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Washington State Department of Transportation
Environmental and Regional Operations
Construction Office
Geotechnical Office
1655 S 2nd Avenue
Tumwater, WA 98512-6951

Approved By:Tony Allen

Signature

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Construction Office
Geotechnical Office
1655 S 2nd Avenue
Tumwater, WA 98512-6951
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6.1 Seismic Design Responsibility and Policy

6.1.1 Responsibility of the Geotechnical Designer

The geotechnical designer is responsible for providing geotechnical/seismic input parameters to the structural engineers for their use in structural design of the transportation infrastructure (e.g., bridges, retaining walls, ferry terminals, etc.). Specific elements to be addressed by the geotechnical designer include the design ground motion parameters, site response, geotechnical design parameters, and geologic hazards. The geotechnical designer is also responsible for providing input for evaluation of soil-structure interaction (foundation response to seismic loading), earthquake-induced earth pressures on retaining walls, and an assessment of the impacts of geologic hazards on the structures.

6.1.2 Geotechnical Seismic Design Policies

6.1.2.1 Seismic Performance Objectives

In general, the AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications shall be followed for structure classification of bridges. Critical, essential, and other structures are defined in AASHTO LRFD Bridge Design Specifications. In the current inventory, most structures are considered “other” with a few being “essential” or “critical”. In keeping with the current seismic design approaches employed both nationally and internationally, geotechnical seismic design shall be consistent with the philosophy for structure design that loss of life and serious injury due to structure collapse are minimized. This performance objective shall be achieved at a seismic hazard level that is consistent with the seismic hazard level required in the AASHTO specifications (e.g., 7 percent probability of exceedance in 75 years for other structures, or lower probability of exceedance such as 2 percent in 50 years for critical or essential bridges, as determined by the State Bridge Engineer – see Section 6.3.1. The definition of structure collapse is provided in the LRFD Bridge Design Manual (BDM) M 23-50. Bridges, regardless of their AASHTO classification, may suffer damage and may need to be replaced after a design seismic event, but they are designed for non-collapse due to earthquake shaking and geologic hazards associated with a design seismic event.

In keeping with the no collapse philosophy, bridge approach embankments and fills through which cut-and-cover tunnels are constructed should be designed to remain stable during the design seismic event because of the potential to contribute to collapse of the structure should they fail. The aerial extent of approach embankment (and embankment surrounding cut-and-cover tunnels) seismic design and mitigation (if necessary) should be such that the structure is protected against instability or loading conditions that could result in collapse. The typical distance of evaluation and mitigation is within 100 feet of the abutment or tunnel wall, but the actual distance should be evaluated on a case-by-case basis. Instability or other seismic hazards such as liquefaction, lateral spread, downdrag, and settlement may require mitigation near the abutment or tunnel wall to ensure that the structure is not compromised during a design seismic event. The geotechnical designer should evaluate the potential for differential settlement between mitigated and non mitigated soils. Additional measures may be required to limit differential settlements to tolerable levels both for static and seismic conditions. The bridge interior pier foundations should also be designed to be adequately stable with regard to liquefaction, lateral flow, and other seismic effects to prevent bridge collapse.

All retaining walls and abutment walls shall be evaluated and designed for seismic stability internally and externally (i.e. sliding and overturning) , with the exception of walls that meet the AASHTO LRFD Bridge Design Manual "no seismic Analysis" provisions. With regard to overall seismic slope stability (often referred to as global stability) involving a retaining wall, with or without liquefaction, the geotechnical designer shall evaluate the impacts of failure due to seismic loading, as well as for liquefied conditions during and after shaking, if failure is predicted to occur. If collapse of the wall is likely during the design seismic event (i.e., does not meet minimum slope stability level of safety requirements during seismic loading in accordance with Sections 6.4.2 and 6.4.3), and if that collapse is likely to cause loss of life or severe injury to the traveling public, the stability of the wall shall be improved such that the life safety of the traveling public during the design seismic event is preserved. As a general guide, walls that are less than 10 feet in height, or walls that are well away from the traveled way, are not likely to cause loss of life or severe injury to the traveling public. Therefore, the wall design may allow these lower height walls, or walls that are well away from the traveled way, to deform, translate, or rotate during a seismic event due to inadequate seismic stability. This also applies to reinforced slopes that are steep enough to require a facing such as a geosynthetic wrap or welded wire form, which is generally required for a face slope steeper that 1.2H:1V.

Note that the policy to stabilize retaining walls for overall stability due to design seismic events may not be practical for walls placed on marginally stable landslide areas or otherwise marginally stable slopes. In general, if the placement of a wall within a marginally stable slope (i.e., marginally stable for static conditions) has only a minor effect on the seismic stability of the landslide or slope, or if the wall has a relatively low risk of causing loss of life or severe injury to the traveling public if wall collapse occurs, the requirement of the wall and slope to meet minimum seismic overall stability requirements may be waived, subject to the approval of the State Geotechnical Engineer. The State Geotechnical Engineer will assess the impact and potential risks caused by wall and slope seismic instability or poor performance, and the magnitude of the effect the presence of the wall could have on the stability of the overall slope during the design seismic event. The effect on the corridor in addition to the portion of the corridor being addressed by the project will be considered. In general, if the presence of the wall could decrease the overall slope stability factor of safety by more than 0.05, the requirement to meet minimum seismic overall slope stability requirements will not be waived, but this requirement may be waived by the State Geotechnical Engineer if the existing slope seismic slope stability safety factor is significantly less than 0.9, subject to the evaluation of the impacts described above.

Cut slopes in soil and rock, fill slopes, and embankments should be evaluated for instability due to design seismic events and associated geologic hazards. Instability associated with cuts and fills is usually not mitigated due to the high cost of applying such a design policy uniformly to all slopes statewide. However, slopes that could cause collapse of an adjacent structure (e.g., a bridge, building, or pipeline) if failure due to seismic loading occurs shall be stabilized.

6.1.2.2 Liquefaction Mitigation for Bridge Widenings

The Policy – For the case where an existing bridge is to be widened and liquefiable soil is present, the foundations for the widened portion of the bridge and bridge approaches should be designed to remain stable during the design seismic event such that bridge collapse does not occur. In addition, if the existing bridge foundation is not stable and could cause collapse of the bridge widening, to the extent practical, measures should be taken to prevent collapse of the existing bridge during the design seismic event. The foundations for the widening should be designed in such a way that the seismic response of the bridge widening can be made compatible with the seismic response of the existing bridge as stabilized in terms of foundation deformation and stiffness. If it is not feasible to stabilize the existing bridge such that it will not cause collapse of the bridge widening during the design seismic event, consideration should be given to replacing the existing bridge rather than widening it. Specific design and mitigation requirements to address the instability in the existing bridge to cause collapse of the new bridge widening will be assessed by the WSDOT Bridge and Geotechnical Offices. In accordance with executive departmental policy, the department may choose to defer liquefaction mitigation for the existing bridge, programming the implementation of the liquefaction mitigation of the existing bridge as part of the overall WSDOT seismic retrofit program. See the *Design Manual* M22-01 Chapter 720 for the specific policy regarding this issue.

Scoping for Bridge Widening and Liquefaction Mitigation – Due to the high cost of liquefaction mitigation, it is extremely important that input be received from the Bridge Office and Geotechnical Office when developing the scope of bridge widening projects where liquefiable soils may be present, so that good project delivery decisions can be made. Therefore, the region project manager should contact the Bridge Office for bridge widening and retaining wall scoping assistance before project funding commitments are made to the legislature and the public. The Bridge Office will work with the Geotechnical Office to assess the potential for liquefaction or other seismic hazards that could affect the cost of the proposed structures.

6.1.2.3 Maximum Considered Depth for Liquefaction

When evaluating liquefaction potential and its impacts to transportation facilities, the maximum considered liquefaction depth below the natural ground surface shall be limited to 80 feet. However, for sites that contain exceptionally loose soils that are apparently highly susceptible to liquefaction to greater depths, effective stress analysis techniques may be used to evaluate the potential for deeper liquefaction and the potential impacts of that liquefaction. The reasons for this depth limitation are as follows:

Limits of Simplified Procedures – The simplified procedures most commonly used to assess liquefaction potential are based on historical databases of liquefied sites with shallow liquefaction (i.e., in general, less than 50 feet). Thus, these empirical methodologies have not been calibrated to evaluate deep liquefaction. In addition, the simplified equation used to estimate the earthquake induced cyclic shear stress ratio (CSR) is based on a stress reduction coefficient, r_d, which is highly variable at depth. For example, at shallow depth (15 feet), r_d ranges from about 0.94 to 0.98. As depth increases, r_d becomes more variable ranging, for example, from 0.40 to 0.80 at a depth of 65 feet. The uncertainty regarding the coefficient r_d and lack of verification of the simplified procedures used to predict liquefaction at depth, as well as some of the simplifying assumptions and empiricism within the simplified method with regard to the calculation of liquefaction resistance (i.e., the cyclic resistance ratio CRR), limit the depth at which these simplified procedures should be used. Therefore, simplified empirical methods should not be used to predict liquefaction at depths greater than 50 to 60 feet, and shall not be used at depths of greater than 80 feet.

Lack of Verification and Complexity of More Rigorous Approaches – Several non-linear, effective stress analysis programs have been developed by researchers and can be used to estimate liquefaction potential at depth. However, there has been little field verification of the ability of these programs to predict liquefaction at depth because there are few well documented sites with deep liquefaction. Key is the ability of these approaches to predict pore pressure increase and redistribution in liquefiable soils during and after ground shaking. Calibration of such pore pressure models has so far been limited to comparison to laboratory performance data test results and centrifuge modeling. Furthermore, these more rigorous methods require considerable experience to obtain and apply the input data required, and to confidently interpret the results. Hence, use of such methods requires independent peer review (see Section 6.3 regarding peer review requirements) by expert(s) in the use of such methods for liquefaction analysis.

Decreasing Impact with Depth – Observation and analysis of damage in past earthquakes suggests that the damaging effects of liquefaction generally decrease as the depth of a liquefiable layer increases. This reduction in damage is largely attributed to decreased levels of relative displacement and the need for potential failure surfaces to extend down to the liquefying layer. The effects of a 10 feet thick soil layer liquefying between depths of 80 and 90 feet will generally be much less severe than those of a layer between the depths of 10 and 20 feet. Note that these impacts are focused on the most damaging effects of liquefaction, such as lateral deformation and instability. Deeper liquefaction can, however, increase the magnitude and impact of vertical movement (settlement) and loading (downdrag) on foundations.

Difficulties Mitigating for Deep Liquefaction – The geotechnical engineering profession has limited experience with mitigation of liquefaction hazards at large depths, and virtually no field case histories on which to reliably verify the effectiveness of mitigation techniques for very deep liquefaction mitigation. In practicality, the costs to reliably mitigate liquefaction by either ground improvement or designing the structure to tolerate the impacts of very deep liquefaction are excessive and not cost effective for most structures.

6.1.3 Governing Design Specifications and Additional Resources

The specifications applicable to seismic design of a given project depend upon the type of facility.

For transportation facilities the following manuals, listed in hierarchical order, shall be the primary source of geotechnical seismic design policy for WSDOT:

1. This *Geotechnical Design* Manual (GDM)
2. AASHTO Guide Specifications for LRFD Seismic Bridge Design
3. AASHTO LRFD Bridge Design Specifications

If a publication date is shown, that version shall be used to supplement the geotechnical design policies provided in this WSDOT GDM. If no date is shown, the most current version, including interim publications of the referenced manuals, as of the WSDOT GDM publication date shall be used. This is not a comprehensive list; other publications are referenced in this WSDOT GDM and shall be used where so directed herein.

Until the AASHTO Guide Specifications for LRFD Bridge Seismic Design are fully adopted in the AASHTO LRFD Bridge Design Specifications, the seismic design provisions in the Guide Specifications regarding foundation design, liquefaction assessment, earthquake hazard assessment, and ground response analysis shall be considered to supersede the parallel seismic provisions in the AASHTO LRFD Bridge Design Specifications.

For seismic design of new buildings and non-roadway infrastructure, the 2012 International Building Code (IBC) (International Code Council, 2012), or most current version should be used.

FHWA geotechnical design manuals, or other nationally recognized design manuals, are considered secondary relative to this WSDOT GDM and the AASHTO manuals (and for buildings, the IBC) listed above regarding WSDOT geotechnical seismic design policy, and may be used to supplement the WSDOT GDM, WSDOT BDM, and AASHTO design specifications.

A brief description of these additional references is as follows:

FHWA Geotechnical Engineering Circular No. 3 (Kavazanjian, et al., 2011) – This FHWA document provides design guidance for geotechnical earthquake engineering for highways. Specifically, this document provides guidance on earthquake fundamentals, seismic hazard analysis, ground motion characterization, site characterization, seismic site response analysis, seismic slope stability, liquefaction, and seismic design of foundations and retaining walls. The document also includes design examples for typical geotechnical earthquake engineering analyses.

Geotechnical Earthquake Engineering Textbook – The textbook titled Geotechnical Earthquake Engineering (Kramer, 1996) provides a wealth of information to geotechnical engineers for seismic design. The textbook includes a comprehensive summary of seismic hazards, seismology and earthquakes, strong ground motion, seismic hazard analysis, wave propagation, dynamic soil properties, ground response analysis, design ground motions, liquefaction, seismic slope stability, seismic design of retaining walls, and ground improvement.

In addition, the following website may be accessed to obtain detailed ground motion data that will be needed for design:

United States Geological Survey (USGS) Website – The USGS National Hazard Mapping Project website (http://earthquake.usgs.gov/hazards/apps/#deaggint) is a valuable tool for characterizing the seismic hazard for a specific site. The website allows the user to identify the peak ground acceleration (PGA) on soft bedrock/very dense or hard soils and spectral acceleration ordinates at periods of 0.2, 0.3 and 1 second for hazard levels of 2, 5 and 10 percent probabilities of exceedance (PE) in 50 years. The website also provides interactive deaggregation of a site's probabilistic seismic hazard. The deaggregation is useful in understanding the contribution of earthquakes of varying magnitude and distance to the seismic hazard at a site and is especially useful for liquefaction hazard evaluations. The website address is http://earthquake.usgs.gov/research/hazmaps/.

The results of the hazards analysis using the 2002 USGS website hazard model at a return period of 5 percent in 50 years are the same as those from the AASHTO hazard analysis maps. However, the USGS has updated their hazards maps (see USGS website for update). The USGS updated hazard results could differ somewhat from the results from the AASHTO hazards maps for the same location. In this case, if the updated hazard results are less conservative than the hazard level from the AASHTO hazard maps, the AASHTO hazard maps shall should be used as the basis for design.

Geotechnical Earthquake Engineering Textbook – The textbook titled *Geotechnical Earthquake Engineering* (Kramer, 1996) provides a wealth of information to geotechnical engineers for seismic design. The textbook includes a comprehensive summary of seismic hazards, seismology and earthquakes, strong ground motion, seismic hazard analysis, wave propagation, dynamic soil properties, ground response analysis, design ground motions, liquefaction, seismic slope stability, seismic design of retaining walls, and ground improvement.

Geotechnical seismic design is a rapidly developing sub-discipline within the broader context of the geotechnical engineering discipline, and new resources such as technical journal articles, as well as academic and government agency research reports, are becoming available to the geotechnical engineer. It is important when using these other resources, as well as those noted above, that a review be performed to confirm that the guidance represents the current state of knowledge and that the methods have received adequate independent review. Where new methods not given in the AASHTO Specifications or herein (i.e., Chapter 6) are proposed in the subject literature, use of the new method(s) shall be approved by the State Geotechnical Engineer for use in the project under consideration.

6.2 Geotechnical Seismic Design Considerations

6.2.1 Overview

The geotechnical designer has four broad options available for seismic design. They are:

- Use specification/code based hazard (Section 6.3.1) with specification/code based ground motion response (Section 6.3.2)
- Use specification/code based hazard (Section 6.3.1) with site specific ground motion response (Appendix 6-A)

- Use site specific hazard (Appendix 6-A) with specification/code based ground motion response (Section 6.3.2)
- Use site specific hazard (Appendix 6-A) with site specific ground motion response (Appendix 6-A)

Geotechnical parameters required for seismic design depend upon the type and importance of the structure, the geologic conditions at the site, and the type of analysis to be completed. For most structures, specification based design criteria appropriate for the site's soil conditions may be all that is required. Unusual, critical, or essential structures may require more detailed structural analysis, requiring additional geotechnical parameters. Finally, site conditions may require detailed geotechnical evaluation to quantify geologic hazards.

6.2.2 Site Characterization and Development of Seismic Design Parameters

As with any geotechnical investigation, the goal is to characterize the site soil conditions and determine how those conditions will affect the structures or features constructed when seismic events occur. In order to make this assessment, the geotechnical designer should review and discuss the project with the structural engineer, as seismic design is a cooperative effort between the geotechnical and structural engineering disciplines. The geotechnical designer should do the following as a minimum:

- Identify, in coordination with the structural designer, structural characteristics (e.g., fundamental frequency/period), anticipated method(s) of structural analysis, performance criteria (e.g., collapse prevention, allowable horizontal displacements, limiting settlements, target load and resistance factors, components requiring seismic design, etc.) and design hazard levels (e.g., 7 percent PE in 75 years).
- Identify, in coordination with the structural engineer, what type of ground motion parameters are required for design (e.g., response spectra or time histories), and their point of application (e.g., mudline, bottom of pile cap, or depth of pile fixity).
- Identify, in coordination with the structural engineer, how foundation stiffness will be modeled and provide appropriate soil stiffness properties or soil/foundation springs.
- Identify potential geologic hazards, areas of concern (e.g. soft soils), and potential variability of local geology.
- Identify potential for large scale site effects (e.g., basin, topographic, and near fault effects).
- Identify, in coordination with the structural designer, the method by which risk-compatible ground motion parameters will be established (specification/code, deterministic, probabilistic, or a hybrid).
- Identify engineering analyses to be performed (e.g. site specific seismic response analysis, liquefaction susceptibility, lateral spreading/slope stability assessments).
- Identify engineering properties required for these analyses.
- Determine methods to obtain parameters and assess the validity of such methods for the material type.
- Determine the number of tests/samples needed and appropriate locations to obtain them.

It is assumed that the basic geotechnical investigations required for nonseismic (gravity load) design have been or will be conducted as described in Chapters 2, 5 and the individual project element chapters (e.g., Chapter 8 for foundations, Chapter 15 for retaining walls, etc.). Typically, the subsurface data required for seismic design is obtained concurrently with the data required for design of the project (i.e., additional exploration for seismic design over and above what is required for nonseismic foundation design is typically not necessary). However, the exploration program may need to be adjusted to obtain the necessary parameters for seismic design. For instance, a seismic cone might be used in conjunction with a CPT if shear wave velocity data is required. Likewise, if liquefaction potential is a significant issue, mud rotary drilling with SPT sampling should be used. In this case, preference shall be given to drill rigs furnished with automatic SPT hammers that have been recently (i.e., within the past 6 months) calibrated for hammer energy. Hollow-stem auger drilling and non-standard samplers (e.g., down-the-hole or wire-line hammers) shall not be used to collect data used in liquefaction analysis and mitigation design, other than to obtain samples for gradation.

The goal of the site characterization for seismic design is to develop the subsurface profile and soil property information needed for seismic analyses. Soil parameters generally required for seismic design include:

- Dynamic shear modulus at small strains or shear wave velocity;
- Shear modulus and material damping characteristics as a function of shear strain;
- Cyclic and post-cyclic shear strength parameters (peak and residual);
- Consolidation parameters such as the Compression Index or Percent Volumetric Strain resulting from pore pressure dissipation after cyclic loading, and
- Liquefaction resistance parameters.

Table 6-1 provides a summary of site characterization needs and testing considerations for geotechnical/seismic design.

Chapter 5 covers the requirements for using the results from the field investigation, the field testing, and the laboratory testing program separately or in combination to establish properties for static design. Many of these requirements are also applicable for seismic design.

For routine designs, in-situ field measurements or laboratory testing for parameters such as the dynamic shear modulus at small strains, shear modulus and damping ratio characteristics versus shear strain, and residual shear strength are generally not obtained. Instead, correlations based on index properties may be used in lieu of in-situ or laboratory measurements for routine design to estimate these values. However, if a site specific ground motion response analysis is conducted, field measurements of the shear wave velocity V_s should be obtained.

Geotechnical Issues	Engineering Evaluations	Required Information for Analyses	Field Testing	Laboratory Testing
Site Response	• source characterization and ground motion attenuation • site response spectra • time history	• subsurface profile (soil, groundwater, depth to rock) • shear wave velocity • shear modulus for low strains • relationship of shear modulus with increasing shear strain, OCR, and PI • equivalent viscous damping ratio with increasing shear strain, OCR, and PI • Poisson's ratio • unit weight • relative density • seismicity (design earthquakes - source, distance, magnitude, recurrence)	• SPT • CPT • seismic cone • geophysical testing (shear wave velocity) • piezometer	• Atterberg limits • grain size distribution • specific gravity • moisture content • unit weight • resonant column • cyclic direct simple shear test • torsional simple shear test • cyclic triaxial tests
Geologic Hazards Evaluation (e.g., liquefaction, lateral spreading, slope stability, faulting)	• liquefaction susceptibility • liquefaction triggering • liquefaction induced settlement • settlement of dry sands • lateral spreading flow failure • slope stability and deformations	• subsurface profile (soil, groundwater, rock) • shear strength (peak and residual) • unit weights • grain size distribution • plasticity characteristics • relative density • penetration resistance • shear wave velocity • seismicity (PGA, design earthquakes, deaggregation data, ground motion time histories) • site topography	• SPT • CPT • seismic cone • Becker penetration test • vane shear test • piezometers • geophysical testing (shear wave velocity)	• grain size distribution • Atterberg Limits • specific gravity • organic content • moisture content • unit weight • soil shear strength tests (static and cyclic) • post-cyclic volumetric strain
Input for Structural Design	• soil stiffness for shallow foundations (e.g., springs) • P-Y data for deep foundations • down-drag on deep foundations • residual strength • lateral earth pressures • lateral spreading/ slope movement loading • post earthquake settlement • Kenematic soil-structure interaction	• subsurface profile (soil, groundwater, rock) • shear strength (peak and residual) • coefficient of horizontal subgrade reaction • seismic horizontal earth pressure coefficients • shear modulus for low strains or shear wave velocity • relationship of shear modulus with increasing shear strain • unit weight • Poisson's ratio • seismicity (PGA, design earthquake, response spectrum, ground motion time histories) • site topography • Interface strength	• CPT • SPT • seismic cone • piezometers • geophysical testing (shear wave velocity, resistivity, natural gamma) • vane shear test • pressuremete	• grain size distribution • Atterberg limits • specific gravity • moisture content • unit weight • resonant column • cyclic direct simple shear test • triaxial tests (static and cyclic) • torsional shear test • direct shear interface tests

Summary of Site Characterization Needs and Testing Considerations for Seismic Design (Adapted From Sabatini, et al., 2002)
Table 6-1

If correlations are used to obtain seismic soil design properties, and site- or region-specific relationships are not available, then the following correlations should be used:

- Table 6-2, which presents correlations for estimating initial shear modulus based on relative density, penetration resistance or void ratio.
- Shear modulus reduction and equivalent viscous damping ratio equations by Darendelli (2001), applicable to all soils, as provided below, or Figure 6-1, which presents shear modulus reduction curves and equivalent viscous damping ratio for sands as a function of shear strain and depth, and, Figures 6-2 and 6-3, which present shear modulus reduction curves and equivalent viscous damping ratio, respectively, as a function of cyclic shear strain and plasticity index for fine grained soils.
- Figures 6-4 through 6-7, which present charts for estimating equivalent undrained residual shear strength for liquefied soils as a function of SPT blowcounts. It is recommended that all these figures be checked to estimate residual strength and averaged using a weighting scheme. Table 6-3 presents an example of a weighting scheme as recommended by Kramer (2007). Designers using these correlations should familiarize themselves with how the correlations were developed, assumptions used, and any limitations of the correlations as discussed in the source documents for the correlations before selecting a final weighting scheme to use for a given project. Alternate correlations based on CPT data may also be considered.

Designers are encouraged to develop region or project specific correlations for these seismic design properties.

Regarding Figure 6-6, two curves are provided, one in which void redistribution is likely, and one in which void redistribution is not likely. Void redistribution becomes more likely if a relatively thick liquefiable layer is capped by relatively impermeable layer. Sufficient thickness of a saturated liquefiable layer is necessary to generate enough water for void redistribution to occur, and need capping by a relatively impermeable layer to prevent pore pressures from dissipating, allowing localized loosening near the top of the confined liquefiable layer. Engineering judgment will need to be applied to determine which curve in Figure 6-6 to use.

When using the above correlations, the potential effects of variations between the dynamic property from the correlation and the dynamic property for the particular soil should be considered in the analysis. The published correlations were developed by evaluating the response of a range of soil types; however, for any specific soil, the behavior of any specific soil can depart from the average, falling either above or below the average. These differences can affect the predicted response of the soil. For this reason sensitivity studies should be conducted to evaluate the potential effects of property variation on the design prediction. In lieu of more specific data on variability of the property in question, the following variations should be investigated with regard to their effect on design:

- In situ shear wave velocity: + 10 to 20 percent
- Shear modulus and viscous damping versus shear strain: + 20 percent
- Residual strength: + 20 percent

For those cases where a single value of the property can be used with the knowledge that the specific property selection will produce safe design results or for cases when the design is not very sensitive to variations in the property being considered, a sensitivity analysis may not be required.

Reference	Correlation	Units(1)	Limitations
Seed et al. (1984)	$G_{max} = 220\ (K_2)_{max}\ (\sigma'_m)^{½}$ $(K_2)_{max} = 20(N_1)_{60}^{1/3}$	kPa	$(K_2)_{max}$ is about 30 for very loose sands and 75 for very dense sands; about 80 to 180 for dense well graded gravels; Limited to cohesionless soils
Imai and Tonouchi (1982)	$G_{max} = 15{,}560\ N_{60}^{0.68}$	kPa	Limited to cohesionless soils
Hardin (1978)	$G_{max} = (6.25/0.3+e_o^{1.3})(P_a\ \sigma'_m)^{0.5}OCR^k$	kP_a(1)(3)	Limited to cohesive soils Pa = atmospheric pressure
Jamiolkowski, et al.. (1991)	$G_{max} = 6.25/(e_o^{1.3})(P_a\ \sigma'_m)^{0.5}OCR^k$	kP_a(1)(3	Limited to cohesive soils Pa = atmospheric pressure
Mayne and Rix (1993)	$G_{max} = 99.5(P_a)^{0.305}(q_c)^{0.695}/(e_0)^{1.13}$	kPa(2)	Limited to cohesive soils Pa = atmospheric pressure

Notes:

(1) 1 kPa = 20.885 psf

(2) P_a and q_c in kPa

(3) The parameter k is related to the plasticity index, PI, as follows:

PI	k
0	0
20	0.18
40	0.30
60	0.41
80	0.48
>100	0.50

Correlations for Estimating Initial Shear Modulus
(Adapted from Kavazanjian, et al., 2011)
Table 6-2

Modulus Reduction Curve (Darendelli, 2001) – The modulus reduction curve for soil, as a function of shear strain, should be calculated as shown in Equations 6-1 and 6-2.

$$\frac{G}{G_{\max}} = \frac{1}{1+\left(\frac{\gamma}{\gamma_r}\right)^a} \qquad (6\text{-}1)$$

where,

G = shear modulus at shear strain γ, in the same units as G_{max}
γ = shear strain (%), and
a = 0.92

γ_r is defined in Equation 6-2 as:

v $$\gamma_r = \left(\phi_1 + \phi_2 \times PI \times OCR^{\phi_3}\right) \times \sigma_0'^{\,\phi_4} \qquad (6\text{-}2)$$

where,

ϕ_1 = 0.0352; ϕ_2 = 0.0010; ϕ_3 = 0.3246; ϕ_4 = 0.3483 (from regression),
OCR = overconsolidation ratio for soil
σ'_0 = effective vertical stress, in atmospheres, and
PI = plastic index, in %

Damping Curve (Darendelli, 2001) – The damping ratio for soil, as a function of shear strain, should be calculated as shown in Equations 6-3 through 6-7.

