficients for Corrugated Box Structure Design

Metric Units			
Depth of Cover (mm)	Live Load Coefficients, K (1/mm)		
	Legal or Permit Vehicles*	Wheel Tractor-Scraper	Live Load Footing Reaction Reduction Factor, R
300	0.000656	0.000755	0.60
600	0.000512	0.000436	0.73
900	0.000381	0.000341	0.88
1200	0.000322	0.000289	1.00
1500	0.000266	0.000236	1.00
2100	0.000246	0.000187	1.00
3000	0.000207	0.000151	1.00

U.S. Customary Units			
Depth of Cover (ft)	Live Load Coefficients, K (1/ft)		
	Legal or Permit Vehicles*	Wheel Tractor-Scraper	Live Load Footing Reaction Reduction Factor, R
1	0.200	0.230	0.60
2	0.156	0.133	0.73
3	0.116	0.104	0.88
4	0.098	0.088	1.00
5	0.081	0.072	1.00
7	0.075	0.057	1.00
10	0.063	0.046	1.00

*Four tires per axle, single or tandem axles, 1500 mm {5 ft.} maximum spacing of load-sharing tandem axle.

Source: Reference 1.

2.2P STRUCTURAL PLATE BOX STRUCTURES WITH CONCRETE RELIEVING SLABS

The procedure for the design of box structures with relieving slabs is similar to that used for structures without relieving slabs, except that the value of K_4 (line load factor) shall be determined using the values presented in Appendix L, Table 2.1.1P-1.

Where used, relieving slabs shall extend a minimum of 1500 mm {5 ft.} beyond the haunch along each side of the

SPECIFICATIONS

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structure and cover all areas over the top of the culvert where traffic may travel, including trafficable shoulders. Conditions with soil under or over the slab shall be considered the same for purposes of analysis.

REFERENCES

Duncan, J. M., and Drawsky, R. H., Design Procedures for Flexible Metal Culvert Structures, Report No. UCB/GT/83-02, Department of Civil Engineering, University of California at Berkeley in cooperation with Kaiser Aluminum and Chemical Sales, Inc., 2nd ed., 1983

Duncan, J. M., Seed, R. B., and Drawsky, R. H., "Design of Corrugated Metal Box Culverts", Transportation Research Record 1008, Transportation Research Board, Washington, DC, 1985

Kaiser Aluminum, Aluminum Box Culvert, DP-125, 8th ed., Kaiser Aluminum and Chemical Company, Oakland, CA, 1983

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PART 4

VOLUME 2

APPENDIX N - PROCEDURE FOR ISSUING A NEW STANDARD OR REVISING AN EXISTING STANDARD

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1.0 PROCEDURE FOR DEVELOPING A NEW STANDARD OR REVISING AN EXISTING STANDARD

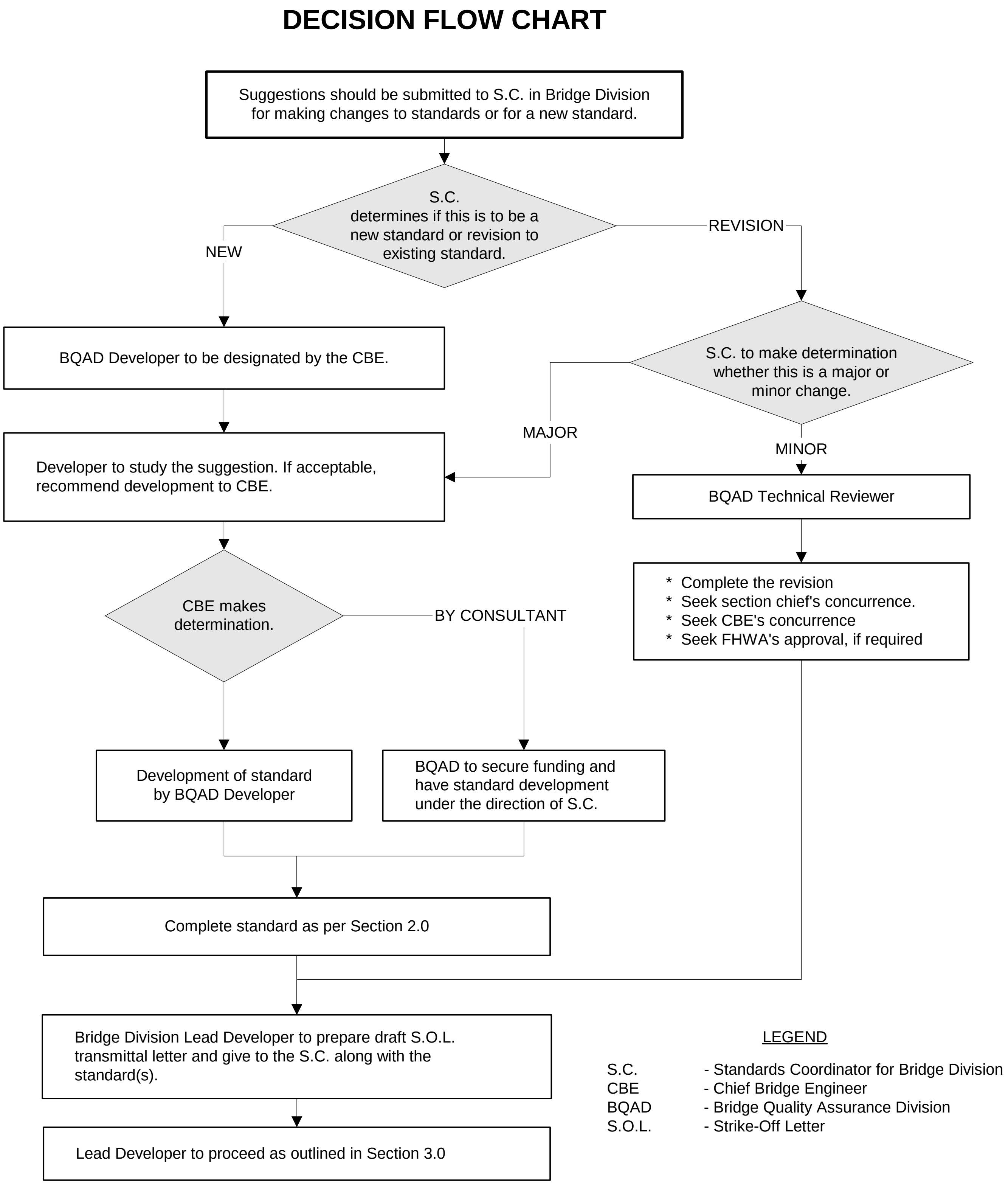


Figure 1.0-1 - Procedure for Developing a New Standard or Revising an Existing Standard

2.0 PROCEDURE FOR PREPARATION OF STANDARD DRAWINGS

1. Requests for New Standards or revisions of existing Standards are sent to the Bridge QA Division's Standard Coordinator (S.C.).

2. S.C. determines if submitted suggestion is to be a new standard or a revision to an existing standard.

3. S.C. along with input from Chief Bridge Engineer (C.B.E.) considers the level of effort required to complete the Standard and, if necessary, contacts the assistance of a Design Consultant. Typically, Standards with minor changes are assigned to a designated BQAD Technical Reviewer or one of three Standard Developers.

4. Reviewers and/or Developers create a red markup set of the standard along with a description of change(s) and submit them to the S.C. for approval.

5. After approval by S.C., documents are given to Lead Developer/CAD Coordinator. Lead Developer determines whether to use Department or Open-end Agreement Consultant CAD resources based on the deadline and overall amount of current Department CAD work.

6. Lead Developer is responsible for sending out all new standards and those standards with significant changes via Clearance Transmittal Letter to Districts, Pennsylvania Turnpike Commission (PTC), FHWA and others for their review. Acrobat "PDF" files of all documents are uploaded to the "Bridge Standards" folder on the Department's ftp site. An e-mail containing a link to the ftp folder is then sent to all reviewers on the Distribution list.

7. Clearance Transmittal Review comments, and both electronic and paper markups are logged in and then complied. These comments are reviewed by the appropriate Developers and final changes are prepared and submitted to S.C. for concurrence.

8. Lead Developer submits final markup set of standard(s) to CAD unit for final changes.

3.0 PROCEDURE FOR ISSUANCE OF STANDARDS

Lead Developer to proceed as follows:

1. Obtain laser quality half-size drawings with date of drawing postdated by about one week.

2. Circulate Standard drawings, Transmittal Letter(s) and issuance Strike-off Letter (S.O.L.) to obtain signatures from the Chief Bridge Engineer, Director of the Bureau of Design and Chief Engineer.

3. Create Acrobat PDF files of signed drawing sheets and Transmittal Letter(s). Print out all drawing sheets and conduct final review. Upon satisfactory review, submit documents to both the Publications Office (Printing Lead Time of 4 weeks) and the Department's Internet Web Master.

4. Upon notification from the Web Master that the Web Site update is complete, BQAD assigns number to S.O.L. and dates it. An Acrobat "PDF" file of S.O.L., including all of its attachments, is created.

5. S.O.L. is e-mailed to all individuals on distribution list.

6. Publications Office sends PDF file of each Publication Change or New Edition to Graphic Services for printing and general distribution.

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 2

APPENDIX O -SOIL-NAILED RETAINING WALLS

APPENDIX O - TABLE OF CONTENTS

SPECIFICATIONS

1.1 GENERAL

Soil-nailed walls may be used to stabilize and retain permanent or temporary cut slopes of weathered rock, granular soils, and clayey soils whose liquidity index is less than 0.2 and undrained shear strength greater than 0.05 MPa {1.04 ksf}. The finished slope may be vertical or at a batter. At present, the permanent facing may be constructed as cast-in-place concrete, shotcrete, or with precast concrete panels.

Because of limited experience with these types of systems, they shall not be used under the following conditions:

(a) For structures greater than 15 000 mm {50 ft.} in height

(b) For retention of granular slopes composed of uniform fine sands or where 1500 mm {5 ft.} cuts would not stay vertically open for the amount of time required for the installation of the nails and the application and subsequent curing of the shotcrete

(c) For retention of cohesive clay slopes exhibiting liquidity indexes greater than 0.2 and undrained shear strengths less than 0.05 MPa {1.04 ksf}

(d) Where the nails would extend beyond the right-of-way limits

(e) For retention, where high groundwater table would generate excessive flows

(f) For retention of frost-susceptible and expansive soils

(g) For retention of weathered rock with weak structural discontinuities that are inclined steeply toward and daylight into the excavation face

A service load design (SLD) approach as defined in the AASHTO Standard Specifications for Highway Bridges, fifteenth edition, (1992); AASHTO Interim Specifications (1993,1994); and DM-4 (1993), and an LRFD approach as defined in the 2004 AASHTO LRFD Bridge Design Specifications shall be used in the design of soil-nailed retaining walls. The most conservative design shall be used, i.e., the longest nail length, largest rebar size, etc.

Contractors specializing in the design and construction of soil-nailed structures shall be responsible for final wall design using the guidelines herein.

COMMENTARY

C1.1

Soil nailing systems are designed to reinforce in situ soil, using passive reinforcements to retain excavations or stabilize or construct vertical or nearly vertical slopes. In soil-nailed retaining structures, the inclusions (nails) are generally steel bars or other metal elements which can resist tensile stresses and bending moments. They are either placed in drilled boreholes and grouted along their total length or driven in the ground. The nails are not prestressed, and their center-to-center spacing (density) is relatively tight, thus providing an anisotropic cohesion. The outside facing of the structure, which ensures local stability between the reinforcement layers, can consist of a thin layer of shotcrete 100 mm to 150 mm {4 in. to 6 in.} thick reinforced with a steel mesh, prefabricated panels, or a cast-in-place concrete veneer. Certain methods of nail installation may be proprietary, as well as certain types of prefabricated facings.

Within Appendix O, the 1992 AASHTO Standard Specifications for Highway Bridges and the 1993 and 1994 AASHTO Interim Specifications will be referred to collectively as AASHTO 92.

The design of soil-nailed walls is a complex problem of soil-structure interaction, strongly influenced by methods of construction. To provide guidance in the design and construction of soil-nailed structures, the Federal Highway Administration (FHWA) has developed a design manual for the design and construction of soil-nailed walls (Byrne, et al, 1998). Preliminary design guidelines, restrictions, and technical considerations developed in earlier editions of this manual have been incorporated herein.

SPECIFICATIONS

1.2 LOADING

Soil-nailed retaining walls shall be investigated for all applicable load combinations from AASHTO 92, Section 3.22, and D3.4.1.1P-3 including, as a minimum:

- lateral earth and water pressures, including any live and dead load surcharge,
- the self weight of the wall, and
- earthquake loads, as specified herein and in DM-4 (1993), Appendix A and D3.10 and A11.9.6.

Walls shall be designed for a minimum live load surcharge equal to 900 mm {3 ft.} of soil, or the actual surcharge, whichever is greater.

In general, temperature and shrinkage deformation effects are not applicable in the design of soil-nailed retaining walls. However, temperature and shrinkage deformation effects shall be considered in the application of soil-nailed retaining walls to underpass widening through end slope removal under an existing bridge abutment.

