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Chapter 16
BRIDGE FOUNDATIONS

16.1 GENERAL

16.1.1 Overview

The satisfactory performance of a bridge structure depends on the proper selection and design of foundations used to support the bridge. This Chapter discusses MDT-specific criteria for the geotechnical design of bridge foundations. These criteria generally follow methods given in Section 3 *Loads and Load Factors* and in Section 10 *Foundations* of the *AASHTO LRFD Bridge Design Specifications*. The *MDT Structures Manual*, which is the responsibility of the Bridge Bureau, discusses the structural design of foundations. The *MDT Hydraulics Manual*, which is the responsibility of the Hydraulics Section, discusses the evaluation of hydraulic scour for bridge foundations.

The function of the bridge foundation is to support loads from the bridge superstructure by 1) spreading concentrated loads over a sufficient area to provide adequate bearing capacity and to limit settlement under the imposed load, or 2) transferring loads through unsuitable foundation strata to suitable strata. Knowledge of the loading conditions, environmental and climatic effects over the life of the structure, plus an understanding of subsurface soil conditions, location and quality of rock, groundwater conditions, local construction practices, and scour and frost effects is necessary to choose the most appropriate foundation type and size.

16.1.2 Responsibilities

Bridge foundation design involves close coordination between the Bridge Bureau and the Geotechnical Section. Where river or stream crossings occur, the Hydraulic Section will also have a key role in the overall design process. The coordination between these Units is summarized in the following Sections.

16.1.2.1 Geotechnical Section/Bridge Bureau Coordination

The Geotechnical Section and Bridge Bureau coordinate both the selection and design of bridge foundations as summarized below. Section 16.2.2 provides additional discussion of this design process. The sequence of coordination and design includes:

1. Geotechnical Information. The project geotechnical specialist prepares a Geotechnical Report for all new bridge projects summarizing the geotechnical information for the site. The Report presents the soil and rock types, bearing capacities and foundation recommendations based on subsurface investigations, lab testing and analyses that have been carried out for the project. The Bridge Bureau comments on the Geotechnical Report and works with the Geotechnical Section to resolve any questions or conflicts.

The bridge designer incorporates the relevant geotechnical information into the bridge design plans. More specifically, the bridge designer prepares the Log of Borings Detail for the Footing Plan Sheet. The information from the Geotechnical Report is transferred to the Footing Plan Sheet in a form suitable for the construction plans. See the *MDT Structures Manual* for additional guidance. The information includes the borehole location and number, elevations and number of blows from Standard Penetration Tests (SPTs). A full set of the log of borings is included in the special provisions of the contract documents.

2. Foundation Type and Design. The foundation type may be spread footings, driven piles or drilled shafts. The selection of the foundation type is a collaborative effort between the Bridge Bureau and Geotechnical Section based on the Geotechnical Report, expected superstructure type, scour potential and other design issues. The Bridge Bureau provides the Geotechnical Section with the applicable loads and a preliminary bridge foundation type and layout. In summary, the coordination works as follows:

   a. Spread Footings. The Geotechnical Section is responsible for determining if the use of spread footings is appropriate. The project geotechnical specialist provides the Bridge Bureau with the appropriate resistance factors for each limit state, nominal footing bearing capacity, settlements for the Service Limit State and base footing elevations.

   b. Driven Piles. The Geotechnical Section is responsible for recommending the use of piles and selecting the pile type. The project geotechnical specialist provides the Bridge Bureau with the appropriate resistance factors, nominal axial pile capacities, lateral deflections for the given lateral loads, settlements for the Service Limit State, design pile toe elevations and required capacity during driving.

   c. Drilled Shafts. Where proposed by the Bridge Bureau, the Geotechnical Section evaluates the use of drilled shafts and provides the Bridge Bureau with resistance factors for each limit state, nominal axial capacities for different shaft diameters, settlement for the Service Limit State, lateral load versus deflections, and shaft length and diameter.

   After the foundation type and basic dimensions are selected, the Bridge Bureau performs the structural design of the foundation.

3. Scour. For hydraulic scour evaluations at existing bridges, the Hydraulics Section, Geotechnical Section and Bridge Bureau participate as an interdisciplinary team to evaluate the existing foundation design and to determine if any corrective actions are warranted.

16.1.2.2 Geotechnical Section/Hydraulics Section Coordination

For bridge crossings over water, the Geotechnical Section will coordinate with the Hydraulics Section to determine realistic scour depths for foundations. The Geotechnical Section characterizes the site conditions including the soil type, soil gradation, rock type, depth to rock and competency of bedrock. The Hydraulics Section evaluates the scour potential based on the information provided by the Geotechnical Section and estimates potential scour depths. Based on the Hydraulics Section evaluation, the Geotechnical Section may present recommendations in the Geotechnical Report pertaining to scour.

If the Hydraulics Section determines that the supporting foundation elements are exposed to stream flow from pier and contraction scour, then a redesign of the foundation may be required. When a redesign of the foundation is required, the Bridge Bureau should resubmit the redesign information (e.g., new foundation layout, sizes, foundation load combinations) to both the Hydraulics Section and the Geotechnical Section. The Geotechnical Section will analyze the new foundation and resubmit the necessary geotechnical information to the Bridge Bureau. The Hydraulics Section will analyze the new foundation and will confirm that the new design is within acceptable limits of general and contraction scour.

For projects where scour protection is required along the river or stream banks or around in-water foundations, the project geotechnical specialist works closely with the Hydraulics Section to identify the most appropriate scour protection system. The type of system can range from use of rip rap and quarry stone to various types of pre-manufactured scour protective systems. These pre-manufactured systems can include geosynthetic products, flexible concrete mat systems and rock-filled gabions. The project geotechnical specialist will often establish the bedding and filter requirements for these systems.

A common practice by MDT includes placing topsoil on top of rip rap to help establish growth of vegetation. The project geotechnical specialist should evaluate this procedure with respect to the potential of topsoil placement impeding drainage of the soil below the riprap and resulting slope stability. Where drainage of soil below the riprap is critical to slope stability, the project geotechnical specialist should recommend against this topsoil placement and document this recommendation in the Geotechnical Report. These determinations are usually evaluated and recommended on a case-by-case basis.

16.1.3 References

For further guidance on the design of bridge foundations, consider the following references:

- AASHTO *LRFD Bridge Design Specifications*;
- FHWA *Design and Construction of Driven Pile Foundations*, FHWA-HI-97-03;
- FHWA Geotechnical Engineering Circular #3, Design Guidance: *Geotechnical Earthquake Engineering*, FHWA-SA-97-077;
- FHWA *Load and Resistance Factor Design (LRFD) for Highway Bridge Substructures*, FHWA HI-98-032;
- FHWA *Drilled Shafts: Construction Procedures and Design Methods*, FHWA-IF-99-025;
- FHWA *Shallow Foundations Reference Manual*, FHWA-NHI-01-023;
- FHWA *Subsurface Investigations – Geotechnical Site Characterization*, NHI-01-031;

- FHWA *Geotechnical Circular #6, Shallow Foundations*, FHWA-SA-02-0540;
- *Design of Pile Foundations*, EM 1110-2-2906, Corps of Engineers;
- NCHRP 343 *Manuals for Design of Bridge Foundations*, Transportation Research Board;
- *Canadian Foundation Engineering Manual*;
- *Navy Foundations and Earth Structures* 7.2, Department of the Navy, Naval Facilities Engineering Command; and
- *Rock Foundations*, EM 1110-1-2908, Corps of Engineers, 1994.

16.2 GENERAL FOUNDATION DESIGN CONSIDERATIONS

16.2.1 General

This Section provides an overview of general foundation design considerations. The overview covers the design process, foundation selection, a review of LRFD design principles and the geotechnical characterization of the site. Specific design procedures for spread footings, driven piles and drilled shafts are presented in Sections 16.3, 16.4 and 16.5, respectively.

- The discussion of design process and foundation selection are provided to give an overall description of the interactive process that occurs during design, particularly with the Bridge Bureau, and to summarize some of the considerations when selecting foundations for bridges.

- The review of LRFD design principles is provided to explain the background for the change in design approach that was implemented within AASHTO. Where preliminary engineering (PFR) for a project was initiated prior to October 1, 2007, either the Allowable Stress Design (ASD) or the Load and Resistance Factor Design (LRFD) methodology presented in the *AASHTO LRFD Bridge Design Specifications* are acceptable methods of design. Use the LRFD methodology for projects where preliminary engineering was initiated after October 1, 2007.

- Specific investigation requirements for the three primary types of foundations (i.e., spread footings, driven piles and drilled shafts) are summarized in Sections 16.3.1, 16.4.1 and 16.5.1, respectively. These specific requirements supplement guidance provided in Chapter 8 on methodologies used to characterize subsurface conditions at a project site. Before implementing the specific requirements, review key elements and requirements related to geotechnical site characterization in Chapter 8.

16.2.2 Foundation Design Process

Chapter 4 discusses the Geotechnical Section's role in the overall design process. A complete discussion on the project development process (OPX2) can be found on the Department's intranet.

16.2.2.1 Field Investigations and Preliminary Foundation Recommendations

The design process is initiated with the Preliminary Field Review (PFR). The Bridge Bureau will request a geotechnical field investigation (Bridge Activity 568) and foundation report for the superstructure. The bridge engineer provides the Geotechnical Section with the approximate location of the foundations and may provide initial estimates of axial service loads for the superstructure based upon assumed foundation types and sizes. The project geotechnical specialist uses this information to develop a scope for the subsurface investigation (Geotechnical Activity 462) and provide a subsurface investigation request to the Field Investigation Unit supervisor.

Upon completion of the field investigation and preliminary geotechnical analyses, the Bridge Design Parameters Meeting is conducted. This meeting is attended by representatives of the

Bridge Bureau, the project geotechnical specialist, Hydraulics Section and Environmental Services Bureau and any others identified by the Bridge Bureau. Possible foundation types and other design items (e.g., scour depths, PDA testing, static load testing, environmental concerns) are discussed at this meeting. Preliminary foundation recommendations (including foundation type) provided at this meeting by the project geotechnical specialist can be modified as additional information or analytical results become available. Informal discussions or meetings between the bridge engineer and the project geotechnical specialist are common and should be performed to address foundation-related issues.

16.2.2.2 Geotechnical Engineering and Final Geotechnical Report

After the Design Parameters Meeting is conducted, the bridge engineer will develop a structural model based on preliminary foundation types, and then determine the loads at the top of pile, top of footing or other locations as agreed upon with the Geotechnical Section (Bridge Activity 560).

The project geotechnical specialist will analyze the foundation with these loads and submit a geotechnical engineering report to the bridge engineer (Geotechnical Activity 466), including special provisions. The report may include the following:

- For deep foundations, this report may include the diameter and tip elevation of drilled shafts, the pile size and tip elevation of driven piles, and an LPILE file containing soil and foundation information.
- For spread footings, the project geotechnical specialist may supply the dimensions of the foundation for global (or external) stability.

The information exchanged between the Geotechnical Section and the Bridge Bureau will vary depending on the type of foundation chosen.

The bridge engineer may use the information to verify and refine the structural model, examine predicted foundation deflections and determine the behavior of the structure under different load conditions (Bridge Activity 578). If the final design loads are greater than the initial loads provided to the project geotechnical specialist, a discussion should take place between the bridge engineer and the project geotechnical specialist to determine the necessity and advisability for further foundation analysis. If necessary, a supplemental geotechnical engineering report (Geotechnical Activity 468) containing any revisions to the 466 Activity Report will be submitted to the Bridge Bureau.

16.2.3 <u>Selection of Foundation Type</u>

The selection of the foundation type involves a number of different factors, ranging from soil conditions to construction costs. On some projects, the selection process will be relatively straightforward for the particular geology and bridge location. However, other times secondary factors (e.g., environment conditions, construction schedule) need to be considered during the selection process. The following Sections summarize the Geotechnical Section's typical practice for selecting foundation types. Where the foundation selection process is not obvious, it will usually be desirable to discuss the alternatives with others in the Geotechnical Section, Bridge Bureau and Construction Engineering Services Bureau.

16.2.3.1 Spread Footings

Spread footing foundations used by MDT for bridge foundations consist of a reinforced concrete slab bearing directly on the founding stratum. The geometry of the concrete slab is determined by structural requirements and the characteristics of supporting components (e.g., soil or rock).

Spread footings are normally used where the bearing capacity is high and settlements will be small (e.g., till materials, rock). Competent material must be near the ground surface (i.e., typically less than 10 ft (3 m) below the ground line) to avoid large excavations, shoring systems and other related construction issues. Spread footings are typically not recommended within rivers or stream crossings because of scour susceptibility and environmental restrictions that may occur during construction. Spread footings are also generally not used on embankment fills unless the proposed fill material has been specified for soil classification, gradation, compaction, etc.; the foundation soils below the embankment are not expected to settle; or the predicted settlement has been mitigated.