Initial step: Compute closed-form expression for Masing Damping for *a* = 1.0 (standard hyperbolic backbone curve):

$$D_{\text{Masing, } a} = 1(\gamma)\ [\%] = = \frac{100}{\pi}\left[4\frac{\gamma - \gamma_r \ln\left(\frac{\gamma+\gamma_r}{\gamma_r}\right)}{\frac{\gamma^2}{\gamma+\gamma_r}} - 2\right] \tag{6-3}$$

For other values of *a* (e.g., a = 0.92, as used to calculate G):

$$D_{\text{Masing, } a}(\gamma)\ [\%] = c_1(D_{\text{masing, } a=1}) + c_2(D_{\text{masing, } a=1})^2 + c_3(D_{\text{masing, } a=1})^3 \tag{6-4}$$

Where,

c_1 = $0.2523 + 1.8618a - 1.1143a^2$
c_2 = $-0.0095 - 0.0710a + 0.0805a^2$
c_3 = $0.0003 + 0.0002a - 0.0005a^2$

Final step: Compute damping ratio as function of shear strain:

$$D(\gamma) = D_{\min} + bD_{Ma\sin g}(\gamma)\left(\frac{G}{G_{\max}}\right)^{0.1} \tag{6-5}$$

Where:

$$D_{\min} = \left(\phi_6 + \phi_7 \times PI \times OCR^{\phi_8}\right) \times \sigma_0'^{\phi_9} \times \left(1 + \phi_{10}\ln(freq)\right) \tag{6-6}$$

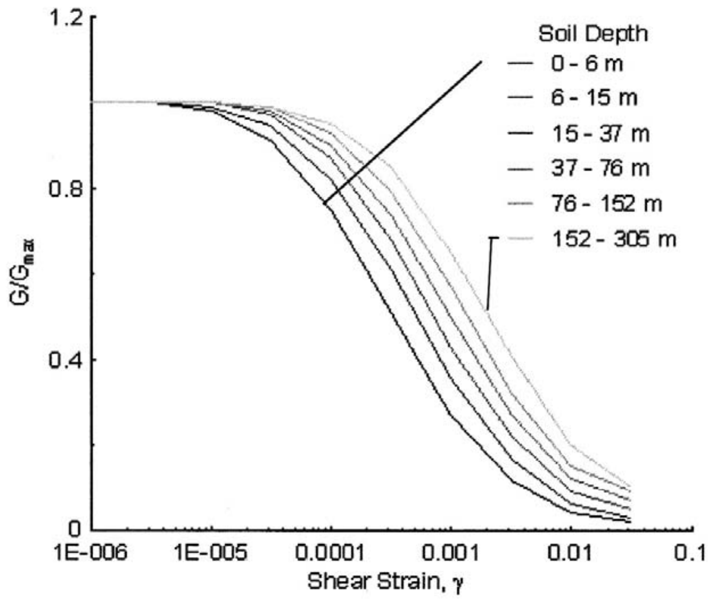
$$b = \phi_{11} + \phi_{12} \times \ln(N) \tag{6-7}$$

Where:

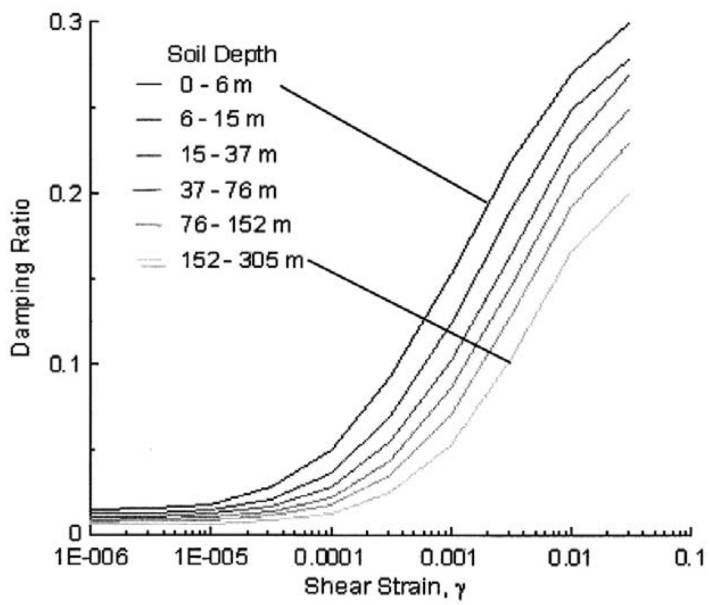
freq = frequency of loading, in Hz
N = number of loading cycles
φ_6 = 0.8005; φ_7 = 0.0129;
φ_8 = -0.1069;
φ_9 = -0.2889; φ_{10} = 0.2919;
φ_{11} = 0.6329; φ_{12} = -0.0057

Model	Weighting Factor
Idriss	0.2
Olson-Stark	0.2
Idriss-Boulanger	0.2
Hybrid	0.4

Weighting Factors for Residual Strength Estimation (Kramer, 2007)
Table 6-3

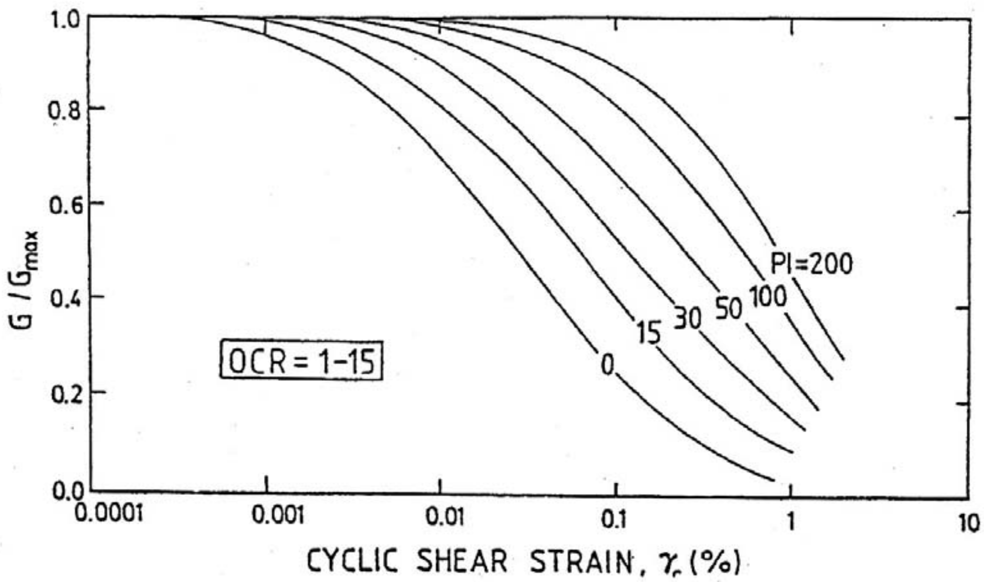


Shear Modulus Reduction Curves

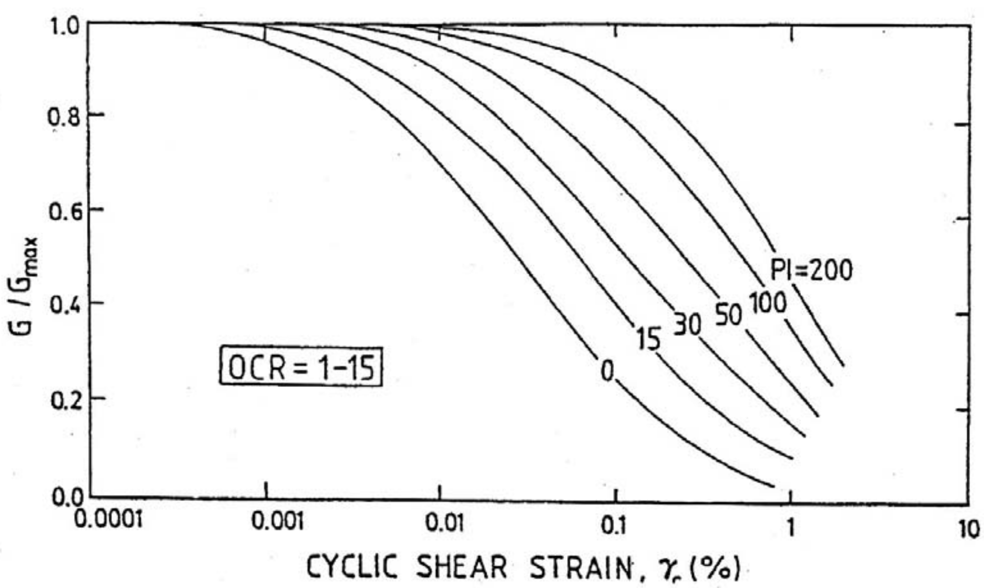


Damping Ratio Curves

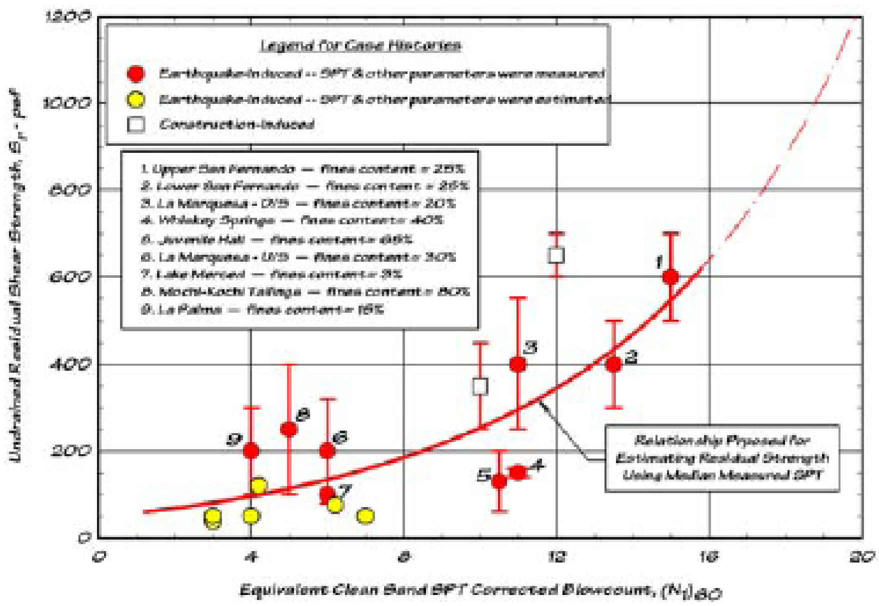
Shear Modulus Reduction and Damping Ratio Curves for Sand (EPRI, 1993)
Figure 6-1



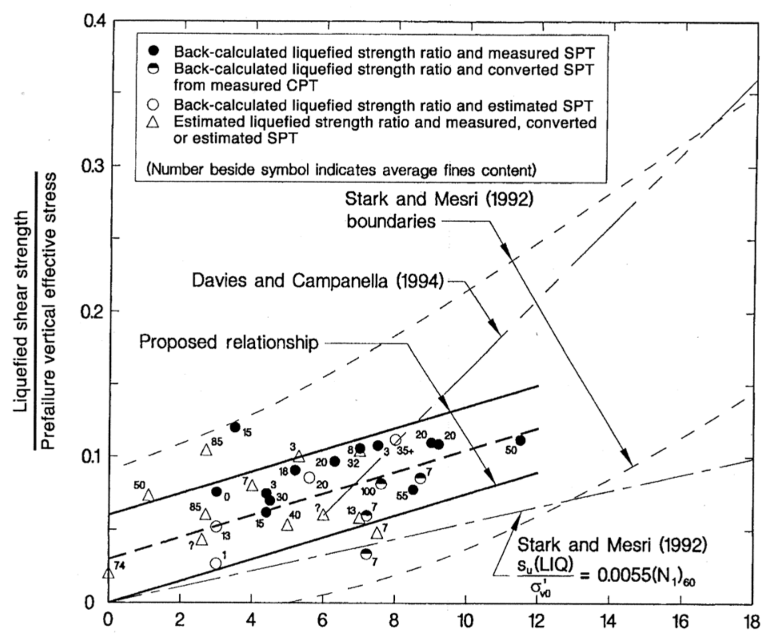
Shear Modulus Reduction Curves for Fine Grained Soils (Vucetic and Dobry, 1991)
Figure 6-2



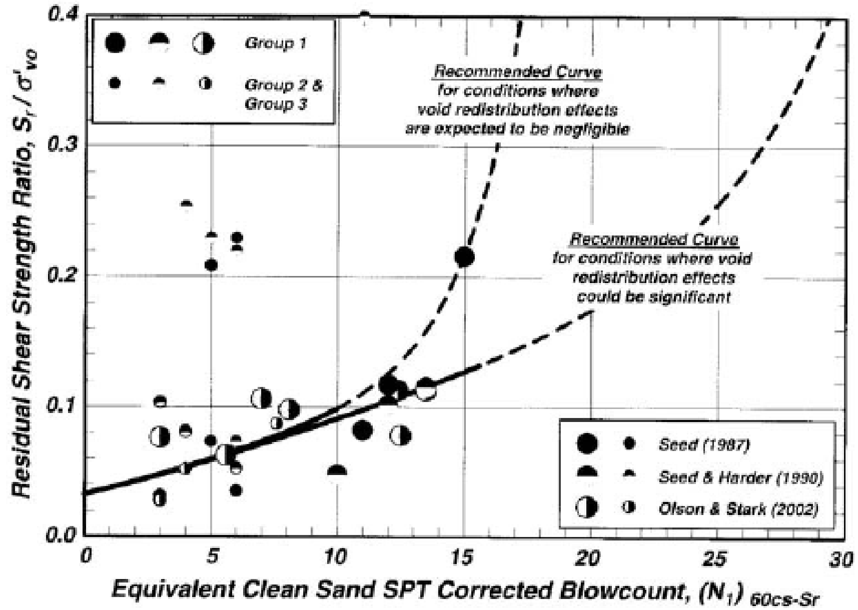
Equivalent Viscous Damping Ratio for Fine Grained Soils (Vucetic and Dobry, 1991)
Figure 6-3



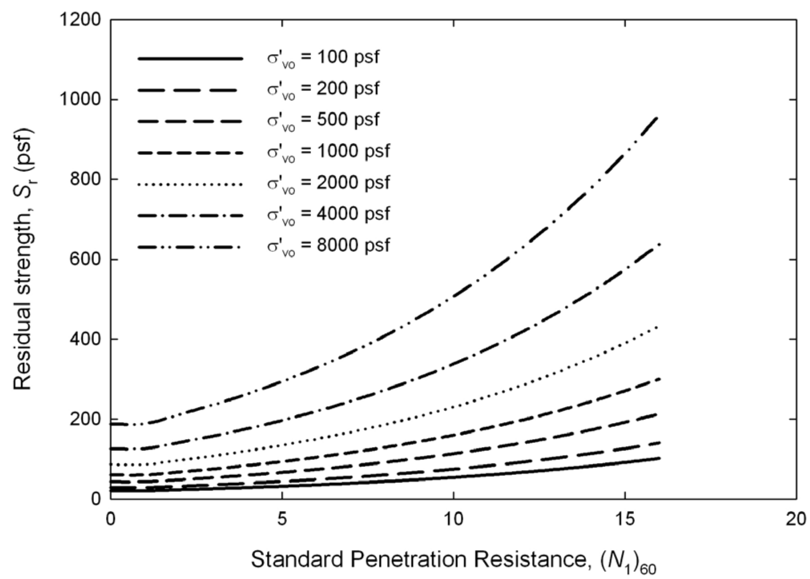
Estimation of Residual Strength from SPT Resistance
(Idriss and Boulanger, 2007)
Figure 6-4



Estimation of Residual Strength Ratio from SPT Resistance
(Olson and Stark, 2002)
Figure 6-5



Estimation of Residual Strength Ratio from SPT Resistance (Idriss and Boulanger, 2007)
Figure 6-6



Variation of Residual Strength Ratio with SPT Resistance and Initial Vertical Effective Stress Using Kramer-Wang Model (Kramer, 2007)
Figure 6-7

6.2.3 Information for Structural Design

The geotechnical designer shall recommend a design earthquake ground motion, and shall evaluate geologic hazards for the project. For code based ground motion analysis, the geotechnical designer shall provide the Site Class B spectral accelerations at periods of 0.2 and 1.0 seconds, the PGA, the site class, and the multipliers to the PGA and spectral accelerations to account for the effect of the site class on the design accelerations. Note that the site class should be determined considering the soils up to the ground surface, not just soil below the foundations. In addition, the geotechnical designer should evaluate the site and soil conditions to the extent necessary to provide the following input for structural design:

- Foundation spring values for dynamic loading (lateral and vertical), as well as geotechnical parameters for evaluation of sliding resistance applicable to the foundation design. If liquefaction is possible, spring values for liquefied conditionsshould also be provided (primarily applies to deep foundations, as in general, shallow footings are not used over liquefied soils).
- Earthquake induced earth pressures (active and passive) for retaining structures and below grade walls, and other geotechnical parameters, such as sliding resistance, needed to complete the seismic design of the wall.
- If requested by the structural designer, passive soil springs to use to model the abutment fill resistance to seismic motion of the bridge.
- Impacts of seismic geologic hazards including fault rupture, liquefaction, lateral spreading, flow failure, and slope instability on the structure, including estimated loads and deformations acting on the structure due to the effects of the geologic hazard.
- If requested by the structural designer, for long bridges, potential for incoherent ground motion effects.
- Options to mitigate seismic geologic hazards, such as ground improvement. Note that seismic soil properties used for design should reflect the presence of the soil improvement.

6.3 Seismic Hazard and Site Ground Motion Response Requirements

For most projects, design code/specification based seismic hazard and ground motion response (referred to as the "General Procedure" in the AASHTO Guide Specifications for LRFD Seismic Bridge Design) are appropriate and shall be used. However, a site specific hazard analysis should be considered in the following situations:

- A more accurate assessment of hazard level is desired, or
- Information about one or more active seismic sources for the site has become available since the USGS/AASHTO Seismic Hazard Maps were developed (USGS 2002), and the new seismic source information may result in a significant change of the seismic hazard at the site.

If the site is located within 6 miles of a known active fault capable of producing a magnitude 5 earthquake and near fault effects are not adequately modeled in the development of ground motion maps used, directivity and directionality effects shall be addressed as described in Article 3.4.3.1 of the AASHTO Guide Specifications for LRFD Seismic Bridge Design and its commentary.

A site specific ground motion response analysis shall be performed in the following situations:

- The facility is identified as critical or essential,
- Sites where geologic conditions are likely to result in un-conservative spectral acceleration values if the generalized code response spectra is used (e.g., a sharp change in impedance between subsurface strata is present, basin effects are present, etc.), or
- Site subsurface conditions are classified as Site Class F.

A site specific ground motion response analysis shall also be conducted for sites where the AASHTO or IBC site classes do not fit the subsurface conditions adequately. There may be other reasons why the general procedure cannot be used, such as the situation where the spectral acceleration coefficient at 1.0 second is greater than the spectral acceleration coefficient at 0.2 second. In such cases, a site specific ground motion analysis should be conducted. A site specific ground motion response analysis may also be conducted for sites where the effects of liquefaction on the ground motion response could be overly conservative.

If a site specific hazard analysis is conducted, it shall be conducted in accordance with AASHTO Guide Specifications for LRFD Seismic Bridge Design and Appendix 6-A. Note that where the response spectrum is developed using a site-specific hazard analysis, a site specific ground motion response analysis, or both, the AASHTO specifications require that the spectrum not be lower than two-thirds of the response spectrum at the ground surface determined using the general procedure of the AASHTO Guide Specifications for LRFD Seismic Bridge Design, Article 3.4.1, adjusted by the site coefficients (F_{pga}) in Article 3.4.2.3 in the region of $0.5T_F$ to $2T_F$ of the spectrum, where T_F is the bridge fundamental period. For other analyses such as liquefaction assessment and retaining wall design, the free field acceleration at the ground surface determined from a site specific analysis should not be less than two-thirds of the PGA multiplied by the specification based site coefficient F_{pga}.

When estimating the minimum ground surface response spectrum using two-thirds of the response spectrum from the specification based procedures provided in the AASHTO Guide Specifications for LRFD Seismic Bridge Design, there are no site coefficients for liquefiable sites or for sites that fall in Site Class F. No consensus currently exists regarding the appropriate site coefficients for these cases. Unless directed otherwise by the State Geotechnical Engineer and the State Bridge Engineer, the following approach shall be used:

- For liquefiable sites, use the specification based site coefficient for soil conditions without any modifications for liquefaction. This approach is believed to be conservative for higher frequency motions (i.e., $T_F < 1.0$ sec). If a site specific ground response analysis is conducted, the response spectrum shall not be lower than two-thirds of the non-liquefied specification based spectrum, unless specifically approved by the State Bridge and Geotechnical Engineers to go lower. When accepting a spectrum lower than the specification based spectrum, the uncertainties in the analysis method should be carefully reviewed, particularly for longer periods (i.e., $T > 1.0$ sec.) where increases in the spectral ordinate may occur. Because of this, for structures that are characterized as having a fundamental period, T_F, greater than 1.0 sec., a site specific ground response analysis shall be conducted if liquefiable soils are determined to be present.

- For Site Class F soils, conduct a site specific ground response analysis. In previous guidance documents, the suggestion was made to use a Site Class E site coefficient for Site Class F soils. Use of F_{pga}, F_a and F_v from Site Class E for Site Class F soils appears to be overly conservative and is not recommended.

If a site specific ground motion analysis to establish a response spectrum that is lower than two-thirds of the specification based spectrum is approved by the State Geotechnical and Bridge Engineers, the site specific analysis shall be independently peer reviewed by someone with expertise in the site specific ground response analysis technique used to conduct the analysis. When the site specific analysis is conducted by a consultant working for the State or a design-builder, the peer reviewer shall not be a staff member of the consultant(s) doing the engineering design for the project, even if not part of the specific team within those consultants doing the project design. The expert peer reviewer must be completely independent of the design team consultant(s).

If a site specific hazard analysis is conducted, it shall be independently peer reviewed in all cases. The peer reviewer shall meet the same requirements as described in the previous paragraph, except that their expertise must be in site specific seismic hazard analyses.

6.3.1 Determination of Seismic Hazard Level

All transportation structures (e.g., bridges, pedestrian bridges, walls, and WSF terminal structures such as docks, wing walls, etc.) classified as "other" (i.e., not critical or essential) by the AASHTO LRFD Bridge Design Specifications are designed for no-collapse based on a hazard level of 7 percent PE in 75 years (i.e., the same as 5 percent PE in 50 years and an approximately 1,000 year return period). Therefore, geotechnical seismic design for these structures shall be consistent with the no collapse design objective and the seismic hazard level used for those structures.

The AASHTO Guide Specifications for LRFD Seismic Bridge Design, or Figures 6-8, 6-9, and 6-10 shall be used to estimate the PGA, 0.2 sec. spectral acceleration (S_s), and 1.0 sec. spectral acceleration values (S_1), respectively, for WSDOT transportation facilities for code/specification based seismic hazard evaluation. By definition, PGA, S_S and S_1 are for Site Class B (very hard or very dense soil or soft rock) conditions. The PGA contours in Figure 6-8, in addition S_s and S_1 in Fgures 6-9 and 6-10, are based on information published by the USGS National Seismic Hazards Mapping Project (USGS, 2002) and published by AASHTO in the AASHTO Guide Specifications for LRFD Seismic Bridge Design. Interpolation between contours in Figure 6-8 should be used when establishing the PGA for Site Class B for a project.

When a transportation structure (e.g., bridges, walls, and WSF terminal structures such as docks, etc.) is designated as critical or essential by WSDOT, a more stringent seismic hazard level may be required by the State Bridge Engineer. If a different hazard level than that specified in the AASHTO LRFD Seismic design specifications is selected, the most current seismic hazard maps from the USGS National Seismic Hazards Mapping Project should be used, unless a site specific seismic hazard analysis is conducted, subject to the approval of the State Bridge Engineer and State Geotechnical Engineer.

If a site specific probabilistic seismic hazard analysis (PSHA) is conducted, it shall be conducted in a manner to generate a uniform-hazard acceleration response spectrum considering a 7 percent probability of exceedance in 75 years for spectral values over the entire period range of interest. This analysis shall follow the same basic approach as used by the USGS in developing seismic hazards maps for AASHTO. In this approach it is necessary to establish the following:

- The contributing seismic sources,
- A magnitude fault-rupture-length or source area relation for each contributing fault or source area to estimate an upper-bound earthquake magnitude for each source zone,
- Median attenuation relations for acceleration response spectral values and their associated standard deviations,
- A magnitude-recurrence relation for each source zone, and
- Weighting factors, with justification, for all branches of logic trees used to establish ground shaking hazards.

AASHTO allows site-specific ground motion hazard levels to be based on a deterministic seismic hazard analysis (DSHA) in regions of known active faults, provided that deterministic spectrum is no less than two-thirds of the probabilistic spectrum (see AASHTO Article 3.10.2.2). This requires that:

- The ground motion hazard at a particular site is largely from known faults (e.g., "random" seismicity is not a significant contributor to the hazard), and
- The recurrence interval for large earthquakes on the known faults are generally less than the return period corresponding to the specified seismic hazard level (e.g., the earthquake recurrence interval is less than a return period of 1,000 years that corresponds to a seismic hazard level of 7 percent probability of exceedance in 75 years).

Currently, these conditions are generally not met for sites in Washington State. Approval by the State Geotechnical Engineer and State Bridge Engineer is required before DSHA-based ground motion hazard level is used on a WSDOT project.

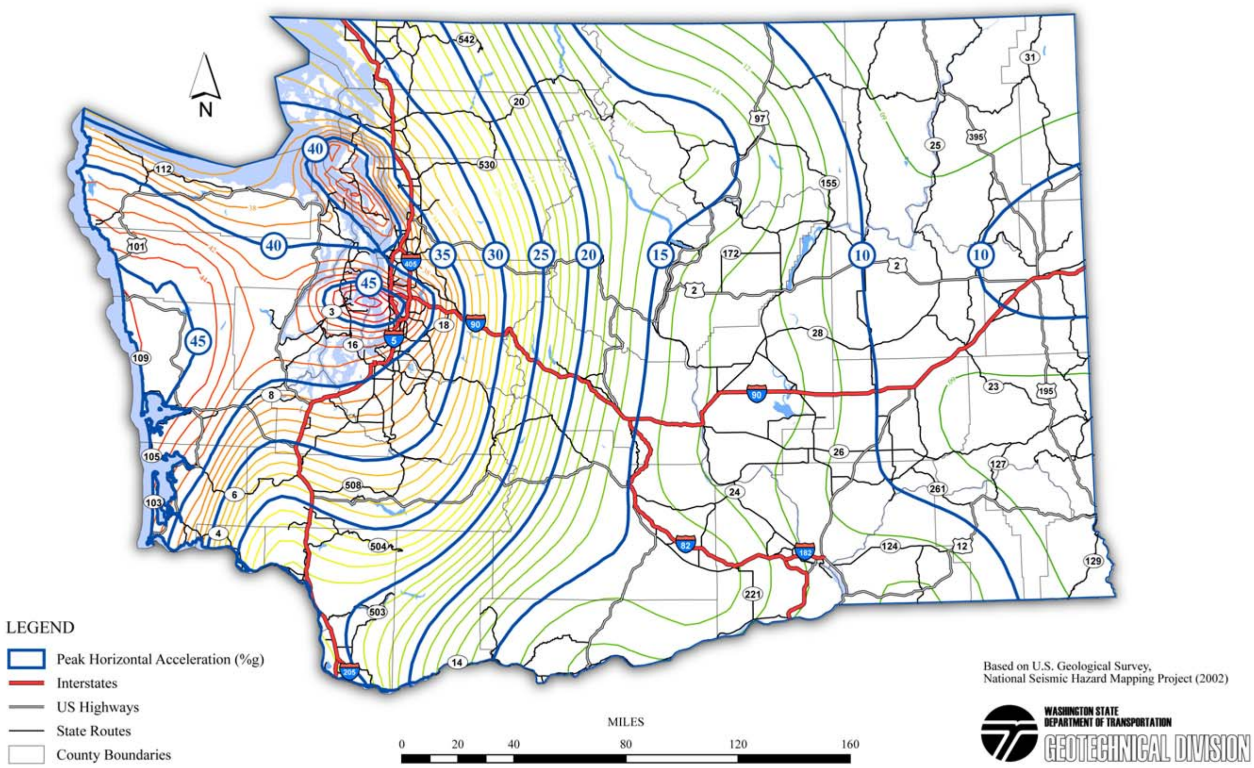
Where use of a deterministic spectrum is appropriate, the spectrum shall be either:

- The envelope of a median spectra calculated for characteristic maximum magnitude earthquakes on known active faults; or
- The deterministic spectra for each fault, and in the absence of a clearly controlling spectrum, each spectrum should be used.

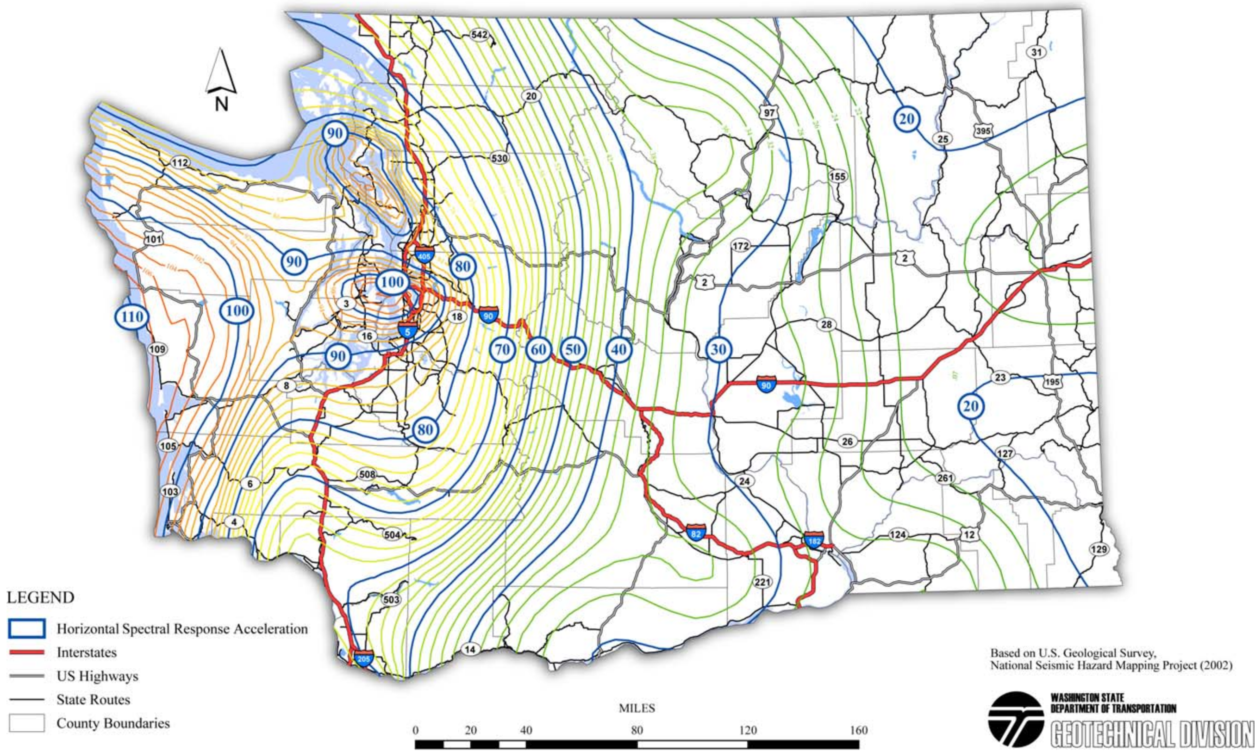
If the site specific deterministic hazard analysis is combined with a site specific ground motion response analysis, the response spectral ordinates may be as low as two-thirds of the response spectrum at the ground surface determined using the specification based procedures in the AASHTO LRFD Seismic Guide Specifications (Articles 3.4.1 and 3.4.2.3) in the region of $0.5T_F$ to $2T_F$. The same would also apply to the free field acceleration A_s in this case.

Uncertainties in source modeling and parameter values shall be taken into consideration in the PSHA and DSHA. Detailed documentation of seismic hazard analysis shall be provided.