COMMENTARY

C1.2

Generally, seismic loading will seldom govern in the design of soil-nailed walls constructed in Pennsylvania (see Section C1.6).

SPECIFICATIONS

1.3 MOVEMENT UNDER THE SERVICE LIMIT STATE

The effects of horizontal and vertical wall movement on existing structures or underground utilities shall be investigated.

Horizontal and vertical displacements associated with the construction of the wall may be estimated using Figure 1. For calculations of settlement behind the wall, linear interpolation shall be used with a maximum value of δ_v at the top of the displaced wall decreasing to a value of zero at a distance of $\delta_h + \lambda$ from the back of the displaced wall as shown in Figure 1.

In addition to the displacements associated with the construction of the wall, any time dependent displacements of the wall shall be considered.

When precast panels, CIP concrete, or a second layer of shotcrete is added to the initial construction facing to provide a permanent facing, additional vertical settlement of the wall shall be considered.

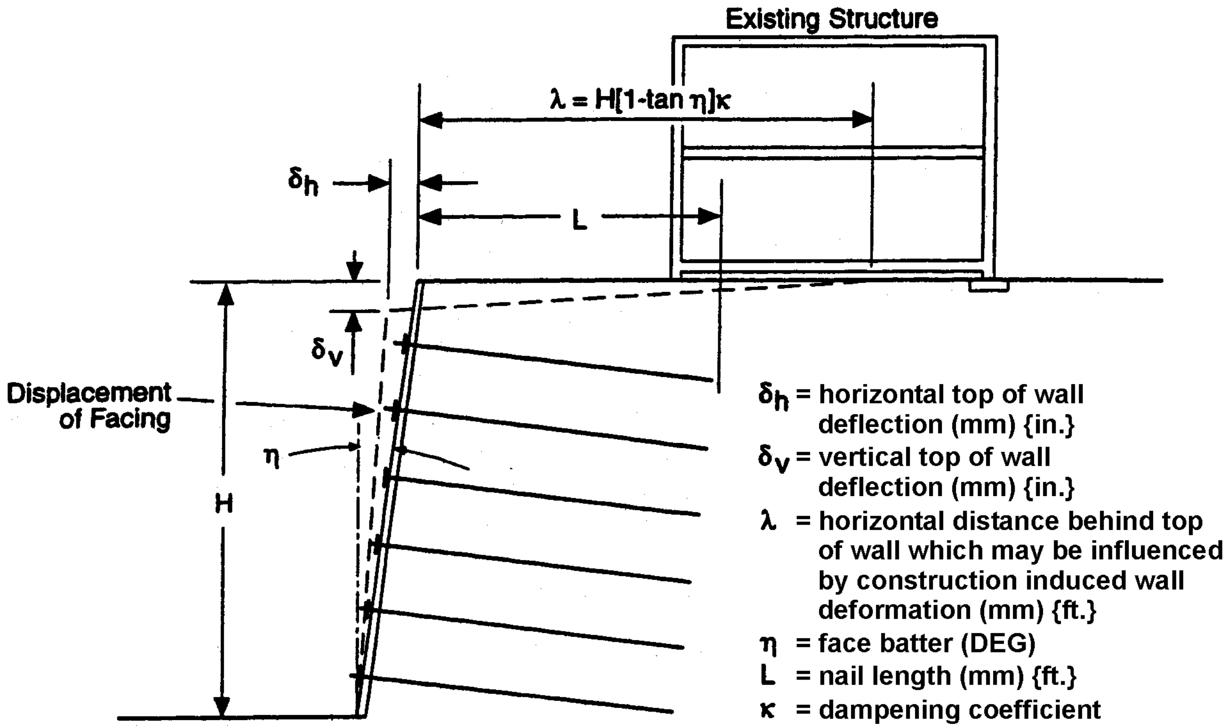
COMMENTARY

C1.3

The horizontal and vertical movements of the wall given in Figure 1 are associated with the construction of the wall. These displacements are the result of the mobilization of the tensile loads within the nails.

Depending on the ground type, post-construction monitoring of wall displacements has indicated that some ongoing movements may occur with time.

For typical construction facings consisting of shotcrete 100 mm {4 in.} thick, the weight of the construction facing is supported by the installed nails. When thicker shotcrete facings are employed during construction, strut nails are usually installed to carry the additional weight. However, when permanent facing is installed after the initial construction facing has been applied, no provisions are made to carry the additional weight of the permanent facing. This additional weight is carried by the soil beneath the wall resulting in additional settlement and an increased potential for a bearing capacity failure of the soil.



Type of Soil	Weathered Rocks Stiff Soils	Sandy Soils	Clayey Soils
δ_h - δ_v coefficient κ	H / 1000 0.8	2 H / 1000 1.25	3 H / 1000 1.5

Figure 1.3-1 - Deformation Behavior of Soil-Nailed Walls (after Clouterre, 1991)

1.4 SAFETY AGAINST SOIL FAILURE

The nailed soil mass shall be treated as a gravity wall and checked for stability against sliding, bearing capacity failure, overturning, and deep-seated foundation failure.

In determining safety against soil failure, the magnitude, location, and inclination of the resultant earth pressure load, applied to the back of the nailed soil mass, shall be taken as specified in Section 5.8.2 of AASHTO 92 and DM-4 (1993) and D11.6.3. However, the soil unit weight, γ, and friction angle, φ, shall be based on the in-situ soil behind the reinforced soil mass. The back and base of the nailed soil mass shall be defined as shown in Figure 1.

C1.4

The consideration of checking the dimensioned structure as a coherent structure subject to sliding and overturning forces is consistent with the present German Design Code, the only known adopted code of practice as outlined by Stocker, et al, (1979). The additional check on bearing capacity failure and deep-seated foundation failure is in accordance with the Manual for Design and Construction Monitoring of Soil Nail Walls (Byrne, et al, 1996).

SPECIFICATIONS

COMMENTARY

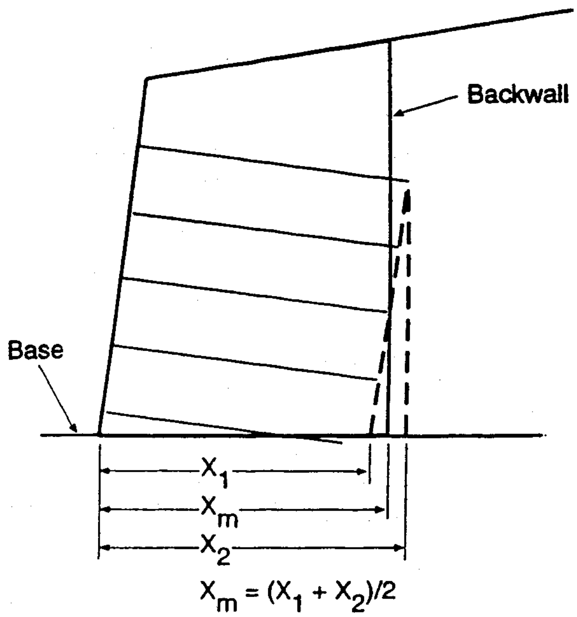


Figure 1.4-1 - Nailed Soil Mass (Byrne, et al, 1996)

For stability computations, live load surcharges shall be applied from a vertical plane beginning at the back of the nailed soil mass. In addition, a saturated soil condition shall be considered in determining stability.

With the following revisions, Figures 5.8.2A, 5.8.2B, and 5.8.2C in AASHTO 92 illustrate stability calculations applicable to soil-nailed walls:

- $L = X_m$
- Revisions as specified in DM-4 (1993), Section 5.8.2.

1.4.1 Sliding

The soil-nailed wall shall be dimensioned to ensure that the factor of safety against sliding specified in DM-4 (1993), Section 5.8.1 and D11.6.3.7 are satisfied. The vertical force, V_2, due to the surcharge load shall not be used in determining the factor of safety against sliding.

1.4.2 Bearing

Bearing capacity shall be evaluated in accordance with Section 5.8.3 of AASHTO 92 and DM-4 (1993). Bearing resistance shall be evaluated in accordance with D11.6.3.2. However, the width of the footing for ultimate bearing capacity shall be taken as shown in Figure 1.4-1.

When precast panels, CIP concrete, or a second layer of shotcrete is added to the initial construction facing to provide a permanent facing, the soil beneath the wall facing shall be investigated for a local bearing capacity failure.

C1.4.2

Refer to the last paragraph of Section C1.3.

1.4.3 Overturning

For stability against overturning, the soil-nailed wall shall be dimensioned to ensure that the factor of safety specified in DM-4 (1993), Section 5.8.1, is satisfied, and the location of the resultant is in accordance with D11.6.3.3.

SPECIFICATIONS

1.4.4 Overall Stability

Overall stability shall be evaluated in accordance with AASHTO 92, Section 5.2.2.3 and D11.6.3.4. For structures loaded with sloping surcharges, general stability analyses shall be performed using Swedish circle methods and yielding a minimum safety factor of 1.5.

1.4.5 Passive Resistance

The passive resistance of soil in front of the wall shall be neglected in stability calculations.

1.4.6 Nail Pullout Capacity

The ultimate pullout resistance per unit length used for preliminary design shall be taken as:

Metric Units: $Q_u = \sigma_b \pi D$ (1.4.6-1)

U.S. Customary Units: $Q_u = 12\sigma_b \pi D$

where:

Q_u = ultimate pullout resistance per unit length (N/mm) {kip/ft}

σ_b = ultimate grout-ground bond stress estimated from Table 1, 2, or 3 (MPa) {ksi}

D = nail drill hole diameter (mm) {in.}

The design pullout resistance in accordance with LRFD used for preliminary design shall be taken as:

$Q = \phi_q\, Qr$

where:

ϕ_q = ground pullout resistance factor
= 0.7, Strength Limit State
= 0.8, Extreme Limit State

Q = Design pullout resistance (N/mm){k/ft}

In calculating the allowable pullout resistance, factors of safety shall be as specified in AASHTO 92, Section 5.7.6.2.

COMMENTARY

C1.4.6

SPECIFICATIONS

Table 1.4.6-1 - Ultimate Bond Stress for Cohesionless Soils

Soil Type	Ultimate Bond Stress	
	MPa	ksi
Non-plastic silt	0.020-0.030	0.0030-0.0045
Loess	0.025-0.075	0.0035-0.0110
Medium dense sand and silty sand/sandy silt	0.050-0.075	0.0070-0.0110
Dense silty sand and gravel	0.080-0.100	0.0115-0.0145
Very dense silty sand and gravel	0.120-0.240	0.0175-0.0345

Table 1.4.6-2 - Ultimate Bond Stress for Cohesive Soils

Soil Type	Ultimate Bond Stress	
	MPa	ksi
Stiff Clay	0.040-0.060	0.0060-0.0085
Stiff Clayey Silt	0.040-0.100	0.0060-0.0145
Stiff Sandy Clay	0.100-0.200	0.0165-0.0290

Table 1.4.6-3 - Ultimate Bond Stress for Rock

Rock Type	Ultimate Bond Stress	
	MPa	ksi
Marl/Limestone	0.300-0.400	0.0435-0.0580
Phillite	0.100-0.300	0.0145-0.0435
Chalk	0.500-0.600	0.0720-0.0865
Soft Dolomite	0.400-0.600	0.0580-0.0865
Fissured Dolomite	0.600-1.000	0.0865-0.1445
Weathered Sandstone	0.200-0.300	0.0290-0.0435
Weathered Shale	0.100-0.150	0.0145-0.0215
Weathered Schist	0.100-0.175	0.0145-0.0255
Basalt	0.500-0.600	0.0720-0.0865

Tables 1 through 3 are based on straight shaft nail drill holes formed by rotary drilling in rock and open hole construction in soils and subsequently grouting by gravity or low pressures.

Field pullout tests shall be conducted to verify the values

COMMENTARY

The tables of ultimate bond stress values are consistent with data available in the literature and shall not be used for final design without verification pullout tests as provided in the Special Provisions.

SPECIFICATIONS

of the ultimate bond stress used for preliminary design using procedures outlined in the specifications. Final design shall be based on field data obtained.

1.5 SAFETY AGAINST STRUCTURAL FAILURE

1.5.1 Global Stability

Soil-nailed structures shall be dimensioned to ensure a minimum factor of safety of 1.5 with respect to global stability.

The global stability of the soil-nailed wall shall be evaluated using slip surface limiting equilibrium methods of analysis modified to incorporate the additional resisting forces provided by the nail reinforcement. The reinforcing contribution of a nail shall be a function of the location at which the associated slip surface intersects the nail, as demonstrated in Figure 1 for a planar slip surface. The available nail strength is limited by the tensile strength of the nail, pullout resistance, or the strength of the nail head. Shear and bending of the nails shall be ignored. Figure 2 shall be used to determine the available nail strength as a function of the location of the intersection of the slip surface with the nail.