The spread footing's advantage is its simplicity in design and construction. Special construction equipment is typically not required; consequently, more contractors can bid this type of construction. The primary disadvantage of the spread footing is that it requires a considerably larger construction area than a pile or drilled shaft foundation. If there is adequate available space at the foundation location, this may not be an issue. However, if the footing is being constructed as part of a retrofit or new bridge along an existing highway, space requirements may preclude efficient use of spread footing foundations.

An important consideration when selecting spread footing foundations is the settlement that will occur under the bridge load. Settlement criteria for the spread footing need to be consistent with the function and type of structure, anticipated service life and consequences of unanticipated movements on service performance. Typically, the settlement for individual footings should be less than 1 in (25 mm). By limiting settlement to 1 in (25 mm), differential settlement will usually be acceptable. Larger settlements may be acceptable if the differential movement between footings is limited. In general, longitudinal angular distortions between adjacent spread footings should not be greater than 0.008 radians in simple spans and 0.004 radians in continuous spans. The level of total settlements should be discussed with the Bridge Bureau wherever the total settlements are estimated to be in excess of 1 in (25 mm) to determine if the settlements are tolerable for the specific application, alternative foundation systems are necessary or ground improvement methods should be considered.

Ground modification techniques may be used to improve poor soil conditions, thereby allowing the use of spread footings where they would not otherwise be appropriate. These techniques are typically used to reduce differential settlement or to avoid potential liquefaction problems. There are a variety of ground modification and soil improvement techniques available. Examples of some methods common in the US include construction of columns of gravel in the ground called stone columns or compaction grouting through the pressure injection of a slow-flowing water/sand/cement mix into a granular soil. Because of the limited number of local contractors specializing in this type of work, coupled with the often relatively remote locations in

Montana, these techniques are often not economical. Nevertheless, a preliminary evaluation of ground modification is often desirable, recognizing that economics may require the use of a deep foundation system.

16.2.3.2 Driven Piles

Driven piles are used where the underlying soils cannot provide adequate bearing capacity or where predicted settlements are excessive for a spread footing based upon the anticipated loading conditions. For these locations, driven piles are used to transfer loads to deeper suitable strata through friction and end bearing. Driven piles are also used where the anticipated depth of scour is excessive. MDT typically uses steel pipe piles and H-piles.

The selected type of pile is determined by the required bearing capacity, length, soil conditions and economic considerations. Piles are typically driven with a single-acting diesel impact hammer. The installation method will depend on subsurface conditions, the pile size and any environmental constraints, which could set limits on permissible vibrations and noise. A significant benefit of the driven pile is that it can be monitored during installation, thereby providing the project geotechnical specialist some confidence that the pile capacity criteria are being satisfied. Field monitoring also allows use of higher resistance factors as summarized in the *AASHTO LRFD Specifications*.

Consider installation factors when evaluating foundation types. For example, pile-driving problems will occur if rock is shallow, in which case the limited depth of penetration may not be sufficient to achieve lateral stability. Damage can also occur to the toe of piles when driving through soil with cobbles and boulders. The cost of predrilling to meet toe elevation requirements for driven piles is often high.

The following items based on MDT's experience should be considered in the selection of driven piles:

- Driven piles are used beneath most abutments and sometimes to support taller retaining walls.
- Footings on piles are used for shorter spans (e.g., grade separations) where scour is not a concern.
- Driven pile foundations are generally less expensive than drilled shafts.
- Pipe piles range in diameter from 14 in to 30 in (356 mm to 762 mm). H-piles will vary from 12 in to 14 in (305 mm to 356 mm).
- The *MDT Standard Specifications* require pipe piles to be filled with concrete. No reinforcement is used, and the concrete is ignored when estimating flexural stiffness.
- Monotube piles are used infrequently in soft ground situations. Prestressed concrete piles are not usually used because of limited fabrication capabilities and transportation constraints.

- H-piles usually have better drivability than pipe piles; however, pipe piles have better lateral load capabilities and are easier to design for bridges with high-skew angles.

There are a number of disadvantages associated with driven piles that must also be considered during the pile selection process, including:

1. Number of Piles. Column loads may require 10 or more driven piles compared to a single drilled shaft.

2. Pile Cap. The space requirements associated with the pile cap may be a serious constraint in some locations. If the groundwater table is high, construction of the pile cap may also require special dewatering work.

3. Pile Toe Elevations. There may be difficulty in achieving the required pile toe elevations in some soils. This situation occurs if a dense surface layer overlies softer material, and the piles have to penetrate the dense soils to penetrate the softer layer. Predrilling or other special construction methods may be required.

4. Noise and Vibrations. Noise and vibrations associated with pile driving can be a significant issue in environmentally sensitive areas, urban locations and near or on potentially unstable slopes.

16.2.3.3 Drilled Shafts

Drilled shaft foundations are constructed by excavating a hole with drilling equipment and placing concrete with reinforcing steel in the excavation. Casing, drilling slurry or both may be necessary to keep the excavation stable. The size of the drilled shaft foundation typically ranges from 3 ft (900 mm) to 10 ft (3000 mm) in diameter. The length of drilled shafts can be up to 200 ft (60 m), although lengths over 100 ft (30 m) require special drilling equipment.

Drilled shaft foundations are selected where one or more of the following conditions apply:

- significant scour is expected,
- there are limits on in-stream work or tight construction zones,
- bridge spans are long and heavy loads are involved,
- depth to rock is shallow,
- earthquake loads are high, or
- driven piles are not economically viable due to high loads or obstructions to driving.

Limitations on vibration or construction noise may also dictate the selection of the drilled shaft foundation. Drilled shafts are typically the most costly foundation alternative relative to spread footings and driven piles.

Consider the following factors before specifying a drilled shaft foundation:

1. Usage. Drilled shafts may be an economical alternative to driven piles. Consider using drilled shafts to resist large axial and lateral loads where deformation tolerances are relatively small. Drilled shafts derive load resistance either as end-bearing shafts transferring load by toe resistance or as friction shafts transferring load by side

resistance. A temporary surface casing extending 10 ft to 15 ft (3 m to 5 m) below the ground surface should be planned for each shaft project. Full depth temporary casing may also be required by special provisions for a specific project.

2. Socketed Shafts. Where casing through overburden soils is required, design the shaft as one size and do not step down when going into formation material.

3. Soil Type. The type of soil and whether the excavation requires casing or special drilling fluids to maintain hole stability will be a factor when evaluating the feasibility of drilled shafts. This is particularly an issue in open gravels or granular materials with significant number of cobbles. Equipment is available (e.g., German Liebherr oscillator or rotator system) that use the casing as part of the drilling system; however, the unit cost for this type of equipment is usually much higher than a more conventional auger rig.

4. Quality Assurance. The quality assurance program is critical for drilled shafts. Fewer drilled shafts are used relative to an equivalent pile foundation; consequently, there is less redundancy in a drilled shaft foundation. Typical practice involves conducting sonic logging surveys of the shaft after the shaft has been installed. If low velocity zones are identified in the shaft, repair or replacement of the shaft may be required.

5. Available Contractors. Fewer contractors have the equipment and experience necessary to install drilled shafts, particularly for large diameter shafts (e.g., 8 ft (2.4 m) and larger) and for shafts that exceed 100 ft (30 m) in length.

6. Capacity Uncertainty. There may be greater uncertainty in the analytical methods used to predict drilled shaft capacity. For sites with well-established bearing materials, this is not a limitation. However, for sites that have less competent materials, there is greater uncertainty in the capacity predictive equations, and this uncertainty can result in greater conservatism to meet load requirements.

Recent work in the area of pressure grouting of the base of the drilled shafts appears to offer some significant capacity benefits in certain soils. Where analyses suggest that capacities may be difficult to reach within reasonable depths, consideration should be given to this alternative. FHWA has participated in research in this area and should be contacted regarding the current status of this work.

16.2.3.4 Alternative Piles

Alternative foundation types are not used commonly by MDT; however, there may be projects that alternative foundations are preferable and cost-effective. Two alternatives that have been used in other locales of the U.S. include the following:

1. Micropiles. Consider using the micropile for seismic retrofitting of foundations. One of the advantages of this pile type is that it has low overhead clearance requirements. Micropiles have also been used for foundation mitigation where scour has undermined bridge foundations.

2. Auger Cast Piles. Auger cast piles (ACP) have not been used to date by MDT. ACP piles are lower in cost than the drilled shaft, because diameters are typically smaller and

the reinforcement is reduced below the upper 10 pile diameters. FHWA Geotechnical Circular No. 8, *Design and Construction of Continuous Flight Auger Piles* documents appropriate design and construction methods of this type of pile.

16.2.4 Foundation Design Using LRFD Principles

The former process for designing foundations used the Allowable Stress Design (ASD) methodology. With the adoption of Load and Resistance Factor Design (LRFD), the approach used for the design of spread footings, pile foundations and drilled shafts has changed. This design approach is summarized in the current version of the *AASHTO LRFD Bridge Design Specifications*. The following Sections provide an overview of pertinent information for LRFD bridge foundation design.

16.2.4.1 Philosophy and History for LRFD Resistance and Load Factors

The *AASHTO LRFD Specifications* introduced a major change when compared to the traditional principles of the *AASHTO Standard Specifications for Highway Bridges* by quantifying the uncertainties in strength and loads that develop in the soil and structure. The uncertainties are quantified by use of load factors to account for the uncertainty in load and resistance factors to account for uncertainties in resistance. The combination of load factor and resistance factor is intended to give a margin of safety between the loads on and the strength of bridge components, thereby achieving the same objectives as ASD. However, by imposing separate load and resistance factors to structural and geotechnical components, a clearer understanding is obtained on the treatment of uncertainty.

The distinction between load and resistance has also led to the use of the terms “demand” and “capacity” when evaluating the performance of the bridge foundations. Demand refers to the loads that are imposed on the structure after the loads are multiplied by an appropriate load factor. These load factors change, depending on the type of load. The capacity refers to the strength of the system, which is defined by the nominal (ultimate) capacity multiplied by a resistance factor. The resistance factor for foundations can range from less than 0.4 to 1.0, depending on the type of method used to determine the capacity. The project geotechnical specialist is usually interested in the capacity to demand ratio (C/D), which is equivalent to the factor of safety for ASD. Note that the nominal capacity is the best estimate of capacity. It should not be a lower bound or “conservative” estimate of capacity. The intent of the resistance factor is to account for uncertainties.

Load and resistance factors for geotechnical design are found in Sections 3 *Loads and Load Factors* and Section 10 *Foundations* of the *AASHTO LRFD Specifications*. These factors originally were selected to achieve a level of safety comparable to the safety factor in the AASHTO *Standard Specifications*. In other words, a foundation designed for the load and resistance factors identified in the *AASHTO LRFD Specifications* would give the same ratio of capacity to demand as the total factor of safety used in the ASD. More recently, efforts have been made to apply reliability methods to the determination of resistance factors appropriate for foundation design, as summarized in the *AASHTO LRFD Specifications*.

16.2.4.2 ASD versus LRFD

In ASD, all loads are combined and considered Service loads. Allowable loads were determined by dividing ultimate capacities by safety factors.

$$\sum Q \leq R_n / FS \qquad \text{(Equation 16.2-1)}$$

Where:

R_n = nominal (ultimate) resistance
FS = factor of safety
ΣQ = summation of force effects

The factor of safety in ASD was determined on the basis of experience considering variables (e.g., type of analysis, type of soil, method of field investigation, performance testing, consequences of failure). Considerable judgment was involved; however, the geotechnical profession had generally reached consensus on what factors of safety to use.

In comparison, LRFD attempts to be much more specific by clearly defining resistance factors based on the strength of material, the variability in resistance and the confidence in the predictive method. Likewise, load factors were selected to account for normal variations in load considering the likelihood of loading. The intent of the LRFD approach is to provide a measure of safety related to the probability of failure. The *AASHTO LRFD Specifications* states that the ultimate resistance (R_n) multiplied by a resistance factor (ϕ) must be greater than or equal to the summation of loads (Q_i) multiplied by corresponding load factor:

$$\sum \eta_i \gamma_i Q_i \leq \phi R_n \qquad \text{(Equation 16.2-2)}$$

Where:

γ_I = load factor generally greater than 1.0

Q_i = load or force effect

ϕ = resistance factor generally less than 1.0 (equal to 1.0 for Service and Extreme Event limit states)

R_n = Nominal (ultimate) resistance

η_i = load modifier as defined in the *AASHTO LRFD Specifications*

The left-hand side of Equation 16.2-2 is the sum of the factored load (force) effects acting on a component; the right-hand side is the factored nominal resistance of the component. Equation 16.2-2 must be considered for all applicable limit state load combinations. The equation is applicable to both superstructures and substructures. The Bridge Bureau uses η_i values of 1.00 for all limit states, because bridges designed in accordance with the *MDT Structures Manual* will demonstrate traditional levels of redundancy and ductility. Rather than penalize less redundant or less ductile bridges, these bridge designs are not encouraged. The Bridge Bureau may on a case-by-case basis designate a bridge to be of special operational importance and specify an appropriate value of η_i.

