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GEOTECHNICAL DESIGN MANUAL - PART 2 OF 4

Main Category:	Civil Engineering
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GEO-118 EXAM PREVIEW

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Exam Preview:

1. Many soil properties used for design are not intrinsic to the soil type but vary depending on conditions. In-situ stresses, changes in stresses, the presence of water, rate and direction of loading, and time can all affect the behavior of soils.
   a. True
   b. False
2. According to the reference material, if in-situ test methods are utilized to determine hydraulic conductivity, which of the following methods should NOT be used?
   a. Slug tests
   b. Packer permeability tests
   c. Piezocone tests
   d. Cone Penetrometer Test
3. According to the reference material, numerical models, used for back-analysis, typically only have 2 degrees of freedom, and high-quality input data is usually required to use such a complex tool for this purpose.
   a. True
   b. False
4. Using Table 5-1, which shows the Correlation of SPT N values to drained friction angle of granular soils, if the drained friction angle of the granular soil is between 35-40°, what is the corresponding SPT value in (blows/ft)?
   a. 4
   b. 10
   c. 30
   d. 50

5. According to the Table 6-2, which shows the Correlations for Estimating Initial Shear Modulus, which of the following is NOT a reference listed in this table?
    a. Jamiolkowski, et al.. (1991
    b. Hardy (1977)
    c. Mayne and Rix (1993)
    d. Imai and Tonouchi (1982)
6. Chapter 6, section 3.1 states that for essential or critical bridges, a two level seismic hazard design is required: the Safety Evaluation Earthquake (SEE) and the Functional Evaluation Earthquake (FEE).
    a. True
    b. False
7. According to the reference material, 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 ___ or if the soil is determined to be liquefiable for the return period of interest.
    a. 1.2
    b. 1.3
    c. 1.4
    d. 1.5
8. According to the reference material, for general slope stability analysis of permanent cuts, fills, and landslide repairs, a minimum safety factor of ___ should be used.
    a. 1.10
    b. 1.20
    c. 1.25
    d. 1.35
9. Table 8-1 provides a summary of information needs and testing considerations for foundation design. Which of the following foundation designs is NOT included in the reference material?
    a. Shallow Foundations
    b. Driven Pile Foundation
    c. Drilled Shaft Foundations
    d. Strip Foundations
10. According to the reference material, elasticity-based methods should be used to estimate the vertical stress increase in subsurface strata due to an embankment loading, or embankment load in combination with other surcharge loads.
    a. True
    b. False

Washington State
Department of Transportation

Geotechnical Design Manual

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PART 2 OF 4

Environmental and Regional Operations
Construction Division
Geotechnical Office

Chapter 5 Engineering Properties of Soil and Rock

5.1 Overview

The purpose of this chapter is to identify, either by reference or explicitly herein, appropriate methods of soil and rock property assessment, and how to use that soil and rock property data to establish the final soil and rock parameters to be used for geotechnical design. The final properties to be used for design should be based on the results from the field investigation, the field testing, and the laboratory testing, used separately or in combination. Site performance data should also be used if available to help determine the final geotechnical properties for design. The geotechnical designer's responsibility is to determine which parameters are critical to the design of the project and then determine those parameters to an acceptable level of accuracy. See Chapter 2, and the individual chapters that cover each geotechnical design subject area, for further information on what information to obtain and how to plan for obtaining that information.

5.2 The Geologic Stratum as the Basis for Property Characterization

The development of soil and rock properties for geotechnical design purposes begins with developing/defining the geologic strata present at the site in question. Therefore, the focus of geotechnical design property assessment and final selection shall be on the individual geologic strata identified at the project site. A geologic stratum is characterized as having the same geologic depositional history, stress history, and degree of disturbance, and generally has similarities throughout the stratum in terms of density, source material, stress history, hydrogeology, and macrostructure. The properties of each stratum shall be consistent with the stratum's geologic depositional and stress history, and macrostructure. Note that geologic units/formations identified in geologic maps may contain multiple geologic strata as defined in this GDM.

Once the geologic strata are defined, Engineering Stratagraphic Units (ESU's) are developed for the purpose of defining zones within the subsurface profile with similar properties for design. If there are multiple geologic strata as previously defined that have approximately the same engineering properties, multiple geologic strata may be grouped into a single ESU to simplify the design. However, soil and rock properties for design should not be averaged across multiple geologic strata except as noted later in this section, or unless averaging the properties results in an insignificant difference in the design outcome. If it is not clear that averaging the properties together will have an insignificant difference in the design outcome, the most conservative value of the property in question for the strata grouped together into one ESU should be used for design, or the strata should not be grouped together into one ESU.

The properties of a given geologic stratum at a project site may vary significantly from point to point within the stratum. In some cases, a measured property value may be closer in magnitude to the measured property value in an adjacent geologic stratum than to the measured properties at another point within the same stratum. It should also be recognized that some properties (e.g., undrained shear strength in normally consolidated clays) may vary as a function of a stratum dimension (e.g., depth below the top of the stratum). Where the property within the stratum varies in this manner, the design parameters shall be developed taking this variation into account, which may result in multiple values of the property within the stratum and therefore multiple ESU's within the stratum.

Since ESU's are defined as zones of soil or rock with consistent engineering properties, properties of ESU's shall not be averaged together, except as noted in the following sentences. For design methods that require a very simplified stratigraphy be used, to create the simplified stratigraphy, a weighted average of the properties from each ESU based on the design ESU thickness should be used to estimate the properties of the simplified ESU for the design method in question. An example of this approach is provided in the AASHTO LRFD Bridge Design Specifications, Article C3.10.3.1, in particular Table 1 of that article. However, there is a significant risk that weaker materials, seams, layers, or structures (e.g., fractures, fissures, slickensides) within a stratum or ESU will dominate the performance of the geotechnical structure being designed, the design properties selected shall reflect the weakest aspects of the stratum or ESU rather than taking a weighted average.

5.3 Influence of Existing and Future Conditions on Soil and Rock Properties

Many soil properties used for design are not intrinsic to the soil type, but vary depending on conditions. In-situ stresses, changes in stresses, the presence of water, rate and direction of loading, and time can all affect the behavior of soils. Prior to evaluating the properties of a given soil, it is important to determine the existing conditions as well as how conditions may change over the life of the project. Future construction such as new embankments may place new surcharge loads on the soil profile or the groundwater table could be raised or lowered. Often it is necessary to determine how subsurface conditions or even the materials themselves will change over the design life of the facility that is constructed. Normally consolidated clays can gain strength with increases in effective stress, and overconsolidated clays may lose strength with time when exposed in cuts, unloaded, or exposed to water. Some construction materials such as weak rock may lose strength due to weathering within the design life of the embankment. These long-term effects shall be considered when selecting properties to use for design.

5.4 Methods of Determining Soil and Rock Properties

Subsurface soil or rock properties are generally determined using one or more of the following methods:

- in-situ testing during the field exploration program;
- laboratory testing, and
- back-analysis based on site performance data

The two most common in-situ test methods for use in soil are the Standard Penetration Test, (SPT) and the cone penetrometer test (CPT). Section 5.4 describes these tests as well as other in-situ tests. The laboratory testing program generally consists of index tests to obtain general information or to use with correlations to estimate design properties, and performance tests to directly measure specific engineering properties.

Back-analysis is used to tie the soil or rock properties to the quantifiable performance of the slope, embankment, wall, or foundation (see Section 5.7).

The detailed measurement and interpretation of soil and rock properties shall be consistent with the guidelines provided in FHWA-IF-02-034, *Evaluation of Soil and Rock Properties*, Geotechnical Engineering Circular No. 5 (Sabatini, et al., 2002), except as specifically indicated herein.

5.5 In-Situ Field Testing

Standards and details regarding field tests such as the Standard Penetration Test (SPT), the Cone Penetrometer Test (CPT), the vane shear test, and other tests and their use provided in Sabatini et al. (2002) should be followed, except as specifically noted herein. Regarding Standard Penetration Tests (SPT), the N-values obtained in the field depend on the equipment used and the skill of the operator, and shall be corrected before they are used in design so that they are consistent with the design method and correlations being used. Many of the correlations developed to determine soil properties are based on N60-values.

SPT N values shall be corrected for hammer efficiency, if applicable to the design method or correlation being used, using the following relationship.

$$N_{60} = (ER/60\%)\ N \qquad (5\text{-}1)$$

Where:

N = uncorrected SPT value (blows/ft)
N60 = SPT blow count corrected for hammer efficiency (blows/ft)
ER = Hammer efficiency expressed as percent of theoretical free fall energy delivered by the hammer system actually used.

The following values for ER may be assumed if hammer specific data are not available:

ER = 60% for conventional drop hammer using rope and cathead
ER = 80% for automatic trip hammer

Hammer efficiency (ER) for specific hammer systems used in local practice may be used in lieu of the values provided. If used, specific hammer system efficiencies shall be developed in general accordance with ASTM D-4633 for dynamic analysis of driven piles or other accepted procedure. See Chapter 3 for additional information on ER, including specific measurements conducted for WSDOT drilling equipment.

Corrections for rod length, hole size, and use of a liner may also be made if appropriate. In general, these are only significant in unusual cases or where there is significant variation from standard procedures. These corrections may be significant for evaluation of liquefaction. Information on these additional corrections may be found in: “Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils”; Publication Number: MCEER-97-0022; T.L. Youd, I.M. Idriss (1997).

N-values are also affected by overburden pressure, and shall be corrected for that effect, if applicable to the design method or correlation being used. N values corrected for both overburden and the efficiency of the field procedures used shall be designated as N160. The overburden correction equation that should be used is:

$$N1_{60} = C_N N_{60} \tag{5-2}$$

Where:

C_N = [0.77 log10 (20/σ′v)], CN < 2.0 (5-3)
C_N = correction factor for overburden
N_{60} = N-value corrected for energy efficiency
σ'_v = vertical effective stress at the location of the SPT N-value (TSF)

In general, correlations between N-values and soil properties should only be used for cohesionless soils, and sand in particular. Caution should be used when using N-values obtained in gravelly soil. Gravel particles can plug the sampler, resulting in higher blow counts and estimates of friction angles than actually exist. Caution should also be used when using N-values to determine silt or clay parameters, due to the dynamic nature of the test and resulting rapid changes in pore pressures and disturbance within the deposit. Correlations of N-values with cohesive soil properties should generally be considered as preliminary. N-values can also be used for liquefaction analysis. See Chapter 6 for more information regarding the use of N-values for liquefaction analysis.

In general design practice, hydraulic conductivity is estimated based on grain size characteristics of the soil strata (see Highway Runoff Manual M 31-16, Section 4-5). In critical applications, the hydraulic conductivity may be determined through in-situ testing. A discussion of field measurement of permeability is presented in Sabatini et al. (2002) and Mayne et al. (2002), and ASTM D 4043 presents a guide for the selection of various field methods. If in-situ test methods are utilized to determine hydraulic conductivity, one or more of the following methods should be used:

- Well pumping tests
- Packer permeability tests
- Seepage Tests
- Slug tests
- Piezocone tests
- Flood tests or Pit Infiltration Tests (PIT) – applies mainly to infiltration facility design – see Section 4-5 of the Highway Runoff Manual (2004) M 31-16.

5.5.1 Well Pumping Tests

Pump tests can be used to provide an estimate of the overall hydraulic conductivity of a geologic formation, and since it is in essence a full scale test, it directly accounts for the layering and directionality of the hydraulic characteristics of the formation. The data provided can be used to determine the requirements for construction dewatering systems for excavations. However, pump tests can be quite expensive and can take a significant amount of time to complete. Furthermore, care must be exercised when conducting this type of test, especially if potentially contaminated zones are present that could be mobilized during pumping. This could also create problems with disposal of the pumped water. Impact to adjacent facilities, such as drinking wells and subsidence caused by dewatering, should be evaluated when planning this type of test. For this test, the method prescribed in ASTM D 4050 should be used. Analysis of the results of pumping tests requires experience and a thorough knowledge of the actual geologic conditions present at the test location. The time-drawdown response curves are unique to a particular geologic condition. Therefore, knowledge of the actual geologic conditions present at the test location is required in order to choose the correct analysis procedure, e.g., whether the aquifer is leaky, unconfined, or bounded, etc.

5.5.2 Packer Permeability Tests

Packer permeability tests can be used to measure the hydraulic conductivity of a specific soil or rock unit. The information obtained is used primarily in seepage studies. This test is conducted by inserting the packer units to the desired test location after the boring has been properly cleaned out. The packers are expanded to seal off the zone being tested, and water is injected into the borehole under constant pressure. Measurements of the flow rate are taken at regular time intervals. Upon completion of testing at a particular depth, the packers are lowered to a new test depth. Test depths should be determined from cores and geophysical logs of the borehole, prior to hydraulic conductivity testing. Note that if the packer test is performed in soil borings, casing must be installed. See Mayne et al. (2002) for additional information on this type of test.

5.5.3 Seepage Tests

Three types of seepage tests are commonly used: falling head, rising head and constant water level methods. In general, either the rising or falling level methods should be used if the hydraulic conductivity is low enough to permit accurate determination of the water level. In the falling head method, the borehole or piezometer is filled with water that is allowed to seep into the soil. The rate of drop of the water surface in the casing is monitored. The rising head method consists of bailing the water out of the borehole and observing the rate of rise until the change becomes negligible. The constant water level method is used if soil is too permeable to allow accurate measurement of the rising or falling water level. General guidance on these types of tests is provided in Mayne et al. (2002).

Boreholes (or in subsequently installed piezometers) in which seepage tests are to be performed should be drilled using only clear water as the drilling fluid. This precludes the formation of a mud cake on the walls of the hole or clogging of the soil pores with drilling mud. The tests can be performed intermittently as the borehole is advanced. In general, the rising head test is preferred because there is less chance of clogging soil pores with suspended sediment.

Data from seepage tests only reflect the hydraulic conditions near the borehole. In addition the actual area of seepage at the base of the borehole may not be accurately known. During the rising head test, there is the danger of the soil at the bottom of the borehole becoming loosened or "quick" if too great a gradient is imposed. However, seepage tests can be used in soils with lower hydraulic conductivities than is generally considered suitable for pumping tests and if large volumes of water do not require disposal. Also note that if the test is conducted inside the piezometer, the hydraulic conductivity measured from this could be influenced by the material placed inside the borehole around the screened pipe.

5.5.4 Slug Tests

These tests are easy to perform and can be performed in a borehole in which a screened pipe is installed. Two types of slug tests are commonly used, falling head and rising head. Falling head slug tests are conducted by lowering a solid object such as a weighted plastic cylinder into the borehole causing an instantaneous water level rise. As the water level gradually returns to static, the rate is recorded. A rising head slug test can then be performed by suddenly removing the slug, causing an instantaneous lowering of the water level. By monitoring the rate of rise or fall of the water level in the borehole, an estimate of the hydraulic conductivity can be determined. For this test, the method prescribed in ASTM D 4044 should be used. However slug tests are not very reliable and may underestimate hydraulic conductivity by one or two orders of magnitude, particularly if the test well has been inadequately developed prior to testing. The test data will not provide an indication of the accuracy of the computed value unless a pumping test is done in conjunction with the slug test. Because the slug tests are short duration, they reflect hydraulic properties of the soil immediately surrounding the well intake.

5.4.5 Piezocone Tests

Details of the equipment and methodology used to conduct the piezocone test are provided in Sabatini et al. (2002). Piezocone data can be useful to estimate the hydraulic conductivity of silts and clays from interpretation of the coefficient of horizontal consolidation, ch, obtained from the piezocone measurements. The procedure involves pushing the cone to the desired depth, followed by recording pore pressures while the cone is held stationary. The test is usually run until 50 percent of the excess pore pressure has dissipated (t50). This requires knowledge of the initial in situ pore pressure at the test location. Dissipation tests are generally effective in silts and clays where large excess pore pressures are generated during insertion of the cone. Hydraulic conductivity can be estimated using various correlations with t50 and coefficient of horizontal consolidation (ch), (see Lunne et al. (1997), and Sabatini et al. (2002)). Estimation of hydraulic conductivity from CPT tests is subject to a large amount of uncertainty, and should be used only as a preliminary estimate of permeability.

5.5.6 Flood Tests

Flood tests or pilot infiltration tests are not always feasible, and in general are only used where unusual site conditions are encountered that are poorly modeled by correlation to soil gradation characteristics, and there is plenty of water available to conduct the test. The key to the success of this type of test is the estimate of the hydraulic gradient during the test, recognizing that the test hydraulic gradient could be much higher than the hydraulic gradient that is likely in service for the facility being designed. For more information, see the Highway Runoff Manual (2004).

5.6 Laboratory Testing of Soil and Rock

Laboratory testing is a fundamental element of a geotechnical investigation. The ultimate purpose of laboratory testing is to utilize repeatable procedures to refine the visual observations and field testing conducted as part of the subsurface field exploration program, and to determine how the soil or rock will behave under the imposed conditions. The ideal laboratory program will provide sufficient data to complete an economical design without incurring excessive tests and costs. Depending on the project issues, testing may range from simple soil classification testing to complex strength and deformation testing.

5.6.1 Quality Control for Laboratory Testing

Improper storage, transportation and handling of samples can significantly alter the material properties and result in misleading test results. The requirements provided in Chapter 3 regarding these issues shall be followed.

Laboratories conducting geotechnical testing shall be either AASHTO accredited or fulfill the requirements of AASHTO R18 for qualifying testers and calibrating/verifications of testing equipment for those tests being performed. In addition, the following guidelines (Mayne et al., 1997) for laboratory testing of soils should be followed:

1. Protect samples to prevent moisture loss and structural disturbance.
2. Carefully handle samples during extrusion of samples; samples must be extruded properly and supported upon their exit from the tube.
3. Avoid long-term storage of soil samples in Shelby tubes.
4. Properly number and identify samples.
5. Store samples in properly controlled environments.
6. Visually examine and identify soil samples after removal of smear from the sample surface.
7. Use pocket penetrometer or miniature vane only for an indication of strength.
8. Carefully select “representative” specimens for testing.
9. Have a sufficient number of samples to select from.
10. Always consult the field logs for proper selection of specimens.
11. Recognize disturbances caused by sampling, the presence of cuttings, drilling mud or other foreign matter and avoid during selection of specimens.
12. Do not depend solely on the visual identification of soils for classification.
13. Always perform organic content tests when classifying soils as peat or organic. Visual classifications of organic soils may be very misleading.
14. Do not dry soils in overheated or underheated ovens.
15. Discard old worn-out equipment; old screens for example, particularly fine (< No. 40) mesh ones need to be inspected and replaced often, worn compaction mold or compaction hammers (an error in the volume of a compaction mold is amplified 30x when translated to unit volume) should be checked and replaced if needed.
16. Performance of Atterberg limits requires carefully adjusted drop height of the Liquid Limit machine and proper rolling of Plastic Limit specimens.
17. Do not use tap water for tests where distilled water is specified.
18. Properly cure stabilization test specimens.

19. Never assume that all samples are saturated as received.
20. Saturation must be performed using properly staged back pressures.
21. Use properly fitted o-rings, membranes, etc. in triaxial or permeability tests.
22. Evenly trim the ends and sides of undisturbed samples.
23. Be careful to identify slickensides and natural fissures. Report slickensides and natural fissures.
24. Also do not mistakenly identify failures due to slickensides as shear failures.
25. Do not use unconfined compression test results (stress-strain curves) to determine elastic modulus values.
26. Incremental loading of consolidation tests should only be performed after the completion of each primary stage.
27. Use proper loading rate for strength tests.
28. Do not guesstimate e-log p curves from accelerated, incomplete consolidation tests.
29. Avoid "Reconstructing" soil specimens, disturbed by sampling or handling, for undisturbed testing.
30. Correctly label laboratory test specimens.
31. Do not take shortcut: such as using non-standard equipment or non-standard test procedures.
32. Periodically calibrate all testing equipment and maintain calibration records.
33. Always test a sufficient number of samples to obtain representative results in variable material.

5.6.2 Developing the Testing Plan

The amount of laboratory testing required for a project will vary depending on availability of preexisting data, the character of the soils and the requirements of the project. Laboratory tests should be selected to provide the desired and necessary data as economically as possible. Specific geotechnical information requirements are provided in the GDM chapters that address design of specific types of geotechnical features. Laboratory testing should be performed on both representative and critical test specimens obtained from geologic layers across the site. Critical areas correspond to locations where the results of the laboratory tests could result in a significant change in the proposed design. In general, a few carefully conducted tests on samples selected to cover the range of soil properties with the results correlated by classification and index tests is the most efficient use of resources.

The following should be considered when developing a testing program:

- Project type (bridge, embankment, rehabilitation, buildings, etc.)
- Size of the project
- Loads to be imposed on the foundation soils
- Types of loads (i.e., static, dynamic, etc.)
- Whether long-term conditions or short-term conditions are in view

- Critical tolerances for the project (e.g., settlement limitations)
- Vertical and horizontal variations in the soil profile as determined from boring logs and visual identification of soil types in the laboratory
- Known or suspected peculiarities of soils at the project location (i.e., swelling soils, collapsible soils, organics, etc.)
- Presence of visually observed intrusions, slickensides, fissures, concretions, etc. in sample – how will it affect results
- Project schedules and budgets
- Input property data needed for specific design procedures

Details regarding specific types of laboratory tests and their use are provided in Sabatini et al. (2002). Specifics regarding what is required in a laboratory testing plan is provided in Section 2.4.

5.7 Back-Analysis Based on Known Performance or Failure

Back-analysis to determine engineering properties of soil or rock is most often used with geotechnical failures. When failures occur, back analysis can be used to model the conditions, and loads which resulted in failure. Back-analysis can also be used in some situations where failure has not occurred but the geotechnical performance can be quantified (e.g., deformations). Back-analysis is a quantitative approach to adjust soil or rock properties to match measureable site performance.

To successfully carry out this approach, it is important to define the site geometry and stratigraphy, geologic history of the subsurface strata to be encountered, loading conditions, ground water conditions, and measurable soil properties. Since there are typically a number of variables to consider in most back-analyses (e.g., soil shear strength and unit weight of each stratum/ESU, the stratigraphy itself, the groundwater regime, the failure or deformation mechanism, the amount of deformation that has occurred, the location of the failure surface, the loading that occurred at the time the observed behavior occurred, etc.), all of the variables need to be defined before conducting the back-analysis so that the parameter of interest can be determined in a meaningful way.

Transient loading such as construction equipment live load shall not be included in the back-analysis, unless the transient load clearly caused failure (i.e., slope failed while equipment was on slope). If transient loads are included in the back-analysis, the rate of loading and its effect on the soil properties shall be addressed in the analysis.

To that end, the parameters used for the back-analysis shall be determined in a way that is consistent with the requirements provided in this manual. The back-analysis is then used to adjust the parameter of interest so that predicted behavior is consistent with the observed behavior. The observed behavior must be measurable in some way so that consistency between the observed and predicted behavior is quantifiably recognizable. If the behavior/performance is not quantifiable, then back-analysis will not be meaningful for determining or verifying design parameters.

If a back-analysis is to be conducted, the considerations and recommendations provided by Duncan and Stark (1992) shall be used. While the Duncan and Stark paper was written with regard to application to back-analysis of slope failures, the principles provided are generally applicable to other back-analysis situations.

5.7.1 Back-Analysis of Slopes

With landslides or slope failures, if the factor of safety for the slope is to be used as the performance measurement, a slope factor of safety of 1.0 shall be used, and shall accurately model the failure surface geometry and failure mechanism (Turner and Schuster 1996). It is important to determine or estimate the conditions that initiated the slope failure to successfully back-analyze the slope failure. See Stark, et al. (2011) for the principles that should be used to conduct slope failure back-analyses and a detailed example.