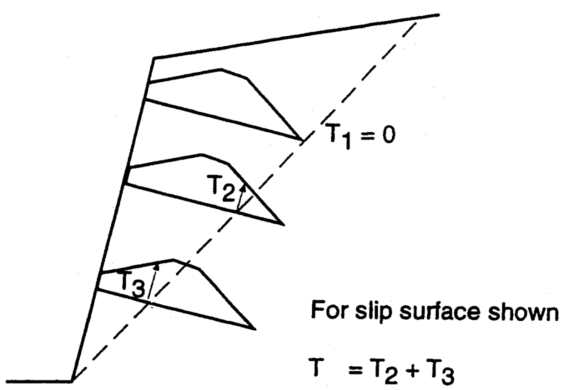


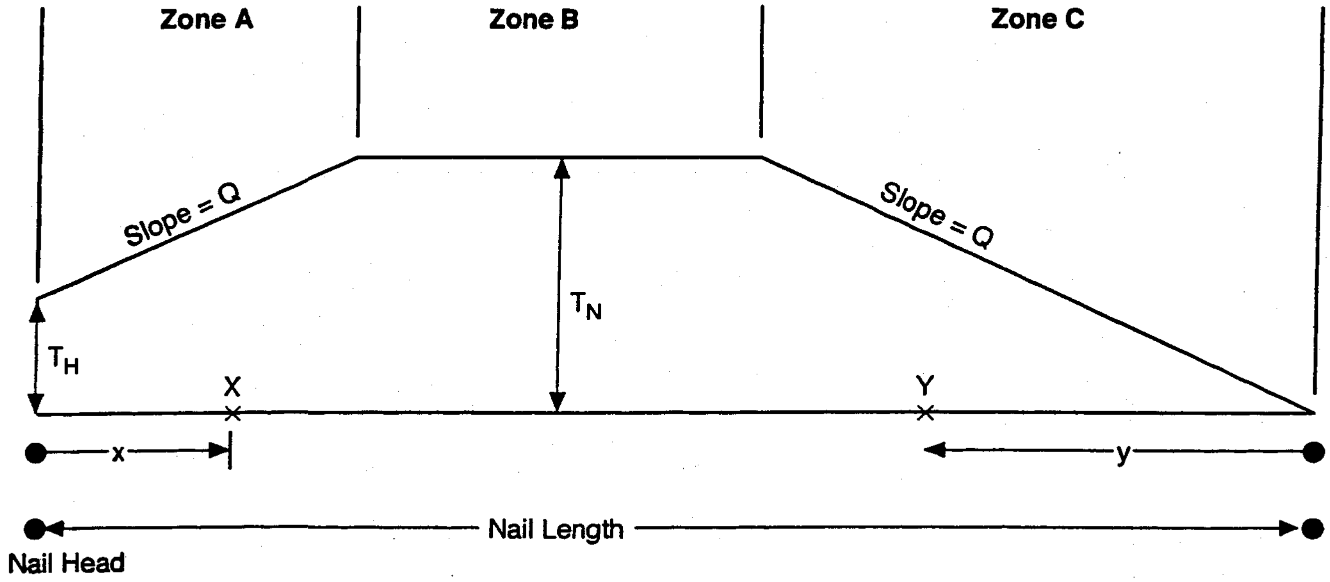
Figure 1.5.1-1 - Nail Reinforcing Contribution (Byrne, et al, 1996)

COMMENTARY

C1.5.1

The Davis (Shen, et al, 1981) or French (Schlosser, et al, 1984) method, modified to meet the requirements herein, may be used. In addition, in using the Davis method, the soil stress ratio, K, shall be taken as the at-rest earth pressure coefficient, K_o. The French method is preferred where complicated soil stratigraphy is present or seepage pressures must be considered.

SPECIFICATIONS COMMENTARY



Slip surfaces intersecting the nail in Zone A at Point X: $T = T_H + Qx$

Slip surfaces intersecting the nail in Zone B: $T = T_N$

Slip surfaces intersecting the nail in Zone C at Point Y: $T = Qy$

T_H = allowable nail head strength in accordance with Section 1.5.2.3

T_N = allowable nail tensile strength as given by Equation 1.5.2.1-1

Q = allowable pullout resistance in accordance with Section 1.4.6

Figure 1.5.1-2 - Available Nail Resistance (after Byrne, et al, 1996)

For design calculation purposes, the nail length pattern used in the analysis of global stability shall be determined from Figure 3. However, it is preferred that nails of uniform length be installed for ease of construction. Shorter nails may be installed in the lower part of the wall provided the appropriate external stability checks are performed.

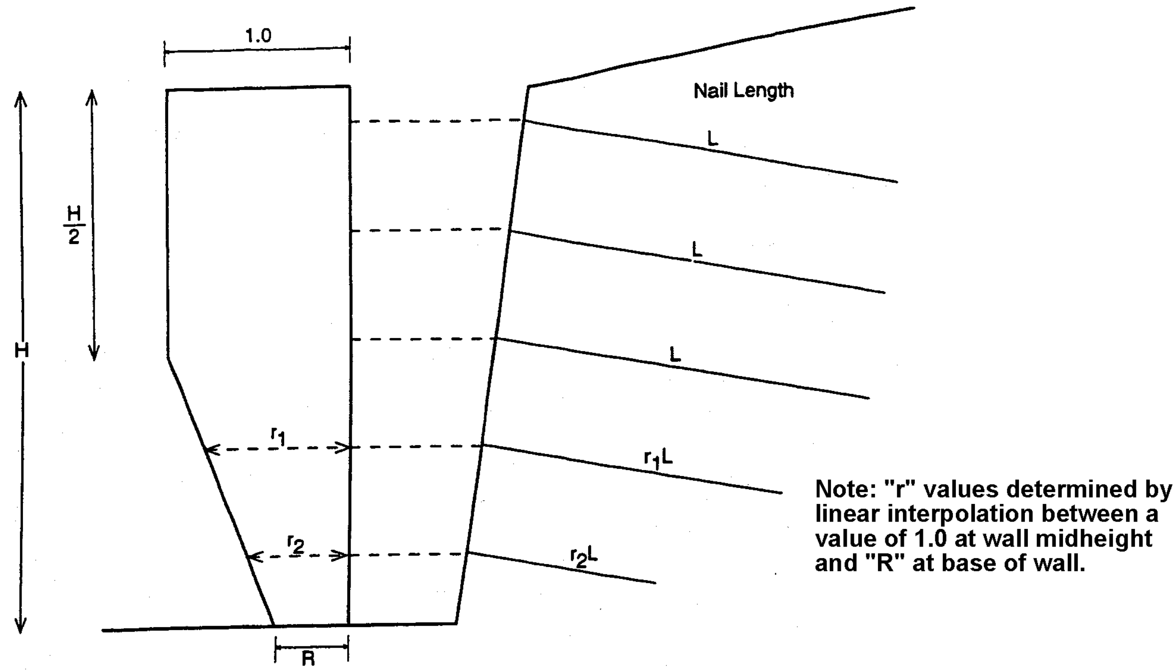
For LRFD design,

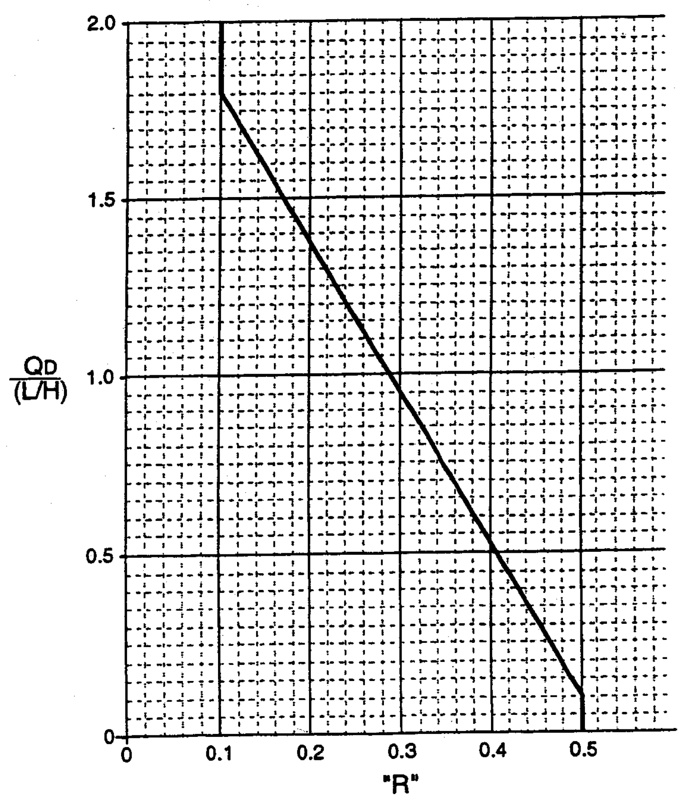
$Q_D = (\phi_q Q_u)/(r_w \gamma s_u s_h)$

where r_w = Soil weight factor from Table 4.6 of the Manual for Design and Construction Monitoring of Soil Nail Walls (Byrne, et al, 1996).

A limitation of the slip surface limit equilibrium method in the design of soil-nailed walls is that it is possible to define a wide variety of nail length patterns that satisfy the specified factor of safety, but result in excessive wall deflections. Performance monitoring of several soil-nail walls has demonstrated that the nails in the upper portion of the wall are more significant than those located lower down in the wall in developing resisting loads and controlling displacements. If nails having the same length as those located in the upper portion of the wall are used in the lower part of the wall, design calculations could overstate their contribution to the global stability of the wall. This can have the effect of indicating shorter nails and/or smaller nail sizes in the upper part of the wall, which is undesirable from a performance standpoint.

SPECIFICATIONS COMMENTARY





L = maximum nail length (mm) {ft.}

H = wall height (mm) {ft.}

Q_D = dimensionless pullout resistance taken as:

Metric Units: $Q_D = \dfrac{Q_u}{FS\ g\ \gamma\ S_H\ S_V\ (10^{-9})}$

U. S. Customary Units: $Q_D = \dfrac{Q_u}{FS\ \gamma\ S_H\ S_V}$

where:

FS = factor of safety as given in Section 1.4.6

Q_u = ultimate pullout resistance as given by Equation 1.4.6-1 (N/mm) {kip/ft}

g = 9.81 m/s^2

γ = unit weight (kg/m^3) {kcf}

S_H = horizontal nail spacing (mm) {ft.}

S_V = vertical nail spacing (mm) {ft.}

Figure 1.5.1-3 - Nail Length Distribution Assumed for Design (after Byrne, et al, 1996)

SPECIFICATIONS

When heterogeneous conditions exist, such as variable soil properties or highly non-uniform surface surcharges, critical slip surfaces that pass through points higher up on the wall shall be investigated.

Frictional and cohesive strength of soils to be used in stability analyses shall be obtained from direct shear or triaxial testing and evaluated at residual strength levels.

COMMENTARY

For relatively uniform surface surcharges and homogeneous soil profiles, the critical slip surfaces resulting in the lowest calculated factors of safety will tend to pass through a point near the toe of the wall. However, when heterogeneities of any type are present, the critical slip surface may not pass through the toe of the wall. An example of this would be when a weak soil overlies a much stronger one. In this case, the critical slip surface may pass through the wall in the vicinity of the contact surface between the weak and strong soil as shown in Figure C1.

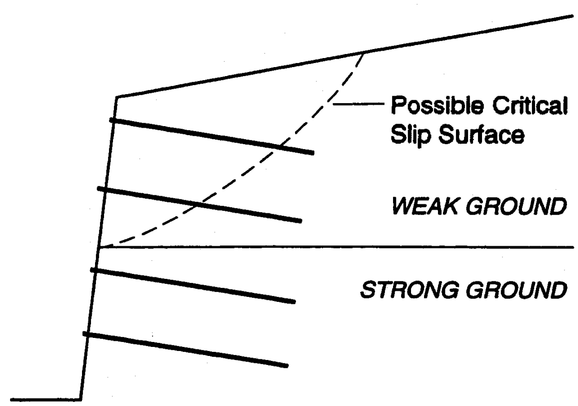


Figure C1.5.1-1 - Potential Critical Slip Surface for Heterogeneous Soil Profiles (Byrne, et al, 1996)

The stability of the wall during its construction shall be investigated. This investigation shall consider temporary construction conditions corresponding to the situation in which the next lift has been excavated, prior to the installation of the nails for that lift. A minimum factor of safety of 1.35 shall be provided.

Construction conditions may control in situations where significant surcharge loads adjacent to the wall exist during construction.

1.5.2 Internal Stability

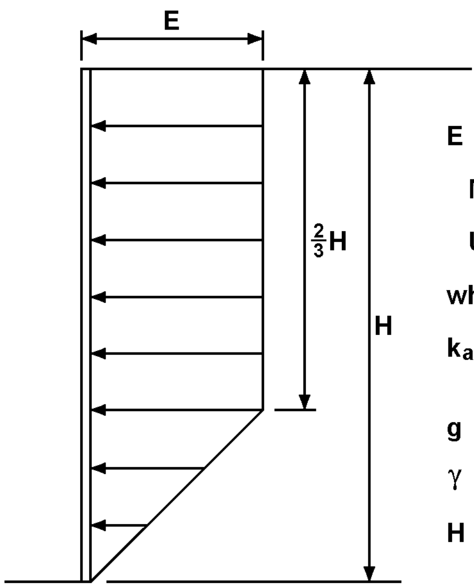
1.5.2.1 SOIL NAIL

The required horizontal component of each nail force shall be computed using the apparent earth pressure distribution given in Figure 1 and any other horizontal pressure components acting on the wall. The total nail force shall be determined based on the nail inclination. The horizontal/vertical nail spacing and nail capacity shall be selected to provide the required total nail force.