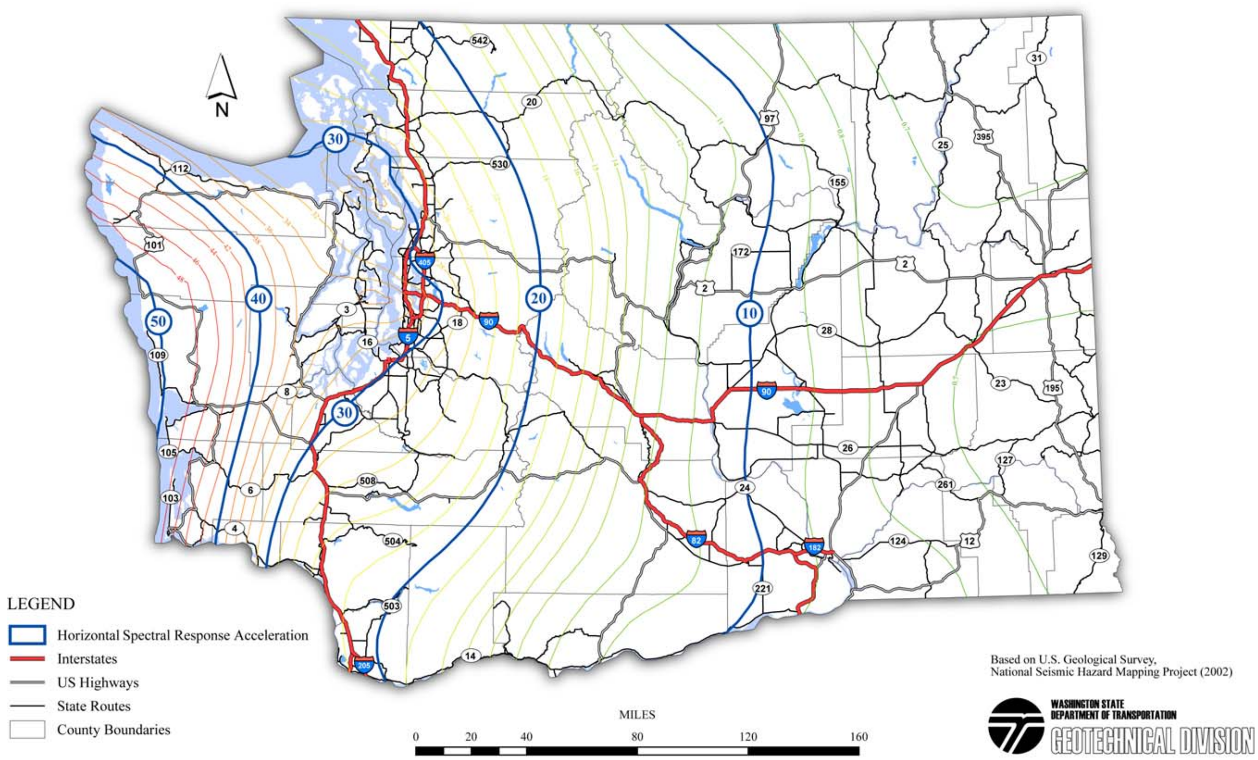
For buildings, restrooms, shelters, and covered walkways, specification based seismic design parameters required by the most current version of the International Building Code (IBC) shall be used. The seismic design requirements of the IBC are based on a hazard level of 2 percent PE in 50 years. The 2 percent PE in 50 years hazard level corresponds to the maximum considered earthquake (MCE). The IBC identifies procedures to develop a maximum considered earthquake acceleration response spectrum, at the ground surface by adjusting Site Class B spectra for local site conditions, similar to the methods used by AASHTO except that the probability of exceedance is lower (i.e., 2 percent in 50 years versus 7 percent in 75 years). However, the IBC defines the design response spectrum as two-thirds of the value of the maximum considered earthquake acceleration response spectrum. The site factors used in IBC are the same as used by AASHTO for modifying the Site Class B spectrum for local site effects. As is true for transportation structures, for critical or unique structures, for sites characterized as soil profile Type F (thick sequence of soft soils in the IBC) or liquefiable soils, or for soil conditions that do not adequately match the specification based soil profile types, site specific response analysis may be required as discussed in Appendix 6-A.



Peak Horizontal Acceleration (%G) for 7% Probability of Exceedance in 75 Years for Site Class B (Adapted From AASHTO 2012)
Figure 6-8



Horizontal Spectral Acceleration at 0.2 Second Period (%g) for 7% Probability of Exceedance in 75 Years with 5% of Critical Damping for Site Class B (Adapted from AASHTO 2012)
Figure 6-9



Horizontal Spectral Acceleration at 1.0 Second Period (%g) for 7% Probability of Exceedance in 75 Years With 5% of Critical Damping for Site Class B (Adapted from AASHTO 2012)
Figure 6-10

6.3.2 Site Ground Motion Response Analysis

The AASHTO Guide Specifications for LRFD Bridge Seismic Design require that site effects be included in determining seismic loads for design of bridges. The guide specifications characterize all subsurface conditions with six Site Classes (A through F) and provides site soil coefficients for PGA (F_{pga}), S_S (F_a), and S_1 (F_v) for five of the Site Classes (A through E). Code/specification based response spectra that include the effect of ground motion amplification or de-amplification from the soil/rock stratigraphy at the site can be developed from the PGA, S_S, S_1 and the Site-Class-based site coefficients F_{pga}, F_a, and F_v. The geotechnical designer shall determine the appropriate site coefficient (F_{pga} for PGA, F_a for S_S, and F_v for S_1) to construct the code/specification based response spectrum for the specific site subsurface conditions. Tables 3.4.2.3-1, 3.4.2.3-2, and 3.4.2.3-3 of the AASHTO Guide Specifications for LRFD Bridge Seismic Design present the values of the Site Coefficients for Soil Classes A through E. No specification based site class values are available for Site Class F, however – in that case, a site specific ground response analysis must be conducted (see the AASHTO Guide Specifications for LRFD Bridge Seismic Design for additional details on site conditions that are considered to be included in Site Class F).

The AASHTO LRFD Bridge Design Specifications do not specifically require that a site specific seismic ground response analyses be completed for sites where liquefaction is anticipated during a design earthquake. The AASHTO Guide Specifications for LRFD Bridge Seismic Design require that the specification based ground motion spectral response for nonliquefied conditions be used unless a site specific ground motion response analysis is conducted. However, as discussed at the beginning of Section 6-3 herein, for structures with a fundamental period, T_F, greater than 1.0 sec., a site specific response analysis shall be conducted if the soils at the site are potentially liquefiable.

Sites that contain a strong impedance contrast, i.e., a boundary between adjacent layers with shear wave velocities that differ by a factor of 2 or more, may benefit from a site-specific seismic ground response analysis. The strong impedance contrast can occur where a thin soil profile (e.g., < 20 to 30 feet) overlies rock or where layers of soft and stiff soils occur.

6.3.3 IBC for Site Response

The IBC, Sections 1613 through 1615, provides procedures to estimate the earthquake loads for the design of buildings and similar structures. Earthquake loads per the IBC are defined by acceleration response spectra, which can be determined through the use of the IBC general response spectrum procedures or through site-specific procedures. The intent of the IBC MCE is to reasonably account for the maximum possible earthquake at a site, to preserve life safety and prevent collapse of the building.

The general response spectrum per the IBC utilizes mapped Maximum Considered Earthquake (MCE) spectral response accelerations at short periods (S_s) and at 1-second (S_1) to define the seismic hazard at a specific location in the United States.

The IBC uses the six site classes, Site Class A through Site Class F, to account for the effects of soil conditions on site response. The geotechnical designer shall identify the appropriate Site Class for the site. Note that the site class should be determined considering the soils up to the ground surface, not just soil below the foundations.

Once the Site Class and mapped values of S_s and S_1 are determined, values of the Site Coefficients F_a and F_v (site response modification factors) can be determined. The Site Coefficients and the mapped spectral accelerations S_s and S_1 can then be used to define the MCE and design response spectra. The PGA at the ground surface may be estimated as 0.4 of the 0.2 sec design spectral acceleration.

For sites where Site Class F soils are present, the IBC requires that a site-specific geotechnical investigation and dynamic site response analysis be completed (see Appendix 6-A). Dynamic site response analysis may not be required for liquefiable soil sites for structures with predominant periods of vibration less than 0.5 seconds.

6.3.4 Adjusting Ground Surface Acceleration to Other Site Classes

The site coefficient F_{pga} to account for the difference in ground response between Class B soil/rock conditions to other site classes with regard to the estimation of acceleration A_s are directly incorporated into the development of the standard response spectra for structural design of bridges and similar structures in the AASHTO LRFD Bridge Design Specifications and for the structural design of buildings and non-transportation related structures in the IBC. However, the PGA shall also be multiplied by F_{pga} to account for the site class when assessing the potential for liquefaction and for the estimation of seismic earth pressures and inertial forces for retaining wall and slope design. For liquefaction assessment and retaining wall and slope design, the site coefficient presented in the AASHTO Guide Specifications for LRFD Bridge Seismic Design shall be used, unless a site specific evaluation of ground response conducted in accordance with these AASHTO Guide specifications and Section 6.3 and Appendix 6-A is performed. Note that the site class should be determined considering the soils up to the ground surface, not just soil below the foundations.

6-3.5 Earthquake Magnitude

Assessment of liquefaction and lateral spreading require an estimate of the earthquake magnitude. The magnitude should be assessed using the seismic deaggregation data for the site, available through the USGS national seismic hazard website (earthquake.usgs.gov/hazards/) as discussed in Appendix 6-A. The deaggregation used shall be for a seismic hazard level consistent with the hazard level used for the structure for which the liquefaction analysis is being conducted (typically, a probability of exceedance of 5 percent in 50 years in accordance with the AASHTO Guide Specifications for LRFD Seismic Bridge Design). Additional discussion and guidance regarding the selection of earthquake magnitude values is provided in the AASHTO Guide Specifications for LRFD Bridge Seismic Design.

6.4 Seismic Geologic Hazards

The geotechnical designer shall evaluate seismic geologic hazards including fault rupture, liquefaction, lateral spreading, ground settlement, and slope instability. The potential effects associated with seismic geologic hazards shall be evaluated by the geotechnical designer.

6.4.1 Fault Rupture

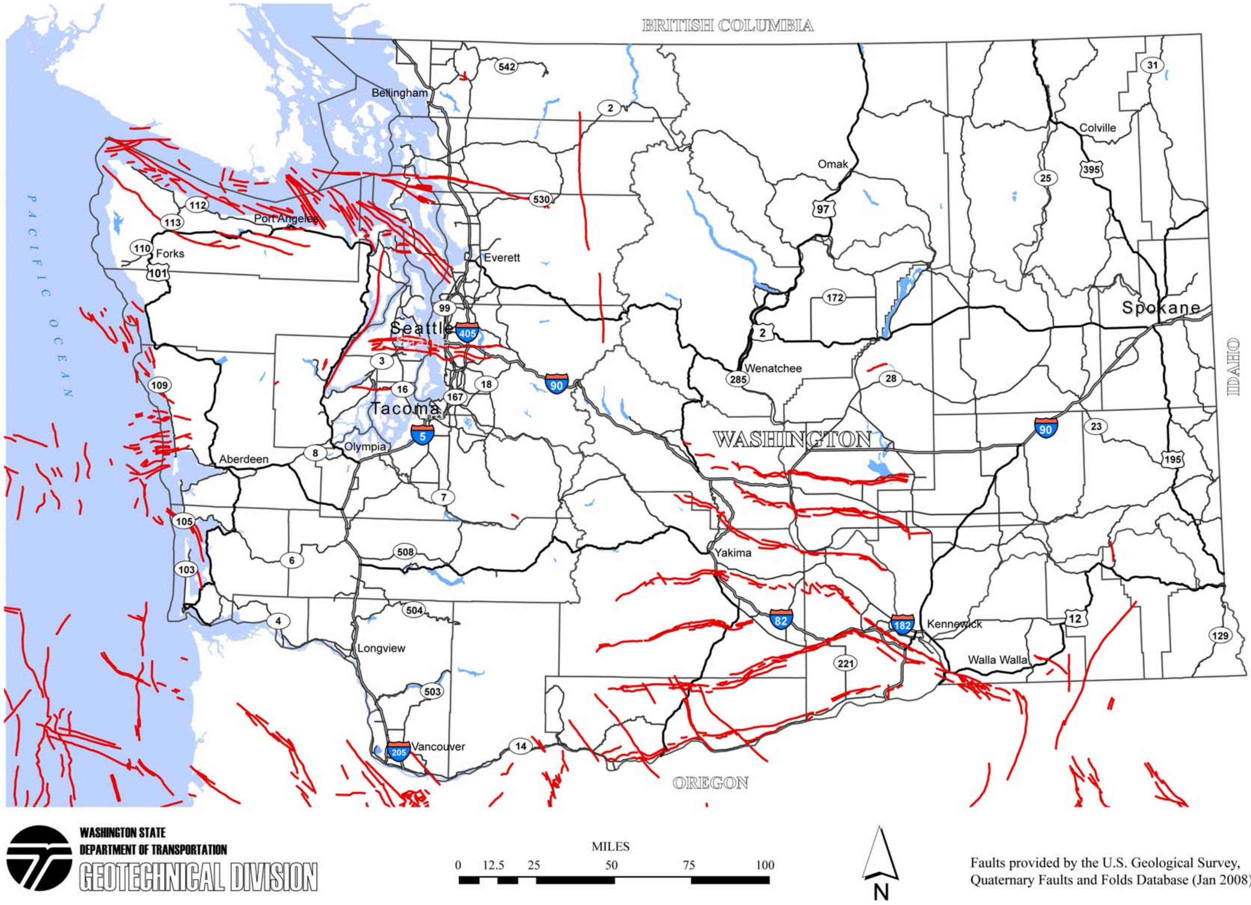
Washington State is recognized as a seismically active region; however, only a relatively small number of active faults have been identified within the state. Thick sequences of recent geologic deposits, heavy vegetation, and the limited amount of instrumentally recorded events on identified faults are some of the factors that contribute to the difficulty in identifying active faults in Washington State. Considerable research is ongoing throughout Washington State to identify and characterize the seismicity of active faults, and new technology makes it likely that additional surface faults will be identified in the near future.

Figure 6-11 presents the earthquake faults in the North American plate considered to be potentially active. The following faults are explicitly included in the 2002 USGS probabilistic hazard maps that were used in the development of the AASHTO seismic hazards maps:

- Seattle Fault Zone
- Southern Whidbey Island Fault
- Utsalady Fault
- Strawberry Point Fault
- Devils Mountain Fault
- Horse Heaven Hills Anticline
- Rattlesnake-Wallula Fault System
- Mill Creek Fault
- Saddle Mountains Fault
- Hite Fault System

The potential impacts of fault rupture include abrupt, large, differential ground movements and associated damage to structures that might straddle a fault, such as a bridge. Until the recent application of advanced mapping techniques (e.g., LIDAR and aeromagnetics) in combination with trenching and age dating of apparent ground offsets, little information was available regarding the potential for ground surface fault rupture hazard in Washington State. However, WSDOT expects that as these techniques are applied throughout the state, additional Holocene faults traces and fault zones will likely be identified, and the understanding of ground surface rupture hazard may change significantly with time.

In view of the advances that will likely be made in the area of fault identification, the potential for fault rupture should be evaluated and taken into consideration in the planning and design of new facilities. These evaluations should incorporate the latest information identifying potential Holocene ground deformation.



Earthquake Faults in Washington State (Adapted from USGS, 2002)
Figure 6-4-1

6.4.2 Liquefaction

Liquefaction has been one of the most significant causes of damage to bridge structures during past earthquakes (ATC-MCEER Joint Venture, 2002). Liquefaction can damage bridges and structures in many ways including:

- Modifying the nature of ground motion;
- Bearing failure of shallow foundations founded above liquefied soil;
- Changes in the lateral soil reaction for deep foundations;
- Liquefaction induced ground settlement;
- Lateral spreading of liquefied ground;
- Large displacements associated with low frequency ground motion;
- Increased earth pressures on subsurface structures;
- Floating of buoyant, buried structures; and
- Retaining wall failure.

Liquefaction refers to the significant loss of strength and stiffness resulting from the generation of excess pore water pressure in saturated, predominantly cohesionless soils. Kramer (1996) provides a detailed description of liquefaction including the types of liquefaction phenomena, evaluation of liquefaction susceptibility, and the effects of liquefaction.

All of the following general conditions are necessary for liquefaction to occur:

- The presence of groundwater, resulting in a saturated or nearly saturated soil.
- Predominantly cohesionless soil that has the right gradation and composition. Liquefaction has occurred in soils ranging from low plasticity silts to gravels. Clean or silty sands and non-plastic silts are most susceptible to liquefaction.
- A sustained ground motion that is large enough and acting over a long enough period of time to develop excess pore-water pressure, equal to the effective overburden stress, thereby significantly reducing effective stress and soil strength,
- The state of the soil is characterized by a density that is low enough for the soil to exhibit contractive behavior when sheared undrained under the initial effective overburden stress.

Methods used to assess the potential for liquefaction range from empirically based design methods to complex numerical, effective stress methods that can model the time-dependent generation of pore-water pressure and its effect on soil strength and deformation. Furthermore, dynamic soil tests such as cyclic simple shear or cyclic triaxial tests can be used to assess liquefaction susceptibility and behavior to guide input for liquefaction analysis and design.

Liquefaction hazard assessment includes identifying soils susceptible to liquefaction, evaluating whether the design earthquake loading will initiate liquefaction, and estimating the potential effects of liquefaction on the planned facility. Liquefaction hazard assessment is required in the AASHTO Guide Specifications for LRFD Seismic Bridge Design if the site Seismic Design Category (SDC) is classified as SDC C or D, and the soil is identified as being potentially susceptible to liquefaction (see Section 6.4.2.1). The SDC is defined on the basis of the site-adjusted spectral acceleration at 1 second (i.e., $S_{D1} = F_v S_1$) where SDC C is defined as $0.30 \leq S_{D1} < 0.5$ and SDC D is defined as $S_{D1} \geq 0.50$.Where loose to very loose, saturated sands are within the subsurface profile such that liquefaction could impact the stability of the structure, the potential for liquefaction in SDC B ($0.15 \leq S_{D1} < 0.3$) should also be considered as discussed in the AASHTO Guide Specifications for LRFD Seismic Bridge Design.

To determine the location of soils that are adequately saturated for liquefaction to occur, the seasonally averaged groundwater elevation should be used. Groundwater fluctuations caused by tidal action or seasonal variations will cause the soil to be saturated only during a limited period of time, significantly reducing the risk that liquefaction could occur within the zone of fluctuation.

For sites that require an assessment of liquefaction, the potential effects of liquefaction on soils and foundations shall be evaluated. The assessment shall consider the following effects of liquefaction:

- Loss in strength in the liquefied layer(s) with consideration of potential for void redistribution due to the presence of impervious layers within or bounding a liquefiable layer

- Liquefaction-induced ground settlement, including downdrag on deep foundation elements
- Slope instability induced by flow failures or lateral spreading

During liquefaction, pore-water pressure build-up occurs, resulting in loss of strength and then settlement as the excess pore-water pressures dissipate after the earthquake. The potential effects of strength loss and settlement include:

- **Slope Instability Due to Flow Failure or Lateral Spreading** – The strength loss associated with pore-water pressure build-up can lead to slope instability. Generally, if the factor of safety against liquefaction is less than approximately 1.2 to 1.3, a potential for pore-water pressure build-up will occur, and the effects of this build-up shall be assessed. If the soil liquefies, slope stability is determined by the residual strength of the soil. The residual strength of liquefied soils can be estimated using empirical methods. Loss of soil resistance can allow abutment soils to move laterally, resulting in bridge substructure distortion and unacceptable deformations and moments in the superstructure.
- **Reduced foundation bearing resistance** – The residual strength of liquefied soil is often a fraction of nonliquefied strength. This loss in strength can result in large displacements or bearing failure. For this reason spread footing foundations are not recommended where liquefiable soils exist unless the spread footing is located below the maximum depth of liquefaction or soil improvement techniques are used to mitigate the effects of liquefaction.
- **Reduced soil stiffness and loss of lateral support for deep foundations** – This loss in strength can change the lateral response characteristics of piles and shafts under lateral load.

Vertical ground settlement will occur as excess pore-water pressures induced by liquefaction dissipate, resulting in downdrag loads on and loss of vertical support for deep foundations. If liquefaction-induced downdrag loads can occur, the downdrag loads shall be assessed as specified in Sections 6.5.3 and 8.12.2.7, and in Article 3.11.8 in the AASHTO LRFD Bridge Design Specifications.

The effects of liquefaction will depend in large part on the amount of soil that liquefies and the location of the liquefied soil with respect to the foundation. On sloping ground, lateral flow, spreading, and slope instability can occur even on gentle slopes on relatively thin layers of liquefiable soils, whereas the effects of thin liquefied layer on the lateral response of piles or shafts (without lateral ground movement) may be negligible. Likewise, a thin liquefied layer at the ground surface results in essentially no downdrag loads, whereas the same liquefied layer deeper in the soil profile could result in large downdrag loads. Given these potential variations, the site investigation techniques that can identify relatively thin layers are a fundamental part of the liquefaction assessment.

The following sections provide requirements for liquefaction hazard assessment and its mitigation.

6.4.2.1 Methods to Evaluate Potential Susceptibility of Soil to Liquefaction

Evaluation of liquefaction potential shall be completed based on soil characterization using in-situ testing such as Standard Penetration Tests (SPT) and Cone Penetration Tests (CPT). Liquefaction potential may also be evaluated using shear wave velocity (V_s) testing and Becker Penetration Tests (BPT) for soils that are difficult to test using SPT and CPT methods, such as gravelly soils; however, these methods are not preferred and are used less frequently than SPT or CPT methods. If the CPT method is used, SPT sampling and soil gradation testing shall still be conducted to obtain direct information on soil gradation parameters for liquefaction susceptibility assessment and to provide a comparison to CPT based analysis.

Simplified screening criteria to assess the potential liquefaction susceptibility of sands and silts based on soil gradation and plasticity indices should be used. In general, gravelly sands through low plasticity silts should be considered potentially liquefiable, provided they are saturated and very loose to medium dense.

If a more refined analysis of liquefaction potential is needed, laboratory cyclic triaxial shear or cyclic simple shear testing may be used to evaluate liquefaction susceptibility and initiation in lieu of empirical soil gradation/PI/density criteria, in accordance with Section 6.4.2.6.

Preliminary Screening – A detailed evaluation of liquefaction potential is required if all of the following conditions occur at a site, and the site Seismic Design Category is classified as SDC C or D:

- The estimated maximum groundwater elevation at the site is determined to be within 50 feet of the existing ground surface or proposed finished grade, whichever is lower.
- The subsurface profile is characterized in the upper 75 feet as having low plasticity silts, sand, or gravelly sand with a measured SPT resistance, corrected for overburden depth and hammer energy ($N1_{60}$), of 25 blows/ft, or a cone tip resistance q_{ciN} of 150, or a geologic unit is present at the site that has been observed to liquefy in past earthquakes. For low plasticity silts and clays, the soil is considered liquefiable as defined by the Bray and Sancio (2006) or Boulanger and Idriss (2006) criteria.

For loose to very loose sand sites [e.g., $(N1)_{60}$, < 10 bpf or q_{c1N}, < 75], a potential exists for liquefaction in SDC B, if the acceleration coefficient, A_s (i.e., PGA × F_{pga}), is 0.15 or higher. The potential for and consequences of liquefaction for these sites will depend on the dominant magnitude for the seismic hazard and just how loose the soil is. As the magnitude decreases, the liquefaction resistance of the soil increases due to the limited number of earthquake loading cycles. Generally, if the magnitude is 6 or less, the potential for liquefaction, even in these very loose soils, is either very low or the extent of liquefaction is very limited. Nevertheless, a liquefaction assessment should be made if loose to very loose sands are present to a sufficient extent to impact bridge stability and A_s is greater than or equal to 0.15. These loose to very loose sands are likely to be present in hydraulically placed fills and alluvial or estuarine deposits near rivers and waterfronts.

If the site meets the conditions described above, a detailed assessment of liquefaction potential shall be conducted. If all conditions are met except that the water table depth is greater than 50 feet but less than 75 feet, a liquefaction evaluation should still be considered, and if deep foundations are used, the foundation tips shall be located below the bottom of the liquefiable soil that is below the water table, or adequately above the liquefiable zone such that the impact of the liquefaction does not cause bridge or wall collapse.

Liquefaction Susceptibility of Silts – Liquefaction susceptibility of silts should be evaluated using the criteria developed by Bray and Sancio (2006) or Boulanger and Idriss (2006) if laboratory cyclic triaxial or cyclic simple shear tests are not conducted. The Modified Chinese Criteria (Finn, et al., 1994) that has been in use in the past has been found to be unconservative based on laboratory and field observations (Boulanger and Idriss, 2006). Therefore, the new criteria proposed by Bray and Sancio or Boulanger and Idriss are recommended. According to the Bray and Sancio criteria, fine-grained soils are considered susceptible to liquefaction if:

- The soil has a water content(w_c) to liquid limit (LL) ratio of 0.85 or more; and
- The soil has a plasticity index (PI) of less than 12.

For fine grained soils that are outside of these ranges of plasticity, cyclic softening resulting from seismic shaking may need to be considered. According to the Boulanger and Idriss (2006) criterion, fine grained soils are considered susceptible to liquefaction if the soil has a PI of less than 7. Since there is a significant difference in the screening criteria for liquefaction of silts in the current literature, for soils that are marginally susceptible or not susceptible to liquefaction, cyclic triaxial or simple shear laboratory testing of undisturbed samples is recommended to assess whether or not the silt is susceptible to liquefaction, rather than relying solely on the screening criteria.

Liquefaction Susceptibility of Gravels – No specific guidance regarding susceptibility of gravels to liquefaction is currently available. The primary reason why gravels may not liquefy is that their high permeability frequently precludes the development of undrained conditions during and after earthquake loading. When bounded by lower permeability layers, however, gravels should be considered susceptible to liquefaction and their liquefaction potential evaluated. A gravel that contains sufficient sand to reduce its permeability to a level near that of the sand, even if not bounded by lower permeability layers, should also be considered susceptible to liquefaction and its liquefaction potential evaluated as such. Becker hammer testing and sampling could be useful for obtaining a representative sample of the sandy gravel that can be used to get an accurate soil gradation for assessing liquefaction potential. Downhole suspension logging (suspension logging in a mud rotary hole, not cased boring) should also be considered in such soils, as high quality V_s testing can overcome the variation in SPT test results caused by the presence of gravels.

6.4.2.2 Determination of Whether or Not a Soil will Liquefy

The most common method of assessing liquefaction involves the use of empirical methods (i.e., Simplified Procedures). These methods provide an estimate of liquefaction potential based on SPT blowcounts, CPT cone tip resistance, BPT blowcounts, or shear wave velocity. This type of analysis shall be conducted as a baseline evaluation, even when more rigorous methods are used. More rigorous, nonlinear, dynamic, effective stress computer models may be used for site conditions or situations that are not modeled well by the simplified methods, subject to the approval of the State Geotechnical Engineer. For situations where simplified (empirical) procedures are not allowed (e.g., to assess liquefaction at depths greater than 50 to 80 ft as described in Section 6.1.2.3), these more rigorous computer models should be used, and independent peer review, as described in Section 6.3, of the results from these more rigorous computer models shall be conducted.

Simplified Procedures – Procedures that should be used for evaluating liquefaction susceptibility using SPT, CPT, V_s, and BPT criteria are provided in Youd et al. (2001). Youd et al. summarize the consensus of the profession up to year 2000 regarding the use of the simplified (i.e., empirical) methods. Since the publication of this consensus paper, various other modifications to the consensus approach have been introduced, including those by Cetin et al. (2004), Moss et al. (2006), Boulanger and Idriss (2006), and Idriss and Boulanger (2008). These more recent modifications to these methods account for additions to the database on liquefaction, as well as refinements in the interpretation of case history data. The updated methods potentially offer improved estimates of liquefaction potential, and should be considered for use.

The simplified procedures are based on comparing the cyclic resistance ratio (CRR) of a soil layer (i.e., the cyclic shear stress required to cause liquefaction) to the earthquake induced cyclic shear stress ratio (CSR). The CRR is a function of the soil relative density as represented by an index property measure (e.g., SPT blowcount), the fines content of the soil taken into account through the soil index property used, the in-situ vertical effective stress as represented by a factor K_σ , an earthquake magnitude scaling factor, and possibly other factors related to the geologic history of the soil. The soil index properties are used to estimate liquefaction resistance based on empirical charts relating the resistance available to specific index properties (i.e., SPT, CPT, BPT or shear wave velocity values) and corrected to an equivalent magnitude of 7.5 using a magnitude scaling factor. The earthquake magnitude is used to empirically account for the duration of shaking or number of cycles.

The basic form of the simplified procedures used to calculate the earthquake induced CSR for the Simplified Method is as shown in Equation 6-8:

$$CSR = 0.65 \frac{A_{max}}{g} \frac{\sigma_o}{\sigma_o'} \frac{r_d}{MSF} \tag{6-8}$$

Where

A_{max} = peak ground acceleration accounting for site amplification effects
g = acceleration due to gravity
σ_o = initial total vertical stress at depth being evaluated
σ_o' = initial effective vertical stress at depth being evaluated
r_d = stress reduction coefficient
MSF = magnitude scaling factor

Note that A_{max} is the PGA times the acceleration due to gravity, since the PGA is actually an acceleration coefficient, and A_{max}/g is equal to A_s.

The factor of safety against liquefaction is defined by Equation 6-9:

$$FS_{liq} = CRR/CSR \tag{6-9}$$

The SPT procedure has been most widely used and has the advantage of providing soil samples for gradation and Atterberg limits testing. The CPT provides the most detailed soil stratigraphy, is less expensive, can provide shear wave velocity measurements, and is more reproducible. If the CPT is used, soil samples shall be obtained using the SPT or other methods so that detailed gradational and plasticity analyses can be conducted. The use of both SPT and CPT procedures can provide a detailed liquefaction assessment for a site.

Where SPT data is used, sampling and testing shall be conducted in accordance with Chapter 3. In addition:

- Correction factors for borehole diameter, rod length, hammer type, and sampler liners shall be used, where appropriate.
- Where gravels or cobbles are present, the use of short interval adjusted SPT N values may be effective for estimating the N values for the portions of the sample not affected by gravels or cobbles.
- Blowcounts obtained when sampling using Dames and Moore or modified California samplers or non-standard hammer weights and drop heights, including wireline and downhole hammers, shall not be used for liquefaction evaluations.

As discussed in Section 6.1.2.2, the limitations of the simplified procedures should be recognized. The simplified procedures were developed from empirical evaluations of field observations. Most of the case history data was collected from level to gently sloping terrain underlain by Holocene-age alluvial or fluvial sediment at depths less than 50 feet. Therefore, the simplified procedures are most directly applicable to these site conditions. Caution should be used for evaluating liquefaction potential at depths greater than 50 feet using the simplified procedures. In addition, the simplified procedures estimate the earthquake induced cyclic shear stress ratio based on a coefficient, r_d, that is highly variable at depth as discussed in Section 6.1.2.2.

As an alternative to the use of the r_d factor, to improve the assessment of liquefaction potential, especially at greater depths, if soft or loose soils are present, equivalent linear or nonlinear site specific, one dimensional ground response analyses may be conducted to determine the maximum earthquake induced shear stresses at depth in the Simplified Method. For example, the linear total stress computer programs ProShake (EduPro Civil Systems, 1999) or Shake2000 (Ordoñez, 2000) may be used for this purpose. Consideration should be given to the consistency of site specific analyses with the procedures used to develop the liquefaction resistance curves. A minimum of seven spectrally matched time histories should be used to conduct these analyses to obtain a reasonably stable mean r_d value as a function of depth.

Nonlinear Effective Stress Methods – An alternative to the simplified procedures for evaluating liquefaction susceptibility is to complete a nonlinear, effective stress site response analysis utilizing a computer code capable of modeling pore water pressure generation and dissipation. This is a more rigorous analysis that requires additional parameters to describe the stress-strain behavior and pore pressure generation characteristics of the soil.

The advantages with this method of analysis include the ability to assess liquefaction potential at all depths, including those greater than 50 feet, and the effects of liquefaction and large shear strains on the ground motion. In addition, pore-water redistribution during and following shaking can be modeled, seismically induced deformation can be estimated, and the timing of liquefaction and its effects on ground motion at and below the ground surface can be assessed.