Typically, the Bridge Bureau prescribes the loads and the Geotechnical Section determines the resistance for foundations. On the resistance side, the project geotechnical specialist has the opportunity to control the determination of resistance factors, through the selection of sampling and testing methods, the procedures used for design and the methods used to monitor the construction process.

16.2.4.3 Limit States in LRFD

The LRFD approach involves three categories relative to the geotechnical design of foundations. These categories are referred to as limit states. The limit state defines a load condition beyond which the element ceases to satisfy the provisions for which it was designed. The three categories are discussed below:

1. Service Limit State. This limit state ensures satisfactory performance for loads that occur on a daily basis. These loads are not modified by load factors, and the soil capacity is not modified by resistance factors. Foundation design at the Service Limit State includes settlement, lateral displacements and bearing resistance using presumptive bearing pressure.

2. Strength Limit State. This limit state ensures that stability, both local and global, is provided under statistically significant load combinations that a bridge is expected to experience in its design life. Extensive distress and structural damage may occur under the Strength Limit State, but overall structural integrity is expected. Foundation design at the Strength Limit State includes axial pile capacity, spread footing bearing capacity, external stability of foundations (e.g., excessive loss of contact, sliding at the base of footing, overturning) and global slope stability. Both load and resistance factors are used in this limit state.

3. Extreme Event Limit State. This limit state ensures the structural survival of a bridge during a major earthquake, flood or when impacted by a vessel, vehicle or ice flow, possibly under scoured conditions. These loads are considered unique occurrences whose return period may be significantly greater than the design life of the bridge. Because of the infrequency of the extreme loads, the load factors for this limit state are usually lower than those used for the Strength Limit State, and the resistance factors are higher. For example, the load factor and resistance factor for seismic loading are usually assumed to be 1.0, because of the low likelihood of this type of loading.

16.2.4.4 Advantages and Disadvantages of LRFD Approaches

As noted in Section 16.2.4.1, the intent of the LRFD approach is to treat uncertainty in a more explicit manner than occurred in ASD. The advantages and limitations are discussed below:

1. ASD Limitations. Before listing the advantages and disadvantages of the LRFD approach, it is important to list the limitations of the ASD method, as these were the primary reasons for abandoning ASD:

- The ASD method does not adequately account for variability of loads and resistances. The factor of safety is applied only to resistance. Loads are considered to be without variation (i.e., deterministic).

- The measure of resistance is in terms of allowable stress rather than allowable resistance.

- Selection of a factor of safety is subjective and does not provide a measure of reliability in terms of probability of failure.

2. <u>LRFD Method</u>. The LRFD approach addresses the fundamental limitations of the ASD approach:

   - The LRFD method accounts for variability in both resistance and load.

   - More uniform levels of safety are achieved based on the strength of soil and rock for different limit states and foundations types.

   - Levels of safety in the superstructure and substructure are more consistent as both are designed using the same loads for predicted or target probabilities of failure.

3. <u>LRFD Limitations</u>. These limitations include:

   - Method for developing and adjusting resistance factors to meet individual situations requires availability of statistical data and probabilistic design methods.

   - Resistance factors vary with design methods and are not consistent.

   - Implementation requires a change in design procedures for engineers accustomed to ASD.

Initially the conversion from ASD to LRFD results in many questions and uncertainties regarding the benefits of this different approach. From a geotechnical standpoint, there will be questions regarding the correctness of resistance factors, and there will be questions regarding the accuracy of the calibration process. However, as more calibrations are performed and the uncertainties of the method are resolved, the project geotechnical specialist should find this approach to be an improvement to ASD. These calibrations will involve collection and review of performance information from field load testing, as well as further analytical studies where the individual contributions to uncertainty in the capacity estimate are quantified.

In cases where the project geotechnical specialist does not feel comfortable with the LRFD approach or during initial designs using the LRFD approach, it will be desirable to perform a check on the design using the ASD method. This check should result in FS values or allowable capacities that are not significantly different from the LRFD results for the same limit state.

16.2.4.5 Foundation Load Determination

Bridge foundation loads are established by the Bridge Bureau. However, the Geotechnical Section plays a key role in the evaluation of loads, particularly seismic earth loads. The following summarize the typical process followed in defining foundation loads:

- The Bridge Bureau will identify the nominal (working) loads for the bridge, as well as the load combination limit states and the load factors to be considered for each limit state. For lateral loading, the Bridge Bureau will identify the range of lateral loads and the fixity at the head of the pile. Typically, no load factor will be used for the Service Limit State. For extreme events (e.g., ice, seismic, scour), the corresponding axial and lateral loads, and their limit states, should be identified as appropriate.

- For seismic analysis, the Geotechnical Section will provide the Bridge Bureau with the site coefficient for the site (i.e., F_a and F_v) based on the Site Class defined in the *AASHTO LRFD Specifications*. The peak ground acceleration (PGA), short period spectral acceleration (S_s) and long period spectral acceleration (S_1) on rock can be obtained from the AASHTO seismic hazard CD or maps, USGS publications and site-specific analysis. The Geotechnical Section should use this peak ground acceleration at the ground surface to estimate the liquefaction potential at the site and the seismic stability of abutment or other slopes that could load the bridge foundations if they were to move. See Chapters 15 and 19 on the design of roadway slopes and embankments, and seismic design, respectively.

- The Geotechnical Section performs the lateral soil-structure interaction analysis with computer programs (e.g., LPILE) to evaluate response of the drilled shaft and pile-supported bridges to Extreme Event I loadings. If soil liquefaction is anticipated, the Geotechnical Section will also provide the Bridge Bureau with foundation downdrag loads due to liquefaction for use in developing the Extreme Event I load combination. The Geotechnical Section will also provide the lateral soil forces that act on the foundation as a result of seismically induced movements of earth retaining structures (e.g., embankments, retaining walls) or lateral soil movements attributable to lateral spread.

- The Bridge Bureau performs the final design of the foundations for the bridge based on input from the Geotechnical Section. If structural members are overstressed or if deflections exceed acceptable limits from any loading combination, the Bridge Bureau will revise the design of the foundation. The revised design may include the adjustment of support member spacing or modification of member sizes. When a revised design of the foundation is required, the Bridge Bureau will resubmit the redesign information (e.g., new foundation layout, sizes, foundation load combinations) to the Geotechnical Section. The Geotechnical Section will analyze the new foundation and resubmit the necessary information to the Bridge Bureau.

16.2.4.6 Tolerable Movements

One of the more difficult tasks of the project geotechnical specialist is quantifying vertical and lateral deformations of the bridge foundations under the loads developed by the bridge engineer. This difficulty is related to the uncertainties of predicting the amount and rate of deformations for foundations supported on soil and the factors that will determine the tolerable movement. These factors can range from the type of bridge superstructure to the type of soil.

Clear guidance on the acceptable vertical and lateral deflections of bridge substructures/ foundations does not exist. In the past, bridge foundations were constructed at sites where deflections were often not an issue. Components used to construct current bridges are generally more flexible than in the past due to advances in design, construction and material technology. In addition, codes have changed from mandatory to optional deflection limits with a trend toward more accurate, optimized design. An acceptable limit for bridge foundation displacement requires engineering judgment based on previous experience, empirical guidelines and structural analyses.

Tolerable deflections are typically based on structural capacity and load-deflection behavior considerations. There is a distinct difference between loads for the Service Limit State and loads for the Strength or Extreme Limit State. LRFD foundation design limits lateral displacements under the Service Limit State, but only requires adequate lateral support under the Strength and Extreme Event Limit State. See the *AASHTO LRFD Bridge Design Specifications* for further discussion of tolerable deformations.

16.2.4.7 Scour Potential at Streams and Rivers

Scour is localized erosion of the channel bed that occurs around flow obstructions (e.g., piers, bridge abutments), at channel contractions (e.g., bridges) and on the outside of channel bends. It can also be the result of long-term erosion of the channel bed that can occur during the life of a structure. Nationally, a number of bridge collapses can be attributed to scour around bridge foundations. In view of the potential implications of scour, the evaluation of scour potential is a particularly important part of the design process for locations where rivers and streams occur.

Scour is a site-specific process that is a function of the flow velocity and duration, the geometry of the structural elements exposed to the flow of water, the geomorphology of the channel and the properties of the foundation and channel bed materials. A multidisciplinary team of hydraulic, geotechnical and structural engineers should evaluate the risk of scour-induced failure at each structure site.

A scour assessment requires a determination of the cumulative effects of the three main components of scour — aggradation/degradation, contraction scour and local (or pier) scour. It also requires an evaluation of potential changes in channel geometry and location that may occur during a structures design life. The amount of scour depends on many factors, including the hydrological characteristics of the site, the hydraulics of the flow and the properties of the streambed materials. Detailed discussion of scour can be found in the FHWA Technical Advisory "Evaluating Scour at Bridges" (1988), FHWA *Riverbed Scour at Bridge Piers* (Copp and Johnson, 1987) and NCHRP Synthesis No. 5, *Scour at Bridge Waterways* (1970).

Use the boring logs to establish the D_{50} values at the streambed surface and to verify depth to bedrock, competency of the bedrock and material conditions. The Hydraulics Section evaluates scour potential based on idealized soil based on the D_{50} of the streambed material.

16.2.5 <u>Geotechnical Characterization of the Site</u>

Adequate subsurface information is required for foundation design. Lack of this information may lead to construction disputes and claims, overly conservative designs with excessively high factors of safety or unsafe designs. Plan subsurface exploration programs to obtain the maximum possible information at minimum cost. A thorough investigation may result in substantial savings in the cost of a foundation in a particular area. In other cases, no amount of detailed information may change the type, cost or performance of the foundation. The project geotechnical specialist must develop a program to obtain an adequate amount of information and data to develop soil parameters for use in design of the foundation system. Soil parameters can be determined from in-situ tests (SPT and CPT), laboratory tests and correlations with index properties. Chapters 8 through 10 provide additional details pertaining to soil testing and measuring soil properties.

16.2.5.1 Soil Profile and Groundwater Conditions

A subsurface profile provides a visual representation of subsurface conditions interpreted from subsurface explorations and laboratory testing. After the soil layer boundaries and descriptions are established, determine the extent and details of any necessary additional laboratory testing (e.g., consolidation and shear strength tests). The final soil profile should include the average physical properties of the soil deposits including unit weight, shear strength and compressibility, as well as, a soil group classification and visual description of each deposit or layer. Also note the observed groundwater level and the presence of other applicable items (e.g., boulders, voids, artesian pressures).

To the extent practical, document the variation in properties within each soil layer. The variation in soil properties should reflect effects of random variation that occurs in natural soils, as well as variations that are introduced through field explorations and laboratory testing. It is often important during the design of foundations to evaluate the effects of these uncertainties on the recommended design. Where significant changes in design recommendation result for small variations in soil property values, it may be necessary to perform additional field explorations and laboratory testing, particularly where these recommendations involve the feasibility or large changes in construction costs.

The soil profile should be characterized at each bridge pier location. Usually, this will require an exploration at each center pier and each end pier. If drilled shafts are used, common practice is to conduct an exploration at each shaft location. Where a drilled shaft foundation is anticipated, it is desirable to leave exploratory borings open for as long as practical to establish whether the hole will stay open or cave. This information is useful in helping to determine if temporary casing will be required during construction or to what elevation the temporary casing will be required.

The following site conditions will warrant special consideration during the field exploration phase:

1. <u>Soft and Compressible Soils</u>. If soils are soft and compressible, it will be important to collect high quality, relatively undisturbed samples for laboratory evaluations of compressibility and strength. This information may be critical for assessing issues

including long-term settlement of spread footings, downdrag on piles and shafts, and settlement of piles and shafts that are not founded in strong-bearing materials.

2. Thin Layers of Soft Soil. Thin layers of soft soil can result in settlement of spread footings or downdrag on shafts and driven piles. As discussed in Chapter 15, these layers can also serve as sliding surface for embankments and slopes, particularly during seismic loading. If slope or embankment movement occurs, spread footings or deep foundations located in the moving soil could be damaged.

3. Liquefiable Soils. Conventional practice is to locate the spread footing below the deepest depth of liquefaction or to improve the ground so that the potential for liquefaction is mitigated. For deep foundations, the toe elevations should be founded below potential liquefiable soils. Liquefaction can result in loss in lateral support within the liquefied zone and downdrag loads on the pile as the liquefied soil settles. The consequences of loss in lateral support or downdrag could be excessive and result in lateral or vertical movement of the foundation system during or following an earthquake. Consider the possibilities of these occurrences during the design.

4. Groundwater. Evaluate the groundwater conditions in the soil borings during the field investigation. When feasible, install and/or monitor piezometers and/or monitoring wells during the various weather and irrigation cycles. To the extent practical, make a determination of all potential groundwater environments beneath the structure (e.g., seasonally high and low groundwater, perched water tables, deep aquifers). Also, evaluate the potential for artesian conditions or cases of excess pore-water pressure as they can reduce the load carrying capacity of the soil and alter the effective stress distribution.