C1.5.2.1

The apparent earth pressure distribution given in Figure 1.5.3.1-1 is based on field measurements of instrumented nails in several soil-nailed walls (Byrne, et al, 1996).

SPECIFICATIONS



E = horizontal earth pressure taken as:

Metric Units: $E = 0.75\ k_a\ g\ \gamma\ H\ (10^{-9})$

U. S. Customary Units: $E = 0.75\ k_a\ \gamma\ H$

where:

k_a = active earth pressure coefficient in accordance with A3.11.5.3

g = 9.81 m/s^2

γ = unit weight (kg/m^3) {kcf}

H = wall height (mm) {ft.}

Figure 1.5.2.1-1 - Apparent Earth Pressure Distribution for Soil Nail

The allowable tensile capacity of the nail shall be taken as:

$$T_N = 0.55\ A_N F_y \qquad (1.5.2.1\text{-}1)$$

where:

T_N = allowable nail tensile strength (N) {kips}

A_N = cross-sectional area of nail (mm^2) {in^2}

F_y = specified yield strength of nail (MPa) {ksi}

For LRFD, nail tesnile strength shall be taken as:

$$T_N = \phi_n\ T_{NN} \qquad (1.5.2.1\text{-}1)$$

where:

T_N = nail tendon tensile strength (N) {kips}

T_{NN} = Tendon yield strength (N) {kips}
= $A_N F_y$

ϕ_n = Tendon strength resistance factor
= 0.9 Strength Limit State
= 1.0 Extreme Limit State

Selection of nail inclination shall consider the location of suitable soil and rock strata and the presence of buried utilities and other geometric constraints.

COMMENTARY

Nail inclinations of 15 degrees are common. Flatter or steeper inclinations may be required due to drill rig access restrictions or to avoid underground obstructions. For tremie-grouted nails, care should be taken to ensure that grout fills the nail hole through the entire length of the nail for inclination angles of less than about 10 degrees due to the increased potential for voids in the grout column.

SPECIFICATIONS

Horizontal/vertical spacing of nails in soil shall not exceed 2100 mm {7 ft.}.

Minimum bar size used for soil nails shall be No. 19 {No. 6}.

COMMENTARY

Common nail spacings are 1500 mm by 1500 mm {5 ft. by 5 ft.} or 1800 mm by 1800 mm {6 ft. by 6 ft.}.

1.5.2.2 SOIL-NAIL PULLOUT

The nail load shall be developed by sufficient embedment beyond the line of maximum tension given in Figure 1.

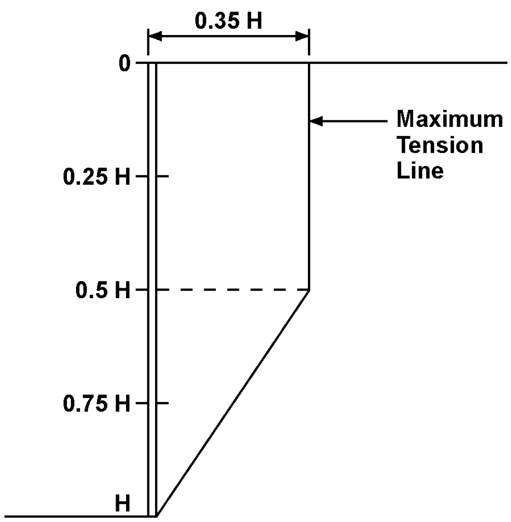


Figure 1.5.2.2-1 - Line of Maximum Tension

C1.5.2.2

The shape of the maximum tension line is actually curvilinear and intercepts the surface at about 0.3H to 0.35H. The shape and location of the maximum tension line is empirically based and is applicable to nearly vertical walls with horizontal backslopes and homogeneous soil conditions. The line of maximum tension given in Figure 1 is an approximation to the curvilinear line and may not apply to heterogeneous soil conditions or sloped back surfaces.

1.5.2.3 NAIL HEAD

The allowable (or LRFD design) nail head strength shall exceed the computed nail head service load (or resistance) for all applicable loads including, but not limited to, horizontal earth pressure, surcharge, water pressure, and seismic loadings. Unless site specific monitoring information is available from walls constructed in similar soils, the nail head load due to horizontal earth pressure may be computed using the apparent earth pressure distribution given in Figure 1.5.2.1-1 multiplied by a nail head reduction factor, F_F, equal to 0.8.

The nominal nail head strength (or LRFD design nail head strength will be limited by the flexural strength of the facing, the punching shear strength of the facing, or in the case of permanent wall facings, the tensile capacity of the headed studs.

The nominal nail head capacity based on the flexural strength of the facing shall be taken as follows:

$$(T_{HN})_F = C_F\left[(M_V)_{NEG} + (M_V)_{POS}\right]\left(\frac{8S_H}{S_V}\right) \qquad (1.5.2.3\text{-}1)$$

where:

$(T_{HN})_F$ = nominal nail head capacity based on the flexural strength of the facing (N) {kips}

C1.5.2.3

Researchers do not have a good understanding of the loads that develop at the nail head due to a lack of sound data in this area. However, the earth pressure distribution given in Figure 1.5.2.1-1 multiplied by a nail head reduction factor of 0.8 may be used until further research becomes available. The nail head reduction factor accounts for the fact that observed nail head loads for typical nail spacings have been significantly less than the loads developed within the nail at the maximum tension line.

The pressure factor, C_F, given in Table 1, accounts for the increased capacity of the facing due to soil arching and was taken as recommended by the FHWA (1996). Equation 1 also comes from the FHWA (1996).

SPECIFICATIONS

C_F = pressure factor for flexure given in Table 1

$(M_V)_{NEG}$ = vertical unit moment resistance at the nail head based on limit state design (N mm/mm) {k ft/ft}

$(M_V)_{POS}$ = vertical unit moment resistance at midspan based on limit state design (N mm/mm) {k ft/ft}

S_H = horizontal spacing of nails (mm) {ft.}

S_V = vertical spacing of nails (mm) {ft.}

When horizontal nail spacings are greater than the vertical spacing of nails or when horizontal unit moment capacities are less than those in the vertical direction, Equation 1 shall also be computed with unit moment capacities corresponding to the horizontal direction and with the vertical nail spacing substituted for the horizontal spacing and vice versa.

Table 1.5.2.3-1 - Pressure Factor for Flexure

Thickness of Facing	C_F for Temporary Facings	C_F for Permanent Facings
100 mm {4 in.}	2.0	1.0
150 mm {6 in.}	1.5	1.0
200 mm {8 in.}	1.0	1.0

When using a C_F value greater than 1.0, the reinforcement ratio (based on gross area) shall not exceed 0.35 percent.

The nominal nail head capacity based on the punching shear strength of the facing shall be taken as:

Metric Units: $$(T_{HN})_v = 0.33\sqrt{f'_c}\,\pi D'_c h_c \qquad (1.5.2.3\text{-}2)$$

U.S. Customary Units: $$(T_{HN})_v = 0.126\sqrt{f'_c}\,\pi D'_c h_c$$

where:

$(T_{HN})_v$ = nominal nail head capacity based on the punching shear strength of the facing (N) {kips}

f'_c = compressive structural design strength of concrete at 28 days, unless another age is specified (MPa) {ksi}

D'_c = diameter of effective punching cone (mm) {in.}

COMMENTARY

For reinforcing ratios (based on gross area) greater than 0.35 percent, Equation 1 tends to over estimate strength when C_F is greater than 1.0.

Equation 2 comes from the FHWA (1996). In determining the nail head capacity based on the punching shear strength of the facing, Equation 2 ignores the resistance provided by the soil as shown in Figure 1.

SPECIFICATIONS

- for bearing plate connections:

$D'_c = b_{PL} + h_c$

where:

b_{PL} = width of bearing plate as shown in Figure 1 (mm) {in.}

h_c = effective cone depth equal to the full thickness of the facing as shown in Figure 1 (mm) {in.}

- for headed stud connections:

$D'_c = S_{HS} + h_c$

where:

S_{HS} = stud spacing as shown in Figure 1 (mm) {in.}

h_c = effective cone depth taken from the top of the headed studs as shown in Figure 1 (mm) {in.}

For headed-stud connections in which the length of the stud is less than half the stud spacing, the strength of the nail head based on the pullout capacity of individual studs shall also be evaluated.

In the case of permanent wall facings, the nominal nail head capacity based on the tensile strength of the headed studs shall be taken as follows:

$$(T_{HN})_T = n\, A_{HS} F_y \qquad (1.5.2.3\text{-}3)$$

where:

$(T_{HN})_T$ = nominal nail head capacity based on the tensile strength of the headed studs (N) {kips}

n = number of studs

A_{HS} = cross-sectional area of stud body (mm^2) {in^2}

F_y = yield strength of stud (MPa) {ksi}

The nominal nail head capacity shall be the minimum determined from Equations 1, 2, and 3.

In determining the allowable nail head strength, the factor of safety shall be based on the limiting failure mode and taken as follows:

For nail head strength limited by:

- facing flexure..1.75
- facing punching shear...1.75

COMMENTARY

Equation 3 was adopted from PCI (1999).

Factors of safety were obtained by dividing the critical load combination for horizontal earth pressure and live load surcharge by the appropriate resistance factor in accordance with LRFD specifications.

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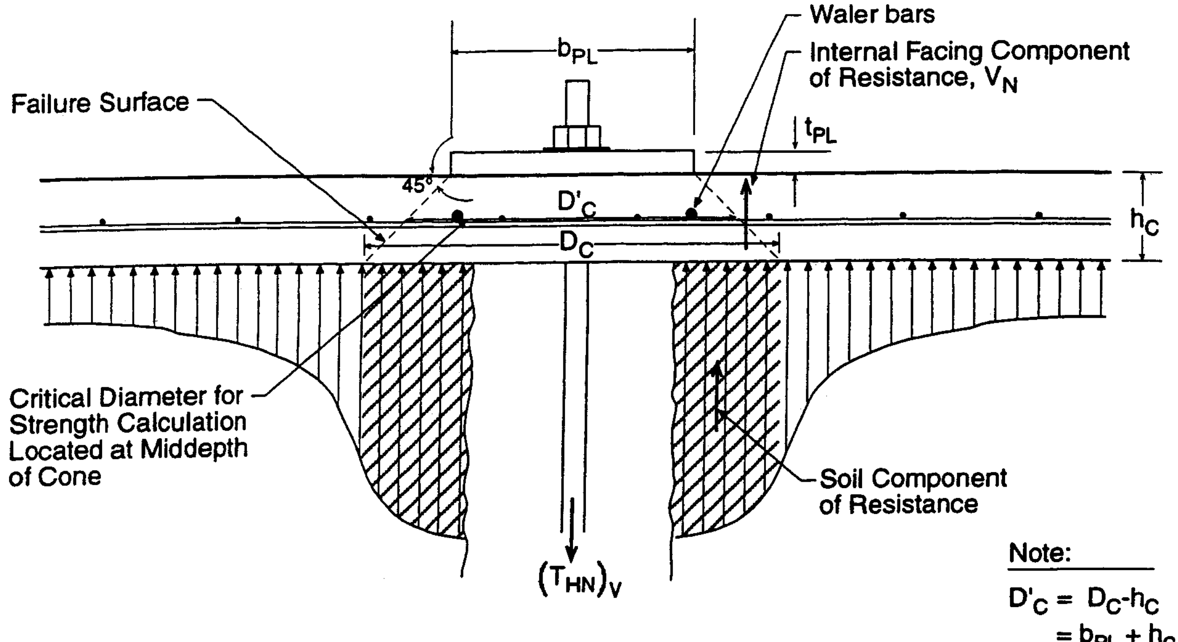
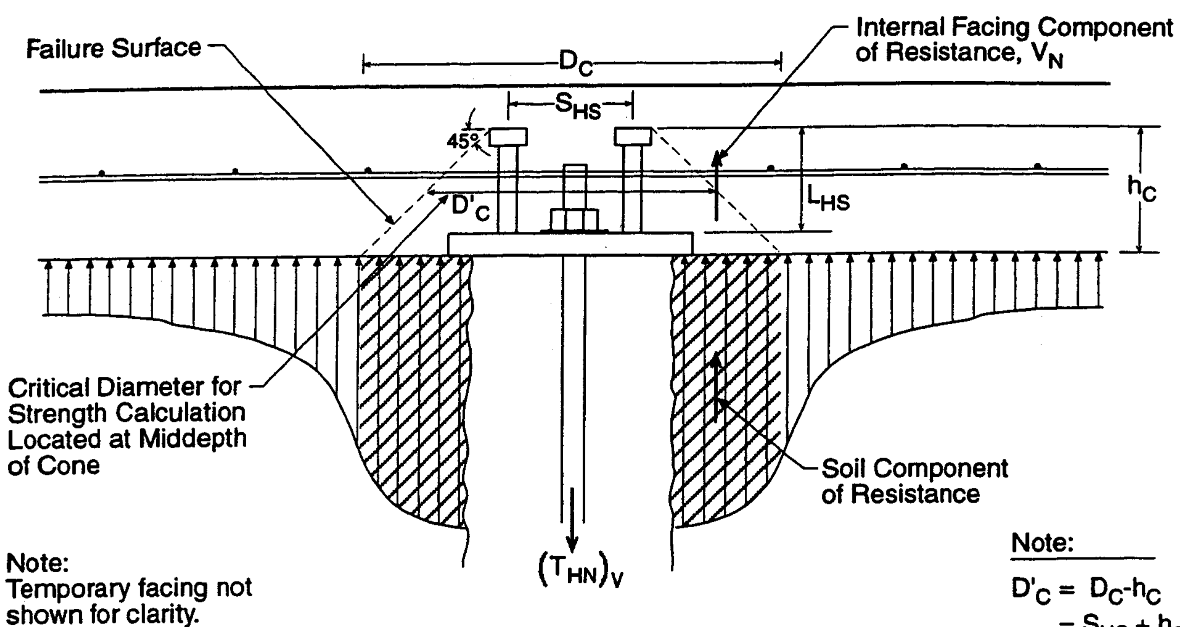
- headed-stud yielding...1.75

For LRFD design, determine the design nail head strength by multiplying the nominal nail head streangth by the corresponding resistance factors from Table 4.7 in the Manual for Design and Construction Monitoring of Soil Nail Walls (Byrne, et al, 1996).