Several one-dimensional non-linear, effective stress analysis programs are available for estimating liquefaction susceptibility at depth, and these methods are being used more frequently by geotechnical designers. However, a great deal of caution needs to be exercised with these programs, as there has been little verification of the ability of these programs to predict liquefaction at depths greater than 50 feet. This limitation is partly the result of the very few well documented sites with pore-water pressure measurements during liquefaction, either at shallow or deep depths, and partly the result of the one-dimensional approximation. For this reason greater reliance must be placed on observed response from laboratory testing or centrifuge modeling when developing the soil and pore pressure models used in the effective stress analysis method. The success of the effective stress model is, therefore, tied in part to the ability of the laboratory or centrifuge modeling to replicate field conditions.

A key issue that can affect the results obtained from nonlinear effective stress analyses is whether or not, or how well, the pore pressure model used addresses soil dilation during shearing. Even if good pore pressure data from laboratory liquefaction testing is available, the models used in some effective stress analysis methods may not be sufficient to adequately model dilation during shearing of liquefied soils. This limitation may result in unconservative predictions of ground response when a deep layer liquefies early during ground shaking. The inability to transfer energy through the liquefied layer could result in "shielding" of upper layers from strong ground shaking, potentially leading to an unconservative site response. See Appendix 6-A for additional considerations regarding modeling accuracies.

Two-dimensional effective stress analysis models can overcome some of these deficiencies, provided that a good soil and pore pressure model is used (e.g., the UBC sand model) – see Appendix 6-A. However, they are even more complex to use and certainly not for novice designers.

It should also be recognized that the results of nonlinear effective stress analyses can be quite sensitive to soil parameters that are often not as well established as those used in equivalent linear analyses. Therefore, it is incumbent upon the user to calibrate the model, evaluate the sensitivity of its results to any uncertain parameters or modeling assumptions, and consider that sensitivity in the interpretation of the results. Therefore, the geotechnical designer shall provide documentation that their model has been validated and calibrated with field data, centrifuge data, and/or extensive sensitivity analyses.

Analysis results from nonlinear effective stress analyses shall not be considered sufficient justification to conclude that the upper 40 to 50 feet of soil will not liquefy as a result of the ground motion dampening effect (i.e., shielding, or loss of energy) caused by deeper liquefiable layers. However, the empirical liquefaction analyses identified in this section may be used to justify that soil layers and lenses within the upper 65 feet of soil will not liquefy. This soil/pore pressure model deficiency for nonlinear effective stress methodologies could be crudely and conservatively addressed by selectively modifying soil parameters and/or turning off the pore pressure generation in given layers to bracket the response.

Due to the highly specialized nature of these more sophisticated liquefaction assessment approaches, approval by the State Geotechnical Engineer is required to use nonlinear effective stress methods for liquefaction evaluation, and independent peer review as described in Section 6.3 shall be conducted.

6.4.2.3 Minimum Factor of Safety Against Liquefaction

Liquefaction hazards assessment and the development of hazard mitigation measures shall be conducted if the factor of safety against liquefaction (Equation 6-9) is less than 1.2 or if the soil is determined to be liquefiable for the return period of interest (e.g., 975 years) using the performance based approach as described by Kramer and Mayfield (2007) and Kramer (2007). Performance based techniques can be accomplished using the WSLIQ software (Kramer, 2007). The hazard level used for this analysis shall be consistent with the hazard level selected for the structure for which the liquefaction analysis is being conducted (typically, a probability of exceedance of 7 percent in 75 years in accordance with the AASHTO Guide Specifications for LRFD Seismic Bridge Design). Liquefaction hazards to be assessed include settlement and related effects, and liquefaction induced instability (e.g., flow failure or lateral spreading), and the effects of liquefaction on foundations.

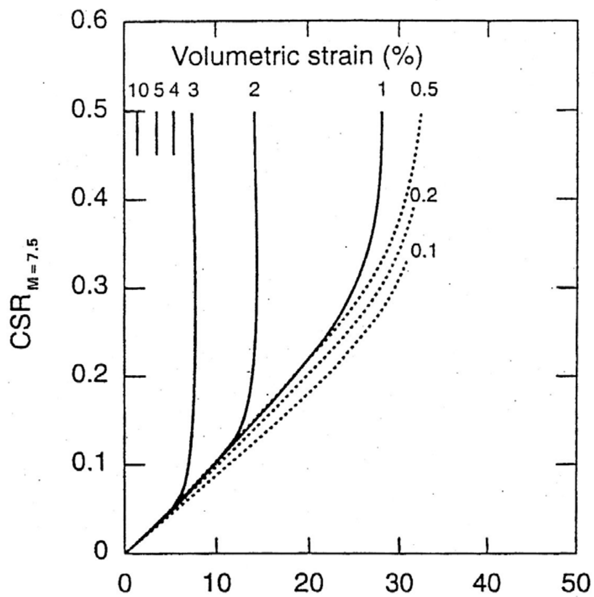
6.4.2.4 Liquefaction Induced Settlement

Both dry and saturated deposits of loose granular soils tend to densify and settle during and/or following earthquake shaking. Settlement of unsaturated granular deposits is discussed in Section 6.5.3. Settlement of saturated granular deposits due to liquefaction shall be estimated using techniques based on the Simplified Procedure, or if nonlinear effective stress models are used to assess liquefaction in accordance with Section 6.5.2.1, such methods may also be used to estimate liquefaction settlement.

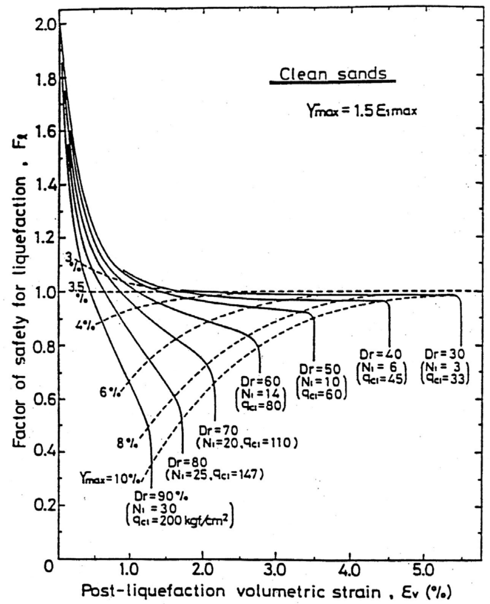
If the Simplified Procedure is used to evaluate liquefaction potential, liquefaction induced ground settlement of saturated granular deposits should be estimated using the procedures by Tokimatsu and Seed (1987) or Ishihara and Yoshimine (1992). The Tokimatsu and Seed (1987) procedure estimates the volumetric strain as a function of earthquake induced CSR and corrected SPT blowcounts. The Ishihara and Yoshimine (1992) procedure estimates the volumetric strain as a function of factor of safety against liquefaction, relative density, and corrected SPT blowcounts or normalized CPT tip resistance. Example charts used to estimate liquefaction induced settlement using the Tokimatsu and Seed procedure and the Ishihara and Yoshimine procedure are presented as Figures 6-12 and 6-13, respectively.

If a more refined analysis of liquefaction induced settlement is needed, laboratory cyclic triaxial shear or cyclic simple shear testing may be used to evaluate the liquefaction induced vertical settlement in lieu of empirical SPT or CPT based criteria, in accordance with Section 6.4.2.6.

The empirically based analyses should be conducted as a baseline evaluation, even when laboratory volumetric strain test results are obtained and used for design, to qualitatively check the reasonableness of the laboratory test results.



Liquefaction Induced Settlement Estimated Using the Tokimatsu and Seed procedure (Tokimatsu and Seed, 1987)
Figure 6-12



Liquefaction Induced Settlement Estimated Using the Ishihara and Yoshimine procedure (Ishihara and Yoshimine, 1992)
Figure 6-13

6.4.2.5 Residual Strength Parameters

Liquefaction induced instability is strongly influenced by the residual strength of the liquefied soil. Instability occurs when the shear stresses required to maintain equilibrium exceed the residual strength of the soil deposit. Evaluation of residual strength of a liquefied soil deposit is one of the most difficult problems in geotechnical practice (Kramer, 1996). A variety of empirical methods are available to estimate the residual strength of liquefied soils. The empirical relationships provided in Figures 6-4 through 6-7 and Table 6-3 shall be used to estimate residual strength of liquefied soil unless soil specific laboratory performance tests are conducted as described below. These procedures for estimating the residual strength of a liquefied soil deposit are based on an empirical relationship between residual undrained shear strength and equivalent clean sand SPT blowcounts or CPT q_{c1n} values, using the results of back-calculation of the apparent shear strengths from case histories of large displacement flow slides. The significant level of uncertainty in these estimates of residual strength should be taken into account in design and evaluation of calculations.

If a more refined analysis of residual strength is needed, laboratory cyclic triaxial shear or cyclic simple shear testing may be used to evaluate the residual strength in lieu of empirical SPT or CPT based criteria, in accordance with Section 6.4.2.6.

The empirically based analyses should be conducted as a baseline evaluation, even when laboratory residual shear strength test results are obtained and used for design, to qualitatively check the reasonableness of the laboratory test results. The final residual shear strength value selected should also consider the shear strain level in the soil that can be tolerated by the structure or slope impacted by the reduced shear strength in the soil (i.e., how much lateral deformation can the structure tolerate?). Numerical modeling techniques may be used to determine the soil shear strain level that results in the maximum tolerable lateral deformation of the structure being designed.

6.4.2.6 Assessment of Liquefaction Potential and Effects Using Laboratory Test Data

If a more refined analysis of liquefaction potential, liquefaction induced settlement, or residual strength of liquefied soil is needed, laboratory cyclic simple shear or cyclic triaxial shear testing may be used in lieu of empirical soil gradation/PI/density (i.e., SPT or CPT based) criteria, if high quality undisturbed samples can be obtained. Laboratory cyclic simple shear or cyclic triaxial shear testing may also be used to evaluate liquefaction susceptibility of and effects on sandy soils from reconstituted soil samples. However, due to the difficulties in creating soil test specimens that are representative of the actual in-situ soil, liquefaction testing of reconstituted soil may be conducted only if approved by the State Geotechnical Engineer. Requests to test reconstituted soil specimens will be evaluated based on how well the proposed specimen preparation procedure mimics the in-situ soil conditions and geologic history.

The number of cycles, and either the cyclic stress ratios (stress-controlled testing) or cyclic shear strain (strain-controlled testing) used during the cyclic testing to liquefy or to attempt to liquefy the soil, should cover the range of the number of cycles and cyclic loading anticipated for the earthquake/ground motion being modeled. Testing to more than one stress or strain ratio should be done to fully capture the range of stress or strain ratios that could occur. Preliminary calculations or computer analyses to estimate the likely cyclic stresses and/or strains anticipated should be conducted to help provide a basis for selection of the cyclic loading levels to be used for the testing. The vertical confining stress should be consistent with the in-situ vertical effective stress estimated at the location where the soil sample was obtained. Therefore K_{o}-consolidation is required in triaxial tests.

Defining liquefaction in these laboratory tests can be somewhat problematic. Theoretically, initial liquefaction is defined as being achieved once the excess pore pressure ratio in the specimen, r_u, is at 100 percent. The assessment of whether or not this has been achieved in the laboratory tested specimen depends on how the pore pressure is measured in the specimen, and the type of soil contained in the specimen. As the soil gets siltier, the greater the possibility that the soil will exhibit fully liquefied behavior (i.e., initial liquefaction) at a measured pore pressure in the specimen of significantly less than 100 percent. A more practical approach that should be used in this case is to use a strain based definition to identify the occurrence of enough cyclic softening to consider the soil to have reached a failure state caused by liquefaction. Typically, if the soil reaches shear strains during cyclic loading of 3 percent or more, the soil, for practical purposes, may be considered to have achieved a state equivalent to initial liquefaction.

Note that if the testing is carried out well beyond initial liquefaction, cyclic triaxial testing is not recommended. In that case, necking of the specimen can occur, making the cyclic triaxial test results not representative of field conditions.

For the purpose of estimating liquefaction induced settlement, after the cyclic shearing is completed, with the vertical stress left on the specimen, the vertical strain is measured as the excess pore pressure is allowed to dissipate.

Note that once initial liquefaction has been achieved, volumetric strains are not just affected by the excess pore pressure generated through cyclic loading, but are also affected by damage to the soil skeleton as cyclic loading continues. Therefore, to obtain a more accurate estimate of post liquefaction settlement, the specimen should be cyclically loaded to the degree anticipated in the field, which may mean continuing cyclic loading after initial liquefaction is achieved.

If the test results are to be used with simplified ground motion modeling techniques (e.g., specification based ground response analysis or total stress site specific ground motion analysis), volumetric strain should be measured only for fully liquefied conditions. If effective stress ground motion analysis (e.g., D-MOD) is conducted, volumetric strain measurements should be conducted at the cyclic stress ratio and number of loading cycles predicted by the effective stress analysis for the earthquake being modeled at the location in the soil profile being modeled, whether or not that combination results in a fully liquefied state. Vertical settlement prediction should be made by using the laboratory test data to develop a relationship between the measured volumetric strain and either the shear strain in the lab test specimens or the excess pore pressure measured in the specimens, and correlating the predicted shear strain or excess pore pressure profile predicted from the effective stress analysis to the laboratory test results to estimate settlement from volumetric strain; however, the shear strain approach is preferred.

To obtain the liquefied residual strength, after the cyclic shearing is completed, the drain lines in the test should be left closed, and the sample sheared statically. If the test results are to be used with simplified ground motion modeling techniques (e.g., specification based ground response analysis or total stress site specific ground motion analysis), residual strength should be measured only for fully liquefied conditions. If effective stress ground motion analysis (e.g., D-MOD) is conducted, residual shear strength testing should be conducted at the cyclic stress ratio and number of loading cycles predicted by the effective stress analysis for the earthquake being modeled at the location in the soil profile being modeled, whether or not that combination results in a fully liquefied state.

See Kramer (1996), Seed. et al. (2003), and Idriss and Boulanger (2008) for additional details and cautions regarding laboratory evaluation of liquefaction potential and its effects.

6.4.2.7 Combining Seismic Inertial Loading with Analyses Using Liquefied Soil Strength

The number of loading cycles required to initiate liquefaction, and hence the time at which liquefaction is triggered, tends to vary with the relative density and composition of the soil (i.e., denser soils require more cycles of loading to cause initial liquefaction). Whether or not the geologic hazards that result from liquefaction (e.g., lateral soil displacement such as flow failure and lateral spreading, reduced soil stiffness and strength, and settlement/downdrag) are concurrent with the strongest portion of the design earthquake ground motion depends on the duration of the motion and the resistance of the soil to liquefaction. For short duration ground motions and/or relatively dense soils, liquefaction may be triggered near the end of shaking. In this case, the structure of interest is unlikely to be subjected to high inertial forces after the soil has reached a liquefied state, and the evaluation of the peak inertial demands on the structure can be essentially decoupled from evaluation of the deformation demands associated with soil liquefaction. However, for long-duration motions (which are usually associated with large magnitude earthquakes such as a subduction zone earthquake as described in GDM Appendix 6-A) and/or very loose soils, liquefaction may be triggered earlier in the motion, and the structure may be subjected to strong shaking while the soil is in a liquefied state.

There is currently no consensus on how to specifically address this issue of timing of seismic acceleration and the development of initial liquefaction and its combined impact on the structure. More rigorous analyses, such as by using nonlinear, effective stress methods, are typically needed to analytically assess this timing issue. Nonlinear, effective stress methods can account for the build-up in pore-water pressure and the degradation of soil stiffness and strength in liquefiable layers. Use of these more rigorous approaches requires considerable skill in terms of selecting model parameters, particularly the pore pressure model. The complexity of the more rigorous approaches is such that approval by the State Geotechnical Engineer to use these approaches is mandatory, and an independent peer reviewer with expertise in nonlinear, effective stress modeling shall be used to review the specific methods used, the development of the input data, how the methods are applied, and the resulting impacts.

While flow failure due to liquefaction is not really affected by inertial forces acting on the soil mass (see Section 6.4.3.1), it is possible that lateral forces on a structure and its foundations due to flow failure may be concurrent with the structure inertial forces if the earthquake duration is long enough (e.g., a subduction zone earthquake). Likewise, for lateral spreading, since seismic inertial forces are acting on the soil during the development of lateral spreading (see Section 6.4.3.1), logically, inertial forces may also be acting on the structure itself concurrently with the development of lateral forces on the structure foundation.

However, there are several factors that may affect the magnitude of the structural inertial loads, if any, acting on the foundation. Brandenberg, et al. (2007a and b) provide examples from centrifuge modeling regarding the combined effect of lateral spreading and seismic structural inertial forces on foundation loads and some considerations for assessing these inertial forces. They found that the total load on the foundation was approximately 40 percent higher on average than the loads caused by the lateral spreading alone. However, the structural column used in this testing did not develop any plastic hinging, which, had it occurred could have resulted in structural inertial loads transmitted to the foundation that could have been as low as one-fourth of what was measured in this testing. Another factor that could affect the potential combination of lateral spreading and structural inertia loads is how close the foundation is to the initiation point (i.e., downslope end) for the lateral spreading, as it takes time for the lateral spread to propagate upslope and develop to its full extent.

The current AASHTO Guide Specifications for seismic design do allow the lateral spreading forces to be decoupled from bridge seismic inertial forces. However, the potential for some combined effect of lateral spread forces with structural inertial loads should be considered if the structure is likely to be subjected to strong shaking while the soil is in a liquefied state, especially if the foundation is located near the toe of the lateral spread or flow failure. In lieu of more sophisticated analyses such as dynamic-stress deformation analyses, for sites where more than 20 percent of the hazard contributing to the peak ground acceleration is from an earthquake with a magnitude of 7.5 or more (i.e., a long duration earthquake where there is potential for strong motion to occur after liquefaction induced lateral ground movement has initiated), it should be assumed that the lateral spreading/flow failure forces on the foundations are combined with 25 percent of the structure inertial forces or the plastic hinge force, whichever is less.

This timing issue also affects liquefaction-induced settlement and downdrag, in that settlement and downdrag do not generally occur until the pore pressures induced by ground shaking begin to dissipate after shaking ceases. Therefore, a de-coupled analysis is appropriate when considering liquefaction downdrag loads.

When considering the effect of liquefaction on the resistance of the soil to structure foundation loads both in the axial (vertical) and lateral (horizontal) directions, two analyses should be conducted to address the timing issue. For sites where liquefaction occurs around structure foundations, structures should be analyzed and designed in two configurations as follows:

- **Nonliquefied Configuration** – The structure should be analyzed and designed, assuming no liquefaction occurs using the ground response spectrum appropriate for the site soil conditions in a nonliquefied state, i.e., using P-Y curves derived from static soil properties.
- **Liquefied Configuration** – The structure as designed in nonliquefied configuration above should be reanalyzed assuming that the layer has liquefied and the liquefied soil provides the appropriate residual resistance for lateral and axial deep foundation response analyses consistent with liquefied soil conditions (i.e., modified P-Y curves, modulus of subgrade reaction, T-Z curves, axial soil frictional resistance). The design spectrum should be the same as that used in nonliquefied configuration. However, this analysis does not include the lateral forces applied to the structure due to liquefaction induced lateral spreading or flow failure, except as noted earlier in this section with regard to large magnitude, long duration earthquakes.

With the approval of the State Bridge and State Geotechnical Engineers, a site-specific response spectrum (for site specific spectral analysis) or nonlinear time histories developed near the ground surface (for nonlinear structural analysis) that accounts for the modifications in spectral content from the liquefying soil may be developed. The modified response spectrum, and associated time histories, resulting from the site-specific analyses at the ground surface shall not be less than two-thirds of the spectrum (i.e., as applied to the spectral ordinates within the entire spectrum) developed using the general procedure described in the AASHTO Guide Specifications for LRFD Bridge Seismic Design, Article 3.4.1, modified by the site coefficients in Article 3.4.2.3. If the soil and bedrock conditions are classified as Site Class F, however, there is no AASHTO general procedure spectrum. In that case, the reduced response spectrum, and associated time histories, that accounts for the effects of liquefaction shall not be less than two-thirds of the site specific response spectrum developed from an equivalent linear total stress analysis (i.e., nonliquefied conditions).

Designing structures for these two configurations should produce conservative results. Typically, the nonliquefied configuration will control the loads applied to the structure and therefore is used to determine the loads within the structure, whereas the liquefied configuration will control the maximum deformations in the structure and is therefore used to design the structure for deformation. In some cases, this approach may be more conservative than necessary, and the designer may use a more refined analysis to assess the combined effect of strong shaking and liquefaction impacts, considering that both effects may not act simultaneously. However, Youd and Carter (2005) suggest that at periods greater than 1 second, it is possible for liquefaction to result in higher spectral accelerations than occur for equivalent nonliquefied cases, all other conditions being equal. Site-specific ground motion response evaluations may be needed to evaluate this potential.

6.4.3 Seismic Slope Instability and Deformation

Slope instability can occur during earthquakes due to inertial effects associated with ground accelerations or due to weakening of the soil induced by the seismic shear strain. Inertial slope instability is caused by temporary exceedance of the soil strength by dynamic earthquake stresses. In general, the soil strength remains unaffected by the earthquake shaking in this case. Weakening instability is the result of soil becoming progressively weaker as shaking occurs such that the shear strength becomes insufficient to maintain a stable slope.

Seismic slope instability analysis is conducted to assess the impact of instability and slope deformation on structures (e.g., bridges, tunnels, and walls). However, in accordance with Section 6.1.2, slopes that do not impact such structures are generally not mitigated for seismic slope instability.

The scope of this section is limited to the assessment of seismic slope instability. The impact of this slope instability on the seismic design of foundations and walls is addressed in Sections 6.5.3 and 6.5.4 for foundations and Sections 15.4.10 through 15.4.12 for walls.

6.4.3.1 Weakening Instability due to Seismic Loading

Weakening instability occurs due to liquefaction or seismic shear strain induced weakening of sensitive fine grained soils. With regard to liquefaction induced weakening instability, earthquake ground motion induces stress and strain in the soil, resulting in pore pressure generation and liquefaction in saturated soil. As the soil strength decreases toward its liquefied residual value, two types of slope instability can occur: flow failure, and lateral spreading. These various types of weakening instability are described in the subsections that follow. How the impact of weakening instability due to liquefaction is addressed for design of structures is specified in Section 6.5.4.

Weakening Instability not Related to Liquefaction – This type of weakening instability depends on the sensitivity of the soil to the shear strain induced by the earthquake ground motion. Sensitive silts and clays fall into this category. For seismic stability design in this scenario, the stability shall be assessed with consideration to the lowest shear strength that is likely to occur during and after shaking. For example, glacially overconsolidated clays will exhibit a significant drop in strength to a residual value as deformation takes place (e.g., see Section 5.13.3). A seismic slope deformation analysis should be conducted to assess this potential. Since it is likely that most of the strong motion will have subsided by the time the deformation required to drop the soil to its residual strength has occurred, the seismic slope stability analysis typically does not need to include inertial forces due to seismic acceleration when seismic stability is evaluated using the residual shear strength of the sensitive silt or clay soil. However, if the deformation analysis shows that enough deformation to drop the soil shear strength to near its residual value can occur before strong motion ceases, then the slope stability analysis shall include seismic inertial forces in combination with the residual shear strength. For silts and clays with low to moderate sensitivity, a strength reduction of 10 to 15 percent to account for cyclic degradation is reasonable for earthquake magnitudes of 7.0 or more (Kavazanjian, et al. 2011). For clays with high sensitivity, cyclic shear strength tests should be conducted to assess the rate of strength reduction.

For this type of weakening instability, the minimum level of safety specified in Section 6.4.3.2 shall be met, considering the weakened state of the soil during and after shaking. Assessment of the impact of this type of instability on structures is addressed in Section 6.5.3 for foundations and Sections 15.4.10 through 15.4.12 for walls.

Liquefaction Induced Flow Failure – Liquefaction can lead to catastrophic flow failures driven by static shearing stresses that lead to large deformation or flow. Such failures are similar to debris flows and are characterized by sudden initiation, rapid failure, and the large distances over which the failed materials move (Kramer, 1996). Flow failures typically occur near the end of strong shaking or shortly after shaking. However, delayed flow failures caused by post-earthquake redistribution of pore water pressures can occur—particularly if liquefiable soils are capped by relatively impermeable layers.

The potential for liquefaction induced flow failures should be evaluated using conventional limit equilibrium slope stability analyses (see Section 6.4.3), using residual undrained shear strength parameters for the liquefied soil, and decoupling the analysis from all seismic inertial forces (i.e., performed with k_h and k_v equal to zero). If the limit equilibrium factor of safety, FS, is less than 1.05, flow failure shall be considered likely. In these instances, the magnitude of deformation is usually too large to be acceptable for design of bridges or structures, and some form of mitigation will likely be needed. The exception is where the liquefied material and any overlying crust flow past the structure and the structure and its foundation system can resist the imposed loads. Where the factor of safety for this decoupled analysis is greater than 1.05 for liquefied conditions, deformation and stability shall be evaluated using a lateral spreading analysis (see the subsection “Lateral Spreading,” especially regarding cautions in conducting these types of analyses).

Residual strength values to be used in the flow failure analysis may be determined from empirical relationships (See Section 6.4.2.5) or from laboratory test results. If laboratory test results are used to assess the residual strength of the soil that is predicted to liquefy and potentially cause a flow failure, the shearing resistance may be very strain dependent. As a default, the laboratory mobilized residual strength value used should be picked at a strain of 2 percent, assuming the residual strength value is determined from laboratory testing as described in Section 6.4.2.6. A higher strain value may be used for this purpose, subject to the approval of the State Geotechnical Engineer and State Bridge Engineer, if it is known that the affected structure can tolerate a relatively large lateral deformation without collapse. Alternatively, numerical modeling may be conducted to develop the relationship between soil shear strain and slope deformation, picking a mobilized residual strength value that corresponds to the maximum deformation that the affected structure can tolerate.

With regard to flow failure prediction, even though there is a possibility that seismic inertial forces may be concurrent with the liquefied conditions (i.e., in long duration earthquakes), it is the static stresses that drive the flow failure and the deformations that result from the failure. The dynamic stresses present have little impact on this type of slope failure. Therefore, slope stability analyses conducted to assess the potential for flow failure resulting from liquefaction, and to estimate the forces that are applied to the foundation due to the movement of the soil mass into the structure, should be conducted without seismic inertial forces (i.e., k_h and k_v acting on the soil mass are set equal to zero).

Lateral Spreading – In contrast to flow failures, lateral spreading can occur when the shear strength of the liquefied soil is incrementally exceeded by the inertial forces induced during an earthquake or when soil stiffness degrades sufficiently to produce substantial permanent strain in the soil. The result of lateral spreading is typically horizontal movement of non-liquefied soils located above liquefied soils, in addition to the liquefied soils themselves. Lateral spreading analysis is by definition a coupled analysis (i.e., directly considers the effect of seismic acceleration), in contrast to a flow failure analysis, which is a decoupled seismic stability analysis.

If the factor of safety for slope stability from the flow failure analysis, assuming residual strengths in all layers expected to experience liquefied conditions, is 1.05 or greater, a lateral spreading/deformation analysis shall be conducted. If the liquefied layer(s) are discontinuous, the slope factor of safety may be high enough that lateral spreading does not need to be considered. This analysis also does not need to be conducted if the depth below the natural ground surface to the upper boundary of the liquefied layers is greater than 50 ft.

The potential for liquefaction induced lateral spreading on gently sloping sites or where the site is located near a free face shall be evaluated using one or more of the following empirical relationships:

- Youd et al. (2002)
- Kramer and Baska (2007)
- Zhang et al. (2004)

These procedures use empirical relationships based on case histories of lateral spreading and/or laboratory cyclic shear test results. Input into these models include earthquake magnitude, source-to-site distance, site geometry/slope, cumulative thickness of saturated soil layers and their characteristics (e.g., SPT N values, average fines content, average grain size). These empirical procedures provide a useful approximation of the potential magnitude of deformation that is calibrated against lateral spreading deformations observed in actual earthquakes. In addition to the cited references for each method, see Kramer (2007) for details on how to carry out these methods. Kramer (2007) provides recommendations on the use of these methods which should be followed.

More complex analyses such as the Newmark time history analysis and dynamic stress deformation models, such as provided in two-dimensional, nonlinear effective stress computer programs (e.g., PLAXIS and FLAC), may also be used to estimate lateral spreading deformations. However, these analysis procedures have not been calibrated to observed performance with regard to lateral movements caused by liquefaction, and there are many complexities with regard to development of input parameters and application of the method to realistic conditions.

The Newmark time history analysis procedure is described in Anderson, et al. (2008) and Kavezanjian, et al. (2011). If a Newmark time history analysis is conducted to obtain an estimate of lateral spreading displacement, the number of cycles to initiate liquefaction for the time histories selected for analysis needs to be considered when selecting a yield acceleration to apply to the various portions of the time history. Initially, the yield acceleration will be high, as the soil will not have liquefied (i.e., non-liquefied soil strength parameters should be used to determine the yield acceleration). As the soil excess pore pressure begins to build up with additional loading cycles, the yield acceleration will begin to decrease. Once initial liquefaction or cyclic softening occurs, the residual strength is then used to determine the yield acceleration. Note that if the yield acceleration applied to the entire acceleration time history is based on residual strength consistent with liquefied conditions, the estimated lateral deformation will likely be overly conservative. To address this issue, an effective stress ground motion analysis (e.g., D-MOD) should be conducted to estimate the build up of pore pressure and the development of liquefaction as the earthquake shaking continues to obtain an improved estimate of the drop in soil shear strength and yield acceleration as a function of time.

Simplified charts based on Newmark-type analyses shall not be used for estimating deformation resulting from lateral spreading. These simplified Newmark type analyses have some empirical basis built in with regard to estimation of deformation. However, they are not directly applicable to lateral spreading, as they were not developed for soil that weakens during earthquake shaking, as is the case for soil liquefaction.

If the more rigorous approaches are used, the empirically based analyses shall still be conducted to provide a baseline of comparison, to qualitatively check the reasonableness of the estimates from the more rigorous procedures, using the more rigorous approaches to evaluate the effect of various input parameters on deformation. See Youd, et al. (2002), Kramer (1996, 2007), Seed, et al. (2003) and Dickenson, et al. (2002) for additional background on the assessment of slope deformations resulting from lateral spreading.

A related issue is how far away the free face must be before lateral spreading need not be considered. Lateral spreading has been observed up to about 1,000 ft from the free face in past earthquakes (Youd, et al., 2002). Available case history data also indicate that deformations at L/H ratios greater than 20, where L is the distance from the free face or channel and H is the height of the free face of channel slope, are typically reduced to less than 20 percent of the lateral deformation at the free face (Idriss and Boulanger, 2008). Detailed analysis of the Youd, et al. database indicates that only two of 97 cases had observable lateral spreading deformation at L/H ratios as large as 50 to 70. If lateral spreading calculations using these empirical procedures are conducted at distances greater than 1,000 ft from the free face or L/H ratios greater than 20, additional evaluation of lateral spreading deformation using more complex or rigorous approaches should also be conducted.

6.4.3.2 Slope Instability Due to Inertial Effects

Even if the soil does not weaken as earthquake shaking progresses, instability can still occur due to the additional inertial forces acting on the soil mass during shaking. Inertial slope instability is caused by temporary exceedance of the soil strength by dynamic earthquake stresses.