5. Problematic Soils. For guidance on the design and construction on problematic soils, review the *FHWA Shallow Foundations Reference Manual*.

16.2.5.2 Shear Strength of Soil

One of the important steps in the characterization process for bridge foundations is the determination of strength properties used for computing the axial and lateral capacity of spread footings, driven piles and drilled shaft foundations. Many useful correlations have been established between the engineering properties of soils and various indirect and classification properties. For small projects or preliminary studies, correlations are often used extensively. In other cases, these correlations serve as alternative sources of design information or for checks against laboratory or in-situ tests results. References for these correlations can be found in Chapters 9 and 10.

Generally, the type of strength information will depend on whether the foundation is being constructed at a cohesionless soil site or a site characterized by cohesive soils. Consider the following:

1. Cohesionless Soil Site. When determining the strength of cohesionless soils, it is usually necessary to rely on the results of in-situ testing methods. These methods include the SPT and the CPT. Other options for in-situ soil property determination include the pressuremeter, the Iowa borehole shear device and the dilatometer.

Empirical correlations are usually used for estimating the friction angle of cohesionless soils from in-situ soil measurements (e.g., SPT blowcounts, CPT sounding results). Alternatively, methods are available for estimating the nominal capacity of driven piles using SPT blowcounts and CPT sounding data directly. These direct methods can provide a relatively accurate estimate of capacity, as long as the soils at the site are generally consistent with the database used to develop the empirical correlations.

2. Cohesive Soil Site. Either of two methods can be used to determine shear strength at cohesive soil sites. One involves collecting undisturbed samples in the field and then conducting triaxial laboratory tests. Typically, this method is used to obtain both the drained and undrained strength parameters for the soil. The alternative method involves the use of in-situ strength measurements. This approach could involve estimating the undrained strength of the soil from CPT sounding information or using in-situ vane shear measurements. Relationships have also been developed for estimating pile capacity from CPT sounding results. Note that use of the SPT blowcount to estimate undrained shear strength or indirect estimates of pile capacity is subject to large uncertainties and therefore should not be used.

Additional discussions of the determination of soil strength are provided in numerous references.

Determination of shear strength parameters should involve consideration of the type of loading relative to the permeability of the soil. Drained strength parameters are appropriate if the soils will drain quickly (e.g., cohesionless soils). Although drained strength parameters can be obtained by laboratory testing, it is very difficult to obtain undisturbed cohesionless soil samples, forcing the laboratory tests to be conducted on reconstituted samples. The reconstitution process introduces enough uncertainties that the preferred approach is to use indirect correlations between in-situ measurements and soil friction angle.

For cohesive soils, the strength under undrained loading is generally less than the drained strength, particularly if consolidation to a higher stress state occurs. In this case, the strength used in estimating the bearing capacity of spread footings and the axial capacity of piles is generally the undrained strength — consistent with the stress state immediately after the load is applied. The exception to this is for those cases where the long-term drained strength is lower than the short-term undrained strength. In this case, it may be necessary to obtain long-term drained properties. This long-term drained strength can be estimated on the basis of effective stress parameters, while the short-term strength is determined from total stress parameters. Good practice is to check both cases (undrained and drained) to determine which one controls the design.

16.3 SPREAD FOOTINGS

16.3.1 General

Spread footings are used to support structures where suitable soils or rock are located at a relatively shallow depth. Factors affecting the size of the footings are the structural loading versus the ability of the soil or rock to resist the applied loads. Where suitable materials lie below the depth that can be excavated economically or where no firm layers are identified in the subsurface exploration, a deep foundation design will normally be used. Soil investigations for spread footings should generally follow the methods summarized in Chapter 8 and the FHWA *Subsurface Investigations Manual*.

16.3.1.1 Design Requirements

The project geotechnical specialist is responsible for providing recommendations on spread footing designs to the Bridge Bureau. Basic design recommendations include bearing capacity, settlement and sliding resistance for spread footings. Also, provide the following:

1. Soil and Rock Sites. Bearing capacity information should include recommendations for nominal (ultimate) bearing capacity as a function of footing width and depth. Also include the recommended resistance factors.

2. Service Limit. Define the limiting bearing pressure to achieve Service limit requirements. Use a Service Limit State of 1 in (25 mm) unless a higher settlement will be accepted by the Bridge Bureau. For sites comprised of cohesive soil layers, also estimate the rate of settlement.

3. Interface Shear. Determine soil parameters for estimating interface shear at the base of the spread footing.

4. Bearing Elevation. Define the bearing elevation of the footing. This elevation should be determined on the basis of acceptable footing performance under Strength, Service and Extreme Limit States, as well as frost depth and scour considerations. Section 16.3.2 provides specific guidance for each of these requirements.

16.3.1.2 Scour Considerations

Scour can undermine shallow foundations or remove sufficient overburden to redistribute foundation forces causing foundation displacement and detrimental stresses to structural elements. One of the hazards of placing a structure in a river or channel is the potential for scour around the foundations. For new structures, design the foundation deep enough that scour protection is not required. Deep foundation systems may be preferable from both constructability and/or cost perspectives at locations where scour is possible.

For existing structures identified as scour susceptible, scour countermeasures are often required to protect foundations from scour conditions that may not have been identified at the time of design. There are four general types of scour protection:

- localized armoring,
- river training,
- modifications to the foundations, and
- monitoring.

Localized armoring techniques are most commonly used and include the following:

- rock riprap,
- gabions and slope mattress,
- precast concrete blocks, and
- grouted riprap.

Discussion of other armoring systems, including concrete slope pavement, grouted fabric, sand/cement bags and soil cement, and a thorough treatise on current technology for stream instability and bridge scour countermeasures, are provided in HEC 23 *Bridge Scour and Stream Instability Countermeasures Experience, Selection, and Design Guidance*. Any assessment of armoring systems should involve a joint effort of the Hydraulics Section and Geotechnical Section.

16.3.2 <u>Geotechnical Design Considerations</u>

16.3.2.1 Bottom of Footings

The depth of footings should be determined in consideration of the character of the foundation materials and the possibility of undermining. Footings not exposed to the action of stream current should be founded on a firm foundation below the frost level or on a firm foundation that is made frost resistant by over excavation of frost-susceptible material to below the frost line and replaced with material that is not frost susceptible.

In cases where spread footings are being considered for use in streams and rivers, consider the following additional guidelines:

1. <u>Footings on Soil</u>. The bottom of footings on soil should be set at least 10 ft (3.0 m) below the channel bottom and below the total scour depth determined for the 100-year flood, Q_{100}.

2. <u>Footings on Rock</u>. Avoid keying into rock at shallow embedment depths. Keying into rock typically involves blasting or other destructive methods that frequently damages and renders the rock structure more susceptible to scour. If footings on smooth massive rock surfaces require lateral restraint, drill and grout steel dowels into the rock below the footing level. The bottom of the footings should be at least 3 ft (1 m) below the surface of scour-resistant rock with the top of the footings at least below the rock surface.

3. <u>Footings on Erodible Rock</u>. Carefully assess weathered rock or other potentially erodible rock formations for scour. The foundation decision should be based upon an analysis of intact rock cores, including rock quality designations and local geology, hydraulic data, and anticipated structure life. An important consideration may be the existence of a high-quality rock formation below a thin weathered zone. For deep

deposits of weathered rock, estimate the potential scour depth for the design flood and then locate the footing base so that the top of the footing is below the estimated contraction plus local scour. The excavation above the top of the spread footing is usually backfilled with the same material that was excavated.

4. Footings Placed on Tremie Seals and Supported on Soil. The location of the top of the footing to be placed on a seal is determined in the same manner as a footing placed directly on the ground. That is, the bottom of the footing is below the estimated scour depth at the design flood. The elevation at the bottom of the footing is the same as the top of the seal. The required seal depth is then calculated assuming that the Contractor will have to dewater the cofferdam to place the footing "in the dry." The seal mass counteracts the buoyant forces that occur when the cofferdam is dewatered. This depth is typically 40% of the head from the bottom of the seal to the normal water elevation. This 40% is simply the ratio of water unit weight to concrete unit weight. To help accommodate construction uncertainties while locating the cofferdam in the channel, the length and width of the seal are usually required as 3 ft (1 m) greater than the dimensions of the footing. This allows for minor "adjustments," if necessary, to position the footing for the pier correctly.

16.3.2.2 Frost Depth

Freezing of the ground occurs during the winter months in all areas of Montana. Footings must be embedded below the maximum depth of frost penetration (frost depth) or the frost susceptible soils should be replaced by soils that are non-frost susceptible (e.g., free draining gravels) to provide adequate frost protection. This is required to prevent heaving of the footings due to volumetric expansions of the subgrade soils from freezing and/or to prevent settling due to loss of shear strength and stiffness from thawing.

The maximum depth of frost penetration generally is established by local experience or from published maps. The use of generalized maps showing large regions is discouraged. Consult local building codes and experience for appropriate design values.

16.3.2.3 Seismic

Seismic hazards should be assessed as part of the foundation design. Shallow foundations are susceptible to excessive movements or damage if the bearing soils are subject to strong ground motion, liquefaction or lateral spreading due to an earthquake. Review FHWA *Geotechnical Circular #3, Geotechnical Earthquake Engineering for Highways* and Chapter 19 of the *MDT Geotechnical Manual* for guidance in assessing seismic hazards at a site, including evaluation of liquefaction and lateral spreading potential.

In general, shallow foundations will not be an appropriate choice for support of bridge piers where there is potential for liquefaction to develop under the design seismic event unless the liquefaction potential is mitigated by ground improvement.

16.3.2.4 Bearing Capacity

The potential mode of failure (critical failure surface) will be dictated by the subsurface conditions in the vicinity of the footing. Where relatively homogeneous soil conditions exist and extend below the footing, the critical failure surface will likely be relatively circular or log spiral shaped. Where subsurface conditions include a particularly weak zone, weak layer or a shallow sloping rock surface, the critical failure surface will likely be planar. In most cases, analyze both modes of failure to determine the more critical failure mode.

Detailed approaches for calculating the bearing capacity of shallow foundations based on theoretical and empirical formulations are available in foundation engineering text books, design guides and manuals. The following publications are recommended for computing bearing capacities:

- *AASHTO LRFD Bridge Design Specifications*,
- FHWA *Geotechnical Circular #6, Shallow Foundations*, and
- FHWA *Shallow Foundations Reference Manual.*

Other factors that can affect bearing capacity include embankment loading, lateral loading, eccentric or inclined loading, vibratory loading from dynamic live loads or earthquake loads, fluctuations of the groundwater level, proximity to a slope or excavation, layered soil profile and removal of overburden. The recommended publications provide equations and charts for addressing these factors.

For spread footings, provide the factored nominal bearing resistance for a given footing width at the bearing elevation to the Bridge Bureau. *(Note: The nominal bearing resistance is what was traditionally called the ultimate bearing capacity.)* The maximum factored design bearing pressure is shown on the Structural Plans for the footing.

16.3.2.5 Settlement

Estimate footing settlements using deformation analyses based on the results of laboratory testing or in-situ testing. Select the soil parameters used in the analyses to reflect the loading history of the ground, the construction sequence and the effect of soil layering.

Consider both total and differential settlements, including time-dependent effects. Total settlement includes the sum of three separate components — elastic settlement, consolidation settlement and secondary settlement. Detailed approaches for calculating these components are available in many foundation engineering text books, design guides and manuals. The following publications are recommended:

- *AASHTO LRFD Bridge Design Specifications*,
- FHWA *Geotechnical Circular #6, Shallow Foundations*, and
- FHWA *Shallow Foundations Reference Manual.*

Other factors that can effect settlement include embankment loading, lateral loading, eccentric or inclined loading, vibratory loading from dynamic live loads or earthquake loads, fluctuations of the groundwater level, proximity to a slope or excavation, and scour of overburden. The recommended publications provide equations and charts for addressing these factors.

16.3.2.6 Footings on IGMs and Rock

A site investigation is required to confirm the consistency and extent of the Intermediate Geomaterial (IGM) or rock formations beneath a shallow foundation.

The *AASHTO LRFD Specifications* and FHWA *Shallow Foundations Reference Manual* provide presumptive bearing resistance values for spread footings located on various types of rock. The bearing resistance values are settlement based (e.g., 1 in (25 mm)) and apply to the Service Limit State. USACE *Rock Foundations* identifies methods for local and wedge failures associated with brittle and ductile rock. Design properties for these analyses are based on the Rock Mass Rating System (RMR) and the likely failure mechanism.