Temporary Bearing-Plate Connection



Permanent Headed-Stud Connection

Figure 1.5.2.3-1 - Punching Shear of Nail-Head Connections (after Byrne, et al, 1996)

SPECIFICATIONS

1.5.3 Facing

A shotcrete facing reinforced with wire mesh may be used for all temporary support systems or for those permanent support systems where aesthetic and environmental concerns would permit it. For permanent applications, a cast-in-place curtain wall or precast panel shall be considered in front of the shotcrete facing.

Cast-in-place concrete facings shall be designed to carry the loads outlined in Section 1.5.2.3. The effects of the temporary facing shall be neglected. When precast concrete facings are used, the shotcrete facing shall be designed as the permanent structural facing. The precast panels and connections from the nail assemblies or shotcrete facing shall be designed to carry the load due to the weight of the facing and the pressure resulting from the previous drainage fill placed between them.

The minimum thickness of shotcrete facing for temporary support shall be 100 mm {4 in.}; for permanent support it shall be 150 mm {6 in.}. The minimum thickness of a cast-in-place facing shall be 200 mm {8 in.}. Minimum cover for mesh in the shotcrete facing shall be 50 mm {2 in.}.

For temporary facings, a minimum of two No. 13 {No. 4} waler bars shall be placed along each nail row and shall be located between the face bearing plate and the back of the shotcrete facing.

For temporary facings, a minimum of two No. 13 {No. 4} waler bars shall be placed along each nail row and shall be located between the face bearing plate and the back of the shotcrete facing.

The moment and shear at the base (centerline of top nail row) of the upper cantilever portion of a soil-nail wall shall be investigated as illustrated in Figure 1. For fill placement and subsequent compaction behind the cantilever, an increased earth pressure shall be considered to account for compaction induced stresses behind the wall.

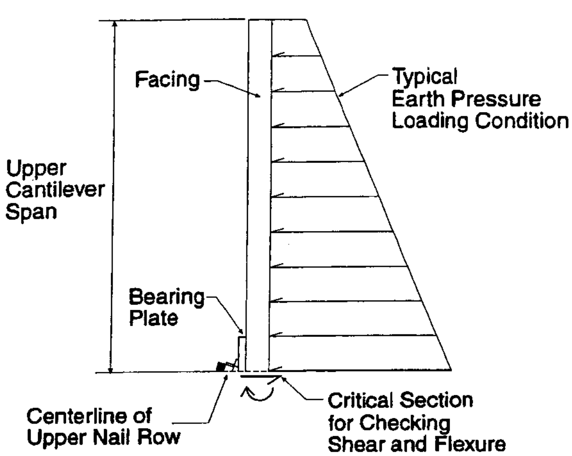


Figure 1.5.3-1 - Upper Cantilever Design Check (Byrne, et al, 1996)

The upper cantilever of a permanent soil-nail wall shall

COMMENTARY

C1.5.3

Waler bars provide for the development of full plastic moment capacity and also provide an element of ductility in the event of a punching shear failure.

Unlike the rest of the wall, the upper cantilever cannot redistribute load by soil arching to adjacent spans, therefore, the moment and shear at the base of the cantilever needs to be checked.

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meet the requirements of Section 8.16.8.4 of AASHTO 92 and DM-4 (1993), and D5.7.3.4 with A5.7.3.4.

The distance between the base of the wall and the bottom row of nails shall not exceed two-thirds the average vertical nail spacing.

Cantilevered end spans shall not exceed two-thirds the average nail spacing.

Shrinkage and temperature reinforcement requirements of Section 8.20 of AASHTO 92 and DM-4 (1993) and D5.10.8 with A5.10.8 shall apply for permanent facing systems.

For permanent applications using shotcrete, consideration shall be given to the introduction of expansion and contraction joints at intervals not exceeding those given in Section 5.5.6.5 of AASHTO 92 and DM-4 (1993) and D11.6.15.

COMMENTARY

If the requirement on the cantilevered end spans is not met, additional design checks are required in this area.

1.5.3.1 STRUT NAIL

For construction facings thicker than 100 mm {4 in.}, consideration shall be given to the installation of strut nails to support the weight of the facing.

C1.5.3.1

For typical construction facings consisting of 100 mm {4 in.}of shotcrete, the soil nails are capable of supporting the weight of the facing through direct shear and bearing on the soil beneath the nails. The maximum thickness of shotcrete facing that can be supported by direct shear and bearing is dependent on the nature of the soil. For relatively thick facings constructed in less competent ground, the soil nails may not be able to support the weight of the facing without large downward and outward displacements of the wall.

1.5.3.2 CONNECTION SYSTEMS

Bearing plates shall have a minimum width and thickness of 200 mm {8 in.} and 19 mm {3/4 in.}.

Headed studs shall extend to at least the middepth of the permanent facing and their heads shall be anchored beyond at least one mat of reinforcement within the permanent facing.

For headed studs, the cross-sectional area of the head shall exceed 2.5 times the cross-sectional area of the body. In addition, the thickness of the head shall exceed one-half the difference between the head diameter and the body diameter.

C1.5.3.2

For bearing plates having a minimum yield stress of 250 MPa {36 ksi} with dimensions less than the specified minimum, flexure and shear of the bearing plate may be critical.

1.6 SEISMIC DESIGN PROVISIONS

For typical soil-nailed walls constructed in Pennsylvania, forces arising from seismic activity need not be considered.

C1.6

For peak ground accelerations below 0.25g, the seismic loading condition will generally not control in the design of soil-nailed walls (Byrne, et al, 1996). Since peak ground acceleration design values do not exceed 0.15g in Pennsylvania, seismic loading will seldom govern in the design of soil-nailed walls.

Should seismic forces be considered, the design engineer shall determine these forces with input from seismic experts with knowledge and experience in seismology and geotechnical engineering.

1.7 CORROSION PROTECTION

Nail head assemblies and nails shall be protected against corrosion consistent with site conditions. The level and extent of corrosion protection shall be a function of the ground

C1.7

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environment and of whether the nail is intended for temporary or permanent applications. For permanent applications in non-aggressive ground, a minimum of double corrosion protection shall be provided. Double corrosion protection can be achieved by resin bond epoxying of the nail and head assembly to a minimum thickness of 0.3 mm {12 mils} in accordance with AASHTO M 284/M 284M and providing minimum grout cover of 38 mm {1.5 in.} along the entire length of the nail. For permanent applications in aggressive ground or for critical structures, nails shall be fully encapsulated. Full encapsulation consists of a nail grouted full-length inside a plastic corrugated tube, placed in an oversized drill hole, and then grouted again against the side of the drill hole. Aggressive site conditions exist whenever one or more of the limiting values specified in Table D11.8.7-1 is exceeded. For all temporary applications, grout cover over the entire nail length shall be adequate. Other potentially corrosive conditions shall be identified and evaluated by the wall designer, and appropriate means of corrosion protection shall be designed by the soil nail wall specialty contractor.

The nail tendon protection, whether epoxy coating or encapsulation, shall extend at least 75 mm {3 in.} into the shotcrete construction facing.

The permanent facing shall provide minimum concrete cover over the nail head assembly in accordance with D5.12.3.

COMMENTARY

If the nail is encapsulated or is an epoxy-coated deformed bar with machine threads at the upper end, the corrosion protection is terminated to expose the bare tendon at the head of the nail in order to allow attachment of the bearing plate and nut. The portion of the exposed tendon within 75 mm {3 in.} of the retained earth is susceptible to corrosion; therefore, the corrosion protection shall extend for at least this distance into the shotcrete construction facing.

1.8 NAIL TESTING

Nail testing for ultimate capacity shall be conducted in accordance with Special Provisions of the Specifications for Soil-Nailed Retaining Walls.

1.9 DRAINAGE

Drainage systems shall be in accordance with A11.8.9 and D11.8.9.

In addition, measures shall be taken to control surface runoff and subsurface flow during construction.

1.10 SUBMITTALS

As a minimum for preliminary design, the information outlined for final design shall be provided with the following changes:

(a) Special Provisions for Soil-Nailed Retaining Walls shall be amended as necessary and included.

(b) Nails shall have no free stressing length.

(c) Type of facing, or shotcrete or cast-in-place concrete (or both) shall be indicated.

As a minimum for final design, the following information

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shall be provided when applicable.

(a) Subsurface exploration information . As a minimum, the following information shall be obtained through the subsurface exploration and testing program for use in design:

(1) Location and thickness of soil and rock units

(2) Engineering properties of in situ soil and rock and granular soil backfill including unit weight, shear strength, and compressibility

(3) Groundwater conditions

(4) Ground surface topography

(5) Geochemistry of soil and groundwater for corrosion potential

(6) Presence of stray electrical currents

(b) Design earth pressures, water pressures, and surcharge loadings (to be included in final plan submission)

(c) Allowable bearing pressures/bearing capacities (to be included in final plan submission)

(d) Design depth of scour if the wall is located adjacent to a stream channel (to be included with foundation stage submission)

(e) Geometric considerations including beginning and ending wall stations, wall profile and alignment, right-of-way limits, utility locations, construction considerations such as traffic restrictions or required construction sequences, and location of wall appurtenances such as drainage outlets, overhead signs and lights, and traffic barriers (to be included in the plans)

(f) References and methods used for analysis including all calculations, computer analyses, assumptions, input, and explanation of all symbols, notations, and formulas (to be included in final plan submission)

(g) Vertical wall element types, sizes, and spacing; hardware details; and erection sequence (to be included in the plans)

(h) Details, dimensions, connections, and schedules of all reinforcing steel for vertical wall elements and facing (to be included in the plans)

(i) Drainage requirements (to be included in the plans)

SPECIFICATIONS COMMENTARY

(j) Corrosion protection and/or accommodation details for the wall elements and hardware (to be included in the plans)

(k) Nail type and estimated capacity, nail inclination, and nail locations and spacings

(l) Description of nail installation procedures including drilling and grouting

(m) Corrosion protection details for the nails and nail head assemblies

(n) Detailed plans for proof, performance/verification, creep (if applicable), and pullout testing of nails including specified load measuring devices, test locations, and testing procedures

(o) Analyses of the stability of the wall at critical stages of construction

1.11 CONTRACTOR'S QUALIFICATIONS

See provisions and requirements under Construction Specifications.

1.12 CONSTRUCTION SPECIFICATIONS

1.12.1 General

Construction specifications for contractor-designed soil-nailed walls shall be governed by the specifications herein. For soil-nailed walls designed by the Department, the following changes shall be made:

(a) Section 1.12.2 - Delete "design and."

(b) Section 1.12.4(a) - Delete in its entirety and substitute, "Provide two sets of shop drawings, material certificates, construction procedures, and detailed construction sequencing plans. For temporary shotcrete facing, provide complete specifications for materials and methods."

(c) Section 1.12.4(g)(3) - Delete "Any modifications of design or construction procedures shall be at no change in the contract prices" and substitute, "Should the nail tests prove that the ultimate bond stress actually being obtained in field production is significantly different than the ultimate bond stress assumed in design, the engineer will make design modifications to increase the nail length or decrease the nail spacing to ensure a stable completed structure. Additional nail lengths or nails beyond those shown on the plans will be paid under an item "Additional Nail Length."

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1.12.2 Description

The furnishing of necessary design and materials as well as the construction of a retaining wall in a cut by internally reinforcing the soil mass with grouted reinforcing elements (nails).

The wall is constructed from the top down as the soil in front of the wall is removed and the nails are installed and grouted at each level. The exposed soil face is immediately protected with a wire mesh reinforced shotcrete facing. A structural cast-in-place concrete or precast concrete facing may subsequently be constructed and suitably attached.

1.12.3 Materials

(a) Soil nails - Section 709.1, Pub. 408 - Thread as necessary. Provide epoxy-coated bars with a minimum thickness of 0.3 mm {12 mils} where required and shown on the plans. Epoxy in accordance with AASHTO M 284/M 284M.

