Chapter 8 Foundation Design

8.1 Overview

This chapter covers the geotechnical design of bridge foundations, cut-and-cover tunnel foundations, foundations for walls, and hydraulic structure foundations (pipe arches, box culverts, flexible culverts, etc.). Chapter 17 covers foundation design for lightly loaded structures, and Chapter 18 covers foundation design for marine structures. Both shallow (e.g., spread footings) and deep (piles, shafts, micro-piles, etc.) foundations are addressed. In general, the load and resistance factor design approach (LRFD) as prescribed in the AASHTO LRFD Bridge Design Specifications shall be used, unless a LRFD design methodology is not available for the specific foundation type being considered (e.g., micro-piles). Structural design of bridge and other structure foundations is addressed in the WSDOT *LRFD Bridge Design Manual* (BDM).

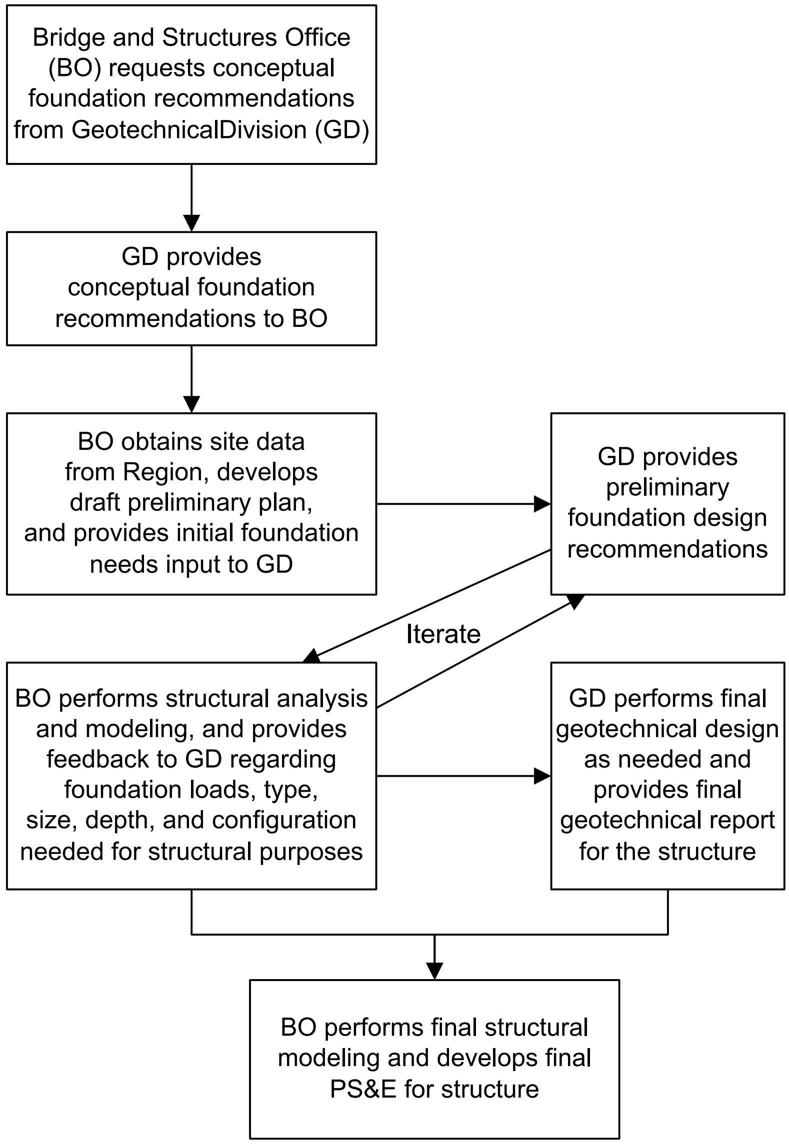
All structure foundations within WSDOT Right of Way or whose construction is administered by WSDOT shall be designed in accordance with the *Geotechnical Design Manual* (GDM) and the following documents:

- *Bridge Design Manual* LRFD M23-50
- *Standard Plans for Road, Bridge, and Municipal Construction* M 21-01
- AASHTO LRFD Bridge Design Specifications, U.S.

The most current versions of the above referenced manuals including all interims or design memoranda modifying the manuals shall be used. In the case of conflict or discrepancy between manuals, the following hierarchy shall be used: those manuals listed first shall supersede those listed below in the list.

8.2 Overall Design Process for Structure Foundations

The overall process for geotechnical design is addressed in Chapters 1 and 23. For design of structure foundations, the overall WSDOT design process, including both the geotechnical and structural design functions, is as illustrated in Figure 8-1.



Overall Design Process for LRFD Foundation Design
Figure 8-1

The steps in the flowchart are defined as follows:

Conceptual Bridge Foundation Design – This design step results in an informal communication/report produced by the Geotechnical Office at the request of the Bridge and Structures Office. This informal communication/report, consistent with what is described for conceptual level geotechnical reports in Chapter 23, provides a brief description of the anticipated site conditions, an estimate of the maximum slope feasible for the bridge approach fills for the purpose of determining bridge length, conceptual foundation types feasible, and conceptual evaluation of potential geotechnical hazards such as liquefaction. The purpose of these recommendations is to provide enough geotechnical information to allow the bridge preliminary plan to be produced. This type of conceptual evaluation could also be applied to other types of structures, such as tunnels or special design retaining walls.

Develop Site data and Preliminary Plan – During this phase, the Bridge and Structures Office obtains site data from the Region (see *Design Manual* Chapters 610, 710, and 730) and develops a preliminary bridge plan (or other structure) adequate for the Geotechnical Office to locate borings in preparation for the final design of the structure (i.e., pier locations are known with a relatively high degree of certainty). The Bridge and Structures Office would also provide the following information to the Geotechnical Office to allow them to adequately develop the preliminary foundation design:

- Anticipated structure type and magnitudes of settlement (both total and differential) the structure can tolerate.
- At abutments, the approximate maximum elevation feasible for the top of the foundation in consideration of the foundation depth.
- For interior piers, the number of columns anticipated, and if there will be single foundation elements for each column, or if one foundation element will support multiple columns.
- At stream crossings, the depth of scour anticipated, if known. Typically, the Geotechnical Office will pursue this issue with the HQ Hydraulics Office.
- Any known constraints that would affect the foundations in terms of type, location, or size, or any known constraints which would affect the assumptions which need to be made to determine the nominal resistance of the foundation (e.g., utilities that must remain, construction staging needs, excavation, shoring and falsework needs, other constructability issues).

Preliminary Foundation Design – This design step results in a memorandum produced by the Geotechnical Office at the request of the Bridge and Structures Office that provides geotechnical data adequate to do the structural analysis and modeling for all load groups to be considered for the structure. The geotechnical data is preliminary in that it is not in final form for publication and transmittal to potential bidders. In addition, the foundation recommendations are subject to change, depending on the results of the structural analysis and modeling and the effect that modeling and analysis has on foundation types, locations, sizes, and depths, as well as any design assumptions made by the geotechnical designer. Preliminary foundation recommendations may also be subject to change depending on the construction staging needs and other constructability issues that are discovered during this design phase. Geotechnical work conducted during this stage typically includes completion of the field exploration program to the final PS&E level, development of foundation types and capacities feasible, foundation depths needed, P-Y curve data and soil spring data for seismic modeling, seismic site characterization and estimated ground acceleration, and recommendations to address known constructability issues. A description of subsurface conditions and a preliminary subsurface profile would also be provided at this stage, but detailed boring logs and laboratory test data would usually not be provided.

Structural Analysis and Modeling – In this phase, the Bridge and Structures Office uses the preliminary foundation design recommendations provided by the Geotechnical Office to perform the structural modeling of the foundation system and superstructure. Through this modeling, the Bridge and Structures Office determines and distributes the loads within the structure for all appropriate load cases, factors the loads as appropriate, and sizes the foundations using the foundation nominal resistances and resistance factors provided by the Geotechnical Office. Constructability and construction staging needs would continue to be investigated during this phase. The Bridge and Structures Office would also provide the following feedback to the Geotechnical Office to allow them to check their preliminary foundation design and produce the Final Geotechnical Report for the structure:

- Anticipated foundation loads (including load factors and load groups used).
- Foundation size/diameter and depth required to meet structural needs.
- Foundation details that could affect the geotechnical design of the foundations.
- Size and configuration of deep foundation groups.

Final Foundation Design – This design step results in a formal geotechnical report produced by the Geotechnical Office that provides final geotechnical recommendations for the subject structure. This report includes all geotechnical data obtained at the site, including final boring logs, subsurface profiles, and laboratory test data, all final foundation recommendations, and final constructability recommendations for the structure. At this time, the Geotechnical Office will check their preliminary foundation design in consideration of the structural foundation design results determined by the Bridge and Structures Office, and make modifications to the preliminary foundation design as needed to accommodate the structural design needs provided by the Bridge and Structures Office. It is possible that much of what was included in the preliminary foundation design memorandum may be copied into the final geotechnical report, if no design changes are needed. This report will also be used for publication and distribution to potential bidders.

Final Structural Modeling and PS&E Development – In this phase, the Bridge and Structures Office makes any adjustments needed to their structural model to accommodate any changes made to the geotechnical foundation recommendations as transmitted in the final geotechnical report. From this, the bridge design and final PS&E would be completed.

Note that a similar design process should be used if a consultant or design-builder is performing one or both design functions.

8.3 Data Needed for Foundation Design

The data needed for foundation design shall be as described in the AASHTO LRFD Bridge Design Specifications, Section 10 (most current version). The expected project requirements and subsurface conditions should be analyzed to determine the type and quantity of information to be developed during the geotechnical investigation. During this phase it is necessary to:

- Identify design and constructability requirements (e.g. provide grade separation, transfer loads from bridge superstructure, provide for dry excavation) and their effect on the geotechnical information needed
- Identify performance criteria (e.g. limiting settlements, right of way restrictions, proximity of adjacent structures) and schedule contraints
- Identify areas of concern on site and potential variability of local geology
- Develop likely sequence and phases of construction and their effect on the geotechnical information needed
- Identify engineering analyses to be performed (e.g. bearing capacity, settlement, global stability)
- Identify engineering properties and parameters required for these analyses
- Determine methods to obtain parameters and assess the validity of such methods for the material type and construction methods
- Determine the number of tests/samples needed and appropriate locations for them.

Table 8-1 provides a summary of information needs and testing considerations for foundation design.

Chapter 5 covers the requirements for how the results from the field investigation, the field testing, and the laboratory testing are to be used separately or in combination to establish properties for design. The specific test and field investigation requirements needed for foundation design are described in the following sections.

Foundation Type	Engineering Evaluations	Required Information for Analyses	Field Testing	Laboratory Testing
Shallow Foundations	• bearing capacity • settlement (magnitude & rate) • shrink/swell of foundation soils (natural soils or embankment fill) • frost heave • scour (for water crossings) • liquefaction • overall slope stability	• subsurface profile (soil, groundwater, rock) • shear strength parameters • compressibility parameters (including consolidation, shrink/swell potential, and elastic modulus) • frost depth • stress history (present and past vertical effective stresses) • depth of seasonal moisture change • unit weights • geologic mapping including orientation and characteristics of rock discontinuities	• SPT (granular soils) • CPT • PMT • dilatometer • rock coring (RQD) • plate load testing • geophysical testing	• 1-D Oedometer tests • soil/rock shear tests • grain size distribution • Atterberg Limits • specific gravity • moisture content • unit weight • organic content • collapse/swell potential tests • intact rock modulus • point load strength test
Driven Pile Foundations	• pile end-bearing • pile skin friction • settlement • down-drag on pile • lateral earth pressures • chemical compatibility of soil and pile • drivability • presence of boulders/ very hard layers • scour (for water crossings) • vibration/heave damage to nearby structures • liquefaction • overall slope stability	• subsurface profile (soil, ground water, rock) • shear strength parameters • horizontal earth pressure coefficients • interface friction parameters (soil and pile) • compressibility parameters • chemical composition of soil/rock (e.g., potential corrosion issues) • unit weights • presence of shrink/swell soils (limits skin friction) • geologic mapping including orientation and characteristics of rock discontinuities	• SPT (granular soils) • pile load test • CPT • PMT • vane shear test • dilatometer • piezometers • rock coring (RQD) • geophysical testing	• soil/rock shear tests • interface friction tests • grain size distribution • 1-D Oedometer tests • pH, resistivity tests • Atterberg Limits • specific gravity • organic content • moisture content • unit weight • collapse/swell potential tests • intact rock modulus • point load strength test
Drilled Shaft Foundations	• shaft end bearing • shaft skin friction • constructability • down-drag on shaft • quality of rock socket • lateral earth pressures • settlement (magnitude & rate) • groundwater seepage/ dewatering/ potential for caving • presence of boulders/ very hard layers • scour (for water crossings) • liquefaction • overall slope stability	• subsurface profile (soil, ground water, rock) • shear strength parameters • interface shear strength friction parameters (soil and shaft) • compressibility parameters • horizontal earth pressure coefficients • chemical composition of soil/rock • unit weights • permeability of water-bearing soils • presence of artesian conditions • presence of shrink/swell soils (limits skin friction) • geologic mapping including orientation and characteristics of rock discontinuities • degradation of soft rock in presence of water and/or air (e.g., rock sockets in shales)	• installation technique test shaft • shaft load test • vane shear test • CPT • SPT (granular soils) • PMT • dilatometer • piezometers • rock coring (RQD) • geophysical testing	• 1-D Oedometer • soil/rock shear tests • grain size distribution • interface friction tests • pH, resistivity tests • permeability tests • Atterberg Limits • specific gravity • moisture content • unit weight • organic content • collapse/swell potential tests • intact rock modulus • point load strength test • slake durability

Summary of Information Needs and Testing Considerations
(Modified After Sabatini, et al., 2002)
Table 8-1

8.3.1 Field Exploration Requirements for Foundations

Subsurface explorations shall be performed to provide the information needed for the design and construction of foundations. The extent of exploration shall be based on variability in the subsurface conditions, structure type, and any project requirements that may affect the foundation design or construction. The exploration program should be extensive enough to reveal the nature and types of soil deposits and/or rock formations encountered, the engineering properties of the soils and/or rocks, the potential for liquefaction, and the ground water conditions. The exploration program should be sufficient to identify and delineate problematic subsurface conditions such as karstic formations, mined out areas, swelling/collapsing soils, existing fill or waste areas, etc.

Borings should be sufficient in number and depth to establish a reliable longitudinal and transverse substrata profile at areas of concern, such as at structure foundation locations, adjacent earthwork locations, and to investigate any adjacent geologic hazards that could affect the structure performance. Requirements for the number and depth of borings presented in the AASHTO LRFD Bridge Design Specifications, Article 10.4.2, should be used. While engineering judgment will need to be applied by a licensed and experienced geotechnical professional to adapt the exploration program to the foundation types and depths needed and to the variability in the subsurface conditions observed, the intent of AASHTO Article 10.4.2 regarding the minimum level of exploration needed should be carried out. Geophysical testing may be used to guide the planning of the subsurface exploration and reduce the requirements for borings. The depth of borings indicated in AASHTO Article 10.4.2 performed before or during design should take into account the potential for changes in the type, size and depth of the planned foundation elements.

AASHTO Article 10.4.2 shall be used as a starting point for determining the locations of borings. The final exploration program should be adjusted based on the variability of the anticipated subsurface conditions as well as the variability observed during the exploration program. If conditions are determined to be variable, the exploration program should be increased relative to the requirements in AASHTO Article 10.4.2 such that the objective of establishing a reliable longitudinal and transverse substrata profile is achieved. If conditions are observed to be homogeneous or otherwise are likely to have minimal impact on the foundation performance, and previous local geotechnical and construction experience has indicated that subsurface conditions are homogeneous or otherwise are likely to have minimal impact on the foundation performance, a reduced exploration program relative to what is specified in AASHTO Article 10.4.2 may be considered. Even the best and most detailed subsurface exploration programs may not identify every important subsurface problem condition if conditions are highly variable. The goal of the subsurface exploration program, however, is to reduce the risk of such problems to an acceptable minimum.

For situations where large diameter rock socketed shafts will be used or where drilled shafts are being installed in formations known to have large boulders, or voids such as in karstic or mined areas, it may be necessary to advance a boring at the location of each shaft.

In a laterally homogeneous area, drilling or advancing a large number of borings may be redundant, since each sample tested would exhibit similar engineering properties. Furthermore, in areas where soil or rock conditions are known to be very favorable to the construction and performance of the foundation type likely to be used (e.g., footings on very dense soil, and groundwater is deep enough to not be a factor), obtaining fewer borings than provided in AASHTO Article 10.4.2 may be justified. In all cases, it is necessary to understand how the design and construction of the geotechnical feature will be affected by the soil and/or rock mass conditions in order to optimize the exploration.

Samples of material encountered shall be taken and preserved for future reference and/or testing. Boring logs shall be prepared in detail sufficient to locate material strata, results of penetration tests, groundwater, any artesian conditions, and where samples were taken. Special attention shall be paid to the detection of narrow, soft seams that may be located at stratum boundaries.

For drilled shaft foundations, it is especially critical that the groundwater regime is well defined at each foundation location. Piezometer data adequate to define the limits and piezometric head in all unconfined, confined, and locally perched groundwater zones should be obtained at each foundation location.

For cut-and-cover tunnels, pipe arches, etc., spacing of investigation points shall be consistent for that required for retaining walls (see Chapter 15), with a minimum of two investigation points spaced adequately to develop a subsurface profile for the entire structure.

8.3.2 Laboratory and Field Testing Requirements for Foundations

General requirements for laboratory and field testing, and their use in the determination of properties for design, are addressed in Chapter 5. In general, for foundation design, laboratory testing should be used to augment the data obtained from the field investigation program, to refine the soil and rock properties selected for design.

Foundation design will typically heavily rely upon the SPT and/or q_c results obtained during the field exploration through correlations to shear strength, compressibility, and the visual descriptions of the soil/rock encountered, especially in non-cohesive soils. The information needed for the assessment of ground water and the hydrogeologic properties needed for foundation design and constructability evaluation is typically obtained from the field exploration through field instrumentation (e.g., piezometers) and in-situ tests (e.g., slug tests, pump tests, etc.). Index tests such as soil gradation, Atterberg limits, water content, and organic content are used to confirm the visual field classification of the soils encountered, but may also be used directly to obtain input parameters for some aspects of foundation design (e.g., soil liquefaction, scour, degree of over-consolidation, and correlation to shear strength or compressibility of cohesive soils). Quantitative or performance laboratory tests conducted on undisturbed soil samples are used to assess shear strength or compressibility of finer grained soils, or to obtain seismic design input parameters such as shear modulus. Site performance data, if available, can also be used to assess design input parameters. Recommendations are provided in Chapter 5 regarding how to make the final selection of design properties based on all of these sources of data.

8.4 Foundation Selection Considerations

Foundation selection considerations to be evaluated include:

- the ability of the foundation type to meet performance requirements (e.g., deformation, bearing resistance, uplift resistance, lateral resistance/deformation) for all limit states, given the soil or rock conditions encountered
- the constructability of the foundation type
- the impact of the foundation installation (in terms of time and space required) on traffic and right-of-way
- the environmental impact of the foundation construction
- the constraints that may impact the foundation installation (e.g., overhead clearance, access, and utilities)
- the impact of the foundation on the performance of adjacent foundations, structures, or utilities, considering both the design of the adjacent foundations, structures, or utilities, and the performance impact the installation of the new foundation will have on these adjacent facilities.
- the cost of the foundation, considering all of the issues listed above.

Spread footings are typically very cost effective, given the right set of conditions. Footings work best in hard or dense soils that have adequate bearing resistance and exhibit tolerable settlement under load. Footings can get rather large in medium dense or stiff soils to keep bearing stresses low enough to minimize settlement, or for structures with tall columns or which otherwise are loaded in a manner that results in large eccentricities at the footing level, or which result in the footing being subjected to uplift loads. Footings are not effective where soil liquefaction can occur at or below the footing level, unless the liquefiable soil is confined, not very thick, and well below the footing level. However, footings may be cost effective if inexpensive soil improvement techniques such as overexcavation, deep dynamic compaction, and stone columns, etc. are feasible. Other factors that affect the desirability of spread footings include the need for a cofferdam and seals when placed below the water table, the need for significant overexcavation of unsuitable soil, the need to place footings deep due to scour and possibly frost action, the need for significant shoring to protect adjacent existing facilities, and inadequate overall stability when placed on slopes that have marginally adequate stability. Footings may not be feasible where expansive or collapsible soils are present near the bearing elevation. Since deformation (service) often controls the feasibility of spread footings, footings may still be feasible and cost effective if the structure the footings support can be designed to tolerate the settlement (e.g., flat slab bridges, bridges with jackable abutments, etc.).

Deep foundations are the best choice when spread footings cannot be founded on competent soils or rock at a reasonable cost. At locations where soil conditions would normally permit the use of spread footings but the potential exists for scour, liquefaction or lateral spreading, deep foundations bearing on suitable materials below such susceptible soils should be used as a protection against these problems. Deep foundations should also be used where an unacceptable amount of spread footing settlement may occur. Deep foundations should be used where right-of-way, space limitations, or other constraints as discussed above would not allow the use of spread footings.

Two general types of deep foundations are typically considered: pile foundations, and drilled shaft foundations. Shaft foundations are most advantageous where very dense intermediate strata must be penetrated to obtain the desired bearing, uplift, or lateral resistance, or where obstructions such as boulders or logs must be penetrated. Shafts may also become cost effective where a single shaft per column can be used in lieu of a pile group with a pile cap, especially when a cofferdam or shoring is required to construct the pile cap. However, shafts may not be desirable where contaminated soils are present, since contaminated soil would be removed, requiring special handling and disposal. Shafts should be used in lieu of piles where deep foundations are needed and pile driving vibrations could cause damage to existing adjacent facilities. Piles may be more cost effective than shafts where pile cap construction is relatively easy, where the depth to the foundation layer is large (e.g., more than 100 feet), or where the pier loads are such that multiple shafts per column, requiring a shaft cap, are needed. The tendency of the upper loose soils to flow, requiring permanent shaft casing, may also be a consideration that could make pile foundations more cost effective. Artesian pressure in the bearing layer could preclude the use of drilled shafts due to the difficulty in keeping enough head inside the shaft during excavation to prevent heave or caving under slurry.

For situations where existing structures must be retrofitted to improve foundation resistance or where limited headroom is available, micro-piles may be the best alternative, and should be considered.

Augercast piles can be very cost effective in certain situations. However, their ability to resist lateral loads is minimal, making them undesirable to support structures where significant lateral loads must be transferred to the foundations. Furthermore, quality assurance of augercast pile integrity and capacity needs further development. Therefore, it is WSDOT policy not to use augercast piles for bridge foundations.

8.5 Overview of LRFD for Foundations

The basic equation for load and resistance factor design (LRFD) states that the loads multiplied by factors to account for uncertainty, ductility, importance, and redundancy must be less than or equal to the available resistance multiplied by factors to account for variability and uncertainty in the resistance per the AASHTO LRFD Bridge Design Specifications. The basic equation, therefore, is as follows:

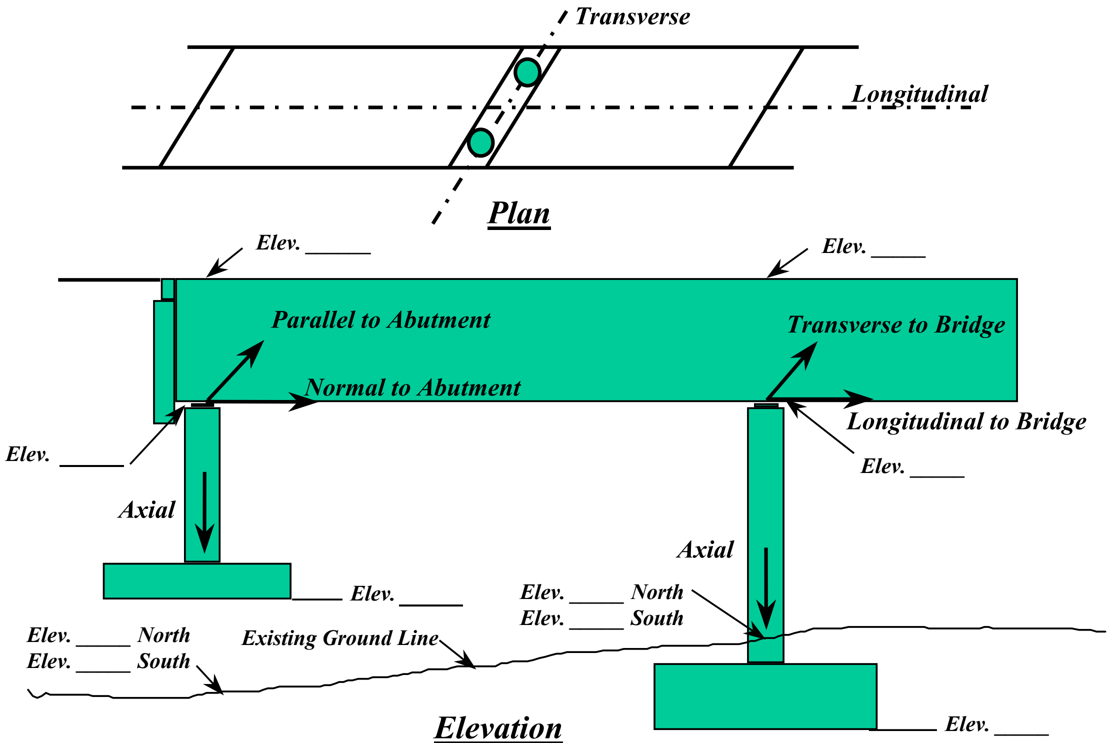
$$\Sigma\eta_\iota\gamma_i Q_i \leq \phi R_n \qquad (8\text{-}1)$$

Where:

η_ι = Factor for ductility, redundancy, and importance of structure
γ_i = Load factor applicable to the i'th load Q_i
Q_i = Load
ϕ = Resistance factor
R_n = Nominal (predicted) resistance

For typical WSDOT practice, η_i should be set equal to 1.0 for use of both minimum and maximum load factors. Foundations shall be proportioned so that the factored resistance is not less than the factored loads.

Figure 8-2 below should be utilized to provide a common basis of understanding for loading locations and directions for substructure design. This figure also indicates the geometric data required for abutment and substructure design. Note that for shaft and some pile foundation designs, the shaft or pile may form the column as well as the foundation element, thereby eliminating the footing element shown in the figure.



Template for Foundation Site Data and Loading Direction Definitions
Figure 8-2

8.6 LRFD Loads, Load Groups and Limit States to be Considered

The specific loads and load factors to be used for foundation design are as found in AASHTO LRFD Bridge Design Specifications and the *LRFD Bridge Design Manual* (BDM).

8.6.1 Foundation Analysis to Establish Load Distribution for Structure

Once the applicable loads and load groups for design have been established for each limit state, the loads shall be distributed to the various parts of the structure in accordance with Sections 3 and 4 of the AASHTO LRFD Bridge Design Specifications. The distribution of these loads shall consider the deformation characteristics of the soil/rock, foundation, and superstructure. The following process is used to accomplish the load distribution (see LRFD BDM Section 7.2 for more detailed procedures):

1. Establish stiffness values for the structure and the soil surrounding the foundations and behind the abutments.
2. For service and strength limit state calculations, use P-Y curves for deep foundations, or use strain wedge theory, especially in the case of short or

intermediate length shafts (see Section 8.13.2.3.3), to establish soil/rock stiffness values (i.e., springs) necessary for structural design. The bearing resistance at the specified settlement determined for the service limit state, but excluding consolidation settlement, should be used to establish soil stiffness values for spread footings for service and strength limit state calculations. For strength limit state calculations for deep foundations where the lateral load is potentially repetitive in nature (e.g., wind, water, braking forces, etc.), use soil stiffness values derived from P-Y curves using non-degraded soil strength and stiffness parameters. The geotechnical designer provides the soil/rock input parameters to the structural designer to develop these springs and to determine the load distribution using the analysis procedures as specified in LRFD BDM Section 7.2 and Section 4 of the AASHTO LRFD Bridge Design Specifications, applying unfactored loads, to get the load distribution. Two unfactored load distributions for service and strength limit state calculations are developed: one using undegraded stiffness parameters (i.e., maximum stiffness values) to determine the maximum shear and moment in the structure, and another distribution using soil strength and stiffness parameters that have been degraded over time due to repetitive loading to determine the maximum deflections and associated loads that result.

3. For extreme event limit state (seismic) deep foundation calculations, use soil strength and stiffness values before any liquefaction or other time dependent degradation occurs to develop lateral soil stiffness values and determine the unfactored load distribution to the foundation and structure elements as described in Step 2, including the full seismic loading. This analysis using maximum stiffness values for the soil/rock is used by the structural designer to determine the maximum shear and moment in the structure. The structural designer then completes another unfactored analysis using soil parameters degraded by liquefaction effects to get another load distribution, again using the full seismic loading, to determine the maximum deflections and associated loads that result. For footing foundations, a similar process is followed, except the vertical soil springs are bracketed to evaluate both a soft response and a stiff response. See Section 6.4.2.7 for additional information on this design issue.

4. Once the load distributions have been determined, the loads are factored to analyze the various components of the foundations and structure for each limit state. The structural and geotechnical resistance are factored as appropriate, but in all cases, the lateral soil resistance for deep foundations remain unfactored (i.e., a resistance factor of 1.0).

Throughout all of the analysis procedures discussed above to develop load distributions, the soil parameters and stiffness values are unfactored. The geotechnical designer must develop a best estimate for these parameters during the modeling. Use of intentionally conservative values could result in unconservative estimates of structure loads, shears, and moments or inaccurate estimates of deflections.

See the AASHTO LRFD Bridge Design Specifications, Article 10.6 for the development of elastic settlement/bearing resistance of footings for static analyses and Chapter 6 for soil/rock stiffness determination for spread footings subjected to seismic loads. See Sections 8.12.2.3 and 8.13.2.3.3, and related AASHTO LRFD Bridge Design Specifications for the development of lateral soil stiffness values for deep foundations.

8.6.2 Downdrag Loads

Regarding downdrag loads, possible development of downdrag on piles, shafts, or other deep foundations shall be evaluated where:

- Sites are underlain by compressible material such as clays, silts or organic soils,
- Fill will be or has recently been placed adjacent to the piles or shafts, such as is frequently the case for bridge approach fills,
- The groundwater is substantially lowered, or
- Liquefaction of loose sandy soil can occur.

Downdrag loads (DD) shall be determined, factored (using load factors), and applied as specified in the AASHTO LRFD Bridge Design Specifications, Section 3. The load factors for DD loads provided in Table 3.4.1-2 of the AASHTO LRFD Bridge Design Specifications shall be used for the strength limit state. This table does not address the situation in which the soil contributing to downdrag in the strength limit state consists of sandy soil, the situation in which a significant portion of the soil profile consists of sandy layers, nor the situation in which the CPT is used to estimate DD and the pile bearing resistance. Therefore, the portion of Table 3.4.1-2 in the AASHTO LRFD Bridge Design Specifications that addresses downdrag loads has been augmented to address these situations as shown in Table 8-3.

Type of Load, Foundation Type, and Method Used to Calculate Downdrag		Load Factor	
		Maximum	Minimum
DD: Downdrag	Piles, α Tomlinson Method	1.4	0.25
	Piles, λ Method	1.05	0.30
	Piles, Nordlund Method, or Nordlund and λ Method	1.1	0.35
	Piles, CPT Method	1.1	0.40
	Drilled shafts, O'Neill and Reese (1999) Method	1.25	0.35

Strength Limit State Downdrag Load Factors
Table 8-3

For the Service and Extreme Event Limit states, a downdrag load factor of 1.0 should be used.

8.6.3 Uplift Loads due to Expansive Soils

In general, uplift loads on foundations due to expansive soils shall be avoided through removal of the expansive soil. If removal is not possible, deep foundations such as driven piles or shafts shall be placed into stable soil. Spread footings shall not be used in this situation.

Deep foundations penetrating expansive soil shall extend to a depth into moisture-stable soils sufficient to provide adequate anchorage to resist uplift. Sufficient clearance should be provided between the ground surface and underside of caps or beams connecting piles or shafts to preclude the application of uplift loads at the pile/cap connection due to swelling ground conditions.

Evaluation of potential uplift loads on piles extending through expansive soils requires evaluation of the swell potential of the soil and the extent of the soil strata that may affect the pile. One reasonably reliable method for identifying swell potential is presented in Chapter 5. Alternatively, ASTM D4829 may be used to evaluate swell potential. The thickness of the potentially expansive stratum must be identified by:

- Examination of soil samples from borings for the presence of jointing, slickensiding, or a blocky structure and for changes in color, and
- Laboratory testing for determination of soil moisture content profiles.