If a settlement estimate is necessary for shallow foundations supported on IGM or rock, a method based on elasticity theory will generally be the best approach. As with any of the methods for estimating settlement that use elasticity theory, a major limitation is the ability to accurately estimate the modulus parameter(s) required by the method. Consider the following:

- The elastic modulus of IGMs and some rocks can be measured in-situ with the pressuremeter, flat or stepped dilatometer, plate load tests or flat jacks. References for these tests can be found in FHWA *The Pressuremeter Test for Highway Applications* (FHWA–IP-89-008), FHWA *The Flat Dilatometer Test* (FHWA-SA-91-044) or the relevant ASTM standard for each test method.

- Alternatively, tables and charts are available that summarize suggested values for Poisson's ratio and elastic modulus obtained from various published sources (e.g., FHWA *Geotechnical Circular #6, Shallow Foundations*, *AASHTO LRFD Bridge Design Specifications*). Corrections to elasticity formulations can be made for complex geological conditions.

The FHWA *Shallow Foundations Reference Manual* includes discussion of layered and anisotropic rock conditions and methods for correcting elasticity-based deformation calculations. Numerous charts and tables are available in the FHWA *Shallow Foundations Reference Manual* that are useful for estimating rock and rock mass properties, including strength and stiffness parameters.

The *AASHTO LRFD Bridge Design Specifications* provides additional guidance regarding footings on rock.

16.3.2.7 Sliding Stability/Resistance

Sliding failure occurs if the force due to the horizontal component of a load exceeds the more critical of either the factored shear resistance of underlying 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.

In most cases, the movement of the structure and foundation will be small. Consequently, if passive resistance is included in the resistance to sliding, the magnitude of passive pressure is commonly taken as 50% of the maximum theoretical passive pressure. Rough footing bases usually result where footings are cast in-situ. Precast concrete footings may have smooth bases.

AASHTO LRFD Bridge Design Specifications and the *FHWA Shallow Foundations Reference Manual* provide procedures and equations for evaluating the sliding of shallow foundations. In many cases a 6 in to 12 in (150 mm to 300 mm) granular bedding material will be placed on the top of the subgrade. The frictional characteristics of this material will determine the sliding resistance.

Keys in footings to develop passive pressure against sliding are not very effective and their economic justification is often over estimated. However, when it becomes necessary to use a key, the project geotechnical specialist should provide recommendations to the bridge designer during preliminary design.

16.4 DRIVEN PILES

16.4.1 General

16.4.1.1 Pile Type Selection

The selection of a pile foundation type for a structure should be based on the specific soil conditions as well as the foundation loading requirements and final performance criteria. A summary of common pile types is provided in the *FHWA Driven Pile Foundation Manual*. Foundation piles can also be classified in terms of their method of load transfer from the pile to the surrounding soil mass. Load transfer can be by side (or shaft) resistance, toe bearing resistance or a combination of both.

Although one pile type may emerge as the only logical choice for a given set of conditions, more often several different types may meet all the requirements for a particular structure. In these cases, the final choice should be made on the basis of a cost analysis that assesses the over-all cost of alternatives. Also evaluate local practices and availability of materials and experienced contractors. For major projects involving large numbers of piles, consider including alternative foundation designs in the contract documents or performing load tests if there is a potential for cost savings. The *FHWA Driven Pile Foundation Manual* provides guidelines for selecting pile type based on subsurface and hydraulic conditions.

The subsurface investigation for driven pile foundations should generally follow the methods summarized in Chapter 8 and the FHWA *Subsurface Investigations Manual*.

16.4.1.2 Design Requirements

The project geotechnical specialist provides design recommendations to the Bridge Bureau for driven pile foundations where spread footing foundations and drilled shafts are not appropriate. The design recommendations include the pile capacity and size, estimated settlement, lateral response to design loads, and constructability of the pile.

The following specific information is provided to the Bridge Bureau:

- The axial capacity in both compression and uplift (where appropriate) at the design tip elevation. Summarize the appropriate resistance factors for the three limit states and whether PDA tests will be required during construction.
- Settlement estimates if the piles will not be founded in rock or extremely dense soils.
- Electronic files from the lateral response analyses (LPILE).
- Recommendations on pile installation, including pile drivability.
- Design toe elevation and required capacity during driving.

16.4.2 Geotechnical Design Considerations

The design methods of estimating load capacity of a single pile have been studied extensively; however, no universally accepted design method of calculating the load capacity has evolved. Analysis of pile capacity for most practical situations is based on empirical equations, experience and judgment. Consequently, a host of analytical formulations exists in the literature. Methods of evaluating pile-load capacity represent approximations, because it is difficult to fully account for the variability of soil types and the differences in the quality of construction and pile installation. In this section, the analytical methods described in the referenced publications are recommended. Alternative methods are not necessarily discouraged, and in fact, may be applicable in some situations. For example, for special structures and loading conditions (e.g., seismic loads), computer programs based on finite difference or finite element methods are commonly used. To obtain an estimate of the pile-load capacity, recourse is made to engineering mechanics, experience, measured observations and to correlations using laboratory and field test results.

16.4.2.1 Spacing

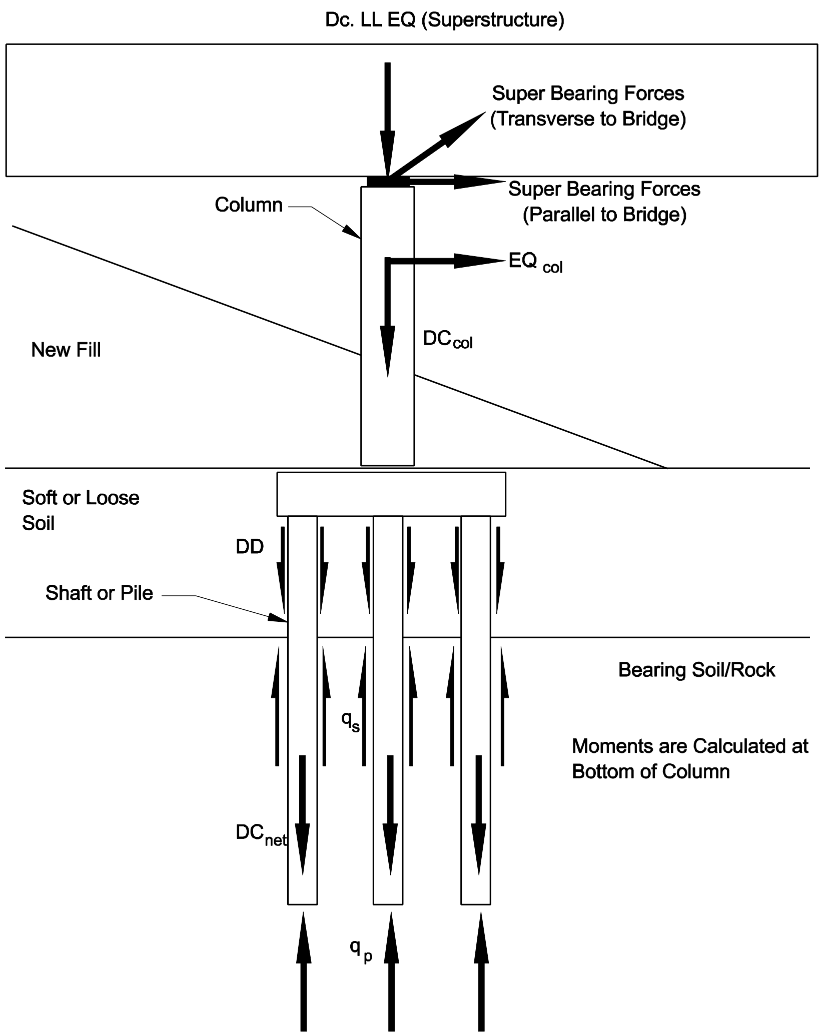
Piles are usually driven at minimum spacings of 3 pile diameters. Closer spacing minimizes the cost of the pile cap. However, driving piles at closer spacing in dense sands and saturated plastic soils can cause heave or lateral ground displacements that may damage or cause misalignment of previously driven piles. Close spacings may be advantageous with loose sands if the loose sands are compacted by driving. The trade-off for this case is that pile driving can become more difficult, requiring careful consideration of the driving sequence.

In determining the spacing of piles, give consideration to the characteristics of the soil and to the length, size, driving tolerance, batter and shape of the piles. If piles are spaced too closely, the axial capacity and lateral resistance of each pile will be reduced. In addition, piles must be spaced to avoid toe interference due to specified driving tolerances. Depending on site conditions and available driving equipment, lateral deviations from the specified location can be as large as 6 in (150 mm) in some cases. A final variation in alignment measured along the longitudinal axis is provided in the *MDT Standard Specifications*.

In general, end-bearing piles should not have center-to-center spacings less than 3 pile diameters. Friction piles, depending on the characteristics of the piles and soil, should have minimum center-to-center spacings of 3 to 5 diameters.

16.4.2.2 Loads and Load Factor Application to Driven Pile Design

The *AASHTO LRFD Specifications* require that load and resistance factors be used to design the piles. Figure 16.4-A provides definitions and typical locations of the forces and moments that act on deep foundations.



Where:

DC_{col} = structure load due to weight of column

EQ_{col} = earthquake inertial force due to weight of column

q_p = ultimate end bearing resistance at base of shaft (unit resistance)

q_s = ultimate side resistance on shaft (unit resistance)

DD = ultimate downdrag load on shaft (total load)

DC_{net} = unit weight of concrete in shaft minus unit weight of soil times the shaft volume below the ground line (may include part of the column if the top of the shaft is deep due to scour or for other reasons)

Figure 16.4-A — DEFINITION AND LOCATION OF FORCES FOR PILE OR SHAFT-SUPPORTED FOOTING

Use the *AASHTO LRFD Bridge Design Specifications* to select load factors for pile foundation analyses and for determining resistance factors for the Strength Limit State. Regionally specific values may be used in lieu of the specified resistance factors, but these should be determined based on substantial statistical data combined with calibration, or substantial successful experience, to justify higher values. Use smaller resistance factors if site or material variability is anticipated to be unusually high or if design assumptions are required that increase performance uncertainty. Design the foundations so that the factored resistance is not less than the factored loads.

16.4.2.3 Factor of Safety

When transitioning from ASD to LRFD design, it will be helpful to check the design conducted according to the *AASHTO LRFD Specifications* against the ASD design. In ASD, the allowable pile resistance (design load) is selected by dividing the ultimate pile capacity by a factor of safety ranging from 2 to 3.5. The range in factor of safety depends on the reliability of the particular static analysis method with consideration of the following items:

- level of confidence of the input parameters,
- variability of the soil and rock,
- proposed pile installation method, and
- level of construction monitoring and construction control.

Experience has shown that construction control methods have a significant influence on pile capacity. In the absence of overriding or extenuating conditions, use the factors of safety presented in Figure 16.4-B. In general, the LRFD design should be consistent with these factors of safety.

Construction Control Method	Factor of Safety
Static load test with wave equation analysis	2.00
Dynamic testing with wave equation analysis	2.25 - 2.5
Indicator piles with wave equation analysis	2.50
Wave equation analysis	2.75 - 3.0
Gates dynamic formula	3.0 - 3.50

Figure 16.4-B — FACTORS OF SAFETY

16.4.2.4 Single-Pile Axial Load Capacity

Static analysis methods are typically used to evaluate pile axial load capacity during the design phase of a project. These analytical methods use soil strength and compressibility properties to determine pile capacity and performance. Guidelines for conducting static capacity analyses of driven pile foundations are provided in the *AASHTO LRFD Bridge Design Specifications*. These guidelines include semi-empirical methods that use estimated or measured soil properties, and methods that directly apply in-situ test results (SPT or CPT measurements). Consider the following:

- Semi-empirical formulations include the use of either total stress (i.e., α-method) or effective stress (i.e., β-method) methods for determining shaft and toe resistances. MDT commonly uses the FHWA computer program DRIVEN to conduct these analyses to determine the toe, shaft and nominal (ultimate) capacities of driven piles. It is useful to determine these capacities versus pile depth.

- In-situ tests are widely used in cohesionless soils because obtaining good quality samples of cohesionless soils is difficult. Two frequently used in-situ test methods for predicting pile capacity are the SPT method and the CPT method. These methods are described in most geotechnical textbooks, manuals and design guides.

A number of methods in addition to those given in *AASHTO LRFD Specifications* are available for estimating pile capacities and may be considered for use on MDT projects on a case-by-case basis.

16.4.2.5 Pile Group Axial Capacity

The axial capacity of a pile group in cohesionless soil is taken as the sum of the resistances of all piles in the group. The group efficiency factor, η, is 1.0 in this case, regardless of whether the pile cap is or is not in contact with the ground.

The efficiency of pile groups in cohesive soil may be diminished from that of the individual pile due to overlapping zones of shear deformation that occur 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. Consider the following:

- If the cap is in firm contact, the soil between the piles and the pile group behaves as a unit. In this case, a block type failure mechanism should be evaluated in addition to evaluating the sum of the individual resistances of each pile in the group.

- 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 should 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; or

    + for intermediate spacings, the value of η may be determined by linear interpolation.