For first time slides, and slides in which the total historical deformation is relatively small, it shall be recognized that the shear strength estimated from the back-analysis is the mobilized shear strength at time of failure, not necessarily the residual shear strength, as the full development of residual strength conditions depends on the amount of deformation that has occurred along the slide failure surface (Hussain et al. 2010, Stark et al. 2011). In first time slides, the back-calculated shear strength is likely to be closer to the fully softened shear strength than the residual shear strength. Laboratory shear strength testing to measure the residual shear strength of the deposit should also be conducted and used in combination with the back-analyzed parameters for design purposes.

5.7.2 Back-Analysis of Soil Settlement Resulting from Changes in Loading

For embankment settlement, the performance measurement to be used is typically the magnitude of settlement measured, the rate at which the settlement occurred, or both. Pore pressure changes that occurred during embankment placement may also be used to help assess the rate of strength gain in soft compressible soils. If the embankment is reinforced with geosynthetic, strain in the geosynthetic should also be measured and used for back-analysis purposes. Monitoring of fill settlement and pore pressure in the soil during construction allows the soil properties and prediction of the rate of future settlement to be refined. For structures such as bridges that experience unacceptable settlement or retaining walls that have excessive deflection, the engineering properties of the soils can be determined if the magnitudes of the loads and structural details are known. As with slope stability analysis, the stratigraphy of the subsurface soil must be adequately known, including the history of the groundwater level at the site.

5.7.3 Back-Analysis of Foundations

Essentially, use of foundation load tests to measure foundation bearing resistance and deflection characteristics is a form of back-analysis, when such data is used to estimate soil properties, enabling the prediction of foundation performance in adjacent areas where the same soil or rock strata are encountered, but the thickness of the strata/ESU's are different.

5.7.4 Use of Numerical Modeling for Back-Analysis

Numerical models typically have many degrees of freedom, and high quality input data is usually required to use such a complex tool for this purpose. If numerical models are used, they shall have gone through a calibration process for a similar situation. Approval by the WSDOT State Geotechnical Engineer is required for use of numerical modeling techniques for the purpose of back-analysis to estimate soil or rock properties. Approval will be based on the adequacy of the numerical model calibration, how well the performance to be modeled is defined and quantified, and how well the variables/input parameters in the model are defined and measured such that a unique value of the parameter of interest can be accurately estimated.

5.8 Engineering Properties of Soil

5.8.1 Laboratory Index Property Testing

Laboratory index property testing is mainly used to classify soils, though in some cases, they can also be used with correlations to estimate specific soil design properties. Index tests include soil gradation and plasticity indices. For soils with greater than 10 percent passing the No. 200 sieve, a decision will need to be made regarding the full soil gradation curve and whether a hydrometer test in addition to sieve testing of the coarser particles (AASHTO T88) is necessary, or if a coarse gradation is sufficient (AASHTO T27). The full gradation range (AASHTO T88) will be needed in the following situations:

- Lateral load analysis of deep foundations using strain wedge theory
- Liquefaction analysis
- Infiltration design, or other analyses that require the determination of hydraulic conductivities
- Other analyses that require a d_{10} size, coefficient of uniformity, etc.

Classification using the coarse sieving only (AASHTO T27) may be adequate for design of MSE walls, general earthwork, footing foundations, gravity walls, and noise walls. These end use needs shall be considered when planning the laboratory investigation for a project.

5.8.2 Laboratory Performance Testing

Laboratory performance testing is mainly used to estimate strength, compressibility, and permeability characteristics of soil and rock. For rock, the focus of laboratory performance testing is typically on the shear strength of the intact rock, or on the shear strength of specific discontinuities (i.e., joint/seam) within the rock mass. See Section 5.9 for additional discussion on rock properties. Soil shear strength may be determined on either undisturbed specimens of finer grained soil (undisturbed specimens of granular soils are very difficult, if not impossible, to get), or disturbed or remolded specimens of fine or coarse grained soil. There are a variety of shear strength tests that can be conducted, and the specific type of test selected depends on the specific application. See Sabatini et al. (2002) for specific guidance on the types of shear strength tests needed for various applications, as well as the chapters in the GDM that cover specific geotechnical design topics.

Disturbed soil shear strength testing is less commonly performed, and is primarily used as supplementary information when performing back-analysis of existing slopes, or for fill material and construction quality assurance when a minimum shear strength is required. It is difficult to obtain very accurate shear strength values of soils in natural deposits through shear strength testing of disturbed (remolded) specimens, since the in-situ density and soil structure is quite difficult to accurately recreate, especially considering the specific in-situ density may not be known. The accuracy of this technique in this case must be recognized when interpreting the results. However, for estimating the shear strength of compacted backfill, more accurate results can be obtained, since the soil placement method, as well as the in-situ density and moisture content, can be recreated in the laboratory with some degree of confidence. The key in the latter case is the specimen size allowed by the testing device, as in many cases, compacted fills have a significant percentage of gravel sized particles, requiring fairly large test specimens <u>and test apparatus</u> (i.e., minimum 3 to 4 inch diameter, or narrowest dimension specimens of 3 to 4 inches).

Typically, a disturbed sample of the granular backfill material (or native material in the case of obtaining supplementary information for back-analysis of existing slopes) is sieved to remove particles that are too large for the testing device and test standard, and is compacted into a mold to simulate the final density and moisture condition of the material. The specimens may or may not be saturated after compacting them and placing them in the shear testing device, depending on the condition that is to be simulated. In general, a drained test is conducted, or if it is saturated, the pore pressure during shearing can be measured (possible for triaxial testing; generally not possible for direct shear testing) to obtained drained shear strength parameters. Otherwise, the test is run slow enough to be assured that the specimen is fully drained during shearing (note that estimating the testing rate to assure drainage can be difficult). Multiple specimens using at least three confining pressures should be tested to obtain a shear strength envelope. See Sabatini et al. (2002) for additional details.

Tests to evaluate compressibility or permeability of existing subsurface deposits must be conducted on undisturbed specimens, and the less disturbance the better. See Sabatini et al. (2002) for additional requirements regarding these and other types of laboratory performance tests that should be followed.

The hydraulic conductivity of a soil is influenced by the particle size and gradation, the void ratio, mineral composition, and soil fabric. In general the hydraulic conductivity, or permeability, increases with increasing grain size; however, the size and shape of the voids also <u>have</u> a significant influence. The smaller the voids <u>are</u>, the lower the permeability. Mineral composition and soil fabric have little effect on the permeability of gravel, sand, and non-plastic silt, but are important for plastic silts and clays. Therefore, relationships between particle size and permeability are available for coarse-grained materials, some of which are presented in the Correlations subsection (Section 5.6.2). In general, for clays, the lower the ion exchange capacity of the soil, the higher the permeability. Likewise, the more flocculated (open) the structure, the higher the permeability.

The methods commonly used to determine the hydraulic conductivity in the laboratory include, the constant head test, the falling head test, and direct or indirect methods during a consolidation test. The laboratory tests for determining the hydraulic conductivity are generally considered quite unreliable. Even with considerable attention to test procedures and equipment design, tests may only provide values within an order of magnitude of actual conditions. Some of the factors for this are:

- The soil in-situ is generally stratified and this is difficult to duplicate in the laboratory.
- The horizontal value of k is usually needed, but testing is usually done on tube samples with vertical values obtained.

- In sand, the horizontal and vertical values of k are significantly different, often on the order of $k_h = 10$ to $100k_v$.
- The small size of laboratory samples leads to boundary condition effects.
- Saturated steady-state soil conditions are used for testing, but partially saturated soil water flow often exists in the field.
- On low permeability soils, the time necessary to complete the tests causes evaporation and equipment leaks to be significant factors.
- The hydraulic gradient in the laboratory is often 5 or more to reduce testing time, whereas in the field it is more likely in the range of 0.1 to 2.

The hydraulic conductivity is expected to vary across the site; however, it is important to differentiate errors from actual field variations. When determining the hydraulic conductivity, the field and laboratory values should be tabulated along with the other known data such as sample location, soil type, grain-size distribution, Atterberg limits, water content, stress conditions, gradients, and test methods. Once this table is constructed, it will be much easier to group like soil types and k values to delineate distinct areas within the site, and eliminate potentially erroneous data.

5.8.3 Correlations to Estimate Engineering Properties of Soil

Correlations that relate in-situ index test results such as the SPT or CPT or laboratory soil index testing may be used in lieu of or in conjunction with performance laboratory testing and back-analysis of site performance data to estimate input parameters for the design of the geotechnical elements of a project. Since properties estimated from correlations tend to have greater variability than measurement using laboratory performance data (see Phoon et al., 1995), properties estimated from correlation to in-situ field index testing or laboratory index testing should be based on multiple measurements within each geologic unit (if the geologic unit is large enough to obtain multiple measurements). A minimum of 3 to 5 measurements should be obtained from each geologic unit as the basis for estimating design properties.

The drained friction angle of granular deposits estimated from SPT measurements shall be determined based on the correlation provided in Table 5-1.

$N1_{60}$ from SPT (blows/ft)	φ (o)
<4	25-30
4	27-32
10	30-35
30	35-40
50	38-43

Correlation of SPT N values to drained friction angle of granular soils (modified after Bowles, 1977 as reported in AASHTO 2012)
Table 5-1

The correlation used is modified after Bowles (1977). The correlation of Peck, Hanson and Thornburn (1974) falls within the ranges specified. Experience should be used to select specific values within the ranges. In general, finer materials, materials with significant silt-sized material, and materials in which the particles are rounded to sub-rounded will fall in the lower portion of the range. Coarser materials with less then 5% fines, and materials in which the particles are sub-angular to angular will fall in the upper portion of the range.

Care should be exercised when using other correlations of SPT results to soil parameters. Some published correlations are based on corrected values (N160) and some are based on uncorrected values (N). The designer shall ascertain the basis of the correlation and use either N160 or N as appropriate. Care shall also be exercised when using SPT blow counts to estimate soil shear strength for soils with gravel, cobbles, or boulders. Gravels, cobbles, or boulders could cause the SPT blow counts to be unrealistically high.

Correlations for other soil properties (other than as specifically addressed above for the soil friction angle) as provided in Sabatini et al. (2002) may be used if the correlation is widely accepted and if the accuracy of the correlation is known. However, such correlations shall not be extrapolated to estimate properties beyond the range of the empirical data used to establish the correlation. Care shall also be exercised when using correlations near the extremities of the empirical basis for the correlations, and the resulting additional uncertainty in the estimated properties shall be addressed in the design in which those properties are used. Local geologic formation-specific correlations may be used if well established by: (1) data comparing the prediction from the correlation to measured high quality laboratory performance data, or (2) back-analysis from full-scale performance of geotechnical elements affected by the geologic formation in question.

Regarding soil hydraulic conductivity, the correlations provided in the Highway Runoff Manual, should be used.

5.9 Engineering Properties of Rock

Engineering properties of rock are controlled by the discontinuities within the rock mass and the properties of the intact rock. Therefore, engineering properties for rock must account for the properties of the intact rock and for the properties of the rock mass as a whole, specifically considering the discontinuities within the rock mass. A combination of laboratory testing of small samples, empirical analysis, and field observations should be employed to determine the requisite engineering properties.

Rock properties can be divided into two categories: intact rock properties and rock mass properties. Intact rock properties are determined from laboratory tests on small samples typically obtained from coring, outcrops or exposures along existing cuts. Common engineering properties typically obtained from laboratory tests include specific gravity, point load strength, compressive strength, tensile strength, shear strength, modulus, and slake durability. Rock mass properties are determined by visual examination and description of discontinuities within the rock mass following the suggested methodology of the International Society of Rock Mechanics (ISRM 1978), and how these discontinuities will affect the behavior of the rock mass when subjected to the proposed construction and loading.

Point load tests should be calibrated to unconfined compression strength test results on the same rock type . Point load tests shall not be used for weak to extremely rock (R0, R1, and R2 rock) with uniaxial compressive strength less than 3600 psi (25 MPa).

The methodology and related considerations provided by Sabatini et al. (2002) should be used to assess the design properties for the intact rock and the rock mass as a whole. However, the portion of Sabatini et al. (2002) that addresses the determination of fractured rock mass shear strength parameters (Hoek and Brown 1988) using the Rock Mechanics Rating (RMR) system is outdated. The original work by Hoek and Brown has been updated and is described in Hoek et al. (2002). The updated method uses a Geological Strength Index (GSI) to characterize the rock mass for the purpose of estimating shear strength parameters, and has been developed based on re-examination of hundreds of tunnel and slope stability analyses in which both the 1988 and 2002 criteria were used and compared to field results. While the 1988 method has been more widely published in national (e.g., FHWA) design manuals than has the updated approach provided in Hoek et al. (2002), considering that the original developers of the method have recognized the short-comings of the 1988 method and have reassessed t through comparison to actual rock slope stability data, WSDOT considers the Hoek, et al. (2002) to be the most accurate methodology. Therefore the Hoek et al. (2002) method should be used for fractured rock mass shear strength determination. Note that this method is only to be used for fractured rock masses in which the stability of the rock slope, or rock surrounding the foundation is not structurally controlled. See Chapter 12 for additional requirements regarding the assessment of rock mass properties.

Some design methods were specifically developed using the older Hoek and Brown (1988) RMR method, such as the design of spread footings on rock in the AASHTO LRFD Bridge Design manual (specifically Article 10.6.3.2). In such cases, the older Hoek and Brown method shall be used until such time that the design procedure has been updated to use the newer GSI index method.

5.10 Determination and Use of Soil Cohesion

Soil cohesion is defined as shear strength resulting from inter-particle attraction effect that is independent of normal stress but varies considerably with water content and rate of loading (Bowles 1979).

The use of cohesion due to inter-particle attraction, such as occurs in clays and clayey silts, for design shall be considered cautiously for long-term design and in general shall not be fully relied upon for long-term loading, unless local experience indicates that a particular value of cohesion in a given geologic unit can be relied upon (note: evidence of that local experience, such as results from previous back-analyses that demonstrate good long-term performance can be reliably achieved, shall be included in the calculation package). If cohesion is used in such cases, it shall be a conservative lower bound value. It is especially important to not rely upon cohesive shear strength if displacement in the soil has occurred in the past or potentially could occur in the future, in fractured or fissured soil, or if moisture content changes over time could occur. In these cases, a drained cohesion value near zero shall be used. For short-term applications, such as in temporary cuts or walls, or during seismic loading, some soil cohesion may be considered for use in design, provided that potential displacement and water content changes are adequately controlled or taken into account. To justify the use of cohesion where structures (e.g., anchored walls) are used to restrain or prevent soil deformation, a deformation analysis of the restraining system shall be conducted to demonstrate that the deformation will be adequately controlled.

Apparent cohesion is defined as the cohesion that results from surface tension due to moisture in unsaturated, but not dry, soils, primarily in sands and non-plastic silts. Apparent cohesion shall not be relied upon for the design of permanent works. For temporary works, apparent cohesion may only be used if the moisture content of the soils can be preserved or controlled and the magnitude of the apparent cohesion is conservatively assessed.

For sands and gravels with 10% fines or less by weight, cohesion shall not be relied upon for both short-term and long-term design situations, as in most cases, most of the cohesion that may be present is apparent cohesion, which is not a reliable source of shear strength.

5.11 Final Selection of Design Values

5.11.1 Overview

After the field and laboratory testing is completed, the geotechnical designer shall review the quality and consistency of the data, and shall determine if the results are consistent with expectations. Once the lab and field data have been collected, the process of final material property selection begins. At this stage, the geotechnical designer generally has several sources of data consisting of that obtained in the field, laboratory test results, and correlations from index testing. In addition, the geotechnical designer may have results of back- analyses, or have experience based on other projects in the area or in similar soil/rock conditions. Therefore, if the results are not consistent with each other or previous experience, the reasons for the differences shall be evaluated, poor data eliminated and trends in data identified. At this stage it may be necessary to conduct additional performance tests to try to resolve discrepancies.

As stated in Section 5.1, the focus of geotechnical design property assessment and final selection is on the individual geologic strata identified at the project site. A geologic stratum is characterized as having the same geologic depositional history, stress history, and degree of disturbance, and generally has similarities throughout the stratum in its density, source material, stress history, and hydrogeology. All of the information that has been obtained up to this point including preliminary office and field reconnaissance, boring logs, CPT soundings etc., and laboratory data are used to determine soil and rock engineering properties of interest and develop a subsurface model of the site to be used for design. Data from different sources of field and lab tests, from site geological characterization of the site subsurface conditions, from visual observations obtained from the site reconnaissance, from historical experience with the subsurface conditions at or near the site, and from the results of back- analyses shall be compared to determine the engineering properties for the various geologic units encountered throughout the site. If soil/rock data from nearby sites in the same or similar geologic unit are considered, site specific test data shall have priority in the selection of design parameters relative to non-site specific historical data for the geologic unit in question at the site.

Often, results from a single test (e.g. SPT N-values) may show significant scatter across a site for a given soil/rock unit. Perhaps data obtained from a particular soil unit for a specific property from two different tests (e.g. field vane shear tests and lab UU tests) do not agree. The validity and reliability of the data and its usefulness in selecting final design parameters shall be evaluated.

After a review of data reliability, a review of the variability of the selected parameters shall be carried out. Variability can manifest itself in two ways: 1) the inherent in-situ variability of a particular parameter due to the variability of the soil unit itself, and 2) the variability associated with estimating the parameter from the various testing methods. From this step, final selection of design parameters can commence, and from there completion of the subsurface profile.

5.11.2 Data Reliability and Variability

Inconsistencies in data shall be examined to determine possible causes and assess any mitigation procedures that may be warranted to correct, exclude, or downplay the significance of any suspect data. The following procedures provide a step-by-step method for analyzing data and resolving inconsistencies as outlined by Sabatini et al. (2002):

1) **Data Validation** – Assess the field and the laboratory test results to determine whether the reported test results are accurate and are recorded correctly for the appropriate material. For lab tests on undisturbed samples consider the effects of sample disturbance on the quality of the data. For index tests (e.g. grain size, compaction) make sure that the sample accurately represents the in-situ condition. Disregard or downplay potentially questionable results (e.g., test results that are potentially invalid due to sample disturbance, affected by recording errors, affected by procedural errors, etc.).

2) **Historical Comparison** – Assess results with respect to anticipated results based on site and/or regional testing and geologic history. If the new results are inconsistent with other site or regional data, it will be necessary to assess whether the new data is anomalous or whether the new site conditions differ from those from which previous data was collected. For example, an alluvial deposit might be expected to consist of medium dense silty sand with SPT blow counts of 30 or less. If much higher blow counts are recorded and the Standard Penetration tests were performed correctly, the reason could be the deposit is actually dense (and therefore higher friction angles can be assumed), or gravel may be present and is influencing the SPT data. Most likely it is the second case, and the engineering properties should probably be adjusted to account for this. But if consideration had not been given as to what to expect, values for properties might be used that could result in an unconservative design. If the reason for the difference between the new site specific test data and the historical data from nearby sites is not clear, then the site specific test data shall be given priority with regard to final selection of design parameters.

3) **Performance Comparison** – Assess results with respect to historic performance of structures at the site or within similar soils as described in Setion 5.7. Compare the results from the back-analyses to the properties determined from field and lab testing for the project site. The newly collected data should be correlated with the

parameters determined from observation of measurable performance and the field and lab tests performed for the previous project.

4) **Correlation Calibration** – If feasible, develop site-specific correlations using the new field and lab data. Assess whether this correlation is within the range of variability typically associated with the correlation based on previous historic data used to develop the generic correlation.

5) **Assess Influence of Test Complexity** – Assess results from the perspective of the tests themselves. Some tests may be easy to run and calibrate, but provide data of a "general" nature, while other tests are complex and subject to operator influence, yet provide "specific" test results. When comparing results from different tests consider which tests have proven to give more accurate or reliable results in the past, or more accurately approximate anticipated actual field conditions. For example, results of field vane shear tests may be used to determine undrained shear strength for deep clays instead of laboratory UU tests because of the differences in stress states between the field and lab samples, and disturbance resulting from the sampling and test specimen preparation. It may be found that certain tests consistently provide high or low values compared to anticipated results.

The result of these five steps is to determine whether or not the data obtained for the particular tests in question is valid. Where it is indicated that test results are invalid or questionable as determined through the five step process described above, the results should be downplayed or thrown out. If the test results are proven to be valid, the conclusion can be drawn that the soil unit itself and its corresponding engineering properties are variable (vertically, aerially, or both).

The next step is to determine the amount of variability that can be expected for a given engineering property in a particular geologic unit, and how that variability should influence the selection of the final design value. Sabatini et al. (2002) list several techniques that can be used:

1) **Experience** – In some cases the geotechnical designer may have accumulated extensive experience in the region such that it is possible to accurately select an average, typical or design value for the selected property, as well as the appropriate variability for the property.

2) **Statistics** – If a geotechnical designer has extensive experience in a region, or there has been extensive testing by others with published or available results, there may be sufficient data to formally establish the average value and the variability (mean and standard deviation) for the specific property. See Sabatini et al. (2002) and Phoon et al. (1995) for information on the variability associated with various engineering properties.

3) **Establish Best-Case and Worst-Case Scenarios** – Based on the experience of the geotechnical designer, it may be possible to establish upper and lower bounds along with the average for a given property.

5.11.3 Time Dependent Considerations

Properties of soil and rock can change over time (see Section 5.3). Examples of time dependent changes include, but are not limited to, the following:

- Material degradation due to weathering, moisture changes, etc.,
- Changes in properties such shear strength due to deformation,
- Changes resulting from short or long-term stress changes (e.g., removal of load due to excavation causing rebound)

When selecting soil and rock properties for design, the potential for these changes to occur during the life of the facility shall be addressed in the final selection of soil and rock properties. For example, if conducting a back analysis of a slope failure, especially if it is a first time slope failure, the back-analysis will determine the mobilized shear strength at the time the failure initiated and therefore may result in a value that is greater than the residual shear strength measured in laboratory testing or determined from correlations. The back-analyzed shear strength may therefore be greater than the shear strength along the post failure shear surface as well as the long-term shear strength that could occur in the future. In such cases, the shear strength that is representative of the long-term condition, i.e., the residual shear strength determined from the laboratory tests and correlations, should be selected for design.

5.11.4 Final Property Selection

Recognizing the variability discussed in the previous section, depending on the amount of variability estimated or measured, the potential impact of that variability (or uncertainty) on the level of safety in the design shall be assessed. If the impact of this uncertainty is likely to be significant, parametric analyses shall be conducted, or more data could be obtained to help reduce the uncertainty. Since the sources of data that could be considered may include measured laboratory data, field test data, performance data (i.e., from back-analyses), and other previous experience with the geologic unit(s) in question, it will not be possible to statistically combine all this data together to determine the most likely property value. Engineering judgment based on experience, combined with parametric analyses as needed, will be needed to make this final assessment and design property determination. At that point, a decision must be made as to whether the final design value selected should reflect the interpreted average value for the property, or a value that is somewhere between the most likely average value and the most conservative estimate of the property. However, the desire for design safety must be balanced with the cost effectiveness and constructability of the design. In some cases, being too conservative with the design could result in an un-constructible design (e.g., the use of very conservative design parameters could result in a pile foundation that must be driven deep into a very dense soil unit that in reality is too dense to penetrate with available equipment).

Note that in Chapter 8, where reliability theory was used to establish load and resistance factors, the factors were developed assuming that mean values for the design properties are used. However, even in those cases, design values that are more conservative than the mean may still be appropriate, especially if there is a significant amount of uncertainty in the assessment of the design properties due, for example, to highly variable site conditions, lack of high quality data to assess property values, or due to widely divergent property values from the different methods used to assess properties within a given geologic unit. The consequence of failure should also bear on the determination of a design parameter. 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 will 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. Note that for those resistance factors that were determined based on calibration by fitting to allowable stress design, consideration for potentially using an average property value is not relevant, and property selection should be based on the considerations discussed previously, which in most cases the property values shall be selected conservatively to be consistent with past practice.