8.6.4 Soil Loads on Buried Structures

For tunnels, culverts and pipe arches, the soil loads to be used for design shall be as specified in Sections 3 and 12 of the AASHTO LRFD Bridge Design Specifications.

8.6.5 Service Limit States

Foundation design at the service limit state shall include:

- Settlements
- Horizontal movements
- Overall stability, and
- Scour at the design flood

Consideration of foundation movements shall be based upon structure tolerance to total and differential movements, rideability and economy. Foundation movements shall include all movement from settlement, horizontal movement, and rotation.

In bridges where the superstructure and substructure are not integrated, settlement corrections can be made by jacking and shimming bearings. Article 2.5.2.3 of the AASHTO LRFD Bridge Design Specifications requires jacking provisions for these bridges. The cost of limiting foundation movements should be compared with the cost of designing the superstructure so that it can tolerate larger movements or of correcting the consequences of movements through maintenance to determine minimum lifetime cost. WSDOT may establish criteria that are more stringent.

The design flood for scour is defined in Article 2.6.4.4.2 and is specified in Article 3.7.5 of the AASHTO LRFD Bridge Design Specifications as applicable at the service limit state.

8.6.5.1 Tolerable Movements

Foundation settlement, horizontal movement, and rotation of foundations shall be investigated using all applicable loads in the Service I Load Combination specified in Table 3.4.1-1 of the AASHTO LRFD Bridge Design Specifications. Transient loads may be omitted from settlement analyses for foundations bearing on or in cohesive soil deposits that are subject to time-dependent consolidation settlements.

Foundation movement criteria shall be consistent with the function and type of structure, anticipated service life, and consequences of unacceptable movements on structure performance. Foundation movement shall include vertical, horizontal and rotational movements. The tolerable movement criteria shall be established by either empirical procedures or structural analyses or by consideration of both.

Experience has shown that bridges can and often do accommodate more movement and/or rotation than traditionally allowed or anticipated in design. Creep, relaxation, and redistribution of force effects accommodate these movements. Some studies have been made to synthesize apparent response. These studies indicate that angular distortions between adjacent foundations greater than 0.008 (RAD) in simple spans and 0.004 (RAD) in continuous spans should not be permitted in settlement criteria (Moulton et al. 1985; DiMillio, 1982; Barker et al. 1991). Other angular distortion limits may be appropriate after consideration of:

- Cost of mitigation through larger foundations, realignment or surcharge,
- Rideability,
- Aesthetics, and,
- Safety.

In addition to the requirements for serviceability provided above, the following criteria (Tables 8-4, 8-5, and 8-6) shall be used to establish acceptable settlement criteria:

Total Settlement at Pier or Abutment	Differential Settlement Over 100 Feet within Pier or Abutment, and Differential Settlement Between Piers	Action
$\Delta H \leq 1$ in	$\Delta H_{100} \leq 0.75$ in	Design and Construct
1 in $< \Delta H \leq 4$ in	0.75 in $< \Delta H_{100} \leq 3$ in	Ensure structure can tolerate settlement
$\Delta H > 4$ in	$\Delta H_{100} > 3$ in	Obtain Approval[1] prior to proceeding with design and Construction

[1]Approval of WSDOT State Geotechnical Engineer and WSDOT Bridge Design Engineer required.

Settlement Criteria for Bridges
Table 8-4

Total Settlement	Differential Settlement Over 100 Feet	Action
$\Delta H \leq 1$ in	$\Delta H_{100} \leq 0.75$ in	Design and Construct
1 in $< \Delta H \leq 2.5$ in	0.75 in $< \Delta H_{100} \leq 2$ in	Ensure structure can tolerate settlement
$\Delta H > 2.5$ in	$\Delta H_{100} > 2$ in	Obtain Approval[1] prior to proceeding with design and Construction

[1]Approval of WSDOT State Geotechnical Engineer and WSDOT Bridge Design Engineer required.

Settlement Criteria for Cut and Cover Tunnels, Concrete Culverts (including box culverts), and Concrete Pipe Arches
Table 8-5

Total Settlement	Differential Settlement Over 100 Feet	Action
$\Delta H \leq 2$ in	$\Delta H_{100} \leq 1.5$ in	Design and Construct
2 in < $\Delta H \leq 6$ in	1.5 in < $\Delta H_{100} \leq 5$ in	Ensure structure can tolerate settlement
$\Delta H > 6$ in	$\Delta H_{100} > 5$ in	Obtain Approval[1] prior to proceeding with design and Construction

[1]Approval of WSDOT State Geotechnical Engineer and WSDOT Bridge Design Engineer required.

Settlement Criteria for Flexible Culverts
Table 8-6

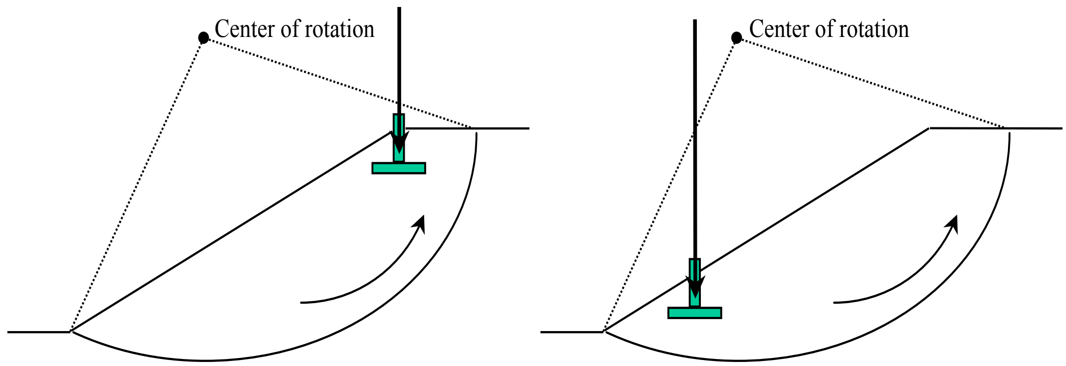
Rotation movements should be evaluated at the top of the substructure unit (in plan location) and at the deck elevation.

The horizontal displacement of pile and shaft foundations shall be estimated using procedures that consider soil-structure interaction (see Section 8.12.2.3). Horizontal movement criteria should be established at the top of the foundation based on the tolerance of the structure to lateral movement, with consideration of the column length and stiffness. Tolerance of the superstructure to lateral movement will depend on bridge seat widths, bearing type(s), structure type, and load distribution effects.

8.6.5.2 Overall Stability

The evaluation of overall stability of earth slopes with or without a foundation unit shall be investigated at the service limit state as specified in Article 11.6.2.3 of the AASHTO LRFD Bridge Design Specifications. Overall stability should be evaluated using limiting equilibrium methods such as modified Bishop, Janbu, Spencer, or other widely accepted slope stability analysis methods. Article 11.6.2.3 recommends that overall stability be evaluated at the Service I limit state (i.e., a load factor of 1.0) and a resistance factor, ϕ_{os} of 0.65 for slopes which support a structural element. For resistance factors for overall stability of slopes that contain a retaining wall, see Chapter 15. Also see Chapter 7 for additional information and requirements regarding slope stability analysis and acceptable safety factors and resistance factors.

Available slope stability programs produce a single factor of safety, FS. Overall slope stability shall be checked to insure that foundations designed for a maximum bearing stress equal to the specified service limit state bearing resistance will not cause the slope stability factor of safety to fall below 1.5. This practice will essentially produce the same result as specified in Article 11.6.2.3 of the AASHTO LRFD Bridge Design Specifications. The foundation loads should be as specified for the Service I limit state for this analysis. If the foundation is located on the slope such that the foundation load contributes to slope instability, the designer shall establish a maximum footing load that is acceptable for maintaining overall slope stability for Service, and Extreme Event limit states (see Figure 8-3 for example). If the foundation is located on the slope such that the foundation load increases slope stability, overall stability of the slope shall be evaluated ignoring the effect of the footing on slope stability, or the foundation load shall be included in the slope stability analysis and the foundation designed to resist the lateral loads imposed by the slope.



Example Where Footing Contributes to Instability of Slope (Left Figure) VS. Example Where Footing Contributes to Stability of Slope (Right Figure)
Figure 8-3

If the slope is found to not be adequately stable, the slope shall be stabilized so that it achieves the required level of safety, or the structure foundation and the structure itself shall be designed to resist the additional load. Loads on foundations due to forces caused by slope instability shall be determined in accordance with Liang (2010) or Vessely, et al. (2007) and Yamasaki, et al. (2013). The load on the deep foundation unit and/or structure shall be determined such that the required level of safety for the slope is achieved. The required level of safety for slope is an FS of 1.5 (or resistance factor of 0.65) for slope instability that can impact a structure, per the AASHTO LRFD Bridge Design Specifications, Articles 10.5.2.3 and 11.6.2.3, designed at the service limit state. For the Extreme Event Limit State, the required minimum level of safety is a FS of 1.1 (resistance factor of 0.9).

8.6.5.3 Abutment Transitions

Vertical and horizontal movements caused by embankment loads behind bridge abutments shall be investigated. Settlement of foundation soils induced by embankment loads can result in excessive movements of substructure elements. Both short and long term settlement potential should be considered.

Settlement of improperly placed or compacted backfill behind abutments can cause poor rideability and a possibly dangerous bump at the end of the bridge. Guidance for proper detailing and material requirements for abutment backfill is provided in Samtani and Nowatzki (2006) and should be followed.

Lateral earth pressure behind and/or lateral squeeze below abutments can also contribute to lateral movement of abutments and should be investigated, if applicable.

In addition to the considerations for addressing the transition between the bridge and the abutment fill provided above, an approach slab shall be provided at the end of each bridge for WSDOT projects, and shall be the same width as the bridge deck. However, the slab may be deleted under certain conditions as described herein and as described in *Design Manual* M22-01, Chapter 720. If approach slabs are to be deleted, a geotechnical and structural evaluation is required. The geotechnical and structural evaluation shall consider, as a minimum, the criteria described below.

1. Approach slabs may be deleted for geotechnical reasons if the following geotechnical considerations are met:
   - If settlements are excessive, resulting in the angular distortion of the slab to be great enough to become a safety problem for motorists, with excessive defined as a differential settlement between the bridge and the approach fill of 8 inches or more, or,
   - If creep settlement of the approach fill will be less than 0.5 inch, and the amount of new fill placed at the approach is less than 20 feet, or
   - If approach fill heights are less than 8 feet, or
   - If more than 2 inches of differential settlement could occur between the centerline and shoulder
2. Other issues such as design speed, average daily traffic (ADT) or accommodation of certain bridge structure details may supersede the geotechnical reasons for deleting the approach slabs.

8.6.6 Strength Limit States

Design of foundations at strength limit states shall include evaluation of the nominal geotechnical and structural resistances of the foundation elements as specified in the AASHTO LRFD Bridge Design Specifications Article 10.5.

8.6.7 Extreme Event Limit States

Foundations shall be designed for extreme events as applicable in accordance with the AASHTO LRFD Bridge Design Specifications.

8.7 Resistance Factors for Foundation Design – Design Parameters

The load and resistance factors provided herein result from a combination of design model uncertainty, soil/rock property uncertainty, and unknown uncertainty assumed by the previous allowable stress design and load factor design approach included in previous AASHTO design specifications. Therefore, the load and resistance factors account for soil/rock property uncertainty in addition to other uncertainties.

It should be assumed that the characteristic soil/rock properties to be used in conjunction with the load and resistance factors provided herein that have been calibrated using reliability theory (see Allen, 2005) are average values obtained from laboratory test results or from correlated field in-situ test results. It should be noted that use of lower bound soil/rock properties could result in overly conservative foundation designs in such cases. However, depending on the availability of soil or rock property data and the variability of the geologic strata under consideration, it may not be possible to reliably estimate the average value of the properties needed for design. In such cases, the geotechnical designer may have no choice but to use a more conservative selection of design parameters to mitigate the additional risks created by potential variability or the paucity of relevant data. Regarding the extent of subsurface characterization and the number of soil/rock property tests required to justify use of the load and resistance factors provided herein, see Chapter 5. For those load and resistance factors determined primarily from calibration by fitting to allowable stress design, this property selection issue is not relevant, and property selection should be based on past practice. For information regarding the derivation of load and resistance factors for foundations, (see Allen, 2005).

8.8 Resistance Factors for Foundation Design – Service Limit States

Resistance factors for the service limit states shall be taken as specified in the AASHTO LRFD Bridge Design Specifications Article 10.5 (most current version).

8.9 Resistance Factors for Foundation Design – Strength Limit States

Resistance factors for the strength limit states for foundations shall be taken as specified in the AASHTO LRFD Bridge Design Specifications Article 10.5 (most current version). Regionally specific values may be used in lieu of the specified resistance factors, but should be determined based on substantial statistical data combined with calibration or substantial successful experience to justify higher values. Smaller resistance factors should be used if site or material variability is anticipated to be unusually high or if design assumptions are required that increase design uncertainty that have not been mitigated through conservative selection of design parameters.

Exceptions with regard to the resistance factors provided in the most current version of AASHTO for the strength limit state are as follows:

- For driven pile foundations, if the WSDOT driving formula is used for pile driving construction control, the resistance factor φ_{dyn} shall be equal to 0.55 (end of driving conditions only). This resistance factor does not apply to beginning of redrive conditions. See Allen (2005b and 2007) for details on the derivation of this resistance factor.
- For driven pile foundations, when using Wave Equation analysis to estimate pile bearing resistance and establish driving criteria, a resistance factor of 0.50 may be used if the hammer performance is field verified. Field verification of hammer performance includes direct measurement of hammer stroke or ram kinetic energy (e.g., ram velocity measurement). The wave equation may be used for either end of drive or beginning of redrive pile bearing resistance estimation.
- For drilled shaft foundations, the requirements in Appendix 8-B shall be met. This appendix essentially provides an update to the AASHTO LRFD drilled shaft design specifications approved by the AASHTO Bridge Subcommittee in June 2013. These new specifications shall be used until the final drilled shaft AASHTO specifications are published in the next edition of the AASHTO LRFD Bridge Design Specifications.

All other resistance factor considerations and limitations provided in the AASHTO LRFD Bridge Design Specifications Article 10.5 shall be considered applicable to WSDOT design practice.

8.10 Resistance Factors for Foundation Design – Extreme Event Limit States

Design of foundations at extreme event limit states shall be consistent with the expectation that structure collapse is prevented and that life safety is protected.

8.10.1 Scour

The resistance factors and their application shall be as specified in the AASHTO LRFD Bridge Design Specifications, Article 10.5.

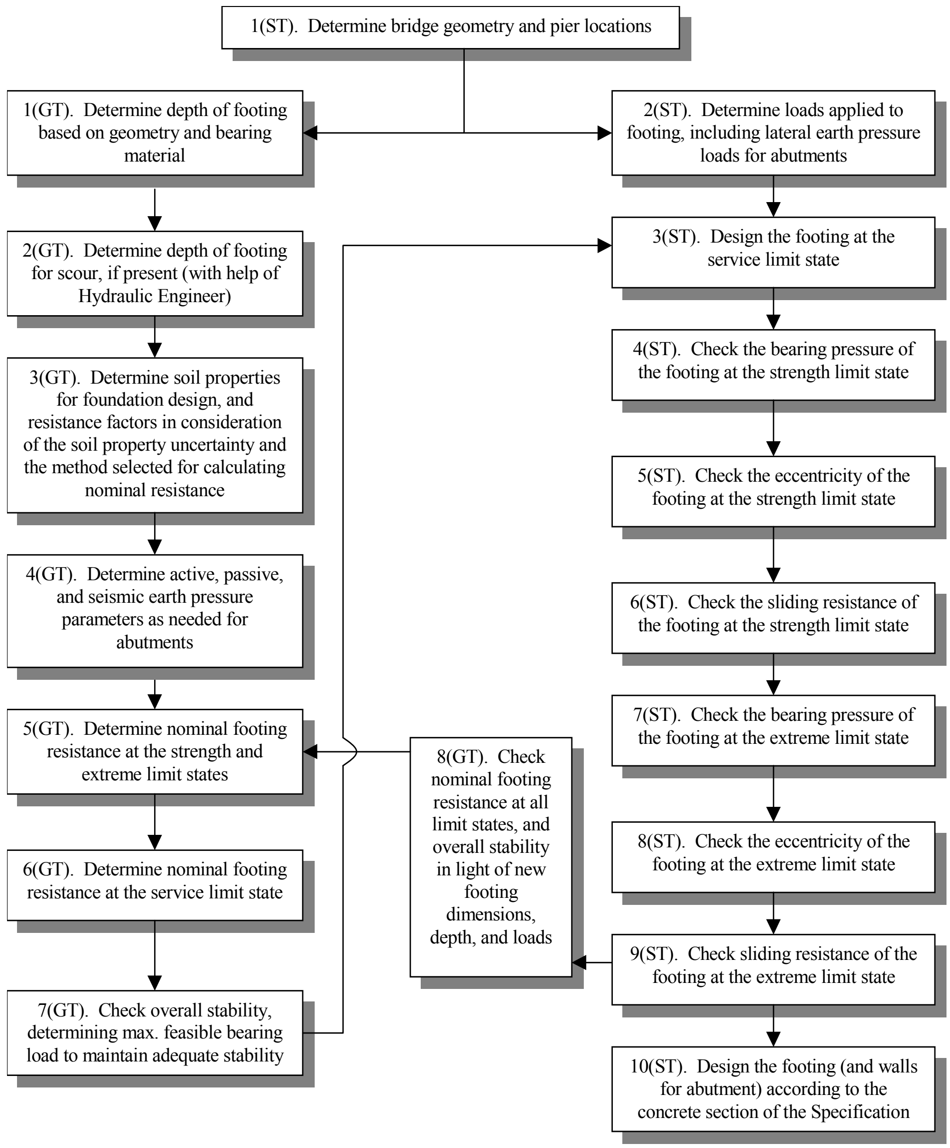
8.10.2 Other Extreme Event Limit States

Resistance factors for extreme event limit states, including the design of foundations to resist earthquake, ice, vehicle or vessel impact loads, shall be taken as 1.0, with the exception of bearing resistance of footing foundations. Since the load factor used for the seismic lateral earth pressure for EQ is currently 1.0, to obtain the same level of safety obtained from the AASHTO Standard Specification design requirements for sliding and bearing, a resistance factor of slightly less than 1.0 is required. For bearing resistance during seismic loading, a resistance factor of 0.90 should be used. For uplift resistance of piles and shafts, the resistance factor shall be taken as 0.80 or less, to account for the difference between compression skin friction and tension skin friction.

Regarding overall stability of slopes that can affect structures, a resistance factor of 0.9, which is equivalent to a factor of safety of 1.1, should in general be used for the extreme event limit state. Section 6.4.3 and Chapter 7 provide additional information and requirements regarding seismic stability of slopes.

8.11 Spread Footing Design

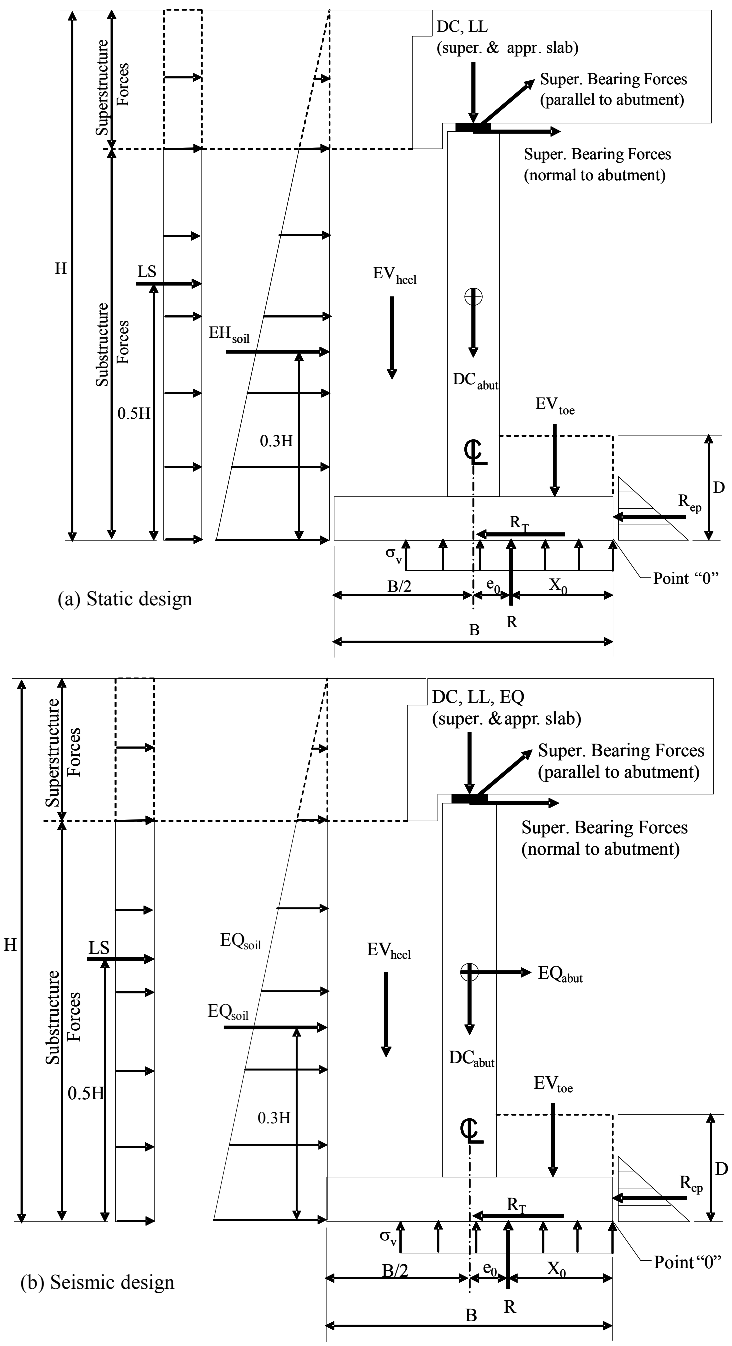
Figure 8-4 provides a flowchart that illustrates the design process, and interaction required between structural and geotechnical engineers, needed to complete a spread footing design. ST denotes steps usually completed by the Structural Designer, while GT denotes those steps normally completed by the geotechnical designer.



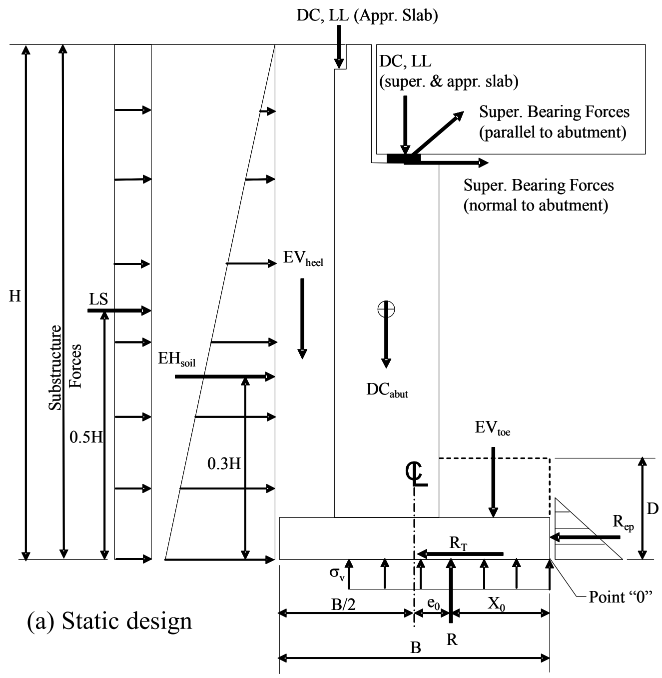
Flowchart for LRFD Spread Footing Design
Figure 8-4

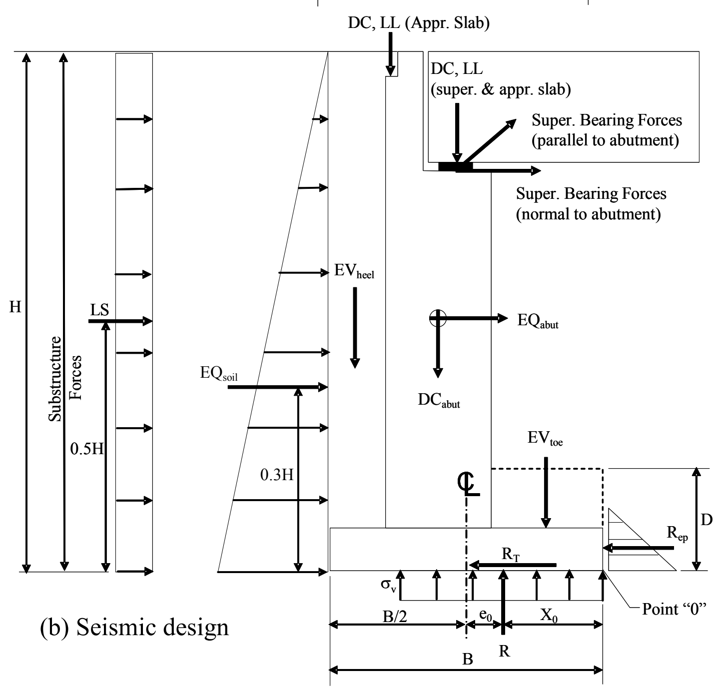
8.11.1 Loads and Load Factor Application to Footing Design

Figures 8-5 and 8-6 provide definitions and locations of the forces and moments that act on structural footings. Note that the eccentricity used to calculate the bearing stress in geotechnical practice typically is referenced to the centerline of the footing, whereas the eccentricity used to evaluate overturning typically is referenced to point O at the toe of the footing. It is important to not change from maximum to minimum load factors in consideration of the force location relative to the reference point used (centerline of the footing, or point "O" at the toe of the footing), as doing so will cause basic statics to no longer apply, and one will not get the same resultant location when the moments are summed at different reference points. The AASHTO LRFD Bridge design Specifications indicate that the moments should be summed about the center of the footing. Table 8-7 identifies when to use maximum or minimum load factors for the various modes of failure for the footing (bearing, overturning, and sliding) for each force, for the strength limit state.



Definition and location of forces for stub abutments
Figure 8-5





Definition and location of forces for L-abutments and interior footings
Figure 8-6

The variables shown in Figures 8-5 and 8-6 are defined as follows:

DC, LL, EQ = vertical structural loads applied to footing/wall (dead load, live load, EQ load, respectively)
DC_{abut} = structure load due to weight of abutment
EQ_{abut} = abutment inertial force due to earthquake loading
EV_{heel} = vertical soil load on wall heel
EV_{toe} = vertical soil load on wall toe
EH_{soil} = lateral load due to active or at rest earth pressure behind abutment
LS = lateral earth pressure load due to live load
EQ_{soil} = lateral load due to combined effect of active or at rest earth pressure plus seismic earth pressure behind abutment
R_{ep} = ultimate soil passive resistance (note: height of pressure distribution triangle is determined by the geotechnical engineer and is project specific)
$R\tau$ = soil shear resistance along footing base at soil-concrete interface
σ_v = resultant vertical bearing stress at base of footing
R = resultant force at base of footing
e_o = eccentricity calculated about point O (toe of footing)
X_o = distance to resultant R from wall toe (point O)
B = footing width
H = total height of abutment plus superstructure thickness

	Load Factor		
Load	**Sliding**	**Overturning, e_o**	**Bearing Stress (e_c, σ_v)**
DC, DC_{abut}	Use min. load factor	Use min. load factor	Use max. load factor
LL, LS	Use transient load factor (e.g., LL)	Use transient load factor (e.g., LL)	Use transient load factor (e.g., LL)
EV_{heel}, EV_{toe}	Use min. load factor	Use min. load factor	Use max. load factor
EH_{soil}	Use max. load factor	Use max. load factor	Use max. load factor

Selection of Maximum or Minimum Spread Footing Foundation Load Factors for Various Modes of Failure for the Strength Limit State
Table 8-7

8.11.2 Footing Foundation Design

Geotechnical design of footings, and all related considerations, shall be conducted as specified in the AASHTO LRFD Bridge Design Specifications Article 10.6 (most current version), except as specified in following paragraphs and sections.

8.11.2.1 Footing Bearing Depth

For footings on slopes, such as at bridge abutments, the footings should be located as shown in the LRFD BDM Section 7.7.1. The footing should also be located to meet the minimum cover requirements provided in LRFD BDM Section 7.7.1.

8.11.2.2 Nearby Structures

Where foundations are placed adjacent to existing structures, the influence of the existing structure on the behavior of the foundation and the effect of the foundation on the existing structures shall be investigated. Issues to be investigated include, but are not limit to, settlement of the existing structure due to the stress increase caused by the new footing, decreased overall stability due to the additional load created by the new footing, and the effect on the existing structure of excavation, shoring, and/or dewatering to construct the new foundation.

8.11.2.3 Service Limit State Design of Footings

Footing foundations shall be designed at the service limit state to meet the tolerable movements for the structure in accordance with Section 8.6.5.1. The nominal unit bearing resistance at the service limit state, q_{serve}, shall be equal to or less than the maximum bearing stress that that results in settlement that meets the tolerable movement criteria for the structure in Section 8.6.5.1, calculated in accordance with the AASHTO LRFD Bridge Design Specifications, and shall also be less than the maximum bearing stress that meets overall stability requirements.

Other factors that may affect settlement, e.g., embankment loading and lateral and/or eccentric loading, and for footings on granular soils, vibration loading from dynamic live loads should also be considered, where appropriate. For guidance regarding settlement due to vibrations, see Lam and Martin (1986) or Kavazanjian, et al., (1997).

8.11.2.3.1 Settlement of Footings on Cohesionless Soils

Based on experience (see also Kimmerling, 2002), the Hough method tends to overestimate settlement of dense sands, and underestimate settlement of very loose silty sands and silts. Kimmerling (2002) reports the results of full scale studies where on average the Hough Method (Hough, 1959) overestimated settlement by an average factor of 1.8 to 2.0, though some of the specific cases were close to 1.0. This does not mean that estimated settlements by this method can be reduced by a factor of 2.0. However, based on successful WSDOT experience, for footings on sands and gravels with $N1_{60}$ of 20 blows/ft or more, or sands and gravels that are otherwise known to be overconsolidated (e.g., sands subjected to preloading or deep compaction), reduction of the estimated Hough settlement by up to a factor of 1.5 may be considered, provided the geotechnical designer has not used aggressive soil parameters to account for the Hough method's observed conservatism. The settlement characteristics of cohesive soils that exhibit plasticity should be investigated using undisturbed samples and laboratory consolidation tests as prescribed in the AASHTO LRFD Bridge Design Specifications.

8.11.2.3.2 Settlement of Footings on Rock

For footings bearing on fair to very good rock, according to the Geomechanics Classification system, as defined in Chapter 5, and designed in accordance with the provisions of this section, elastic settlements may generally be assumed to be less than 0.5 inches.

8.11.2.3.3 Bearing Resistance at the Service Limit State Using Presumptive Values

Regarding presumptive bearing resistance values for footings on rock, bearing resistance on rock shall be determined using empirical correlation the Geomechanic Rock Mass Rating System, RMR, as specified in Chapter 5.

8.11.2.4 Strength Limit State Design of Footings

The design of spread footings at the strength limit state shall address the following limit states:

- Nominal bearing resistance, considering the soil or rock at final grade, and considering scour as specified in the AASHTO LRFD Bridge Design Specifications Section 10:
- Overturning or excessive loss of contact; and
- Sliding at the base of footing.

The LRFD Bridge Design Manual allows footings to be inclined on slopes of up to 6H:1V. Footings with inclined bases steeper than this should be avoided wherever possible, using stepped horizontal footings instead. The maximum feasible slope of stepped footing foundations is controlled by the maximum acceptable stable slope for the soil in which the footing is placed. Where use of an inclined footing base must be used, the nominal bearing resistance determined in accordance with the provisions herein should be further reduced using accepted corrections for inclined footing bases in Munfakh, et al (2001).

8.11.2.4.1 Theoretical Estimation of Bearing Resistance

The footing bearing resistance equations provided in the AASHTO LRFD Bridge Design Specifications have no theoretical limit on the bearing resistance they predict. However, WSDOT limits the nominal bearing resistance for strength and extreme event limit states to 120 KSF on soil. Values greater than 120 KSF should not be used for foundation design in soil.

8.11.2.4.2 Plate Load Tests for Determination of Bearing Resistance in Soil

The nominal bearing resistance may be determined by plate load tests, provided that adequate subsurface explorations have been made to determine the soil profile below the foundation. The nominal bearing resistance determined from a plate load test may be extrapolated to adjacent footings where the subsurface profile is confirmed by subsurface exploration to be similar.