For cohesive soil sites, soil settlement will usually occur after pile installation as excess pore-water from the pile installation dissipates and as secondary compression within the soil occurs. This settlement will often result in separation of the soil from the base of the footing, resulting in the cap no longer being in firm contact with the soil. In view of the potential for settlement, normal design practice is to check group capacity assuming the cap is in contact for initial loading conditions and the cap is not in firm contact with the soil. If the capacity with settlement is lower than the capacity for full contact, the lower of the two capacities is used for conservatism.

16.4.2.6 Settlement

Most settlement analyses for driven pile foundations are based on empirical methods and provide only an approximation of the actual settlement because of the complex load-transfer mechanism that occurs with axially loaded deep foundations. Nonetheless, settlement of single piles and pile groups should be calculated and compared to the performance objectives established for the structure to confirm that calculated settlements are within acceptable limits.

Driven piles are not often used as a single or individual foundation element to support a structure; consequently, settlement analyses of single piles are not commonly conducted. If settlement estimates for a single pile are necessary, the USACE *Design of Pile Foundations Manual* describes elasticity-based methods and the t-z curve method for calculating settlement of single piles. Consider the potential for consolidation-related settlements when making this evaluation.

An empirical approach known as the equivalent footing method is typically used to calculate the settlement of a group of piles. The pile group is treated as an equivalent footing that is founded at an effective depth below the ground surface. For uniform clays sites, the effective depth is two-thirds of the pile embedment in the bearing stratum. For sand sites, the effective depth depends on the soil conditions below the toe of the pile group. After the effective footing depth is defined, procedures for shallow foundation settlement are then applied to the equivalent footing to determine settlement as described in the *AASHTO LRFD Bridge Design Specifications*. Note that the width of the equivalent footing given in *AASHTO LRFD Specifications* changes according to the soil profile.

An alternative approach to the evaluation of settlements for pile groups is to determine the neutral plane for the pile group and to perform the settlement analysis at this location. The neutral plane is determined on the basis of the resistance along the side and at the toe of the pile, as described in the *Canadian Foundation Engineering Manual* (CFEM). This approach can be particularly useful where non-uniform soil layering occurs. The CFEM refers to this method as the unified method. Capacity and settlement are evaluated together with this approach. The UNIPILE computer program discussed in Chapter 13 uses this model to evaluate pile capacity and settlement for pile groups.

16.4.2.7 Downdrag Loads

The current *AASHTO LRFD Specifications* require that the foundation be designed so that the available factored geotechnical resistance is greater than the factored loads applied to the pile, including drag loads (downdrag) at the Strength Limit State. The nominal pile resistance available to support structure loads plus downdrag is estimated by considering only the positive skin and toe resistance below the lowest layer contributing to the downdrag. The structure should also be designed to meet settlement limits resulting from downdrag and the applied loads in accordance with the AASHTO *LRFD Bridge Design Specifications* and the structural limits resulting from the combination of downdrag plus structure loads. The LRFD approach for downdrag is illustrated in Figure 16.4-C.

The nominal bearing resistance of the pile needed to resist the factored loads, including downdrag, is:

$$R_n = (\Sigma\gamma_i Q_i)/\varphi_{dyn} + \gamma_p DD/\varphi_{dyn} \qquad 16.4\text{-}1$$

Where:

R_n = the nominal bearing resistance of the pile needed to resist the factored loads

For allowable stress design, this equation can be re-written using a factor of safety (FS), as:

$$R_n = FS \cdot Q_i + FS \cdot DD = R_{ult} - DD \qquad 16.4\text{-}2$$

The total driving resistance, R_{ndr}, needed to obtain R_n, can be computed using the following equation, which accounts for the skin friction that must be overcome during pile driving that does not contribute to the design resistance of the pile:

$$R_{ndr} = R_{Sdd} + R_n \qquad 16.4\text{-}3$$

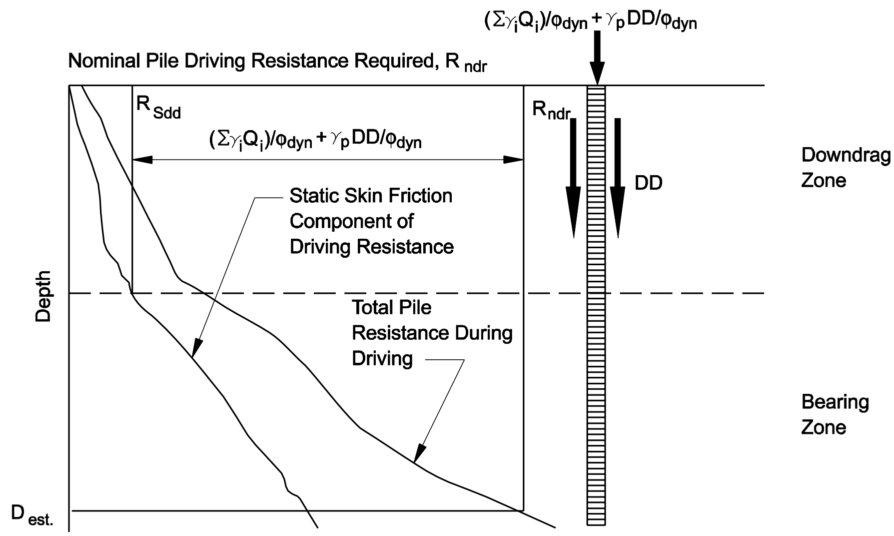
Where:

R_{ndr} = nominal pile driving resistance required. Note that R_{Sdd} remains unfactored in the analysis to determine R_{ndr}

For projects where downdrag loads are having a significant effect on the pile-length determination, the project geotechnical specialist may want to give further consideration to alternatives to the approach given in the *LFRD Specifications*. One option is to conduct a load-displacement analysis using a computer program to evaluate the potential effects of downdrag on the pile using a displacement based approach rather than limit equilibrium method. Computer programs (e.g., APILE) described in Chapter 13 can be used for this evaluation.

16.4.2.8 Piles on Rock

When a rock mass is relatively close to the ground surface and lateral loads are large, it may be necessary to drive steel pipe piles or H-piles into the surface of the rock or to bore the piles some distance into the rock mass. The term “pile drill and socket” applies to a pile that is bored through soil overburden and then drilled into underlying rock a distance of 1 or 2 pile diameters.



R_{Sdd} = skin friction that must be overcome during driving through downdrag zone

Q_p = $(\Sigma\gamma_i Q_i)$ = factored load per pile, excluding downdrag load

DD = downdrag load per pile

$D_{est.}$ = estimated pile length needed to obtain desired nominal resistance per pile

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

Figure 16.4-C — DESIGN OF PILE FOUNDATIONS FOR DOWNDRAG

For pile foundations that are driven to rock, the exact area of contact with rock, the depth of penetration into rock and the quality of rock are largely unknown. Therefore, the determination of load capacity of driven piles on rock should be made based on driving observations, local experience and load tests. The allowable bearing pressure for design of piles on rock will be governed by the rock strength and settlements associated with defects in the rock. For tight joints or joints smaller than a fraction of an inch (mm), rock compressibility is reflected by the rock quality designation (RQD).

Structural capacity typically governs the axial resistance of piles socketed into competent rock. Nevertheless, it is important to check the geotechnical capacity. As a check on the axial capacity of piles driven into rock, *AASHTO LRFD Bridge Design Specifications* provide an approximate empirical method that takes into account the spacing and width of discontinuities within the rock mass and the size and depth of the rock socket. Allowable bearing capacity on unweathered rock should normally be based on the strength of intact rock and on the influence of joints and shear zones.

16.4.2.9 Piles in Intermediate Geomaterials

Deposits of intermediate geomaterials (IGM) exist in many locations in Montana and are commonplace in some areas. These materials exhibit a great variety of physical properties. Consequently, early recognition of the presence of IGM and the need for a pile foundation solution are essential for the planning and execution of an effective site investigation and foundation design.

Various terms have been used to describe or classify an IGM, including weak rock, indurated soil, soft rock and formation material. IGM are intrinsically weak (i.e., they have undergone a limited amount of gravitational compaction and cementation). They are products of the disintegration, weathering and alteration of previously stronger rocks. Classification of IGM based on material properties is not standardized; however, a common definition is that IGM will have a uniaxial compressive strength in the range of 12.5 ksf to 260 ksf (600 kPA to 12,500 kPa) and a stiffness modulus in the range of 2,100 ksf to 21,000 ksf (100 MPa to 1,000 MPa).

The following types of IGM have been encountered in Montana:

- weak shale;
- weak sandstone;
- mudstone, claystone and siltstone; and
- very dense sandy gravel.

The *AASHTO LRFD Specifications* recommend that piles bearing in IGM be designed using the same empirical methods that are used for cohesive and cohesionless soils. Experience indicates that static and dynamic analytical methods are not always reliable for predicting the design depth and capacity of piles driven into IGM. Predictions obtained from design calculations should be tempered and checked using knowledge gained from past projects with similar conditions. Because of the inherent uncertainty associated with these materials, it may be necessary to conduct a CAPWAP signal matching test including a dynamic wave equation analysis during pile installation and restrike to verify capacities. The FHWA Manual *Design and Construction of Driving Pile Foundations* encourages the use of static load tests in IGM materials to determine capacities and help quantify the various unknowns. Evaluate the need to perform a static load test for large projects or where large numbers of piles are anticipated.

16.4.2.10 Uplift

The design of piles for uplift conditions has become increasingly important for structures subject to seismic loading. Where piles are subjected to uplift forces or a moment that results in a net tensile force, investigate the pile group for resistance to pullout, structural ability to resist tension forces, and structural ability to transmit tension forces from the piles to the pile cap or footing. In some cases, uplift capacity determines the minimum pile penetration requirements.

Uplift capacity is typically determined for the pile group as if it acts as a foundation unit, as described in FHWA *Design and Construction of Driven Pile Foundations* and the *AASHTO LRFD Bridge Design Specifications*. In these references, the capacity of a single pile is first determined. The uplift capacity of the group is simply the sum of the uplift capacities of the individual piles. In fine-grained cohesive soils, where loading is assumed to occur under undrained conditions, the single pile shaft resistance is generally considered equal in compression and in uplift. However, in cohesionless or free-draining soils, the relationship between compression and uplift resistance is not as clear. Studies described in the literature indicate the uplift shaft resistance in cohesionless soil may vary from 70% to 100% of the compression shaft resistance.

For allowable stress design, the FHWA *Design and Construction of Driven Pile Foundations* suggests using a single pile design uplift capacity of 1/3 of the ultimate shaft resistance calculated from any of the static analysis methods, except for the Meyerhof (SPT) method, which should not be used. For LRFD design, *AASHTO LRFD Bridge Design Specifications* provide resistance factors for axial tension, which are lower than those for compression. The reason for the lower resistance factor (or higher factor of safety) is that once a pile begins to fail in uplift, the resistance progressively decreases with movement. This behavior is in contrast to most piles loaded in compression where an increase in capacity with movement eventually occurs.

16.4.2.11 Scour

Scour occurs as a result of flowing water eroding away material from the streambed and stream banks. Scour around bridge foundations can create a severe safety hazard. Therefore, design bridge foundations to survive the effects of possible scour.

Perform geotechnical analyses of bridge foundations assuming that the soil above the estimated scour line has been removed and is not available to provide bearing or lateral support. Scour is classified as follows:

1. Local (or Pier) Scour. Local scour affects materials only in the immediate vicinity of a substructure unit. Soil resistance in the scour zone provides resistance at the time of driving that cannot be accounted on for long-term support. Consequently, for determining long-term axial capacity, ignore the shaft resistance in the scour zone. However, for conducting drivability analyses, use the full shaft resistance. For pile capacity calculations in local scour cases, only consider the reduction in soil resistance in the scour zone, and the effective overburden pressure is unchanged.

2. Channel Degradation Scour. Channel degradation/contraction scour is where streambed materials are removed over a large area. Soil resistance in the scour zone provides resistance at the time of driving that cannot be counted on for long-term support. Therefore, like the local scour condition, ignore the shaft resistance in the scour zone for long-term pile support considerations, but not for drivability considerations. In contrast to local scour, pile capacity calculations in channel degradation scour cases should also include a reduction of effective overburden pressure due to removal of the streambed materials. This reduction in effective stresses can have a significant effect on the calculated shaft and toe resistances.

These two scour types need to be added together to determine the axial and lateral capacity.

Scour is usually evaluated for the 100-year flood. The FHWA recommends that:

- The top of the pile cap should be located below the depth of channel contraction scour to reduce obstruction to flow and to minimize local scour.
- A few long piles should be used rather than many short piles. This results in higher safety against pile failure due to scour.

16.4.2.12 Dynamic Pile-Driving Analyses

Where piles are installed using impact driving methods, evaluate the drivability of the pile foundation design by conducting a dynamic driving analysis. The dynamic analysis is used to:

- confirm that the design pile section can be installed to the desired depth and ultimate capacity with reasonable size hammers,
- develop capacity versus blowcount relationships, and
- evaluate compressive and tension stresses during driving to confirm that they are within the allowable driving stresses specified in the *AASHTO LRFD Bridge Design Specifications*.