The process and examples to make the final determination of properties to be used for design provided by Sabatini et al. (2002) shall be followed, subject to the specific requirements in the GDM. Local experience with certain engineered and naturally occurring geologic units encountered in the state of Washington is summarized in Sections 5.12 and 5.13. The final selection of design properties for the engineered and naturally occurring geologic units described in these two GDM sections shall be consistent with the experience cited in these two GDM sections.

The documentation required to justify the selection of design parameters is specified in Section 23.3.2.

5.11.5 Development of the Subsurface Profile

While Section 5.8 generally follows a sequential order, it is important to understand that the selection of design values and production of a subsurface profile is more of an iterative process. The development of design property values should begin and end with the development of the subsurface profile. Test results and boring logs will likely be revisited several times as the data are developed and analyzed before the relation of the subsurface units to each other and their engineering properties are finalized.

The ultimate goal of a subsurface investigation is to develop a working model that depicts major subsurface ESU's exhibiting distinct engineering characteristics. The end product is the subsurface profile, a two dimensional or, if necessary, a three dimensional depiction of the site stratigraphy. The following steps outline the creation of the subsurface profile:

1) Complete the field and lab work and incorporate the data into the preliminary logs.
2) Lay out the logs relative to their respective field locations and compare and match up the different soil and rock units at adjacent boring locations, if possible. However, caution should be exercised when attempting to connect units in adjacent borings, as the stratigraphy commonly is not linear or continuous between borings. Field descriptions and engineering properties will aid in the comparisons.
3) Group, or possibly split up, the subsurface geologic strata based on engineering properties to create ESU's.
4) Create cross sections by plotting borings at their respective elevations and positions horizontal to one another with appropriate scales. If appropriate, two cross sections should be developed that are at right angles to each other so that lateral trends in stratigraphy can be evaluated when a site contains both lateral and transverse extents (i.e. a building or large embankment).

5) Analyze the profile to see how it compares with expected results and knowledge of geologic (depositional) history. Have anomalies and unexpected results encountered during exploration and testing been adequately addressed during the process? Make sure that all of the subsurface features and properties pertinent to design have been addressed.

5.12 Selection of Design Properties for Engineered Materials

This section provides guidelines for the selection of properties that are commonly used on WSDOT projects such as engineered fills. The engineering properties are based primarily on gradation and compaction requirements, with consideration of the geologic source of the fill material typical for the specific project location. For materials such as common borrow where the gradation specification is fairly broad, a wider range of properties will need to be considered.

Common Borrow – Per the WSDOT *Standard Specifications*, common borrow may be virtually any soil or aggregate either naturally occurring or processed which is substantially free of organics or other deleterious material, and is non-plastic. The specification allows for the use of more plastic common borrow when approved by the engineer. On WSDOT projects this material will generally be placed at 90 percent (Method B) or 95 percent (Method C) of Standard Proctor compaction. Because of the variability of the materials that may be used as common borrow, the estimation of an internal friction angle and unit weight should be based on the actual material used. A range of values for the different material properties is given in Table 5-2. Lower range values should be used for finer grained materials compacted to Method B specifications. In general during design, the specific source of borrow is not known. Therefore, it is not prudent to select a design friction angle that is near or above the upper end of the range unless the geotechnical designer has specific knowledge of the source(s) likely to be used, or unless quality assurance shear strength testing is conducted during construction. Depending on location, common borrow will may have a fines content sufficient to be moisture sensitive. This moisture sensitivity may affect the design property selection if it is likely that placement conditions are likely to be marginal due to the timing of construction.

Select Borrow – The requirements for select borrow ensure that the mixture will be granular and contain at least a minimal amount of gravel-size material. The materials are likely to be poorly graded sand and contain enough fines to be moderately moisture sensitive (the specification allows up to 10 percent fines). Select Borrow is not an all weather material. Triaxial or direct shear strength testing on material that meets Select Borrow gradation requirements indicates that drained friction angles of 38 to 45 degrees are likely when the soil is well compacted. Even in it loosest state, shear strength testing of relatively clean sands meeting Select Borrow requirements has indicated values of 30 to 35 degrees. However, these values are highly dependent on the geologic source of the material. Surficial deposits that particles which have been minimally transported/reworked (i.e. colluvium, some glacial deposits) can have more subangular to angular soil particles and hence, high shear strength values. Windblown, beach, or alluvial sands that have been rounded through significant transport could have significantly lower shear strength values. Left-overs from processed materials (e.g., scalpings) could also have relative low friction angles depending on the uniformity of the material and the degree of rounding in the soil particles. A range of values for shear strength and unit weight based on previous experience for well compacted Select Borrow is provided in Table 5-2. In general, during design the specific source of borrow is not known. Therefore, it is not prudent to select a design friction angle that is near or above the upper end of the range unless the geotechnical designer has specific knowledge of the source(s) likely to be used or unless quality assurance shear strength testing is conducted during construction. Select Borrow with significant fines content may sometimes be modeled as having a temporary or apparent cohesion value from 50 to 200 psf, subject to the requirements for the use of cohesion as specified in Section 5.10. If a cohesion value is used, the friction angle should be reduced so as not to increase the overall strength of the material. For long-term analysis, all the borrow material should be modeled with no cohesive strength.

Gravel Borrow – The gravel borrow specification should ensure a reasonably well graded sand and gravel mix. Because the fines content is under 7 percent, the material is only slightly moisture sensitive. However, in very wet conditions, material with lower fines content should be used. Larger diameter triaxial shear strength testing performed on well graded mixtures of gravel with sand that meet the Gravel Borrow specification indicate that very high internal angles of friction are possible, approaching 50 degrees, and that shear strength values less than 40 degrees are not likely. However, lower shear strength values are possible for Gravel Borrow from naturally occurring materials obtained from non-glacially derived sources such as wind blown or alluvial deposits. In many cases, processed materials are used for Gravel Borrow, and in general, this processed material has been crushed, resulting in rather angular particles and very high soil friction angles. Its unit weight can approach that of concrete if very well graded. A range of values for shear strength and unit weight based on previous experience is provided in Table 5-2. In general during design the specific source of borrow is not known. Therefore, it is not prudent to select a design friction angle that is near or above the upper end of the range unless the geotechnical designer has specific knowledge of the source(s) likely to be used or unless quality assurance shear strength testing is conducted during construction.

Gravel Backfill for Walls – Gravel backfill for walls is a free draining material that is generally used to facilitate drainage behind retaining walls. This material has similarities to Gravel Borrow, but generally contains fewer fines and is freer draining. Gravel backfill for Walls is likely to be a processed material and if crushed is likely to have a very high soil friction angle. A likely range of material properties is provided in Table 5-2.

Material	WSDOT Standard Specification	Soil Type (USCS classification)	φ (degrees)	Cohesion (psf)	Total Unit Weight (pcf)
Common Borrow	9-03.14(3)	ML, SM, GM	30 to 34	0	115 to 130
Select Borrow	9-03.14(2)	GP, GP-GM, SP, SP-SM	34 to 38	0	120 to 135
Gravel Borrow	9-03.14(1)	GW, GW-GM, SW, SW-SM	36 to 40	0	130 to 145
Gravel Backfill for Walls	9-03.12(2)	GW, GP, SW, SP	36 to 40	0	125 to 135

Presumptive Design Property Ranges for Compacted Borrow and Other WSDOT *Standard Specification* Materials
Table 5-2

Rock Embankment – Embankment material is considered rock embankment if 25 percent of the material is over 4 inches in diameter. Compactive effort is based on a method specification. Because of the nature of the material, compaction testing is generally not feasible. The specification allows for a broad range of material and properties such that the internal friction angle and unit weight can vary considerably based on the amount and type of rock in the fill. Rock excavated from cuts consisting of siltstone, sandstone and claystone may break down during the compaction process, resulting in less coarse material. Also, if the rock is weak, failure may occur through the rock fragments rather than around them. In these types of materials, the strength parameters may resemble those of earth embankments. For existing embankments, the soft rock may continue to weather with time, if the embankment materials continue to become wet. For embankments constructed of sound rock, the strength parameters may be much higher. For compacted earth embankments with sound rock, internal friction angles of up to 45 degrees may be reasonable. Unit weights for rock embankments generally range from 130 to 140 pcf.

Quarry Spalls and Rip Rap – Quarry spalls, light loose rip rap and heavy loose rip rap created from shot rock are often used as fill material below the water table or in shear keys in slope stability and landslide mitigation applications. WSDOT Standard Specification Section 9-13 provides minimum requirements for degradation and specific gravity for these materials. Therefore sound rock must be used for these applications. For design purposes, typical values of 105 to 120 pcf for the unit weight (this considers the large amount of void space due to the coarse open gradation of this type of material) and internal angles of friction of about 40 to 45 degrees should be used.

Wood Fiber – Wood fiber fills have been used by WSDOT for over 30 years in fill heights up to about 40 feet. The wood fiber has generally been used as light-weight fill material over soft soil to improve embankment stability. Wood fiber has also been used in emergency repair because rain and wet weather does not affect the placement and compaction of the embankment. Only fresh wood fiber should be used to prolong the life of the fill, and the maximum particle size should be 6 inches or less. The wood fiber is generally compacted in lifts of about 12 inches with two passes of a track dozer. Presumptive design values of 50 pcf for unit weight and an internal angle of friction of about 40 degrees may be used for the design of the wood fiber fills (Allen et al., 1993).

To mitigate the effects of leachate, the amount of water entering the wood should be minimized. Generally topsoil caps of about 2 feet in thickness are used. The pavement section should be a minimum of 2 feet (a thicker section may be needed depending on the depth of wood fiber fill). Wood fiber fill will experience creep settlement for several years and some pavement distress should be expected during that period. Additional information on the properties and durability of wood fiber fill is provided in Kilian and Ferry (1993).

Geofoam – Geofoam has been used as lightweight fill on WSDOT projects since 1995. Geofoam ranges in unit weight from about 1 to 2 pcf. Geofoam constructed from expanded polystyrene (EPS) is manufactured according to ASTM standards for minimum density (ASTM C 303), compressive strength (ASTM D 1621) and water absorption (ASTM C 272). Type I and II are generally used in highway applications. Bales of recycled industrial polystyrene waste are also available. These bales have been used to construct temporary haul roads over soft soil. However, these bales should not be used in permanent applications.

5.13 Properties of Predominant Geologic Units in Washington

This section contains a brief discussion of soil and rock types common to Washington state that have specific engineering properties that need consideration.

5.13.1 Loess

Loess is a windblown (eolian) soil consisting mostly of silt with minor amounts of sand and clay (Higgins et al., 1987). Due to its method of deposition, loess has an open (honeycomb) structure with very high void ratios. The clay component of loess plays a pivotal role because it acts as a binder (along with calcium carbonate in certain deposits) holding the structure together. However, upon wetting, either the water soluble calcium carbonate bonds dissolve or the large negative pore pressures within the clay that are holding the soil together are reduced and the soil can undergo shear failures and/or settlements.

Loess deposits encompass a large portion of southeastern Washington. Loess typically overlies portions of the Columbia River Basalt Group and is usually most pronounced at the tops of low hills and plateaus where erosion has been minimal (Joseph, 1990). Washington loess has been classified into four geologic units: Palouse Loess, Walla Walla Loess, Ritzville Loess, and Nez Perce Loess. However, these classifications hold little relevance to engineering behavior. For engineering purposes loess can generally be classified into three categories based on grain size: clayey loess, silty loess, and sandy loess (see Chapter 10).

Typical index and performance properties measured in loess are provided in Table 5-3, based on the research results provided in Report WA-RD 145.2 (Higgins and Fragazy, 1988). Density values typically increase from west to east across the state with corresponding increase in clay content. Higgins and Fragaszy observed that densities determined from Shelby tube samples in loess generally result in artificially high values due to disturbance of the open soil structure and subsequent densification. Studies of shear strength on loess have indicated that friction angles are usually fairly constant for a given deposit and are typically within the range of 27 to 29 degrees using CU tests. These studies have also indicated that cohesion values can be quite variable and depend on the degree of consolidation, moisture content and amount of clay binder. Research has shown that at low confining pressures, loess can lose all shear strength upon wetting.

Type of Loess	Liquid Limit	Plasticity Index	Dry Density (pcf)	Angle of Internal Friction (o)
Clayey	33 to 49	11 to 27	70 to 90, with maximum of up to 95 to 98 (generally increases with clay content)	27 to 29 from CU tests
Silty	14 to 32	0 to 11		
Sandy	Nonplastic	Nonplastic		

Typical Measured Properties For Loess Deposits in Washington State
Table 5-3

The possibility of wetting induced settlements shall be considered for any structure supported on loess by performing collapse tests. Collapse tests are usually performed as either single ring (ASTM D 5333) or double ring tests. Double ring tests have the advantage in that potential collapse can be estimated for any stress level. However, two identical samples must be obtained for testing. Single ring tests have the advantage in that they more closely simulate actual collapse conditions and thus give a more accurate estimate of collapse potential. However, collapse potential can only be estimated for a particular stress level, so care must be taken to choose an appropriate stress level for sample inundation during a test. When designing foundations in loess, it is important to consider long term conditions regarding possible changes in moisture content throughout the design life of the project. Proper drainage design is crucial to keeping as much water as possible from infiltrating into the soil around the structure. A possible mitigation technique could include overexcavation and recompaction to reduce or eliminate the potential for collapse settlement.

Loess typically has low values of permeability and infiltration rates. When designing stormwater management facilities in loess, detention ponds should generally be designed for very low infiltration rates.

Application of the properties of loess to cut slope stability is discussed in Chapter 10.

5.13.2 Peat/Organic Soils

Peats and organic soils are characterized by very low strength, very high compressibility (normally or slightly under-consolidated), low hydraulic conductivity, and having very important time-consolidation effects. Often associated with wetlands, ponds and near the margins of shallow lakes, these soils pose special challenges for the design of engineering transportation projects. Deep deposits (+100 feet in some cases) with very high water content, highly compressibility, low strength and local high groundwater conditions require careful consideration regarding settlement and stability of earth fill embankments, support for bridge foundations, and locating culverts.

The internal structure of peat, either fibrous or granular, affects its capacity for retaining and releasing water and influences its strength and performance. With natural water content often ranging from 200-600 percent (over 100 for organic silts and sands) and wet unit weight ranging from 70 to 90 pcf, it can experience considerable shrinkage (>50%) it dries. Rewetting usually cannot restore its original volume or moisture content. Under certain conditions, dried peat will oxidize and virtually disappear. Undisturbed sampling for laboratory testing is difficult. Field vane testing is frequently used to evaluate in place shear strength, though in very fibrous peats, reliable shear strength data is difficult to obtain even with the field vane shear test. Initial undisturbed values of 100 to 400 psf are not uncommon but remolded (residual) strengths can be 30 to 50 % less (Schmertmann, 1967). Vane shear strength, however, is a function of both vane size and peat moisture content. Usually, the lower the moisture of the peat and the greater its depth, the higher is its strength. Strength increases significantly when peat is consolidated, and peak strength only develops after large deformation has taken place. Due to the large amount of strain that can occur when embankment loads are placed on peats and organic soils, residual strengths may control the design.

Vertical settlement is also a major concern for constructing on organic soils. The amount of foundation settlement and the length of time for it to occur are usually estimated from conventional laboratory consolidation tests. Secondary compression can be quite large for peats and must always be evaluated when estimating long-term settlement. Based on experience in Washington State, compression index values based on vertical strain ($C_{c\varepsilon}$) typically range from 0.1 to 0.3 for organic silts and clays, and are generally above 0.3 to 0.4 for peats. The coefficient of secondary compression ($C_{\alpha\varepsilon}$) is typically equal to $0.05C_{c\varepsilon}$ to $0.06C_{c\varepsilon}$ for organic silts and peats, respectively.

5.13.3 Glacial Deposits

Till – Till is an unsorted and unstratified accumulation of glacial sediment deposited directly by glacial ice. Till is a heterogeneous mixture of different sized material with particle sizes ranging in size from clay to boulders. Although the matrix proportions of silt and clay vary from place to place, the matrix generally consists of silty sand or sandy silt (Troost and Booth, 2003). Tills in Washington are deposited by either continental glaciers or alpine glaciers. Many of the tills in Washington, especially those associated with continental glaciers, have been overridden by the advancing continental ice sheet and are highly over consolidated, but not all tills have been consolidated by glacial ice. Tills deposited by alpine glaciers are most commonly found in and along the valley margins of the Olympic Mountains and Cascade Range, and are commonly not over consolidated.

Glacial till is often found near the surface in the Puget Sound Lowland area. The Puget Sound Lowland is a north-south trending trough bordered by the Cascade Mountains to the east and the Olympic Mountains to the west. The most recent glaciation, the Vashon Stade of the Fraser Glaciation occupied the Puget Sound region between roughly 18,000 to 13,000 years ago. Glacial till deposited by this glaciation extends as far south as the Olympia area.

Till that has been glacially overridden generally has very high unit weights and very high soil strength even when predominantly fine grained. Because if its inherent strength and density, it provides good bearing resistance, has very small strain under applied loads, and exhibits good stand up times even in very steep slopes. Typicall properties for glacially overridden tills range from 40 to 45 degrees for internal friction angle with cohesion values of 100 to 1,000 psf. Unit weights used for design are typically in the range of 130 to 140 pcf for glacially overridden till. The cohesion component of the shear strength can typically be relied upon due to the relatively high fines content of this geologic unit combined with its heavily overconsolidated nature and locked in stress history. Furthermore, very steep, high exposures of till in the Puget Sound region have demonstrated long-term stability that cannot be explained without the presence of significant soil cohesion, verifying the reliability of this soil cohesion. However, where these till units are exposed, the upper 2 to 5 feet is often weathered and is typically medium dense to dense. The glacial till generally grades to dense to very dense below the weathered zone. This upper weathered zone, when located on steep slopes, has often been the source of slope instability and debris flows during wet weather. Glacial till that is exposed as a result of excavation, slope instability, or other removal of overlying material will degrade and lose strength with weathering. If the till unit is capped with a younger deposit and had been previously weathered, weathered till zones can be present at depth as well.

The dense nature of glacially overriden till tends to make excavation and pile installation difficult. It is not uncommon to have to rip till with a dozer or utilize large excavation equipment. Permeability in till is relatively low because of the fines content and the density. However, localized pockets and seams of sand with higher permeability that may also be water bearing are occasional encountered in till units. These localized pockets and seams may contribute slope stability problems.

Till that has not been glacially overridden and over consolidated should be treated as normally consolidated materials consistent with the till's grain size distribution. Accordingly, tills that tend to be finer grained will exhibit lower strength and higher strain than tills which are skewed toward the coarser fraction.

Wet weather construction in till is often difficult because of the relatively high fines content of till soils. When the moisture content is more than a few percent above the optimum moisture content and the till is disturbed or unconfined, till soils become muddy and unstable, and operation of equipment on these soils can become difficult. Within till cobble and boulder-sized material can be encountered at any time. Boulders in till deposits can range from a foot or two in diameter to tens of feet. In some areas cobble, boulders, and cobble/boulder mixtures can be nested together, making excavation very difficult.

Outwash – Outwash is a general term for sorted sediment that has been transported and deposited by glacial meltwater, usually in a braided stream environment. Typically, the sediment becomes finer grained with increasing distance from the glacier terminus.

Outwash tends to be more coarse grained and cleaner (fewer fines) than till. When it has been overridden by advancing ice, its strength properties are similar to till, but the cohesion is much lower due to a lack of fines, causing this material to have greater difficulty standing without raveling in a vertical cut, and in general can more easily cave in open excavations or drilled holes. Typically, the shear strength of glacially overridden (advance) outwash ranges from 40 to 45 degrees, with near zero cohesion for clean deposits. Since it contains less fines, it is more likely to have relatively high permeability and be water bearing. In very clean deposits, non-displacement type piles (e.g., H-piles) can "run" despite the very dense nature of the material.

Outwash that has not been glacially overridden may be indistinguishable from alluvial deposits. When normally consolidated outwash is encountered it exhibits strengths, densities, and other physical properties that are consistent with alluvium, with friction angles generally less than 40 degrees and little or no cohesion.

Within outwash, cobble and boulder-sized material can be encountered at any time. Boulders in outwash deposits can range from a foot or two in diameter to tens of feet. In some areas cobbles, boulders, and cobble/boulder mixtures can be nested together, making excavation very difficult.

Glacial Marine Drift (GMD) – Drift is a collective term used to describe all types of glacial sedimentary deposits, regardless of the size or amount of sorting. The term includes all sediment that is transported by a glacier, whether it is deposited directly by a glacier or indirectly by running water that originates from a glacier. In the Pacific Northwest, practitioners have commonly referred to fine-grained glacial sediments deposited in marine water as Glacial Marine Drift, or sometimes just Marine Drift.

In addition to sand and fine-grained materials, glacial marine drift contains variable amounts of clastic debris from melting icebergs, floating ice, and gravity currents. Most commonly glacial marine drift consists of poorly graded granular material within a clayey matrix. Composition varies from gravelly, silty sand with a trace of clay to silty sand and silty clay with varying percentages of sand and gravel. Because of the marine environment, it can contain shell and wood fragments, and occasional cobbles and boulders.

In and around Bellingham, the glacial marine drift typically consists of unsorted, unstratified silt and clay with varying amounts of sand, gravel, cobbles and occasional boulders, with small percentages of shells and wood. It is typically found at the surface or below Holocene age deposits. The upper portion of this unit, sometimes to about 15 feet of depth, can be quite stiff as a result of desiccation or partial ice contact in upland areas. This stiffer desiccated zone typically grades from medium stiff to very soft with depth. The entire glaciomarine drift profile can be stiff when only a thin section of the drift mantles bedrock at shallow depths. Conversely, the entire profile is typically soft in the Blaine area and can be soft when in low, perennially saturated areas. This geologic unit can be very thick (150 feet or more).

The properties of this unit are extremely variable, varying as a function of location, depth, loading history, saturation and other factors. The soft to medium stiff glaciomarine drift typically has very low shear strength, very low permeability and high compressibility. Based on vane shear and laboratory testing of this unit, the soft portion of this unit below the stiff crust typically has undrained shear strengths of approximately 500 to 1000 psf, and can be as low as 200 to 300 psf. The upper stiff crust is typically stronger, and may be capable of supporting lightly loaded footing supported structures. Atterberg limits testing will typically classify the softer material as a low plasticity clay; although, it can range to high plasticity. Consolidation parameters are variable, with the compression index (CC) in the range of 0.06 to over 0.2. Time rates of consolidation can also be quite variable.

Wet weather construction in glaciomarine drift is very difficult because of the relatively high clay content of these soils. When the moisture content of these soils is more than a few percent above the optimum moisture content, they become muddy and unstable, and operation of equipment can become very difficult. Localized sandy and gravelly layers in the drift can be saturated and are capable of producing significant amounts of water in cuts.

Glaciolacustrine – Glaciolacustrine deposits form in glacial meltwater lakes that may occur during both advancing and recessional glacial episodes. Glaciolacustrine deposits are commonly stratified and tend to be fine grained, typically consisting of silt and clay and often with sand laminae. Glaciolacustrine deposits accumulated during glacial advances may be overridden by the ice, causing the deposits to be highly overconsolidated and typically very stiff to hard. An example of glacially overridden undisturbed laminated silt/clay deposits is provided in Figure 5-1. When not glacially overridden, such as during the last glacial recessional period, glaciolacustrine deposits may behave similarly to other normally consolidated lacustrine deposits.