Plate load tests have a limited depth of influence and furthermore may not disclose the potential for long-term consolidation of foundation soils. Scale effects shall be addressed when extrapolating the results to performance of full scale footings. Extrapolation of the plate load test data to a full scale footing should be based on the design procedures provided herein for settlement (service limit state) and bearing resistance (strength and extreme event limit state), with consideration to the effect of the stratification (i.e., layer thicknesses, depths, and properties). Plate load test results should be applied only within a sub-area of the project site for which the subsurface conditions (i.e., stratification, geologic history, properties) are relatively uniform.

8.11.2.4.3 Bearing Resistance of Footings on Rock

For design of bearing of footings on rock, the competency of the rock mass should be verified using the procedures for RMR rating in Chapter 5.

8.11.2.5 Extreme Event Limit State Design of Footings

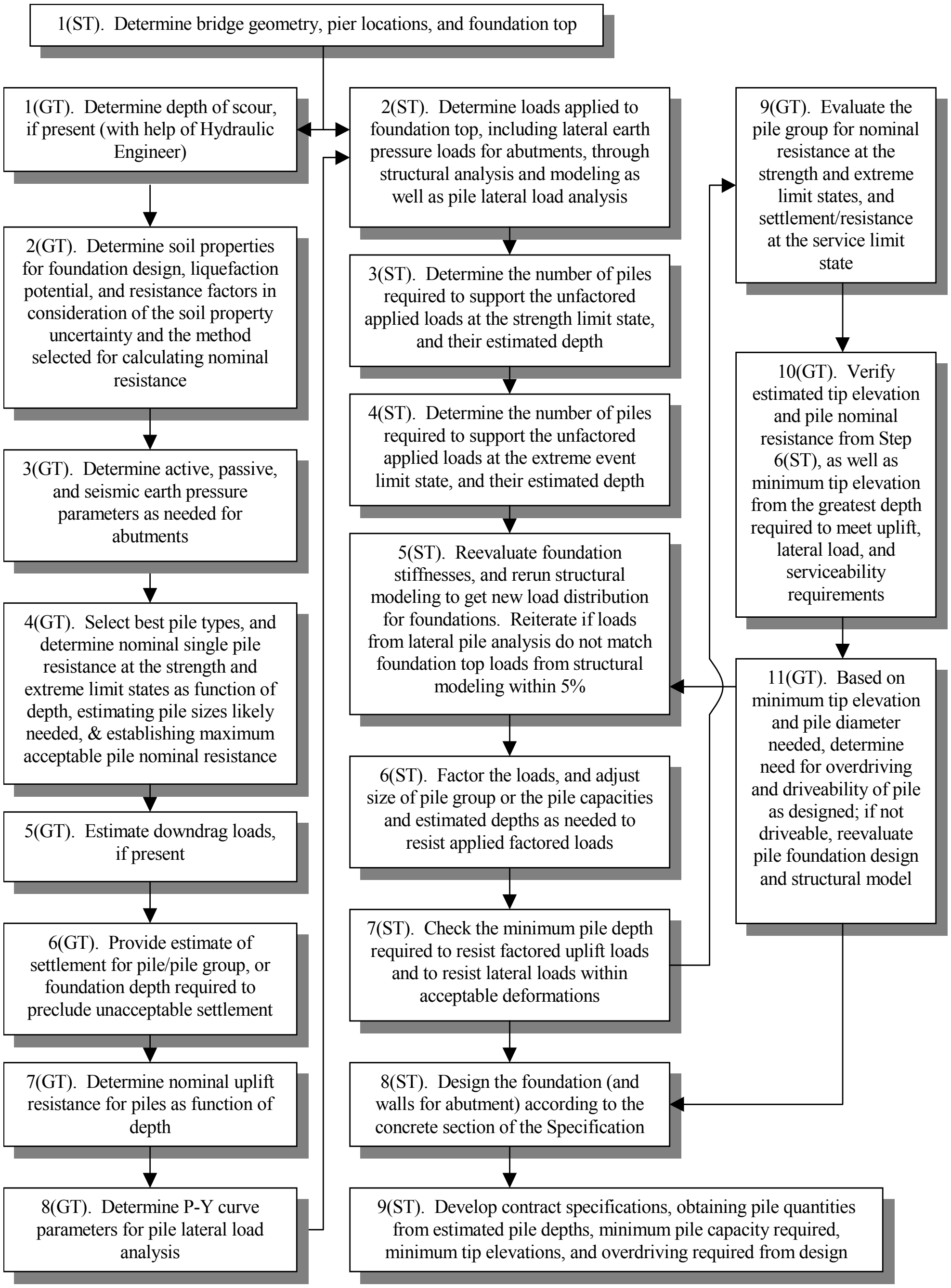
Footings shall not be located on or within liquefiable soil. Footings may be located on liquefiable soils that have been improved through densification or other means so that they do not liquefy. Footings may also be located above liquefiable soil in a non-liquefiable layer if the footing is designed to meet all Extreme Event limit states. In this case, liquefied soil parameters shall be used for the analysis (see Chapter 6). The footing shall be stable against an overall stability failure of the soil (see Section 8.6.5.2) and lateral spreading resulting from the liquefaction (see Chapter 6).

Footings located above liquefiable soil but within a non-liquefiable layer shall be designed to meet the bearing resistance criteria established for the structure for the Extreme Event Limit State. The bearing resistance of a footing located above liquefiable soils shall be determined considering the potential for a punching shear condition to develop, and shall also be evaluated using a two layer bearing resistance calculation conducted in accordance with the AASHTO LRFD Bridge Design Specifications Section 10.6, assuming the soil to be in a liquefied condition. Settlement of the liquefiable zone shall also be evaluated to determine if the extreme event limit state criteria for the structure the footing is supporting are met. Settlement due to liquefaction shall be evaluated as specified in Section 6.4.2.4.

For footings, whether on soil or on rock, the eccentricity of loading at the extreme limit state shall not exceed one-third (0.33) of the corresponding footing dimension, B or L, for γ_{EQ} = 0.0 and shall not exceed four-tenths (0.40) of the corresponding footing dimension, B or L, for γ_{EQ} = 1.0. If live loads act to reduce the eccentricity for the Extreme Event I limit state, γ_{EQ} shall be taken as 0.0.

8.12 Driven Pile Foundation Design

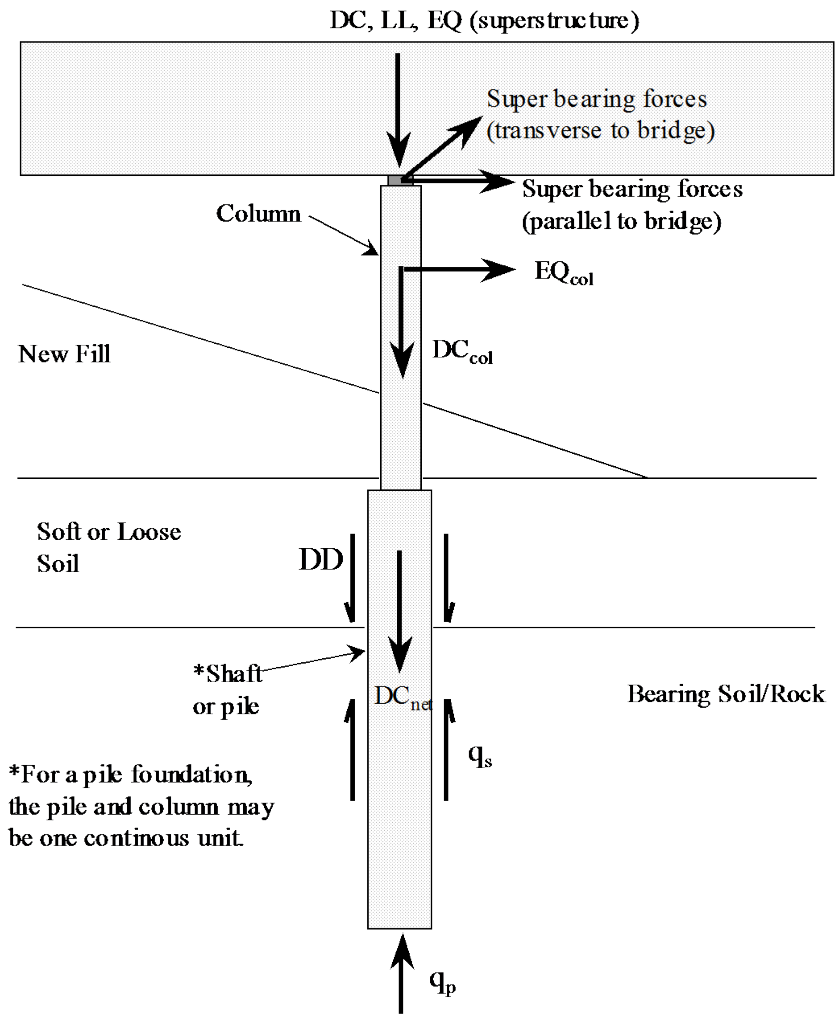
Figure 8-7 provides a flowchart that illustrates the design process, and interaction required between structural and geotechnical engineers, needed to complete a driven pile foundation design. ST denotes steps usually completed by the Structural Designer, while GT denotes those steps normally completed by the geotechnical designer.



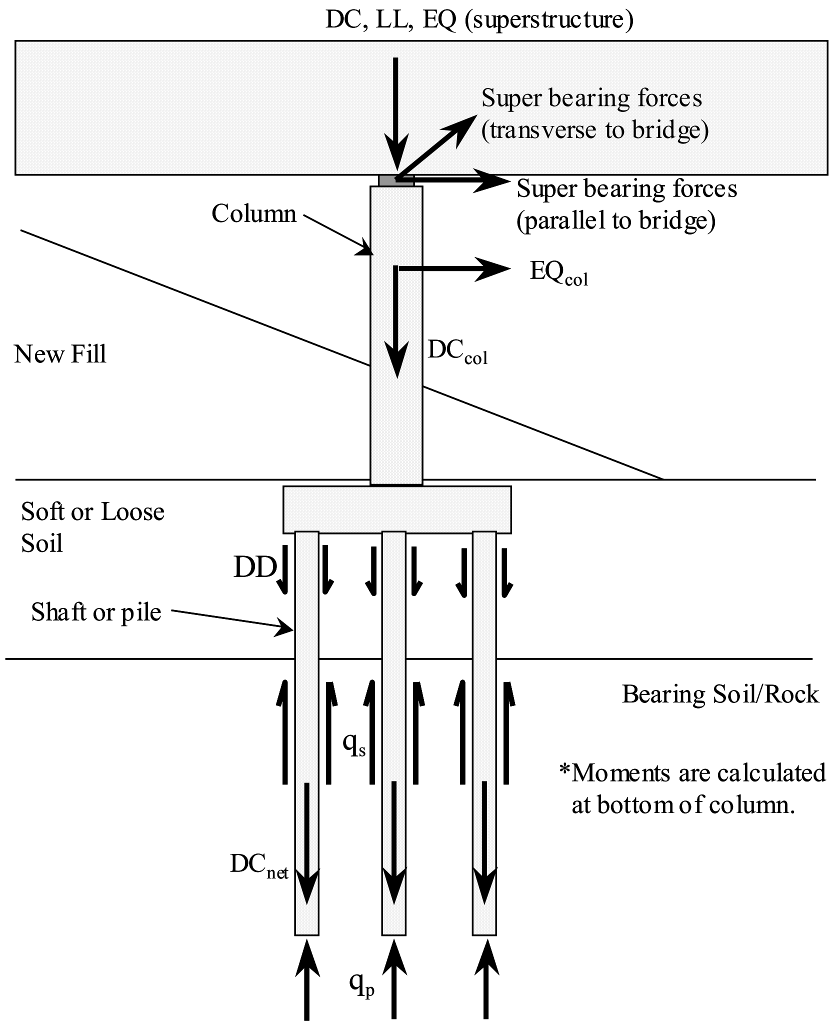
Design Flowchart for Pile Foundation Design
Figure 8-7

8.12.1 Loads and Load Factor Application to Driven Pile Design

Figures 8-8 and 8-9 provide definitions and typical locations of the forces and moments that act on deep foundations such as driven piles. Table 8-8 identifies when to use maximum or minimum load factors for the various modes of failure for the pile (bearing, uplift, and lateral loading) for each force, for the strength limit state.



Definition and Location of Forces for Integral Shaft Column or Pile Bent
Figure 8-8



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 down drag load on shaft (total load)

DC_{net} = unit weight of concrete in shaft minus unit weight of soil times the

shaft volume below the groundline (may include part of the column if the top of the shaft is deep due to scour or for other reasons

Definition and Location of Forces for Pile or Shaft Supported Footing
Figure 8-9

All other forces are as defined previously.

	Load Factor		
Load	**Bearing Stress**	**Uplift**	***Lateral Loading**
DC, DC_{col}	Use max. load factor	Use min. load factor	Use max load factor
LL	Use transient load factor (e.g., LL)	Use transient load factor (e.g., LL)	Use transient load factor (e.g., LL)
DC_{net}	Use max. load factor	Use min. load factor	N/A
DD	Use max. load factor	Treat as resistance, and use resistance factor for uplift	N/A

*Use unfactored loads to get force distribution in structure, then factor the resulting forces for final structural design.

Selection of Maximum or Minimum Deep Foundation Load Factors for Various Modes of Failure for the Strength Limit State
Table 8-8

All forces and load factors are as defined previously.

The loads and load factors to be used in pile foundation design shall be as specified in Section 3 of the AASHTO LRFD Bridge Design Specifications. Computational assumptions that shall be used in determining individual pile loads are described in Section 4 of the AASHTO LRFD Bridge Design Specifications.

8.12.2 Driven for Pile Foundation Geotechnical Design

Geotechnical design of driven pile foundations, and all related considerations, shall be conducted as specified in the AASHTO LRFD Bridge Design Specifications Article 10.7 (most current version), except as specified in following paragraphs and sections:

8.12.2.1 Driven Pile Sizes and Maximum Resistances

In lieu of more detailed structural analysis, the general guidance on pile types, sizes, and nominal resistance values provided in Table 8-9 may be used to select pile sizes and types for analysis. The Geotechnical Office limits the maximum nominal pile resistance for 24 inch piles to 1500 KIPS and 18 inch piles to 1,000 KIPS, and may limit the nominal pile resistance for a given pile size and type driven to a given soil/rock bearing unit based on experience with the given soil/rock unit. Note that this 1500 KIP limit for 24 inch diameter piles applies to closed end piles driven to bearing on to glacially overconsolidated till or a similar geologic unit. Open-ended piles, or piles driven to less competent bearing strata, should be driven to a lower nominal resistance. The maximum resistance allowed in that given soil/rock unit may be increased by the WSDOT Geotechnical Office per mutual agreement with the Bridge and Structures Office if a pile load test is performed.

Nominal pile Resistance (KIPS)	Pile Type and Diameter (in.)			
	Closed End Steel Pipe/ Cast-in-Place Concrete Piles	*Precast, Prestressed Concrete Piles	Steel H-Piles	Timber Piles
120	-	-	-	See WSDOT Standard Specs.
240	-	-	-	See WSDOT Standard Specs.
330	12 in.	13 in.	-	-
420	14 in.	16 in.	12 in.	-
600	18 in. nonseismic areas, 24 in. seismic areas	18 in.	14 in.	-
900	24 in.	Project Specific	Project Specific	-

*Precast, prestressed concrete piles are generally not used for highway bridges, but are more commonly used for marine work.

Typical Pile Types and Sizes for Various Nominal Pile Resistance Values
Table 8-9

8.12.2.2 Minimum Pile Spacing

Center-to-center pile spacing should not be less than the greater of 30 IN or 2.5 pile diameters or widths. A center-to-center spacing of less than 2.5 pile diameters may be considered on a case-by-case basis, subject to the approval of the WSDOT State Geotechnical Engineer and Bridge Design Engineer.

8.12.2.3 Determination of Pile Lateral Resistance

Pile foundations are subjected to horizontal loads due to wind, traffic loads, bridge curvature, vessel or traffic impact and earthquake. The nominal resistance of pile foundations to horizontal loads shall be evaluated based on both soil/rock and structural properties, considering soil-structure interaction. Determination of the soil/rock parameters required as input for design using soil-structure interaction methodologies is presented in Chapter 5.

See Article 10.7.2.4 in the AASHTO LRFD Bridge Design Specifications for detailed requirements regarding the determination of lateral resistance of piles.

Empirical data for pile spacings less than 3 pile diameters is very limited. If, due to space limitations, a smaller center-to-center spacing is used, subject to the requirements in Section 8.12.2.2, based on extrapolation of the values of P_m in Article 10.7.2.4 of the AASHTO LRFD Bridge Design Specifications, the following values of P_m at a spacing of no less than 2D may be used:

- For Row 1, $P_m = 0.45$
- For Row 2, $P_m = 0.33$
- For Row 3, $P_m = 0.25$

These values were extrapolated by fitting curves to the AASHTO Article 10.7.2.4 P_m values. A similar technique should be used to interpolate to intermediate values of foundation element spacing.

8.12.2.4 Batter Piles

WSDOT design preference is to avoid the use of batter piles unless no other structural option is available.

8.12.2.5 Service Limit State Design of Pile Foundations

Driven pile foundations shall be designed at the service limit state to meet the tolerable movements for the structure being supported in accordance with Section 8.6.5.1.

Service limit state design of driven pile foundations includes the evaluation of settlement due to static loads, and downdrag loads if present, overall stability, lateral squeeze, and lateral deformation.

Lateral analysis of pile foundations is conducted to establish the load distribution between the superstructure and foundations for all limit states, and to estimate the deformation in the foundation that will occur due to those loads. This section only addresses the evaluation of the lateral deformation of the foundation resulting from the distributed loads.

8.12.2.5.1 Overall Stability

The provisions of Section 8.6.5.2 shall apply.

8.12.2.5.2 Horizontal Pile Foundation Movement

The horizontal movement of pile foundations shall be estimated using procedures that consider soil-structure interaction as specified in Section 8.12.2.3.

8.12.2.6 Strength Limit State Geotechnical Design of Pile Foundations

8.12.2.6.1 Nominal Axial Resistance Change after Pile Driving

Setup as it relates to the WSDOT dynamic formula is discussed further in Section 8.12.2.6.4(a) and Allen (2005b, 2007).

8.12.2.6.2 Scour

If a static analysis method is used to determine the final pile bearing resistance (i.e., a dynamic analysis method is not used to verify pile resistance as driven), the available bearing resistance, and the pile tip penetration required to achieve the desired bearing resistance, shall be determined assuming that the soil subject to scour is completely removed, resulting in no overburden stress at the bottom of the scour zone.

Pile design for scour is illustrated in Figure 8-11, where,

R_{scour} = skin friction which must be overcome during driving through scour zone (KIPS)

Q_p = $(\Sigma\gamma_i Q_i)$ = factored load per pile (KIPS)

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

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})

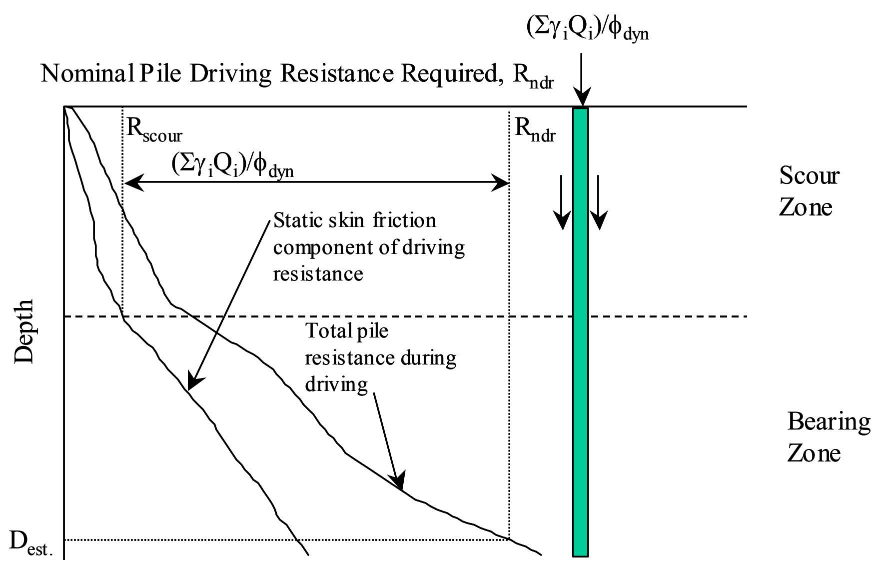
From Equation 8-1, the summation of the factored loads ($\Sigma\gamma_i Q_i$) must be less than or equal to the factored resistance (φR_n). Therefore, the nominal resistance R_n must be greater than or equal to the sum of the factored loads divided by the resistance factor φ. Hence, the nominal bearing resistance of the pile needed to resist the factored loads is therefore,

$$R_n = (\Sigma\gamma_i Q_i)/\phi_{dyn} \quad (8\text{-}2)$$

If dynamic pile measurements or dynamic pile formula are used to determine final pile bearing resistance during construction, the resistance that the piles are driven to must be adjusted to account for the presence of the soil in the scour zone. The total driving resistance, R_{ndr}, needed to obtain R_n, accounting for the skin friction that must be overcome during pile driving that does not contribute to the design resistance of the pile is as follows:

$$R_{ndr} = R_{scour} + R_n \quad (8\text{-}3)$$

Note that R_{scour} remains unfactored in this analysis to determine R_{ndr}.



Design of Pile Foundations for Scour

Figure 8-11

8.12.2.6.3 Downdrag

The foundation should be designed so that the available factored geotechnical resistance is greater than the factored loads applied to the pile, including the downdrag, at the strength limit state. The nominal pile resistance available to support structure loads plus downdrag shall be estimated by considering only the positive skin and tip resistance below the lowest layer contributing to the downdrag. The pile foundation shall be designed to structurally resist the downdrag plus structure loads.

Pile design for downdrag is illustrated in Figure 8-12,

Where:

R_{Sdd} = skin friction which must be overcome during driving through downdrag zone (KIPS)

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

DD= downdrag load per pile (KIPS)

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

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

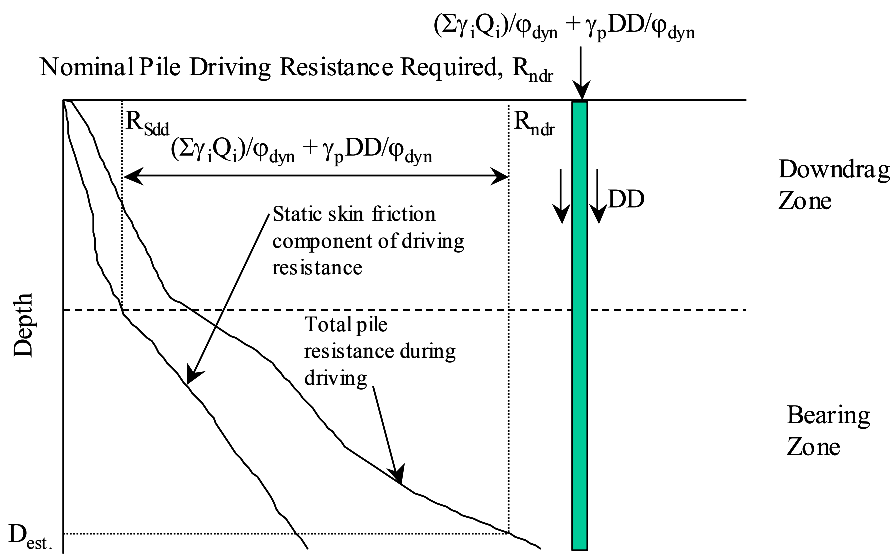
Similar to the derivation of Equation 8-2, the nominal bearing resistance of the pile needed to resist the factored loads, including downdrag, is therefore,

$$R_n = (\Sigma\gamma_i Q_i)/\phi_{dyn} + \gamma_p DD/\phi_{dyn} \quad (8\text{-}4)$$

The total nominal driving resistance, R_{ndr}, needed to obtain R_n, accounting for the skin friction that must be overcome during pile driving that does not contribute to the design resistance of the pile, is as follows:

$$R_{ndr} = R_{Sdd} + R_n \quad (8\text{-}5)$$

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



Design of Pile Foundations for Downdrag

Figure 8-12

In the instance where it is not possible to obtain adequate geotechnical resistance below the lowest layer contributing to downdrag (e.g., friction piles) to fully resist the downdrag, or if it is anticipated that significant deformation will be required to mobilize the geotechnical resistance needed to resist the factored loads including the downdrag load, the structure should be designed to tolerate the settlement resulting from the downdrag and the other applied loads in accordance with the AASHTO LRFD Bridge Design Specifications, Article 10.7.

The static analysis procedures in the AASHTO LRFD Bridge Design Specifications, Article 10.7 may be used to estimate the available pile resistance to withstand the downdrag plus structure loads to estimate pile lengths required to achieve the required bearing resistance. For this calculation, it should be assumed that the soil subject to downdrag still contributes overburden stress to the soil below the downdrag zone.

Resistance may also be estimated using a dynamic method per the AASHTO LRFD Bridge Design Specifications, Article 10.7, provided the skin friction resistance within the zone contributing to downdrag is subtracted from the resistance determined from the dynamic method during pile installation. The skin friction resistance within the zone contributing to downdrag may be estimated using the static analysis methods specified in the AASHTO LRFD Bridge Design Specifications, Article 10.7, from signal matching analysis, or from pile load test results. Note that the static analysis method may have a bias, on average over or under predicting the skin friction. The bias of the method selected to estimate the skin friction within and above the downdrag zone should be taken into account as described in the AASHTO LRFD Bridge Design Specifications, Article 10.7.

8.12.2.6.4 Determination of Nominal Axial Pile Resistance in Compression

If a dynamic formula is used to establish the driving criterion in lieu of a combination of dynamic measurements with signal matching, wave equation analysis, and/or pile load tests, the WSDOT Pile Driving Formula from the WSDOT *Standard Specifications for Roads, Bridge, and Municipal Construction* Section 6-05.3(12) shall be used, unless otherwise specifically approved by the WSDOT State Geotechnical Engineer.

The hammer energy used to calculate the nominal (ultimate) pile resistance during driving in the WSDOT and other driving formulae described herein is the developed energy. The developed hammer energy is the actual amount of gross energy produced by the hammer for a given blow. This value will never exceed the rated hammer energy (rated hammer energy is the maximum gross energy the hammer is capable of producing, i.e., at its maximum stroke).

The development of the WSDOT pile driving formula is described in Allen (2005b, 2007). The nominal (ultimate) pile resistance during driving using this method shall be taken as:

$$R_{ndr} = F \times E \times Ln\,(10N) \quad (8\text{-}6)$$

Where:

R_{ndr} = driving resistance, in TONS
F = 1.8 for air/steam hammers
= 1.2 for open ended diesel hammers and precast concrete or timber piles
= 1.6 for open ended diesel hammers and steel piles
= 1.2 for closed ended diesel hammers
= 1.9 for hydraulic hammers
= 0.9 for drop hammers
E = developed energy, equal to W times H[1], in feet-kips
W = weight of ram, in kips
H = vertical drop of hammer or stroke of ram, in feet
N = average penetration resistance in blows per inch for the last 4 inches of driving
Ln = the natural logarithm, in base "e"

[1]For closed-end diesel hammers (double-acting), the developed hammer energy (E) is to be determined from the bounce chamber reading. Hammer manufacturer calibration data may be used to correlate bounce chamber pressure to developed hammer energy. For double acting hydraulic and air/steam hammers, the developed hammer energy shall be calculated from ram impact velocity measurements or other means approved by the Engineer. For open ended diesel hammers (single-acting), the blows per minute may be used to determine the developed energy (E).

Note that R_{ndr} as determined by this driving formula is presented in units of TONS rather than KIPS, to be consistent with the WSDOT *Standard Specifications for Road, Bridge, and Municipal Construction* M 41-10. The above formula applies only when:

1. The hammer is in good condition and operating in a satisfactory manner;
2. A follower is not used;
3. The pile top is not damaged;
4. The pile head is free from broomed or crushed wood fiber;
5. The penetration occurs at a reasonably quick, uniform rate; and the pile has been driven at least 2 feet after any interruption in driving greater than 1 hour in length.
6. There is no perceptible bounce after the blow. If a significant bounce cannot be avoided, twice the height of the bounce shall be deducted from "H" to determine its true value in the formula.
7. For timber piles, bearing capacities calculated by the formula above shall be considered effective only when it is less than the crushing strength of the piles.
8. If "N" is greater than or equal to 1.0 blow/inch.

As described in detail in Allen (2005b, 2007), Equation 8-6 should not be used for nominal pile bearing resistances greater than approximately 1,000 KIPS (500 TONS), or for pile diameters greater than 30 inches, due to the paucity of data available to verify the accuracy of this equation at higher resistances and larger pile diameters, and due to the increased scatter in the data. Additional field testing and analysis, such as the use of a Pile Driving Analyzer (PDA) combined with signal matching, or a pile load test, is recommended for piles driven to higher bearing resistance and pile diameters larger than 30 inches.

As is true of most driving formulae, if they have been calibrated to pile load test results, the WSDOT pile driving formula has been calibrated to N values obtained at end of driving (EOD). Since the pile nominal resistance obtained from pile load tests are typically obtained days, if not weeks, after the pile has been driven, the gain in pile resistance that typically occurs with time is in effect correlated to the EOD N value through the driving formula. That is, the driving formula assumes that an "average" amount of setup will occur after EOD when the pile nominal resistance is determined from the formula (see Allen, 2005b, 2007). Hence, the WSDOT driving formula shall not be used in combination with the resistance factor φ_{dyn} provided in **Section 8.9** for beginning of redrive (BOR) N values to obtain nominal resistance. If pile foundation nominal resistance must be determined based on restrike (BOR) driving resistance, dynamic measurements in combination with signal matching analysis and/or pile load test results should be used.

Since driving formulas inherently account for a moderate amount of pile resistance setup, it is expected that theoretical methodologies such as the wave equation will predict lower nominal bearing resistance values for the same driving resistance N than empirical methodologies such as the WSDOT driving formula. This should be considered when assessing pile drivability if it is intended to evaluate the pile/hammer system for contract approval purposes using the wave equation, but using a pile driving formula for field determination of pile nominal bearing resistance.

If a dynamic (pile driving) formula other than the one provided here is used, subject to the approval of the State Geotechnical Engineer, it shall be calibrated based on measured load test results to obtain an appropriate resistance factor, consistent with the AASHTO LRFD Bridge Design Specifications, Article 10.7 and Allen (2005b, 2007).

If a dynamic formula is used, the structural compression limit state cannot be treated separately as with the other axial resistance evaluation procedures unless a drivability analysis if performed. Evaluation of pile drivability, including the specific evaluation of driving stresses and the adequacy of the pile to resist those stresses without damage, is strongly recommended. When drivability is not checked, it is necessary that the pile design stresses be limited to values that will assure that the pile can be driven without damage. For steel piles, guidance is provided in Article 6.15.2 of the AASHTO LRFD Bridge Design Specifications for the case where risk of pile damage is relatively high. If pile drivability is not checked, it should be assumed that the risk of pile damage is relatively high. For concrete piles and timber piles, no specific guidance is available in Sections 5 and 8, respectively, of the AASHTO LRFD Bridge Design Specifications regarding safe design stresses to reduce the risk of pile damage. In past practice (see AASHTO 2002), the required nominal axial resistance has been limited to $0.6\,f'_c$ for concrete piles and 2,000 psi for timber piles if pile drivability is not evaluated.

8.12.2.6.5 Nominal Horizontal Resistance of Pile Foundations

The nominal resistance of pile foundations to horizontal loads shall be evaluated based on both geomaterial and structural properties. The horizontal soil resistance along the piles should be modeled using P-Y curves developed for the soils at the site, as specified in Section 8.12.2.3. For piles classified as short or intermediate as defined in Section 8.13.2.4.3, Strain Wedge Theory (Norris, 1986; Ashour, et al., 1998) may used.

The applied loads shall be factored loads and they must include both horizontal and axial loads. The analysis may be performed on a representative single pile with the appropriate pile top boundary condition or on the entire pile group. If P-Y curves are used, they shall be modified for group effects. The P-multipliers Article 10.7.2.4 of the AASHTO LRFD Bridge Design Specifications and Section 8.12.2.3 should be used to modify the curves. If strain wedge theory is used, P-multipliers shall not be used, but group effects shall be addressed through evaluation of the overlap between shear zones formed due to the passive wedge that develops in front of each pile in the group as lateral deflection increases. If the pile cap will always be embedded, the P-Y horizontal resistance of the soil on the cap face may be included in the horizontal resistance.

8.12.2.7 Extreme Event Limit State Design of Pile Foundations

For the applicable factored loads (see AASHTO LRFD Bridge Design Specifications, Section 3) for each extreme event limit state, the pile foundations shall be designed to have adequate factored axial and lateral resistance. For seismic design, all soil within and above liquefiable zones shall not be considered to contribute axial compressive resistance. Downdrag resulting from liquefaction induced settlement shall be determined as specified in Section 6.5.3 and the AASHTO LRFD Bridge Design Specifications (Article 3.11.8), and shall be included in the loads applied to the foundation. Static downdrag loads shall not be combined with seismic downdrag loads due to liquefaction.