In projects involving driven pile foundations, conduct the wave equation analyses using the computer program GRLWEAP. The wave equation analysis is usually conducted at two different phases of the project:

- during design using an assumed pile hammer and system, and
- at the onset of construction using the actual pile hammer system as submitted by the Contractor.

Although a wave equation analyses is required, the Gates formula is sometimes used to check wave equation analysis results. The Gates dynamic formula is described in FHWA *Driven Pile Foundations Publication*. Detailed information on the wave equation analysis is also provided in the FHWA publication and in the help files of the GRLWEAP program.

Dynamic monitoring of force and acceleration at the pile head during pile installation may be considered on any project involving driven piles to verify geotechnical capacity, both after driving and at a later time (restrike) to evaluate setup or relaxation. The dynamic monitoring is accomplished with a pile driving analyzer (PDA). Results of the PDA monitoring include stresses in the pile and an estimate of pile capacity. Dynamic monitoring using signal matching and wave equation analyses (e.g., CAPWAP) may be necessary for piles installed in difficult subsurface conditions, in soils with obstructions or boulders, in weak rock, where piles bear on steeply sloping bedrock surfaces, or other conditions in which uncertainties in the underlying strata exist. Dynamic monitoring may also be necessary on complex projects, projects involving large numbers of piles or situations in which the structural loads (axial and/or lateral) are relatively high.

16.4.2.13 Lateral Loading

Pile foundations are subjected to horizontal loads due to wind, traffic, bridge curvature, ice jams, vessel impact and earthquake. Evaluate the nominal resistance of pile foundations to horizontal loads based on both soil/rock properties and pile structural properties. Soil-structure interaction is a vital consideration in lateral pile analyses.

Design of laterally loaded piles should include evaluation of both the pile structural response and soil deformation to lateral loads. Determine the factor of safety against ultimate soil failure. In addition, calculate the pile deformation under the design loading conditions and compare it to the foundation performance criteria. Acceptable lateral deflections generally need to be evaluated on a case-by-case basis in consultation with the bridge engineer. Acceptable deflections are based upon numerous factors, some of which include loading type (e.g., service, extreme), type of superstructure, bridge location, etc.

The following two methods should both be used to perform lateral load analyses on piles:

1. Brom's Manual Computation Method. This method is a limit-equilibrium analysis that is used to compute the factor of safety against soil failure. Brom's method provides an estimate of the ultimate lateral load and pile deflections at the ground surface. The FHWA *Driven Pile Foundations Publication* provides a detailed procedure for conducting Brom's method of analysis.

2. Reese's Computer Method (LPILE). Reese developed a lateral pile analysis method called the P-y method that can be used to solve the nonlinear soil-structure interaction problem. In this method, P is the soil resistance per unit pile length and y is the lateral soil or pile deflection. Soil properties have the largest influence on the shape of the P-y curves. However, the P-y curves also depend upon depth, soil stress-strain relationships, pile width, water table location and loading condition (static or cyclic). Procedures for constructing P-y curves and for conducting lateral pile analyses by numerically solving the beam-on-elastic-foundation equation are well established. Chapter 13 discusses software that is available for analyzing single piles and pile groups.

The Geotechnical Section and Bridge Bureau use the P-y approach when evaluating the lateral response of driven piles. The analyses are conducted to estimate the deflection, moment and shear as a function of load at the head of the pile and the depth below the ground surface. In this analysis, loads are not proportional to displacements and the law of superposition cannot be used. In addition, the application of safety factors to soil parameters used in LPILE is inappropriate. The recommended technique is to use upper bound and lower bound soil values to check limit states. Upper bound values will control for shear in the foundation element, while lower bound values will control for deflection and moment.

During the P-y analyses, the following additional factors must be addressed:

1. Fixity at the Head. The fixity at the head of the pile can range from fixed to free. Usually a fixed-head model is used for smaller diameter piles; however, as loads and pile sizes increase, some rotation at the pile head can occur. This response will change the load-deflection response of the pile. The Bridge Bureau should provide input on the pile-head

fixity and, if this is not provided, the project geotechnical specialist should request this information from the Bridge Bureau. If there is uncertainty in the fixity, one option is to provide results for fixed- and free-head conditions.

2. Sloping Ground. Many abutment piles will be close to the approach fill slope. This can modify the P-y curves in the direction of the slope. Various methods exist for estimating P-y curves on sloping ground. Because the lateral loads are often the result of seismic event, one common approach is to assume that the P-y curve for level ground is appropriate, based on the assumption that the cyclic response in the longitudinal direction will balance between in-slope and out-of-slope response.

3. Repeated Cycles. Repeated cycles of load can affect the lateral response of a pile. The cyclic option in commercially available software was introduced to account for degradation in soil resistance that occurs with thousands of cycles of wave loading to an offshore platform. This cyclic option is not applicable for the limited number of cycles associated with earthquake loading. Normal practice is to use the P-y curves without modification for cycle effects when evaluating seismic response.

4. Liquefying Soils. Soil liquefaction will change the P-y curves in cohesionless soil. In the past, the approach was to assume that the liquefied soil responds as a soft clay and then represent the liquefied soil by the P-y curve for soft clays. The strength in the soft clay curve was defined by the residual strength of the liquefied soils (see Chapter 19 for further discussion on liquefied soil strength) and the ε_{50} was assumed to be approximately 0.02. Results from blast-load studies at Treasure Island, California and the Cooper River Bridge in South Carolina have suggested that the soft-clay model results in a stiffness that is too high, and that little reaction is developed until large displacements have developed. The computer programs L-PILE and DF-SAP (see Chapter 13) have incorporated these experimental results in explicit models for representing liquefied soil. No consensus currently (2008) exists on the most appropriate model. For this reason, MDT typically assumes the soil has zero strength in the liquefied zone.

5. Pile Stiffness. Consider the stiffness of the pile and how this stiffness changes with load. This issue is particularly important for concrete piles and drilled shafts, where section modulus of the pile or shaft can changed from uncracked to cracked during loading, resulting in a change to the predicted deformations, moments and shears. As long as steel piles are used, this issue is not critical. However, for projects that involve concrete piles, bored piles or drilled shafts, evaluate the effects of modulus change. The Bridge Bureau can provide direction on this issue.

Another important consideration during the lateral loading of a pile group is the potential contributions of the pile cap. If the pile cap will always be embedded, the P-y horizontal resistance of the soil on the cap face may be included as part of the overall lateral resistance of the foundation system. The passive pressure of the soil in front of the cap limits the horizontal resistance at the face of the cap. Use the *AASHTO LRFD Specifications* to estimate the passive pressure and include the wall friction in the friction estimate. The amount of displacement to mobilize this resistance ranges from 0.02 to 0.1 times the cap thickness. A good rule-of-thumb is to assume that the displacement to mobilize passive resistance is 0.05 times the cap thickness for well-compacted granular soils and 0.1 times the thickness for cohesive soils. The contribution from shear along the side of the cap also contributes to the resisting force; however, this resistance is usually small relative to the passive pressure contribution. Base shear should be neglected from the resistance calculation on the basis that the soil will usually settle away from the cap.

When analyzing the total capacity from a group of piles and the pile cap, consider the amount of displacement to mobilize the reaction from the cap and the piles. This evaluation often means developing a force-displacement curve for the cap and the pile group, and then developing a model that will result in compatibility of displacements.

16.4.2.14 Pile Group Lateral Capacity

Multiple rows of piles will have less resistance than the sum of the single individual piles because of pile-soil-pile interactions that take place in the pile group. Consequently, piles in pile groups can have less resistance to lateral load than piles in the lead row. The pile cap results in equal displacement of all piles in the cap, and therefore, the pile-soil-pile interaction (also called shadowing effect) results in the lateral capacity of a pile group being less than the sum of the lateral capacities of the individual piles comprising the group. Consequently, laterally loaded pile groups may have group efficiencies less than 1, depending on the spacing of the piles.

When the P-y method of analysis is used to evaluate a laterally loaded pile group, reduce the values of P by a multiplier (P_m), which results in softened (less stiff) soil response curves for the piles. Suggested multipliers are provided in Figure 16.4.D, which are based on the center-to-center pile spacing (D) and the row number in the direction of loading. An exception to these P_m values occurs when a single row of piles is loaded in a direction that is perpendicular to the row. In this case, the group reduction factor is 1.0 if the pile spacing is 5D or greater. The group reduction factor is 0.7 for a pile spacing of 3D.

Pile Center-to-Center Spacing (in the direction of loading)	**Pile Load Modifiers, P_m**		
	Row 1	**Row 2**	**Row 3 and Higher**
3D	0.7	0.5	0.35
5D	1.0	0.85	0.7

Figure 16.4-D — PILE LOAD MODIFIERS, PM, FOR MULTIPLE ROW SHADING

16.5 DRILLED SHAFTS

16.5.1 General

Drilled shaft foundations are used where neither spread footings nor driven piles are suitable for the site. Like the driven pile, drilled shafts are used where deep deposits of soft soils occur, or where near-surface soils are susceptible to scour, soil liquefaction or lateral spreading. Drilled shafts are also used where loads are large, where right-of-way and space limitations preclude the use of shallow foundations or driven pile foundations, and where noise and vibrations from pile driving are not acceptable.

16.5.1.1 Design Requirements

Drilled shafts are designed for both axial and lateral loading conditions. The two principal design considerations for drilled shafts under axial loads are the nominal load capacity and settlement. The nominal (ultimate) load capacity of a drilled shaft may be governed by either the structural capacity of the drilled shaft or the bearing capacity of the soil. Drilled shafts that are subjected to lateral loads must also be safe against ultimate failure of the soil or the concrete shaft and excessive lateral deflection.

The project geotechnical specialist provides design recommendations to the Bridge Bureau for drilled shaft foundations where spread footing foundations and driven pile foundations are not appropriate. The design recommendations include the shaft capacity, settlement, lateral response and constructability. The following is also provided to the Bridge Bureau:

1. Axial Capacity. Provide the axial capacity in both compression and uplift, where appropriate, for the given design embedment length. Also, include the appropriate resistance factors for the three limit states.

2. Settlement Estimates. Provide settlement estimates if the drilled shafts will not be founded in rock or extremely dense soils. These settlement estimates should include both the immediate settlement and any long-term consolidation settlement.

3. Lateral Response. The lateral response analyses should show the displacements, moments and shears as a function of depth and for a range of loads that could be imposed on the head of the shaft. This information is provided electronically with LPILE files. The fixity at the head (provided by Bridge Bureau) of the shaft should be considered in these evaluations.

4. Installation Recommendations. Include any recommendations on shaft installation, including use of temporary or permanent casing. Note that the Drilled Shaft Special Provision requires full-depth temporary casing to the bottom of the shaft, unless specifically modified for the project. During casing extraction, the contractor is required to maintain a level of fresh concrete in the casing a minimum of 5 ft (1.5 m) above the hydrostatic water level or drilling fluid level outside the casing, or a minimum of 5 ft (1.5 m) above the bottom of the casing, whichever is higher.

5. Depth. Provide the design tip elevation of the drilled shaft.

16.5.1.2 Site Characterization for Drilled Shaft Foundations

The site characterization requirements for drilled shaft foundations are similar to those used for driven pile foundations. The following additional factors should be considered or evaluated during the investigation:

1. Cobbles and Boulders. Construction of a shaft can be affected by the presence of cobbles and boulders and, therefore, the site characterization effort should try to quantify these effects through the review of drilling information and geologic reviews. Because of the importance of cobbles and boulders to the shaft construction process, normal practice is to conduct a geotechnical exploration at center of each shaft location.

2. Gravel and Cobbles. Identify the presence of open gravel and cobble layers, as these materials may require the use of casing or special drilling muds to avoid hole collapse or excessive loss in drilling muds during construction.

3. Explorations. Explorations should extend at least 20 ft (6 m) or 5 shaft diameters, whichever is greater, below the likely toe of the shaft. If hard bearing material or rock is located less than 20 ft (6 m), the depth of exploration can be stopped 10 ft (3 m) into the hard bearing material.

4. Socketed. If the shaft is going to be socketed in rock, the exploration should extend at least 2 shaft diameters below the planned toe elevation of the shaft.

5. Groundwater. As part of the site characterization effort, it is also very important to establish the location of the groundwater table and whether groundwater is perched or involves artesian conditions. These conditions will have an important effect on the drilling methods selected by the shaft construction contractor.

16.5.1.3 Construction of Drilled Shafts

Details of the construction procedures are critical with regard to the performance of the drilled shafts. Therefore, construction methods must be carefully controlled in order for the foundation to function as designed. Different subsurface conditions warrant different methods of construction. Three methods commonly used in the United States include the dry method, the casing method and the wet method. Detailed descriptions of these methods, along with examples of possible construction problems, are described in FHWA *Drilled Shafts: Construction Procedures and Design Methods*.