Example of Glacially Overridden Laminated Clay Exposed in Highway Excavation on Beacon Hill Near The Intersection SR-5 and SR-90
Figure 5-1

Fine-grained, glacially overridden deposits are widespread in the Puget Sound region, and have been encountered on projects in the Seattle area in the vicinity of SR-5, SR-90, SR-99, SR-405, and SR-520. These fine-grained deposits may be glaciolacustrine in origin associated with one of the more than six continental glaciations that have inundated the region during the Pleistocene. In the Seattle area, the most recent (Vashon Stade) of these advance glaciolacustrine units are named the Lawton Clay. This deposit can be more than 150 feet thick in the Seattle area (Troost and Booth, 2003). Additionally, fine-grained bedded units may be associated with interglacial periods (i.e., Olympia Beds) that may be somewhat similar in initial appearance to glaciolacustrine deposits. The widespread presence in the Seattle area of both glacial and interglacial, fine-grained, overconsolidated deposits has led many geotechnical practitioners to refer to any such deposit as “Seattle clay”, often irrespective of its age or origin. Collectively, these fine-grained overconsolidated deposits are often a primary material affecting engineering design in the Seattle area.

Extensive disturbance of these fine-grained, overconsolidated deposits is commonly observed, evidenced by fracturing and slickensides. A slickenside is a condition in which relative movement has occurred along the fracture, and is discernible by its shiny and commonly striated fracture surface. More extreme disturbance may involve disoriented/transported blocks within a matrix of intensely sheared and fractured silt and clay.

There are a variety of causes that may lead to post-depositional disturbance of these glaciolacustrine deposits. Vertical stresses and subsequent dewatering and consolidation through ice loading can induce fracturing, sometimes producing predictable fracture sets/networks. Lateral stresses induced by ice movement/flow can cause considerable deformation, shearing and translational movements (sometimes termed “shoving”) within the underlying sediments, a process referred to as glaciotectonics (e.g., Figure 5-2). Following deglaciation, stress relief associated with unloading, isostacy, exhumation, and erosion can induce further fracturing within the sediments. Another post-depositional disturbance mechanism causing extensive fracturing and disturbance of these deposits is landsliding on exposed slopes that occurred between glacial episodes and following the last glaciation. Figure 5-3 shows a tilted laminated clay block that was overridden and smeared by a subsequent glacial advance. Figure 5-4 shows a deep (approximately 40 feet) test pit exposing layers of weathered clay, water-bearing gravel, and unweathered clay, illustrating the highly variable structure and depositional environment that can occur in these reconsolidated landslide deposits. These reconsolidated landslide deposits, in particular, can become highly unstable when exposed in excavations or natural slopes. Ground motions and crustal deformation induced by regionally active tectonic processes are another source of disturbance to these deposits.

Figure 5-2(a)

Exposure Near the East End of Sr-520 Illustrating Fractured and Sheared Structure Within Glacially Overridden Clay Deposit Believed to be Due to Glaciotectonics (a) Overview of Exposure, (b) Close-Up Showing Clay Structure
Figure 5-2(b)

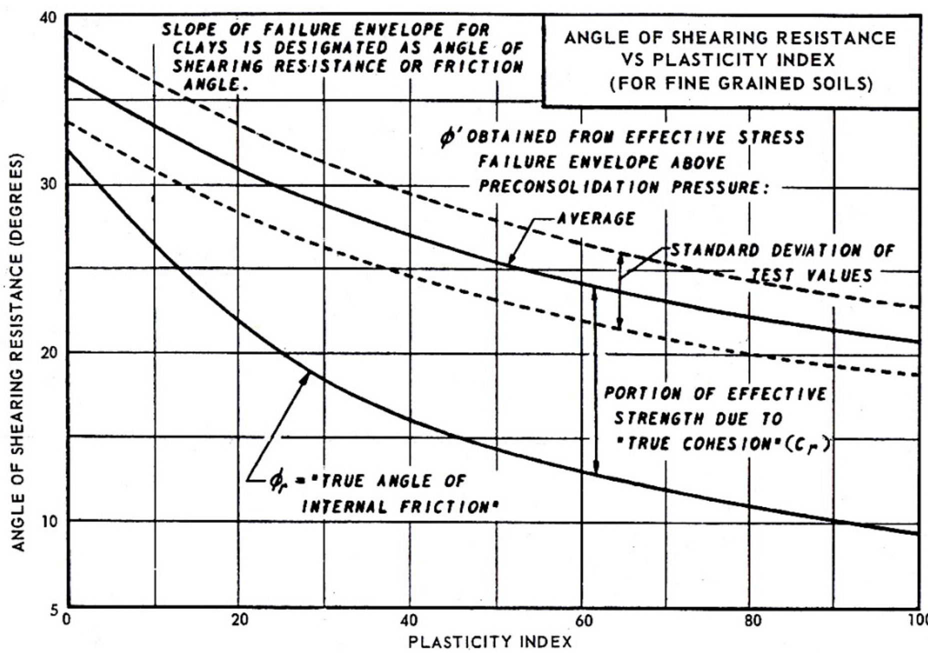
Example on Beacon Hill of Highly Disturbed Glacially Overconsolidated Clay Associated with a Paleolandslide Deposit; Note Near-Vertical Orientation of Laminae/Bedding Within the Landslide Block
Figure 5-3

Test Pit on Beacon Hill Showing Depositional Sequence Within a Glacially Overconsolidated Clay, Paleolandslide Deposit
Figure 5-4

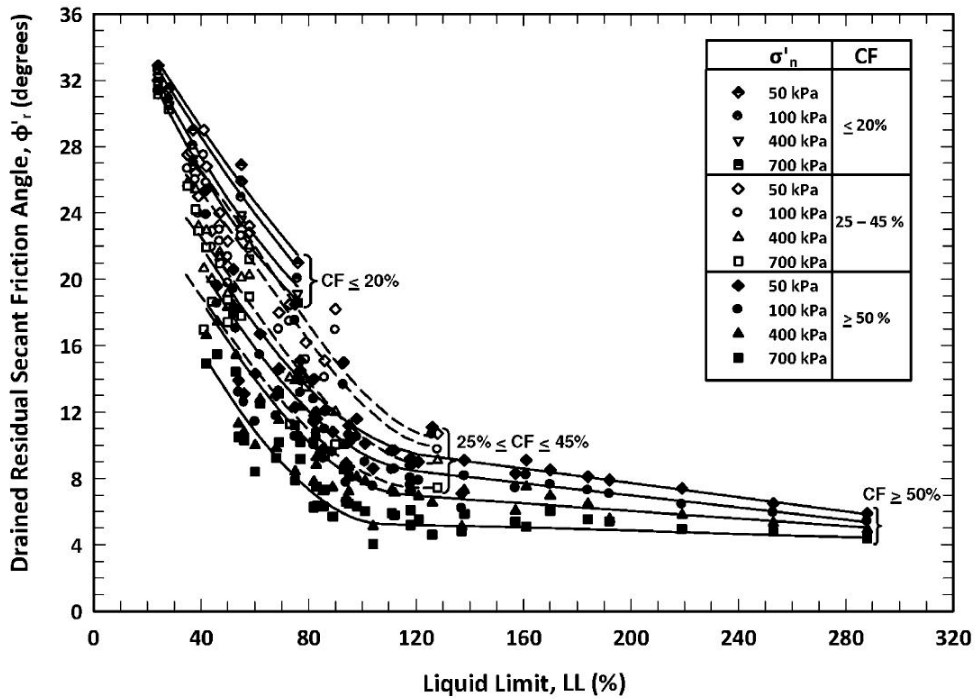
One of the most important geotechnical characteristics of these fine-grained overconsolidated deposits is that they generally have high in situ lateral stresses. Relaxation of these locked in stresses have created significant slope stability problems in both open and shored excavations. As excavations are completed, these deposits experience a lateral elastic rebound, which leads to their internal weakening. The failure mechanism is thought to consist of shear movement and/or tensional opening along pre-existing fractures. Depending on the extent of disturbance, failure surfaces/ zones may need to shear along existing fractures and through intact clay blocks to fully develop. Linkage of fractures and subsequent hydrostatic pressure buildup within them can then further displace larger blocks/masses. With movement comes a drastic reduction in shear strength (often to a residual state) within these larger blocks/ masses, which then lead to progressive slope failures. Such instability occurred in the downtown Seattle area when cuts were made within these deposits to construct Interstate 5 and Interstate 90. Fine, water-bearing sand laminae within the silts and clays often further exacerbate instability in exposures, not only in open cuts, but also in the form of caving in relatively small diameter shaft excavations.

Based on considerable experience, the long-term design of project geotechnical elements affected by these fine-grained overconsolidated deposits should be based on residual strength parameters. However, exceptions to this are provided in the paragraphs that follow.

For these deposits, the relationship between the residual friction angle and the plasticity index as reported in NAVFAC DM7 generally works well for estimating the residual shear strength (see Figure 5-5). The Stark and Hussain (2013) correlations for residual strength (see Figure 5-6) also work well for these deposits. In practice, shear strength values that have been estimated based on back-analysis of landslides and cut slope failures in this region are in the range of 13 to 17 degrees.



Correlation Between Residual Shear Strength of Overconsolidated Clays and Plasticity Index (After NAVFAC, 1971)
Figure 5-5



Correlation Between Residual Shear Strength of Overconsolidated Clays and Plasticity Index, Clay Fraction Cf, and Effective Normal Stress (After Stark and Hussain 2013)
Figure 5-6

Correlations with index soil properties such as the plasticity index, such as shown in Figure 5-5, or such as provided in Stark and Hussain (2013) in Figure 5-6, can be used to estimate the residual shear strength of soil. Laboratory tests on the site specific soils should be conducted, if possible, to measure the residual friction angle. When laboratory shear strength tests are conducted to determine the residual friction angle, high displacement tests such as the ring shear test should be used.

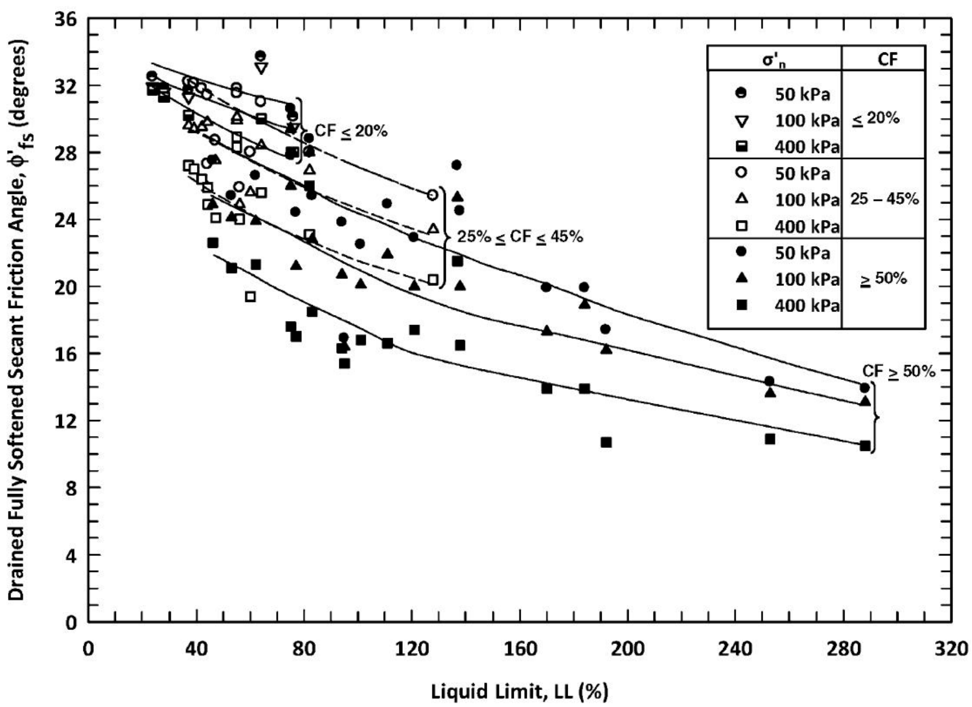
Designing for residual shear strength of the clay is a reasonable and safe approach in these fine-grained glacially consolidated soils, and is the default approach in post-depositionally disturbed deposits of fine-grained glacially consolidated soil, though there may be limited cases where a slightly higher shear strength could be used for design. For example, the glacially overridden clay deposits described earlier (e.g., figures 5-2 through 5-4) have been broken up enough to warrant the use of residual shear strength in most cases. If more detailed investigation is conducted (e.g., through back-analysis of previous slope failures or marginally stable slopes at the site in question, extensive laboratory shear strength testing, other possible testing or evaluation techniques, and consideration of site geological history of the strata in question) and demonstrates the shear strength of the existing deposit is greater than its residual value, higher design shear strengths may be justified, provided that any potential future deformation of the clay strata is prevented. In no case, however, in these glaciolacustrine deposits that have been post-depositionally disturbed due to phenomenon such as landsliding, glacial shoving, and shearing due to fault activity, shall a shear strength greater than the fully softened shear strength be used for design, even if future deformation of the clay deposit can be fully restrained. This applies to both temporary and permanent designs.

Note that the fully softened friction angle for clays is defined in Mesri and Shahien (2003) as:

"The fully softened strength envelope (often defined for stiff clays and shales by peak strength of reconstituted normally consolidated specimens) …."

In essence, this fully softened shear strength reflects the strength of an overconsolidated clay that has been disturbed, but the "plate-shaped" clay particles have not been fully aligned. This is in contrast to the situation in which a clay has been sufficiently sheared to reach a state of residual strength, such as along a landslide failure surface or along slickensides, in which all the clay particles have been aligned, producing the lowest possible shear strength.

Stark and Hussain (2013) provide recommended correlations to estimate the fully softened shear strength (see Figure 5-7) that should be used to estimate the fully softened shear strength, if laboratory site specific shear strength test data are not available. Alternatively, laboratory testing could be conducted to establish the fully softened shear strength. Guidelines regarding the type of laboratory testing required are provided in Stark, et al. (2005), and additional considerations for laboratory testing are provided in Stark and Hussain (2013).



Correlation Between Fully Softened Shear Strength of Overconsolidated Clays and Plasticity Index (After Stark and Hussain 2013)
Figure 5-7

Intact deposits of glacially overridden clays and clayey silts (i.e., those not subjected to the geologic disturbance processes described previously) may be designed for shear strengths approaching their peak values provided that (1) the clay has not been subject to deformation resulting from previous construction or erosion that caused unloading of the clay, or (2) the clay is deep enough to not be affected and will not be subject to unloading and deformation in the planned construction. Structures (e.g., tieback walls) designed to restrain the clay to prevent deformation may be used in combination with shear strengths near their peak values if previous construction that could potentially have caused removal/unloading of the clay has not occurred prior to the construction of the restraining structure. Otherwise, residual shear strength should be used for design within the clay. Intact glacially overridden clay that is deep enough below the final ground surface to not be affected by potential unloading may be designed for shear strength near its peak value.

As with most fine grained soils, wet weather construction in overconsolidated silt/clay is generally difficult. When the moisture content of these soils is more than a few percent above the optimum moisture content, they become muddy and unstable, and operation of equipment on these soils can become difficult.

Groundwater modeling of these glacially overridden clays can be very complex. Where below the groundwater surface, these clays may visually appear moist or dry. However, even with that appearance these clays can be saturated. Because they are fine grained and highly compact, water generally does not freely flow from these soils. More freely flowing ground water may be present in these deposits in localized or thin sand or gravel seams (e.g., Figure 5-4), between laminations in the clay, and within fissures in the clay, whereas the intact portions of the clay appear to be moist. The water within these fissures and sand or gravel seams is often hydraulically connected, having a similar effect with regard to stresses and stability as occurs in fractured rock masses that contain water. Due to the nature of the clay and the tendency of the clay surfaces within boreholes to become smeared during drilling, standard standpipe piezometers may take a very long time to stabilize adequately to get accurate water level readings – electrical piezometers, such as vibrating wire, should be used to get more accurate water level readings within a reasonable period of time.

Even though this geologic deposit is generally fine-grained, due to the highly overconsolidated nature of this deposit, settlement can generally be considered elastic in nature, and settlement, for the most part, occurs as the load is applied. This makes placement of spread footings on this deposit feasible if designed for relatively low bearing stress, and provided the footing is not placed on a slope that could allow an overall stability failure due to the footing load (see Chapter 8).

For additional discussion on geotechnical characterization and design in glacially overconsolidated clays, see Mesri and Shahien (2003) and Stark, et al. (2005).

5.13.4 Colluvium and Talus

Colluvium is a general term used to describe soil and rock material that has been transported through rainwash, sheetwash and downslope creep that collect on or at the base of slopes. Colluvium is typified by poorly sorted mixtures of soil and rock particles ranging in size from clay to large boulders. Talus is a gravitationally derived deposit that forms downslope of steep rock slopes, comprised of a generally loose assemblage of coarse, angular rock fragments of varied size and shape. Talus is commonly collectively referred with the term colluvium.

Colluvium is a very common deposit, encompassing upwards of 90 percent of the ground surface in mountainous areas. Colluvial deposits are typically shallow (less than about 25 to 30 feet thick), with thickness increasing towards the base of slopes. Colluvium commonly directly overlies bedrock on unglaciated slopes and intermixes with alluvial material in stream bottoms.

Subsurface investigations in colluvium using drilling equipment are often complicated by because of the heterogeneity of the deposit and possible presence of cobbles boulders. In addition, site access and safety issues also can pose problems. Test pits and trenches offer alternatives to conventional drilling that may provide better results. Subsurface investigations in talus can be especially difficult. Engineering properties of talus are extremely difficult to determine in the laboratory or in situ. A useful method for determining shear strength properties in both colluvium and talus is to analyze an existing slope failure. For talus, this may be the only way to estimate shear strength parameters. Talus deposits can be highly compressible because of the presence of large void spaces. Colluvial and talus slopes are generally marginally stable. In fact, talus slopes are usually inclined at the angle of repose of the constituent material. Cut slopes in colluvium often result in steepened slopes beyond the angle of repose, resulting in instability. Slope instability is often manifested by individual rocks dislodging from the slope face and rolling downslope. While the slope remains steeper than the angle of repose, a continuous and progressive failure will occur.

Construction in colluvium is usually difficult because of the typical heterogeneity of deposits and corresponding unfavorable characteristics such as particle size, strength variations and large void spaces. In addition, there is the possibility of long-term creep movement. Large settlements are also possible in talus. Foundations for structures in talus should extend through the deposit and bear on more competent material. Slope failures in colluvium are most often caused by infiltration of water from intense rainfall. Modifications to natural slopes in the form of cut slopes, construction of drainage ditches, and impropery channelized stormwater are ways that water can infiltrate into a colluvial soil and initiate a slope failure. Careful consideration must be given to the design of drainage facilities to prevent saturation of colluvial deposits.

5.13.5 Columbia River Sand

These sands are located in the Vancouver area, and both up and down river along the Columbia River west of the Cascades. These sands may have been deposited by backwaters from the glacial Lake Missoula catastrophic floods. The sands are poorly graded and range from loose to medium dense. The sand is susceptible to liquefaction if located below the water table. The sands do not provide a significant amount of frictional resistance for piles, and non-displacement piles may tend to run in these deposits. Based on the observed stability of slopes in this formation, soil friction angles of 28o to 32o should be expected.

5.13.6 Columbia Basin Basalts

The basalt flows that dominate the Columbia Basin were erupted into a structural and topographic low between the northern Rocky Mountains and the rising Cascade Range. During periods between the flows, erosion took place and tuffs, sandstones, and conglomerates were deposited on top of basalt flows (Thorsen, 1989). In some areas lake beds formed. The resulting drainage systems and lakes were responsible for the extensive layer of sediments between, interfingering with, and overlying the basalt flows. These interbedded sediments are generally thicker in areas peripheral to the flows, especially in and along the western margin of the basin. During the interludes between flows, deep saprolites formed on some flow surfaces. Present topographic relief on the basin has been provided largely by a series of east-west trending anticlinal folds, by the cutting of catastrophic glacial meltwater floods, and by the Columbia River system.

The most obvious evidence of bedrock slope failures in the basin is the presence of basalt talus slopes fringing the river canyons and abandoned channels. Such talus are generally standing at near the angle of repose.

Bedrock failures are most commonly in the form of very large slumps, slump flows, and translational landslides, controlled by weak interbeds or palagonite zones between flows. Most of these are ancient failures and occur in areas of regional tilting or are associated with anticlinal ridges. The final triggering, in many cases, appears to have been oversteepening of slopes or removal of toe support.

Along I-82, SR-12, and SR-410 on the western margin of the province and in a structural basin near Pasco, layers of weak sediments interfinger with basalt flows. Some of these sediments are compact enough to be considered siltstone or sandstone and are rich in montmorillionite. Slumps and translation failures are common in some places along planes sloping as little as 8 degrees. Most landslides are associated with pre-existing failure surfaces developed by folding and or ancient landslides. In the Spokane and Grande Ronde areas thick sections of sediments make up a major part of the landslide complexes.

5.13.7 Latah Formation

Much of Eastern Washington is underlain with thick sequences of basaltic flow rock. These flows spread out over a vast area that now comprises what is commonly known as the Columbia Plateau physiographic province (see Section 5.9.6). Consisting of extrusive volcanic rocks, they make up the Columbia River Basalt Group (Griggs, 1959). This geologic unit includes numerous basalt formations, each of which includes several individual flows that are commonly separated from one another by sedimentary lacustrine deposits (Smith et al., 1989). In the Spokane area, these sedimentary rock units are called the Latah Formation.

Most of the sedimentary layers between the basalt flows range from claystone to fine-grained sandstone in which very finely laminated siltstone is predominant. The fresh rock ranges in color from various shades of gray to almost white, tan and rust. Because of its generally poorly indurated state, the Latah rarely outcrops. It erodes rapidly and therefore is usually covered with colluvium or in steeper terrain, it is hidden under the rubble of overlying basaltic rocks.

The main engineering concern for the Latah Formation is its potential for rapid deterioration by softening and eroding when exposed to water and cyclic wetting and drying (Hosterman, 1969). The landslide potential of this geologic unit is also of great engineering concern. While its undisturbed state can often justify relatively high bearing resistance, foundation bearing surfaces need to be protected from precipitation and groundwater. Construction drainage is important and should be planned in advance of excavating. Bearing surface protection measures often include mud slabs or gravel blankets.

In the Spokane area, landslide deposits fringe many of the buttes (Thorsen, 1989). Disoriented blocks of basalt lie in a matrix of disturbed silts. The Latah Formation typically has low permeability. The basalt above it is often highly fractured, and joints commonly fill with water. Although this source of groundwater may be limited, when it is present, and the excavation extends through the Latah-basalt contact, the Latah will often erode (pipe) back under the basalt causing potential instability. The Latah is also susceptible to surface erosion if left exposed in steep cuts. Shotcrete is often used to provide a protective coating for excavation surfaces. Fiber-reinforced shotcrete and soil nailing are frequently used for temporary excavation shoring.

The Latah Formation has been the cause of a number of landslides in northeast Washington and in Idaho. Measured long-term shear strengths have been observed to be in the range of 14 to 17 degrees. It is especially critical to consider the long-term strength of this formation when cutting into this formation or adding load on this formation.

5.13.8 Coastal Range Siltstone/Claystone

The Coast Range, or Willapa Hills, are situated between the Olympic Mountains to the north and the Columbia River to the south. Thick sequences of Tertiary sedimentary and volcanic rocks are present. The rocks are not intensely deformed but have been subjected to compressional tectonism and have been somewhat folded and faulted (Lasmanis, 1991). The Willapa Hills have rounded topography, deep weathering profiles, and typically thick residual soil development. The interbedded sandstone and fine-grained sedimentary formations are encountered in highway cuts. The material from these cuts has been used in embankments. Some of the rock excavated from these cuts will slake when exposed to air and water and cause settlement of the embankment, instability and pavement distortion.