The available factored geotechnical resistance should be greater than the factored loads applied to the pile, including the downdrag, at the extreme event limit state. The pile foundation shall be designed to structurally resist the downdrag plus structure loads.

Pile design for liquefaction downdrag is illustrated in Figure 8-13, where,

R_{Sdd} = skin friction which 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_{seis} = resistance factor for seismic conditions

γ_p = load factor for downdrag

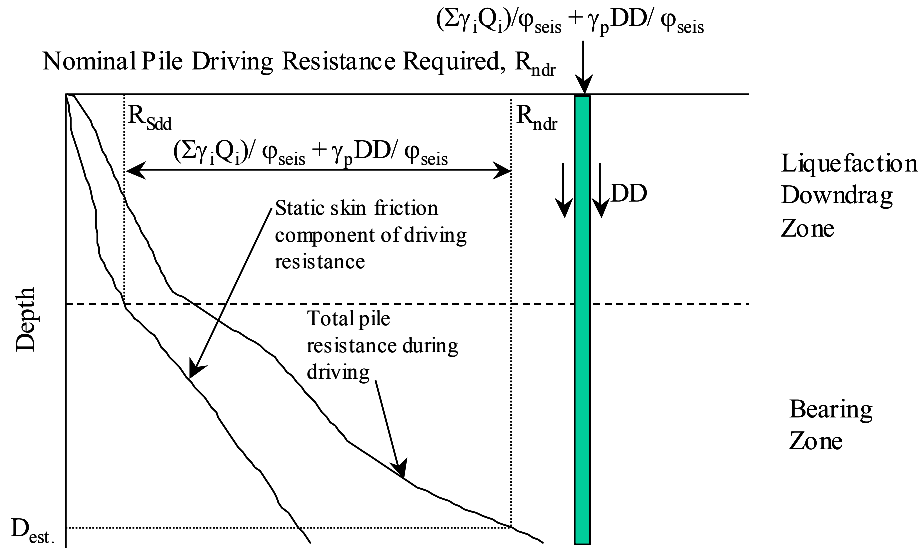
The nominal bearing resistance of the pile needed to resist the factored loads, including downdrag, is therefore,

$$R_n = (\Sigma\gamma_i Q_i)/\phi_{seis} + \gamma_p DD/\phi_{seis} \qquad (8\text{-}7)$$

The total driving resistance, R_{ndr}, needed to obtain R_n, accounting for the skin friction that must be overcome during pile driving that does not contribute to the design resistance of the pile, is as follows:

$$R_{ndr} = R_{Sdd} + R_n \qquad (8\text{-}8)$$

Note that R_{Sdd} remains unfactored in this analysis to determine R_{ndr}.



Design of Pile Foundations for Liquefaction Downdrag
Figure 8-13

In the instance where it is not possible to obtain adequate geotechnical resistance below the lowest layer contributing to downdrag (e.g., friction piles) to fully resist the downdrag, or if it is anticipated that significant deformation will be required to mobilize the geotechnical resistance needed to resist the factored loads including the downdrag load, the structure should be designed to tolerate the settlement resulting from the downdrag and the other applied loads in accordance with AASHTO LRFD Bridge Design Specifications.

The static analysis procedures in AASHTO LRFD Bridge Design Specifications may be used to estimate the available pile resistance to withstand the downdrag plus structure loads to estimate pile lengths required to achieve the required bearing resistance. For this calculation, it should be assumed that the soil subject to downdrag still contributes overburden stress to the soil below the downdrag zone.

Resistance may also be estimated using a dynamic method per AASHTO LRFD Bridge Design Specifications, provided the skin friction resistance within the zone contributing to downdrag is subtracted from the resistance determined from the dynamic method during pile installation. The skin friction resistance within the zone contributing to downdrag may be estimated using the static analysis methods specified in AASHTO LRFD Bridge Design Specifications, from signal matching analysis, or from pile load test results. Note that the static analysis method may have a bias, on average over or under predicting the skin friction. The bias of the method selected to estimate the skin friction within and above the downdrag zone should be taken into account as described in AASHTO LRFD Bridge Design Specifications.

Downdrag forces estimated using these methods may be conservative, as the downdrag force due to liquefaction may be between the full static shear strength and the liquefied shear strength acting along the length of the deep foundation elements (see **Section 6.5.3**).

The pile foundation shall also be designed to resist the horizontal force resulting from lateral spreading, if applicable, or the liquefiable soil shall be improved to prevent liquefaction and lateral spreading. For lateral soil resistance of the pile foundation, if P-Y curves are used, the soil input parameters should be reduced to account for liquefaction. To determine the amount of reduction, the duration of strong shaking and the ability of the soil to fully develop a liquefied condition during the period of strong shaking should be considered.

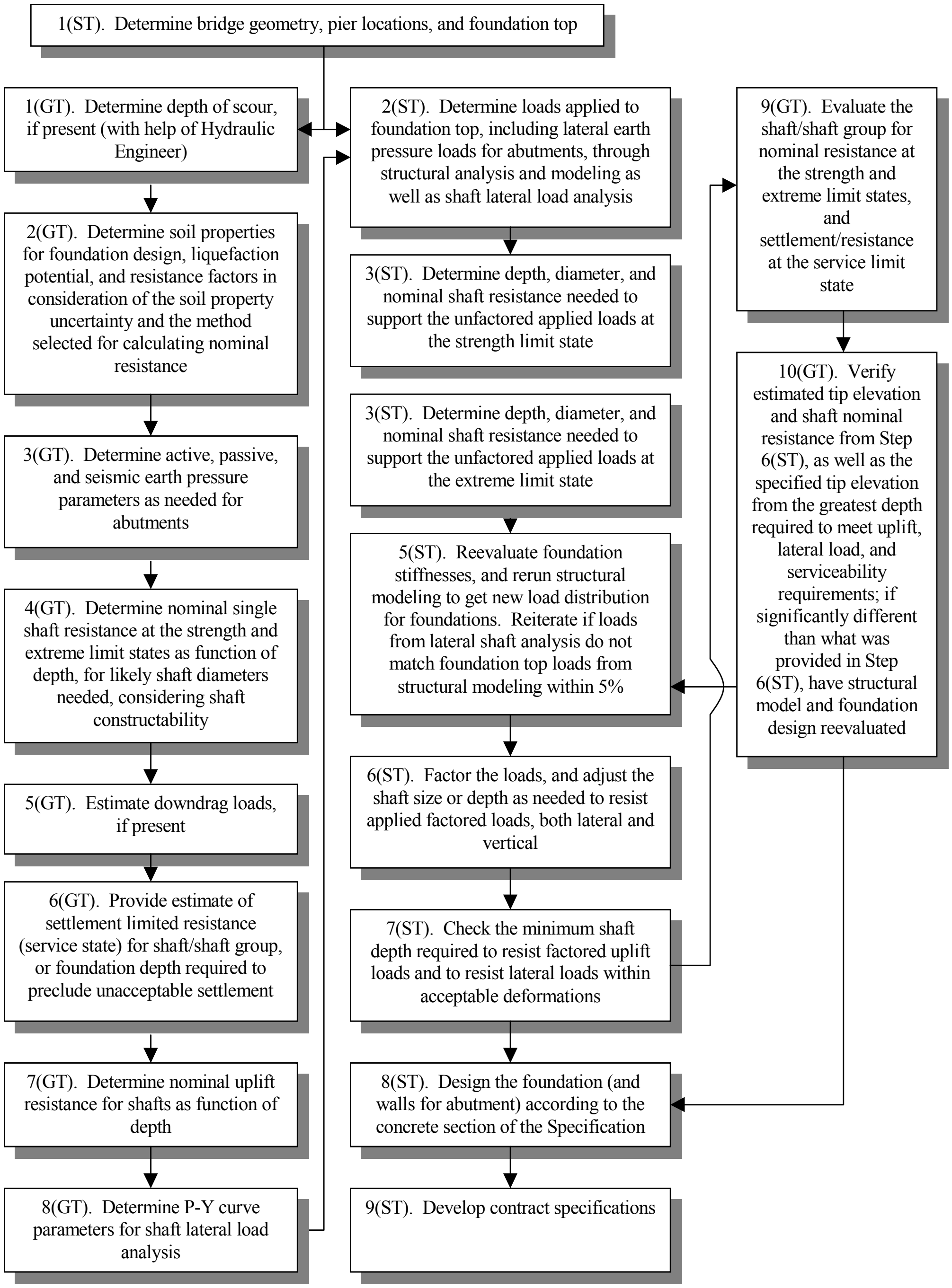
Regarding the reduction of P-Y soil strength and stiffness parameters to account for liquefaction, see Section 6.5.1.2.

The force resulting from flow failure/lateral spreading should be calculated as described in Chapter 6.

When designing for scour at the extreme event limit state, the pile foundation design shall be conducted as described in Section 8.12.4.5, and the AASHTO LRFD Bridge Design Specifications. The resistance factors and the check flood per the AASHTO Bridge Design Specifications shall be used.

8.13 Drilled Shaft Foundation Design

Figure 8-14 provides a flowchart that illustrates the design process, and interaction required between structural and geotechnical engineers, needed to complete a drilled shaft foundation design. ST denotes steps usually completed by the Structural Designer, while GT denotes those steps normally completed by the Geotechnical Designer.



Design Flowchart For Drill Shaft Foundation Design
Figure 8-14

8.13.1 Loads and Load Factor Application to Drilled Shaft Design

Figures 8-8 and 8-9 provide definitions and typical locations of the forces and moments that act on deep foundations such as drilled shafts. Table 8-8 identifies when to use maximum or minimum load factors for the various modes of failure for the shaft (bearing capacity, uplift, and lateral loading) for each force, for the strength limit state.

The loads and load factors to be used in shaft foundation design shall be as specified in Section 3 of the AASHTO LRFD Bridge Design Specifications. Computational assumptions that shall be used in determining individual shaft loads are described in Section 4 of the AASHTO LRFD specifications.

8.13.2 Drilled Shaft Geotechnical Design

Geotechnical design of drilled shaft foundations, and all related considerations, shall be conducted as specified in the AASHTO LRFD Bridge Design Specifications Article 10.8 (2012 version, but as revised/supplemented in Appendix 8-B until the next edition of the AASHTO LRFD specifications, which will contain the revised drilled shaft design specifications provided in Appendix 8-B, are published), except as specified in following paragraphs and sections:

8.13.2.1 General Considerations

The provisions of Section 8.13 and all subsections shall apply to the design of drilled shafts. Throughout these provisions, the use of the term “drilled shaft” shall be interpreted to mean a shaft constructed using either drilling or casing plus excavation equipment and related technology. These provisions shall also apply to shafts that are constructed using casing advancers that twist or rotate casings into the ground concurrent with excavation rather than drilling. The provisions of this section are not applicable to drilled piles installed with continuous flight augers that are concreted as the auger is being extracted (e.g., this section does not apply to the design of augercast piles).

Shaft designs should be reviewed for constructability prior to advertising the project for bids.

8.13.2.2 Nearby Structures

Where shaft foundations are placed adjacent to existing structures, the influence of the existing structure on the behavior of the foundation, and the effect of the foundation on the existing structures, including vibration effects due to casing installation, should be investigated. In addition, the impact of caving soils during shaft excavation on the stability of foundations supporting adjacent structures should be evaluated. For existing structure foundations that are adjacent to the proposed shaft foundation, and if a shaft excavation cave-in could compromise the existing foundation in terms of stability or increased deformation, the design should require that casing be advanced as the shaft excavation proceeds.

8.13.2.3 Service Limit State Design of Drilled Shafts

Drilled shaft foundations shall be designed at the service limit state to meet the tolerable movements for the structure being supported in accordance with Section 8.6.5.1.

Service limit state design of drilled shaft foundations includes the evaluation of settlement due to static loads, and downdrag loads if present, overall stability, lateral squeeze, and lateral deformation.

Lateral analysis of shaft foundations is conducted to establish the load distribution between the superstructure and foundations for all limit states, and to estimate the deformation in the foundation that will occur due to those loads. This section only addresses the evaluation of the lateral deformation of the foundation resulting from the distributed loads.

8.13.2.3.1 Horizontal Movement of Shafts and Shaft Groups

The provisions of Section 8.12.2.3 and Appendix 8-B shall apply.

8.13.2.3.2 Overall Stability

The provisions of Section 8.6.5.2 shall apply.

8.13.2.4 Strength Limit State Geotechnical Design of Drilled Shafts

The nominal shaft geotechnical resistances that shall be evaluated at the strength limit state include:

- Axial compression resistance,
- Axial uplift resistance,
- Punching of shafts through strong soil into a weaker layer,
- Lateral geotechnical resistance of soil and rock strata,
- Resistance when scour occurs, and
- Axial resistance when downdrag occurs.

If very strong soil, such as glacially overridden tills or outwash deposits, is present, and adequate performance data for shaft axial resistance in the considered geological soil deposit is available, the nominal end bearing resistance may be increased above the limit specified for bearing in soil in the AASHTO LRFD Bridge Design Specifications up to the loading limit that performance data indicates will produce good long-term performance. Alternatively, load testing may be conducted to validate the value of bearing resistance selected for design.

8.13.2.4.1 Scour

The effect of scour shall be considered in the determination of the shaft penetration. Resistance after scour shall be based on the applicable provisions of Section 8.12.2.6.2 and the AASHTO LRFD Bridge Design Specifications Section 10. The shaft foundation shall be designed so that the shaft penetration after the design scour event satisfies the required nominal axial and lateral resistance. For this calculation, it shall be assumed that the soil lost due to scour does not contribute to the overburden stress in the soil below the scour zone. The shaft foundation shall be designed to resist debris loads occurring during the flood event in addition to the loads applied from the structure.

The resistance factors are those used in the design without scour. The axial resistance of the material lost due to scour shall not be included in the shaft resistance.

8.13.2.4.2 Downdrag

The nominal shaft resistance available to support structure loads plus downdrag shall be estimated by considering only the positive skin and tip resistance below the lowest layer contributing to the downdrag. For this calculation, it shall be assumed that the soil contributing to downdrag does contribute to the overburden stress in the soil below the downdrag zone. In general, the available factored geotechnical resistance should be greater than the factored loads applied to the shaft, including the downdrag, at the strength limit state.

In the instance where it is not possible to obtain adequate geotechnical resistance below the lowest layer contributing to downdrag (e.g., friction shafts) to fully resist the downdrag, the structure should be designed to tolerate the settlement resulting from the downdrag and the other applied loads.

8.13.2.4.3 Nominal Horizontal Resistance of Shaft and Shaft Group Foundations

The provisions of Section 8.12.2.6.5 and Appendix 8-B shall apply. For shafts classified as short or intermediate, when laterally loaded, the shaft maintains a lateral deflection pattern that is close to a straight line. A shaft is defined as short if its length, L, to relative stiffness ratio (L/T) is less than or equal to 2, intermediate when this ratio is less than or equal to 4 but greater than 2, and long when this ratio is greater than 4, where relative stiffness, T, is defined as:

$$T = \left(\frac{EI}{f}\right)^{0.2} \qquad (8\text{-}9)$$

where,

E = the shaft modulus

I = the moment of inertia for the shaft, and EI is the bending stiffness of the shaft, and

f = coefficient of subgrade reaction for the soil into which the shaft is embedded as provided in NAVFAC DM 7.2 (1982)

For shafts classified as short or intermediate as defined above, strain wedge theory (Norris, 1986; Ashour, et al., 1998) may be used to estimate the lateral resistance of the shafts in lieu of P-Y methods.

The design of horizontally loaded drilled shafts shall account for the effects of interaction between the shaft and ground, including the number of shafts in the group. When strain wedge theory is used to assess the lateral load response of shaft groups, group effects shall be addressed through evaluation of the overlap between shear zones formed due to the passive wedge that develops in front of each shaft in the group as lateral deflection increases.

8.13.2.5 Extreme Event Limit State Design of Drilled Shafts

The provisions of Section 8.12.2.7 shall apply, except that for liquefaction downdrag, the nominal shaft resistance available to support structure loads plus downdrag shall be estimated by considering only the positive skin and tip resistance below the lowest layer contributing to the downdrag. For this calculation, it shall be assumed that the soil contributing to downdrag does contribute to the overburden stress in the soil below the downdrag zone. In general, the available factored geotechnical resistance should be greater than the factored loads applied to the shaft, including the downdrag, at the strength limit state. The shaft foundation shall be designed to structurally resist the downdrag plus structure loads.

In the instance where it is not possible to obtain adequate geotechnical resistance below the lowest layer contributing to downdrag (e.g., friction shafts) to fully resist the downdrag, the structure should be designed to tolerate the settlement resulting from the downdrag and the other applied loads.

8.14 Micropiles

Micropiles shall be designed in accordance with Articles 10.5 and 10.9 of the AASHTO LRFD Bridge Design Specifications. Additional background information on micropile design may be found in the FHWA Micropile Design and Construction Guidelines Implementation Manual, Publication No. FHWA-SA-97-070 (Armour, et al., 2000).

8.15 Proprietary Foundation Systems

Only proprietary foundation systems that have been reviewed and approved by the WSDOT New Products Committee, and subsequently added to Appendix 8-A of this manual, may be used for structural foundation support.

In general, proprietary foundation systems shall be evaluated based on the following:

1. The design shall rely on published and proven technology, and should be consistent with the AASHTO LRFD Bridge Design Specifications and this geotechnical design manual. Deviations from the AASHTO specifications and this manual necessary to design the foundation system must be fully explained based on sound geotechnical theory and supported empirically through full scale testing.
2. The quality of the foundation system as constructed in the field is verifiable.
3. The foundation system is durable, and through test data it is shown that it will have the necessary design life (usually 75 years or more).
4. The limitations of the foundation system in terms of its applicability, capacity, constructability, and potential impact to adjacent facilities during and after its installation (e.g., vibrations, potential subsurface soil movement, etc.) are clearly identified.

8.16 Detention Vaults

8.16.1 Overview

Requirements for sizing and locating detention/retention vaults are provided in the *Highway Runoff Manual*. Detention/retention vaults as described in this section include wet vaults, combined wet/detention vaults and detention vaults. For specific details regarding the differences between these facilities, please refer to Chapter 5 of the *WSDOT Highway Runoff Manual*. For geotechnical and structural design purposes, a detention vault is a buried reinforced concrete structure designed to store water and retain soil, with or without a lid. The lid and the associated retaining walls may need to be designed to support a traffic surcharge. The size and shape of the detention vaults can vary. Common vault widths vary from 15 feet to over 60 feet. The length can vary greatly. Detention vaults over a 100 feet in length have been proposed for some projects. The base of the vault may be level or may be sloped from each side toward the center forming a broad V to facilitate sediment removal. Vaults have specific site design elements, such as location with respect to right-of-way, septic tanks and drain fields. The geotechnical designer must address the adequacy of the proposed vault location and provide recommendations for necessary set-back distances from steep slopes or building foundations.

8.16.2 Field Investigation Requirements

A geotechnical reconnaissance and subsurface investigation are critical for the design of all detention vaults. All detention vaults, regardless of their size, will require an investigation of the underlying soil/rock that supports the structure.

The requirements for frequency of explorations provided in Table 8-10 should be used. Additional explorations may be required depending on the variability in site conditions, vault geometry, and the consequences should a failure occur.

Vault surface area (ft^2)	Exploration points (minimum)
<200	1
200 - 1000	2
1000 – 10,000	3
>10,000	3 - 4

Minimum Exploration Requirements for Detention Vaults
Table 8-10

The depth of the borings will vary depending on the height of soil being retained by the vault and the overall depth of the vault. The borings should be extended to a depth below the bottom elevation of the vault a minimum of 1.5 times the height of the exterior walls. Exploration depth should be great enough to fully penetrate soft highly compressible soils (e.g., peat, organic silt, soft fine grained soils) into competent material of suitable bearing resistance (e.g., very stiff to hard cohesive soil, dense cohesionless soil or bedrock). Since these structures may be subjected to hydrostatic uplift forces, a minimum of one boring must be instrumented with a piezometer to measure seasonal variations in ground water unless the ground water depth is known to be well below the bottom of the vault at all times.

8.16.3 Design Requirements

A detention vault is an enclosed buried structure surrounded by three or more retaining walls. Therefore, for the geotechnical design of detention vault walls, design requirements provided in Chapter 15 are applicable. Since the vault walls typically do not have the ability to deform adequately to allow active earth pressure conditions to develop, at rest conditions should be assumed for the design of the vault walls (see Chapter 15.

If the seasonal high ground water level is above the base of the vault, the vault shall be designed for the uplift forces that result from the buoyancy of the structure. Uplift forces should be resisted by tie-down anchors or deep foundations in combination with the weight of the structure and overburden material over the structure.

Temporary shoring may be required to allow excavation of the soil necessary to construct the vault. See Chapter 15 for guidelines on temporary shoring. If a shoring wall is used to permanently support the sides of the vault or to provide permanent uplift resistance to buoyant forces, the shoring wall(s) shall be designed as permanent wall(s).

8.17 References

AASHTO, 2012, *LRFD Bridge Design Specifications*, American Association of State Highway and Transportation Officials, Sixth Edition, Washington, D.C., USA. (**Note**: Most current edition shall be used).

Allen, T. M., 2005, *Development of Geotechnical Resistance Factors and Downdrag Load Factors for LRFD Foundation Strength Limit State Design*, Publication No. FHWA-NHI-05-052, Federal Highway Administration, Washington, DC, 41 pp.

Allen, T. M., 2005b, *Development of the WSDOT Pile Driving Formula and Its Calibration for Load and Resistance Factor Design (LRFD)*, WSDOT Research Report WA-RD 610.1, Olympia, WA, 45 pp.

Allen, T. M., 2007, "Development of New Pile-Driving Formula and Its Calibration for Load and Resistance Factor Design," *Transportation Research Record 2004*, TRB, National Research Council, Washington, DC., pp. 20-27.

Armour, T., Groneck, T., Keeley, J., and Sharma, S., 2000, Micropile Design and Construction Guidelines Implementation Manual, FHWA-SA-97-070, 376 pp.

Ashour, M., and Norris, G. M., 1999, "Liquefaction and Undrained Response Evaluation of Sands from Drained Formulation," ", ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 125, No. 8, pp. 649-658.

Ashour, M., and Norris, G. M., 2003, "Lateral Loaded Pile Response in Liquefiable Soil", ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 129, No. 6, pp. 404-414.

Ashour, M., Norris, G. M., and Pilling, P., 1998, "Lateral Loading of a Pile in layered Soil Using the Strain Wedge Model," *ASCE Journal of Geotechnical and Geoenvironmental Engineering,* ASCE, Vol. 124, No. 4, pp. 303-315.

Ashour, M., Norris, G. M., and Pilling, P., 2002, "Strain Wedge Model Capability of Analyzing Behavior of Laterally Loaded Isolated Piles, Drilled Shafts, and Pile Groups," *ASCE Journal of Geotechnical and Geoenvironmental Engineering,* Vol. 7, No. 4, pp. 245-254.

Barker, R. M., J. M. Duncan, K. B. Rojiani, P. S. K. Ooi, C. K. Tan, and S. G. Kim. 1991. "Manuals for the Design of Bridge Foundations." *NCHRP Report 343*. TRB, National Research Council, Washington, DC.

Broms, B.B., 1964a. Lateral Resistance of Piles in Cohesive Soil. ASCE, Journal for Soil Mechanics and Foundation Engineering, Vol 90, SM2, 27 – 63.

Broms, B.B., 1964b. Lateral Resistance of Piles in Cohesionless Soil. ASCE, Journal for Soil Mechanics and Foundation Engineering, Vol 90, SM3, 123 – 156.

DiMillio, A. F., 1982. "Performance of Highway Bridge Abutments Supported by Spread Footings on Compacted Fill," *Report No. FHWA/RD-81/184*, (NTIS PB83-201822). (FHWA Staff Study).

Hannigan, P.J., G.G. Goble, G.E. Likins and F. Rausche, 2006. "Design and Construction of Driven Pile Foundations" - Vol. I and II, Federal Highway Administration Report No. FHWA-HI-05-042, Federal Highway Administration, Washington, D.C., 822 pp.

Hoek, E., C. Carrazna-Torres, and B. Corkum, 2002. “Hoek-Brown Failure Criterion – 2002 Edition,” *5th North American Rock Mechanics Symposium and 17th Tunneling Association of Canada Conference:* NARMS-TAC, pp. 267-271.

Hough, B.K. 1959. “Compressibility as the Basis for Soil Bearing Value,” *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 85, Part 2.

Ishihara, K., and Yoshimine, M., 1992. “Evaluation of settlements in sand deposits following liquefaction during earthquakes.” Soils and Foundations, JSSMFE, Vol. 32, No. 1, March, pp. 173-188.

Kavazanjian, E., Jr., Matasoviæ, T. Hadj-Hamou and Sabatini, P.J. 1997. “Geotechnical Engineering Circular No. 3, Design Guidance: Geotechnical Earthquake Engineering for Highways,” *Report No. FHWA-SA-97-076*, Federal Highway Administration, Washington, D.C.

Kimmerling, R. E. 2002. Geotechnical Engineering Circular 6. *Report No. FHWA-SA-02-054,* Federal Highway Administration, Washington, D.C.

Kyfor, Z. G., Schnore, A. R., Carlo, T. A., and Bailey, P. F., 1992. Static Testing of Deep Foundations. Report No. FHWA-SA-91-042, U. S. Department of Transportation, Federal Highway Administration, Office of Technology Applications, Washington D. C., 174

Lam, I. P, and G. R. Martin. 1986. “Seismic Design of Highway Bridge Foundations.” Vol. 2, *Design Procedures and Guidelines*. FHWA/RD-86/102, Federal Highway Administration, U.S. Department of Transportation, Washington, DC, p. 18.

Liang, R. Y., 2010, *Field Intrumentation, Monitoring of Drilled Shafts for Landslide Stabilization and Development of Pertinent Design Method*, Report No. FHWA/OH-2010/15, Ohio Department of Transportation, 238 pp.

Moulton, L. K., H. V. S. GangaRao, and G. T. Halverson. 1985. “Tolerable Movement Criteria for Highway Bridges.” *FHWA/RD-85/107*. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, p. 118.

Munfakh, G., Arman, A., Collin, J.G., Hung, J.C.-J., and Brouillette, R.P. 2001. “Shallow Foundations Reference Manual,” *Publication No. FHWA-NHI-01-023*, Federal Highway Administration, Washington, D.C.

Norris, G. M., 1986, “Theoretically based BEF Laterally Loaded Pile Analysis: Proceedings, Third International Conference on Numerical Methods in Offshore Piling, Nantes, France, pp. 361-386.

Reese, L. C. 1984. “Handbook on Design of Piles and Drilled Shafts Under Lateral Load.” *FHWA-IP-84/11*, Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

Reese, L. C., 1986. Behavior of Piles and Pile Groups Under Lateral Load. Report No. FHWA/RD-85/106, U. S. Department of Transportation, Federal Highway Administration, Office of Engineering and Highway Operations Research and Development, Washington D. C., 311

Sabatini, P.J, Bachus, R.C, Mayne, P.W., Schneider, J.A., Zettler, T.E. (2002), Geotechnical Engineering Circular 5 (GEC5) - Evaluation of Soil and Rock Properties.

Report No FHWA-IF-02-034. Federal Highway Administration, U.S. Department of Transportation.

Samtani, N. C., and Nowatzki, E. A., 2006. *Soils and Foundations Workshop Reference Manual, Volumes 1 and 2*. Washington, DC, National Highway Institute Publication NHI-06-088 and NHI-06-089, Federal Highway Administration, 1056 pp.

Seed, R. B. and Harder, L. F. Jr., 1990. SPT-Based Analysis of Cyclic Pore Pressure Generation and Undrained Residual Strength. Proceedings, H. B. Bolton Seed Memorial Symposium, J. M. Duncan Editor, BiTech Publishers, Vol 2, 351-376.

Tokimatsu, K. and Bolton Seed, B. 1987. Evaluation of Settlements in Sands due to Earthquake Shaking, Journal of Geotechnical Engineering, ASCE, 113, 8, 861-878.

Vessely, A., Yamasaki, K., and Strom, R., 2007. "Landslide Stabilization Using Piles." *Proc. of the 1st N. American Conf. on Landslides*, Vail, Colorado, Assoc. of Environmental & Engineering Geologists, pp. 1173-1183.

Williams, M.E., M. McVay and M.I. Hoit, 2003. "LRFD Substructure and Foundation Design Programs", Proceedings of the 2003 International Bridge Conference, June 9-11, Pittsburgh, Pa.

WSDOT, 2012, Bridge Design Manual LRFD, M 23-50 (**Note**: Most current edition shall be used.)

WSDOT, 2012, *Standard Specifications for Road, Bridge, and Municipal Construction*, M 41-01 (**Note**: Most current edition shall be used).

Yamasaki, K., Strom, R. W., Gunsolus, R. A., and D. Andrew Vessely, 2013, "Long-Term Performance of Landslide Shear Piles," *ASCE Geo-Congress 2013*, pp. 1936-1950.

Appendix 8-A Approved Proprietary Foundation Systems (Vacant)

Appendix 8-B Approved AASHTO LRFD Bridge Design Specifications Drill Shaft Design Provisions

Approved AASHTO LRFD Bridge Design Specifications – Drilled Shaft Design Provisions – Approved June 2013

The AASHTO approved design provisions that follow update Section 10 of the 2012 AASHTO LRFD Bridge Design Specifications and shall be used until these updated provisions are published in the next edition of the AASHTO specifications." The strike-through text shown in the pages that follow in this appendix represent text, tables, and figures that will be removed from Section 10 of the 2012 AASHTO LRFD Bridge Design Specifications, and the underlined text, tables, and figures represent what will be added to Section 10 of the 2012 AASHTO LRFD Bridge Design Specifications.

1

ATTACHMENT A — 2013 AGENDA ITEM __ - T-15

10.1—SCOPE – ***NO CHANGES – NOT SHOWN***

10.2—DEFINITIONS

ONE ADDITION BELOW – THE REMAINDER STAYS THE SAME

GSI—Geologic Strength Index

10.3—NOTATION

ONE ADDITION BELOW – THE REMAINDER STAYS THE SAME

s, m, a = fractured rock mass parameters (10.4.6.4)

10.4—SOIL AND ROCK PROPERTIES

10.4.1—Informational Needs – ***NO CHANGES – NOT SHOWN***

10.4.2—Subsurface Exploration

Subsurface explorations shall be performed to provide the information needed for the design and construction of foundations. The extent of exploration shall be based on variability in the subsurface conditions, structure type, and any project requirements that may affect the foundation design or construction. The exploration program should be extensive enough to reveal the nature and types of soil deposits and/or rock formations encountered, the engineering properties of the soils and/or rocks, the potential for liquefaction, and the groundwater conditions. The exploration program should be sufficient to identify and delineate problematic subsurface conditions such as karstic formations, mined out areas, swelling/collapsing soils, existing fill or waste areas, etc.

Borings should be sufficient in number and depth to establish a reliable longitudinal and transverse substrata profile at areas of concern such as at structure foundation locations and adjacent earthwork locations, and to investigate any adjacent geologic hazards that could affect the structure performance.

C10.4.2

The performance of a subsurface exploration program is part of the process of obtaining information relevant for the design and construction of substructure elements. The elements of the process that should precede the actual exploration program include a search and review of published and unpublished information at and near the site, a visual site inspection, and design of the subsurface exploration program. Refer to Mayne et al. (2001) and Sabatini et al. (2002) for guidance regarding the planning and conduct of subsurface exploration programs.

The suggested minimum number and depth of borings are provided in Table 10.4.2-1. While engineering judgment will need to be applied by a licensed and experienced geotechnical professional to adapt the exploration program to the foundation types and depths needed and to the variability in the subsurface conditions observed, the intent of Table 10.4.2-1 regarding the minimum level of exploration needed should be carried out. The depth of borings indicated in Table 10.4.2-1 performed before or during design should take into account the potential for changes in the type, size and depth of the planned foundation elements.

As a minimum, the subsurface exploration and testing program shall obtain information adequate to analyze foundation stability and settlement with respect to:

- Geological formation(s) present,
- Location and thickness of soil and rock units,
- Engineering properties of soil and rock units, such as unit weight, shear strength and compressibility,
- Groundwater conditions,
- Ground surface topography, and
- Local considerations, e.g., liquefiable, expansive or dispersive soil deposits, underground voids from solution weathering or mining activity, or slope instability potential.

Table 10.4.2-1 shall be used as a starting point for determining the locations of borings. The final exploration program should be adjusted based on the variability of the anticipated subsurface conditions as well as the variability observed during the exploration program. If conditions are determined to be variable, the exploration program should be increased relative to the requirements in Table 10.4.2-1 such that the objective of establishing a reliable longitudinal and transverse substrata profile is achieved. If conditions are observed to be homogeneous or otherwise are likely to have minimal impact on the foundation performance, and previous local geotechnical and construction experience has indicated that subsurface conditions are homogeneous or otherwise are likely to have minimal impact on the foundation performance, a reduced exploration program relative to what is specified in Table 10.4.2-1 may be considered.