(b) Steel welded wire fabric - Section 709.3, Pub. 408

(c) Cast-in-place concrete - Provide Class AA concrete for structural concrete facing where required and shown on the plans, conforming to the requirements of Section 704, Pub. 408.

(d) Precast concrete - Provide precast concrete facing where required and shown on the plans, conforming to the requirements for panels for mechanically stabilized earth walls.

(e) Permanent structural shotcrete facing:

(1) Materials:

- Cement - Section 701, Pub. 408
- Water - Section 720.1, Pub. 408
- Fine aggregate - Type A, Section 703.1, Pub. 408
- Air entraining admixtures for wet mix - Section 711.3(d), Pub. 408
- Accelerating additives and plasticizers - AASHTO M 194

Additives are to be non-corrosive to steel. They are to contain calcium chloride and/or prevent other detrimental effects such as cracking and excessive shrinkage.

(2) Shotcrete quality - Produced by either dry mix or wet mix process achieving a minimum

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compressive strength of 21 MPa {3 ksi} in seven days and 32 MPa {4.6 ksi} in 28 days.

(3) Pre-constructing testing - Test specimens to be made by each application crew using the equipment, materials, mixture proportions, and procedures proposed for the job.

A test panel at least 750 mm by 750 mm {30 in by 30 in.} shall be made for each mixture being considered and for each shooting position to be encountered in the job. The test panels are to be fabricated to the same thickness as the structure, but not less than 100 mm {4 in.}. Take at least five 100 mm {4 in.} diameter cores from each panel for testing in accordance with AASHTO T 24.

(4) Mixture proportions - Provide samples for testing. Submit for acceptance the recommended mixture proportion, source of materials, and all test results. Select mixture proportion on the basis of compressive strength tests of specimens continuously moist cured until tested at 28 days. For mixture acceptance purposes, average core compressive strength shall be equal to at least 1.2 times the required compressive strength specified in (2) above.

(5) Batching and mixing - Aggregate and cement may be batched by weight or volume. Provide mixing equipment capable of thoroughly mixing the materials in sufficient quantity to maintain placing continuity. For dry mix process, provide capability to discharge all mixed materials without any carry-over from one batch to the next. Provide ready mix shotcrete complying with AASHTO M 157 for wet mix process.

(6) Delivery equipment - For dry mix process, provide equipment capable of discharging the aggregate-cement mixture into the delivery hose and delivering a continuous, smooth stream of uniformly mixed material to the discharge nozzle. Equip the discharge nozzle with a manually operated water injection system (water ring) for directing an even distribution of water through the aggregate-cement mixture. The water valve shall be capable of ready adjustment to vary the quantity of water and shall be convenient to the nozzleman. Provide water pressure at the discharge nozzle sufficiently greater than the operating air pressure to ensure that the water is intimately mixed with the other materials. If the line water pressure is inadequate, introduce a water pump into the line. Ensure that the water pressure is steady (non-

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pulsating). Clean delivery equipment thoroughly at the end of each shift. Inspect equipment parts, especially the nozzle lines and water ring, at regular intervals and replace as required.

For wet mix process, provide equipment capable of delivering the premixed materials accurately, uniformly, and continuously through the delivery hose. Follow recommendations of the equipment manufacturer on the type and size of nozzle to be used, and on cleaning, inspection, and maintenance of the equipment. Ready mixed shotcrete is to be delivered in transit mixers complying with AASHTO M 157.

Provide a supply of clean, dry air adequate for maintaining sufficient nozzle velocity for all parts of the work and, if required, for simultaneous operation of a suitable blow pipe for clearing away rebound.

(7) Safety requirements - Maintain safety in all areas where shotcrete is to be applied, including dust protection. Causticity of cement and accelerating hardening admixtures may cause skin and respiratory irritation unless safety measures are taken in addition to providing required ventilation. During the application of shotcrete, provide nozzlemen and helpers with gloves and adequate protective clothing.

(8) Finish - Provide undisturbed final layer of shotcrete as applied from nozzle without hand finishing, unless otherwise specified.

(9) Curing - Immediately after finishing, keep shotcrete continuously moist for at least 24 hours. Use one of the following materials or methods:

- Ponding or continuous sprinkling
- Absorptive mat or fabric, sand, or other covering kept continuously wet
- Curing compounds - (Refer to AASHTO M 148). On natural gun or flash finishes, use the application rate of 2.5 m^2/L {100 ft^2/gal}. Do not use curing compounds on any surfaces against which additional shotcrete or other cementitious finishing materials are to be bonded unless positive measures, such as sandblasting, are taken to completely remove curing compounds prior to the application of such additional materials.

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Provide additional final curing immediately following the initial curing and before the shotcrete has dried. Use one of the following materials or methods:

- Continue the method used in initial curing
- Application of impervious sheet material conforming to AASHTO M 171

Continue curing for the first seven days after shotcreting or until the required strength is obtained. During the curing period, maintain shotcrete above 4º C {40º F} and in a moist condition as specified.

(10) Construction testing - Cut cores from the structure and test in accordance with AASHTO T 24. Three cores from 90 m^2 {1,000 ft^2} of facing or per shift are required.

Alternately, make one test panel with minimum dimensions of 450 mm by 450 mm by 100 mm {18 in. by 18 in. by 4 in.} gunned in the same position as the work represented for each 90 m^2 {1,000 ft^2} of completed facing. Gunn panels during the course of the work by the contractor's regular nozzleman. Field cure panels in the same manner as the work; however, the test specimens shall be soaked in water for a minimum of 40 hours prior to testing. Cut a minimum of three cores from each panel for testing in accordance with AASHTO T 24. The average compressive strength of each core of a set of three cores shall equal or exceed 85 percent of the compressive strength specified.

(f) Temporary shotcrete facing - Provide for approval, materials, methods, and control procedures.

(g) Grout - Provide a neat cement grout to be used in soil nail anchorage consisting of a pumpable mixture capable of reaching a cube strength of 21 MPa {3 ksi} in accordance with AASHTO T 106. Chemical additives that can control, bleed, or retard set in the grout are to be used only when approved in writing.

(h) Fasteners and attachment devices - Provide high-strength nuts conforming to AASHTO M 291. Provide plates and shims conforming to AASHTO M 183M/M 183. Provide plastic centralizers of a minimum diameter 13 mm {1/2 in.} smaller than the nominal diameter of the drill hole.

(i) Horizontal drains - Provide as required and shown on the plans slotted and unslotted PVC pipe conforming to AASHTO M 278. Install to the depths directed by

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the engineer, which will not exceed the maximum depths shown on the plans. Insure that the hole does not collapse prior to the insertion of the slotted drain. Only the front 300 mm {12 in.} of drain pipe shall be unslotted.

(j) Wall drains - Provide as required and shown on the plans, prefabricated, fully wrapped, preformed drains. The core, not less than 6 mm {1/4 in.} thick or more than 13 mm {1/2 in.} thick, shall be either a preformed grid of embossed plastic or a system of plastic pillars and interconnections forming a semirigid mat. When covered with filter fabric, the core material shall be capable of maintaining a drainage void for the entire height of permeable liner. Provide a polypropylene geotextile having a minimum weight of 200 g/m^2 {6 oz/yd^2} as the filter fabric.

1.12.4 Construction

(a) Submittals - Provide two sets of design drawings, calculations, material certificates, construction procedures, and detailed construction sequencing plans, including excavation sequence for approval. Provide sufficient details in the design drawings to eliminate a need for shop drawings. For temporary shotcrete facing, provide complete specifications for materials and methods.

Assume all risks for work performed without approved plans.

(b) Qualifications - Submit proof of two projects on which contractor has designed and/or installed soil nails or ground anchors in the past two years. The contractor's staff on this project is to include a supervising engineer with at least three years of experience in the design and construction of anchored walls.

Drilling operators and foreman are to have a minimum of two years experience installing soil nails or permanent ground anchors with the contractor's organization. Submit documentation that project personnel have appropriate qualifications. Inadequate proof of personnel qualifications shall be cause for withholding wall design approval. Changes to previously approved personnel must be approved in writing.

Provide shotcreting nozzle operators with one year of experience in the application of shotcrete on projects of comparable nature. Shotcreting nozzle operators who have worked under the immediate supervision of a foreman or instructor for at least two years are also qualified.

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(c) Excavation - In conformance with Section 203, Pub. 408, and to the limits and construction stages indicated.

(d) Shotcreting - After each stage cut, and in anticipation of shotcreting, clean surfaces of all loose material, mud, rebound from previously placed shotcrete, and other foreign matter that will prevent bond of shotcrete. Use weep holes, drain pipes, or other methods to control seepage. Where used, provide a weep hole, a 600 mm {2 ft.} long, 50 mm {2 in.} diameter, slotted drain pipe (Schedule 40 PVC) placed in pre-drilled holes sloped 5 percent to drain. Protect against contamination during shotcreting to ensure proper functioning. Dampen surface before shotcreting.

Install permanent drainage as specified.

Apply shotcrete with the same equipment and the same technique as those used in the approved test panels. Nozzle operators performing the test panels are to be the same operators used to place shotcrete in the work. Measuring pins shall be installed on 1500 mm {5 ft.} centers in each direction. The pins shall be non-corrosive and designed to prevent infiltration of water through the shotcrete. Other methods may be approved to establish whether the required minimum thickness of shotcrete is being applied if the contractor can satisfactorily demonstrate the reliability of these other methods.

When a layer of shotcrete is to be covered by a succeeding layer at a later time, it shall first be allowed to develop its initial set. Then remove all laitance and loose material, and rebound by brooming or scraping. Remove laitance that has been allowed to take final set by sandblasting, and thoroughly clean surface.

Do not shotcrete if ambient temperature is less than 4 C {40 F}. Maintain curing temperature as specified under Section 1.12.3(e)(9) of these specifications.

Firmly position the wire fabric to prevent vibration while the shotcrete is being applied. Lap mesh 1 1/2 squares in both directions. Bend tie wires flat in the plane of the mesh and do not form large knots. Provide a minimum cover of 50 mm {2 in.} of shotcrete.

Control thickness, method of support, air pressure, and water content of shotcrete to preclude sagging or sloughing off.

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Fill first horizontal and vertical corners and any area where rebound cannot escape or be blown free.

Hold nozzle at a distance and angle that will place the material behind the reinforcement before any material is allowed to accumulate on its face. In the dry mix process, additional water may be added to the mix when encasing reinforcement, to facilitate a smooth flow of material behind the bars. Do not place shotcrete through more than one layer of reinforcing steel rods or mesh in one application unless preconstruction tests have demonstrated that steel is properly encased.

Taper construction joints over a minimum distance of 300 mm {12 in.} to a thin edge, and thoroughly wet before placing any adjacent section.

Repair surface defects as soon as possible after initial placement of the shotcrete. All shotcrete that lacks uniformity, that exhibits segregation, honeycombing, or lamination, or that contains any dry patches, slugs, voids, or sand pockets shall be removed and replaced with fresh shotcrete.

Do not repair core holes with shotcrete. Instead, fill solid with patching mortar after cleaning and thoroughly dampening.

Use the following precautions during shotcreting:

(1) Do not place shotcrete if drying or stiffening of the mix takes place at any time prior to delivery to the nozzle.

(2) Do not use rebound or previously expended material in the shotcrete mix.

(3) The area to which shotcrete is to be applied shall be clean and free of rebound or overspray.

(4) Discontinue shotcreting when temperature drops below 4º C {40º F} or when shotcrete cannot be protected.

(e) Nail installation - Drill holes for soil nails at the location shown. Provide the soil nail length necessary to develop adequate load capacity to satisfy testing acceptance criteria for the design load required, but not less than the length shown on approved plans. Casing may be necessary to maintain a clean open hole drilled to the size and inclination shown. Drilling methods and grouting pressure are at the option of the contractor. At the point of entry, the nail angle shall be within plus or minus 3 degrees of that shown on the approved plans. Subsidence or physical damage by such operations

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shall be cause for immediate cessation of operations and repair at the contractor's expense.

Inject grout at the lowest point of the drill hole. Pump grout through grout tubes, casing, hollow-stem augers, and drill rods until the hole is filled to prevent air voids. Fill with grout progressively from the bottom to top. Provide grouting equipment capable of continuous mixing and producing a grout free of lumps. Place a nail in each drilled hole within 15 minutes of the grout injection.

Place centralizers at 3000 mm {10 ft.} intervals in total length, and ensure that no less than 38 mm {1.5 in.} of grout cover is achieved at all locations along the tendon.

Lightly stress installed nails to take up any slack after the grout has reached a compressive strength of at least 10 MPa {1.5 ksi}.

(f) Construction sequencing - Follow the construction sequence on the approved plans closely.

(g) Nail testing:

(1) Equipment - Provide a dial gauge capable of measuring to 0.025 mm {0.001 in.} to measure movement. A hydraulic jack and gauge calibrated as a unit shall be used to apply the test load. Provide pressure gauge graduated in 500 MPa {100 psi} increments or less and use to measure the applied load. Apply test load incrementally.

(2) Pullout testing - Install one nail per horizontal row, but no more than 3 percent of the total number of nails as non-service nails, and load test to pullout failure (maintained movement without increased load). Install and test at each level at a rate consistent with construction operations. Choose test length of nail to cause pullout failure prior to steel yield, but not at less than 2400 mm {8 ft.}. Provide a minimum ungrouted zone 900 mm {3 ft.} long to the face. The method of installation and size of the drill hole shall be the same as for the production nails.