Locally thick clayey residual soils are present and extensive areas are underlain by sedimentary and volcanic rocks that are inherently weak. Tuffaceous siltstone and tilted sedimentary rocks with weak interbeds are common. The volcanic units are generally altered and or mechanically weak as a result of brecciation. Large and small-scale deep-seated and shallow landsliding are widespread geomorphic processes in this province. The dominant forms of landsliding are translational landslides, earthflows or slump-earthflows, and debris flows (Thorsen, 1989). Many of these are made up of both soil and bedrock. Reactivation of landslide in some areas can be traced to stream cutting along the toe of a slide.

5.13.9 Troutdale Formation

The Troutdale Formation consists of poorly to moderately consolidated and weakly lithified silt, sand and gravel deposited by the ancestral Columbia River. These deposits can be divided into two general parts; a lower gravel section containing cobbles, and upper section that contains volcanic glass sands. The formation is typically a terrestrial deposit found in and proximal to the present-day flood plain of the Columbia River and the Portland Basin. The granular components of the formation are typically well-rounded as a result of the depositional environment and are occasionally weakly cemented. Occasional boulders have been found in this formation. Excavation for drilled shafts and soldier piles in these soils can be very difficult because of the boulders and cemented sands.

Slope stability issues have been observed in the Troutdale Formation. Significant landslides have occurred in this unit in the Kelso area. Wet weather construction can be difficult if the soils have significant fines content. As described above, when the moisture content of soil with relatively high fines content rises a few percent above optimum, the soils become muddy and unstable. Permeability in this geologic unit varies based on the fines content or presence of lenses or layers of cemented and/or fine-grained material.

5.13.10 Marine Basalts - Crescent Formation

The Crescent Formation basalts were erupted close to the North American shoreline in a marine setting during Eocene time (Lasmanis, 1991). The formation consists mostly of thick submarine basalt flows, which commonly formed as pillow lavas. The Crescent Formation was deposited upon continentally derived marine sediments and is locally interbedded with sedimentary rocks. The Crescent Formation extends from the Willapa Hills area to the Olympic Peninsula. During the middle Eocene, the Crescent Formation was deformed during accretion to North America. The pillow basalts have extensive zones of palagonite and interstitial clay. Along the Olympic Peninsula the basalts are generally highly fractured and are often moderately weathered to decomposed.

The properties of the marine basalts are variable and depend on the amount of fracturing, mineralogy, alteration and weathering. Borrow from cut sections is generally suitable for use in embankments; however, it may not be suitable for use as riprap or quarry spalls because of degradation and slaking characteristics. All marine basalts should be tested for degradation before use as riprap or quarry spalls in permanent applications.

5.13.11 Mélange Rocks on Olympic Peninsula

During the middle Miocene, convergence of the Juan de Fuca plate with the North American plate accelerated to the point that sedimentary, volcanic, and metamorphic rocks along the west flank of the Olympics were broken, jumbled, and chaotically mixed to form a mélange (Thorsen, 1989). This formation is known as the Hoh rock assemblage. Hoh mélange rocks are exposed along 45 miles of the western coast. Successive accretionary packages of sediments within the core of the mountains are composed of folded and faulted Hoh and Ozette mélange rock. Typical of mélange mixtures, which have been broken, sheared and jumbled together by tectonic collision, the Hoh includes a wide range of rock types. Resistant sandstone and conglomerated sequences are extensively exposed in headlands and terraces along the Olympic coast. The mélange rocks may include pillow basalt, deep ocean clay and submarine fan deposits. Slopes in tilted sedimentary rocks that have been extensively altered and/or contain weak interbeds have been undercut by wave action in places along the Strait of Juan de Fuca. Slump flows or bedding plane block glides form along the interbeds.

Because of the variability of the mélange rocks and the potential for failure planes, caution should be used when designing cuts. A robust field exploration program is essential to determine the geometry and properties of the soil and rock layers.

5.14 Application of the Observational Method to Adjust Design Properties

The observational method as described by Peck (1969) and Wu (2008) may be used to adjust design parameters based on measured performance during construction. This approach may be used in the following ways:

- Planning during design that measurements will be taken and observations will be made during construction to verify the design assumptions used, or
- To address unexpected performance during construction.

The application of the observational method includes the following elements (Peck, 1969):

1. "Exploration sufficient to establish at least the general nature, pattern and properties of the deposits, but not necessarily in detail.
2. Assessment of the most probable conditions and the most unfavorable conceivable deviations from these conditions. In this assessment geology often plays a major role.
3. Establishment of the design based on a working hypothesis of behavior anticipated under the most probable conditions.
4. Selection of quantities to be observed as construction proceeds and calculation of their anticipated values on the basis of the working hypothesis.
5. Calculation of values of the same quantities under the most unfavorable conditions compatible with the available data concerning the subsurface conditions.
6. Selection in advance of a course of action or modification of design for every foreseeable significant deviation of the observational findings from those predicted on the basis of the working hypothesis.
7. Measurement of quantities to be observed and evaluation of actual conditions.
8. Modification of design to suit actual conditions."

If the observational method is to be used as part of the design process, the design shall meet the requirements of this manual, adjusting the design as needed during construction to be consistent with the performance observed.

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Chapter 6 Seismic Design

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 is followed for structure classification of bridges, except that the designation "other" is replaced with "normal" in the WSDOT *Bridge Design Manual LRFD* (BDM) M 23-50.

In keeping with the current seismic design approaches employed both nationally and internationally, geotechnical seismic design shall be consistent with the philosophy identified in the WSDOT BDM for structure seismic design which defines the structure performance objectives for the Safety Evaluation Earthquake (SEE) and the Functional Evaluation Earthquake (FEE). For the SEE, the performance objective requires that the structure be designed for non-collapse due to earthquake shaking and geologic hazards associated with a design seismic event so that loss of life and serious injury due to structure collapse are minimized. This is the primary performance objective for bridges classified as "normal". 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, which is an approximate return period of 1,000 years). Geotechnical design associated with structures shall be consistent with this performance objective and design hazard level.

For the FEE, the performance objective requires minimal to no earthquake damage and that the structure remain in full service after the earthquake. For bridges classified as "essential" or "critical", a two level seismic design is required: the SEE as defined above, except that the damage due to the earthquake is limited to minimal to moderate and limited service for the structure is expected after the earthquake, and the Functional Evaluation Earthquake (FEE). This FEE performance objective shall be achieved at a hazard level of 30 percent probability of exceedance in 75 years (or 210-year return period). Geotechnical design associated with structures shall also be consistent with this performance objective and design hazard level for essential and critical bridges. See the BDM Chapter 4, for additional details regarding the performance objectives and associated design requirements. See GDM Section 6-3.1 for requirements to assess the hazard level.

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 or inadequate performance of the structure should they fail or deform excessively. 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 or inadequate performance. 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. For "normal" bridges, the seismic stability of the bridge approach embankment in the lateral direction may not be required if instability in the lateral direction will not significantly damage the bridge and will not cause a life safety issue. The bridge interior pier foundations should also be designed to be adequately stable with regard to liquefaction, lateral spreading, flow failure, and other seismic effects to prevent bridge collapse for "normal" bridges when considering the FEE and which otherwise could compromise the functioning of essential and critical bridges for both the SEE and FEE hazard levels.

All retaining walls and abutment walls, including reinforced slopes steeper than 0.5H:1V, which shall be considered to be a wall (see Section 15-5.6), shall be evaluated and designed for seismic stability internally and externally (i.e. sliding, eccentricity, and bearing capacity), with the exception of walls that meet the AASHTO *LRFD Bridge Design Manual* "No Seismic Analysis" provisions in AASHTO Article 11.5.4.2. Noise walls, as well as reinforced slopes steeper than 1.2H:1V, shall also be evaluated for seismic stability.

With regard to seismic overall slope stability (often referred to as global stability) involving a retaining wall/reinforced slope as defined above, or noise wall, the geotechnical designer shall evaluate the impacts of failure due to seismic loading, as well as for liquefied conditions after shaking. If the wall seismic global stability does not meet the requirements in Sections 6-4.2 and 6-4.3, collapse of the wall/reinforced slope or noise wall *shall* be considered likely and assumed to cause loss of life or severe injury to the public if the following are true:

- The maximum wall/reinforced slope height is greater than 10 feet in height and
- The wall/reinforced slope is close enough to the traveled way such that collapse of the wall/reinforced slope or the slope that it supports will cause an abrupt elevation change within part or all of the traveled way, or will result in debris from the collapsed wall and the material that it supports being deposited on part or all of the traveled way, or other adjacent facility/structure.

If the above two bullets are true, the stability of the wall/reinforced slope or noise wall shall be improved such that the life safety of the public is preserved. If the maximum wall/ reinforced slope or noise wall or noise wall height is less than 10 ft, but the second bullet is still true, the potential for wall/reinforced slope collapse shall be evaluated to assess the severity of the impact to the traveled way and to the potential for life safety issues to occur. Similarly, if the wall height is greater than 10 ft, but it is not near the traveled way as defined above, the potential for wall/reinforced slope or noise wall collapse shall be evaluated to assess the severity of the impact to the public and the potential for life safety issues to occur. In either of these cases, if it is determined that failure of the wall will compromise the life safety of the public, the stability of the wall/reinforced slope or noise wall shall be improved such that the life safety of the public is preserved.

Note that the policy to stabilize retaining walls/reinforced slopes and noise walls for overall stability due to design seismic events may not be practical for walls/reinforced slopes or noise walls placed on marginally stable landslide areas or otherwise marginally stable slopes. In general, if the placement of a wall/reinforced slope 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/reinforced slope has a relatively low risk of causing loss of life or severe injury to the traveling public if collapse occurs, the requirement of the wall/reinforced slope and slope above and/or below the structure 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/reinforced slope 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/reinforced slope 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. However, this requirement may be waived by the State Geotechnical Engineer if the seismic slope stability safety factor for the existing slope (for the design earthquake ground motion) 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*

Bridge widenings require special considerations, as the existing bridge to be widened may not be adequately stabilized to resist the forces imparted to the bridge due to liquefaction effects such as downdrag and lateral spreading loads/deformations. See BDM Section 4.3 for bridge widening seismic design and existing bridge seismic retrofit policies.

To assess the effect of liquefaction induced foundation loading and deformation on the existing and widened bridge stability, the geotechnical engineer provides the structural engineer with the following:

- depth and extent of soil that is likely to liquefy for the applicable hazard level (i.e., for the SEE for normal bridges, and the SEE and FEE hazard levels for essential and critical bridges,

- liquefaction induced downdrag loads and settlement,
- p-y curve parameters for the soil in both a liquefied and not liquefied state,
- the lateral spreading soil deformation profile (i.e., free field displacements), and
- the lateral loads acting on the foundation elements if flow failure is likely.

With this information, the structural designer can then determine the seismic stability of the existing bridge and bridge widening, and the need for structural strengthening of the existing bridge. If that is not feasible, the geotechnical engineer assesses the need for ground improvement to prevent the liquefaction from occurring. If ground improvement is needed, the geotechnical engineer also provides a ground improvement design.

Note that the foundation loads caused by flow failure are affected by the foundation details and therefore may require some design iteration between the geotechnical and structural designer.

Details on the liquefaction analysis, mitigation needed if the bridge cannot be designed to resist the forces and soil deformation anticipated, and the input the geotechnical designer provides to the structural designer regarding liquefaction and its effect, are provided in Sections 6-4.2 and 6-5 of this GDM.

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 to predict liquefaction at depths greater than 50 to 60 feet should be based on a site response analysis to obtain an appropriate, site-specific stress reduction profile, provided that sufficient subsurface data are available and that variability in the input ground motions is considered.

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. For example, the effect of a 10 feet thick soil layer liquefying between depths of 80 and 90 feet will generally be much less severe than the effect 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*.

With regard to seismic hazard levels, the AASHTO *Guide Specifications for LRFD Seismic Bridge Design* and the AASHTO *LRFD Bridge Design Specifications* are based on the 2002 USGS website hazard model at a return period of 975 years (i.e., a probability of exceedance of approximately 7 percent in 75 years). The GDM and BDM seismic design requirements have been updated to use the 2014 USGS website hazard model at a probability of exceedance of 7 percent in 75 years and shall be considered to supersede the AASHTO specifications. Note that the USGS website refers to this hazard level as 5% in 50 years.

For seismic design of new buildings and non-roadway infrastructure, the International Building Code (IBC) (International Code Council), 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.

FHWA *LRFD Seismic Analysis and Design of Bridges Reference Manual* (Marsh et al., 2014) – This manual adapts and updates FHWA Geotechnical Engineering Circular No. 3 to be applicable to LRFD for Bridges and their foundations. This manual includes both geotechnical and structural 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.

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 https://earthquake.usgs.gov/hazards/hazmaps is a valuable source for information regarding the mapping seismic hazard in the United States, and specifically on the details of the hazard model underlying the 2014 mapping. The website also includes a Unified Hazard Tool which allows the user to extract hazard curves and deaggregations for various return periods of interest for the 2008 and 2014 seismic hazard maps. This tool can be found at the following address: https://earthquake.usgs.gov/hazards/interactive

The results of the hazards analysis using the 2002 USGS website hazard model at a probability of exceedance 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, and the new 2014 hazard maps and deaggregation data shall be used for seismic design (see USGS website for update and figures later in this GDM chapter).

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 6-3.2.1), also referred to as the General Procedure
- Use specification/code based hazard (Section 6-3.1) with site specific ground motion response (Section 6-3.2.2 and Appendix 6-A)
- Use site specific hazard (Section 6-3.1 and Appendix 6-A) with specification/code based ground motion response (Section 6-3.2.1)
- Use site specific hazard (Section 6-3.1 and Appendix 6-A) with site specific ground motion response (Section 6-3.2.2 and 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 or 30 percent 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.

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

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 and 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 • Kinematic 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 shear strength	• CPT • SPT • seismic cone • piezometers • geophysical testing (shear wave velocity, resistivity, natural gamma) • vane shear test • pressuremeter	• 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

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 Darendeli (2001) as provided in equations 6-1 through 6-7, applicable to all soils except peats and gravels.
- For gravels, shear modulus reduction and viscous damping relationships provided in Rollins, et al. (1998).
- For peats, shear modulus reduction and viscous damping relationships provided in Kramer (1996, 2000).
- Figures 6-1 through 6-3, which present charts for estimating equivalent undrained residual shear strength for liquefied soils as a function of SPT blowcounts. These figures primarily apply to sands and silty sands. 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. For silts, laboratory testing using cyclic simple shear or cyclic triaxial testing should be conducted (see GDM Section 6-4.2.6).

Designers are encouraged to develop region or project specific correlations for these seismic design properties. Other well accepted correlations in peer reviewed publications may be used, subject to the approval of the State Geotechnical Engineer.

Regarding Figure 6-3, 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 6-3 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.

For those cases where a single value of the property can be used with the knowledge that the design is not very sensitive to variations in the property being considered, a sensitivity analysis may not be required.

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

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 P_a = 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 P_a = atmospheric pressure
Mayne and Rix (1993)	$G_{max} = 99.5(P_a)^{0.305}(q_c)^{0.695}/(e_0)^{1.13}$	kP_a (2)	Limited to cohesive soils P_a = 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

Modulus Reduction Curve (Darendeli, 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.

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

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:

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

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 (Darendeli, 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

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

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

Figure 6-1 Estimation of Residual Strength Ratio from SPT Resistance (Olson and Stark, 2002)

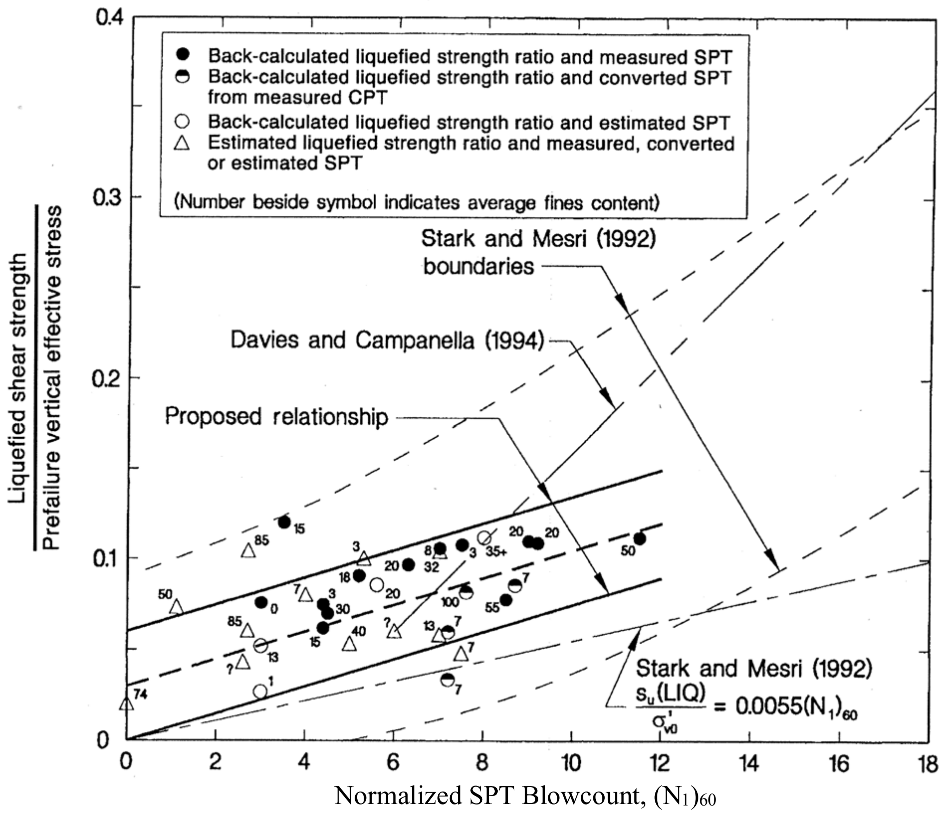


Figure 6-2 Estimation of Residual Strength Ratio from SPT Resistance (Idriss and Boulanger, 2007)

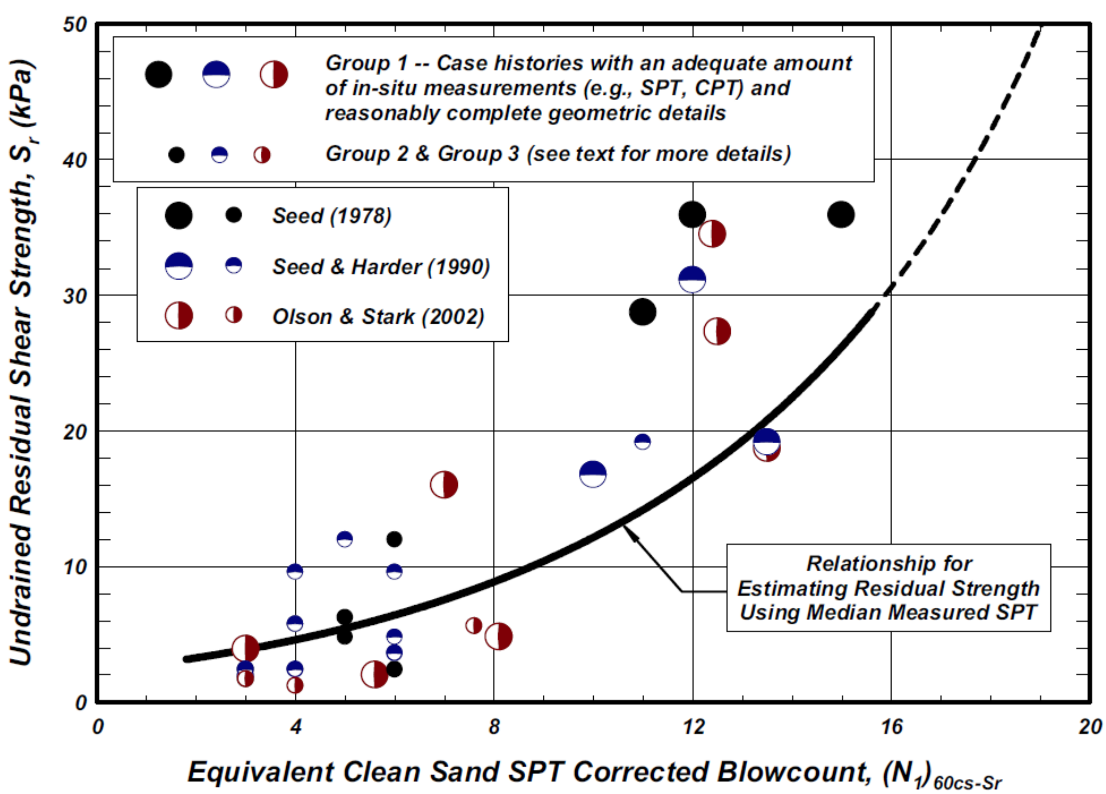


Figure 6-3 Estimation of Residual Strength Ratio from SPT Resistance (Idriss and Boulanger, 2007)

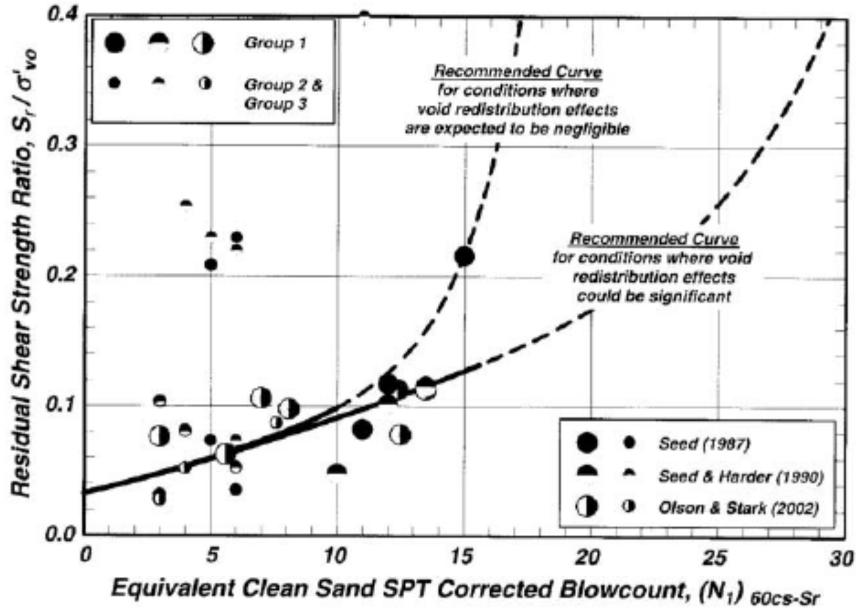
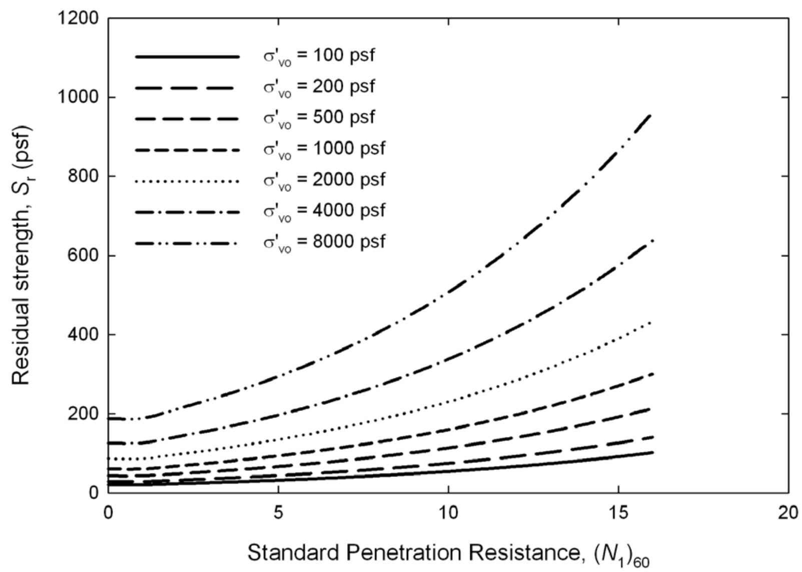


Figure 6-4 Variation of Residual Strength Ratio with SPT Resistance and Initial Vertical Effective Stress Using Kramer-Wang Model (Kramer, 2007)



6-2.3 Information for Structural Design

The geotechnical designer shall recommend a design earthquake ground motion based on the SEE for normal bridges and both the SEE and FEE for essential and critical bridges, and shall evaluate geologic hazards for the project. For code based ground motion analysis, the geotechnical designer shall provide the Site Class B/C boundary spectral accelerations at periods of 0.2 and 1.0 seconds, the PGA, the site class, and site coefficients for the PGA and spectral accelerations to account for the effect of the site class on the design accelerations.