If requested by the Owner or as required by law, boring and penetration test holes shall be plugged.

Laboratory and/or in-situ tests shall be performed to determine the strength, deformation, and permeability characteristics of soils and/or rocks and their suitability for the foundation proposed.

This Table should be used only as a first step in estimating the number of borings for a particular design, as actual boring spacings will depend upon the project type and geologic environment. In areas underlain by heterogeneous soil deposits and/or rock formations, it will probably be necessary to drill more frequently and/or deeper than the minimum guidelines in Table 10.4.2-1 to capture variations in soil and/or rock type and to assess consistency across the site area. For situations where large diameter rock socketed shafts will be used or where drilled shafts are being installed in formations known to have large boulders, or voids such as in karstic or mined areas, it may be necessary to advance a boring at the location of each shaft. Even the best and most detailed subsurface exploration programs may not identify every important subsurface problem condition if conditions are highly variable. The goal of the subsurface exploration program, however, is to reduce the risk of such problems to an acceptable minimum.

In a laterally homogeneous area, drilling or advancing a large number of borings may be redundant, since each sample tested would exhibit similar engineering properties. Furthermore, in areas where soil or rock conditions are known to be very favorable to the construction and performance of the foundation type likely to be used, e.g., footings on very dense soil, and groundwater is deep enough to not be a factor, obtaining fewer borings than provided in Table 10.4.2-1 may be justified. In all cases, it is necessary to understand how the design and construction of the geotechnical feature will be affected by the soil and/or rock mass conditions in order to optimize the exploration.

Borings may need to be plugged due to requirements by regulatory agencies having jurisdiction and/or to prevent water contamination and/or surface hazards.

Parameters derived from field tests, e.g., driven pile resistance based on cone penetrometer testing, may also be used directly in design calculations based on empirical relationships. These are sometimes found to be more reliable than analytical calculations, especially in familiar ground conditions for which the empirical relationships are well established.

3

Table 10.4.2-1—Minimum Number of Exploration Points and Depth of Exploration (modified after Sabatini et al., 2002)

Application	Minimum Number of Exploration Points and Location of Exploration Points	Minimum Depth of Exploration
Retaining Walls	A minimum of one exploration point for each retaining wall. For retaining walls more than 100 ft in length, exploration points spaced every 100 to 200 ft with locations alternating from in front of the wall to behind the wall. For anchored walls, additional exploration points in the anchorage zone spaced at 100 to 200 ft. For soil-nailed walls, additional exploration points at a distance of 1.0 to 1.5 times the height of the wall behind the wall spaced at 100 to 200 ft.	Investigate to a depth below bottom of wall at least to a depth where stress increase due to estimated foundation load is less than ten percent of the existing effective overburden stress at that depth and between one and two times the wall height. Exploration depth should be great enough to fully penetrate soft highly compressible soils, e.g., peat, organic silt, or soft fine grained soils, into competent material of suitable bearing capacity, e.g., stiff to hard cohesive soil, compact dense cohesionless soil, or bedrock.
Shallow Foundations	For substructure, e.g., piers or abutments, widths less than or equal to 100 ft, a minimum of one exploration point per substructure. For substructure widths greater than 100 ft, a minimum of two exploration points per substructure. Additional exploration points should be provided if erratic subsurface conditions are encountered. ~~To reduce design and construction risk due to subsurface condition variability and the potential for construction claims, at least one exploration per shaft should be considered for large diameter shafts (e.g., greater than 5 ft in diameter), especially when shafts are socketed into bedrock.~~	Depth of exploration should be: • great enough to fully penetrate unsuitable foundation soils, e.g., peat, organic silt, or soft fine grained soils, into competent material of suitable bearing resistance, e.g., stiff to hard cohesive soil, or compact to dense cohesionless soil or bedrock ; • at least to a depth where stress increase due to estimated foundation load is less than ten percent of the existing effective overburden stress at that depth; and • if bedrock is encountered before the depth required by the second criterion above is achieved, exploration depth should be great enough to penetrate a minimum of 10 ft into the bedrock, but rock exploration should be sufficient to characterize compressibility of infill material of near-horizontal to horizontal discontinuities. Note that for highly variable bedrock conditions, or in areas where very large boulders are likely, more than 10 ft or rock core may be required to verify that adequate quality bedrock is present.
Deep Foundations	For substructure, e.g., bridge piers or abutments, widths less than or equal to 100 ft, a minimum of one exploration point per substructure. For substructure widths greater than 100 ft, a minimum of two exploration points per substructure. Additional exploration points should be provided if erratic subsurface conditions are encountered, especially for the case of shafts socketed into bedrock. To reduce design and construction risk due to subsurface condition variability and the potential for construction claims, at least one exploration per shaft should be considered for large diameter shafts (e.g., greater than 5 ft in diameter), especially when shafts are socketed into bedrock.	In soil, depth of exploration should extend below the anticipated pile or shaft tip elevation a minimum of 20 ft, or a minimum of two times the ~~maximum~~ minimum pile group dimension, whichever is deeper. All borings should extend through unsuitable strata such as unconsolidated fill, peat, highly organic materials, soft fine-grained soils, and loose coarse-grained soils to reach hard or dense materials. For piles bearing on rock, a minimum of 10 ft of rock core shall be obtained at each exploration point location to verify that the boring has not terminated on a boulder. For shafts supported on or extending into rock, a minimum of 10 ft of rock core, or a length of rock core equal to at least three times the shaft diameter for isolated shafts or two times the ~~maximum~~ minimum shaft group dimension, whichever is greater, shall be extended below the anticipated shaft tip elevation to determine the physical characteristics of rock within the zone of foundation influence. Note that for highly variable bedrock conditions, or in areas where very large boulders are likely, more than 10 ft or rock core may be required to verify that adequate quality bedrock is present.

10.4.3—Laboratory Tests – ***NO CHANGES – NOT SHOWN***

10.4.4—In-Situ Tests – ***NO CHANGES – NOT SHOWN***

10.4.5—Geophysical Tests – ***NO CHANGES- NOT SHOWN***

10.4.6—Selection of Design Properties

10.4.6.1—General – ***NO CHANGES – NOT SHOWN***

10.4.6.2—Soil Strength – ***NO CHANGES – NOT SHOWN***

10.4.6.3—Soil Deformation – ***NO CHANGES – NOT SHOWN***

10.4.6.4—Rock Mass Strength

The strength of intact rock material should be determined using the results of unconfined compression tests on intact rock cores~~, splitting tensile tests on intact rock cores, or point load strength tests on intact specimens of rock~~.

~~The rock should be classified using the rock mass rating system (RMR) as described in Table 10.4.6.4-1. For each of the five parameters in the Table, the relative rating based on the ranges of values provided should be evaluated. The rock mass rating (RMR) should be determined as the sum of all five relative ratings. The RMR should be adjusted in accordance with the criteria in Table 10.4.6.4-2. The rock classification should be determined in accordance with Table 10.4.6.4-3.~~ Except as noted for design of spread footings in rock, for a rock mass that contains a sufficient number of “randomly” oriented discontinuities such that it behaves as an isotropic mass, and thus its behavior is largely independent of the direction of the applied loads, the strength of the rock mass should first be classified using its geological strength index (GSI) as described in Figures 10.4.6.4-1 and 10.4.6.4-2 and then assessed using the Hoek-Brown failure criterion.

C10.4.6.4

Point load strength index tests may be used to assess intact rock compressive strength in lieu of a full suite of unconfined compression tests on intact rock cores provided that the point load test results are calibrated to unconfined compression strength tests. Point load strength index tests rely on empirical correlations to intact rock compressive strength. The correlation provided in the ASTM point load test procedure (ASTM D 5731) is empirically based and may not be valid for the specific rock type under consideration. Therefore, a site specific correlation with uniaxial compressive strength test results is recommended. Point load strength index tests should not be used for weak to very weak rocks (< 2200 psi /15 MPa).

~~Because of the importance of the discontinuities in rock, and the fact that most rock is much more discontinuous than soil~~Because the engineering behavior of rock is strongly influenced by the presence and characteristics of discontinuities, emphasis is placed on visual assessment of the rock and the rock mass. The application of a rock mass classification system essentially assumes that the rock mass contains a sufficient number of “randomly” oriented discontinuities such that it behaves as an isotropic mass, and thus its behavior is largely independent of the direction of the applied loads. It is generally not appropriate to use such classification systems for rock masses with well defined, dominant structural fabrics or where the orientation of discrete, persistent discontinuities controls behavior to loading.

The GSI was introduced by Hoek et al. (1995) and Hoek and Brown (1997), and updated by Hoek et al. (1998) to classify jointed rock masses. Marinos et al. (2005) provide a comprehensive summary of the applications and limitations of the GSI for jointed rock masses (Figure 10.4.6.4-1) and for heterogeneous rock masses that have been tectonically disturbed (Figure 10.4.6.4-2). Hoek et al. (2005) further distinguish heterogeneous sedimentary rocks that are not tectonically disturbed and provide several diagrams for determining GSI values for various rock mass conditions. In combination with rock type and uniaxial compressive strength of intact rock (q_u), GSI provides a practical means to assess rock mass strength and rock mass modulus for foundation design using the Hoek-Brown failure criterion (Hoek et al. 2002).

The design procedures for spread footings in rock provided in Article 10.6.3.2 have been developed using the rock mass rating (RMR) system. For design of foundations in rock in Articles 10.6.2.4 and 10.6.3.2, classification of the rock mass should be according to the RMR system. For additional information on the RMR system, see Sabatini et al. (2002).

Other methods for assessing rock mass strength, including in-situ tests or other visual systems that have proven to yield accurate results may be used in lieu of the specified method.

6

~~Table 10.4.6.4-1—Geomechanics Classification of Rock Masses~~

	~~Parameter~~		~~Ranges of Values~~						
~~1~~	~~Strength of intact rock material~~	~~Point load strength index~~	~~>175 ksf~~	~~85–175 ksf~~	~~45–85 ksf~~	~~20–45 ksf~~	~~For this low range, uniaxial compressive test is preferred~~		
		~~Uniaxial compressive strength~~	~~>4320 ksf~~	~~2160–4320 ksf~~	~~1080–2160 ksf~~	~~520–1080 ksf~~	~~215–520 ksf~~	~~70–215 ksf~~	~~20–70 ksf~~
	~~Relative Rating~~		~~15~~	~~12~~	~~7~~	4	~~2~~	~~1~~	~~0~~
~~2~~	~~Drill core quality RQD~~		~~90% to 100%~~	~~75% to 90%~~	~~50% to 75%~~	~~25% to 50%~~	~~<25%~~		
	~~Relative Rating~~		~~20~~	~~17~~	~~13~~	~~8~~	~~3~~		
~~3~~	~~Spacing of joints~~		~~>10 ft~~	~~3–10 ft~~	~~1–3 ft~~	~~2 in.–1 ft~~	~~<2 in.~~		
	~~Relative Rating~~		~~30~~	~~25~~	~~20~~	~~10~~	~~5~~		
4	~~Condition of joints~~		• ~~Very rough surfaces~~ • ~~Not continuous~~ • ~~No separation~~ • ~~Hard joint wall rock~~	• ~~Slightly rough surfaces~~ • ~~Separation <0.05 in.~~ • ~~Hard joint wall rock~~	• ~~Slightly rough surfaces~~ • ~~Separation <0.05 in.~~ • ~~Soft joint wall rock~~	• ~~Slicken-sided surfaces or~~ • ~~Gouge <0.2 in. thick or~~ • ~~Joints open 0.05–0.2 in.~~ • ~~Continuous joints~~	• ~~Soft gouge >0.2 in. thick or~~ • ~~Joints open >0.2 in.~~ • ~~Continuous joints~~		
	~~Relative Rating~~		~~25~~	~~20~~	~~12~~	~~6~~	~~0~~		
~~5~~	~~Groundwater conditions~~ ~~(use one of the three evaluation criteria as appropriate to the method of exploration)~~	~~Inflow per 30 ft tunnel length~~	~~None~~	~~<400 gal./hr.~~	~~400–2000 gal./hr.~~	~~>2000 gal./hr.~~			
		~~Ratio = joint water pressure/ major principal stress~~	~~0~~	~~0.0–0.2~~	~~0.2–0.5~~	~~>0.5~~			
		~~General Conditions~~	~~Completely Dry~~	~~Moist only (interstitial water)~~	~~Water under moderate pressure~~	~~Severe water problems~~			
	~~Relative Rating~~		~~10~~	~~7~~	4	~~0~~			

~~Table 10.4.6.4-2—Geomechanics Rating Adjustment for Joint Orientations~~

~~Strike and Dip Orientations of Joints~~		~~Very Favorable~~	~~Favorable~~	~~Fair~~	~~Unfavorable~~	~~Very Unfavorable~~
~~Ratings~~	~~Tunnels~~	~~0~~	~~–2~~	~~–5~~	~~–10~~	~~–12~~
	~~Foundations~~	~~0~~	~~–2~~	~~–7~~	~~–15~~	~~–25~~
	~~Slopes~~	~~0~~	~~–5~~	~~–25~~	~~–50~~	~~–60~~

7

~~Table 10.4.6.4-3—Geomechanics Rock Mass Classes Determined from Total Ratings~~

~~RMR Rating~~	~~100–81~~	~~80–61~~	~~60–41~~	~~40–21~~	~~<20~~
~~Class No.~~	~~I~~	~~II~~	~~III~~	~~IV~~	~~V~~
~~Description~~	~~Very good rock~~	~~Good rock~~	~~Fair rock~~	~~Poor rock~~	~~Very poor rock~~

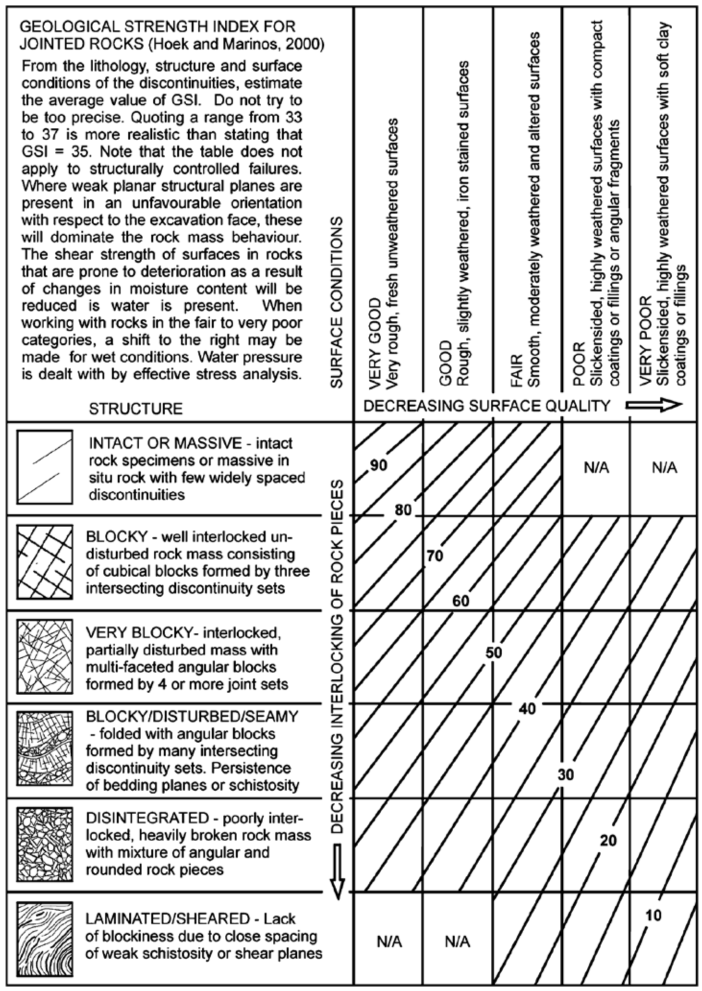


Figure 10.4.6.4-1—Determination of GSI for Jointed Rock Mass (Hoek and Marinos, 2000)

8

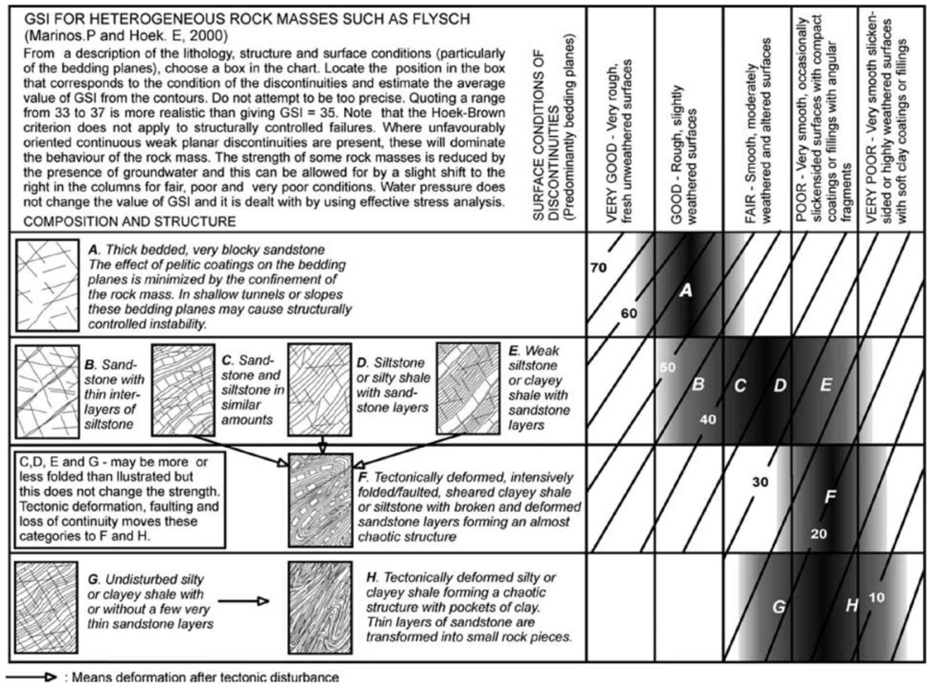


Figure 10.4.6.4-2—Determination of GSI for Tectonically Deformed Heterogeneous Rock Masses (Marinos and Hoek 2000)

The ~~shear~~ strength of ~~fractured~~jointed rock masses should be evaluated using the ~~Hoek and Brown~~ Hoek-Brown failure criterion (Hoek et al., 2002). This nonlinear strength criterion is expressed in its general form as: ~~criteria in which the shear strength is represented as a curved envelope that is a function of the uniaxial compressive strength of the intact rock, q_u, and two dimensionless constants m and s. The values of m and s as defined in Table 10.4.6.4-4 should be used.~~

~~The shear strength of the rock mass should be determined as:~~

$$\tau = \left(\cot\phi'_i - \cos\phi'_i\right) m \frac{q_u}{8} \qquad (10.4.6.4\text{-}1)$$

~~in which:~~

$$\phi'_i = \tan^{-1}\left\{4h\cos^2\left[30 + 0.33\sin^{-1}\left(h^{-\frac{3}{2}}\right)\right] - 1\right\}^{-\frac{1}{2}}$$

$$h = 1 + \frac{16\left(m\sigma'_n + s q_u\right)}{(3m^2 q_u)}$$

~~This method was developed by Hoek (1983) and Hoek and Brown (1988, 1997). Note that the instantaneous cohesion at a discrete value of normal stress can be taken as:~~

$$c_i = \tau - \sigma'_n \tan\phi'_i \qquad (C10.4.6.4\text{-}1)$$

~~The instantaneous cohesion and instantaneous friction angle define a conventional linear Mohr envelope at the normal stress under consideration. For normal stresses significantly different than that used to compute the instantaneous values, the resulting shear strength will be unconservative. If there is considerable variation in the effective normal stress in the zone of concern, consideration should be given to subdividing the zone into areas where the normal stress is relative constant and assigning separate strength parameters to each zone. Alternatively, the methods of Hoek (1983) may be used to compute average values for the range of normal stresses expected.~~

9

~~where:~~

~~τ = the shear strength of the rock mass (ksf)~~

~~ϕ'_r = the instantaneous friction angle of the rock mass (degrees)~~

~~q_u = average unconfined compressive strength of rock core (ksf)~~

~~σ'_n = effective normal stress (ksf)~~

~~m, s = constants from Table 10.4.6.4-4 (dim)~~

$$\sigma_1' = \sigma_3' + q_u\left(m_b \frac{\sigma_3'}{q_u} + s\right)^a \qquad (10.4.6.4\text{-}1)$$

in which:

$$s = e^{\left(\frac{GSI-100}{9-3D}\right)} \qquad (10.4.6.4\text{-}2)$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{\frac{-GSI}{15}} - e^{\frac{-20}{3}}\right) \qquad (10.4.6.4\text{-}3)$$

where:

e = 2.718 (natural or Naperian log base)

D = disturbance factor (dim)

σ'_1 and σ'_3 = principal effective stresses (ksf)

q_u = average unconfined compressive strength of rock core (ksf)

m_b, s, and a = empirically determined parameters

The value of the constant m_i should be estimated from Table 10.4.6.4-1, based on lithology. Relationships between GSI and the parameters m_b, s, and a, according to Hoek et al. (2002) are as follows:

$$m_b = m_i e^{\left(\frac{GSI-100}{28-14D}\right)} \qquad (10.4.6.4\text{-}4)$$

10

~~Table 10.4.6.4-4—Approximate Relationship between Rock-Mass Quality and Material Constants Used in Defining Nonlinear Strength (Hoek and Brown, 1988)~~

~~Rock Quality~~	~~Constants~~	~~Rock Type~~ ~~A = Carbonate rocks with well developed crystal cleavage—*dolomite, limestone and marble*~~ ~~B = Lithified argrillaceous rocks—*mudstone, siltstone, shale and slate (normal to cleavage)*~~ ~~C = Arenaceous rocks with strong crystals and poorly developed crystal cleavage—*sandstone and quartzite*~~ ~~D = Fine grained polymineralllic igneous crystalline rocks—*andesite, dolerite, diabase and rhyolite*~~ ~~E = Coarse grained polyminerallic igneous & metamorphic crystalline rocks—*amphibolite, gabbro gneiss, granite, norite, quartz-diorite*~~				
		~~A~~	~~B~~	~~C~~	~~D~~	~~E~~
~~INTACT ROCK SAMPLES~~ ~~Laboratory size specimens free from discontinuities.~~ ~~CSIR rating: *RMR* = 100~~	~~*m*~~ ~~*s*~~	~~7.00~~ ~~1.00~~	~~10.00~~ ~~1.00~~	~~15.00~~ ~~1.00~~	~~17.00~~ ~~1.00~~	~~25.00~~ ~~1.00~~
~~VERY GOOD QUALITY ROCK MASS~~ ~~Tightly interlocking undisturbed rock with unweathered joints at 3–10 ft~~ ~~CSIR rating: *RMR* = 85~~	~~*m*~~ ~~*s*~~	~~2.40~~ ~~0.082~~	~~3.43~~ ~~0.082~~	~~5.14~~ ~~0.082~~	~~5.82~~ ~~0.082~~	~~8.567~~ ~~0.082~~
~~GOOD QUALITY ROCK MASS~~ ~~Fresh to slightly weathered rock, slightly disturbed with joints at 3–10 ft~~ ~~CSIR rating: *RMR* = 65~~	~~*m*~~ ~~*s*~~	~~0.575~~ ~~0.00293~~	~~0.821~~ ~~0.00293~~	~~1.231~~ ~~0.00293~~	~~1.395~~ ~~0.00293~~	~~2.052~~ ~~0.00293~~
~~FAIR QUALITY ROCK MASS~~ ~~Several sets of moderately weathered joints spaced at 1–3 ft~~ ~~CSIR rating: *RMR* = 44~~	~~*m*~~ ~~*s*~~	~~0.128~~ ~~0.00009~~	~~0.183~~ ~~0.00009~~	~~0.275~~ ~~0.00009~~	~~0.311~~ ~~0.00009~~	~~0.458~~ ~~0.00009~~
~~POOR QUALITY ROCK MASS~~ ~~Numerous weathered joints at 2 to 12 in.; some gouge. Clean compacted waste rock.~~ ~~CSIR rating: *RMR* = 23~~	~~*m*~~ ~~*s*~~	~~0.029~~ $\cancel{3 \times 10^{-6}}$	~~0.041~~ $\cancel{3 \times 10^{-6}}$	~~0.061~~ $\cancel{3 \times 10^{-6}}$	~~0.069~~ $\cancel{3 \times 10^{-6}}$	~~0.102~~ $\cancel{3 \times 10^{-6}}$
~~VERY POOR QUALITY ROCK MASS~~ ~~Numerous heavily weathered joints spaced <2 in. with gouge. Waste rock with fines.~~ ~~CSIR rating: *RMR* = 3~~	~~*m*~~ ~~*s*~~	~~0.007~~ $\cancel{1 \times 10^{-7}}$	~~0.010~~ $\cancel{1 \times 10^{-7}}$	~~0.015~~ $\cancel{1 \times 10^{-7}}$	~~0.017~~ $\cancel{1 \times 10^{-7}}$	~~0.025~~ $\cancel{1 \times 10^{-7}}$

Table 10.4.6.4-1—Values of the Constant m_i by Rock Group (after Marinos and Hoek 2000; with updated values from Rocscience, Inc., 2007)

Rock type	Class	Group	Texture: Coarse	Texture: Medium	Texture: Fine	Texture: Very fine
SEDIMENTARY	Clastic		Conglomerate (21 ± 3) Breccia (19 ± 5)	Sandstone 17 ± 4	Siltstone 7 ± 2 Greywacke (18 ± 3)	Claystone 4 ± 2 Shale (6 ± 2) Marl (7 ± 2)
SEDIMENTARY	Non-Clastic	Carbonates	Crystalline Limestone (12 ± 3)	Sparitic Limestone (10 ± 5)	Micritic Limestone (8 ± 3)	Dolomite (9 ± 3)
SEDIMENTARY	Non-Clastic	Evaporites		Gypsum 10 ± 2	Anhydrite 12 ± 2	
SEDIMENTARY	Non-Clastic	Organic				Chalk 7 ± 2
METAMORPHIC	Non Foliated		Marble 9 ± 3	Hornfels (19 ± 4)) Metasandstone (19 ± 3)	Quartzite 20 ± 3	
METAMORPHIC	Slightly foliated		Migmatite (29 ± 3)	Amphibolite 26 ± 6	Gneiss 28 ± 5	
METAMORPHIC	Foliated*			Schist (10 ± 3)	Phyllite (7 ± 3)	Slate 7 ± 4
IGNEOUS	Plutonic	Light	Granite 32 ± 3 Granodiorite (29 ± 3)	Diorite 25 ± 5		
IGNEOUS	Plutonic	Dark	Gabbro 27 ± 3 Norite 20 ± 5	Dolerite (16 ± 5)		
IGNEOUS	Hypabyssal		Porphyries (20 ± 5)		Diabase (15 ± 5)	Peridotite (25 ± 5)
IGNEOUS	Volcanic	Lava		Rhyolite (25 ± 5) Andesite 25 ± 5	Dacite (25 ± 3)) Basalt (25 ± 5)	
IGNEOUS	Volcanic	Pyroclastic	Agglomerate (19 ± 3)	Volcanic breccia (19 ± 5)	Tuff (13 ± 5)	

Disturbance to the foundation excavation caused by the rock removal methodology should be considered through the disturbance factor D in Eqs. 10.4.6.4-2 through 10.4.6.4-4.

The disturbance factor, D, ranges from 0 (undisturbed) to 1 (highly disturbed), and is an adjustment for the rock mass disturbance induced by the excavation method. Suggested values for various tunnel and slope excavations can be found in Hoek et al. (2002). However, these values may not directly applicable to foundations. If using blasting techniques to remove the rock in a shaft foundation, due to its confined state, a disturbance factor approaching 1.0 should be considered, as the blast energy will tend to radiate laterally into the intact rock, potentially disturbing the rock. If using rock coring techniques, much less disturbance is likely and a disturbance factor approaching 0 may be considered. If using a down hole hammer to break up the rock, the disturbance factor is likely between these two extremes.

Where it is necessary to evaluate the strength of a single discontinuity or set of discontinuities, the strength along the discontinuity should be determined as follows:

- For smooth discontinuities, the shear strength is represented by a friction angle of the parent rock material. To evaluate the friction angle of this type of discontinuity surface for design, direct shear tests on samples should be performed. Samples should be formed in the laboratory by cutting samples of intact core or, if possible, on actual discontinuities using an oriented shear box.
- For rough discontinuities the nonlinear criterion of Barton (1976) should be applied or, if possible, direct shear tests should be performed on actual discontinuities using an oriented shear box.

The range of typical friction angles provided in Table C10.4.6.4-1 may be used in evaluating measured values of friction angles for smooth joints.

Table C10.4.6.4-1—Typical Ranges of Friction Angles for Smooth Joints in a Variety of Rock Types (modified after Barton, 1976; Jaeger and Cook, 1976)

Rock Class	Friction Angle Range	Typical Rock Types
Low Friction	20–27°	Schists (high mica content), shale, marl
Medium Friction	27–34°	Sandstone, siltstone, chalk, gneiss, slate
High Friction	34–40°	Basalt, granite, limestone, conglomerate

Note: Values assume no infilling and little relative movement between joint faces.

When a major discontinuity with a significant thickness of infilling is to be investigated, the shear strength will be governed by the strength of the infilling material and the past and expected future displacement of the discontinuity. Refer to Sabatini et al. (2002) for detailed procedures to evaluate infilled discontinuities.

10.4.6.5—Rock Mass Deformation

The elastic modulus of a rock mass (E_m) shall be taken as the lesser of the intact modulus of a sample of rock core (E_R) or the modulus determined from ~~one of the following equations:~~ Table 10.4.6.5-1.

C10.4.6.5

~~Table 10.4.6.5-1 was developed by O'Neill and Reese (1999) based on a reanalysis of the data presented by Carter and Kulhawy (1988) for the purposes of estimating side resistance of shafts in rock.~~ Methods for establishing design values of E_m include:

13

~~$E_m = 145\left(10^{\frac{RMR-10}{40}}\right)$ (10.4.6.5-1)~~

~~where:~~

~~E_m = Elastic modulus of the rock mass (ksi)~~

~~$E_m \leq E_i$~~

~~E_i = Elastic modulus of intact rock (ksi)~~

~~RMR = Rock mass rating specified in Article 10.4.6.4.~~

~~or~~

~~$E_m = \left(\frac{E_m}{E_i}\right)E_i$ (10.4.6.5-2)~~

~~where:~~

~~E_m = Elastic modulus of the rock mass (ksi)~~

~~E_m/E_i = Reduction factor determined from Table 10.4.6.5-1 (dim)~~

~~E_i = Elastic modulus of intact rock from tests (ksi)~~

For critical or large structures, determination of rock mass modulus (E_m) using in-situ tests ~~may be warranted~~ should be considered. Refer to Sabatini et al. (2002) for descriptions of suitable in-situ tests.

- Empirical correlations that relate E_m to strength or modulus values of intact rock (q_u or E_R) and GSI
- Estimates based on previous experience in similar rocks or back-calculated from load tests
- In-situ testing such as pressuremeter test

Empirical correlations that predict rock mass modulus (E_m) from GSI and properties of intact rock, either uniaxial compressive strength (q_u) or intact modulus (E_R), are presented in Table 10.4.6.5-1. The recommended approach is to measure uniaxial compressive strength and modulus of intact rock in laboratory tests on specimens prepared from rock core. Values of GSI should be determined for representative zones of rock for the particular foundation design being considered. The correlation equations in Table 10.4.6.5-1 should then be used to evaluate modulus and its variation with depth. If pressuremeter tests are conducted, it is recommended that measured modulus values be calibrated to the values calculated using the relationships in Table 10.4.6.5-1.

Preliminary estimates of the elastic modulus of intact rock may be made from Table C10.4.6.5-1. Note that some of the rock types identified in the Table are not present in the U.S.

It is extremely important to use the elastic modulus of the rock mass for computation of displacements of rock materials under applied loads. Use of the intact modulus will result in unrealistic and unconservative estimates.