Pseudo-static slope stability analyses shall be used to evaluate the seismic stability of slopes and embankments. The pseudo-static analysis consists of conventional limit equilibrium static slope stability analysis as described in Chapter 7 completed with horizontal and vertical pseudo-static acceleration coefficients (k_h and k_v) that act upon the critical failure mass. Kramer (1996) provides a detailed summary of pseudo-static analysis procedures.

For earthquake induced slope instability, with or without soil strength loss resulting from deformation induced by earthquake shaking, the target factor of safety for the pseudo-static slope stability analysis is 1.1. When bridge foundations or retaining walls are involved, the LRFD approach shall be used, in which case a resistance factor of 0.9 shall be used for slope stability. Note that available slope stability programs produce a single factor of safety, FS. The specified resistance factor of 0.9 for slope stability is essentially the inverse of the FS that should be targeted in the slope stability program, which in this case is 1.1, making 0.9 the maximum resistance factor to be obtained when conducting pseudo-static slope stability analyses. If liquefaction effects dominate the stability of the slope and its deformation response (i.e., flow failure or lateral spreading occur), the procedures provided in Section 6.4.3.1 shall be used.

Unless a more detailed deformation analysis is conducted, a default horizontal pseudo-static coefficient, kh, of 0.5As and a vertical pseudo-static coefficient, kv, equal to zero shall be used when seismic (i.e., pseudo-static) stability of slopes is evaluated, not considering liquefaction. This value of kh assumes that limited deformation of the slope during earthquake shaking is acceptable (i.e., 1 to 2 inches) and considers some wave scattering effects.

Due to the fact that the soil is treated as a rigid body in pseudo-static limit equilibrium analyses, and that the seismic inertial force is proportional to the square of the failure surface radius whereas the resistance is proportional to just the radius, the tendency is for the failure surface to move deeper and farther uphill relative to the static failure surface when seismic inertial loading is added. That is, the pseudo-static analysis assumes that the k_h value applies uniformly to the entire failure mass regardless of how big the failure mass becomes. Since the soil mass is far from rigid, this can be an overly conservative assumption, in that the average value of k_h for the failure mass will likely decrease relative to the input value of k_h used for the stability assessment due to wave scattering effects.

The default value of k_h should be increased to near 1.0 A_s if a structure within or at the toe of the potentially unstable slope cannot tolerate any deformation. If slope movement can be tolerated, a reduced value of k_h applied to the slope in the stability analysis may be used by accounting for both wave scattering (i.e., height) effects and deformation effects through a more detailed deformation based analysis. See Anderson, et al. (2008) and Kavezanjiam, et al. (2011) for the specific procedures to do this.

Deformation analyses should be employed where an estimate of the magnitude of seismically induced slope deformation is required, or to reduce kh for pseudo-static slope stability analysis below the default value of 0.5As as described above. Acceptable methods of estimating the magnitude of seismically induced slope deformation are as provided in Anderson, et al. (2008) and Kavezanjian, et al. (2011), and include Newmark sliding block (time history) analysis as well as simplified procedures developed from Newmark analyses and numerical modeling. For global and sliding seismic stability analyses for walls, the procedures provided in the AASHTO LRFD Bridge Design Specifications should be used (specifically see Articles 11.6.5.2, 11.6.5.3, and Appendix A11).

6.4.4 Settlement of Dry Sand

Seismically induced settlement of unsaturated granular soils (dry sands) is well documented. Factors that affect the magnitude of settlement include the density and thickness of the soil deposit and the magnitude of seismic loading. The most common means of estimating the magnitude of dry sand settlement are through empirical relationships based on procedures similar to the Simplified Procedure for evaluating liquefaction susceptibility. The procedures provided by Tokimatsu and Seed (1987) for dry sand settlement should be used. The Tokimatsu and Seed approach estimates the volumetric strain as a function of cyclic shear strain and relative density or normalized SPT N values. The step by step procedure is provided in FHWA Geotechnical Engineering Circular No. 3 (Kavazanjian, et al., 2011).

Since settlement of dry sand will occur during earthquake shaking with downdrag forces likely to develop before the strongest shaking occurs, the axial forces caused by this phenomenon should be combined with the full spectral ground motion applied to the structure.

6.5 Input for Structural Design

6.5.1 Foundation Springs

Structural dynamic response analyses incorporate the foundation stiffness into the dynamic model of the structure to capture the effects of soil structure interaction. The foundation stiffness is typically represented as a system of equivalent springs using a foundation stiffness matrix. The typical foundation stiffness matrix incorporates a set of six primary springs to describe stiffness with respect to three translational and three rotational components of motion. Springs that describe the coupling of horizontal translation and rocking modes of deformation may also be used.

The primary parameters for calculating the individual spring stiffness values are the foundation type (shallow spread footings or deep foundations), foundation geometry, dynamic soil shear modulus, and Poisson's Ratio.

6.5.1.1 Shallow Foundations

For evaluating shallow foundation springs, the WSDOT Bridge and Structures Office requires values for the dynamic shear modulus, G, Poisson's ratio, and the unit weight of the foundation soils. The maximum, or low-strain, shear modulus G_0 can be estimated using index properties and the correlations presented in Table 6-2. Alternatively, the maximum shear modulus can be calculated using Equation 6-10 below, if the shear wave velocity is known:

$$G_0 = \frac{\gamma}{g}(V_s)^2 \tag{6-10}$$

Where:

G_0 = low strain, maximum dynamic shear modulus
γ = soil unit weight
V_s = shear wave velocity
g = acceleration due to gravity

The maximum dynamic shear modulus is associated with small shear strains (typically less than 0.0001 percent). As the seismic ground motion level increases, the shear strain level increases, and dynamic shear modulus decreases. If the specification based general procedure described in Section 6.3 is used, the effective shear modulus, G, should be calculated in accordance with Table 4-7 in FEMA 356 (ASCE 2000), reproduced below as Table 6-3 for convenience. Note that $S_{XS}/2.5$ in the table is essentially equivalent to A_s (i.e., $PGAxF_{pga}$). This table reflects the dependence of G on both the shear strain induced by the ground motion and on the soil type (i.e., G drops off more rapidly as shear strain increases for softer or looser soils).

This table must be used with some caution, particularly where abrupt variations in soil profile occur below the base of the foundation. If the soil conditions within two foundation widths (vertically) of the bottom of the foundation depart significantly from the average conditions identified for the specific site class, a more rigorous method may be required. The more rigorous method may involve conducting one-dimensional equivalent linear ground response analyses using a program such as SHAKE to estimate the average effective shear strains within the zone affecting foundation response.

	Effective Peak Acceleration, $S_{XS}/2.5$			
Site Class	$S_{XS}/2.5 = 0$	$S_{XS}/2.5 = 0.1$	$S_{XS}/2.5 = 0.4$	$S_{XS}/2.5 = 0.8$
A	1.00	1.00	1.00	1.00
B	1.00	1.00	0.95	0.90
C	1.00	0.95	0.75	0.60
D	1.00	0.90	0.50	0.10
E	1.00	0.60	0.05	*
F	*	*	*	*

Notes: Use straight-line interpolation for intermediate values of $S_{xs}/2.5$.

* Site-specific geotechnical investigation and dynamic site response analyses shall be performed.

Effective Shear Modulus Ratio (G/G_0)
Table 6-3 (After ASCE 2000)

Alternatively, site specific measurements of shear modulus may be obtained. Measured values of shear modulus may be obtained from laboratory tests, such as the cyclic triaxial, cyclic simple shear, or resonant column tests, or they may be obtained from in-situ field testing. If the specification based general procedure is used to estimate ground motion response, the laboratory or in-situ field test results may be used to calculate G_0. Then the table from FEMA 356 (ASCE, 2000) reproduced above can be used to determine G/G_0. However, caution should be exercised when using laboratory testing to obtain this parameter due to the strong dependency of this parameter on sample disturbance. Furthermore, the low-strain modulus developed from lab test should be adjusted for soil age if the footing is placed on native soil. The age adjustment can result in an increase in the lab modulus by a factor of 1.5 or more, depending on the quality of the laboratory sample and the age of the native soil deposit. The age adjustment is not required if engineered fill will be located within two foundation widths of the footing base. The preferred approach is to measure the shear wave velocity, V_s, through in-situ testing in the field, to obtain G_0.

If a detailed site specific ground response analysis is conducted, either Figures 6-1 and 6-2 may be used to estimate G in consideration of the shear strains predicted through the site specific analysis (the effective shear strain, equal to 65 percent of the peak shear strain, should be used for this analysis), or laboratory test results may be used to determine the relationship between G/G_0 and shear strain.

Poisson's Ratio, v, should be estimated based on soil type, relative density/consistency of the soils, and correlation charts such as those presented in Chapter 5 or in the textbook, *Foundation Analysis and Design* (Bowles, 1996). Poisson's Ratio may also be obtained from field measurements of p- and s-wave velocities.

Once G and v are determined, the foundation stiffness values should be calculated as shown in FEMA 356 (ASCE, 2000).

6.5.1.2 Deep Foundations

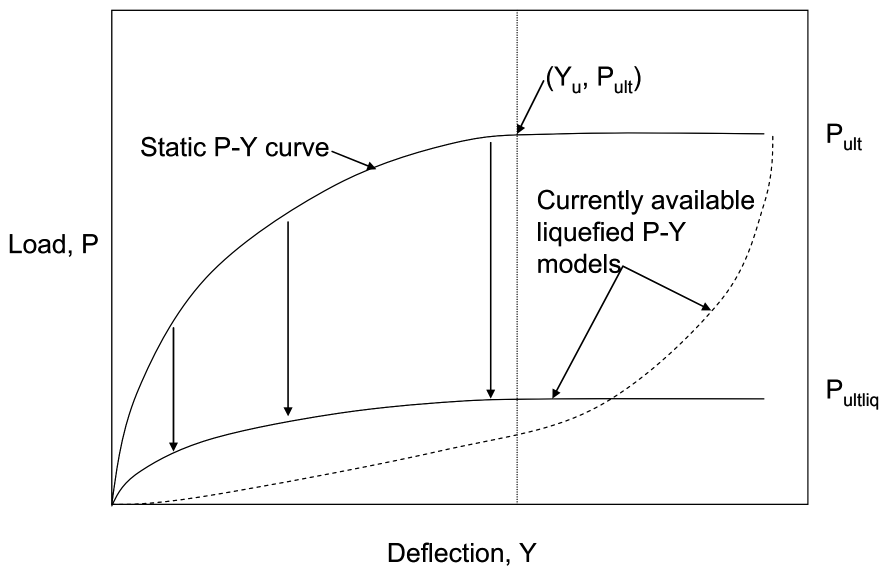
Lateral soil springs for deep foundations shall be determined in accordance with Chapter 8.

Existing deep foundation lateral load analysis computer programs, and the methodologies upon which they are based, do provide approaches for modeling the response of liquefied soil to lateral deep foundation loads. These approaches, and their limitations, are as follows:

- The computer program L-Pile Plus version 5.0 (Reese, et al., 2004) includes P-Y curves for liquefied sands that are intended to more accurately model the strain hardening behavior observed from liquefied soils. However, that particular model tends to predict too soft a response and is very limited regarding the conditions it can consider.
- A similar approach can be used with the DFSAP computer program (Singh, et al., 2006), which is based on the Strain Wedge Model (see Chapter 8 for additional information on the strain wedge model). DFSAP has an option built in to the program for estimating liquefied lateral stiffness parameters and lateral spread loads on a single pile or shaft. However, the accuracy of the liquefied soil stiffness and predicted lateral spread loads using strain wedge theory, in particular the DFSAP program, has not been well established (see Discussion and Closure of "Response of 0.6 m Cast-in-Steel-Shell Pile in Liquefied Soil under Lateral Loading by Thomas J. Weaver, Scott A. Ashford, and Kyle M. Rollins, 2005, ASCE, Vol. 131, No. 1, pp. 94-102", ASCE 2006, pp. 1238-1241.

Weaver, et al. (2005) and Rollins, et al. (2005) provided a comparison between the various methods of developing P-Y parameters for liquefied soil and the measured lateral load response of a full scale pile foundation in liquefied soil (i.e., liquefied using blast loading). They concluded that none of the simplified methods that utilize adjusted soil parameters applied to static P-Y clay or sand models accurately predicted the measured lateral pile response to load due to the difference in curve shape for static versus liquefied conditions (i.e., convex, or strain softening, versus concave, or strain hardening, shape, respectively). Furthermore, in fully liquefied sand, there appears to be virtually no lateral soil resistance for the first 1 to 2 inches of lateral movement, based on their observations. However, available static P-Y curve models reduced adequately to account for the loss of strength caused by liquefaction, such as a p-multiplier approach, could provide an approximate prediction of the measured P-Y response. Rollins, et al. (2005) also concluded that group reduction factors for lateral pile resistance can be neglected in fully liquefied sand (i.e., $R_u > 0.9$), and that group reduction effects reestablish quickly as pore pressures dissipate. Furthermore, they observed that group reduction factors were applicable in soil that is not fully liquefied.

If the demand on the foundation during earthquake shaking is not very high, but the soil still liquefies, the convex-up shape of the static P-Y curves may also result in an under-prediction of the deformation for liquefied conditions. Assuming that the static (i.e., convex up) P-Y curve is reduced to liquefied conditions using a p-multiplier or similar approach, relatively low seismic foundation loading may not be great enough to get past the early steeper portion of the liquefied soil P-Y curve and on to the flatter portion of the curve where deformation can increase fairly readily in response to the applied load. This could possibly result in an unconservative estimate of lateral foundation deformation for the liquefied condition as well.



Conceptual P-Y Curve Model For Liquefied Conditions
Figure 6-15

The liquefied P-Y curves should be estimated using one of two options. These options are as follows:

1. Use the static sand model and the P-multiplier approach as provided by Brandenberg, et al. (2007b) and Boulanger, et al. (2003) to reduce P_{ult} calculated for the static P-Y curve to a liquefied value. This approach is illustrated conceptually in Figure 6-15. The p-multiplier, m_p, used to reduce the static curve to a liquefied curve is determined from Figure 6-16. The p-multiplier approach is primarily applicable to use in L-Pile or a similar computer program.
2. Use the static sand model, using the residual strength and the overburden stress at the depth at which the residual strength was calculated to estimate a reduced soil friction value. The reduced soil friction angle is calculated using the inverse tangent of the residual undrained shear strength divided by the effective vertical stress at which the residual shear strength was determined or measured , i.e., $\phi_{reduced} = \tan^{-1}(S_r/\sigma'_{vo})$, where S_r is the residual shear strength and σ'_{v0} is the effective vertical stress. Use the reduced soil friction angle (i.e., for liquefied conditions) to generate

the liquefied P-Y curves. This approach is applicable to both the strain wedge (DFSAP computer program) and L-Pile computer program methods. The entire static curve needs to be reduced from static to liquefied conditions, as illustrated in Figure 6-15. Parameters representing the initial stiffness of the P-Y curves may also need to be reduced in a manner similar to the reduction applied to obtain P_{ultliq}. For the DFSAP computer program, this adjustment to liquefied conditions would be applied to E_{50}. For L-Pile, this adjustment would be applied to the modulus of subgrade reaction, k. For both approaches, the soil unit weight should not be adjusted for liquefied conditions.

If the first option is selected, the p-multiplier values should be selected from Figure 6-16, Brandenberg (2005) curve. If the second option is selected, residual (i.e., liquefied) soil shear strength should be estimated using a method that considers the effect of overburden stress (e.g., Figures 6-5 through 6-7).

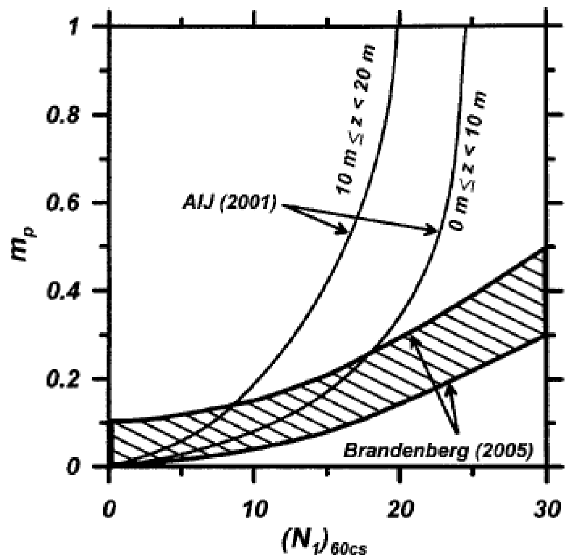
The p-multiplier values represent fully liquefied conditions. Note that for partially liquefied conditions, the p-multipliers can be increased from those values shown in the table, linearly interpolating between the tabulated values and 1.0 based on the pore pressure ratio, r_u, achieved during shaking (e.g., Dobry, et al., 1995). For Option 2, a partially liquefied shear strength may be used to calculate the reduced friction angle and P_{ultliq}.

If Option 2 is selected and the residual shear strengths are based on laboratory test data, the strain at which the liquefied shear strength is determined may be a key factor, as the residual strength can be highly strain dependent. If empirical correlations are used to estimate the residual shear strength, the soil conditions those empirical residual shear strengths represent relative to the soil conditions at the site in question should be considered when picking residual shear strength values to use in the P-Y curve development.

In general, if the liquefied P-Y curves result in foundation lateral deformations that are less than approximately 2 inches near the foundation top for the liquefied state, the liquefied P-Y curves should be further evaluated to make sure the parameters selected to create the liquefied P-Y curves represent realistic behavior in liquefied soil.

For pile or shaft groups, for fully liquefied conditions, P-Y curve reduction factors to account for foundation element spacing and location within the group may be set at 1.0. For partially liquefied conditions, the group reduction factors shall be consistent with the group reduction factors used for static loading.

For other deep foundation soil springs, i.e., axial (t-z) and tip (q-z), the methodology described above for P-Y curves should also be used to assess the effects of liquefaction on t-z and q-z curves.



Recommended P-Multipliers for Liquefied Soil
(After Brandenberg, et al., 2007b)
Figure 6-16

6.5.2 Earthquake Induced Earth Pressures on Retaining Structures

The Mononobe-Okabe pseudo-static method shall be used to estimate the seismic lateral earth pressure, as specified in Chapter 15. Alternatively, slope stability analyses may be used to calculate seismic earth pressures using the same k_h value that would be used for Mononobe-Okabe analysis, and should be used for situations in which Mononobe-Okabe analysis is not applicable (see Chapter 15). Due to the high rate of loading that occurs during seismic loading, the use of undrained strength parameters in the slope stability analysis should be considered for soils other than clean coarse grained sands and gravels and sensitive silts and clays that could weaken during shaking.

6.5.3 Earthquake Induced Slope Failure Loads on Structures

If the pseudo-static slope stability analysis conducted in accordance with Section 6.4.3.2 results in a safety factor of less than 1.1 (or a resistance factor that is greater than 0.9 for LRFD), the slope shall be stabilized or the structure shall be designed to resist the slide force. For earthquake induced slope failure loads applied to structure foundations and bridge abutments, the lateral force applied to the structure is the force needed to restore the slope level of safety to the required minimum value. But this assumes that the structure and its foundations can be designed to resist the slide loading and the deformation required to mobilize the necessary resistance. If the structural designer determines that the structure cannot resist the slide load and the deformation it causes, then the slope shall be stabilized to restore its level of safety to the required minimum values (i.e., FS > 1.1 or a resistance factor of 0.9 or less). See Section 8.6.5.2 for procedures to estimate the slide force on a foundation element.

Landslides and slope instability induced by seismic loading not induced by liquefaction should be considered to be concurrent with the structure seismic loading. Therefore, the structure seismic loads and the seismically induced landslide/slope instability forces should be coupled. Also note that when foundation elements are located within a mass that becomes unstable during seismic loading, the potential for soil below the foundation to move away from the foundation, thereby reducing its lateral support, shall be considered.

6.5.4 Lateral Spread and Flow Failure Loads on Structures Due to Liquefaction

Short of doing a rigorous dynamic stress-deformation analysis, there are two different approaches to estimate the lateral spread/flow failure induced load on deep foundations systems— displacement based approach and a force based approach. Displacement based approaches are more prevalent in the United States. A force based approach has been specified in the Japanese codes and is based on case histories from past earthquakes, especially the pile foundation failures observed during the 1995 Kobe earthquake. Overviews of both approaches are presented below.

6.5.4.1 Displacement Based Approach

The recommended displacement based approach for evaluating the impact of liquefaction induced lateral spreading and flow failure loads on deep foundation systems is presented in, *Guidelines on Foundation Loading and Deformation Due to Liquefaction Induced Lateral Spreading* (Caltrans 2012) located at

www.dot.ca.gov/research/structures/peer_lifeline_program/docs/guidelines_on_foundation_loading_jan2012.pdf

Additional background on the Caltrans procedure is provided in Ashford, et al. (2011). This procedure provides methods to evaluate deep foundation systems that partially restrain the ground movement caused by lateral spreading/flow failure, and those foundation systems in which the ground can freely flow around them.

To be consistent with the design provisions in this GDM, the Caltrans procedure shall be modified as follows:

- Assessment of liquefaction potential shall be in accordance with Section 6.4.2.2.
- Determination of liquefied residual strengths shall be in accordance with Section 6.4.2.5.
- Lateral spread deformations shall be estimated using methods provided in Section 6.4.3.1.
- The combination of seismic inertial loading and kinematic loading from lateral spreading or flow failure shall be in accordance with Section 6.4.2.7.
- Deep foundation springs shall be determined using Section 6.5.1.2.

6.5.4.2 Force Based Approaches

A force based approach to assess lateral spreading induced loads on deep foundations is specified in the Japanese codes. The method is based on back-calculations from pile foundation failures caused by lateral spreading (see Yokoyama, et al., 1997 for background on this method) The pressures on pile foundations are simply specified as follows:

- The liquefied soil exerts a pressure equal to 30 percent of the total overburden pressure (lateral earth pressure coefficient of 0.30 applied to the total vertical stress).

- Non-liquefied crustal layers exert full passive pressure on the foundation system.

Data from simulated earthquake loading of model piles in liquefiable sands in centrifuge tests indicate that the Japanese Force Method is an adequate design method (Finn, et al., 2004) and therefore may be used to estimate lateral spreading and flow failure forces on bridge foundations.

6.5.4.3 Dynamic Stress-Deformation Approaches

Seismically induced slope deformations and their effect on foundations can be estimated through a variety of dynamic stress-deformation computer models such as PLAXIS, DYNAFLOW, FLAC, and OpenSees. These methods can account for varying geometry, soil behavior, and pore pressure response during seismic loading and the impact of these deformations on foundation loading. The accuracy of these models is highly dependent upon the quality of the input parameters and the level of model validation performed by the user for similar applications.

In general, dynamic stress deformation models should not be used for routine design due to their complexity, and due to the sensitivity of deformation estimates to the constitutive model selected and the accuracy of the input parameters. If dynamic stress deformation models are used, they should be validated for the particular application. Dynamic stress-deformation models shall not be used for design on WSDOT projects without the approval of the State Geotechnical Engineer. Furthermore, independent peer review as specified in Section 6.3 shall be conducted.

6.5.5 Downdrag Loads on Structures Due to Liquefaction

Downdrag loads on foundations shall be determined in accordance with Article 3.11.8 of the AASHTO LRFD Bridge Design Specifications, GDM Chapter 8, and as specified herein.

The AASHTO LRFD Bridge Design Specifications, Article 3.11.8, recommend the use of the nonliquefied skin friction in the layers within and above the liquefied zone that do not liquefy, and a skin friction value as low as the residual strength within the soil layers that do liquefy, to calculate downdrag loads for the extreme event limit state. In general, vertical settlement and downdrag cannot occur until the pore pressures generated by the earthquake ground motion begin to dissipate after the earthquake shaking ceases. At this point, the liquefied soil strength will be near its minimum residual strength. At some point after the pore pressures begin to dissipate, and after some liquefaction settlement has already occurred, the soil strength will begin to increase from its minimum residual value. Therefore, the actual shear strength of soil along the sides of the foundation elements in the liquefied zone(s) may be higher than the residual shear strength corresponding to fully liquefied conditions, but still significantly lower than the nonliquefied soil shear strength. Very little guidance on the selection of soil shear strength to calculate downdrag loads due to liquefaction is available; therefore some engineering judgment may be required to select a soil strength to calculate downdrag loads due to liquefaction.

6.5.6 Mitigation Alternatives

The two basic options to mitigate the lateral spread induced loads on the foundation system are to design the structure to accommodate the loads or improve the ground such that the hazard does not occur.

Structural Options (design to accommodate imposed loads) – See Sections 6.5.4.1 (displacement based approach) and 6.5.4.2 (force based approach) for more details on the specific analysis procedures. Once the forces and/or displacements caused by the lateral spreading have been estimated, the structural designer should use those estimates to analyze the effect of those forces and/or displacements will have on the structure to determine if designing the structure to tolerate the deformation and/or lateral loading is structurally feasible and economical.

Ground Improvement – It is often cost prohibitive to design the bridge foundation system to resist the loads and displacements imposed by liquefaction induced lateral loads, especially if the depth of liquefaction extends more than about 20 feet below the ground surface and if a non-liquefied crust is part of the failure mass. Ground improvement to mitigate the liquefaction hazard is the likely alternative if it is not practical to design the foundation system to accommodate the lateral loads.

The primary ground improvement techniques to mitigate liquefaction fall into three general categories, namely densification, altering the soil composition, and enhanced drainage. A general discussion regarding these ground improvement approaches is provided below. Chapter 11, Ground Improvement, should be reviewed for a more detailed discussion regarding the use of these techniques.

Densification and Reinforcement – Ground improvement by densification consists of sufficiently compacting the soil such that it is no longer susceptible to liquefaction during a design seismic event. Densification techniques include vibro-compaction, vibro-flotation, vibro-replacement (stone columns), deep dynamic compaction, blasting, and compaction grouting. Vibro-replacement and compaction grouting also reinforce the soil by creating columns of stone and grout, respectively. The primary parameters for selection include grain size distribution of the soils being improved, depth to groundwater, depth of improvement required, proximity to settlement/vibration sensitive infrastructure, and access constraints.

For those soils in which densification techniques may not be fully effective to densify the soil adequately to prevent liquefaction, the reinforcement aspect of those methods may still be used when estimating composite shear strength and settlement characteristics of the improved soil volume. See Chapter 11 for details and references that should be consulted for guidance in establishing composite properties for the improved soil volume.

If the soil is reinforced with vertical structural inclusions (e.g., drilled shafts, driven piles, but not including the structure foundation elements) but not adequately densified to prevent the soil from liquefying, the design of the ground improvement method should consider both the shear and moment resistance of the reinforcement elements. For vertical inclusions that are typically not intended to have significant bending resistance (e.g., stone columns, compaction grout columns, etc.), the requirement to resist the potential bending stresses caused by lateral ground movement may be waived, considering only shear resistance of the improved soil plus inclusions, if all three of the following conditions are met:

- The width and depth of the improved soil volume are equal to or greater than the requirements provided in Figure 6-18,
- three or more rows of reinforcement elements to resist the forces contributing to slope failure or lateral spreading are used, and
- the reinforcement elements are spaced center-to-center at less than 5 times the reinforcement element diameter or 10 feet, whichever is less.

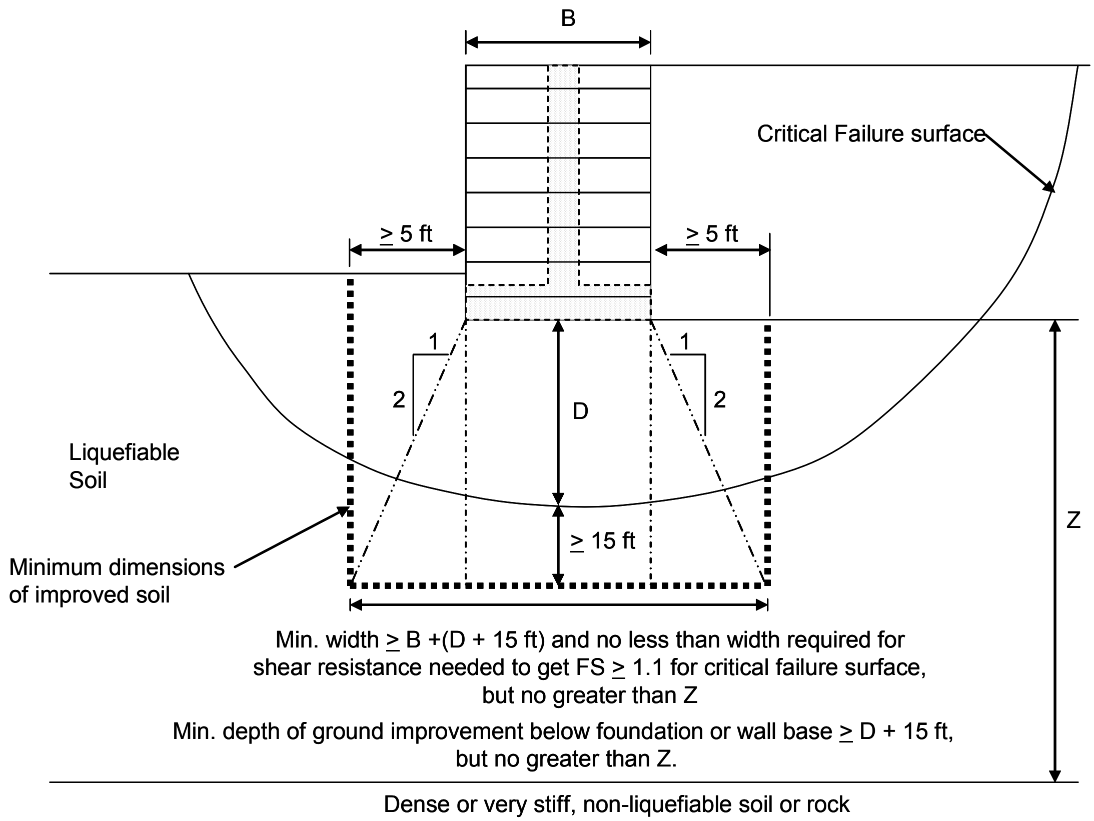
Figure 6-18 shows the improved soil volume as centered around the wall base or foundation. However, it is acceptable to shift the soil improvement volume to work around site constraints, provided that the edge of the improved soil volume is located at least 5 feet outside of the wall or foundation being protected. Greater than 5 feet may be needed to insure stability of the foundation, prevent severe differential settlement due to the liquefaction, and to account for any pore pressure redistribution that may occur during or after liquefaction initiation.

For the case where a "collar" of improved soil is placed outside and around the foundation, bridge abutment or other structure to be protected from the instability that liquefaction can cause, assume "B" in Figure 6-18 is equal to zero (i.e., the minimum width of improved ground is equal to D + 15 feet, but no greater than "Z").

If the soil is of the type that can be densified through the use of stone columns, compaction grout columns, or some other means to improve the soil such that it is no longer susceptible to liquefaction within the improved soil volume, Figure 6-18 should also be used to establish the minimum dimensions of the improved soil.