16.5.1.3.1 Clay Sites

The effects of installing a drilled shaft into clay are different from those of installing a pile. If the clay is homogeneous so that the excavation will remain open and dry, the clay will creep toward the axis of the excavation accompanied by vertical subsidence of the ground surface. The creep and subsidence will be substantial if the clay is weak, but minimal for stronger overconsolidated clays. Disturbance and stress relief due to drilling will cause some loss of shear strength at the surface of the borehole, which must be addressed during design.

The placement of fluid concrete in the excavation will impose a lateral stress on the sides of the excavation, the magnitude of which is dependent on the fluidity and rate of placement of the concrete. If the excavation is drilled dry, moisture from the fluid concrete can migrate into the clay and cause some additional softening. This problem can be important in concrete that is mixed with a high water-cement ratio in which much more water than is needed to hydrate the cement is used in batching. Whether the excavation in the clay is wet or dry, there is evidence to show that there is an interaction between the clay and particles of cement and/or products of cement hydration, with a consequent strengthening of the bond between the concrete and the clay. This interaction results in a larger strength at the interface than the softened strength that exists just after the concrete placement.

16.5.1.3.2 Sand Sites

If the sand in a drilled-shaft excavation is prevented from collapsing by driving a casing into place, the behavior of the sand around the perimeter (shaft) of the casing will be similar to that of a driven pile. The sand will heave at the base of the excavation resulting in lower unit end bearing than for a driven pile. The end-bearing load-deformation behavior may be adversely affected by construction practices that fail to remove cuttings that have been suspended in drilling slurries during borehole excavation. The Drilled Shaft Special Provision limits the amount of loose or disturbed material in the bottom of the shaft to 1 in (25 mm) after cleaning.

The placing of concrete with high workability (cohesive mixes with high slump) will impose stresses against the sides and base of the excavation that are larger than those from the slurry. The fluid concrete could then cause a slight densification of the sand adjacent to the wall and base of the drilled shaft. Concrete with a low slump will bulk and not collapse under its own weight. In addition to producing potential defects (e.g., honeycomb; voids) in the concrete, this effect causes the lateral stress against the sides of the excavation to be less than would occur had the concrete been fluid. The resistance along the sides are to some extent dependent on this concrete pressure. Low-slump concrete can also have a negative effect on geomaterial resistance.

As with clay, the properties of sand around a drilled shaft can be very different from the in-situ properties. The subsurface investigation should be designed to reveal as well as practical the in-situ characteristics of the sands, especially its density and grain-size distribution. The parameters selected for the design of a drilled shaft in sand will then be adjusted by the design method according to the best estimate of the properties of the sand that exist around the drilled shaft as built.

16.5.1.3.3 Rock Sites

The requirement to bear on or penetrate rock strata often dictates the use of drilled shaft foundations. One of the important considerations of rock-socketed drilled shafts is the condition of the side of the borehole. High values of side shearing resistance can develop because of dilation that occurs between a rough surface at the boundary of the concrete and the mating surface in the rock. Upward or downward movement of the concrete shaft caused by applying axial loads produces lateral compression of the rock and, as a result, higher lateral stresses along the concrete-rock interface than existed after the concrete was placed. The increased lateral stresses can in turn increase the strength of the rock if pore pressures dissipate rapidly. Either the rock or concrete finally fails by some manner of shearing through the respective asperities, at a high value of resistance.

Construction practices that cause the concrete-rock interface to be smooth, rather than rough, can have a profoundly negative effect on the side shearing resistance that develops in rock sockets. For example, in argillaceous (clay based) rock (e.g., shale, mudstone, slate), the presence of free water in the borehole during drilling (e.g., minor inflow of water from a small perched aquifer near the surface, intentional introduction of water by the Contractor to aid in excavating cuttings) can cause the surface of the rock to become fully softened or “smeared,” so that any effect of borehole roughness is almost completely masked. The rough interface with degraded (smeared) rock behaves very similarly to the smooth interface, and the behavior of a drilled shaft with a smeared interface is closer to the behavior of a drilled shaft in a mass of soil that has properties of the degraded rock, rather than one with a rough interface in the original rock.

16.5.2 Geotechnical Design Considerations

16.5.2.1 Spacing

The center-to-center spacing of drilled shafts should be the greater of 3.0 diameters or the spacing required to avoid interaction between adjacent shafts. Larger spacing than 3.0 diameters may be necessary when drilling operations are anticipated to be difficult. If closer spacing is necessary because of project constraints, address the sequence of construction in the contract documents, and evaluate the interaction between adjacent shafts for group effects and a corresponding reduction in group efficiency.

16.5.2.2 Movement

Horizontal movements occur at bridge abutments and piers due to lateral forces from earth pressure, wind loads, stream flow forces, braking forces of vehicles and earthquakes. Lateral movements of abutments and piers must be limited to prevent damage to bearings and expansion joints (functional and structural damage) and poor ride quality.

Excessive movements of foundations supporting bridges may lead to discontinuities in the slope of the riding surface, damage to the bridge superstructure or substructure, jamming of bearings and expansion joints or even collapse. It is necessary in bridge design to estimate the maximum settlement and lateral movement anticipated in the foundations and to ensure that they fall within tolerable limits. Acceptable lateral deflections generally need to be evaluated on a case-by-case basis in consultation with the structural engineer because acceptable deflections will be based upon numerous factors, some of which include loading type (e.g., service, extreme), type of superstructure, bridge location, etc.

Load tests on instrumented drilled shafts have shown that the movement required to mobilize shaft resistance in drilled shafts is smaller than that required to mobilize end bearing. The shaft capacity in clays is fully mobilized when the settlement is less than 1% of the shaft diameter. The end bearing of drilled shafts in clay, however, is not mobilized until the shaft settles about 2% to 5% of its diameter. For drilled shafts in sands, the side resistance is fully mobilized at settlements less than 1% of its diameter. However, very large displacements are required to fully mobilize the end bearing of drilled shafts in sands and the tolerable settlement will usually be exceeded much before the end bearing is fully mobilized. This is an important design consideration when the working load acting on the drilled shaft exceeds the shaft resistance. In this case, larger settlements may be required to mobilize the portion of the end bearing that supports the load not carried by the side resistance. For design purposes, the "ultimate" end-bearing capacity of drilled shafts in sand is usually limited to the capacity mobilized at a settlement of 5% of the diameter of the drilled shaft.

16.5.2.3 Shaft Capacity

The axial capacity of a drilled shaft is the sum of its toe and shaft capacities. During failure, the shear stress at the interface of the drilled shaft and soil reaches a limiting value. This can occur under both compressive and upward (tensile) loads.

Drilled shafts in saturated clays are usually designed using total stress analyses where the undrained shear strength of the clay is used. Long-term loads will lead to an increase in the shear strength of the clay around the shaft as the clay consolidates with time. Associated with this consolidation will be some settlement of the foundation. However, there is a possibility that negative pore pressures can develop along the sides of drilled shafts in heavily overconsolidated clay or shale, leading to soil softening over time. As a result, total stress methods of analyzing drilled shafts in heavily overconsolidated clay or shale may be unconservative. In this case, FHWA *Drilled Shafts: Construction Procedures and Design Methods* suggests using the undrained shear strength measured on a triaxial or direct shear specimen that has previously been allowed to imbibe water. The purpose of the imbibition is to approximate the long-term softening behavior.

A drilled shaft will fail in compression when the loads exceed the structural or soil capacity. The structural capacity of the shaft is often greater than the ultimate soil capacity except when the shaft bears on sound rock. Nevertheless, always check the adequacy of the drilled shaft against structural failure. Also check the tensile capacity of a drilled shaft where the shaft is subjected to uplift loads.

Semi-empirical methods may be used to estimate the resistance of drilled shafts in cohesive soils. Drilled shafts in cohesive soils should be designed by total and effective stress methods for undrained and drained loading conditions, respectively. Shafts in cohesionless soils should be designed using effective stress methods for drained loading conditions or by empirical methods based on in-situ test results. Equations and procedures for conducting these analyses are described in the *Engineering Manual for Drilled Shafts* (Barker et al, 1991), *AASHTO LRFD Bridge Design Specifications* and FHWA *Drilled Shafts: Construction Procedures and Design Methods*. The methods described in these references for shaft capacity in IGMs may not yield reliable results for conditions encountered in Montana. Verify calculated shaft capacities in IGMs using alternative methods and experience.

When calculating shaft resistance, friction along the upper 5 ft (1.5 m) of the shaft is ignored to account for the effects of seasonal moisture changes, disturbance during construction, cyclic lateral loading and lower lateral stresses. Resistance along the lower 1.0-diameter length above the shaft toe (or top of enlarged base) is ignored due to the development of tensile cracks in the soil and a corresponding reduction in lateral stress and side resistance.

If the temporary casing cannot be retrieved during construction of the shaft, this is defined as a shaft defect by the special provisions. The special provision requires the Contractor to perform corrective action using a method approved by MDT.

16.5.2.4 Resistance Factors

Resistance factors for side resistance and toe resistance are available in the *AASHTO LRFD Bridge Design Specifications*. An important task in the design of drilled shafts involves the choice of an appropriate resistance factor for each design mode (i.e., axial or lateral capacity). Previously, the choice was made based on values that have been used in the past in a given location, tempered with the judgment of the geotechnical engineer. The *AASHTO LRFD Specifications* are much more explicit, with resistance factors that vary with method of analysis, geology and form of loading. If load tests are conducted, the resistance factor also changes. Review the *AASHTO LRFD Specifications* for a discussion of these alternatives.

The value used for the resistance factor depends, in part, on the level of uncertainty that exists in the quantification of design parameters, which depends directly on the uncertainty of the soil data obtained in the subsurface investigation. Where a site is highly variable, relatively few geomaterial samples are tested and/or uncertainty is high, use a lower resistance factor, unless very conservative values for the soil parameters have been selected. In allowable stress design, MDT typically uses a 2.5 factor of safety for axially loaded drilled shafts.

Uncertainty can be reduced by making a boring at the location of every drilled shaft on the project (this approach is highly recommended) and a minimum of one boring per bent is required. This may also reduce the probability of Contractor claims.

16.5.2.5 Lateral Loading

Laterally loaded drilled shafts will fail in flexure if the induced bending moment exceeds the moment capacity of the shafts. The structural capacity of the drilled shaft is dependent on both the moment and axial load. Structural adequacy is checked using load-moment interaction diagrams. These are envelopes of the combinations of moment and axial load that would cause structural failure.

The ultimate geotechnical capacity is usually not a controlling factor in the design of drilled shafts to resist lateral loads. The governing criterion in lateral load design is usually either maximum tolerable deflection or structural capacity. The Broms method should be used to calculate the factor of safety against lateral soil failure. The lateral deflection of single drilled shafts and groups of drilled shafts may be estimated using the procedures described in the *Engineering Manual for Drilled Shafts* (Barker et al, 1991) or by conducting a P-y analysis using the computer software LPILE.

MDT’s Geotechnical Section and Bridge Bureau normally use the LPILE method of evaluating drilled shaft behavior during lateral loading. Consider the following factors when conducting LPILE analyses on drilled shafts:

1. Side Shear. Conventional P-y curves were developed from load tests on smaller diameter pipe piles. Information suggests that these P-y curves may not account for the side shear that develops on a large diameter drilled shaft. Modifications to the P-y curve to account for this side shear may be appropriate. Strain wedge procedures have been developed that may better account for side shear. This approach is taken in the computer program DF-SAP developed by the Washington Department of Transportation; see Chapter 13.

2. Base Shear. The shear developed at the base of a shaft can be significant and can be explicitly modeled in the LPILE program. Whether this shear can be counted on will depend on the construction method. For dry holes where good cleanout occurs, it may be reasonable to include the shear capacity. Otherwise, this contribution should usually be ignored or assigned very low values.

3. Displacement. The amount of displacement allowed at the toe of a drilled shaft during lateral loading can be a question during the LPILE analyses. Usually movements of up to 1 in (25 mm) during seismic loading are permissible, because of the low likelihood of occurrence of these loads and the limited consequences of the movement. However, this performance objective should be discussed with the Bridge Bureau.

16.5.2.6 Settlement

Settlement of a drilled shaft foundation should be evaluated as a single shaft or as a group, whichever is applicable. Estimate the settlement considering:

- short-term settlement;
- consolidation settlement, if constructed in cohesive soils; and
- axial compression of the shaft.

Dimensionless load-settlement curves for drilled shafts in cohesive and cohesionless soils are presented in the *AASHTO LRFD Bridge Design Specifications* for both side resistance and end bearing conditions. These curves include elastic shortening of the shaft. They represent the immediate response of the shaft to load. Add consolidation settlement to settlement estimated from the load-settlement curves if the shaft is founded in a compressible clay layer (an atypical situation).

Address the group settlement of drilled shafts using the same approach for driven piles; see the *AASHTO LRFD Specifications*.