~~Table 10.4.6.5-1—Estimation of E_m Based on RQD (after O'Neill and Reese, 1999)~~

~~RQD (percent)~~	~~E_m/E_i~~	
	~~Closed Joints~~	~~Open Joints~~

~~*RQD* (percent)~~	~~E_m/E_i~~	
	~~Closed Joints~~	~~Open Joints~~
~~100~~	~~1.00~~	~~0.60~~
~~70~~	~~0.70~~	~~0.10~~
~~50~~	~~0.15~~	~~0.10~~
~~20~~	~~0.05~~	~~0.05~~

Table 10.4.6.5-1—Estimation of E_m Based on GSI

Expression	Notes/Remarks	Reference
$E_m(GPa) = \sqrt{\frac{q_u}{100}}\,10^{\frac{GSI-10}{40}}$ for $q_u \leq 100$ MPa $E_m(GPa) = 10^{\frac{GSI-10}{40}}$ for $q_u > 100$ MPa	Accounts for rocks with $q_u < 100$ MPa; *note* q_u in MPa	Hoek and Brown (1997); Hoek et al. (2002)
$E_m = \frac{E_R}{100} e^{GSI/21.7}$	Reduction factor on intact modulus, based on GSI	Yang (2006)
Notes: E_R = modulus of intact rock, E_m = equivalent rock mass modulus, GSI = geological strength index, q_u = uniaxial compressive strength. 1 MPa = 20.9 ksf.		

Table C10.4.6.5-1—Summary of Elastic Moduli for Intact Rock (modified after Kulhawy, 1978)

Rock Type	No. of Values	No. of Rock Types	Elastic Modulus, ~~E_i~~E_R (ksi $\times 10^3$)			Standard Deviation (ksi $\times 10^3$)
			Maximum	Minimum	Mean	
Granite	26	26	14.5	0.93	7.64	3.55
Diorite	3	3	16.2	2.48	7.45	6.19
Gabbro	3	3	12.2	9.8	11.0	0.97
Diabase	7	7	15.1	10.0	12.8	1.78
Basalt	12	12	12.2	4.20	8.14	2.60
Quartzite	7	7	12.8	5.29	9.59	2.32
Marble	14	13	10.7	0.58	6.18	2.49
Gneiss	13	13	11.9	4.13	8.86	2.31
Slate	11	2	3.79	0.35	1.39	0.96
Schist	13	12	10.0	0.86	4.97	3.18
Phyllite	3	3	2.51	1.25	1.71	0.57
Sandstone	27	19	5.68	0.09	2.13	1.19
Siltstone	5	5	4.76	0.38	2.39	1.65
Shale	30	14	5.60	0.001	1.42	1.45
Limestone	30	30	13.0	0.65	5.7	3.73
Dolostone	17	16	11.4	0.83	4.22	3.44

Poisson's ratio for rock should be determined from tests on intact rock core.

Where tests on rock core are not practical, Poisson's ratio may be estimated from Table C10.4.6.5-2.

Table C10.4.6.5-2—Summary of Poisson's Ratio for Intact Rock (modified after Kulhawy, 1978)

Rock Type	No. of Values	No. of Rock Types	Poisson's Ratio, ν Maximum	Minimum	Mean	Standard Deviation
Granite	22	22	0.39	0.09	0.20	0.08
Gabbro	3	3	0.20	0.16	0.18	0.02
Diabase	6	6	0.38	0.20	0.29	0.06
Basalt	11	11	0.32	0.16	0.23	0.05
Quartzite	6	6	0.22	0.08	0.14	0.05
Marble	5	5	0.40	0.17	0.28	0.08
Gneiss	11	11	0.40	0.09	0.22	0.09
Schist	12	11	0.31	0.02	0.12	0.08
Sandstone	12	9	0.46	0.08	0.20	0.11
Siltstone	3	3	0.23	0.09	0.18	0.06
Shale	3	3	0.18	0.03	0.09	0.06
Limestone	19	19	0.33	0.12	0.23	0.06
Dolostone	5	5	0.35	0.14	0.29	0.08

10.4.6.6—Erodibility of Rock - *NO CHANGES – NOT SHOWN*

10.5—LIMIT STATES AND RESISTANCE FACTORS

10.5.1—General – *NO CHANGES – NOT SHOWN*

10.5.2—Service Limit States – *NO CHANGES – NOT SHOWN*

10.5.3—Strength Limit States – *NO CHANGES – NOT SHOWN*

10.5.4—Extreme Events Limit States – *NO CHANGES – NOT SHOWN*

10.5.5—Resistance Factors

10.5.5.1—Service Limit States – *NO CHANGES – NOT SHOWN*

10.5.5.2—Strength Limit States

10.5.5.2.1—General - ***NO CHANGES – NOT SHOWN***

10.5.5.2.2—Spread Footings - ***NO CHANGES – NOT SHOWN***

10.5.5.2.3—Driven Piles - ***NO CHANGES – NOT SHOWN***

10.5.5.2.4—Drilled Shafts

Resistance factors shall be selected based on the

C10.5.5.2.4

The resistance factors in Table 10.5.5.2.4-1 were method used for determining the nominal shaft resistance. When selecting a resistance factor for shafts in clays or other easily disturbed formations, local experience with the geologic formations and with typical shaft construction practices shall be considered.

Where the resistance factors provided in Table 10.5.5.2.4-1 are to be applied to a single shaft supporting a bridge pier, the resistance factor values in the Table should be reduced by 20 percent. Where the resistance factor is decreased in this manner, the η_R factor provided in Article 1.3.4 shall not be increased to address the lack of foundation redundancy.

The number of static load tests to be conducted to justify the resistance factors provided in Table 10.5.5.2.4-1 shall be based on the variability in the properties and geologic stratification of the site to which the test results are to be applied. A site, for the purpose of assessing variability, shall be defined ~~in accordance with Article 10.5.5.2.3.~~as a project site, or a portion of it, where the subsurface conditions can be characterized as geologically similar in terms of subsurface stratification, i.e., sequence, thickness, and geologic history of strata, the engineering properties of the strata, and groundwater conditions.

developed using either statistical analysis of shaft load tests combined with reliability theory (Paikowsky et al., 2004), fitting to allowable stress design (ASD), or both. Where the two approaches resulted in a significantly different resistance factor, engineering judgment was used to establish the final resistance factor, considering the quality and quantity of the available data used in the calibration. The available reliability theory calibrations were conducted for the Reese and O'Neill (1988) method, with the exception of shafts in cohesive intermediate geo-materials (IGMs), in which case the O'Neill and Reese (1999) method was used. In Article 10.8, the O'Neill and Reese (1999) method is recommended. See Allen (2005) for a more detailed explanation on the development of the resistance factors for shaft foundation design, and the implications of the differences in these two shaft design methods on the selection of resistance factors.

~~The information in the commentary to Article 10.5.5.2.3 regarding the number of load tests to conduct considering site variability applies to drilled shafts as well.~~

For single shafts, lower resistance factors are specified to address the lack of redundancy. See Article C10.5.5.2.3 regarding the use of η_R.

Where installation criteria are established based on one or more static load tests, the potential for site variability should be considered. The number of load tests required should be established based on the characterization of site subsurface conditions by the field and laboratory exploration and testing program. One or more static load tests should be performed per site to justify the resistance factor selection ~~as discussed in Article C10.5.5.2.3, applied to drilled shafts installed within the site. See Article C10.5.5.2.3 for details on assessing site variability as applied to selection and use of load tests~~.

Site variability is the most important consideration in evaluating the limits of a site for design purposes. Defining the limits of a site therefore requires sufficient knowledge of the subsurface conditions in terms of general geology, stratigraphy, index and engineering properties of soil and rock, and groundwater conditions. This implies that the extent of the exploration program is sufficient to define the subsurface conditions and their variation across the site.

A designer may choose to design drilled shaft foundations for strength limit states based on a calculated nominal resistance, with the expectation that load testing results will verify that value. The question arises whether to use the resistance factor associated with the design equation or the higher value allowed for load testing. This choice should be based on engineering judgment. The potential risk is that axial resistance measured by load testing may be lower than the nominal resistance used for design, which could require increased shaft dimensions that may be problematic, depending upon the capability of the drilled shaft equipment mobilized for the project and other project-specific factors.

For the specific case of shafts in clay, the resistance factor recommended by Paikowsky et al. (2004) is much lower than the recommendation from Barker et al. (1991). Since the shaft design method for clay is nearly the same for both the 1988 and 1999 methods, a resistance factor that represents the average of the two resistance factor recommendations is provided in Table 10.5.5.2.4-1. This difference may point to the differences in local geologic formations and local construction practices, pointing to the importance of taking such issues into consideration when selecting resistance factors, especially for shafts in clay.

Cohesive IGMs are materials that are transitional between soil and rock in terms of their strength and compressibility, such as ~~residual soils, glacial tills, or very weak rock. See Article C10.8.2.2.3 for a more detailed definition of an IGM.~~clay shales or mudstones with undrained shear strength between 5 and 50 ksf.

Since the mobilization of shaft base resistance is less certain than side resistance due to the greater deformation required to mobilize the base resistance, a lower resistance factor relative to the side resistance is provided for the base resistance in Table 10.5.5.2.4-1. O'Neill and Reese (1999) make further comment that the recommended resistance factor for tip resistance in sand is applicable for conditions of high quality control on the properties of drilling slurries and base cleanout procedures. If high quality control procedures are not used, the resistance factor for the O'Neill and Reese (1999) method for tip resistance in sand should be also be reduced. The amount of reduction should be based on engineering judgment.

Shaft compression load test data should be extrapolated to production shafts that are not load tested as specified in Article 10.8.3.5.6. There is no way to verify shaft resistance for the untested production shafts, other than through good construction inspection and visual observation of the soil or rock encountered in each shaft. Because of this, extrapolation of the shaft load test results to the untested production shafts may introduce some uncertainty. Statistical data are not available to quantify this at this time. Historically, resistance factors higher than 0.70, or their equivalent safety factor in previous practice, have not been used for shaft foundations. If the recommendations in Paikowsky, et al. (*2004*) are used to establish a resistance factor when shaft static load tests are conducted, in consideration of site variability, the resistance factors recommended by Paikowsky, et al. for this case should be reduced by 0.05, and should be less than or equal to 0.70 as specified in Table 10.5.5.2.4-1.

This issue of uncertainty in how the load test is applied to shafts not load tested is even more acute for shafts subjected to uplift load tests, as failure in uplift can be more abrupt than failure in compression. Hence, a resistance factor of 0.60 for the use of uplift load test

18

results is recommended.

19

Table 10.5.5.2.4-1—Resistance Factors for Geotechnical Resistance of Drilled Shafts

Method/Soil/Condition			Resistance Factor
Nominal Axial Compressive Resistance of Single-Drilled Shafts, φ_{stat}	Side resistance in clay	α-method (~~O'Neill and Reese, 1999~~ Brown et al., 2010)	0.45
	Tip resistance in clay	Total Stress (~~O'Neill and Reese, 1999~~ Brown et al., 2010)	0.40
	Side resistance in sand	β-method (~~O'Neill and Reese, 1999~~ Brown et al., 2010)	0.55
	Tip resistance in sand	~~O'Neill and Reese (1999)~~ Brown et al., (2010)	0.50
	Side resistance in cohesive IGMs	~~O'Neill and Reese (1999)~~ Brown et al., (2010)	0.60
	Tip resistance in cohesive IGMs	~~O'Neill and Reese (1999)~~ Brown et al., (2010)	0.55
	Side resistance in rock	~~Horvath and Kenney (1979)~~ ~~O'Neill and Reese (1999)~~ Kulhawy et al. (2005) Brown et al. (2010)	0.55
	Side resistance in rock	Carter and Kulhawy (1988)	0.50
	Tip resistance in rock	Canadian Geotechnical Society (1985) Pressuremeter Method (Canadian Geotechnical Society, 1985) ~~O'Neill and Reese (1999)~~Brown et al. (2010)	0.50
Block Failure, φ_{b1}	Clay		0.55
Uplift Resistance of Single-Drilled Shafts, φ_{up}	Clay	α-method (~~O'Neill and Reese, 1999~~ Brown et al., 2010)	0.35
	Sand	β-method (~~O'Neill and Reese, 1999~~ Brown et al., 2010)	0.45
	Rock	~~Horvath and Kenney (1979)~~ ~~O'Neill and Reese (1999)~~ Kulhawy et al. (2005) Brown et al. (2010)	0.40
Group Uplift Resistance, φ_{ug}	Sand and clay		0.45
Horizontal Geotechnical Resistance of Single Shaft or Shaft Group	All materials		1.0
Static Load Test (compression), φ_{load}	All Materials		0.70
Static Load Test (uplift), φ_{upload}	All Materials		0.60

10.5.5.2.5—Micropiles - ***NO CHANGES – NOT SHOWN***

10.5.5.3—Extreme Limit States – ***NO CHANGES – NOT SHOWN***

10.6—SPREAD FOOTINGS

10.6.1—General Considerations – ***NO CHANGES – NOT SHOWN***

10.6.2—Service Limit State Design

10.6.2.1—General – ***NO CHANGES – NOT SHOWN***

10.6.2.2—Tolerable Movements – ***NO CHANGES – NOT SHOWN***

10.6.2.3—Loads – ***NO CHANGES – NOT SHOWN***

10.6.2.4—Settlement Analyses

10.6.2.4.1—General - ***NO CHANGES – NOT SHOWN***

10.6.2.4.2—Settlement of Footings on Cohesionless Soils - ***NO CHANGES – NOT SHOWN***

10.6.2.4.3—Settlement of Footings on Cohesive Soils - ***NO CHANGES – NOT SHOWN***

10.6.2.4.4—Settlement of Footings on Rock

For footings bearing on fair to very good rock, according to the Geomechanics Classification system~~, as defined in Article 10.4.6.4,~~ and designed in accordance with the provisions of this Section, elastic settlements may generally be assumed to be less than 0.5 in. When elastic settlements of this magnitude are unacceptable or when the rock is not competent, an analysis of settlement based on rock mass characteristics shall be made.

Where rock is broken or jointed (relative rating of ten or less for *RQD* and joint spacing), the rock joint condition is poor (relative rating of ten or less) or the criteria for fair to very good rock are not met, a settlement analysis should be conducted, and the influence of rock type, condition of discontinuities, and degree of weathering shall be considered in the settlement analysis.

C10.6.2.4.4

In most cases, it is sufficient to determine settlement using the average bearing stress under the footing.

Where the foundations are subjected to a very large load or where settlement tolerance may be small, settlements of footings on rock may be estimated using elastic theory. The stiffness of the rock mass should be used in such analyses.

The accuracy with which settlements can be estimated by using elastic theory is dependent on the accuracy of the estimated rock mass modulus, E_m. In some cases, the value of E_m can be estimated through empirical correlation with the value of the modulus of elasticity for the intact rock between joints. For unusual or poor rock mass conditions, it may be necessary to determine the modulus from in-situ tests, such as plate loading and pressuremeter tests.

The elastic settlement of footings on broken or jointed rock, in feet, should be taken as:

- For circular (or square) footings:

$$\rho = q_o\left(1-\nu^2\right)\frac{rI_p}{144\,E_m} \quad (10.6.2.4.4\text{-}1)$$

in which:

$$I_p = \frac{\left(\sqrt{\pi}\right)}{\beta_z} \quad (10.6.2.4.4\text{-}2)$$

- For rectangular footings:

$$\rho = q_o\left(1-\nu^2\right)\frac{BI_p}{144\,E_m} \quad (10.6.2.4.4\text{-}3)$$

in which:

$$I_p = \frac{\left(L/B\right)^{1/2}}{\beta_z} \quad (10.6.2.4.4\text{-}4)$$

where:

q_o = applied vertical stress at base of loaded area (ksf)

ν = Poisson's Ratio (dim)

r = radius of circular footing or $B/2$ for square footing (ft)

I_p = influence coefficient to account for rigidity and dimensions of footing (dim)

E_m = rock mass modulus (ksi)

β_z = factor to account for footing shape and rigidity (dim)

Values of I_p should be computed using the β_z values presented in Table 10.6.2.4.2-1 for rigid footings. Where the results of laboratory testing are not available, values of Poisson's ratio, ν, for typical rock types may be taken as specified in Table C10.4.6.5-2. Determination of the rock mass modulus, E_m, should be based on the methods described in ~~Article 10.4.6.5~~ Sabatini (2002).

The magnitude of consolidation and secondary settlements in rock masses containing soft seams or other material with time-dependent settlement characteristics should be estimated by applying procedures specified in Article 10.6.2.4.3.

10.6.2.5—Overall Stability – *NO CHANGES* –

NOT SHOWN

10.6.2.6—Bearing Resistance at the Service Limit State

10.6.2.6.1—Presumptive Values for Bearing Resistance ***– NO CHANGES – NOT SHOWN***

10.6.2.6.2—Semiempirical Procedures for Bearing Resistance

Bearing resistance on rock shall be determined using empirical correlation to the Geomechanic Rock Mass Rating System, RMR~~, as specified in Article 10.4.6.4~~. Local experience should be considered in the use of these semi-empirical procedures.

If the recommended value of presumptive bearing resistance exceeds either the unconfined compressive strength of the rock or the nominal resistance of the concrete, the presumptive bearing resistance shall be taken as the lesser of the unconfined compressive strength of the rock or the nominal resistance of the concrete. The nominal resistance of concrete shall be taken as 0.3 f'_c.

10.6.3—Strength Limit State Design

10.6.3.1—Bearing Resistance of Soil – ***NO CHANGES – NOT SHOWN***

10.6.3.2—Bearing Resistance of Rock

10.6.3.2.1—General

The methods used for design of footings on rock shall consider the presence, orientation, and condition of discontinuities, weathering profiles, and other similar profiles as they apply at a particular site.

For footings on competent rock, reliance on simple and direct analyses based on uniaxial compressive rock strengths and *RQD* may be applicable. For footings on less competent rock, more detailed investigations and analyses shall be performed to account for the effects of weathering and the presence and condition of discontinuities.

The designer shall judge the competency of a rock mass by taking into consideration both the nature of the intact rock and the orientation and condition of discontinuities of the overall rock mass. Where engineering judgment does not verify the presence of competent rock, the competency of the rock mass should be verified using the procedures for *RMR* rating ~~in Article 10.4.6.4~~.

C10.6.3.2.1

The design of spread footings bearing on rock is frequently controlled by either overall stability, i.e., the orientation and conditions of discontinuities, or load eccentricity considerations. The designer should verify adequate overall stability at the service limit state and size the footing based on eccentricity requirements at the strength limit state before checking nominal bearing resistance at both the service and strength limit states.

The design procedures for foundations in rock have been developed using the RMR rock mass rating system. Classification of the rock mass should be according to the RMR system. For additional information on the RMR system, see Sabatini et al. (2002).

10.6.3.2.2—Semiempirical Procedures - ***NO CHANGES – NOT SHOWN***

10.6.3.2.3—Analytic Method - ***NO CHANGES – NOT SHOWN***

10.6.3.2.4—Load Test - ***NO CHANGES – NOT SHOWN***

10.6.3.3—Eccentric Load Limitations – ***NO CHANGES – NOT SHOWN***

10.6.3.4—Failure by Sliding – ***NO CHANGES – NOT SHOWN***

10.6.4—Extreme Event Limit State Design – ***NO CHANGES – NOT SHOWN***

10.6.5—Structural Design – ***NO CHANGES – NOT SHOWN***

10.7—DRIVEN PILES – ***NO CHANGES – NOT SHOWN***

10.8—DRILLED SHAFTS

10.8.1—General

10.8.1.1—Scope - ***NO CHANGES – NOT SHOWN***

10.8.1.2—Shaft Spacing, Clearance, and Embedment into Cap - ***NO CHANGES – NOT SHOWN***

10.8.1.3—Shaft Diameter and Enlarged Bases - ***NO CHANGES – NOT SHOWN***

10.8.1.4—Battered Shafts - ***NO CHANGES – NOT SHOWN***

10.8.1.5—Drilled Shaft Resistance

Drilled shafts shall be designed to have adequate axial and structural resistances, tolerable settlements, and tolerable lateral displacements.

C10.8.1.5

The drilled shaft design process is discussed in detail in Drilled Shafts: Construction Procedures and Design Methods (~~O'Neill and Reese, 1999~~ Brown, et al., 2010).

The axial resistance of drilled shafts shall be determined through a suitable combination of subsurface investigations, laboratory and/or in-situ tests, analytical methods, and load tests, with reference to the history of past performance. Consideration shall also be given to:

- The difference between the resistance of a single shaft and that of a group of shafts;
- The resistance of the underlying strata to support the load of the shaft group;
- The effects of constructing the shaft(s) on adjacent structures;
- The possibility of scour and its effect;
- The transmission of forces, such as downdrag forces, from consolidating soil;
- Minimum shaft penetration necessary to satisfy the requirements caused by uplift, scour, downdrag, settlement, liquefaction, lateral loads and seismic conditions;
- Satisfactory behavior under service loads;
- Drilled shaft nominal structural resistance; and
- Long-term durability of the shaft in service, i.e., corrosion and deterioration.

Resistance factors for shaft axial resistance for the strength limit state shall be as specified in Table 10.5.5.2.4-1.

The method of construction may affect the shaft axial and lateral resistance. The shaft design parameters shall take into account the likely construction methodologies used to install the shaft.

The performance of drilled shaft foundations can be greatly affected by the method of construction, particularly side resistance. The designer should consider the effects of ground and groundwater conditions on shaft construction operations and delineate, where necessary, the general method of construction to be followed to ensure the expected performance. Because shafts derive their resistance from side and tip resistance, which is a function of the condition of the materials in direct contact with the shaft, it is important that the construction procedures be consistent with the material conditions assumed in the design. Softening, loosening, or other changes in soil and rock conditions caused by the construction method could result in a reduction in shaft resistance and an increase in shaft displacement. Therefore, evaluation of the effects of the shaft construction procedure on resistance should be considered an inherent aspect of the design. Use of slurries, varying shaft diameters, and post grouting can also affect shaft resistance.

Soil parameters should be varied systematically to model the range of anticipated conditions. Both vertical and lateral resistance should be evaluated in this manner.

Procedures that may affect axial or lateral shaft resistance include, but are not limited to, the following:

- Artificial socket roughening, if included in the design nominal axial resistance assumptions.
- Removal of temporary casing where the design is dependent on concrete-to-soil adhesion.
- The use of permanent casing.
- Use of tooling that produces a uniform cross-section where the design of the shaft to resist lateral loads cannot tolerate the change in stiffness if telescoped casing is used.

It should be recognized that the design procedures provided in these Specifications assume compliance to construction specifications that will produce a high quality shaft. Performance criteria should be included in the construction specifications that require:

- Shaft bottom cleanout criteria,
- Appropriate means to prevent side wall movement or failure (caving) such as temporary casing, slurry, or a combination of the two,
- Slurry maintenance requirements including minimum slurry head requirements, slurry testing requirements, and maximum time the shaft may be left open before concrete placement.

If for some reason one or more of these performance criteria are not met, the design should be reevaluated and the shaft repaired or replaced as necessary.

10.8.1.6—Determination of Shaft Loads

10.8.1.6.1—General - ***NO CHANGES – NOT SHOWN***

10.8.1.6.2—Downdrag

The provisions of Articles 10.7.1.6.2 and 3.11.8 shall apply for determination of load due to downdrag.

For shafts with tip bearing in a dense stratum or rock where design of the shaft is structurally controlled,~~and~~ downdrag shall be considered at the strength and extreme event limit states.

For shafts with tip bearing in soil, downdrag shall not be considered at the strength and extreme limit states if settlement of the shaft is less than failure criterion.

C10.8.1.6.2

See commentary to Articles ~~10.7.1.6.2 and~~ 3.11.8.

Downdrag loads may be estimated using the α-method, as specified in Article 10.8.3.5.1b, ~~for calculating~~ to calculate negative shaft ~~resistance~~ friction. As with positive shaft resistance, ~~the top 5.0 ft and a bottom length taken as one shaft diameters~~ shaft length assumed to not contribute to nominal side resistance should also be assumed to not contribute to downdrag loads.

When using the α-method, an allowance should be made for a possible increase in the undrained shear strength as consolidation occurs. Downdrag loads may also come from cohesionless soils above settling cohesive soils~~, requiring granular soil friction methods be used in such zones to estimate downdrag loads~~. The downdrag caused by settling cohesionless soils may be estimated using the β method presented in Article 10.8.3.5.2.

Downdrag occurs in response to relative downward deformation of the surrounding soil to that of the shaft, and may not exist if downward movement of the drilled shaft in response to axial compression forces exceeds the vertical deformation of the soil. The response of a drilled shaft to downdrag in combination with the other forces acting at the head of the shaft therefore is complex and a realistic evaluation of actual limit states that may occur requires careful consideration of two issues: (1) drilled shaft load-settlement behavior, and (2) the time period over which downdrag occurs relative to the time period over which nonpermanent components of load occur. When these factors are taken into account, it is appropriate to consider different downdrag forces for evaluation of geotechnical strength limit states than for structural strength limit states. These issues are addressed in Brown et al. (2010).

10.8.1.6.3—Uplift - ***NO CHANGES – NOT SHOWN***

10.8.2—Service Limit State Design

10.8.2.1—Tolerable Movements - ***NO CHANGES – NOT SHOWN***

10.8.2.2—Settlement

10.8.2.2.1—General - ***NO CHANGES – NOT***

SHOWN

10.8.2.2.2—Settlement of Single-Drilled Shaft

The settlement of single-drilled shafts shall be estimated ~~in consideration of~~ as a sum of the following:

- Short-term settlement resulting from load transfer,
- Consolidation settlement if constructed ~~in~~ where cohesive soils~~s~~ exists beneath the shaft tip, and
- Axial compression of the shaft.

The normalized load-settlement curves shown in Figures 10.8.2.2.2-1 through 10.8.2.2.2-4 should be used to limit the nominal shaft axial resistance computed as specified for the strength limit state in Article 10.8.3 for service limit state tolerable movements. Consistent values of normalized settlement shall be used for limiting the base and side resistance when using these Figures. Long-term settlement should be computed according to Article 10.7.2 using the equivalent footing method and added to the short-term settlements estimated using Figures 10.8.2.2.2-1 through 10.8.2.2.2-4.

~~Other methods for evaluating shaft settlements that may be used are found in O'Neill and Reese (1999).~~

C10.8.2.2.2

O'Neill and Reese (1999) have summarized load-settlement data for drilled shafts in dimensionless form, as shown in Figures 10.8.2.2.2-1 through 10.8.2.2.2-4. These curves do not include consideration of long-term consolidation settlement for shafts in cohesive soils. Figures 10.8.2.2.2-1 and 10.8.2.2.2-2 show the load-settlement curves in side resistance and in end bearing for shafts in cohesive soils. Figures 10.8.2.2.2-3 and 10.8.2.2.2-4 are similar curves for shafts in cohesionless soils. These curves should be used for estimating short-term settlements of drilled shafts.

The designer should exercise judgment relative to whether the trend line, one of the limits, or some relation in between should be used from Figures 10.8.2.2.2-1 through 10.8.2.2.2-4.

The values of the load-settlement curves in side resistance were obtained at different depths, taking into account elastic shortening of the shaft. Although elastic shortening may be small in relatively short shafts, it may be substantial in longer shafts. The amount of elastic shortening in drilled shafts varies with depth. O'Neill and Reese (1999) have described an approximate procedure for estimating the elastic shortening of long-drilled shafts.

~~Settlements induced by loads in end bearing are different for shafts in cohesionless soils and in cohesive soils. Although drilled shafts in cohesive soils typically have a well-defined break in a load-displacement curve, shafts in cohesionless soils often have no well-defined failure at any displacement. The resistance of drilled shafts in cohesionless soils continues to increase as the settlement increases beyond five percent of the base diameter. The shaft end bearing R_p is typically fully mobilized at displacements of two to five percent of the base diameter for shafts in cohesive soils. The unit end bearing resistance for the strength limit state (see Article 10.8.3.3) is defined as the bearing pressure required to cause vertical deformation equal to five percent of the shaft diameter, even though this does not correspond to complete failure of the soil beneath the base of the shaft.~~

Induced settlements for isolated drilled shafts are different for elements in cohesive soils and in cohesionless soils. In cohesive soils, the failure threshold, or nominal axial resistance corresponds to mobilization of the full available side resistance, plus the full available base resistance. In cohesive soils, the failure threshold has been shown to occur at an average normalized deformation of 4 percent of the shaft diameter. In cohesionless soils, the failure threshold is the force corresponding to mobilization of the full side resistance, plus the base resistance corresponding to settlement at a defined failure criterion. This has been traditionally defined as the bearing pressure required to cause vertical deformation equal to 5 percent of the shaft diameter, even though this does not correspond to complete failure of the soil beneath the base of the shaft. Note that nominal base resistance in cohesionless soils is calculated according to the empirical correlation given by Eq. 10.8.3.5.2c-1 in terms of N-value. That relationship was developed using a base resistance corresponding to 5 percent normalized displacement. If a normalized displacement other than 5 percent is used, the base resistance calculated by Eq. 10.8.3.5.2c-1 must be corrected.

The curves in Figures 10.8.2.2.2-1 and 10.8.2.2.2-3 also show the settlements at which the side resistance is mobilized. The shaft skin friction R_s is typically fully mobilized at displacements of 0.2 percent to 0.8 percent of the shaft diameter for shafts in cohesive soils. For shafts in cohesionless soils, this value is 0.1 percent to 1.0 percent.

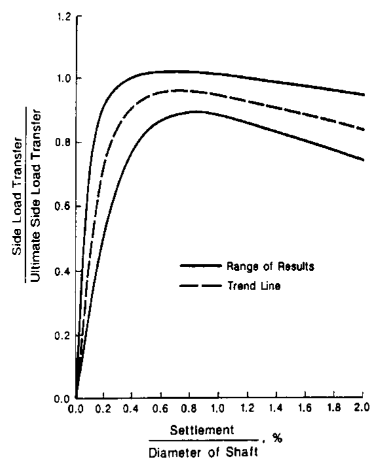


Figure 10.8.2.2.2-1 Normalized Load Transfer in Side Resistance versus Settlement in Cohesive Soils (from O'Neill and Reese, 1999)

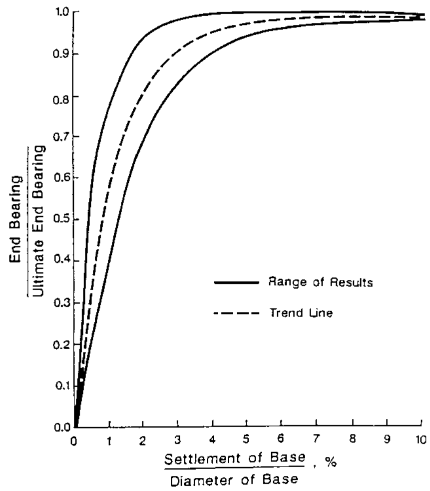


Figure 10.8.2.2.2-2—Normalized Load Transfer in End Bearing versus Settlement in Cohesive Soils (from O'Neill and Reese, 1999)

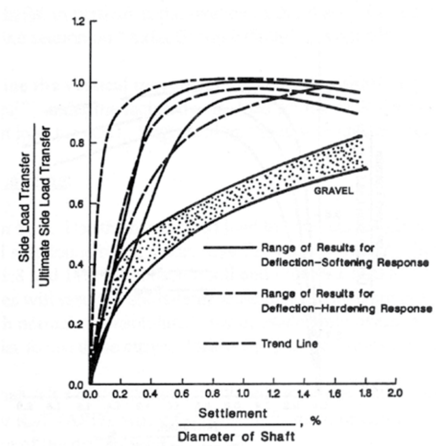


Figure 10.8.2.2.2-3—Normalized Load Transfer in Side Resistance versus Settlement in Cohesionless Soils (from O'Neill and Reese, 1999)

The deflection-softening response typically applies to cemented or partially cemented soils, or other soils that exhibit brittle behavior, having low residual shear strengths at larger deformations. Note that the trend line for sands is a reasonable approximation for either the deflection-softening or deflection-hardening response.

The normalized load-settlement curves require separate evaluation of an isolated drilled shaft for side and base resistance. Brown et al. (2010) provide alternate normalized load-settlement curves that may be used for estimation of settlement of a single drilled shaft considering combined side and base resistance. The method is based on modeling the average load deformation behavior observed from field load tests and incorporates the load test data used in development of the curves provided by O'Neill and Reese (1999). Additional methods that consider numerical simulations of axial load transfer and approximations based on elasto-plastic solutions are available in Brown et al. (2010).

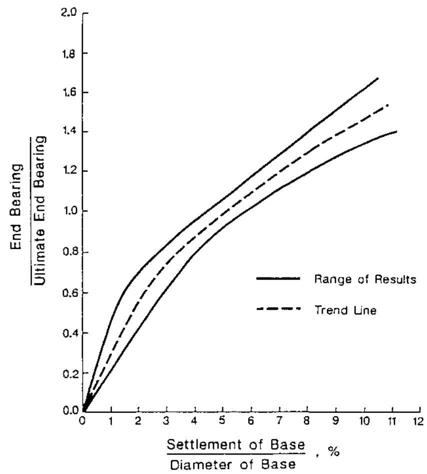


Figure 10.8.2.2.2-4—Normalized Load Transfer in End Bearing versus Settlement in Cohesionless Soils (from O'Neill and Reese, 1999)

10.8.2.2.3—Intermediate Geomaterials (IGMs)

For detailed settlement estimation of shafts in IGMs, the procedures ~~provided by O'Neill and Reese (1999)~~ described by Brown et al. (2010) should be used.