If it is desired to use dimensions of the ground improvement that are less than the minimums illustrated in Figure 6-18, more sophisticated analyses to determine the effect of using reduced ground improvement dimensions should be conducted (e.g., effective stress two dimensional analyses such as FLAC). The objectives of these analyses include prevention of soil shear failure and excessive differential settlement during liquefaction. The amount of differential settlement allowable for this limit state will depend on the tolerance of the structure being protected to such movement without collapse. Use of smaller ground improvement area dimensions shall be approved of the WSDOT State Geotechnical Engineer and shall be independently peer reviewed in accordance with Section 6.3.

Another reinforcement technique that may be used to mitigate the instability caused by liquefaction is the use of geosynthetic reinforcement as a base reinforcement layer. In this case, the reinforcement is designed as described in Chapter 9, but the liquefied shear strength is used to conduct the embankment base reinforcement design.



Minimum Dimensions for Soil Improvement Volume Below Foundations and Walls
Figure 6-18

Altering Soil Composition – Altering the composition of the soil typically refers to changing the soil matrix so that it is no longer susceptible to liquefaction. Example ground improvement techniques include permeation grouting (either chemical or micro-fine cement), jet grouting, and deep soil mixing. These types of ground improvement are typically more costly than the densification/reinforcement techniques, but may be the most effective techniques if access is limited, construction induced vibrations must be kept to a minimum, and/or the improved ground has secondary functions, such as a seepage barrier or shoring wall.

Drainage Enhancements – By improving the drainage properties of soils susceptible to liquefaction, it may be possible to prevent the build-up of excess pore water pressures, and thus liquefaction. However, drainage improvement is not considered adequately reliable by WSDOT to prevent excess pore water pressure buildup due to liquefaction due to drainage path time for pore pressure to dissipate, and due to the potential for drainage structures to become clogged during installation and in service. In addition, with drainage enhancements some settlement is still likely. Therefore, drainage enhancements shall not be used as a means to mitigate liquefaction. However, drainage enhancements may provide some potential benefits with densification and reinforcement techniques such as stone columns.

6.6 References

AASHTO, 2012. *AASHTO LRFD Bridge Design Specifications*, Sixth Edition.

AASHTO, 2011, *AASHTO Guide Specifications for LRFD Seismic Bridge Design*, Second Edition.

Anderson, D. E., Lam, P., Martin, G., and Wang, J., 2008, *Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes, and Embankments*, NCHRP Report 611, Transportation Research Board, Washington, DC, 137 pp.

ASCE, 2000, Prestandard and Commentary for the Seismic Rehabilitation of Buildings, FEMA 356, 518 pp.

Ashford, S. A., Boulanger, R. W., and Brandenberg, S. J., 2011, Recommended Design Practice for Pile Foundations in Laterally Spreading Ground, PEER Report 2011/04, Berkeley, CA, 68 pp.

ATC-MCEER Joint Venture, 2002. *Comprehensive Specification for the Seismic Design of Bridges*, NCHRP Report 472, National Cooperative Highway Research Program, Washington DC.

Atwater, Brian F., 1996. *Coastal Evidence for Great Earthquakes in Western Washington*. Assessing Earthquake Hazards and Reducing Risk in the Pacific Northwest, USGS Professional Paper 1560 Vol. 1: pp. 77-90.

Bakun, W.H., Haugerud, R.A., Hopper, M.G., and Ludwin, R.S., 2002. "The December 1872 Washington State Earthquake." *Bulletin of the Seismological Society of America*, Vol. 92, No. 8, pp. 3239-3258.

Bommer, J. J., and Acevedo, A. B., 2004, The Use of Real Earthquake Accelerograms as Input to Dynamic Analysis, Journal of Earthquake Engineering, Vol. 8, Special Issue 1, Imperial College Press, pp. 43-91.

Ashford, S. A., Boulanger, R. W., and Brandenberg, S. J., 2011, Recommended Design Practice for Pile Foundations in Laterally Spreading Ground, PEER Report 2011/04, Berkeley, CA, 68 pp.

Boulanger, R.W. and Idriss, I.M., 2006, "Liquefaction Susceptibility Criteria for Silts and Clays," *Journal of Geotechnical and Geoenvironmental Engineering, ASCE* Vol. 132, No. 11, Nov., pp. 1413-1426.

Boulanger, R. W., Kutter, B. L., Brandenberg, S. J., Singh, P., and Chang, D., 2003, *Pile Foundations in Liquefied and Laterally Spreading Ground During Earthquakes: Centrifuge Experiments & Analyses*, College of Engineering, University of California at Davis, 205 pp.

Bowles, J.E., 1996. *Foundation Analysis and Design*, Fifth Edition. The McGraw-Hill Companies, Inc., New York.

Brandenberg, S. J., Boulanger, R. W., Kutter, B. L., and Chang, D., 2007b, "Static Pushover Analyses of Pile Groups in Liquefied and Laterally Spreading Ground in Centrifuge Tests," *ASCE Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 133, No. 9, pp. 1055-1066.

Bray, J. D., and Sancio, R. B., 2006, "Assessment of the Liquefaction Susceptibility of Fine Grained Soils, *ASCE Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 132, No. 9, pp. 1165-1177.

Caltrans, 2012, *Guidelines on Foundation Loading and Deformation Due to Liquefaction Induced Lateral Spreading*, Sacramento, CA, 44 pp. (http://www.dot.ca.gov/research/structures/peer_lifeline_program/docs/guidelines_on_foundation_loading_jan2012.pdf).

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

Dickenson, S. E., McCullough, N. J., Barkau, M. G., and Wavra, B. J., 2002, *Assessment and Mitigation of Liquefaction Hazards to Bridge Approach Embankments in Oregon*, Final Report SPR 361, FHWA-OR-RD-03-04, 210 pp.

Discussion and Closure of "Response of 0.6 m Cast-in-Steel-Shell Pile in Liquefied Soil under Lateral Loading by Thomas J. Weaver, Scott A. Ashford, and Kyle M. Rollins, 2005, *ASCE Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 131, No. 1, pp. 94-102", ASCE 2006, pp. 1238-1241.

Dobry, R., Taboada, V, and Liu., L., 1995. "Centrifuge modeling of liquefaction effects during earthquakes." *Proc. 1st Intl. Conf. On Earthquake Geotechnical Engineering*, K. Ishihara, ed., Tokyo, Japan, Vol. 3, pp. 1291-1324.

Darendeli, M., 2001, *Development of a New Family of Normalized Modulus Reduction and Material Damping Curves*, Ph.D. Dissertation, Department. of Civil Engineering, University of Texas, Austin, 362 pp.

EduPro Civil Systems, Inc., 1999. *ProShake Version 1.10*, Computer Software.

Electrical Power Research Institute (EPRI), 1993. *Guidelines for Site Specific Ground Motions*. Palo Alto, CA. Electrical Power Research Institute, November-TR-102293.

Finn, W.D. Liam, Ledbetter, R.H. and Wu, G., 1994. "Liquefaction in Silty Soils: Design and Analysis." *Ground Failures Under Seismic Conditions*, Geotechnical Special Publication 44. ASCE, New York, New York, pp. 51-76.

Finn, W.D. Liam and Fujita, N., 2004. "Behavior of Piles in Liquefiable Soils during Earthquakes: Analysis and Design Issues." *Proceedings: Fifth International Conference on Case Histories in Geotechnical Engineering*, New York, New York, April 13-17, 2004.

Goter, S.K., 1994. *Earthquakes in Washington and Oregon; 1872-1993, 1994*, USGS Open-File Report No. 94-226A.

International Code Council, Inc., 2006. *2006 International Building Code*. Country Club Hills, IL.

Idriss, I. M., and Boulanger, R. W., 2007, "Residual Shear Strength of Liquefied Soils," *Proceedings of the 27th USSD Annual Meeting and Conference, Modernization and Optimization of Existing Dams and Reservoirs*.

Idriss, I. M., and Boulanger, R. W., 2008, *Soil Liquefaction During Earthquakes*, Earthquake Engineering Research Institute (EERI), MNO-12, 226 pp.

Imai, T. and Tonouchi, K., 1982, "Correlation of N-Value with S-Wave Velocity and Shear Modulus," Proc. 2nd European Symposium on Penetration Testing, Amsterdam, The Netherlands, pp. 67-72.

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.

Jamiolkowski, M., D.C.F. LoPresti, and M. Manassero, 1991, "Evaluation of Relative Density and Shear Strength of Sands from Cone Penetration Test and Flat Dilatometer Test," Soil Behavior and Soft Ground Construction (GSP 119), American Society of Civil Engineers, Reston, Va., pp. 201–238.

Jibson R. and Jibson M., 2003. *Java Program for using Newmark's Method and Simplified Decoupled Analysis to Model Slope Deformations During Earthquakes*. Computer Software. USGS Open File Report 03-005.

Johnson, S.Y., Dadisman, S.V., Childs, J.R., and Stanley, W.D., 1999. "Active Tectonics of the Seattle Fault and Central Puget Sound, Washington: Implications for Earthquake Hazards." *Geological Society of America Bulletin*, Vol. 111, No. 7, pp. 1042-1053.

Kavazanjian, E., Wang, J.-N. J., Martin, G. R., Shamsabadi, A., Lam, I. (P.), Dickenson, S. E., and Hung, C. J., 2011, Geotechnical Engineering Circular #3, *LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations.* Report No. FHWA-NHI-11-032.

Kramer, S.L., 1996 U.S. Department of Transportation, Federal Highway Administration, Washington, D.C. *Geotechnical Earthquake Engineering*. Prentice-Hall, Inc., Upper Saddle River, NJ.

Kramer, S. L., 2007. *Evaluation of Liquefaction Hazards in Washington State*, Washington State Department of Transportation, Report WA-RD 668.1, 312 pp.

Kramer, S.L. and Baska, D.A., 2007. "Estimation of permanent displacements due to lateral spreading," Journal of Geotechnical and Geoenvironmental Engineering, ASCE

Kramer, S.L. and Paulsen, S.B., 2004. "Practical Use of Geotechnical Site Response Models." *PEER Lifelines Program Workshop on the Uncertainties in Nonlinear Soil Properties and the Impact on Modeling Dynamic Soil Response*. Berkeley, CA. March 18-19, 2004.

Kramer, S.L. and Mayfield, R. T., 2007, "Return Period of Soil Liquefaction," *ASCE Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 133, No. 7, pp. 802-813.

Lee, M. and Finn, W., 1978. *DESRA-2, Dynamic Effective Stress Response Analysis of Soil Deposits with Energy Transmitting Boundary Including Assessment of Liquefaction Potential*. Soil Mechanics Series No. 38, Dept. of Civil Engineering, University of British Columbia, Vancouver, B.C.

Matasović, Neven, and Ordóñez, Gustovo, 2007, *D-MOD2000*, GeoMotions, LLC, Computer Software.

Mayne, P.W. and Rix, G.J., 1993, "Gmax-qc Relationships for Clays," Geotechnical Testing Journal, ASTM, Vol. 16, No. 1, pp. 54-60.

McGuire, R.K., 2004. *Seismic Hazard and Risk Analysis*. Monograph MNO-10, Earthquake Engineering Research Institute, Oakland, CA. 221 pp.

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

Olson, S.M. and Stark, T.D., 2002. "Liquefied Strength Ratio from Liquefaction Flow Failure Case Histories," *Canadian Geotechnical Journal*, Vol. 39, June, pp 629-647.

Ordoñez, G.A., 2000. *Shake 2000*, Computer Software.

Rollins, K. M., Gerber, T. M., Lane, J. D., and Ashford, S. A., 2005, "Lateral Resistance of a Full-Scale Pile Group in Liquefied Sand," *ASCE Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 131, No. 1, pp. 115-125.

Reese, L. C., Wang, S. T. Isenhower, W. M., and Arrellaga, J. A., 2004, Computer Program L-Pile Plus, Version 5.0, Technical Manual, ENSOFT, Inc, .Austin, Texas.

Sabatini, P.J., Bachus, R.C., Mayne, P.W., Schneider, J.A., and Zettler, T.E., 2002. *Geotechnical Engineering Circular No. 5, Evaluation of Soil and Rock Properties*, Report No. FHWA-IF-02-034. U.S. Department of Transportation, Federal Highway Administration, Washington, D.C.

Satake, Kenji, et al., 1996. "Time and Size of a Giant Earthquake in Cascadia Inferred from Japanese Tsunami Records of January 1700." *Nature*, Vol. 379, pp. 247-248.

Seed, R. B., Cetin, K. O., Moss, R. E. S., Kammerer, A. M., Pestana, J. M., Riemer, M. F., Sancio, R. B., Bray, J. D., Kayen, R. E., Faris, A., 2003, *Recent Advances in Soil Liquefaction Engineering: A unified and Consistent Framework*, Earthquake Engineering Research Center, Univ. of CA Berkeley, Report No. EERC 2003-06, 71 pp.

Seed. H.B., Wong, R.T., Idriss, I.M. and Tokimatsu, K., 1984, "Moduli and Damping Factors for Dynamic Analyses of Cohesionless Soils," Report No. UCB/EERC-84/14, Earthquake Engineering Research Center, University of California, Berkeley, California.

Singh, J.P., Ashour, M., and Norris, G., 2006, *Laterally and Axially Loaded Deep Foundation Systems – Computer Program DFSAP*, WSDOT Report, 172 pp.

Stewart, J.P., Archuleta, R.J., and Power, M.S., 2008, *Earthquake Spectra, Special Issue on the Next Generation Attenuation Project*, 24(1), EERI.

Tokimatsu, K. and Seed, H.B., 1987. "Evaluation of Settlement in Sands Due to Earthquake Shaking." *ASCE Journal of Geotechnical Engineering*, Vol. 113, No. 8, August 1987.

United States Geological Survey, 2002. *Earthquake Hazards Program*. Website link: www.eqhazmaps.usgs.gov/

Vucetic, M. and Dobry, R. (1991). *Effect of Soil Plasticity on Cyclic Response*. Journal of Geotechnical Engineering, Vo. 117, No. 1, pp. 89-107.

Washington State Department of Transportation, 2012, *LRFD Bridge Design Manual*. Publication Number M23-50.

Washington State Department of Transportation, 2012, *Design Manual*, Publication Number M22-01.

Weaver, T. J., Ashford, S. A., and Rollins, K. M., 2005, "Response of a 0.6 m Cast-in-Steel-Shell Pile in Liquefied Soil Under Lateral Loading," *ASCE Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 131, No. 1, pp. 94-102.

Yelin, T.S., Tarr, A.C., Michael, J.A., and Weaver, C.S., 1994. *Washington and Oregon Earthquake History and Hazards*. USGS, Open File Report 94-226B.

Yokoyame, K., Tamura, K., and Matsuo, O, 1997, "Design Methods of Bridge Foundations against Soil Liquefaction-induced Ground Flow," *Second Italy-Japan Workshop on Seismic Design and Retrofit of Bridges*, Rome, Italy, pp. 1-23.

Youd, T.L.; Idriss, I.M.; Andrus, R.D.; Arango, I.; Castro, G.; Christian, J.T.; Dobry, R.; Finn, W.D.; Harder, L.; Hynes, M.E.; Ishihara, K.; Koester, J.P.; Liao, S.S.C.; Marcuson, W.F.; Martin, G.R.; Mitchell, J.K.; Moriwaki, Y.; Power, M.S.; Robertson, P.K.; Seed, R.B. and Stokoe, K.H., 2001. "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils." *ASCE Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 127, No. 10, pp. 817-833.

Youd, T.L.; Hansen, C.M. and Bartlett, S.F., 2002. "Revised Multilinear Regression Equations for Prediction of Lateral Spread Displacement.". *ASCE Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 128, No. 12, pp. 1007-1017.

Youd, T.L. and Carter, B.L., 2005. "Influence of Soil Softening and Liquefaction on Spectral Acceleration," *Journal of Geotechnical and Geoenvironmental Engineering, ASCE*, Vol. 131, No. 7, pp. 811-825.

Zhang, G., Robertson, P. K., Brachman, R. W. I.,2004, Estimating Liquefaction-Induced Lateral Displacements Using the Standard Penetration Test or Cone Penetration Test, *ASCE Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 130, No. 8, pp. 861-871.

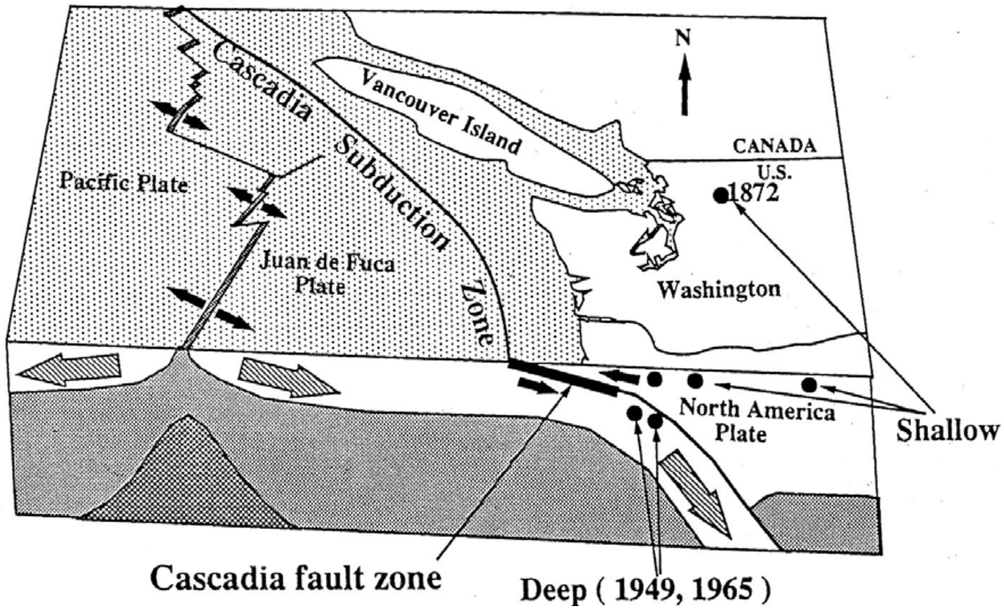
Site specific seismic hazard and response analyses shall be conducted in accordance with Section 6.3 and the AASHTO Guide Specifications for LRFD Seismic Bridge Design. When site specific hazard characterization is conducted, it shall be conducted using the design hazard levels specified in Section 6.3.1.

6-A.1 Background Information for Performing Site Specific Analysis

Washington State is located in a seismically active region. The seismicity varies throughout the state, with the seismic hazard generally more severe in Western Washington and less severe in Eastern Washington. Earthquakes as large as magnitude 8 to 9 are considered possible off the coast of Washington State. The regional tectonic and geologic conditions in Washington State combine to create a unique seismic setting, where some earthquakes occur on faults, but more commonly historic earthquakes have been associated with large broad fault zones located deep beneath the earth's surface. The potential for surface faulting exists, and as discussed in this appendix a number of surface faults have been identified as being potential sources of seismic ground shaking; however, surface vegetation and terrain have made it particularly difficult to locate surface faults. In view of this complexity, a clear understanding of the regional tectonic setting and the recognized seismic source zones is essential for characterizing the seismic hazard at a specific site in Washington State.

6-A.1.1 Regional Tectonics

Washington State is located at the convergent continental boundary known as the Cascadia Subduction Zone (CSZ). The CSZ is the zone where the westward advancing North American Plate is overriding the subducting Juan de Fuca Plate. The CSZ extends from mid-Vancouver Island to Northern California. The interaction of these two plates results in three potential seismic source zones as depicted on Figure 6-A-1. These three seismic source zones are: (1) the shallow crustal source zone, (2) the CSZ Benioff or intraplate source zone, and (3) the CSZ interplate or interface source zone.



The Three Potential Seismic Source Zones Present in the Pacific Northwest (Yelin et al., 1994)
Figure 6-A-1

6-A.1.2 Seismic Source Zones

If conducting a site specific hazard characterization, as a minimum, the following source zones should be evaluated (all reported magnitudes are moment magnitudes):

Shallow Crustal Source Zone – The shallow crustal source zone is used to characterize shallow crustal earthquake activity within the North American Plate throughout Washington State. Shallow crustal earthquakes typically occur at depths ranging up to 12 miles. The shallow crustal source zone is characterized as being capable of generating earthquakes up to about magnitude 7.5. Large shallow crustal earthquakes are typically followed by a sequence of aftershocks.

The largest known earthquakes associated with the shallow crustal source zone in Washington State include an event on the Seattle Fault about 900 AD and the 1872 North Cascades earthquake. The Seattle Fault event was believed to have been magnitude 7 or greater (Johnson, 1999), and the 1872 North Cascades earthquake is estimated to have been between magnitudes 6.8 and 7.4. The location of the 1872 North Cascades earthquake is uncertain; however, recent research suggests the earthquake's intensity center was near the south end of Lake Chelan (Bakun et al, 2002). Other large, notable shallow earthquakes in and around the state include the 1936 Milton Freewater, Oregon magnitude 6.1 earthquake and the North Idaho magnitude 5.5 earthquake (Goter, 1994).

Benioff Source Zone – CSZ Benioff source zone earthquakes are also referred to as intraplate, intraslab, or deep subcrustal earthquakes. Benioff zone earthquakes occur within the subducting Juan de Fuca Plate between depths of 20 and 40 miles and typically have no large aftershocks. Extensive faulting results as the Juan de Fuca Plate is forced below the North American plate and into the upper mantle. Benioff zone earthquakes primarily contribute to the seismic hazard within Western Washington.

The Olympia 1949 (M = 7.1), the Seattle 1965 (M = 6.5), and the Nisqually 2001 (M = 6.8) earthquakes are considered to be Benioff zone earthquakes. The Benioff zone is characterized as being capable of generating earthquakes up to magnitude 7.5. The recurrence interval for large earthquakes originating from the Benioff source zone is believed to be shorter than for the shallow crustal and CSZ interpolate source zones—damaging Benioff zone earthquakes in Western Washington occur every 30 years or so. The deep focal depth of these earthquakes tends to dampen the shaking intensity when compared to shallow crustal earthquakes of similar magnitudes.

CSZ Interplate Source Zone – The Cascadia Subduction Zone (CSZ) is an approximately 650-mile long thrust fault that extends along the Pacific Coast from mid-Vancouver Island to Northern California. CSZ interplate earthquakes result from rupture of all or a portion of the convergent boundary between the subducting Juan de Fuca plate and the overriding North American plate. The fault surfaces approximately 50 to 75 miles off the Washington coast. The width of the seismogenic portion of the CSZ interplate fault varies along its length. As the fault becomes deeper, materials being faulted become ductile and the fault is unable to store mechanical stresses. CSZ interplate earthquakes primarily contribute to the seismic hazard within Western Washington, though not as great as the Benioff source mechanism for much of western Washington. This is particularly the case for the I-5 corridor because of the distance of the CSZ interplate source to the I-5 corridor.

The CSZ is considered as being capable of generating earthquakes of magnitude 8 to magnitude 9. No earthquakes on the CSZ have been instrumentally recorded; however, through the geologic record and historical records of tsunamis in Japan, it is believed that the most recent CSZ event occurred in the year 1700 (Atwater, 1996 and Satake, et al, 1996). Recurrence intervals for CSZ interplate earthquakes are thought to be on the order of 400 to 600 years. Paleogeologic evidence suggests five to seven interplate earthquakes may have been generated along the CSZ over the last 3,500 years at irregular intervals.

6-A.2 Design Earthquake Magnitude

In addition to identifying the site's source zones, the design earthquake(s) produced by the source zones must be characterized for use in evaluating seismic geologic hazards such as liquefaction and lateral spreading. Typically, design earthquake(s) are defined by a specific magnitude, source-to-site distance, and ground motion characteristics.

The following guidelines should be used for determining a site's design earthquake(s):

The design earthquake should consider hazard-compatible events occurring on crustal and subduction-related sources.

More than one design earthquake may be appropriate depending upon the source zones thatcontribute to the site's seismic hazard and the impact that these earthquakes may have on site response.

The design earthquake should be consistent with the design hazard level prescribed in Section 6.3.1.

The USGS interactive deaggregation tool (http://earthquake.usgs.gov/hazards/) provides a summary of contribution to seismic hazard for earthquakes of various magnitudes and source to site distances for a given hazard level and may be used to evaluate relative contribution to ground motion from seismic sources. Note that magnitudes presented in the deaggregation data represent contribution to a specified hazard level and should not simply be averaged for input into analyses such as liquefaction and lateral spreading. Instead, the deaggregation data should be used to assess the relative contribution to the probabilistic hazard from the various source zones. If any source zone contributes more than about 10 percent of the total hazard, design earthquakes representative from each of those source zones should be used for analyses.

For liquefaction or lateral spreading analysis, one of the following approaches should be used to account for the earthquake magnitude, in order of preference:

> Use all earthquake magnitudes applicable at the specific site (from the deggregation) using the multiple scenario or performance based approaches for liquefaction assessment as described by Kramer and Mayfield (2007) and Kramer (2007). These techniques can be accomplished using the WSLIQ software (Kramer, 2007). The hazard level used for this analysis shall be consistent with the hazard level selected for the structure for which the liquefaction analysis is being conducted (typically, a probability of exceedance of 7 percent in 75 years in accordance with the AASHTO Guide Specifications for LRFD Seismic Bridge Design).
>
> If a single or a few larger magnitude earthquakes dominate the deaggregation, the magnitude of the single dominant earthquake or the mean of the few dominant earthquakes in the deaggregation should be used.
>
> For routine design, a default moment magnitude of 7.0 should be used for western Washington and 6.0 for eastern Washington, except within 30 miles of the coast where Cascadia Subduction zone events contribute significantly to the seismic hazard. In that case, the geotechnical designer should use a moment magnitude of 8.0. These default magnitudes should not be used if they represent a smaller hazard than shown in the deaggregation data. Note that these default magnitudes are intended for use in simplified empirically based liquefaction and lateral spreading analysis only and should not be used for development of the design ground motion parameters.

6-A.3 Probabilistic and Deterministic Seismic Hazard Analyses

Probabilistic seismic hazard analysis (PSHA) and deterministic seismic hazard analysis (DSHA) can be completed to characterize the seismic hazard at a site. A DSHA consists of evaluating the seismic hazard at a site for an earthquake of a specific magnitude occurring at a specific location. A PSHA consists of completing numerous deterministic seismic hazard analyses for all feasible combinations of earthquake magnitude and source to site distance for each earthquake source zone. The result of a PSHA is a relationship of the mean annual rate of exceedance of the ground motion parameter of interest with each potential seismic source considered. Since the PSHA provides information on the aggregate risk from each potential source zone, it is more useful in characterizing the seismic hazard at a site if numerous potential sources could impact the site. The USGS 2002 probabilistic hazard maps on the USGS website and as published in AASHTO (2012) are based on PSHA.

PSHAs and DSHAs may be required where the site is located close to a fault, long-duration ground motion is expected, or if the importance of the bridge is such that a longer exposure period is required by WSDOT. For a more detailed description and guidelines for development of PSHAs and DSHAs, see Kramer (1996) and McGuire (2004).

Site specific hazard analysis should include consideration of topographic and basin effects, fault directivity and near field effects.

At a minimum, seismic hazard analysis should consider the following sources:

Cascadia subduction zone interpolate (interface) earthquake

Cascadia subduction zone intraplate (Bennioff) earthquake

Crustal earthquakes associated with non-specific or diffuse sources (potential sources follow). These sources will account for differing tectonic and seismic provinces and include seismic zones associated with Cascade volcanism

Earthquakes on known and potentially active crustal faults. The following list of potential seismic sources may be used for hazard assessment and site response development. The applicability of these sources will depend on their proximity to the site.

Seattle Fault Zone	Horse Heaven Hills Anticline
Southern Whidbey Island Fault	Rattlesnake-Wallula Fault System
Utsalady Fault	Mill Creek Fault
Strawberry Point Fault	Saddle Mountains Fault
Devils Mountain Fault	Hite Fault System

When PSHA or DSHA are performed for a site, the following information shall be included as a minimum in project documentation and reports:

Overview of seismic sources considered in analysis

Summary of seismic source parameters including length/boundaries, source type, slip rate, segmentation, maximum magnitude, recurrence models and relationships used, source depth and geometry. This summary should include the rationale behind selection of source parameters.

Assumptions underlying the analysis should be summarized in either a table (DSHA) or in a logic tree (PSHA)

The 2002 USGS probabilistic hazard maps as published in AASHTO (2007) essentially account for regional seismicity and attenuation relationships, recurrence rates, maximum magnitude of events on know faults or source zones, and the location of the site with respect to the faults or source zones. The USGS data is sufficient for most sites, and more sophisticated seismic hazard analyses are generally not required; the exceptions may be to capture the effects of sources not included in the USGS model, to assess near field or directivity influences, or to incorporate topographic impacts or basin effects.

The AASHTO seismic hazard maps do not explicitly account for the effects of near-fault motions (i.e., ground motion directivity or pulse effects) or bedrock topography (i.e., so called basin effects). These effects modify ground motions, particularly at certain periods, for sites located near active faults (typically with 6 miles) or for sites where significant changes in bedrock topography occurs. For specific requirements regarding near fault effects, see the AASHTO Guide Specifications for LRFD Seismic Bridge Design.

6-A.4 Selection of Attenuation Relationships

Attenuation relationships describe the decay of earthquake energy as it travels from the seismic source to the project site. Many of the newer published relationships are capable of accommodating site soil conditions as well as varying source parameters (e.g., fault type, location relative to the fault, near-field effects, etc.) In addition, during the past 10 years, specific attenuation relationships have been developed for Cascadia subduction zone sources. For both deterministic and probabilistic hazard assessments, attenuation relationships used in analysis should be selected based on applicability to both the site conditions and the type of seismic source under consideration. Rationale for the selection of and assumptions underlying the use of attenuation relationships for hazard characterization shall be clearly documented.

Attenuation relationships used in developing the USGS/AASHTO Seismic Hazard Maps for these Guide Specifications do not include the Next Generation Attenuation (NGA) relationships developed in 2006 and 2007. It is recommended that the NGA relationships (Stewart, et al., 2008) be used for any future site-specific studies for modeling crustal sources.

If deterministic methods are used to develop design spectra, the spectral ordinates should be developed using a range of ground motion attenuation relationships consistent with the source mechanisms. At least three to four attenuation relationships should be used.

6-A.5 Site Specific Ground Response Analysis

6-A.5.1 Design/Computer Models

Site specific ground response analyses are most commonly done using one-dimensional equivalent-linear or non linear procedures. A one dimensional analysis is generally based on the assumption that soils and ground surface are laterally uniform and horizontal and that ground surface motions can be modeled by vertically propagating shear wave through laterally uniform soils. The influence of vertical motions, surface waves, laterally non-uniform soil conditions, incoherence and spatial variation of ground motions are not accounted for in conventional, one-dimensional analyses (Kavazanjian, et al., 2011). A variety of site response computer models are available to geotechnical designers for dynamic site response analyses. In general, there are three classes of dynamic ground response models: 1) one dimensional equivalent linear, 2) one dimensional nonlinear, and 3) multi-dimension models.