Grout in place each test nail as part of a regular production grouting process. After grouting, do not load for a minimum of three days.

Perform pullout test by incrementally loading the nail in accordance with the following schedule. Measure nail movement and record to the nearest 0.025 mm {0.001 in.} with respect to an

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independent fixed reference point at each increment of load. Monitor the test with a pressure gauge. The load hold period shall start as soon as the test load is applied. Movement shall be recorded at 1 minute, 2, 3, 4, 5, 6 and 10 minutes. Each increment of load shall be no greater than 15 percent of the steel yield strength of the nail. Terminate loading at failure or earlier, at the option of the contractor, if the design friction limit is demonstrated.

(3) Acceptance criteria - The nail is acceptable if the developed friction limit at failure is greater than the design friction limit. Unacceptable test results shall result in modification to design and/or construction procedures. Any modification of design or construction procedures shall be at no change in the contract prices. Graphs shall be plotted during the test of deflection against load.

(h) Cast-in-place concrete facing - At the completion of the sequenced construction and where required, construct cast-in-place structural facing in accordance with the provisions of Section 1001, Pub. 408.

(i) Precast concrete facing - At the completion of the sequenced construction and where required, construct precast concrete structural facing in accordance with the provisions for precast concrete panels for mechanically stabilized embankments. Attachment devices are to be shown on the plans as to size and material composition.

1.12.5 Measurement and Payment

Lump sum.

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PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 2

APPENDIX P - JACKING AND SUPPORTING THE SUPERSTRUCTURE

The following sections are excerpts from the AASHTO Maintenance Manual for Roadways and Bridges 2007 and are provided for guidance in developing shoring of superstructures.

Jacking and Supporting the Superstructure
Bridge rehabilitation and superstructure repair often require jacking to provide load transfer and bridge support while repairs are made. If jacking is required, both the safety of any vehicular traffic continuing to use the bridge while maintenance and repair are conducted and the safety of all working personnel must be considered.

- Jacks and jacking supports must be straight, plumb, and of sufficient capacity to support the portion of the bridge being lifted.
- Place jacks at points that will not damage the structure; reinforce jacking points on the structure if necessary.
- Before jacking, check deck joints for offsets that might be damaged from differential movement between spans during jacking; to prevent damage, check railings and disconnect them, if needed.
- Uniformly raise and lower jacks to distribute the jacking load evenly and prevent overstressing or twisting the bridge.
- Position blocks adjacent to the jacks to increase their height as the structure is raised, minimizing any loss of support if a jack fails during the jacking operation.
- Do not permit traffic on the bridge while it is supported by jacks. If traffic is permitted on the bridge while it is supported by blocks, a vertical transition slope should be provided to avoid abrupt changes in the road surface, to provide a safe riding surface, to prevent damage to the bridge, and to minimize any vertical acceleration loading from the traffic.

Jacking the Superstructure. The procedure should be designed by a Professional Engineer licensed in the Commonwealth of Pennsylvania for each jacking set-up, ensuring that it is adequate for the job to be undertaken. This design and review must account for the following factors (16):

- Dead-load reaction to bear on the jacks.
- If the bridge cannot be closed to traffic during jacking (preferable), then the expected live load on the jacks must be included.
- The size and number of jacks required.
- The location of the jacks.
- Any temporary bents or cribbing required to support the jacks.
- Any modifications to bridge structural members required at the jacking points so that the bridge members can sustain the jacking pressure.
- Sufficient space at deck joints to permit differential movement between spans.
- Defining the height to which the structure needs to be jacked, jacking only as high as is absolutely necessary to conduct the required maintenance.

Generally, the steps and precautions included within a jacking procedure to ensure successful, safe operation are as follows:

1. Construct the necessary bents and cribbing to support the jacks when it is not possible to locate supports on the existing substructure. An adequate foundation to prevent differential settlement is very important.
2. As necessary, reinforce bridge members to withstand the force of the jacks.
3. If necessary, disconnect railing and utilities.

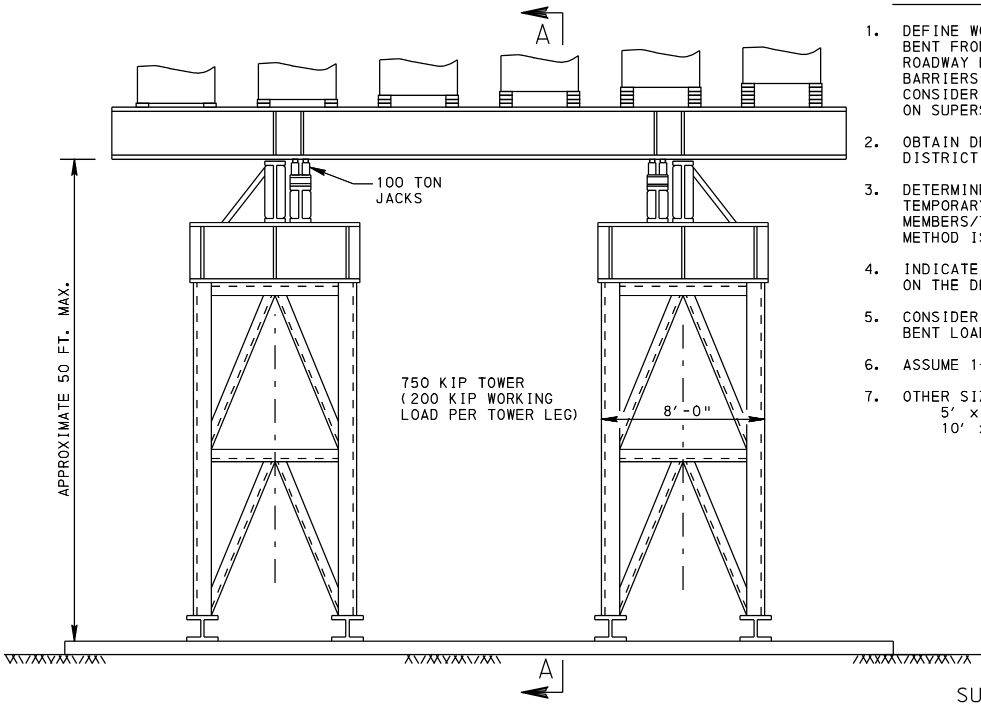
4. Place jacks snugly in position.
5. Restrict vehicular traffic on the span while it is supported by jacks.
6. Raise the span by jacking. Pressure gauges should be used to ensure that all of the jacks are lifting the span evenly.
7. Use observers placed at strategic points to watch for signs of structural distress because of jacking.
8. Jack, block, and rejack until the required position is achieved.
9. Protect joints and provide a transition to the span with steel plates if traffic is maintained while the span is on blocks.
10. Check periodically to ensure that there is no differential settlement.
11. After the repairs are completed, remove the blocks using the jacks.
12. After the span has been lowered into place, ensure that the deck joints are functioning properly, that the alignment has not been changed, that there is adequate space for expansion without debris or restriction in the joins, and that the joint seal is watertight.

The attached schematics are provided for guidance in developing shoring plans:

- Partially Supported Superstructure

- Fully Supported Superstructure

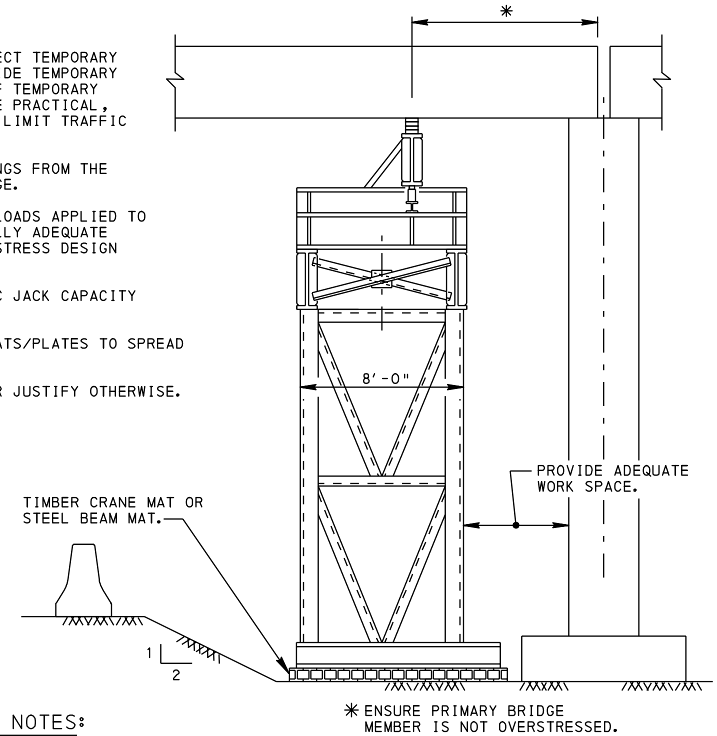
- Partially Supported Superstructure Alternate Shoring Method



ELEVATION

GUIDANCE NOTES:

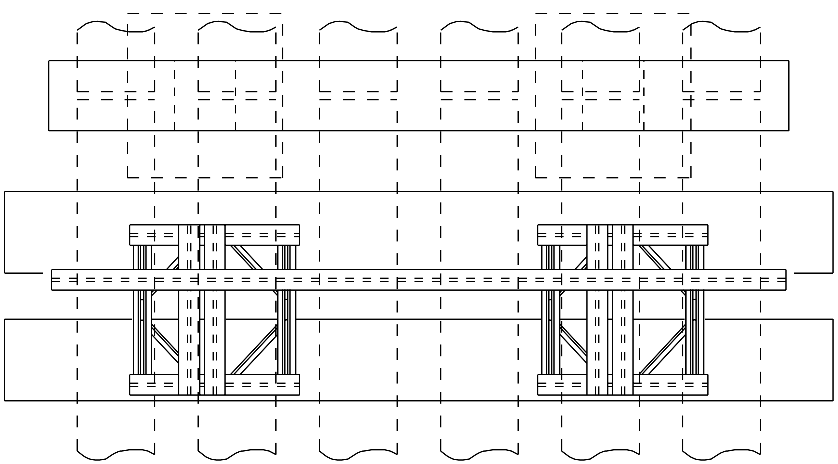
1. DEFINE WORKING SPACE LIMITS AND PROTECT TEMPORARY BENT FROM TRAFFIC AS REQUIRED. PROVIDE TEMPORARY ROADWAY BARRIER(S) AND TERMINATION OF TEMPORARY BARRIERS PER RC-58M AS NEEDED. WHERE PRACTICAL, CONSIDER USE OF TEMPORARY BARRIER TO LIMIT TRAFFIC ON SUPERSTRUCTURE.
2. OBTAIN DESIGN AND CONSTRUCTION DRAWINGS FROM THE DISTRICT OFFICE OF THE EXISTING BRIDGE.
3. DETERMINE ANTICIPATED DEAD AND LIVE LOADS APPLIED TO TEMPORARY BENT AND PROVIDE STRUCTURALLY ADEQUATE MEMBERS/TOWERS. USE OF THE WORKING STRESS DESIGN METHOD IS ACCEPTABLE.
4. INDICATE TOWER CAPACITY AND HYDRAULIC JACK CAPACITY ON THE DRAWINGS.
5. CONSIDER USE OF LARGE DISTRIBUTION MATS/PLATES TO SPREAD BENT LOAD OVER A LARGE AREA.
6. ASSUME 1-2 T.S.F. BEARING CAPACITY OR JUSTIFY OTHERWISE.
7. OTHER SIZE TOWERS:
   5' × 5' - 650^{K}/TOWER
   10' × 10' - 385^{K}/ LEG × 4 LEGS



SECTION A-A

SUGGESTED CONSTRUCTION NOTES:

1. PROVIDE TEMPORARY BENT AND BRACING AS REQUIRED TO MAINTAIN STRUCTURAL STABILITY.
2. PROVIDE MATERIALS AND WORKMANSHIP IN ACCORDANCE WITH PUBLICATION 408-07.
3. EVALUATE IN-SITU FOUNDATION MATERIAL AND OVER EXCAVATE ANY UNSUITABLE FOUNDATION MATERIAL AND BACK FILL WITH COMPACTED NO. 2A COARSE AGGREGATE WHEN REQUIRED.
4. DIVERT SURFACE WATER RUNOFF AWAY FROM THE TEMPORARY BENT FOUNDATION.
5. PROVIDE SHIMS AND BRACING AS REQUIRED TO MAINTAIN STRUCTURAL STABILITY AND SUPERSTRUCTURE ALIGNMENT. BENT COMPONENTS ESPECIALLY SHIMS AND POINT OF EXISTING SUPERSTRUCTURE SUPPORT SHALL BE TACKED/BOLTED/GUIDED OR OTHERWISE SECURED SUFFICIENT TO PRECLUDE UNINTENDED STRUCTURE MOVEMENTS WHILE SUPPORT HAS BEEN TRANSFERRED TO THE TEMPORARY BENT.
6. PROVIDE TEMPORARY BRACING CALCULATIONS AND DRAWINGS SEALED BY A PROFESSIONAL ENGINEER REGISTERED IN PENNSYLVANIA.
7. DO NOT CAUSE UNACCEPTABLE OVERSTRESS TO OTHER BRIDGE COMPONENTS.
8. CERTIFY FALSE WORK PER PUB. 408, SECTION 105.03 (c).
9. JACK SIMULTANEOUSLY WHEREVER POSSIBLE. ALTERNATELY JACK WITH ASSIST POINT.