C10.8.2.2.3

IGMs are defined by ~~O'Neill and Reese (1999)~~ Brown et al. (2010) as follows:

- *Cohesive IGM*—clay shales or mudstones with an S_u of 5 to 50 ksf, and
- ~~*Cohesionless*—granular tills or granular residual soils with $N1_{60}$ greater than 50 blows/ft.~~

10.8.2.2.4—Group Settlement

The provisions of Article 10.7.2.3 shall apply. Shaft group effect shall be considered for groups of 2 shafts or more.

C10.8.2.2.4

See commentary to Article 10.7.2.3.

O'Neill and Reese (1999) summarize various studies on the effects of shaft group behavior. These studies were for groups that consisted of 1 × 2 to 3 × 3 shafts. These studies suggest that group effects are relatively unimportant for shaft center-to-center spacing of 5*D* or greater.

10.8.2.3—Horizontal Movement of Shafts and Shaft Groups

The provisions of Articles 10.5.2.1 and 10.7.2.4 shall apply.

For shafts socketed into rock, the input properties used to determine the response of the rock to lateral loading shall consider both the intact shear strength of the rock and the rock mass characteristics. The designer shall also consider the orientation and condition of discontinuities of the overall rock mass. Where specific adversely oriented discontinuities are not present, but the rock mass is fractured such that its intact strength is

C10.8.2.3

See commentary to Articles 10.5.2.1 and 10.7.2.4.

For shafts socketed into rock, approaches to developing p-y response of rock masses include both a weak rock response and a strong rock response. For the strong rock response, the potential for brittle fracture should be considered. If horizontal deflection of the rock mass is greater than 0.0004b, a lateral load test to evaluate the response of the rock to lateral loading should be considered. Brown et al. (2010) provide a considered compromised, the rock mass shear strength parameters should be assessed using the procedures for *GSI* rating in Article 10.4.6.4. For lateral deflection of the rock adjacent to the shaft greater than 0.0004b, where b is the diameter of the rock socket, the potential for brittle fracture of the rock shall be considered.

summary of a methodology that may be used to estimate the lateral load response of shafts in rock. Additional background on lateral loading of shafts in rock is provided in Turner (2006).

These methods for estimating the response of shafts in rock subjected to lateral loading use the unconfined compressive strength of the intact rock as the main input property. While this property is meaningful for intact rock, and was the key parameter used to correlate to shaft lateral load response in rock, it is not meaningful for fractured rock masses. If the rock mass is fractured enough to justify characterizing the rock shear strength using the GSI, the rock mass should be characterized as a c-ϕ material, and confining stress (i.e., σ'_3) present within the rock mass should be considered when establishing a rock mass shear strength for lateral response of the shaft. If the P-y method of analysis is used to model horizontal resistance, user-specified P-y curves should be derived. A method for developing hyperbolic P-y curves is described by Liang et al. (2009).

10.8.2.4—Settlement Due to Downdrag - *NO CHANGES – NOT SHOWN*

10.8.2.5—Lateral Squeeze - *NO CHANGES – NOT SHOWN*

10.8.3—Strength Limit State Design

10.8.3.1—General - *NO CHANGES – NOT SHOWN*

10.8.3.2—Groundwater Table and Buoyancy - *NO CHANGES – NOT SHOWN*

10.8.3.3—Scour - *NO CHANGES – NOT SHOWN*

10.8.3.4—Downdrag

~~The provisions of Article 10.7.3.7 shall apply.~~

The foundation should be designed so that the available factored axial geotechnical resistance is greater than the factored loads applied to the shaft, including the downdrag, at the strength limit state. The nominal shaft resistance available to support structure loads plus downdrag shall be estimated by considering only the positive skin and tip resistance below the lowest layer contributing to the downdrag. The drilled shaft shall be designed structurally to resist the downdrag plus structure loads.

C10.8.3.4

~~See commentary to Article 10.7.3.7.~~

The static analysis procedures in Article 10.8.3.5 may be used to estimate the available drilled shaft nominal side and tip resistances to withstand the downdrag plus other axial force effects.

Nominal resistance may also be estimated using an instrumented static load test provided the side resistance within the zone contributing to downdrag is subtracted from the resistance determined from the load test.

As stated in Article C10.8.1.6.2, ~~that~~ it is appropriate to apply different downdrag forces for evaluation of geotechnical strength limit states than for structural strength limit states. A drilled shaft with its tip bearing in stiff material, such as rock or hard soil, would be expected to limit settlement to very small values. In this case, the full downdrag force could occur in combination with the other axial force effects, because downdrag will not be reduced if there is little or no downward movement of the shaft. Therefore, the factored force effects resulting from all load components, including full factored downdrag, should be used to check the structural strength limit state of the drilled shaft.

A rational approach to evaluating this strength limit state will incorporate the force effects occurring at this magnitude of downward displacement. This will include the factored axial force effects transmitted to the head of the shaft, plus the downdrag loads occurring at a downward displacement defining the failure criterion. In many cases, this amount of downward displacement will reduce or eliminate downdrag. For soil layers that undergo settlement exceeding the failure criterion (for example, 5% of B for shafts bearing in sand), downdrag loads are likely to remain and should be included. This approach requires the designer to predict the magnitude of downdrag load occurring at a specified downward displacement. This can be accomplished using the hand calculation procedure described in Brown et al. (2010) or with commercially available software.

When downdrag loads are determined to exist at a downward displacement defining failure, evaluation of drilled shafts for the geotechnical strength limit state in compression should be conducted under a load combination that is limited to permanent loads only, including the calculated downdrag load at a settlement defining the failure criterion, but excluding nonpermanent loads, such as live load, temperature changes, etc. See Brown et al. (2010) for further discussion.

When analysis of a shaft subjected to downdrag shows that the downdrag load would be eliminated in order to achieve a defined downward displacement, evaluation of geotechnical and structural strength limit states in compression should be conducted under the full load combination corresponding to the relevant strength limit state, including the non-permanent components of load, but not including downdrag.

10.8.3.5—Nominal Axial Compression Resistance of Single Drilled Shafts - *NO CHANGES – NOT SHOWN*

10.8.3.5.1—Estimation of Drilled Shaft Resistance in Cohesive Soils

10.8.3.5.1a—General - ***NO CHANGES – NOT SHOWN***

10.8.3.5.1b—Side Resistance

The nominal unit side resistance, q_s, in ksf, for shafts in cohesive soil loaded under undrained loading conditions by the α-Method shall be taken as:

C10.8.3.5.1b

The α-method is based on total stress. For effective stress methods for shafts in clay, see ~~O'Neill and Reese (1999)~~ Brown et al. (2010).

The adhesion factor is an empirical factor used to

$$q_s = \alpha S_u \quad (10.8.3.5.1b\text{-}1)$$

in which:

$$\alpha = 0.55 \text{ for } \frac{S_u}{p_a} \le 1.5 \quad (10.8.3.5.1b\text{-}2)$$

$$\alpha = 0.55 - 0.1\left(S_u / p_a - 1.5\right)$$
$$\text{for } 1.5 \le S_u / p_a \le 2.5 \quad (10.8.3.5.1b\text{-}3)$$

where:

S_u = undrained shear strength (ksf)

α = adhesion factor (dim)

p_a = atmospheric pressure (= 2.12 ksf)

The following portions of a drilled shaft, illustrated in Figure 10.8.3.5.1b-1, should not be taken to contribute to the development of resistance through skin friction:

- At least the top 5.0 ft of any shaft;
- ~~For straight shafts, a bottom length of the shaft taken as the shaft diameter;~~
- Periphery of belled ends, if used; and
- ~~Distance above a belled end taken as equal to the shaft diameter.~~

When permanent casing is used, the side resistance shall be adjusted with consideration to the type and length of casing to be used, and how it is installed.

Values of α for contributing portions of shafts excavated dry in open or cased holes should be as specified in Eqs. 10.8.3.5.1b-2 and 10.8.3.5.1b-3.

correlate the results of full-scale load tests with the material property or characteristic of the cohesive soil. The adhesion factor is usually related to S_u and is derived from the results of full-scale pile and drilled shaft load tests. Use of this approach presumes that the measured value of S_u is correct and that all shaft behavior resulting from construction and loading can be lumped into a single parameter. Neither presumption is strictly correct, but the approach is used due to its simplicity.

Steel casing will generally reduce the side resistance of a shaft. No specific data is available regarding the reduction in skin friction resulting from the use of permanent casing relative to concrete placed directly against the soil. Side resistance reduction factors for driven steel piles relative to concrete piles can vary from 50 to 75 percent, depending on whether the steel is clean or rusty, respectively (Potyondy, 1961). Greater reduction in the side resistance may be needed if oversized cutting shoes or splicing rings are used.

If open-ended pipe piles are driven full depth with an impact hammer before soil inside the pile is removed, and left as a permanent casing, driven pile static analysis methods may be used to estimate the side resistance as described in Article 10.7.3.8.6.

The upper 5.0 ft of the shaft is ignored in estimating R_n, to account for the effects of seasonal moisture changes, disturbance during construction, cyclic lateral loading, and low lateral stresses from freshly placed concrete. ~~The lower 1.0-diameter length above the shaft tip or top of enlarged base is ignored due to the development of tensile cracks in the soil near these regions of the shaft and a corresponding reduction in lateral stress and side resistance.~~

Bells or underreams constructed in stiff fissured clay often settle sufficiently to result in the formation of a gap above the bell that will eventually be filled by slumping soil. Slumping will tend to loosen the soil immediately above the bell and decrease the side resistance along the lower portion of the shaft.

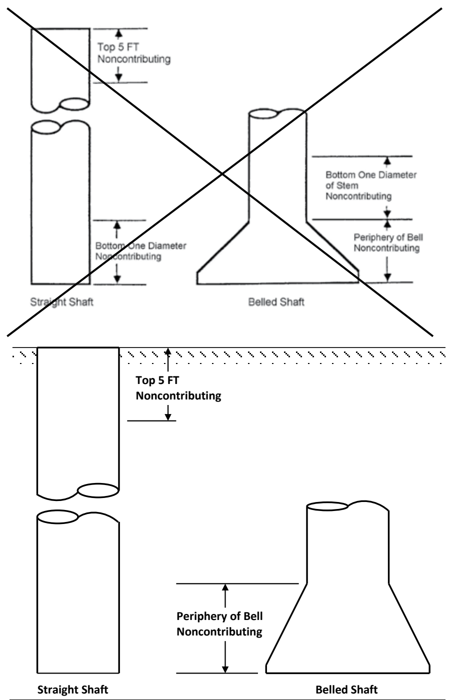


Figure 10.8.3.5.1b-1—Explanation of Portions of Drilled Shafts Not Considered in Computing Side Resistance (~~O'Neill and Reese, 1999~~ Brown et al., 2010)

The value of α is often considered to vary as a function of S_u. Values of α for drilled shafts are recommended as shown in Eqs. 10.8.3.5.1b-2 and 10.8.3.5.1b-3, based on the results of back-analyzed, full-scale load tests. ~~This recommendation is based on eliminating the upper 5.0 ft and lower 1.0 diameter of the shaft length during back-analysis of load test results.~~ The load tests were conducted in insensitive cohesive soils. Therefore, if shafts are constructed in sensitive clays, values of α may be different than those obtained from Eqs. 10.8.3.5.1b-2 and 10.8.3.5.1b-3. Other values of α may be used if based on the results of load tests.

The depth of 5.0 ft at the top of the shaft may need to be increased if the drilled shaft is installed in expansive clay, if scour deeper than 5.0 ft is anticipated, if there is substantial groundline deflection from lateral loading, or if there are other long-term loads or construction factors that could affect shaft resistance.

~~A reduction in the effective length of the shaft contributing to side resistance has been attributed to horizontal stress relief in the region of the shaft tip, arising from development of outward radial stresses at the toe during mobilization of tip resistance. The influence of this effect may extend for a distance of 1*B* above the tip (O'Neill and Reese, 1999). The effectiveness of enlarged bases is limited when *L*/*D* is greater than 25.0 due to the lack of load transfer to the tip of the shaft.~~

The values of α obtained from Eqs. 10.8.3.5.1b-2 and 10.8.3.5.1b-3 are considered applicable for both compression and uplift loading.

10.8.3.5.1c—Tip Resistance

For axially loaded shafts in cohesive soil, the nominal unit tip resistance, q_p, by the total stress method as provided in ~~O'Neill and Reese (1999)~~ Brown et al. (2010) shall be taken as:

$$q_p = N_c S_u \leq 80.0 \text{ ksf} \quad (10.8.3.5.1c\text{-}1)$$

in which:

$$N_c = 6\left[1 + 0.2\left(\frac{Z}{D}\right)\right] \leq 9 \quad (10.8.3.5.1c\text{-}2)$$

where:

D = diameter of drilled shaft (ft)

Z = penetration of shaft (ft)

C10.8.3.5.1c

These equations are for total stress analysis. For effective stress methods for shafts in clay, see ~~O'Neill and Reese (1999)~~ Brown et al. (2010).

The limiting value of 80.0 ksf for q_p is not a theoretical limit but a limit based on the largest measured values. A higher limiting value may be used if based on the results of a load test, or previous successful experience in similar soils.

S_u = undrained shear strength (ksf)

The value of S_u should be determined from the results of in-situ and/or laboratory testing of undisturbed samples obtained within a depth of 2.0 diameters below the tip of the shaft. If the soil within 2.0 diameters of the tip has S_u <0.50 ksf, the value of N_c should be multiplied by 0.67.

10.8.3.5.2—Estimation of Drilled Shaft Resistance in Cohesionless Soils

10.8.3.5.2a—General

Shafts in cohesionless soils should be designed by effective stress methods for drained loading conditions or by empirical methods based on in-situ test results.

C10.8.3.5.2a

The factored resistance should be determined in consideration of available experience with similar conditions.

~~Although many field load tests have been performed on drilled shafts in clays, very few have been performed on drilled shafts in sands.~~ The shear strength of cohesionless soils can be characterized by an angle of internal friction, ϕ_f, or empirically related to its *SPT* blow count, *N*. Methods of estimating shaft resistance and end bearing are presented below. Judgment and experience should always be considered.

10.8.3.5.2b—Side Resistance

~~The nominal axial resistance of drilled shafts in cohesionless soils by the β-method shall be taken as~~The side resistance for shafts in cohesionless soils shall be determined using the β method, take as:

~~$q_s = \beta\sigma'_v \leq 4.0$ for $0.25 \leq \beta \leq 1.2$ (10.8.3.5.2b-1)~~

~~in which, for sandy soils:~~

- ~~for $N_{60} \geq 15$:~~

~~$\beta = 1.5 - 0.135\sqrt{z}$ (10.8.3.5.2b-2)~~

- ~~for $N_{60} < 15$:~~

~~$\beta = \frac{N_{60}}{15}(1.5 - 0.135\sqrt{z})$ (10.8.3.5.2b-3)~~

~~where:~~

~~σ'_v = vertical effective stress at soil layer mid-depth (ksf)~~

~~β = load transfer coefficient (dim)~~

~~z = depth below ground, at soil layer mid-depth (ft)~~

C10.8.3.5.2b

~~O'Neill and Reese (1999) provide additional discussion of computation of shaft side resistance and recommend allowing β to increase to 1.8 in gravels and gravelly sands, however, they recommend limiting the unit side resistance to 4.0 ksf in all soils.~~

~~O'Neill and Reese (1999) proposed a method for uncemented soils that uses a different approach in that the shaft resistance is independent of the soil friction angle or the *SPT* blow count. According to their findings, the friction angle approaches a common value due to high shearing strains in the sand caused by stress relief during drilling.~~

35

~~N_{60} = average *SPT* blow count (corrected only for hammer efficiency) in the design zone under consideration (blows/ft)~~

~~Higher values may be used if verified by load tests.~~

~~For gravelly sands and gravels, Eq. 10.8.3.5.2b-4 should be used for computing β where $N_{60} \geq 15$. If $N_{60} < 15$, Eq. 10.8.3.5.2b-3 should be used.~~

~~The detailed development of Eq. 10.8.3.5.2b-4 is provided in O'Neill and Reese (1999).~~

$$\cancel{\beta = 2.0 - 0.06(z)^{0.75}} \quad \text{(10.8.3.5.2b-4)}$$

$$q_s = \beta\,\sigma'_v \quad \text{(10.8.3.5.2b-1)}$$

in which:

$$\beta = \left(1-\sin\varphi'_f\right)\left(\frac{\sigma'_p}{\sigma'_v}\right)^{\sin\varphi'_f}\tan\varphi'_f \quad \text{(10.8.3.5.2b-2)}$$

where:

β = load transfer coefficient (dim)

φ'_f = friction angle of cohesionless soil layer (°)

σ'_p = effective vertical preconsolidation stress

σ'_v = vertical effective stress at soil layer mid-depth

The correlation for effective soil friction angle for use in the above equations shall be taken as:

$$\varphi'_f = 27.5 + 9.2\log\left[\left(N_1\right)_{60}\right] \quad \text{(10.8.3.5.2b-3)}$$

where:

$(N_1)_{60}$ = SPT N-value corrected for effective overburden stress

The preconsolidation stress in Eq. 10.8.3.5.2b-2 should be approximated through correlation to SPT N-values. For sands:

$$\frac{\sigma'_p}{p_a} = 0.47\left(N_{60}\right)^m \quad \text{(10.8.3.5.2b-4)}$$

where:

m = 0.6 for clean quartzitic sands

m = 0.8 for silty sand to sandy silts

p_a = atmospheric pressure (same units as σ'_p, 2.12 ksf or 14.7 psi)

The method described herein is based on axial load tests on drilled shafts as presented by Chen and Kulhawy (2002) and updated by Kulhawy and Chen (2007). This method provides a rational approach for relating unit side resistance to N-values and to the state of effective stress acting at the soil-shaft interface. This approach replaces the previously used depth-dependent β-method developed by O'Neill and Reese (1999), which does not account for variations in N-value or effective stress on the calculated value of β. Further discussion, including the detailed development of Eq. 10.8.3.5.2b-2, is provided in (Brown et al. 2010).

For gravelly soils:

$$\frac{\sigma'_p}{p_a} = 0.15\left(N_{60}\right) \qquad (10.8.3.5.2b\text{-}5)$$

When permanent casing is used, the side resistance shall be adjusted with consideration to the type and length of casing to be used, and how it is installed.

Steel casing will generally reduce the side resistance of a shaft. No specific data is available regarding the reduction in skin friction resulting from the use of permanent casing relative concrete placed directly against the soil. Side resistance reduction factors for driven steel piles relative to concrete piles can vary from 50 to 75 percent, depending on whether the steel is clean or rusty, respectively (Potyondy, 1961). Casing reduction factors of 0.6 to 0.75 are commonly used. Greater reduction in the side resistance may be needed if oversized cutting shoes or splicing rings are used.

If open-ended pipe piles are driven full depth with an impact hammer before soil inside the pile is removed, and left as a permanent casing, driven pile static analysis methods may be used to estimate the side resistance as described in Article 10.7.3.8.6.

10.8.3.5.2c—Tip Resistance

The nominal tip resistance, q_p, in ksf, for drilled shafts in cohesionless soils by the ~~O'Neill and Reese (1999)~~ method described in Brown et al. (2010) shall be taken as:

$$\cancel{\text{for } N_{60} \leq 50,\ q_p = 1.2N_{60}} \qquad (10.8.3.5.2c\text{-}1)$$

$$\text{If } N_{60} \leq 50, \text{ then } q_p = 1.2N_{60}$$

where:

N_{60} = average *SPT* blow count (corrected only for hammer efficiency) in the design zone under consideration (blows/ft)

The value of q_p in Eq. 10.8.3.5.2c-1 should be limited to 60 ksf, unless greater values can be justified though load test data.

~~Cohesionless soils with *SPT-N*60 blow counts greater than 50 shall be treated as intermediate geomaterial (IGM) and the tip resistance, in ksf, taken as:~~

$$\cancel{q_p = 0.59\left[N_{60}\left(\frac{p_a}{\sigma'_v}\right)\right]^{0.8}\sigma'_v} \qquad \cancel{(10.8.3.5.2c\text{-}2)}$$

~~where:~~

~~p_a = atmospheric pressure (= 2.12 ksf)~~

~~σ'_v = vertical effective stress at the tip elevation of~~

C10.8.3.5.2c

~~O'Neill and Reese (1999)~~ Brown et al. (2010) provide additional discussion regarding the computation of nominal tip resistance and on tip resistance in specific geologic environments.

~~See O'Neill and Reese (1999) for background on IGMs.~~

~~the shaft (ksf)~~

~~N_{60} should be limited to 100 in Eq. 10.8.3.5.2c-2 if higher values are measured.~~

10.8.3.5.3—Shafts in Strong Soil Overlying Weaker Compressible Soil - ***NO CHANGES – NOT SHOWN***

10.8.3.5.4—Estimation of Drilled Shaft Resistance in Rock

10.8.3.5.4a—General

Drilled shafts in rock subject to compressive loading shall be designed to support factored loads in:

- Side-wall shear comprising skin friction on the wall of the rock socket; or
- End bearing on the material below the tip of the drilled shaft; or
- A combination of both.

The difference in the deformation required to mobilize skin friction in soil and rock versus what is required to mobilize end bearing shall be considered when estimating axial compressive resistance of shafts embedded in rock. Where end bearing in rock is used as part of the axial compressive resistance in the design, the contribution of skin friction in the rock shall be reduced to account for the loss of skin friction that occurs once the shear deformation along the shaft sides is greater than the peak rock shear deformation, i.e., once the rock shear strength begins to drop to a residual value.

C10.8.3.5.4a

Methods presented in this Article to calculate drilled shaft axial resistance require an estimate of the uniaxial compressive strength of rock core. Unless the rock is massive, the strength of the rock mass is most frequently controlled by the discontinuities, including orientation, length, and roughness, and the behavior of the material that may be present within the discontinuity, e.g., gouge or infilling. The methods presented are semi-empirical and are based on load test data and site-specific correlations between measured resistance and rock core strength.

Design based on side-wall shear alone should be considered for cases in which the base of the drilled hole cannot be cleaned and inspected or where it is determined that large movements of the shaft would be required to mobilize resistance in end bearing.

Design based on end-bearing alone should be considered where sound bedrock underlies low strength overburden materials, including highly weathered rock. In these cases, however, it may still be necessary to socket the shaft into rock to provide lateral stability.

Where the shaft is drilled some depth into sound rock, a combination of sidewall shear and end bearing can be assumed (Kulhawy and Goodman, 1980).

If the rock is degradable, use of special construction procedures, larger socket dimensions, or reduced socket resistance should be considered.

<u>Factors that should be considered when making an engineering judgment to neglect any component of resistance (side or base) are discussed in Article 10.8.3.5.4d. In most cases, both side and base resistances should be included in limit state evaluation of rock-socketed shafts.</u>

For drilled shafts installed in karstic formations, exploratory borings should be advanced at each drilled shaft location to identify potential cavities. Layers of compressible weak rock along the length of a rock socket and within approximately three socket diameters or more below the base of a drilled shaft may reduce the resistance of the shaft.

For rock that is stronger than concrete, the concrete shear strength will control the available side friction, and the strong rock will have a higher stiffness, allowing significant end bearing to be mobilized before the side wall shear strength reaches its peak value. Note that concrete typically reaches its peak shear strength at about 250 to 400 microstrain (for a 10-ft long rock socket, this is approximately 0.5 in. of deformation at the top of the rock socket). If strains or deformations greater than the value at the peak shear stress are anticipated to mobilize the desired end bearing in the rock, a residual value for the skin friction can still be used. Article 10.8.3.5.4d provides procedures for computing a residual value of the skin friction based on the properties of the rock and shaft.

10.8.3.5.4b—Side Resistance

~~For drilled shafts socketed into rock, shaft resistance, in ksf, may be taken as (Horvath and Kenney, 1979):~~

$$\cancel{q_s = 0.65\alpha_E p_a \left(q_u/p_a\right)^{0.5} < 7.8 p_a \left(f'_c/p_a\right)^{0.5}} \quad \text{~~(10.8.3.5.4b-1)~~}$$

~~where:~~

~~q_u = uniaxial compressive strength of rock (ksf)~~

~~p_a = atmospheric pressure (= 2.12 ksf)~~

~~α_E = reduction factor to account for jointing in rock as provided in Table 10.8.3.5.4b-1~~

~~f'_c = concrete compressive strength (ksi)~~

~~Table 10.8.3.5.4b-1—Estimation of α_E (O'Neill and Reese, 1999)~~

~~E_m/E_i~~	~~α_E~~
~~1.0~~	~~1.0~~
~~0.5~~	~~0.8~~
~~0.3~~	~~0.7~~
~~0.1~~	~~0.55~~
~~0.05~~	~~0.45~~

For drilled shafts socketed into rock, unit side resistance, q_S in ksf, shall be taken as (Kulhawy et al., 2005):

$$\frac{q_S}{p_a} = C\sqrt{\frac{q_u}{p_a}} \quad \text{(10.8.3.5.4b-1)}$$

where:

C10.8.3.5.4b

~~Eq. 10.8.3.5.4b-1 applies to the case where the side of the rock socket is considered to be smooth or where the rock is drilled using a drilling slurry. Significant additional shaft resistance may be achieved if the borehole is specified to be artificially roughened by grooving. Methods to account for increased shaft resistance due to borehole roughness are provided in Section 11 of O'Neill and Reese (1999).~~

~~Eq. 10.8.3.5.4b-1 should only be used for intact rock. When the rock is highly jointed, the calculated q_s should be reduced to arrive at a final value for design. The procedure is as follows:~~

~~Step 1. Evaluate the ratio of rock mass modulus to intact rock modulus, i.e., E_m/E_i, using Table C10.4.6.5-1.~~

~~Step 2. Evaluate the reduction factor, α_E, using Table 10.8.3.5.4b-1.~~

~~Step 3. Calculate q_s according to Eq. 10.8.3.5.4b-1.~~

Eq. 10.8.3.5.4b-1 is based on regression analysis of load test data as reported by Kulhawy et al. (2005) and includes data from pervious studies by Horvath and Kenney (1979), Rowe and Armitage (1987), Kulhawy and Phoon (1993), and others. The recommended value of the regression coefficient C = 1.0 is applicable to "normal" rock sockets, defined as sockets constructed with conventional equipment and resulting in nominally clean sidewalls without resorting to special procedures

p_a = atmospheric pressure taken as 2.12 ksf

C = regression coefficient taken as 1.0 for normal conditions

q_u = uniaxial compressive strength of rock (ksf)

If the uniaxial compressive strength of rock forming the sidewall of the socket exceeds the drilled shaft concrete compressive strength, the value of concrete compressive strength (f'_c) shall be substituted for q_u in Eq. 10.8.3.5.4b-1.

For fractured rock that caves and cannot be drilled without some type of artificial support, the unit side resistance shall be taken as:

$$\frac{q_s}{p_a} = 0.65\alpha_E\sqrt{\frac{q_u}{p_a}} \qquad (10.8.3.5.4b\text{-}2)$$

The joint modification factor, α_E is given in Table 10.8.3.5.4b-1 based on RQD and visual inspection of joint surfaces.

Table 10.8.3.5.4b-1—Estimation of α_E (O'Neill and Reese, 1999)

RQD (%)	Joint Modification Factor, α_E	
	Closed joints	Open or gouge-filled jo
100	1.00	0.85
70	0.85	0.55
50	0.60	0.55
30	0.50	0.50
20	0.45	0.45

or artificial roughening. Rock that is prone to smearing or rapid deterioration upon exposure to atmospheric conditions, water, or slurry are outside the "normal" range and may require additional measures to insure reliable side resistance. Rocks exhibiting this type of behavior include clay shales and other argillaceous rocks. Rock that cannot support construction of an unsupported socket without caving is also outside the "normal" and will likely exhibit lower side resistance than given by Eq. 10.8.3.5.4b-1 with C = 1.0. For additional guidance on assessing the magnitude of C, see Brown, et al. (2010).

Shafts are sometimes constructed by supporting the hole with temporary casing or by grouting the rock ahead of the excavation. When using these construction methods, disturbance of the sidewall results in lower unit side resistances. Based on O'Neill and Reese (1999) and as discussed in Brown et al. (2010), the reduction in side resistance can be related empirically to the RQD and joint conditions.

10.8.3.5.4c—Tip Resistance

End-bearing for drilled shafts in rock may be taken as follows:

- If the rock below the base of the drilled shaft to a depth of 2.0*B* is either intact or tightly jointed, i.e., no compressible material or gouge-filled seams, and the depth of the socket is greater than 1.5*B* ~~(O'Neill and Reese, 1999)~~:

C10.8.3.5.4c

If end bearing in the rock is to be relied upon, and wet construction methods are used, bottom clean-out procedures such as airlifts should be specified to ensure removal of loose material before concrete placement.

The use of Eq. 10.8.3.5.4c-1 also requires that there are no solution cavities or voids below the base of the drilled shaft.

$$q_p = 2.5q_u \qquad (10.8.3.5.4c\text{-}1)$$

- If the rock below the base of the shaft to a depth of 2.0B is jointed, the joints have random orientation, and the condition of the joints can be evaluated as:

$$\cancel{q_p = \left[\sqrt{s} + \sqrt{(m\sqrt{s} + s)}\right] q_u} \qquad \cancel{(10.8.3.5.4c\text{-}2)}$$

~~where:~~

~~s, m = fractured rock mass parameters and are specified in Table 10.4.6.4-4~~

~~q_u = unconfined compressive strength of rock (ksf)~~

$$q_p = A + q_u\left[m_b\left(\frac{A}{q_u}\right) + s\right]^a \qquad (10.8.3.5.4c\text{-}2)$$

In which:

$$A = \sigma'_{vb} + q_u\left[m_b\frac{(\sigma'_{v,b})}{q_u} + s\right]^a \qquad (10.8.3.5.4c\text{-}3)$$

where:

σ'_{vb} = vertical effective stress at the socket bearing elevation (tip elevation)

s, a, and m_b = Hoek-Brown strength parameters for the fractured rock mass determined from GSI (see Article 10.4.6.4)

q_u = uniaxial compressive strength of intact rock

Eq. 10.8.3.5.4c-1 should be used as an upper-bound limit to base resistance calculated by Eq. 10.8.2.5.4c-2, unless local experience or load tests can be used to validate higher values.

For further information see ~~O'Neill and Reese (1999)~~Brown et al. (2010).

~~Eq. 10.8.3.5.4c-2 is a lower bound solution for bearing resistance for a drilled shaft bearing on or socketed in a fractured rock mass. This method is appropriate for rock with joints that are not necessarily oriented preferentially and the joints may be open, closed, or filled with weathered material. Load testing will likely indicate higher tip resistance than that calculated using Eq. 10.8.3.5.4c-2. Resistance factors for this method have not been developed and must therefore be estimated by the designer.~~ Bearing capacity theory provides a framework for evaluation of base resistance for cases where the bearing rock can be characterized by its GSI. Eq. 10.8.3.5.4c-2 (Turner and Ramey, 2010) is a lower bound solution for bearing resistance of a drilled shaft bearing on or socketed into a fractured rock mass. Fractured rock describes a rock mass intersected by multiple sets of intersecting joints such that the strength is controlled by the overall mass response and not by failure along pre-existing structural discontinuities. This generally applies to rock that can be characterized by the descriptive terms shown in Figure 10.4.6.4-1 (*e.g.*, "blocky", "disintegrated", etc.).

10.8.3.5.4d—Combined Side and Tip Resistance

Design methods that consider the difference in shaft movement required to mobilize skin friction in rock versus what is required to mobilize end bearing~~, such as the methodology provided by O'Neill and Reese (1999),~~ shall be used to estimate axial compressive resistance of shafts embedded in rock.