One-Dimensional Equivalent Linear Models – One-dimensional equivalent linear site response computer codes, such as ProShake (EduPro Civil Systems, 1999) or Shake2000 (Ordoñez, 2000), use an iterative total stress approach to estimate the nonlinear, inelastic behavior of soils. These programs use an average shear modulus and material damping over the entire cycle of loading to approximate the hysteresis loop.

The equivalent linear model provides reasonable results for small strains (less than about 1 to 2 percent) (Kramer and Paulsen, 2004). Equivalent linear analysis should be used with caution where large strain is likely to occur.

One-Dimensional Nonlinear Models – One-dimensional, nonlinear computer codes, such as D-MOD 2000, or DESRA, use direct numerical integration of the incremental equation of motion in small time steps and account for the nonlinear soil behavior through use of constitutive soil models. Depending upon the constitutive model used, these programs can model pore water pressure buildup and permanent deformations. The accuracy of nonlinear models depends on the proper selection of parameters used by constitutive soil model and the ability of the constitutive model to represent the response of the soil to ground shaking.

Another issue that can affect the accuracy of the model is how the G/G_{max} and damping relations are modeled and the ability of the design model to adapt those relations to site specific data. Additionally, the proper selection of a Rayleigh damping value can have a significant effect on the modeling results. In general, a value of 1 to 2% is needed to maintain numerical stability. It should be recognized that the Rayleigh damping will act in addition to hysteretic damping produced by the nonlinear, inelastic soil model. Rayleigh damping should therefore be limited to the smallest value that provides the required numerical stability. The results of analyses using values greater than 1 to 2% should be interpreted with great caution.

See Section 6.4.2.2 for specific issues related to liquefaction modeling when using one-dimensional nonlinear analysis methods.

Two and Three Dimensional Models – Two- and three-dimensional site response analyses can be performed using computer codes, such as QUAD4, PLAXIS, FLAC, and DYNAFLOW, and use both equivalent linear and nonlinear models. Many attributes of the two- and three-dimensional models are similar to those described above for the one-dimensional equivalent linear and nonlinear models. However, the two- and three-dimensional computer codes typically require significantly more model development and computational time than one-dimensional analyses. The important advantages of the two- and three-dimensional models include the ability to consider soil anisotropy, irregular soil stratigraphy, surface waves, irregular topography, and soil-structure interaction. Another advantage with the two- and three-dimensional models is that seismically induced permanent displacements can be estimated. Successful application of these codes requires considerable knowledge and experience. Expert peer review of the analysis shall be conducted, in accordance with Section 6.3 unless approval to not conduct the peer review is obtained from the State Geotechnical Engineer.

6-A.5.2 Input Parameters for Site Specific Response Analysis

The input parameters required for both equivalent-linear and nonlinear site specific ground response analysis include the site stratigraphy (including soil layering and depth to rock or rock-like material), dynamic properties for each stratigraphic layer (including soil and rock stiffness, e.g., shear wave velocity), and ground motion time histories. Soil and rock parameters required by the equivalent linear models include the shear wave velocity or initial (small strain) shear modulus and unit weight for each layer, and curves relating the shear modulus and damping ratio as a function of shear strain (See Figures 6-1 through 6-3).

The parameters required for cyclic nonlinear soil models generally consist of a backbone curve that models the stress strain path during cyclic loading and rules for loading and unloading, stiffness degradation, pore pressure generation and other factors (Kramer, 1996). More sophisticated nonlinear soil constitutive models require definition of yield surfaces, hardening functions, and flow rules. Many of these models require specification of multiple parameters whose determination may require a significant laboratory testing program.

One of the most critical aspects of the input to a site-specific response analysis is the soil and rock stiffness and impedance values or shear wave velocity profile. Great care should be taken in establishing the shear wave velocity profile – it should be measured whenever possible. Equal care should be taken in developing soil models, including shear wave velocity profiles, to adequately model the potential range and variability in ground motions at the site and adequately account for these in the site specific design parameters (e.g., spectra). A long bridge, for example, may cross materials of significantly different stiffness (i.e., velocities) and/or soil profiles beneath the various bridge piers and abutments. Because different soil profiles can respond differently, and sometimes (particularly when very soft and/or liquefiable soils are present) very differently, great care should be taken in selecting and averaging soil profiles and properties prior to performing the site response analyses. In most cases, it is preferable to analyze the individual profiles and then aggregate the responses rather than to average the soil properties or profiles and analyze only the averaged profile.

A suite of ground motion time histories is required for both equivalent linear and nonlinear site response analyses as described in Section 6-A.6. The use of at least three input ground motions is required and seven or more is preferred for site specific ground response analysis (total, regardless of the number of source zones that need to be considered. Guidelines for selection and development of ground motion time histories are also described in Section 6-A.6.

6-A.6 Analysis Using Acceleration-Time Histories

The site specific analyses discussed in Section 6.3 and in this appendix are focused on the development of site specific design spectra and use in other geotechnical analyses. However, site specific time histories may be required as input in nonlinear structural analysis.

Time history development and analysis for site-specific ground response or other analyses shall be conducted as specified in the AASHTO Guide Specifications for LRFD Seismic Bridge Design. For convenience, Article 3.4.4 and commentary of the AASHTO Guide Specifications is provided below:

> *Earthquake acceleration time histories will be required for site-specific ground motion response evaluations and for nonlinear inelastic dynamic analysis of bridge structures. The time histories for these applications shall have characteristics*

that are representative of the seismic environment of the site and the local site conditions, including the response spectrum for the site.

Response-spectrum-compatible time histories shall be developed from representative recorded earthquake motions. Analytical techniques used for spectrum matching shall be demonstrated to be capable of achieving seismologically realistic time series that are similar to the time series of the initial time histories selected for spectrum matching. The recorded time histories should be scaled to the approximate level of the design response spectrum in the period range of significance unless otherwise approved by the Owner. At least three response-spectrum-compatible time histories shall be used for representing the design earthquake (ground motions having 7 percent probability of exceedance in 75 years) when conducting dynamic ground motion response analyses or nonlinear inelastic modeling of bridges.

- *For site-specific ground motion response modeling single components of separate records shall be used in the response analysis. The target spectrum used to develop the time histories is defined at the base of the soil column. The target spectrum is obtained from the USGS/AASHTO Seismic Hazard Maps or from a site-specific hazard analysis as described in Article 3.4.3.1.*
- *For nonlinear time history modeling of bridge structures, the target spectrum is usually located at or close to the ground surface, i.e., the rock spectrum has been modified for local site effects. Each component of motion shall be modeled. The issue of requiring all three orthogonal components (x, y, and z) of design motion to be input simultaneously shall be considered as a requirement when conducting a nonlinear time-history analysis. The design actions shall be taken as the maximum response calculated for the three ground motions in each principal direction.*

If a minimum of seven time histories are used for each component of motion, the design actions may be taken as the mean response calculated for each principal direction.

For near-field sites (D < 6 miles) the recorded horizontal components of motion selected should represent a near-field condition and that they should be transformed into principal components before making them response-spectrum-compatible. The major principal component should then be used to represent motion in the fault-normal direction and the minor principal component should be used to represent motion in the fault-parallel direction.

Characteristics of the seismic environment of the site to be considered in selecting time-histories include: tectonic environment (e.g., subduction zone; shallow crustal faults in western United States or similar crustal environment; eastern United States or similar crustal environment); earthquake magnitude; type of faulting (e.g., strike-slip; reverse; normal); seismic-source-to-site distance; basin effects, local site conditions; and design or expected ground-motion characteristics (e.g., design response spectrum; duration of strong shaking; and special ground-motion characteristics such as near-fault characteristics). Dominant earthquake magnitudes and distances, which contribute principally to the probabilistic design response spectra at a site, as determined from national ground motion maps, *can be obtained from deaggregation information on the U.S. Geological Survey website: http://geohazards.cr.usgs.gov/.*

It is desirable to select time-histories that have been recorded under conditions similar to the seismic conditions at the site listed above, but compromises are usually required because of the multiple attributes of the seismic environment and the limited data bank of recorded time-histories. Selection of time-histories having similar earthquake magnitudes and distances, within reasonable ranges, are especially important parameters because they have a strong influence on response spectral content, response spectral shape, duration of strong shaking, and near-source ground-motion characteristics. It is desirable that selected recorded motions be somewhat similar in overall ground motion level and spectral shape to the design spectrum to avoid using very large scaling factors with recorded motions and very large changes in spectral content in the spectrum-matching approach. If the site is located within 6 miles of an active fault, then intermediate-to-long-period ground-motion pulses that are characteristic of near-source time-histories should be included if these types of ground motion characteristics could significantly influence structural response. Similarly, the high short-period spectral content of near-source vertical ground motions should be considered.

Ground-motion modeling methods of strong-motion seismology are being increasingly used to supplement the recorded ground-motion database. These methods are especially useful for seismic settings for which relatively few actual strong-motion recordings are available, such as in the central and eastern United States. Through analytical simulation of the earthquake rupture and wave-propagation process, these methods can produce seismologically reasonable time series.

Response spectrum matching approaches include methods in which time series adjustments are made in the time domain (Lilhanand and Tseng, 1988; Abrahamson, 1992) and those in which the adjustments are made in the frequency domain (Gasparini and Vanmarcke, 1976; Silva and Lee, 1987; Bolt and Gregor, 1993). Both of these approaches can be used to modify existing time-histories to achieve a close match to the design response spectrum while maintaining fairly well the basic time-domain character of the recorded or simulated time-histories. To minimize changes to the time-domain characteristics, it is desirable that the overall shape of the spectrum of the recorded time-history not be greatly different from the shape of the design response spectrum and that the time-history initially be scaled so that its spectrum is at the approximate level of the design spectrum before spectrum matching.

When developing three-component sets of time histories by simple scaling rather than spectrum matching, it is difficult to achieve a comparable aggregate match to the design spectra for each component of motion when using a single scaling factor for each time-history set. It is desirable, however, to use a single scaling factor to preserve the relationship between the components. Approaches for dealing with this scaling issue include:

- *Use of a higher scaling factor to meet the minimum aggregate match requirement for one component while exceeding it for the other two,*
- *Use of a scaling factor to meet the aggregate match for the most critical component with the match somewhat deficient for other components, and*

- *Compromising on the scaling by using different factors as required for different components of a time-history set.*

While the second approach is acceptable, it requires careful examination and interpretation of the results and possibly dual analyses for application of the horizontal higher horizontal component in each principal horizontal direction.

The requirements for the number of time histories to be used in nonlinear inelastic dynamic analysis and for the interpretation of the results take into account the dependence of response on the time domain character of the time histories (duration, pulse shape, pulse sequencing) in addition to their response spectral content.

Additional guidance on developing acceleration time histories for dynamic analysis may be found in publications by the Caltrans Seismic Advisory Board Adhoc Committee (CSABAC) on Soil-Foundation-Structure Interaction (1999) and the U.S. Army Corps of Engineers (2000). CSABAC (1999) also provides detailed guidance on modeling the spatial variation of ground motion between bridge piers and the conduct of seismic soil-foundation-structure interaction (SFSI) analyses. Both spatial variations of ground motion and SFSI may significantly affect bridge response. Spatial variations include differences between seismic wave arrival times at bridge piers (wave passage effect), ground motion incoherence due to seismic wave scattering, and differential site response due to different soil profiles at different bridge piers. For long bridges, all forms of spatial variations may be important. For short bridges, limited information appears to indicate that wave passage effects and incoherence are, in general, relatively unimportant in comparison to effects of differential site response (Shinozuka et al., 1999; Martin, 1998). Somerville et al. (1999) provide guidance on the characteristics of pulses of ground motion that occur in time histories in the near-fault region.

In addition to the information sources cited above, Kramer (1996) and Bommer and Acevedo (2004) provide excellent guidance on the selection, scaling, and use of time histories for ground motion characterization and dynamic analysis.

Final selection of time histories to be used will depend on two factors:

- How well the response spectrum generated from the scaled time histories matches the design response spectrum, and
- Similarity of the fault mechanisms for the time histories to those of recognized seismic source zones that contribute to the site's seismic hazard Also, if the earthquake records are used in the site specific ground response model as bedrock motion, the records should be recorded on sites with bedrock characteristics. The frequency content, earthquake magnitude, and peak bedrock acceleration should also be used as criteria to select earthquake time histories for use in site specific ground response analysis.

The requirements in the first bullet are most important to meet if the focus of the seismic modeling is structural and foundation design. The requirements in the second bullet are most important to meet if liquefaction and its effects are a major consideration in the design of the structure and its foundations. Especially important in the latter case is the duration of strong motion.

Note that a potential issue with the use of a spectrum-compatible motion that should be considered is that in western Washington, the uniform hazard spectrum (UHS) may have significant contributions from different sources that have major differences in magnitudes and site-to-source distances. The UHS cannot conveniently be approximated by a single earthquake source. For example, the low period (high frequency) part of the UHS spectrum may be controlled by a low-magnitude, short-distance event and the long period (low frequency) portion by a large-magnitude, long-distance event. Fitting a single motion to that target spectrum will therefore produce an unrealistically energetic motion with an unlikely duration. Using that motion as an input to an analysis involving significant amounts of nonlinearity (such as some sort of permanent deformation analysis, or the analysis of a structure with severe loading) can lead to overprediction of response (soil and/or structural). However, if the soil is overloaded by this potentially unrealistically energetic prediction of ground motion, the soil could soften excessively and dampen a lot of energy (large strains), more than would be expected in reality, leading to an unconservative prediction of demands in the structure.

To address this potential issue, time histories representing the distinctly different seismic sources (e.g., shallow crustal versus subduction zone) should be spectrally matched or scaled to correspondingly distinct, source-specific spectra. A source-specific spectrum should match the UHS or design spectrum over the period range in which the source is the most significant contributor to the ground motion hazard, but will likely be lower than the UHS or design spectrum at other periods for which the source is not the most significant contributor to the hazard. However, the different source-spectra in aggregate should envelope the UHS or design spectrum. Approval by the State Geotechnical Engineer and State Bridge Engineer is required for use of source-specific spectra and time histories.

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).

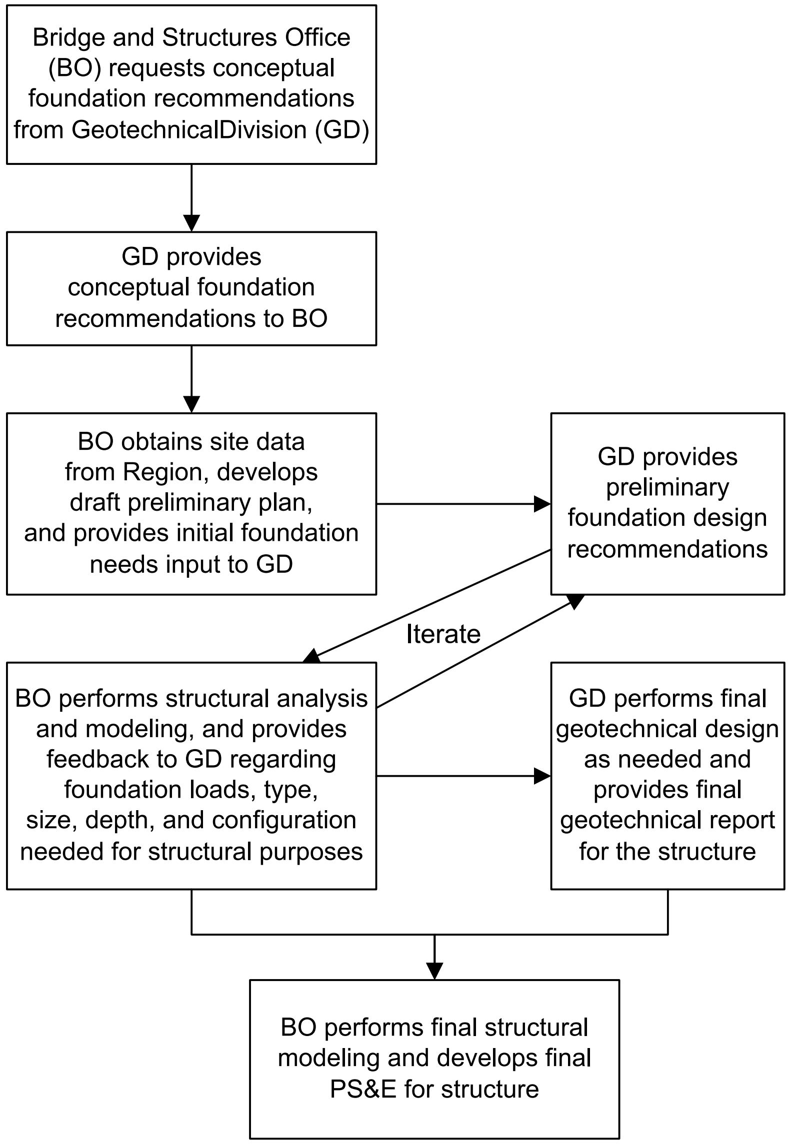
All structure foundations within WSDOT Right of Way or whose construction is administered by WSDOT shall be designed in accordance with the *Geotechnical Design Manual* (GDM) and the following documents:

- *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:

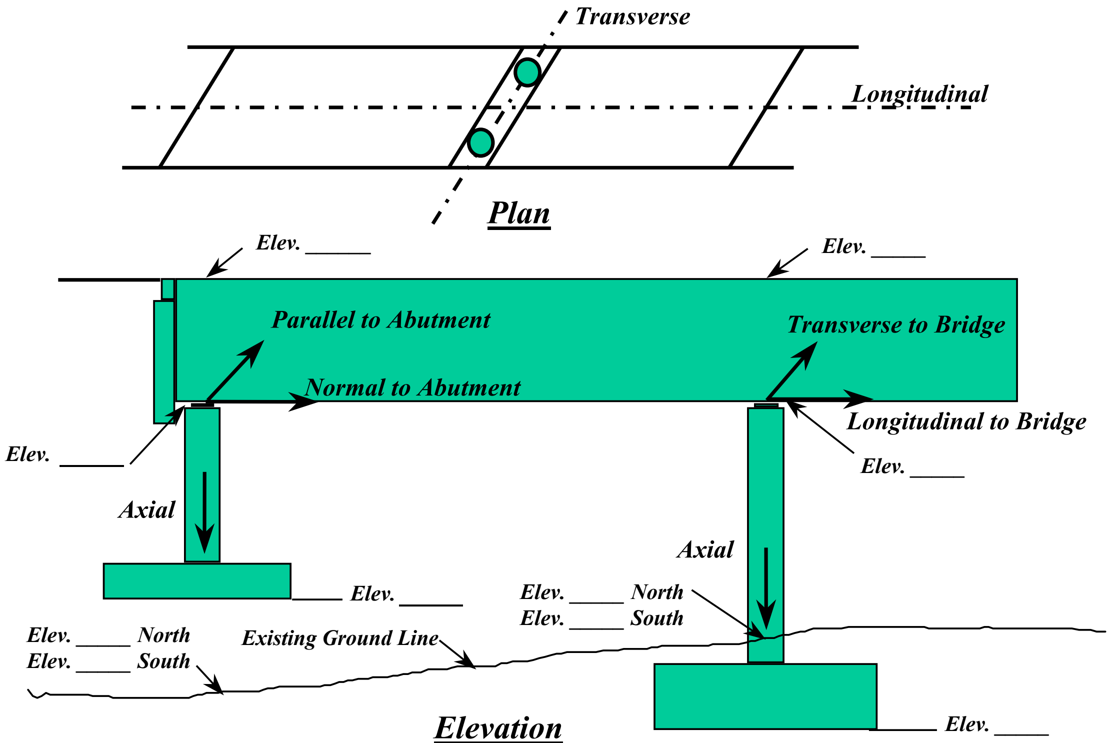
$$\Sigma\eta_\iota\gamma_i\, Q_i \leq \phi R_n \qquad (8\text{-}1)$$

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 <u>for the strength limit state</u>. 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
ΔH ≤ 2 in	$\Delta H_{100} \le 1.5$ in	Design and Construct
2 in < ΔH ≤ 6 in	1.5 in < $\Delta H_{100} \le 5$ in	Ensure structure can tolerate settlement
ΔH > 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

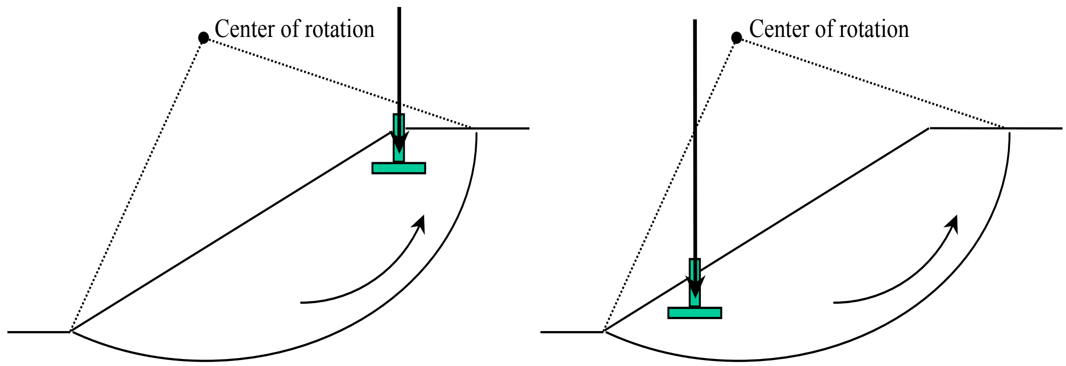
Rotation movements should be evaluated at the top of the substructure unit (in plan location) and at the deck elevation.

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.

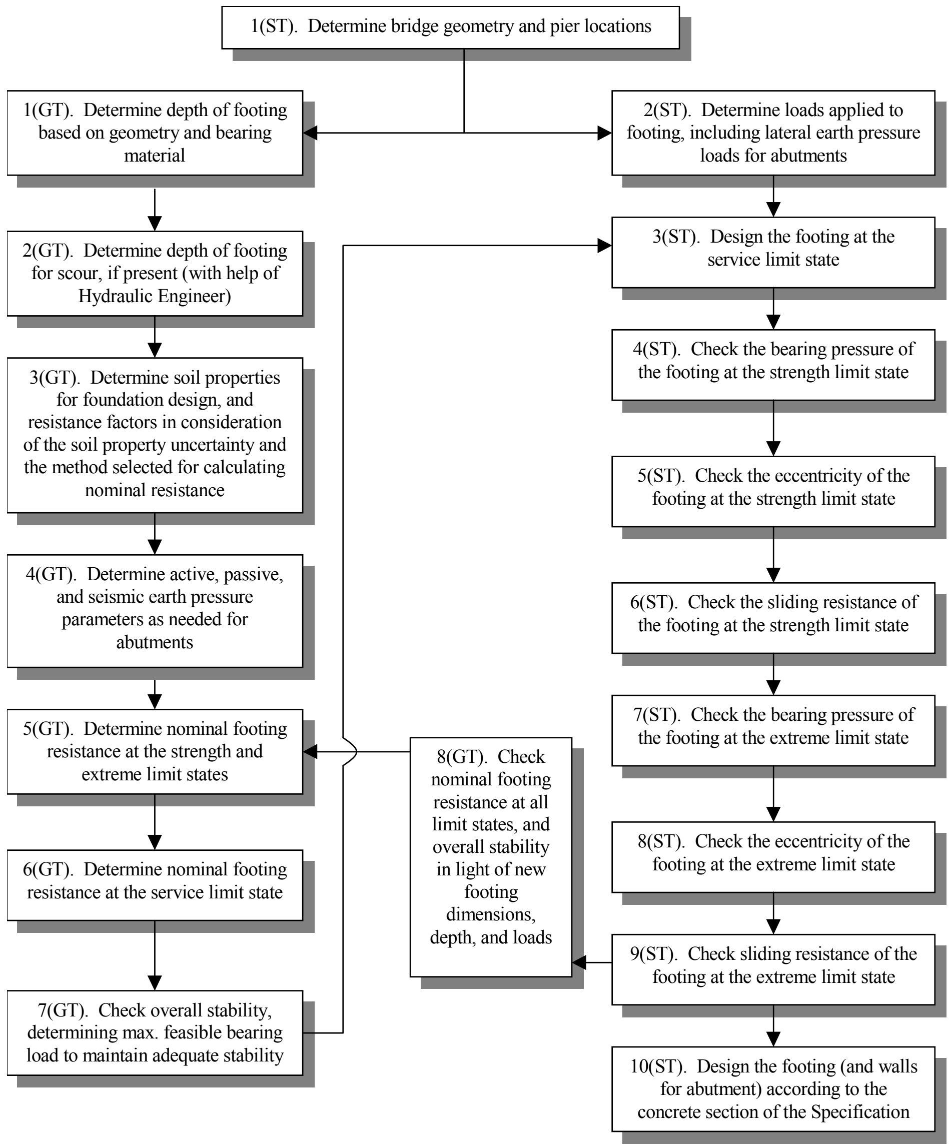
8.10.2 Other Extreme Event Limit States

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

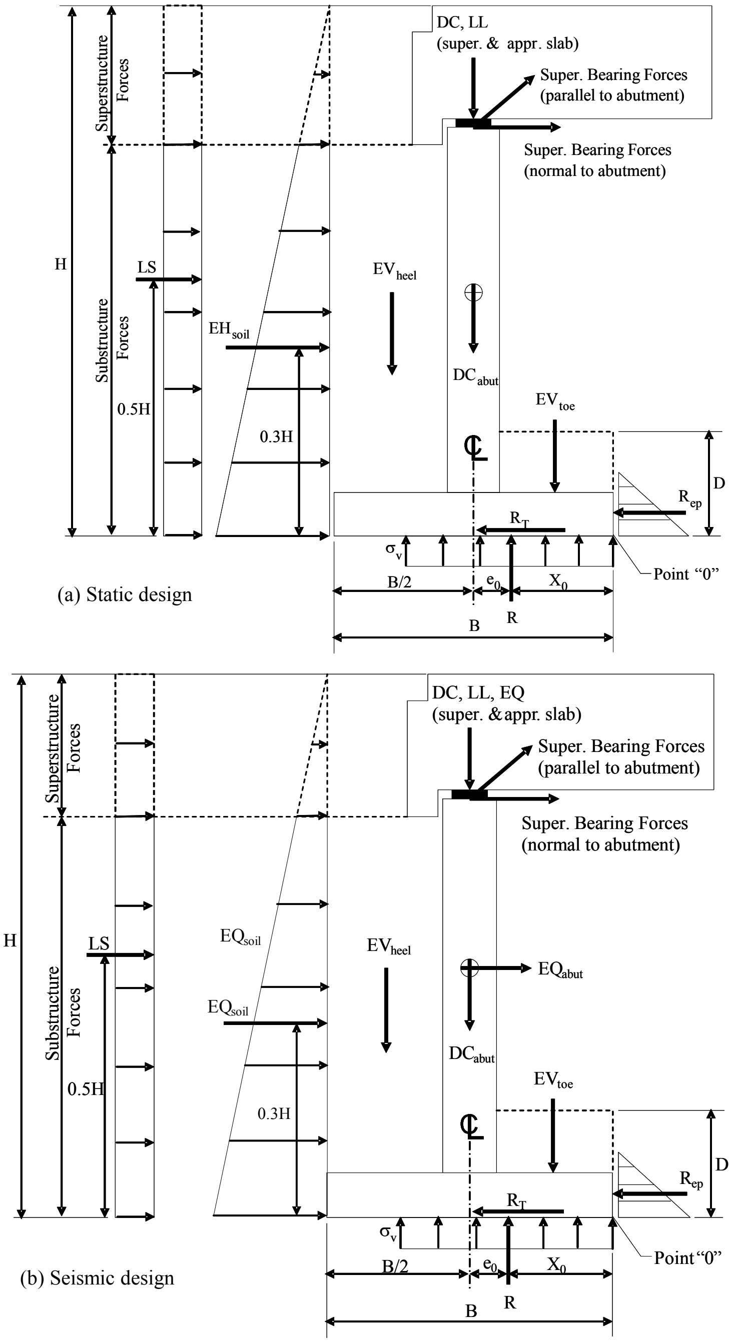
Figure 8-4 provides a flowchart that illustrates the design process, and interaction required between structural and geotechnical engineers, needed to complete a spread footing design. ST denotes steps usually completed by the Structural Designer, while GT denotes those steps normally completed by the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



Flowchart for LRFD Spread Footing Design
Figure 8-4

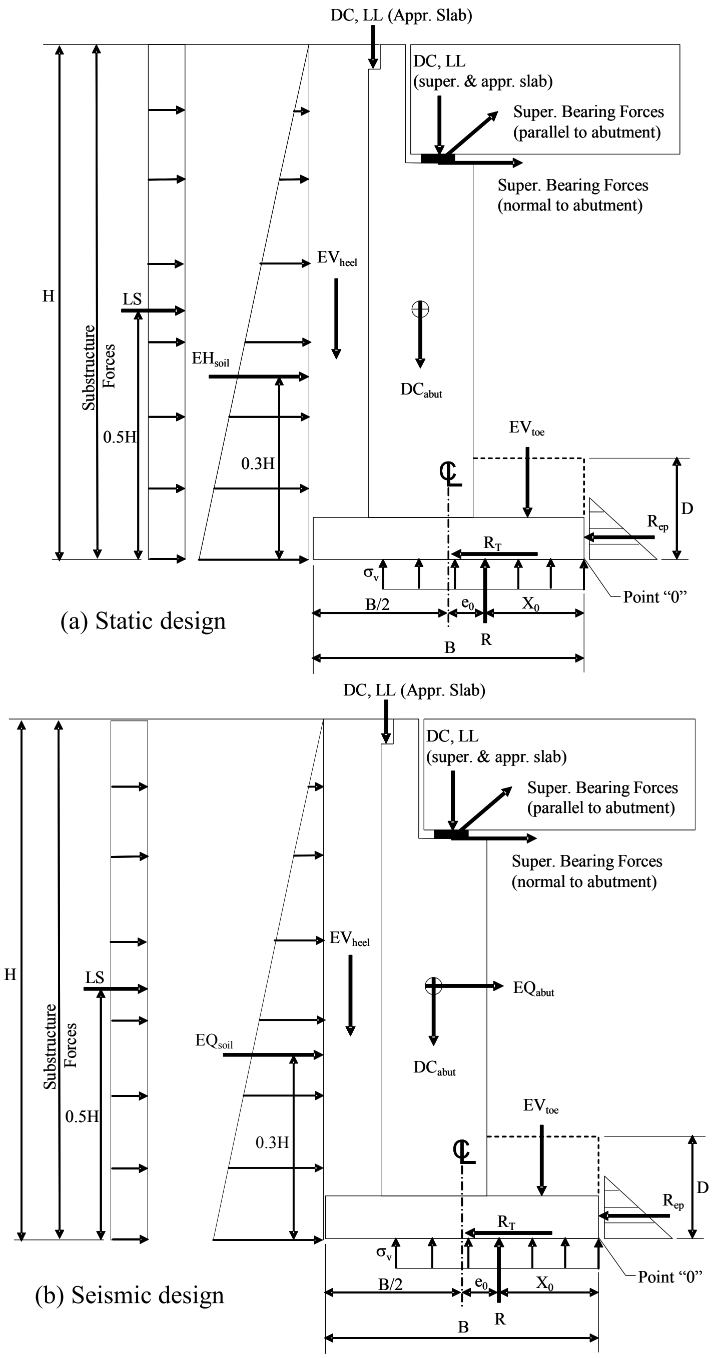
8.11.1 Loads and Load Factor Application to Footing Design