In addition, the geotechnical designer should evaluate the site and soil conditions to the extent necessary to provide the following input for structural design, with consideration to the structure classification (i.e., normal, essential, or critical bridges) and the hazard level required (i.e., SEE for normal bridges, and both SEE and FEE for essential and critical bridges):

- 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 conditions should 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, except that the 2014 seismic hazard data and maps described previously shall be used instead of the 2002 hazard information provided in the AASHTO Specifications. However, a site specific hazard or ground motion response analysis is required in situations for which the General Procedure is not applicable, and may also be considered for situations in which the General Procedure is applicable.

6-3.1 Determination of Seismic Hazard Level

All transportation structures (e.g., bridges, pedestrian bridges, walls, , etc.) classified as "other" or "normal" (i.e., not critical or essential) are designed for the SEE (see Section 6-1.2.1) based on a hazard level of 7 percent PE in 75 years (i.e., an approximately 1,000 year return period). For essential or critical bridges, a two level seismic hazard design is required: the Safety Evaluation Earthquake (SEE) and the Functional Evaluation Earthquake (FEE). In this case, the SEE hazard level is as defined above. The FEE is based on a hazard level of 30 percent probability of exceedance in 75 years (or 210-year return period).

For buildings on terminal structures, the design hazard level shall be consistent with IBC requirements, which uses a risk adjusted 2,475 year event as its basis (MCER).

The AASHTO *Guide Specifications for LRFD Seismic Bridge Design* shall be used for WSDOT transportation facilities for code/specification based seismic hazard evaluation, except that Figures 6-5, 6-6, and 6-7 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 the SEE. By definition for Figures 6-5, 6-6, and 6-7, PGA, S_S and S_1 are for the Site Class B/C boundary (very hard or very dense soil or soft rock) conditions. The PGA contours in Figure 6-5, in addition S_s and S_1 in Figures 6-6 and 6-7, are based on information published by the USGS National Seismic Hazards Mapping Project (USGS, 2014) and supersede the AASHTO *LRFD Bridge Design Specifications* and the AASHTO *Guide Specifications for LRFD Seismic Bridge Design*. Interpolation between contours in Figures 6-5, 6-6, and 6-7 should be used when establishing the PGA for the Site Class B/C boundary for a project. High resolution images of these three acceleration maps are provided in Appendix 6-B.

Figure 6-5 Peak Horizontal Acceleration (%G) for 7% Probability of Exceedance in 75 Years for Site Class B/C Boundary (Adapted From USGS 2014)

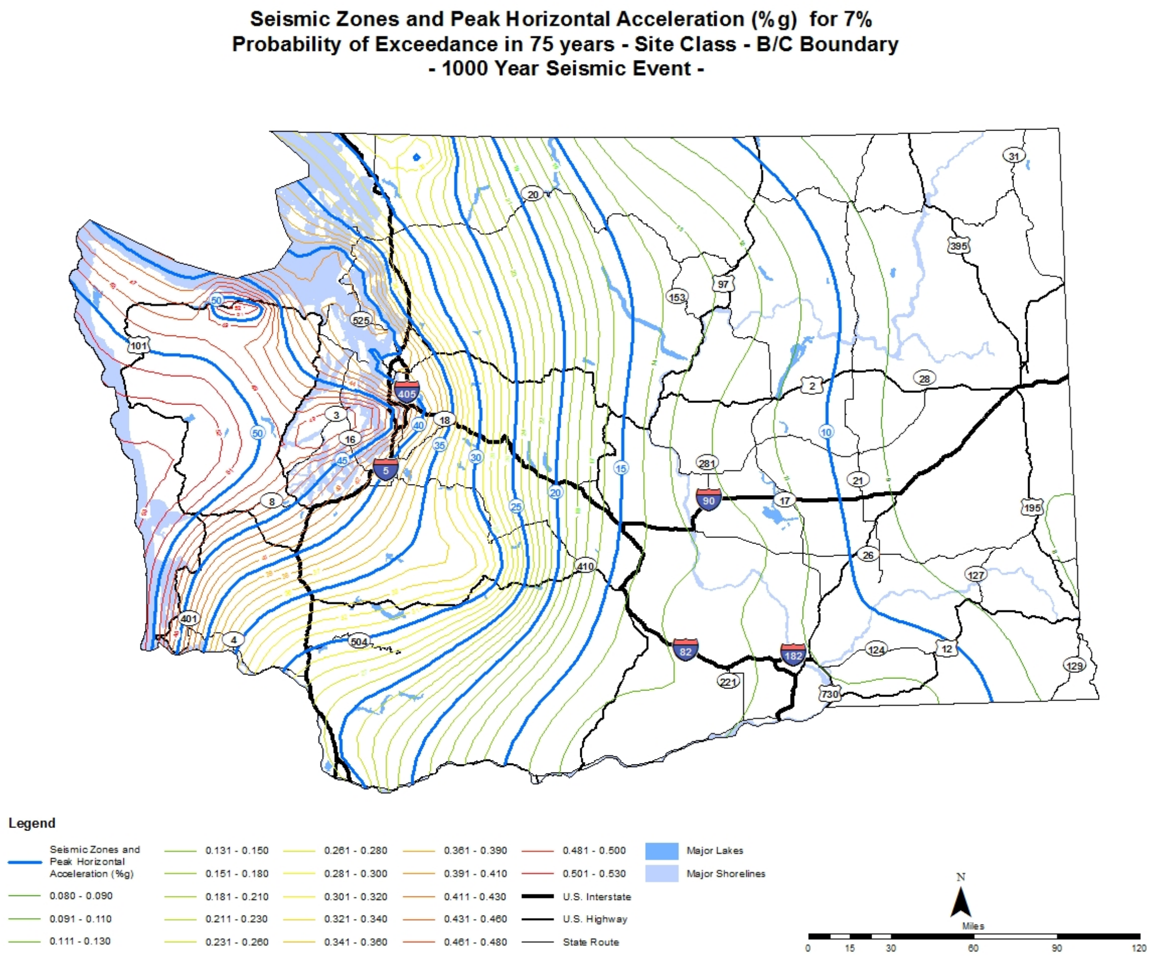


Figure 6-6 Horizontal Spectral Acceleration at 0.2 Second Period (%g) for 7% Probability of Exceedance in 75 Years with 5% of Critical Damping for Site Class B/C Boundary (Adapted from USGS 2014)

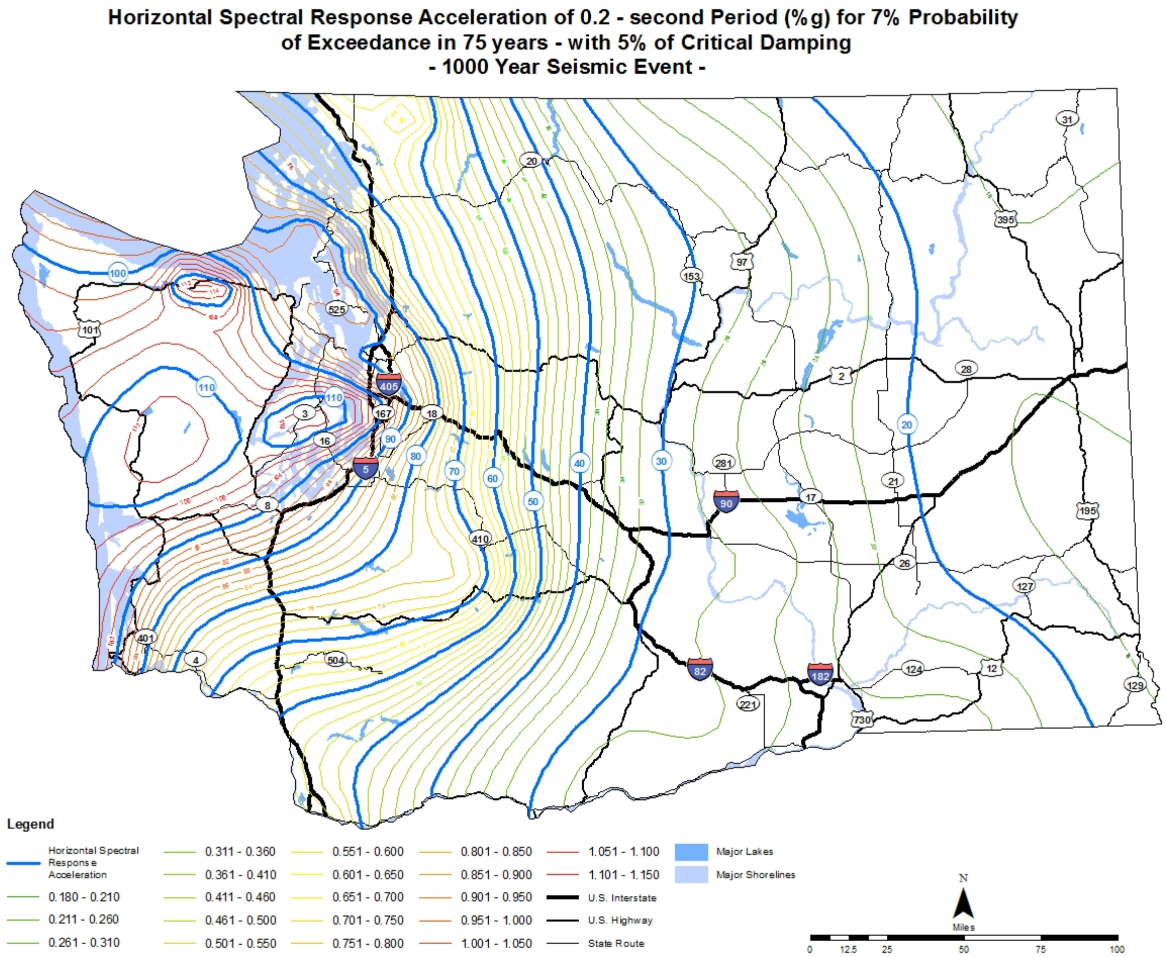
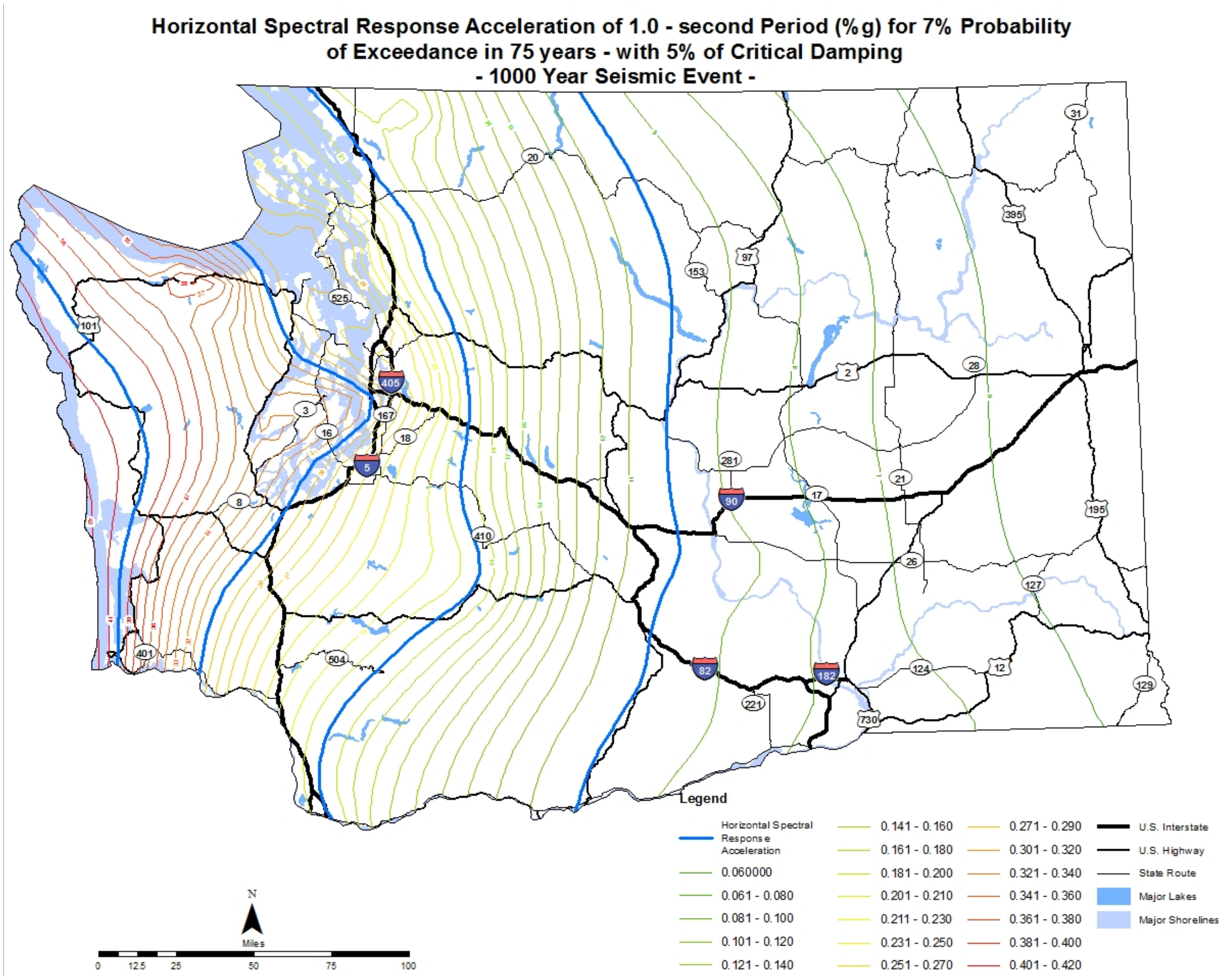


Figure 6-7 Horizontal Spectral Acceleration at 1.0 Second Period (%g) for 7% Probability of Exceedance in 75 Years With 5% of Critical Damping for Site Class B/C Boundary (Adapted from USGS 2014)



To obtain the PGA, 0.2 sec. spectral acceleration (S_s), and 1.0 sec. spectral acceleration values (S_1) for the FEE i.e., 30 percent probability of exceedance in 75 years (or 210-year return period), go to the USGS website at:
https://earthquake.usgs.gov/hazards/interactive

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 herein and 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.

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 Seismic Hazard Maps specified herein (USGS 2014) were developed, 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 or greater 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.

If a site specific hazard analysis is conducted, it shall be conducted in accordance with AASHTO *Guide Specifications for LRFD Seismic Bridge Design* and GDM Appendix 6-A.

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 and for the 2014 maps included in this GDM chapter. 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 ground motion attenuation equations 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.

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, and shelters, specification based seismic design parameters required by the most current version of the International Building Code (IBC) shall be used. For covered pedestrian walkways, the AASHTO *LRFD Bridge Design Specifications* or AASHTO *Guide Specifications for LRFD Seismic Bridge Design* shall be used.

The seismic design requirements of the IBC are based on a hazard level of 2 percent PE in 50 years which has been risk adjusted. The 2 percent PE in 50 years hazard level corresponds to the maximum considered earthquake (MCE), and the risk adjusted earthquake (MCER) corresponds to 1 percent probability of collapse in 50 years. The IBC identifies procedures to develop a maximum considered earthquake acceleration response spectrum, at the ground surface by adjusting Site Class B/C boundary 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. 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.

6-3.2 Site Ground Motion Response Analysis

6-3.2.1 *General Procedure*

The AASHTO Guide Specifications for LRFD Bridge Seismic Design require that site effects be included in determining seismic loads for design of bridges. Article 3.4.1 of the AASHTO Guide Specifications for LRFD Bridge Seismic Design (also Article 3.10.4.1 of the AASHTO *LRFD Bridge Design Specifications*) provide requirements for developing a design response spectrum when using the General Procedure. When conducting a seismic design based on the General Procedure, the site response spectrum shall be developed in accordance with the AASHTO Guide Specifications for LRFD Bridge Seismic Design, except that the USGS 2014 deaggregation/ground motions as depicted in Figures 6-5, 6-6, and 6-7 shall be used to establish the PGA, Ss, and S1 accelerations used as input. With regard to characterization of the site subsurface conditions, Tables 6-4, 6-5, and 6-6 shall be used as input to establish the site seismic response spectrum instead of the site coefficients provided in the AASHTO specifications.

The guide specifications characterize all subsurface conditions with six Site Classes (A through F). The site soil coefficients for PGA (F_{pga}), SS (F_a), and S_1 (F_v) provided in the Guide Specifications are updated herein for use with the 2014 seismic acceleration maps. Site soil coefficients for five of the Site Classes (A through E) are provided in Tables 6-4, 6-5, and 6-6. 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. Note that the site class should be determined considering the soils up to the ground surface, not just soil below the foundations.

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.

Table 6-4 Values of Site Coefficient, F_{pga}, for Peak Ground Acceleration

Site Class	Mapped Peak Ground Acceleration Coefficient (PGA)					
	PGA ≤ 0.10	PGA = 0.2	PGA = 0.3	PGA = 0.4	PGA = 0.5	PGA ≥ 0.6
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.9	0.9	0.9	0.9	0.9	0.9
C	1.3	1.2	1.2	1.2	1.2	1.2
D	1.6	1.4	1.3	1.2	1.1	1.1
E	2.4	1.9	1.6	1.4	1.2	1.1
F	*	*	*	*	*	*

* Site-specific response geotechnical investigation and dynamic site response analysis should be considered.

Note: Use straight line interpolation for intermediate values of *PGA*.

Table 6-5 Values of Site Coefficient, F_a, for 0.2-sec Period Spectral Acceleration

Site Class	Mapped Spectral Acceleration Coefficient at Period 0.2 sec (Ss)					
	Ss ≤ 0.25	Ss = 0.50	Ss = 0.75	Ss = 1.00	Ss = 1.25	Ss ≥ 1.50
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.9	0.9	0.9	0.9	0.9	0.9
C	1.3	1.3	1.2	1.2	1.2	1.2
D	1.6	1.4	1.2	1.1	1.0	1.0
E	2.4	1.7	1.3	1.0	0.9	0.9
F	*	*	*	*	*	*

* Site-specific response geotechnical investigation and dynamic site response analysis should be considered,

Note: Use straight line interpolation for intermediate values of S_s.

Table 6-6 Values of Site Coefficient, F_v, for 1.0-sec Period Spectral Acceleration

Site Class	Mapped Spectral Acceleration Coefficient at Period 1.0 sec (S1)					
	S1 ≤ 0.1	S1 = 0.2	S1 = 0.3	S1 = 0.4	S1 = 0.5	S1 ≥ 0.6
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.8	0.8	0.8	0.8	0.8	0.8
C	1.5	1.5	1.5	1.5	1.5	1.4
D	2.4	2.2	2.0	1.9	1.8	1.7
E	4.2	3.3	2.8	2.4	2.2	2.0
F	*	*	*	*	*	*

* Site-specific response geotechnical investigation and dynamic site response analysis should be considered,
Note: Use straight line interpolation for intermediate values of S_1.

6-3.2.2 *Site Specific Ground Motion Response Analysis*

When to Conduct: 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., within the upper 100 ft a sharp change in impedance between subsurface strata is present, etc.), or
- Site subsurface conditions are classified as Site Class F, and in some cases Site Class E as identified in Table 6-5.

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 should also be considered for sites where:

- the effects of liquefaction on the ground motion response could be overly conservative.
- basin effects could have a strong impact on the ground motion. However, the current (2014) acceleration maps partially consider basin effects. Whether or not basin effects should be considered for a particular site will be determined on a case by case basis as directed by the State Geotechnical Engineer and State Bridge Engineer.

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 as specified in the AASHTO *Guide Specifications for LRFD Seismic Bridge Design*, Article 3.4.1. For this comparison, the general procedure response spectrum is adjusted by the site coefficients (e.g., F_{pga}) in Tables 6-4, 6-5, and 6-6 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}.

No site coefficients are available for Site Class F and in some cases Site Class E. In these cases, a site specific ground response analysis shall 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). Furthermore, there are no site coefficients for liquefiable soils. No consensus currently exists regarding the appropriate site coefficients for these cases. 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* and as provided herein, 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., TF < 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, TF, greater than 1.0 sec., a site specific ground response analysis shall be conducted if liquefiable soils are determined to be present.

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 are not specifically considered in the site soil coefficients and a site- specific seismic ground response analysis should be conducted. 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.

How to Conduct: Input ground motion (i.e., acceleration time histories) selection and processing (e.g., matching through scaling with consideration to a target spectrum) for site specific ground motion response analyses should be conducted using procedures provided in Kramer et al. (2012). A WSDOT website link to the ground motion selection and processing tool cited in that reference (i.e., a modified version of SigmaSpectra with a ground motion database developed for Washington) is as follows: http://www.wsdot.wa.gov/Business/MaterialsLab/GeotechnicalServices.htm

Additional background and guidance on the subject of input ground motion selection and processing to produce a site specific base rock spectrum for conducting a site specific ground motion response analysis is provided in Kramer (1996), Bommer and Acevedo (2004), NEHRP (2011), and Kavazanjian, et al. (2011).

Once the input (i.e., base rock) ground motions are established, the frequency domain site specific response spectra needed for structure design (also commonly referred to as a site response analysis) is developed based on the requirements in Appendix 6-A.5. For the more complex sites or structures, a nonlinear time history analysis may be necessary. Appendix 6-A.6 provides requirements for conducting time history analysis to obtain the needed ground motions for structure design.

See Appendix 6-A for additional requirements and guidance regarding site specific ground response analyses, including requirements for time history analyses. Matasovic and Hashash (2012) also provide a good overview of the process used to conduct site specific ground motion response analysis from development of input ground motions to development of the structure design response spectra.

6-3.3 Need for Peer Review of Site Specific Hazard and Ground Motion Response Analyses

If a site specific hazard analysis is conducted, it shall be independently peer reviewed in all cases by someone with expertise in site specific seismic hazard analyses. When the site specific hazard 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).

A site specific ground motion response analysis to establish a response spectrum that is lower than two-thirds of the specification based spectrum shall be approved by the State Geotechnical and Bridge Engineers. If the site specific response analysis is conducted for this purpose, the site specific analysis shall be independently peer reviewed. The peer reviewer shall meet the same requirements as described in the previous paragraph, except that their expertise must be in the site specific ground motion response analysis technique used to conduct the analysis.

6-3.4 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.

6-3.5 Determination of A_s for Geotechnical Seismic Design

The ground acceleration A_s is determined by multiplying the PGA from Figure 6-8, which provides the ground acceleration for Class B/C rock/soil conditions, by its site coefficient F_{pga} (Table 6-4) to determine A_s for other site classes. A_s determined in this manner is used for 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 Table 6-4 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.

6-3.6 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 are 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. The best source of fault information that can be considered for design is the USGS at the following website: https://earthquake.usgs.gov/hazards/qfaults

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.

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.

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 using the residual strength of the soil to assess the potential for flow failure. 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. See Section 6-4.3.1 for additional requirements to assess flow failure and lateral spreading.