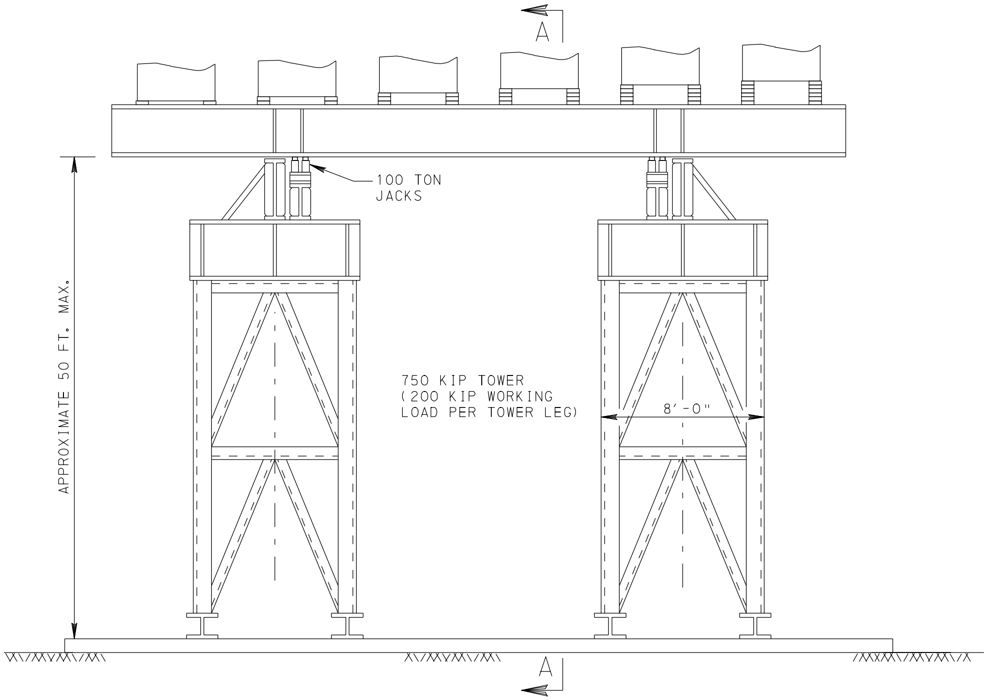


PLAN

GENERAL REFERENCE ONLY
NOT FOR CONSTRUCTION

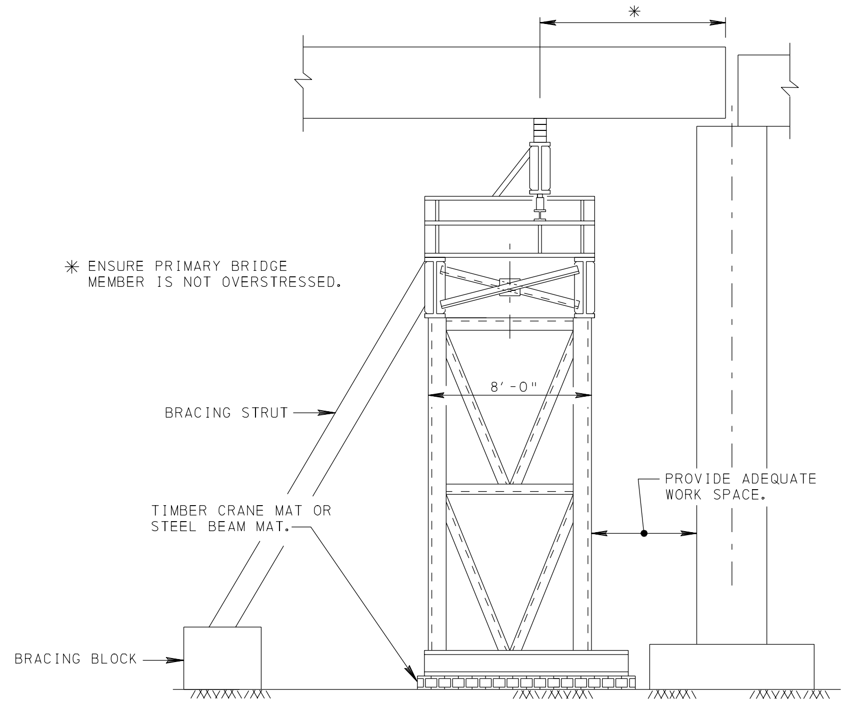
TEMPORARY SHORING

PARTIALLY SUPPORTED SUPERSTRUCTURE

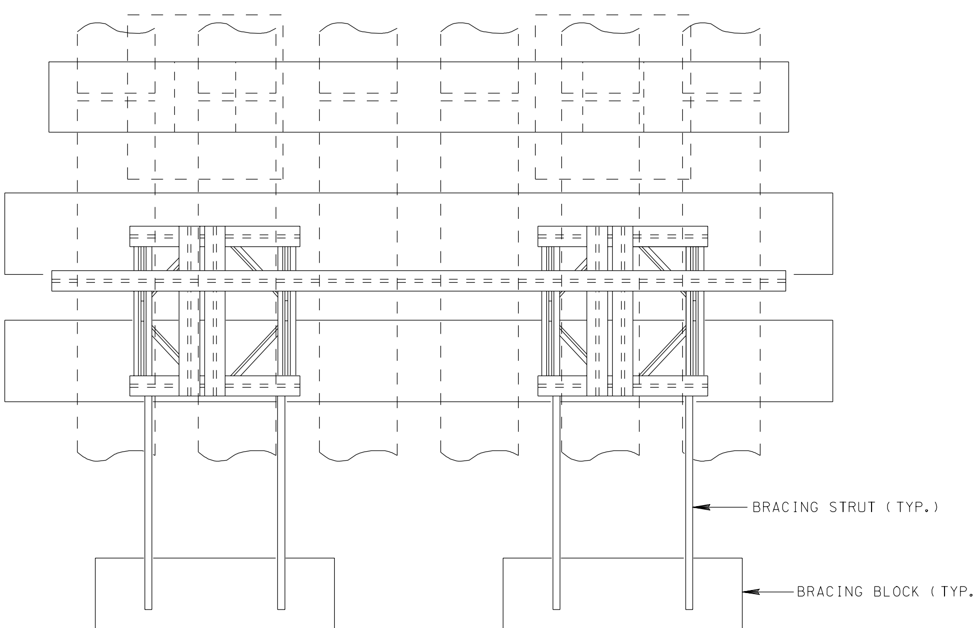


ELEVATION

STRUT BRACING NOT SHOWN FOR CLARITY



SECTION A-A



PLAN

GUIDANCE NOTES:

1. SEE GUIDANCE NOTES 1 & 2, AND 4 THRU 7 ON SHEET 1.
2. DETERMINE ANTICIPATED DEAD, LIVE LOADS, AND WIND LOADS APPLIED TO TEMPORARY BENT, PROVIDE STRUCTURALLY ADEQUATE MEMBERS/TOWERS AND BRACING. USE OF THE WORKING STRESS DESIGN METHOD IS ACCEPTABLE.
3. WIND FORCES INCLUDE WIND EFFECT ON SUPER-STRUCTURE.

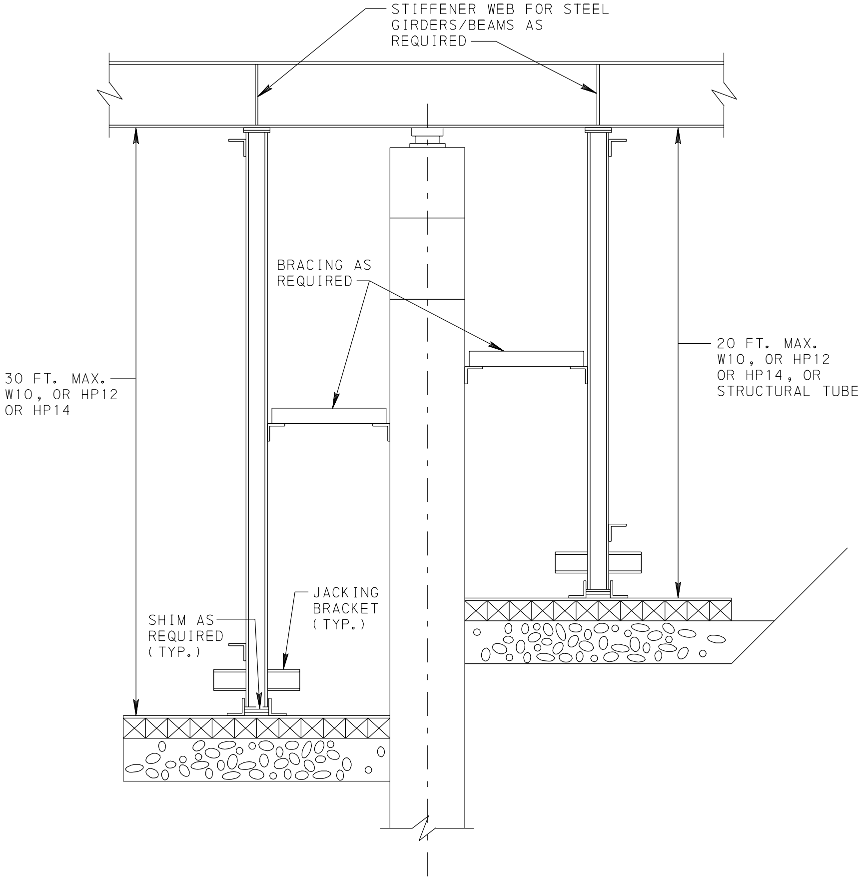
SUGGESTED CONSTRUCTION NOTES:

1. SEE SHEET 1 FOR SUGGESTED CONSTRUCTION NOTES.

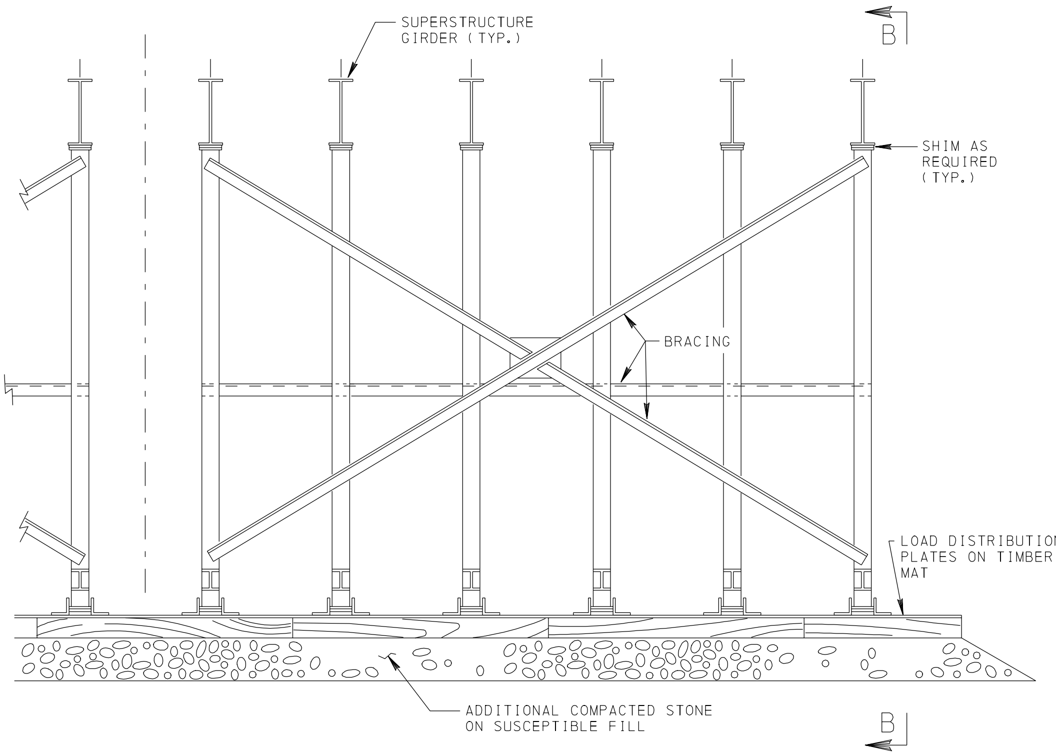
GENERAL REFERENCE ONLY
NOT FOR CONSTRUCTION

TEMPORARY SHORING

FULLY SUPPORTED SUPERSTRUCTURE



VIEW B-B



ELEVATION

NOTES:

1. FOR GUIDANCE NOTES SEE SHEET 1.
2. FOR SUGGESTED CONSTRUCTION NOTES SEE SHEET 1.

GENERAL REFERENCE ONLY
NOT FOR CONSTRUCTION

TEMPORARY SHORING

PARTIALLY SUPPORTED SUPERSTRUCTURE
ALTERNATE SHORING METHOD