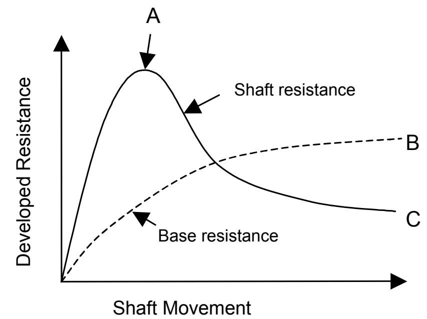
C10.8.3.5.4d

~~Typically, the axial compression load on a shaft socketed into rock is carried solely in shaft side resistance until a total shaft movement on the order of 0.4 in. occurs.~~

~~Designs which consider combined effects of side friction and end-bearing of a drilled shaft in rock require that side friction resistance and end bearing resistance be evaluated at a common value of axial displacement, since maximum values of side friction~~

~~and end-bearing are not generally mobilized at the same displacement.~~

~~Where combined side friction and end-bearing in rock is considered, the designer needs to evaluate whether a significant reduction in side resistance will occur after the peak side resistance is mobilized. As indicated in Figure C10.8.3.5.4d-1, when the rock is brittle in shear, much shaft resistance will be lost as vertical movement increases to the value required to develop the full value of q_p. If the rock is ductile in shear, i.e., deflection softening does not occur, then the side resistance and end-bearing resistance can be added together directly. If the rock is brittle, however, adding them directly may be unconservative. Load testing or laboratory shear strength testing, e.g., direct shear testing, may be used to evaluate whether the rock is brittle or ductile in shear.~~



~~**Figure C10.8.3.5.4d-1—Deflection Softening Behavior of Drilled Shafts under Compression Loading (after O'Neill and Reese, 1999).**~~

~~The method used to evaluate combined side friction and end-bearing at the strength limit state requires the construction of a load-vertical deformation curve. To accomplish this, calculate the total load acting at the head of the drilled shaft, Q_{T1}, and vertical movement, w_{T1}, when the nominal shaft side resistance (Point *A* on Figure C10.8.3.5.4d-1) is mobilized. At this point, some end bearing is also mobilized. For detailed computational procedures for estimating shaft resistance in rock, considering the combination of side and tip resistance, see O'Neill and Reese (1999).~~

A design decision to be addressed when using rock sockets is whether to neglect one or the other component of resistance (side or base). For example, design based on side resistance alone is sometimes assumed for cases in which the base of the drilled hole cannot be cleaned and inspected or where it is determined that large downward movement of the shaft would be required to mobilize tip resistance. However, before making a decision to omit tip resistance, careful consideration should be given to applying available methods of quality construction and inspection that can provide confidence in tip resistance. Quality construction practices can result in adequate clean-out at the base of rock sockets, including those constructed by wet methods. In many cases, the cost of quality control and assurance is offset by the economies achieved in socket design by including tip resistance. Load testing provides a means to verify tip resistance in rock.

Reasons cited for neglecting side resistance of rock sockets include (1) the possibility of strain-softening behavior of the sidewall interface (2) the possibility of degradation of material at the borehole wall in argillaceous rocks, and (3) uncertainty regarding the roughness of the sidewall. Brittle behavior along the sidewall, in which side resistance exhibits a significant decrease beyond its peak value, is not commonly observed in load tests on rock sockets. If there is reason to believe strain softening will occur, laboratory direct shear tests of the rock-concrete interface can be used to evaluate the load-deformation behavior and account for it in design. These cases would also be strong candidates for conducting field load tests. Investigating the sidewall shear behavior through laboratory or field testing is generally more cost-effective than neglecting side resistance in the design. Application of quality control and assurance through inspection is also necessary to confirm that sidewall conditions in production shafts are of the same quality as laboratory or field test conditions.

Materials that are prone to degradation at the exposed surface of the borehole and are prone to a "smooth" sidewall generally are argillaceous sedimentary rocks such as shale, claystone, and siltstone. Degradation occurs due to expansion, opening of cracks and fissures combined with groundwater seepage, and by exposure to air and/or water used for drilling. Hassan and O'Neill (1997) note that this behavior is most prevalent in cohesive IGM's and that in the most severe cases degradation results in a smear zone at the interface. Smearing may reduce load transfer significantly. As reported by Abu-Hejleh et al. (2003), both smearing and smooth sidewall conditions can be prevented in cohesive IGM's by using roughening tools during the final pass with the rock auger or by grooving tools. Careful inspection prior to concrete placement is required to confirm roughness of the sidewalls. Only when these measures cannot be confirmed would there be cause for neglecting side resistance in design.

Analytical tools for evaluating the load transfer behavior of rock socketed shafts are given in Turner (2006) and Brown et al. (2010).

10.8.3.5.5—Estimation of Drilled Shaft Resistance in Intermediate Geomaterials (IGMs)

For detailed base and side resistance estimation procedures for shafts in cohesive IGMs, the procedures

C10.8.3.5.5

See Article 10.8.2.2.3 for a definition of an IGM. ~~For convenience, since a common situation is to tip~~

provided by ~~O'Neill and Reese (1999)~~ Brown et al. (2010) should be used.

~~the shaft in a cohesionless IGM, the equation for tip resistance in a cohesionless IGM is provided in Article C10.8.3.5.2c.~~

10.8.3.5.6—Shaft Load Test - ***NO CHANGES – NOT SHOWN***

10.8.3.6—Shaft Group Resistance - ***NO CHANGES – NOT SHOWN***

10.8.3.7—Uplift Resistance - ***NO CHANGES – NOT SHOWN***

10.8.3.8—Nominal Horizontal Resistance of Shaft and Shaft Groups - ***NO CHANGES – NOT SHOWN***

10.8.3.9—Shaft Structural Resistance - ***NO CHANGES – NOT SHOWN***

10.8.4—Extreme Event Limit State

The provisions of Article 10.5.5.3 and 10.7.4 shall apply.

C10.8.4

See commentary to Articles 10.5.5.3 and 10.7.4.

10.9—MICROPILES – ***NO CHANGES – NOT SHOWN***

10.10—REFERENCES

AASHTO. 1988. *Manual on Subsurface Investigations*, First Edition, MSI-1. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2002. *Standard Specifications for Highway Bridges*, 17th Edition, HB-17. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2011. *AASHTO Guide Specifications for LRFD Seismic Design*, Second Edition, LRFDSEIS-2. American Association of State Highway and Transportation Officials, Washington, DC.

Allen, T. M. 2005. *Development of Geotechnical Resistance Factors and Downdrag Load Factors for LRFD Foundation Strength Limit State Design*, FHWA-NHI-05-052, Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

Abu-Hejleh, N, M.W. O'Neill, D. Hannerman, and W.J. Atwoll. 2003. *Improvement of the Geotechnical Axial Design Methodology for Colorado's Drilled Shafts Socketed in Weak Rock*. Report No. CDOT-DTD-R-2003-6, Colorado Department of Transportation, Denver, 192 p.

Allen, T. M., 2005, *Development of Geotechnical Resistance Factors and Downdrag Load Factors for LRFD Foundation Strength Limit State Design*, Publication No. FHWA-NHI-05-052, Federal Highway Administration, Washington, DC, 41 pp.

Allen, T. M., 2007, "Development of New Pile-Driving Formula and Its Calibration for Load and Resistance Factor Design," *Transportation Research Record: Journal of the Transportation Research Board*, No. 2004, Transportation Research Board of the National Academies, Washington, D.C., pp. 20–27.

Arman, A., N. Samtani, R. Castelli, and G. Munfakh. 1997. *Subsurface Investigations: Training Course in Geotechnical and Foundation Engineering*, FHWA-HI-97-021. Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

Ashour, M., G. M. Norris, and P. Pilling. 1998, "Lateral Loading of a Pile in Layered Soil Using the Strain Wedge Model," *ASCE Journal of Geotechnical and Geoenvironmental Engineering*. American Society of Civil Engineers, Reston, VA, Vol. 124, No. 4, pp. 303–315.

ASTM D 5731-07, *Standard Test Method for Determination of the Point Load Strength Index of Rock and Application to Rock Strength Classifications*.

ASTM D6429, *Standard Guide for Selecting Surface Geophysical Methods*

Baguelin, F., J. F. Jezequel, and D. H. Shields. 1987. *The Pressuremeter and Foundation Engineering*. Trans Tech Publications, Clausthal-Zellerfeld, Germany, p. 617.

Barkdale, R. D., and R. C. Bachus. 1983. *Design and Construction of Stone Columns—Vol. 1*, FHWA/RD-83/02C. Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

Barker, R. M., J. M. Duncan, K. B. Rojiani, P. S. K. Ooi, C. K. Tan, and S. G. Kim. 1991. *Manuals for the Design of Bridge Foundations*. *NCHRP Report 343*. Transportation Research Board, National Research Council, Washington, DC.

Barton, N. R. 1976. "The Shear Strength of Rock and Rock Joints," *Engineering Geology*. Elsevier Science Publishing Company, Inc., New York, NY, Vol. 7, pp. 287–332.

Bieniawski, Z. T. 1984. *Rock Mechanics Design in Mining and Tunneling*. A. A. Balkema, Rotterdam/Boston, p. 272.

Bogard, J. D., and H. Matlock. 1990. "Application of Model Pile Tests to Axial Pile Design." In *Proc., 22nd Annual Offshore Technology Conference*, Houston, TX, Vol. 3, pp. 271–278.

Boulanger, R. W., and I. M. Idriss. 2006. "Liquefaction Susceptibility Criteria for Silts and Clays," *Journal of Geotechnical and Geoenvironmental Engineering*. American Society of Civil Engineers, Reston, VA, Vol. 132, No. 11, pp. 1413–1426.

Bowles, J. E. 1977. *Foundation Analysis and Design*, Second Edition. McGraw–Hill Book Company, New York, NY.

Bowles, J. E. 1988. *Foundation Analysis and Design*, 4th Edition. McGraw–Hill Book Company, New York, NY, p. 1004.

Bray, J. D., and R. B. Sancio. 2006. "Assessment of the Liquefaction Susceptibility of Fine-Grained Soils," *Journal of Geotechnical and Geoenvironmental Engineering*. American Society of Civil Engineers, Reston, VA, Vol. 132, No. 9, pp. 1165–1176.

Bray, J. D., R.B. Sancio, M. F. Riemer, and T. Durgunoglu. 2004. "Liquefaction Susceptibility of Fine-Grained Soils." In *Proc., 11th International Conference on Soil Dynamics and Earthquake Engineering and 3rd International Conference On Earthquake Geotechnical Engineering*, Berkeley, CA, January 2004, Doolin, Kammerer, Nogami, Seed and Towhata, Eds., Vol. 1, pp. 655–662.

Broms, B. B. 1964a. "Lateral Resistance of Piles in Cohesive Soil," *ASCE Journal for Soil Mechanics and Foundation Engineering*. American Society of Civil Engineers, Reston, VA, Vol. 90, SM2, pp. 27–63.

Broms, B.B. 1964b. "Lateral Resistance of Piles in Cohesionless Soil," *ASCE Journal for Soil Mechanics and Foundation Engineering*. American Society of Civil Engineers, Reston, VA, Vol. 90, SM3, pp. 123–156.
Campanella, R. G. 1994. "Field Methods for Dynamic Geotechnical Testing: An Overview of Capabilities and Needs." In *Dynamic Geotechnical Testing II,* Special Technical Publication No. 1213, American Society for Testing and Materials, Philadelphia, PA, pp. 3–23.

Brown, D.A., J.P. Turner, and R.J. Castelli, 2010 *Drilled Shafts: Construction Procedures and LRFD Design Methods – Geotechnical Engineering Circular No. 10*, FHWA NHI-10-016. National Highway Institute, Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

Canadian Geotechnical Society. 1985. *Canadian Foundation Engineering Manual*, Second Edition. Bitech Publishers, Ltd., Vancouver, British Columbia, Canada, p. 460.

Carter, J. P., and F. H. Kulhawy. 1988. *Analysis and Design of Foundations Socketed into Rock*, Report No. EL-5918. Empire State Electric Engineering Research Corporation and Electric Power Research Institute, New York, NY, p. 158.

Cetin, K. O., R. B. Seed, A. Der Kiureghian, K. Tokimatsu, L. F, Harder, R. E. Kayen, and R. E. S. Moss. 2004. "Standard Penetration Test-Based Probabilistic and Deterministic Assessment of Seismic Soil Liquefaction Potential," *Journal of Geotechnical and Geoenvironmental Engineering*. American Society of Civil Engineers, Reston, VA, Vol. 130, No. 12, pp. 1314–1340.

Chen, Y-J, and F.H. Kulhawy, 2002. "Evaluation of Drained Axial Capacity for Drilled Shafts," *Geotechnical Special Publication No. 116, Deep Foundations 2002*. ASCE, Reston, VA, pp. 1200-1214.

Cheney, R. and R. Chassie. 2000. *Soils and Foundations Workshop Reference Manual*, NHI-00-045. National Highway Institute, Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

Davisson, M. T. 1972. "High Capacity Piles." *Proceedings Lecture Series, Innovations in Foundation Construction*, Illinois Section, American Society of Civil Engineers, Reston, VA.

Davisson, M. T., and K. E. Robinson. 1965. "Bending and Buckling of Partially Embedded Piles." In *Proc., Sixth International Conference S. M. and F. E.* University of Toronto Press, Montreal, Canada, pp. 243–246.

DiMillio, A. F. 1982. *Performance of Highway Bridge Abutments Supported by Spread Footings on Compacted Fill*, FHWA/RD-81/184 (NTIS PB83-201822). FHWA Staff Study.

Duncan, J. M., and A. L. Buchignani. 1976. *An Engineering Manual for Settlement Studies*. Department of Civil Engineering, University of California at Berkley, Berkley, CA, p. 94. Geotechnical engineering report.

Duncan, J. M. 2000. "Factors of Safety and Reliability in Geotechnical Engineering," *Journal of Geotechnical and Geoenvironmental Engineering*. American Society of Civil Engineers, Reston, VA, Vol. 126, No. 4, pp. 307–316.

Elias, V. 1990. Durability*/Corrosion of Soil Reinforced Structures*, FHWA/R-89/186. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, p. 173.

Elias, V., J. Welsh, J. Warren, and R. Lukas. 2000. *Ground Improvement Technical Summaries—Vols. 1 and 2*, Demonstration Project 116, FHWA-SA-98-086. Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

EPRI. 1983. *Transmission Line Structure Foundations for Uplift-Compression Loading*, EL-2870. Electric Power Research Institute, Palo Alto, CA.

Esrig, M. E., and R. C. Kirby. 1979. "Advances in General Effective Stress Method for the Prediction of Axial Capacity for Driven Piles in Clay." In *Proc., 11th Annual Offshore Technology Conference*, pp. 437–449.

Fang, H. Y. 1991. *Foundation Engineering Handbook*, Second Edition. Van Nostrand Reinhold, New York, NY.

Gifford, D. G., J. R. Kraemer, J. R. Wheeler, and A. F. McKown. 1987. Spread *Footings for Highway Bridges,* FHWA/RD-86/185. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, p. 229.

Goodman, R. E. 1989. "Introduction." In *Rock Mechanics*, Second Edition, John Wiley and Sons, Inc., New York, NY, p. 562.

Hannigan, P. J., G. G. Goble, G. Thendean, G. E. Likins, and F. Rausche. 2006. *Design and Construction of Driven Pile Foundations*, FHWA-NHI-05-042 and NHI-05-043, Federal Highway Administration, U.S. Department of Transportation, Washington, D.C., Vols. I and II.

Hansen, B. J. 1970. "A Revised and Extended Formula for Bearing Capacity," *Bulletin.* Danish Geotechnical Institute, Copenhagen, Vol. 28.

Hara, A., T. Ohta, M. Niwa, S. Tanaka, and T. Banno. 1974. "Shear Modulus and Shear Strength of Cohesive Soils," *Soils and Foundations*. Vol. 14, No. 3, pp. 1–12.

Hassan, K.M. and M.W. O'Neill. 1997. "Side Load-Transfer Mechanisms in Drilled Shafts in soft Argillaceous Rock," *Journal of Geotechnical And Geoenvironmental Engineering,* Vol. 123, No. 2, ASCE, pp.272-280.

~~Hoek, E. 1983. "Strength of Jointed Rock Masses," *Geotechnique*. Thomas Telford Ltd., London, Vol. 33, No. 3, pp. 187–223.~~

~~Hoek, E., and E. T. Brown. 1988. "The Hoek-Brown Failure Criterion—A 1988 Update." In *Proc., 15th Canadian Rock Mechanics Symposium*, Toronto, Canada.~~

Hoek, E., and E. T. Brown. 1997. "Practical Estimates of Rock Mass Strength," *International Journal of Rock Mechanics and Mining Sciences.* Available at http://www.rocscience.com/.

Hoek, E., P.K.Kaiser, and W.F. Bawden. 1995. *Support of Underground Excavations in Hard Rock*. AA Balkema, Rotterdam, 215 p.

Hoek E, P. Marinos, and M. Benissi. 1998. "Applicability of the Geological Strength Index (GSI) Classification for Weak and Sheared Rock Masses—The Case of the Athens Schist Formation." *Bulletin of Engineering Geology and the Environment*, Vol. 57, pp. 151–160.

Hoek, E. and P. Marinos. 2000. Prediciting tunnel squeezing problems in weak heterogeneous rock masses, *Tunnels and Tunneling International*, pp. 1-21.

Hoek, E., C. Carranza-Torres, and B. Corkum. 2002. "Hoek-Brown Failure Criterion – 2002 Edition," In *Proc., North American Rock Mechanics Society Meeting*. Toronto, Canada

Hoek, E., P. Marinos and V. Marinos. 2005. "Characterization and Engineering Properties of Tectonically Undisturbed but Lithologically Varied Sedimentary Rock Masses." *International Journal of Rock Mechanics and Mining Sciences*, Vol. 42, pp. 277-285.

Holtz, R. D., and W. D. Kovacs. 1981. *An Introduction to Geotechnical Engineering*. Prentice–Hall, Englewood Cliffs, NJ.

Horvath, R. G., and T. C. Kenney. 1979. "Shaft Resistance of Rock Socketed Drilled Piers." In *Proc., Symposium on Deep Foundations*. American Society of Civil Engineers, Atlanta, GA, pp. 182–214.

Hough, B. K. 1959. "Compressibility as the Basis for Soil Bearing Value," *Journal of the Soil Mechanics and Foundations Division*. American Society of Civil Engineers, Reston, VA, Vol. 85, Part 2.

Jaeger, J. C., and N. G. W. Cook. 1976. *Fundamentals of Rock Mechanics*. Chapman & Hall, London.

Janbu, N. 1963. "Soil Compressibility as Determined By Oedometer and Triaxial Tests." In *Proc, 3rd European Conference of Soil Mechanics and Foundation Engineering*. Wiesbaden, Germany, Vol. 1.

Janbu, N. 1967. "Settlement Calculations Based on Tangent Modulus Concept," *Bulletin No. 2, Soil Mechanics and Foundation Engineering Series*. The Technical University of Norway, Trondheim, Norway, p. 57.

Kavazanjian, E., Jr., T. Hadj-Hamou Matasoviæ, and P. J. Sabatini. 1997. *Geotechnical Engineering Circular No. 3, Design Guidance: Geotechnical Earthquake Engineering for Highways*, FHWA-SA-97-076, Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

Kimmerling, R. E. 2002. *Geotechnical Engineering Circular 6, Shallow Foundations*, FHWA-SA-02-054, Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

Komurka, V. E., A. B. Wagner, and T. B. Edil. 2003. *Estimating Soil/Pile Set-Up*, Wisconsin Highway Research Program #0092-00-14, Wisconsin Department of Transportation, Madison, WI.

Kramer, S. L. 1996. *Geotechnical Earthquake Engineering*, Prentice–Hall, Upper Saddle River, NJ.

Kulhawy, F. H. 1978. "Geomechanical Model for Rock Foundation Settlement," *Journal of the Geotechnical Engineering Division*. American Society of Civil Engineers, New York, NY, Vol. 104, No. GT2, pp. 211–227.

Kulhawy, F. H., and R. E. Goodman. 1980. "Design of Foundations on Discontinuous Rock." In *Proc., International Conference on Structural Foundations on Rock*, International Society for Rock Mechanics, Vol. I, pp. 209–220.

Kulhawy, F. H., and R. E. Goodman. 1987. "Foundations in Rock." In *Ground Engineering Reference Manual*, F. G. Bell, ed. Butterworths Publishing Co., London, Chapter 55.

Kulhawy, F.H. and K.K. Phoon. 1993. "Drilled Shaft Side Resistance in Clay Soil to Rock," *Geotechnical Special Publication No. 38: Design and Performance of Deep Foundations*, ASCE, New York, pp. 172-183.

Kulhawy, F.H., W.A. Prakoso, and S.O. Akbas. 2005. "Evaluation of Capacity of Rock Foundation Sockets," *Alaska Rocks 2005, Proceedings, 40th U.S. Symposium on Rock Mechanics,* American Rock Mechanics Association, Anchorage, AK, 8 p.

Kulhawy, F.H. and Y-R Chen. 2007. "Discussion of 'Drilled Shaft Side Resistance in Gravelly Soils' by Kyle M. Rollins, Robert J. Clayton, Rodney C. Mikesell, and Bradford C. Blaise," *Journal of Geotechnical and Geoenvironmental Engineering,* ASCE, Vol. 133, No. 10, pp. 1325-1328.

Kyfor, Z. G., A. R. Schnore, T. A. Carlo, and P. F. Bailey. 1992. *Static Testing of Deep Foundations*, FHWA-SA-91-042, Federal Highway Administration, Office of Technology Applications, U. S. Department of Transportation, Washington D. C., p. 174.

Lam, I. P, and G. R. Martin. 1986. "Design Procedures and Guidelines." In *Seismic Design of Highway Bridge Foundations*, FHWA/RD-86/102, Federal Highway Administration, U.S. Department of Transportation, Washington, DC, Vol. 2, p. 18.

Lambe, T.W. and R. V. Whitman. 1969. *Soil Mechanics.* New York, John Wiley and Sons, Inc.

Liang, R, K Yang and J Nusairat. 2009. "P-y Criterion for Rock Mass" *Journal of Geotechnical and Geoenvironmental Engineering,* ASCE, Vol 135, No. 1, pp. 26-36.

Lukas, R. G. 1995. *Geotechnical Engineering Circular No. 1—Dynamic Compaction*, FHWA-SA-95-037. Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

Marinos, P. and E. Hoek. 2000. "GSI: A Geologically Friendly Tool for Rock Mass Strength Estimation," In *Proc., Geo-Engineering 2000, International Conference on Geotechncial and Geological Engineering.* Melbourne, Austrailia, pp. 1422-1440

Marinos, V., P. Marinos and E. Hoek. 2005. The Geological Strength index: applications and limitations, *Bulletin of Engineering Geology and the Environment*, Vol. 64, pp. 55-65.

Mayne, P. W., B. R.Christopher, and J. DeJong. 2001. *Manual on Subsurface Investigations*, FHWA NHI-01-031. Federal Highway Administration, U.S. Department of Transportation, Washington, D.C.

Meyerhof, G. G. 1953. "The Bearing Capacity of Foundations under Eccentric and Inclined Loads." In *Proc., 3rd International Conference on Soil Mechanics and Foundation Engineering*, Switzerland, pp. 440–445.

Meyerhof, G. G. 1956. "Penetration Tests and Bearing Capacity of Cohesionless Soils," *Journal of the Soil Mechanics and Foundation Division*. American Society of Civil Engineers, Reston, VA, Vol. 82, No. SM1, pp. 866-1–866-19.

Meyerhof, G. G. 1957. "The Ultimate Bearing Capacity of Foundations on Slopes." In *Proc., 4th International Conference on Soil Mechanics and Foundation Engineering*, London.

Meyerhof, G.G. 1976. "Bearing Capacity and Settlement of Pile Foundations," *Journal of the Geotechnical Engineering Division*. American Society of Civil Engineers, New York, NY, Vol. 102, No. GT3, pp. 196–228.

Morgano, C. M. and B. White. 2004. "Identifying Soil Relaxation from Dynamic Testing." In *Proc., 7th International Conference on the Application of Stresswave Theory to Piles 2004*, Petaling Jaya, Selangor, Malaysia.

Moulton, L. K., H. V. S. GangaRao, and G. T. Halverson. 1985. *Tolerable Movement Criteria for Highway Bridges*, FHWA/RD-85/107. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, p. 118.

Moss, R. E. S., R. B. Seed, R. E. Kayen, J. P. Stewart, A. Der Kiureghian, and K. O. Cetin. 2006. "CPT-Based Probabilistic and Deterministic Assessment of In Situ Seismic Soil Liquefaction Potential," *Journal of Geotechnical and Geoenvironmental Engineering*. American Society of Civil Engineers, Reston, VA, Vol. 132, No. 8, pp. 1032–1051.

Munfakh, G., A. Arman, J. G. Collin, J. C.-J. Hung, and R. P. Brouillette. 2001. *Shallow Foundations Reference Manual*, FHWA-NHI-01-023. Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

Nordlund, R. L. 1979. *Point Bearing and Shaft Friction of Piles in Sand*. Missouri-Rolla 5th Annual Short Course on the Fundamentals of Deep Foundation Design.

Norris, G. M. 1986. "Theoretically Based BEF Laterally Loaded Pile Analysis." In *Proc., 3rd International Conference on Numerical Methods in Offshore Piling*, Nantes, France, pp. 361–386.

Nottingham, L., and J. Schmertmann. 1975. *An Investigation of Pile Capacity Design Procedures*. Final Report D629 to Florida Department of Transportation from Department of Civil Engineering, University of Florida, Gainesville, FL, p. 159.

Olson, S. M., and T. D. Stark. 2002. "Liquefied Strength Ratio from Liquefaction Flow Failure Case Histories," *Canadian Geotechnical Journal*. NRC Research Press, Ottawa, ON, Canada, Vol. 39, pp 629–647.

O'Neill, M. W. and L. C. Reese. 1999. *Drilled Shafts: Construction Procedures and Design Methods*, FHWA-IF-99-025, Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

Paikowsky, S. G., C. Kuo, G. Baecher, B. Ayyub, K. Stenersen, K. O'Malley, L. Chernauskas, and M. O'Neill. 2004. *Load and Resistance Factor Design (LRFD) for Deep Foundations*, NCHRP Report 507. Transportation Research Board, National Research Council, Washington, DC.

Peck, R. B., W. E. Hanson, and T. H. Thornburn. 1974. *Foundation Engineering*, Second Edition. John Wiley and Sons, Inc., New York, NY, p. 514.

Potyondy, J. G. 1961. "Skin Friction Between Various Soils and Construction Materials," *Géotechnique*. Vol. 11, No. 4, pp. 339–353.

Poulos, H. G., and E. H. Davis. 1974. *Elastic Solutions for Soil and Rock Mechanics*. John Wiley and Sons, Inc., New York, NY, p. 411.

Prakash, S., and H. D. Sharma. 1990. *Pile Foundations in Engineering Practice*. John Wiley and Sons, Inc., New York, NY, p. 734.

Prandtl, L. 1921. "Über die eindingungsfestigkeit (harte) plasticher baustoffe und die festigkeit von schneiden." *Zeitschrift für Angewandte Mathematik und Mechanik*, Vol. 1, No. 1, pp. 15–20.

Rausche, F., F. Moses, and G. G. Goble. 1972. "Soil Resistance Predictions from Pile Dynamics," *Journal of the Soil Mechanics and Foundation Division*. American Society of Civil Engineers, Reston, VA, Vol. 98, No. SM9, pp. 917–937.

Rausche, F., G. G.Goble, and G. Likins. 1985. "Dynamic Determination of Pile Capacity," *Journal of Geotechnical Engineering*. American Society of Civil Engineers, Reston, VA, Vol. 111, No. 3, pp. 367–383.

Reese, L. C. 1984. *Handbook on Design of Piles and Drilled Shafts Under Lateral Load*, FHWA-IP-84/11. Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

Reese, L. C. 1986. *Behavior of Piles and Pile Groups Under Lateral Load*, FHWA/RD-85/106. Federal Highway Administration, Office of Engineering and Highway Operations Research and Development, U.S. Department of Transportation, Washington DC, p. 311.

Reese, L. C., and M. W. O'Neill. 1988. *Drilled Shafts: Construction Procedures and Design Methods*, FHWA-HI-88-042 or ADSC-TL-4. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, p. 564.

Reissner, H. 1924. "Zum erddruckproblem." In *Proc., 1st Intem. Congress of Applied Mechanics*," Delft, Germany, pp. 295–311.

Richardson, E. V., and S. R. Davis. 2001. *Hydraulic Engineering Circular 18, Evaluating Scour at Bridges*, Fourth Edition, FHWA NHI-01-001. Hydraulic Engineering Circular No. 18, Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

Rowe, R.K. and H.H. Armitage. 1987. "A Design Method for Drilled Piers in Soft Rock," *Canadian Geotechnical Journal,* Vol. 24, pp. 126-142.

Sabatini, P. J, R. C Bachus, P. W. Mayne, J. A. Schneider, and T. E. Zettler. 2002. *Geotechnical Engineering Circular 5 (GEC5)—Evaluation of Soil and Rock Properties*, FHWA-IF-02-034. Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

Seed, R. B., and L. F. Harder. 1990. SPT-Based Analysis of Pore Pressure Generation and Undrained Residual Strength. In *Proc., H. B. Seed Memorial Symposium*, Berkeley, CA, May 1990. BiTech Ltd., Vancouver, BC, Canada, Vol. 2, pp. 351–376.

Skempton, A. W. 1951. "The Bearing Capacity of Clays." In *Proc., Building Research Congress*, Vol. 1, pp. 180–189.

Smith, S. P. 1994. "Preliminary Procedure to Predict Scour in Bedrock."

Sowers, G. F. 1979. *Introductory Soil Mechanics and Foundations: Geotechnical Engineering*. MacMillan Publishing Co., New York, NY, p. 621.

Terzaghi, K., and R. B. Peck. 1967. *Soil Mechanics in Engineering Practice*, Second Edition. John Wiley and Sons, Inc., New York, NY, p. 729.

Terzaghi, K., R. G. Peck, and G. Mesri. 1996. *Soil Mechanics in Engineering Practice*. John Wiley and Sons, Inc., New York, NY, pp. 549.

Tomlinson, M. J. 1980. *Foundation Design and Construction*, Fourth Edition. Pitman Publishing Limited, 128 Long Acre, London WC2E 9AN, UK.

Tomlinson, M. J. 1986. *Foundation Design and Construction*, Fifth Edition. Longman Scientific and Technical, London, p. 842.

Tomlinson, M. J. 1987. *Pile Design and Construction Practice*. Viewpoint Publication, p. 415.

Turner, J.P. 2006. *NCHRP Synthesis 360: Rock-Socketed Shafts for Highway Structure Foundations*, Transportation Research Board, National Research Council, Washington, D.C., 148 p.

Turner, J. P., and S. B. Ramey. 2010. "Base Resistance of Drilled Shafts in Fractured Rock," In *Proc., The Art of Foundation Engineering Practice Congress 2010*. American Society of Civil Engineers, Reston, VA, pp. 687-701.

U.S. Department of the Navy. 1982. *Soil Mechanics. Design Manual 7.1,* NAVFAC DM-7.1. Naval Facilities Engineering Command, Alexandria, VA, p. 348.

Vesic', A. S. 1970. *Research on Bearing Capacity of Soils*.

Vesic', A. S. 1975. "Bearing Capacity of Shallow Foundations." In *Foundation Engineering Handbook*, H. Winterkorn, and H. Y. Fang, eds. Van Nostrand Reinhold Co., New York, NY, Chapter 3, pp. 121–147.

Vesic', A. S. 1973. "Analysis of Ultimate Loads of Shallow Foundations," *Journal of the Soil Mechanics and Foundations Division*. American Society of Civil Engineers, Reston, VA, Vol. 99, No. SM1. pp. 45–73.

Vijayvergiya, V. N., and J. A. Focht, Jr. 1972. "A New Way to Predict the Capacity of Piles in Clay." In *Proc., 4th Annual Offshore Technology Conference*, Vol. 2, pp. 865–874.

Williams, M. E., M. McVay, and M. I. Hoit. 2003. "LRFD Substructure and Foundation Design Programs." In *Proc., 2003 International Bridge Conference*, June 9–11, Pittsburgh, PA.

Winterkorn, H. F., and H. Y. Fang. 1975. *Foundation Engineering Handbook*. Van Nostrand Reinhold Company, New York, NY, p. 751.

Wyllie, D. C. 1999. *Foundations on Rock*. Routledge, New York, NY.

Yazdanbod, A., S. A. Sheikh, and M. W. O'Neill. 1987. "Uplift of Shallow Underream in Jointed Clay." In *Proc., Foundations for Transmission Line Towers*, J. L. Briaud, ed. American Society of Civil Engineers, Reston, VA, pp. 110–127.

Youd, T. L., and I. M. Idriss. 1997. *Proc., NCEER Workshop on Evaluation of Liquefaction Resistance of Soils*, MCEER-97-0022.

Youd, T. L. and B. L. Carter. 2005. "Influence of Soil Softening and Liquefaction on Spectral Acceleration," *Journal of Geotechnical and Geoenvironmental Engineering*. American Society of Civil Engineers, Reston, VA, Vol. 131, No. 7, pp. 811–825.

Youd, et al. 2001. "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance in Soils," *Journal of Geotechnical and Geoenvironmental Engineering*. American Society of Civil Engineers, Reston, VA, Vol. 127, No. 10, pp. 817–833.

Zhang, L. M., W. H. Tang, and C. W. W. Ng. 2001. "Reliability of Axially Loaded Driven Pile Groups," *Journal of Geotechnical and Geoenvironmental Engineering*. American Society of Civil Engineers, Reston, VA, Vol. 127, No. 12, pp. 1051–1060.

51

APPENDIX A10—SEISMIC ANALYSIS AND DESIGN OF FOUNDATIONS – *NO CHANGES – NOT SHOWN*