- **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 should be used 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 (see Andrus and Stokoe 2000); 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 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 less, or a cone tip resistance q_{ciN} of 150 or less, 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. See Idriss and Boulanger (2008) for additional information that relates liquefaction susceptibility to the depositional environment and geologic age of the deposit.

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, 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 – Other than through correlation to shear wave velocity as described in Andrus and Stokoe (2000), 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, or sonic coring, 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, Vs, 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, 2014), 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. National Academies of Sciences, Engineering, and Medicine (2016) provides the most recent consensus report on liquefaction and should be consulted to obtain the most up to date consensus guidance on this subject.

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
$\sigma_o{}'$ = 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), Shake2000 (Ordoñez, 2000), or DEEPSOIL (Hashash, et al., 2016) 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 time histories (see Section 6-3.2.2 and Appendix 6-A) 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 Anderson, et al. 2011 for additional explanation and guidance regarding effective stress modeling). 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). Note that for silts and low plasticity clays, a factor of safety is not calculated – the basis for determining whether or not liquefaction will occur is through cyclic simple shear or cyclic triaxial testing, or just whether or not the liquefaction susceptibility criteria are met. 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*). While performance based techniques can be accomplished using the WSLIQ software (Kramer, 2007), the performance based option (as well as the multi-hazard option) in that software uses the 2002 USGS ground motions and has not been updated to include more recent ground motion data that would be consistent with the ground motions used to produce the 2014 USGS seismic maps. Until that software is updated to use the new ground motion database, the multi-hazard and performance based options in WSLIQ shall not be used. 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 6-4.4. 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 6-4.4.2, 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. Updated procedures for estimating liquefaction settlement using CPT data are also provided in Zhang, et al. (2002). Example charts used to estimate liquefaction induced settlement using the Tokimatsu and Seed procedure and the Ishihara and Yoshimine procedure are presented as Figures 6-8 and 6-9, 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.

Figure 6-8 Liquefaction Induced Settlement Estimated Using the Tokimatsu and Seed procedure (Tokimatsu and Seed, 1987)

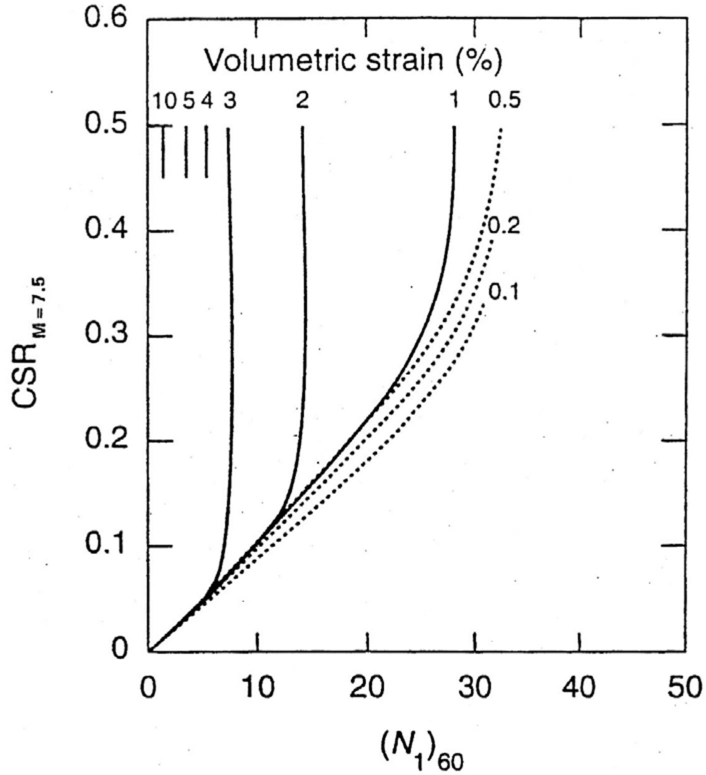
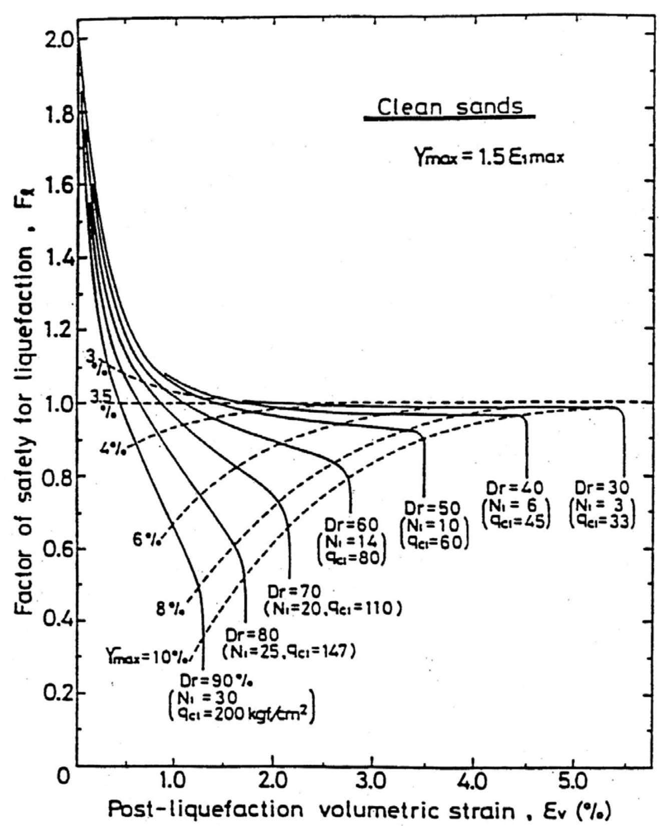


Figure 6-9 Liquefaction Induced Settlement Estimated Using the Ishihara and Yoshimine procedure (Ishihara and Yoshimine, 1992)



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 6-1 through 6-3 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 calculation results. See Section 6-2.2 for additional requirements regarding this issue.

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 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 increases. 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., DEEPSOIL) 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., DEEPSOIL) 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 account 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 Section 6-3.2 of this chapter. 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 account for the effects of liquefaction shall not be less than two-thirds of the site specific response spectrum developed from an equivalent linear or nonlinear total stress analysis (i.e., nonliquefied conditions), or alternatively a Site Class E response spectrum could be used for this purpose instead of the equivalent total stress analysis.

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, including reinforced slopes steeper than 1.2H:1V and noise 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. It should be noted, however, that the dataset used to develop these lateral spreading correlations is very limited for the upper end of earthquake magnitude (e.g., Mw > 8). Therefore, the potential for error in the estimate is greater for these very large magnitude earethquakes. 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., DEEPSOIL) 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. The more rigorous approaches should be used 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.

At locations close to the free face (e.g., $L/H < 5$), displacement mechanisms more closely related to localized instabilities such as slumping could become more dominant. This should be considered when estimating displacements close to the free face.

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 (e.g., weakening instability due to strength loss in clays), 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, k_h, 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 k_h 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 k_h 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:

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

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 6-7 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.

Table 6-7 Effective Shear Modulus Ratio (G/G_0)
(After ASCE 2000)

Site Class	Effective Peak Acceleration, SXS/2.5			
	SXS/2.5 = 0	SXS/2.5 = 0.1	SXS/2.5 = 0.4	SXS/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.

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. However, if soil liquefaction is likely to occur, then the effect of liquefaction on both the shape and the magnitude of the P-Y curves provided in this section shall be followed.

Available models used to estimate P-Y curves for liquefied soil vary considerably, which may affect the accuracy of the predicted behavior during liquefaction. Typical approaches that have been used in the past to address the effect of liquefaction on P-Y curves include the following:

1. Use the soft clay P-Y model, using the undrained residual strength as the cohesive strength for development of the P-Y curve as suggested by Wang and Reese (1998);
2. Use the static sand P-Y curve model, but with the peak shear strength reduced by a p-multiplier as recommended by Brandenberg, et al. (2007b) and Boulanger, et al. (2003);
3. Assume that the liquefied soil provides no resistance to lateral movement; and
4. Liquefied sand model as developed by Rollins, et al. (2005a, 2005b), and as applied in deep foundation lateral load analysis computer programs such as LPile (Isenhower and Wang 2015).

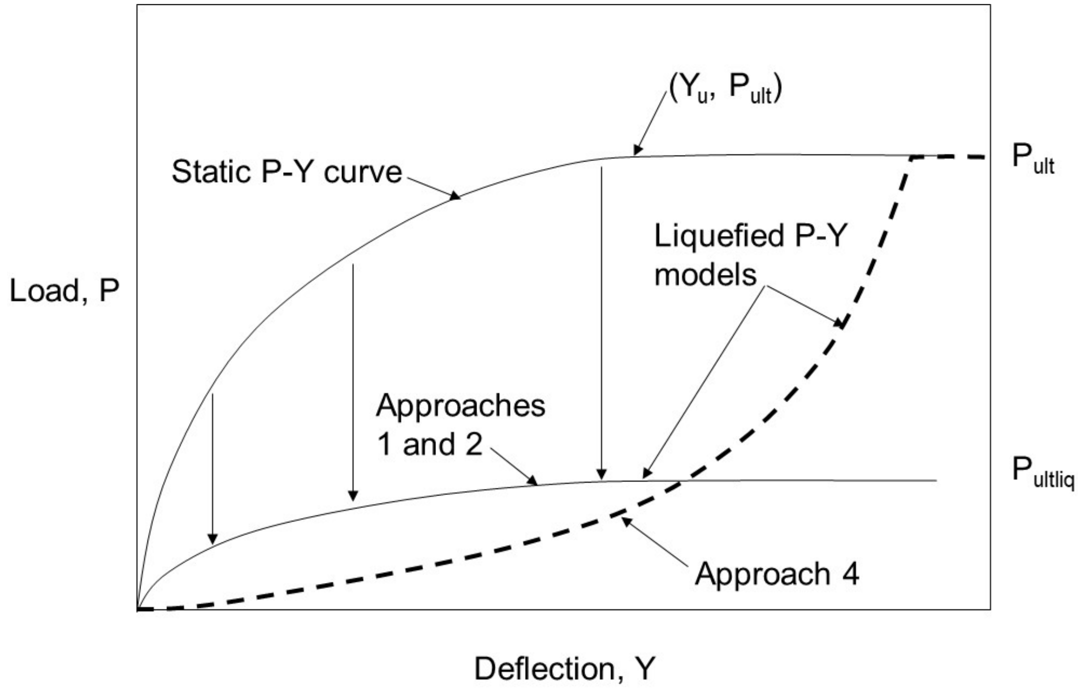
These approaches are conceptually illustrated in Figure 6-10.

Weaver, et al. (2005) and Rollins, et al. (2005a) provided comparisons between the various methods for 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 (i.e., approaches 1 and 2 identified above) 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 P-Y curves that will result from approaches 1 and 2, versus concave, or strain hardening, shape that will result from approach 4, respectively). Since the strain softening model is rather steeply increasing as a function of displacement at lower stress levels, the use of that model could be unconservative for moderate earthquakes in that there is not enough load to get past the steeper portion of the P-Y curve. They also found that the third approach (i.e., assume the liquefied soil has no shear strength), was overly conservative. The concave, or strain hardening, shape most accurately modeled the observed behavior of the piles tested in liquefied conditions (Weaver, et al. 2005; Rollins, et al. 2005a).

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.

Therefore, the expressions developed by Rollins, et al. (2005a, 2005b) and contained within LPile (Isenhower and Wang 2015) should be used to develop liquefied soil P-Y curves.

Figure 6-10 Conceptual P-Y Curve Models For Liquefied Conditions



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.

6-5.2 Earthquake Induced Earth Pressures on Retaining Structures

The procedures specified in the AASHTO *LRFD Bridge Design Specifications* shall be used to determine earth pressures acting on retaining walls during a seismic event. Due to the high rate of loading that occurs during seismic loading, the use of undrained strength parameters in the slope stability analysis may 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 and, as applied for WSDOT projects, *Design Procedure for Bridge Foundations Subject to Liquefaction-Induced Lateral Spreading* (Arduino, et al. 2017) located at: www.wsdot.wa.gov/research/reports/fullreports/874-2.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 (restrained case), and those foundation systems in which the ground can freely flow around them (unrestrained case). In general, the restrained case is used for bridge abutments, and the unrestrained case is used for interior bridge piers. However, to make a final determination, the spacing of the foundation elements, their stiffness as well as the stiffness of the superstructure, and the overall geometry of the structure may need to be considered.

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).
- The nonliquefied "crust" above the liquefied layer that moves with the liquefied layer is equal to the passive pressure of the nonliquefied layer soil moving against the foundation as later flow occurs.
- In both cases, the width of the pressure acting on the foundations is applied to the full foundation group width supporting the bridge pier. However, nothing was discussed in Yokoyama, et al. (1997) regarding the maximum center-center spacing of foundation elements that would result in the force being based in the full foundation group width. For a single foundation element supporting a bridge pier (e.g., a caisson or large diameter shaft), the width over which this lateral pressure is applied may be assumed to be the foundation width.
- 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 above the liquefied zone that do not liquefy but will settle, 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.

The neutral plane theory approach to assessing downdrag due to liquefaction may also be used, subject to the approval of the WSDOT State Geotechnical Engineer. See Muhunthan et al. (2017) for guidance.

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 6-11,
- 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.

The effect of any lateral or vertical deformation of the vertical inclusions on the structure the improved ground supports shall be taken into account in the design of the supported structure.

Figure 6-11 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.

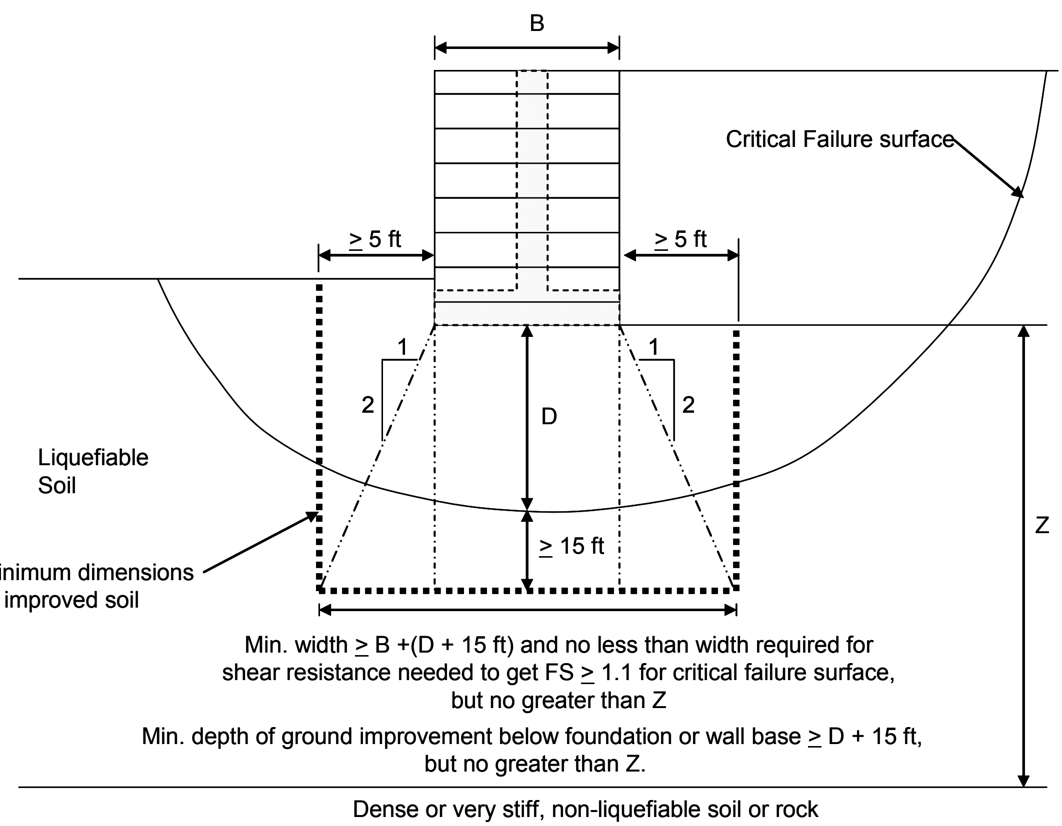
For the case where a “collar” of improved soil is placed outside and around the foundation, bridge abutment or other structure to be protected from the instability that liquefaction can cause, assume “B” in Figure 6-11 is equal to zero (i.e., the minimum width of improved ground is equal to D + 15 feet, 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 6-11 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 6-11, 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.

Figure 6-11 Minimum Dimensions for Soil Improvement Volume Below Foundations and Walls



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 for the following reasons:

- The drainage path time for pore pressure to dissipate may be too long,
- There is a potential for drainage structures to become clogged during installation and in service, and
- 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.

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6-7 Appendices

Appendix 6-A Site Specific Seismic Hazard and Site Response

Appendix 6-B High Resolution Seismic Acceleration Maps

Appendix 6-A Site Specific Seismic Hazard and Site Response

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.

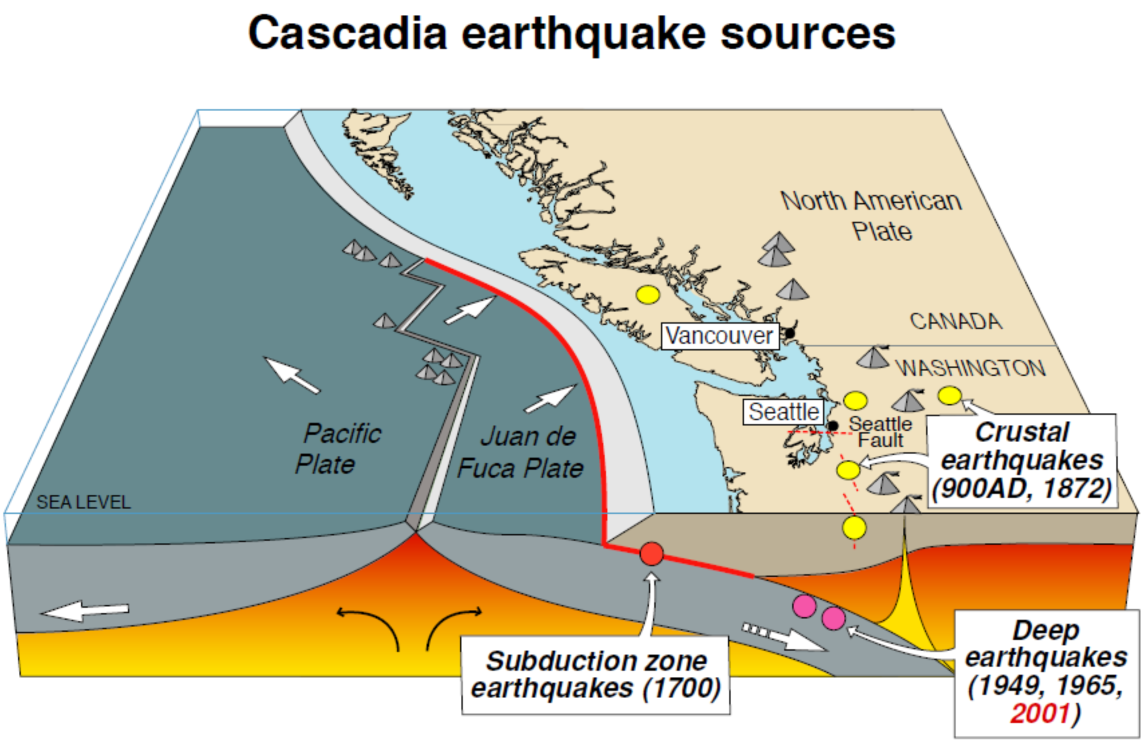
6-A.1 Background Information for Performing Site Specific Analysis

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 lies at the boundary between two crustal tectonic plates, where the offshore Juan de Fuca plate moves northeastward, converging with and subducting beneath the continental North American 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 deep CSZ Benioff or intraplate source zone, and (3) the CSZ interplate or interface source zone (i.e., the Cascadia Subduction Zone).

Figure 6-A-1 The Three Potential Seismic Source Zones Present in the Pacific Northwest (USGS 2017)



	Source	Affected area	Max. Size	Recurrence
	Subduction Zone	W.WA, OR, CA	M 9	500-600 yr
	Deep Juan de Fuca plate	W.WA, OR,	M 7+	30-50 yr
	Crustal faults	WA, OR, CA	M 7+	Hundreds of yr?

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.

Crustal seismicity is generally characterized using two types of models: known fault source models (such as the Seattle Fault zone, South Whidbey Island fault system, and the Tacoma fault), and seismicity-based background sources (which are based on historical data from earthquakes on unidentified or uncharacterized faults).

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 earthquake (magnitude 6.1) and the North Idaho earthquake (magnitude 5.5) (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 interplate source zones—anecdotally, Benioff zone earthquakes in Western Washington occur every 15 to 35 years or so, based on recent history. 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 is approximately 50 to 60 miles wide and varies along its length. As the fault becomes deeper, materials being faulted become ductile and the fault is unable to store mechanical stresses.

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 that contribute 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 (https://earthquake.usgs.gov/hazards/interactive/) 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. Since this chapter has been updated to require the use of the 2014 maps and associated data, it is required to use the 2014 deaggregation data. 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). 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*). While performance based techniques can be accomplished using the WSLIQ software (Kramer, 2007), the performance based option in that software uses the 2002 USGS ground motions and has not been updated to include more recent ground motion data that would be consistent with the ground motions used to produce the 2014 USGS seismic maps. Until that software is updated to use the new ground motion database, the performance based option in WSLIQ shall not be used.

If a single or a few larger magnitude earthquakes dominate the deaggregation, the magnitude of the single dominant earthquake or the weighted mean of the few dominant earthquakes in the deaggregation (weighted by the percent contribution of each source) 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 2014 probabilistic hazard maps on the USGS website 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), McGuire (2004), and Baker (2013).

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 interplate (interface) earthquake
- Cascadia subduction zone intraplate (Benioff) 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 best source of fault information that can be considered for design is the USGS at the following website: https://earthquake.usgs.gov/hazards/qfaults

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 2014 USGS probabilistic hazard maps as published herein essentially account for regional seismicity and attenuation relationships, recurrence rates, maximum magnitude of events on known 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 2014 USGS hazard maps only capture the effects of near- fault motions (i.e., ground motion directivity or pulse effects) or bedrock topography (i.e., so called basin effects) in a limited manner. 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.

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. See Matasović and Hashash (2012) for a good overview of the types of models available for site specific ground response analysis, their advantages, and their limitations.

One-Dimensional Equivalent Linear Models – One-dimensional equivalent linear site response computer codes, such as ProShake (EduPro Civil Systems, 1999) or Shake2000 (Ordoñez, 2000), and DEEPSOIL (Hashash, et al. 2016) 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). A-priori thresholds to evaluate differences between analyses and determine if a nonlinear analysis is needed (or if an equivalent linear analysis is acceptable) are provided in Kim et al. (2016). Additional information on the use and comparison of equivalent linear and nonlinear models is provided in Kaklamanos, et al. (2013, 2015), and Kim and Hashash (2013).

One-Dimensional Nonlinear Models – One-dimensional, nonlinear computer codes, such as D-MOD 2000, DESRA, and DEEPSOIL 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. Additional information regarding Rayleigh damping as well as newer damping models is provided in Kwok, et al. (2007), and Phillips and Hashash (2009).

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, DYNAFLOW, LSDYNA, and OPENSEES, 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. Furthermore, these modeling platforms are better equipped for nonlinear effective stress analysis for liquefiable sites and can incorporate models that can capture large strain dilation (e.g., UBCSand). 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 Section 6-2.2).

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 may 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 are 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: https://earthquake.usgs.gov/hazards/interactive.