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 <u>shall</u> 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

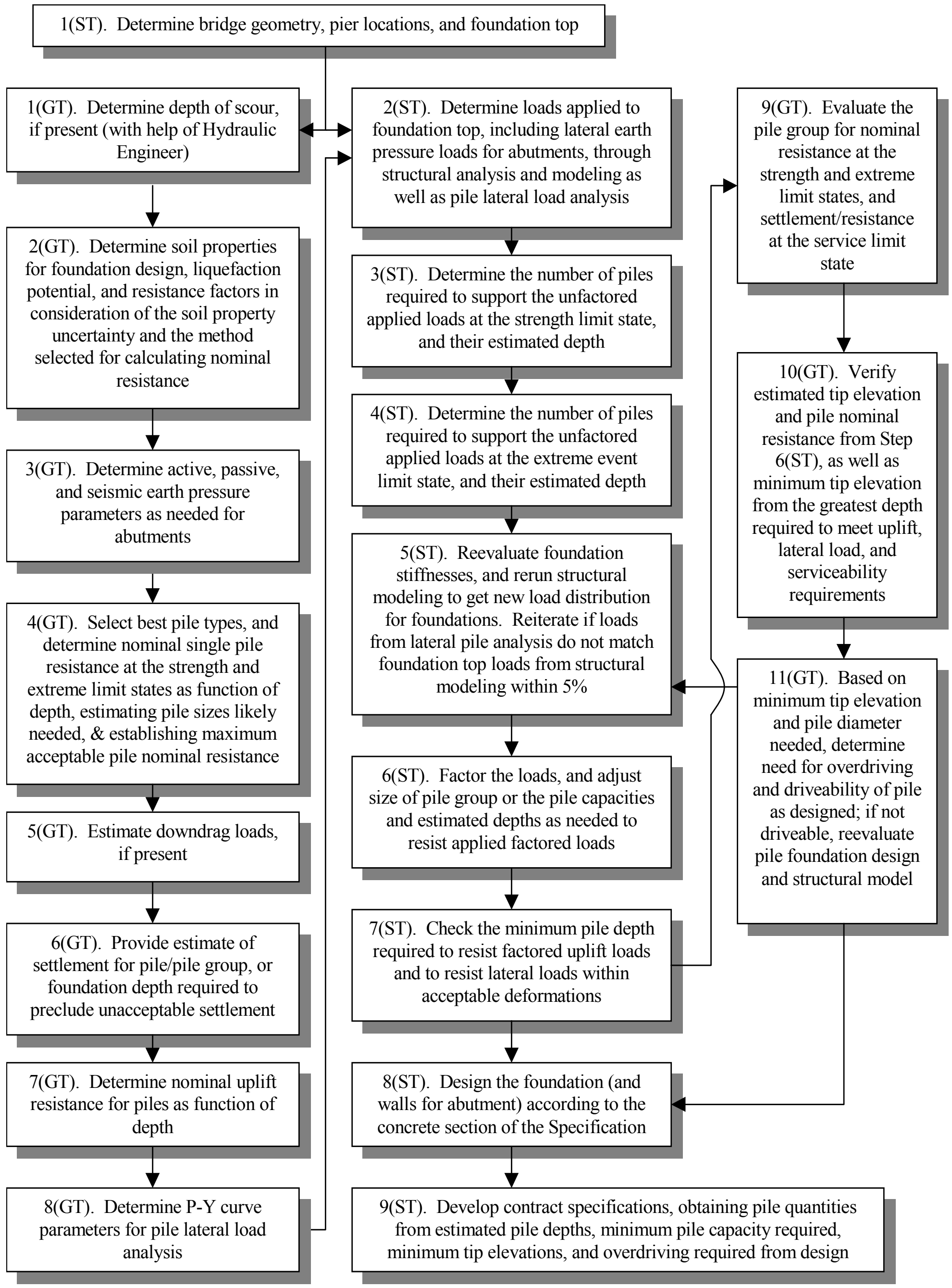
Footings shall not be located on or within liquefiable soil. Footings may be located on liquefiable soils that have been improved through densification or other means so that they do not liquefy. Footings may also be located above liquefiable soil in a non-liquefiable layer if the footing is designed 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

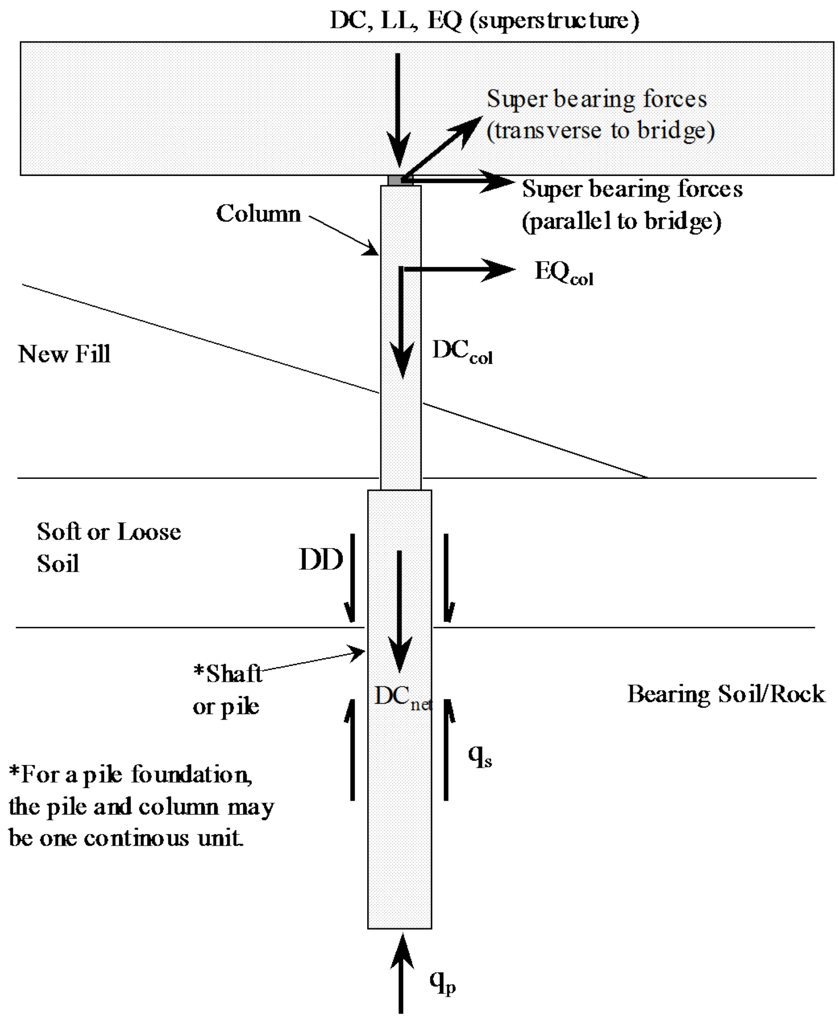
Figure 8-7 provides a flowchart that illustrates the design process, and interaction required between structural and geotechnical engineers, needed to complete a driven pile foundation design. ST denotes steps usually completed by the Structural Designer, while GT denotes those steps normally completed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



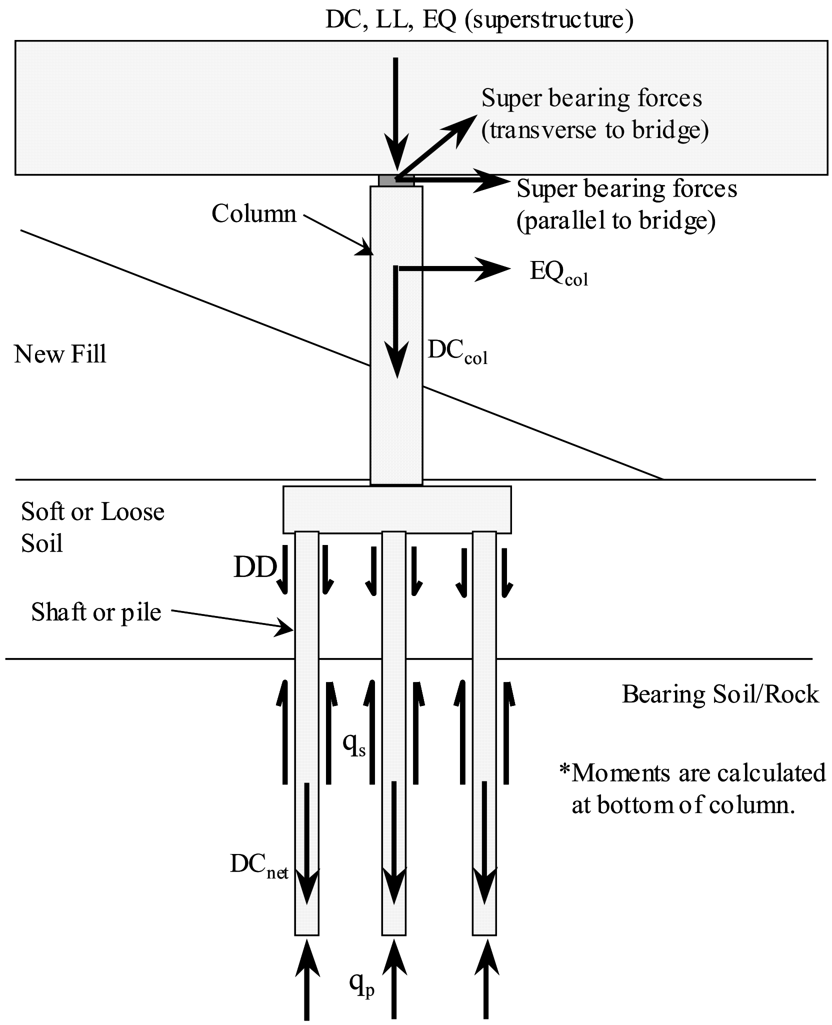
Design Flowchart for Pile Foundation Design
Figure 8-7

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})

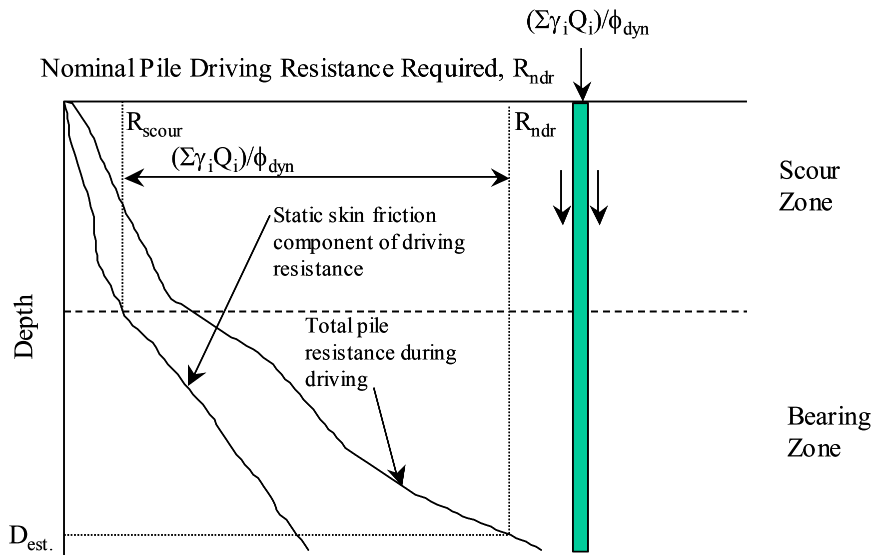
From Equation 8-1, the summation of the factored loads ($\Sigma\gamma_i Q_i$) must be less than or equal to the factored resistance (φR_n). Therefore, the nominal resistance R_n must be greater than or equal to the sum of the factored loads divided by the resistance factor φ. Hence, the nominal bearing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

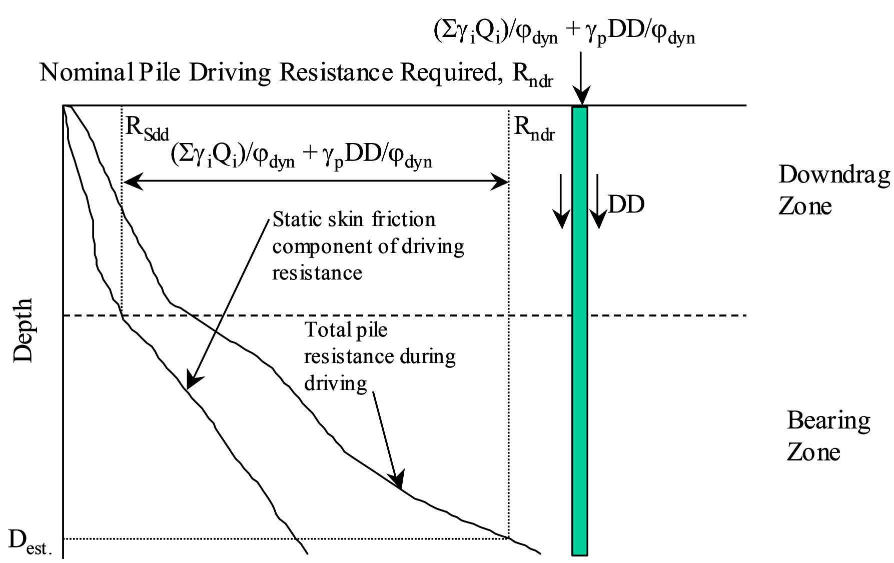
Similar to the derivation of Equation 8-2, the nominal bearing resistance of the pile needed to resist the factored loads, including downdrag, is therefore,

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

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:

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

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

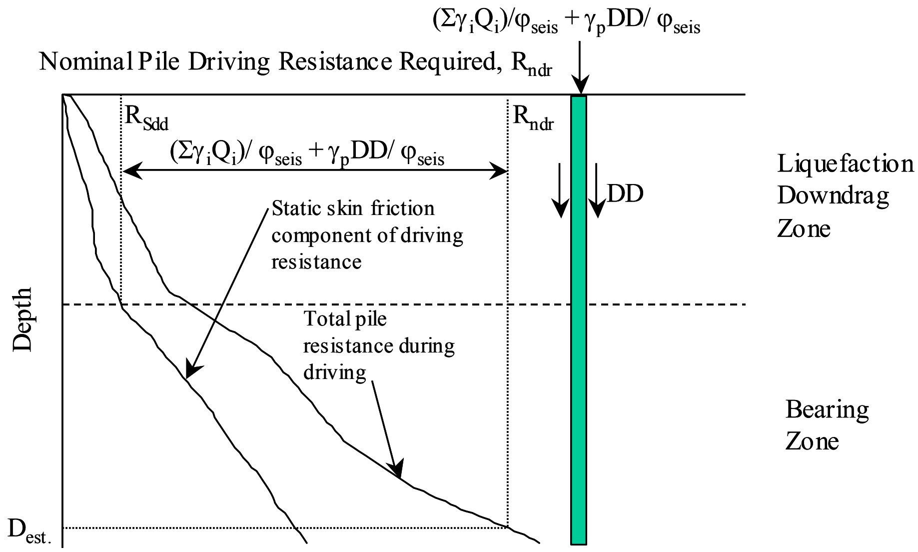
The nominal bearing resistance of the pile needed to resist the factored loads, including downdrag, is therefore,

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

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.

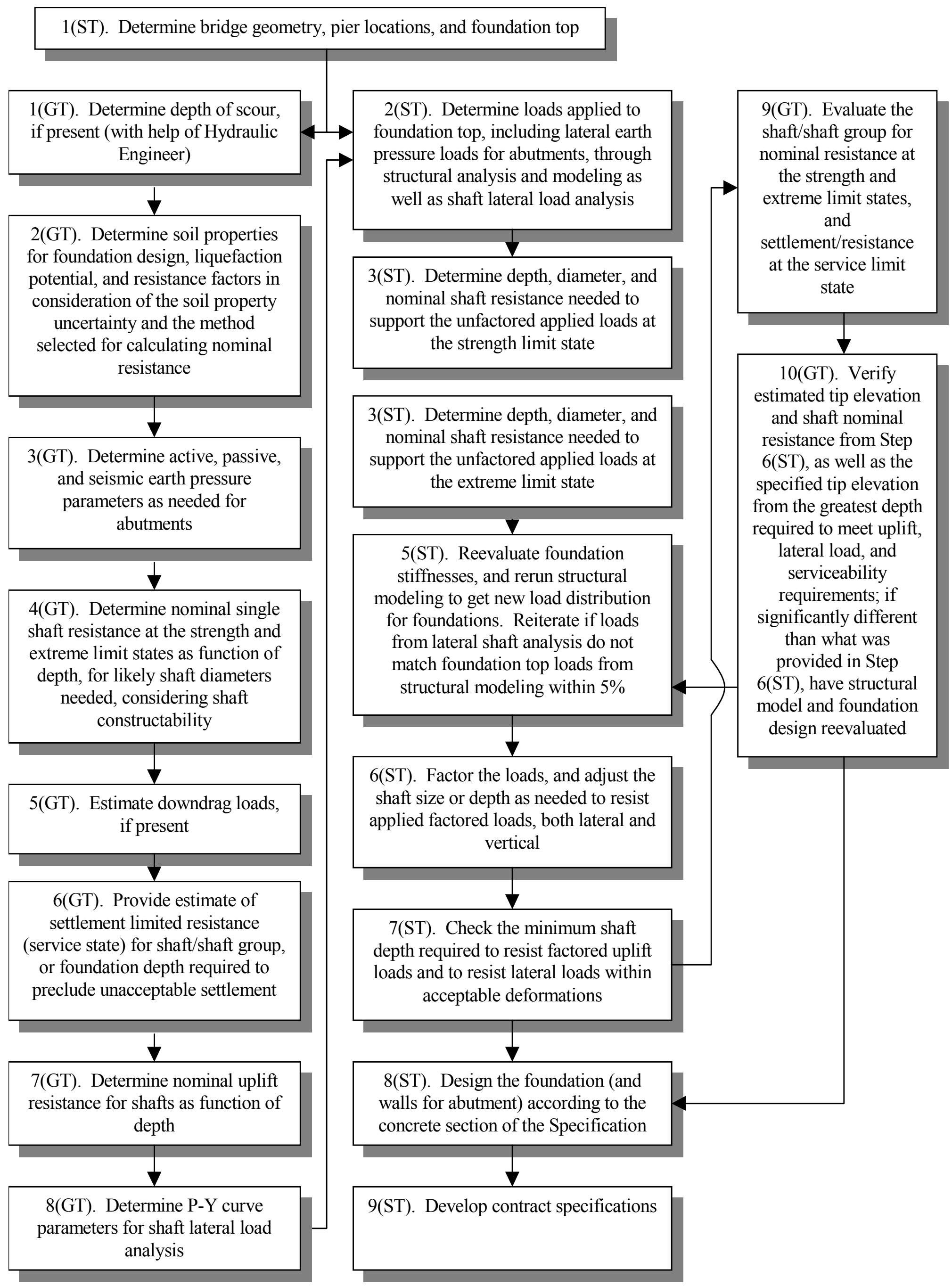
Regarding the reduction of P-Y soil strength and stiffness parameters to account for liquefaction, see Section 6.5.1.2.

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.

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

<u>*GSI—Geologic Strength Index*</u>

10.3—NOTATION

ONE ADDITION BELOW – THE REMAINDER STAYS THE SAME

<u>*s, m, a* = fractured rock mass parameters (10.4.6.4)</u>

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.

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.

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.

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

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.

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.

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. <u>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.</u>	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~~ <u>minimum</u> 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~~ <u>minimum</u> 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.

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

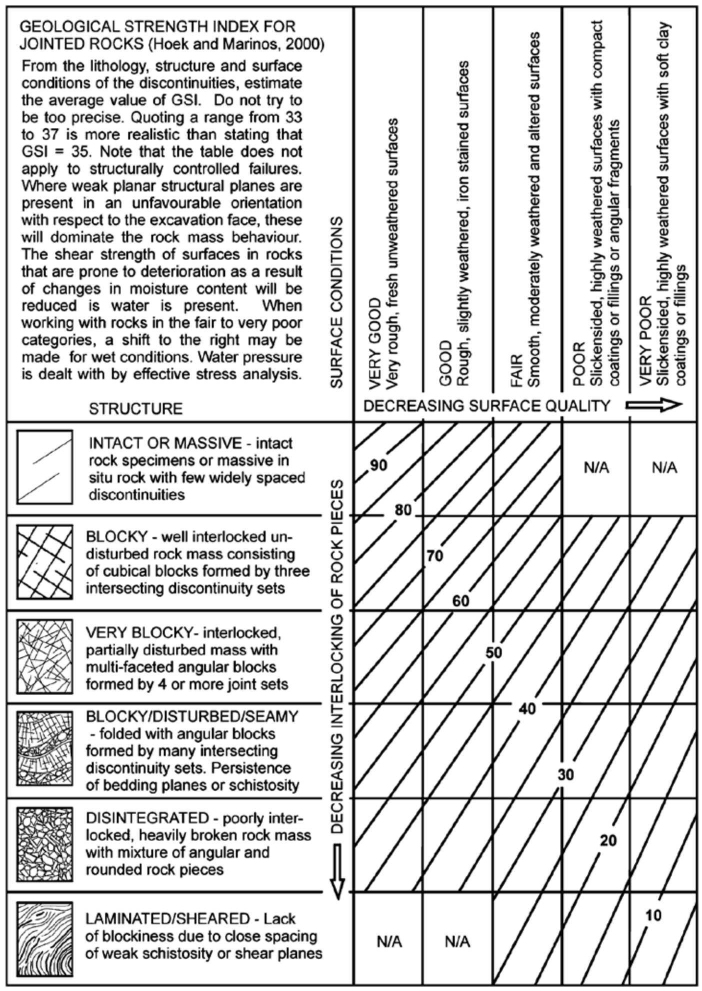
	~~Parameter~~		~~Ranges of Values~~						
~~1~~	~~Strength of intact rock material~~	~~Point load strength index~~	~~>175 ksf~~	~~85–175 ksf~~	~~45–85 ksf~~	~~20–45 ksf~~	~~For this low range, uniaxial compressive test is preferred~~		
		~~Uniaxial compressive strength~~	~~>4320 ksf~~	~~2160–4320 ksf~~	~~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~~

~~**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~~



<u>**Figure 10.4.6.4-1—Determination of GSI for Jointed Rock Mass (Hoek and Marinos, 2000)**</u>

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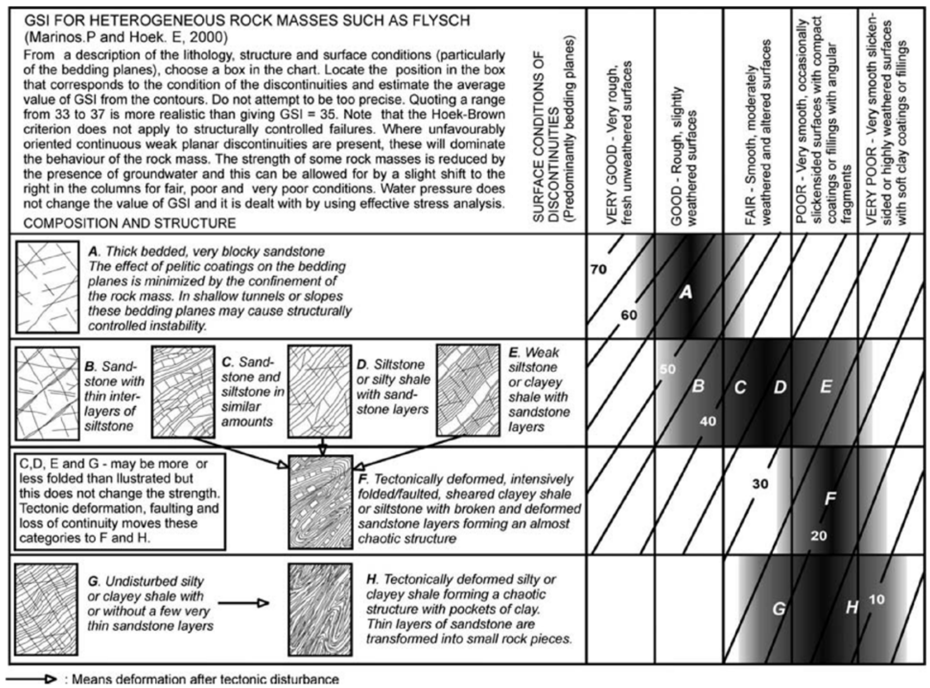

Figure 10.4.6.4-2—Determination of GSI for Tectonically Deformed Heterogeneous Rock Masses (Marinos and Hoek 2000)

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:~~

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

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

~~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.~~

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~~where:~~

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

~~ϕ′ᵢ = 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:

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

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

~~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 polymineralic 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~~ ~~3×10^{-6}~~	~~0.041~~ ~~3×10^{-6}~~	~~0.061~~ ~~3×10^{-6}~~	~~0.069~~ ~~3×10^{-6}~~	~~0.102~~ ~~3×10^{-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~~ ~~1×10^{-7}~~	~~0.010~~ ~~1×10^{-7}~~	~~0.015~~ ~~1×10^{-7}~~	~~0.017~~ ~~1×10^{-7}~~	~~0.025~~ ~~1×10^{-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:

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

~~where:~~

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

~~E_m ≤ 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 $q_u \le 100$ 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 $\times 10^3$) Maximum	Minimum	Mean	Standard Deviation (ksi $\times 10^3$)
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

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results is recommended.

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:

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

in which:

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

- For rectangular footings:

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

in which:

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

where:

q_o = applied vertical stress at base of loaded area (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.

<u>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).</u>

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~~ $\cancel{R_p}$ ~~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.~~

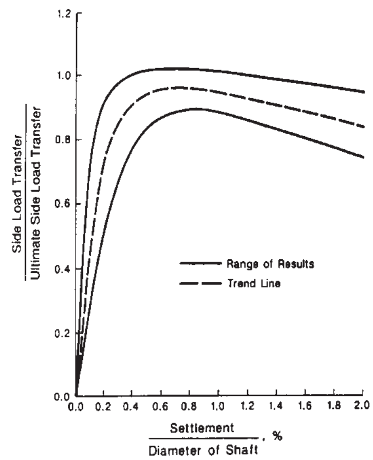


Figure 10.8.2.2.2-1 Normalized Load Transfer in Side Resistance versus Settlement in Cohesive Soils (from O'Neill and Reese, 1999)

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.

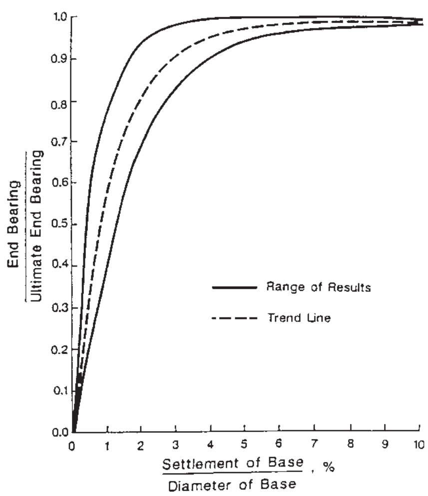


Figure 10.8.2.2.2-2—Normalized Load Transfer in End Bearing versus Settlement in Cohesive Soils (from O'Neill and Reese, 1999)

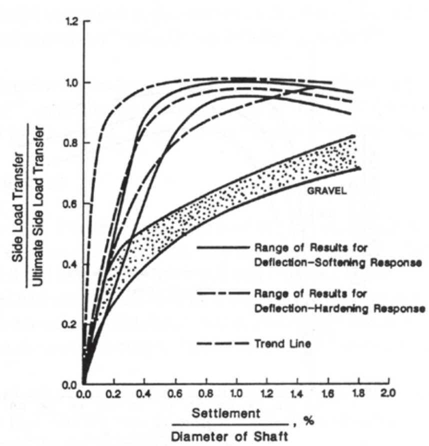


Figure 10.8.2.2.2-3—Normalized Load Transfer in Side Resistance versus Settlement in Cohesionless Soils (from O'Neill and Reese, 1999)

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).

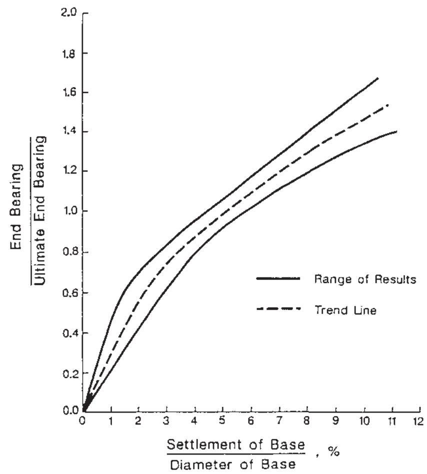


Figure 10.8.2.2.2-4—Normalized Load Transfer in End Bearing versus Settlement in Cohesionless Soils (from O'Neill and Reese, 1999)

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.

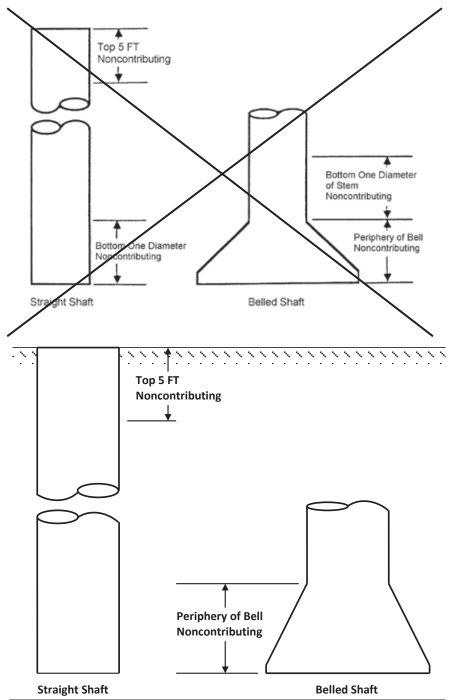


Figure 10.8.3.5.1b-1—Explanation of Portions of Drilled Shafts Not Considered in Computing Side Resistance (~~O'Neill and Reese, 1999~~ Brown et al., 2010)

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:

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

in which:

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

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:

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

~~in which, for sandy soils:~~

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

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

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

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

~~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.~~

~~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).~~

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

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

in which:

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

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:

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

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:

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

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:

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

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:

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

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

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 $SPT\text{-}N_{60}$ 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.

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.

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):~~

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

~~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):

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

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:

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

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 2.0*B* is jointed, the joints have random orientation, and the condition of the joints can be evaluated as:

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

~~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.

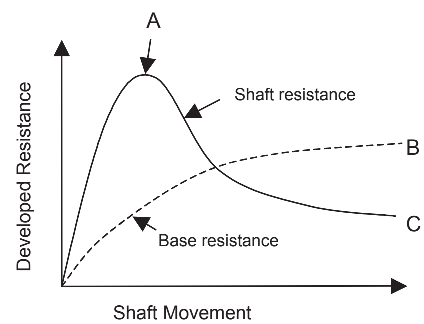
C10.8.3.5.4d

~~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.

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