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), Bommer and Acevedo (2004), NEHRP (2011), and Kramer, et al. (2012), should be consulted for specific requirements 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:

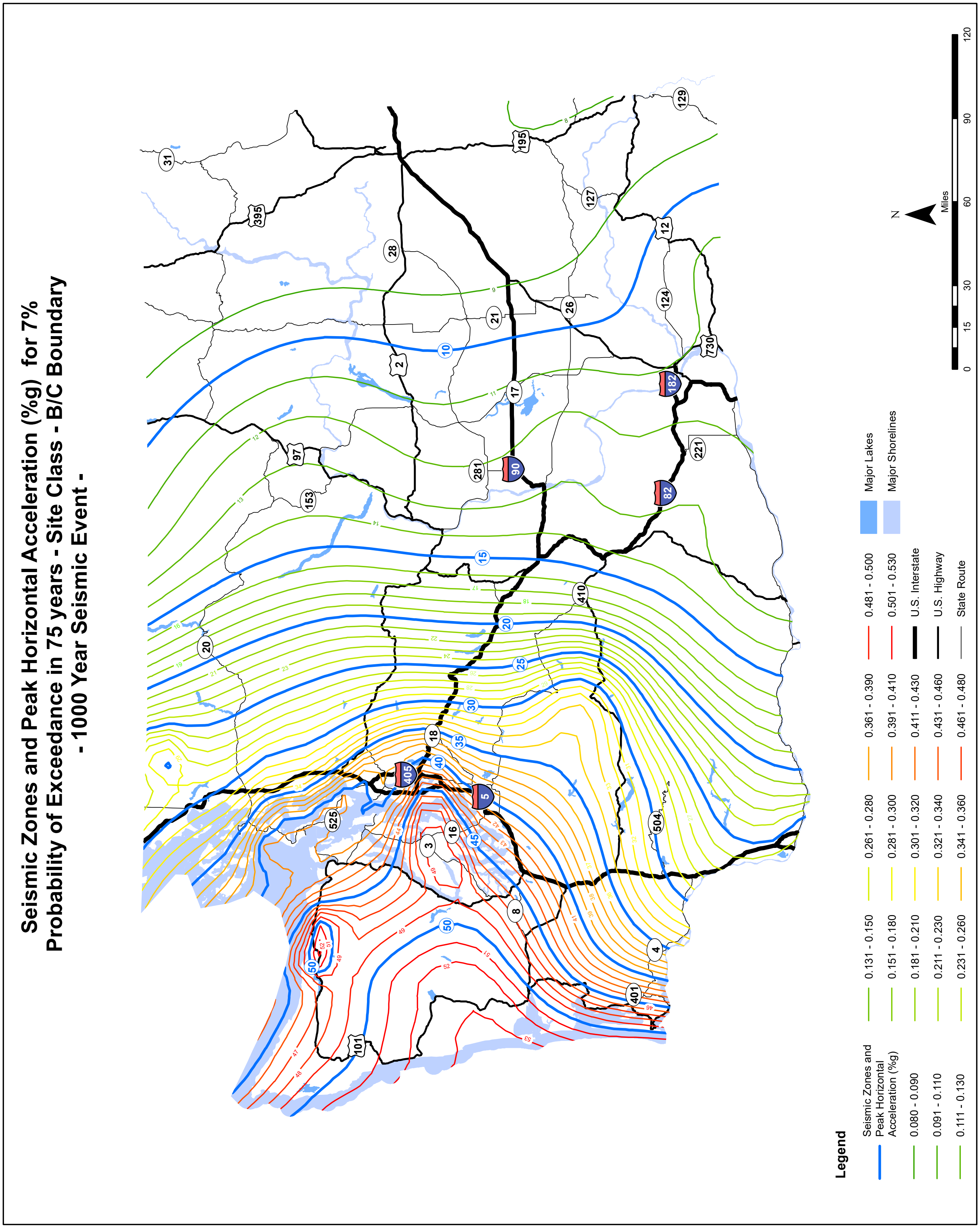
- How well the response spectrum generated from the scaled time histories matches the design response spectrum, and
- Similarity of the fault mechanisms for the time histories to those of recognized seismic source zones that contribute to the site's seismic hazard Also, if the earthquake records are 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.

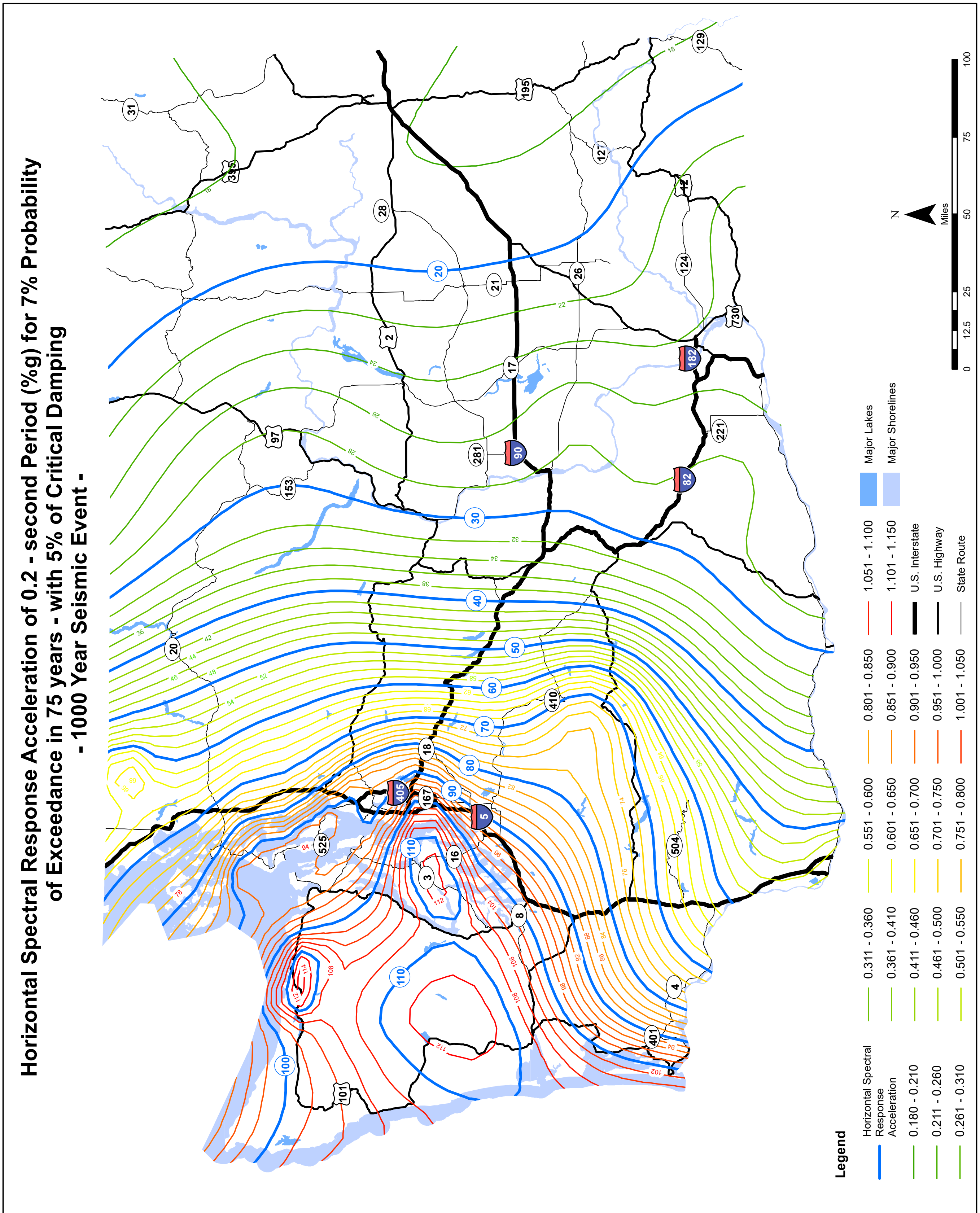
The requirements in the first bullet are most important to meet if the focus of the seismic modeling is structural and foundation design. The requirements in the second bullet are most important to meet if liquefaction and its effects are a major consideration in the design of the structure and its foundations.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

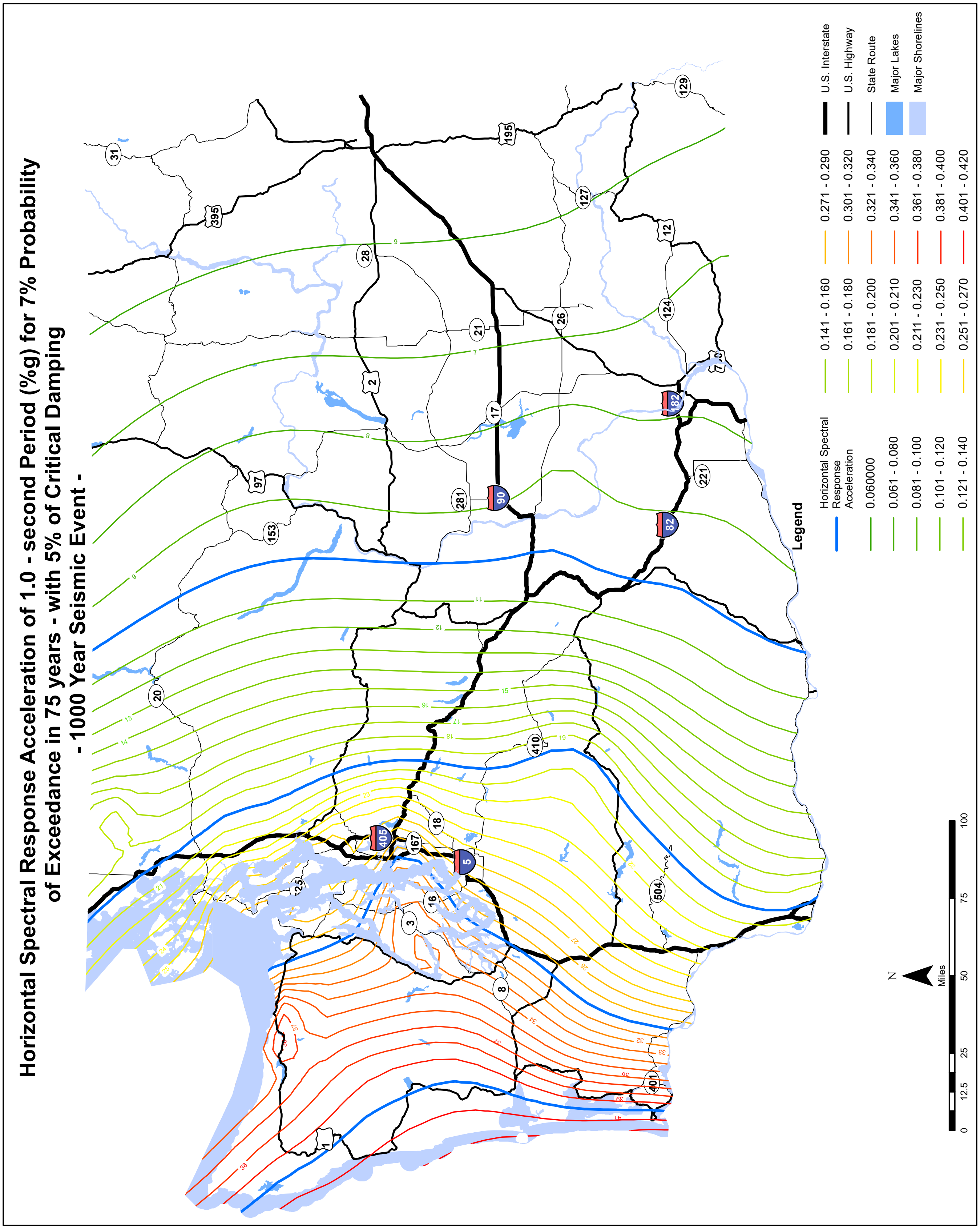
Note that a potential issue with the use of a spectrum-compatible motion that should be considered is that in western Washington, the uniform hazard spectrum (UHS) may have significant contributions from different sources that have major differences in magnitudes and site-to-source distances. The UHS 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.

Appendix 6-B High Resolution Seismic Acceleration Maps




Horizontal Spectral Response Acceleration of 0.2 - second Period (%g) for 7% Probability
of Exceedance in 75 years - with 5% of Critical Damping
- 1000 Year Seismic Event -
Legend
Horizontal Spectral Response Acceleration
0.180 - 0.210
0.211 - 0.260
0.261 - 0.310
0.311 - 0.360
0.361 - 0.410
0.411 - 0.460
0.461 - 0.500
0.501 - 0.550
0.551 - 0.600
0.601 - 0.650
0.651 - 0.700
0.701 - 0.750
0.751 - 0.800
0.801 - 0.850
0.851 - 0.900
0.901 - 0.950
0.951 - 1.000
1.001 - 1.050
1.051 - 1.100
1.101 - 1.150
U.S. Interstate
U.S. Highway
State Route
Major Lakes
Major Shorelines
N
Miles
0 12.5 25 50 75 100


Horizontal Spectral Response Acceleration of 1.0 - second Period (%g) for 7% Probability
of Exceedance in 75 years - with 5% of Critical Damping
- 1000 Year Seismic Event -
Legend
Horizontal Spectral Response Acceleration
0.060000
0.061 - 0.080
0.081 - 0.100
0.101 - 0.120
0.121 - 0.140
0.141 - 0.160
0.161 - 0.180
0.181 - 0.200
0.201 - 0.210
0.211 - 0.230
0.231 - 0.250
0.251 - 0.270
0.271 - 0.290
0.301 - 0.320
0.321 - 0.340
0.341 - 0.360
0.361 - 0.380
0.381 - 0.400
0.401 - 0.420
U.S. Interstate
U.S. Highway
State Route
Major Lakes
Major Shorelines
N
Miles
0 12.5 25 50 75 100

Chapter 7 Slope Stability Analysis

7.1 Overview

Slope stability analysis is used in a wide variety of geotechnical engineering problems, including, but not limited to, the following:

- Determination of stable cut and fill slopes
- Assessment of overall stability of retaining walls, including global and compound stability (includes permanent systems and temporary shoring systems)
- Assessment of overall stability of shallow and deep foundations for structures located on slopes or over potentially unstable soils, including the determination of lateral forces applied to foundations and walls due to potentially unstable slopes
- Stability assessment of landslides (mechanisms of failure, and determination of design properties through back-analysis), and design of mitigation techniques to improve stability
- Evaluation of instability due to liquefaction

Types of slope stability analyses include rotational slope failure, translational failure, irregular surfaces of sliding, and infinite slope failure. Stability analysis techniques specific to rock slopes, other than highly fractured rock masses that can in effect be treated as soil, are described in Chapter 12. Detailed stability assessment of landslides is described in Chapter 13.

7.2 Development of Design Parameters and Other Input Data for Slope Stability Analysis

The input data needed for slope stability analysis is described in Chapter 2 for site investigation considerations, Chapters 9 and 10 for fills and cuts, and Chapter 13 for landslides. Chapter 5 provides requirements for the assessment of design property input parameters.

Detailed assessment of soil and rock stratigraphy is critical to the proper assessment of slope stability, and is in itself a direct input parameter for slope stability analysis. It is important to define any thin weak layers present, the presence of slickensides, etc., as these fine details of the stratigraphy could control the stability of the slope in question. Knowledge of the geologic nature of the strata present at the site and knowledge of past performance of such strata may also be critical factors in the assessment of slope stability. See Chapter 5 for additional requirements and discussion regarding the determination and characterization of geologic strata and the determination of ESU's for design purposes.

Whether long-term or short-term stability is in view, and which will control the stability of the slope, will affect the selection of soil and rock shear strength parameters used as input in the analysis. For short-term stability analysis, undrained shear strength parameters should be obtained. For long-term stability analysis, drained shear strength parameters should be obtained. For assessing the stability of landslides, residual shear strength parameters will be needed, since the soil has in such has typically deformed enough to reach a residual value. For highly overconsolidated clays, such as the Seattle clays (e.g., Lawton Formation), if the slope is relatively free to deform after the cut is made or is otherwise unloaded, even if a structure such as a wall is placed to retain the slope after that deformation has already occurred, residual shear strength parameters should be obtained and used for the stability analysis. See Chapter 5 for requirements on the development of shear strength parameters.

Detailed assessment of the groundwater regime within and beneath the slope/landslide mass is also critical. Detailed piezіometric data at multiple locations and depths within and below the slope will likely be needed, depending on the geologic complexity of the stratigraphy and groundwater conditions. Potential seepage at the face of the slope must be assessed and addressed. In some cases, detailed flow net analysis may be needed. If seepage does exit at the slope face, the potential for soil piping should also be assessed as a slope stability failure mechanism, especially in highly erodable silts and sands. If groundwater varies seasonally, long-term monitoring of the groundwater levels in the soil should be conducted. If groundwater levels tend to be responsive to significant rainfall events, the long-term groundwater monitoring should be continuous, and on-site rainfall data collection should also be considered.

7.3 Design Requirements

Limit equilibrium methods shall be used to assess slope stability. The Modified Bishop, simplified Janbu, Spencer, or other widely accepted slope stability analysis methods should be used for rotational, translational and irregular surface failure mechanisms. Each limit equilibrium method varies with regard to assumptions used and how stability is determined. Therefore, a minimum of two limit equilibrium methods should be used and compared to one another to ensure that the the level of safety in the slope is accurately assessed. In cases where the stability failure mechanisms anticipated are not well modeled by limit equilibrium techniques, or if deformation analysis of the slope is required, more sophisticated analysis techniques (e.g., finite difference methods such as is used by the computer program FLAC) may be used in addition to the limit equilibrium methodologies. Since these more sophisticated methods are quite sensitive to the quality of the input data and the details of the model setup, including the selection of constitutive models used to represent the material properties and behavior, limit equilibrium methods should also be used in such cases, and input parameters should be measured or assessed from back-analysis techniques whenever possible. If the differences in the results are significant, the reasons for the differences shall be assessed with consideration to any available field observations to assess the correctness of the design model used. If the reasons for the differences cannot be assessed, and if the FLAC model provides a less conservative result than the limit equilibrium based methods, the limit equilibrium based methods shall govern the design.

If the potential slope failure mechanism is anticipated to be relatively shallow and parallel to the slope face, with or without seepage affects, an infinite slope analysis should be conducted. Typically, slope heights of 15 to 20 feet or more are required to have this type of failure mechanism. For infinite slopes consisting of cohesionless soils that are either above the water table or that are fully submerged, the factor of safety for slope stability is determined as follows:

$$FS = \frac{Tan\phi}{Tan\beta} \tag{7-1}$$

Where:
ϕ = the angle of internal friction for the soil
β = the slope angle relative to the horizontal

For infinite slopes that have seepage at the slope face, the factor of safety for slope stability is determined as follows:

$$FS = \left(\frac{\gamma_b}{\gamma s}\right)\frac{Tan\phi}{Tan\beta} \tag{7-2}$$

Where:
γb = the buoyant unit weight of the soil
γs = the saturated unit weight of the soil

Considering that the buoyant unit weight is roughly one-half of the saturated unit weight, seepage on the slope face can reduce the factor of safety by a factor of two, a condition which should obviously be avoided through some type of drainage if at all possible; otherwise much flatter slopes will be needed. When using the infinite slope method, if the FS is near or below 1.0 to 1.15, severe erosion or shallow slumping is likely. Vegetation on the slope can help to reduce this problem, as the vegetation roots add cohesion to the surficial soil, improving stability. Note that conducting an infinite slope analysis does not preclude the need to check for deeper slope failure mechanisms, such as would be assessed by the Modified Bishop or similar methods listed above.

Translational (block) or noncircular searches are generally more appropriate for modeling thin weak layers or suspected planes of weakness, and for modeling stability of long natural slopes or of geologic strata with pronounced shear strength anisotropy (e.g., due to layered/bedded macrostructure or pre-existing fracture patterns). If there is a disparately strong unit either below or above a thin weak unit, the user must ensure that the modeled failure plane lies within the suspected weak unit so that the most critical failure surface is modeled as accurately as possible. Circular searches for these types of conditions should generally be avoided as they do not generally model the most critical failure surface.

For very simplified cases, design charts to assess slope stability are available. Examples of simplified design charts are provided in NAVFAC DM-7 (US Department of Defense, 2005). These charts are for a c-φ soil, and apply only to relatively uniform soil conditions within and below the cut slope. They do not apply to fills over relatively soft ground, as well as to cuts in primarily cohesive soils. Since these charts are for a c-φ soil, a small cohesion will be needed to perform the calculation. If these charts are to be used, it is recommended that a cohesion of 50 to 100 psf be used in combination with the soil friction angle obtained from SPT correlation for relatively clean sands and gravels. For silty to very silty sands and gravels, the cohesion could be increased to 100 to 200 psf, but with the friction angle from SPT correlation (see Chapter 5) reduced by 2 to 3 degrees, if it is not feasible to obtain undisturbed soil samples suitable for laboratory testing to measure the soil shear strength directly. This should be considered general guidance, and good engineering judgment should be applied when selecting soil parameters for this type of an analysis. Simplified design charts shall only be used for final design of non-critical slopes that are approximately 10 feet in height or less and that are consistent with the simplified assumptions used by the design chart. Simplified design charts may be used as applicable for larger slopes for preliminary design.

The detailed guidance for slope stability analysis provided by Abramson, et al. (1996) should be used.

For additional design requirements for temporary slopes, including application of the applicable WAC's, see Sections 15.7 and 9.5.5.

7.4 Resistance Factors and Safety Factors for Slope Stability Analysis

For overall stability analysis of walls and structure foundations, design shall be consistent with Chapters 6, 8 and 15 and the AASHTO LRFD Bridge Design Specifications. For slopes adjacent to but not directly supporting structures, a maximum resistance factor of 0.75 should be used. For foundations on slopes that support structures such as bridges and retaining walls, a maximum resistance factor of 0.65 should be used. <u>This reduced resistance factor also applies if the slope is not directly supporting the structure, but if slope failure occurred, it could impact and damage the structure.</u> Exceptions to this could include minor walls that have a minimal impact on the stability of the existing slope, in which the 0.75 resistance factor may be used. Since these resistance factors are combined with a load factor of 1.0 (overall stability is assessed as a service limit state only), these resistance factors of 0.75 and 0.65 are equivalent to a safety factor of 1.3 and 1.5, respectively.

For general slope stability analysis of permanent cuts, fills, and landslide repairs, a minimum safety factor of 1.25 should be used. Larger safety factors should be used if there is significant uncertainty in the analysis input parameters. The Monte Carlo simulation features now available in some slope stability computer programs may be used for this purpose, from which a probability of failure can be determined, provided a coefficient of variation for each of the input parameters can be ascertained. For considerations regarding the statistical characterization of input parameters, see Allen, et al. (2005). For minimum safety factors and resistance factors for temporary cuts, see Section 15.7.

For seismic analysis, if seismic analysis is conducted (see Chapter 6 for policies on this issue), a maximum resistance factor of 0.9 should be used for slopes involving or adjacent to walls and structure foundations. This is equivalent to a safety factor of 1.1. For other slopes (cuts, fills, and landslide repairs), a minimum safety factor of 1.05 shall be used.

Conditions	Probability of Failure, Pf
Unacceptable in most cases	> 0.1
Temporary structures with no potential life loss and low repair cost	0.1
Slope of riverbank at docks, no alternative docks, pier shutdown threatens operations	0.01 to 0.02
Low consequences of failure, repairs when time permits, repair cost less than cost to go to lower Pf	0.01
Existing large cut on interstate highway	0.01 to 0.02
New large cut (i.e., to be constructed) on interstate highway	0.01 or less
Acceptable in most cases except if lives may be lost	0.001
Acceptable for all slopes	0.0001
Unnecessarily low	0.00001

Slope Stability – Probability of Failure (Adapted From Santamarina, et al., 1992)
Table 7-1

7.5 References

Abramson, L., Boyce, G., Lee, T., and Sharma, S., 1996, *Slope Stability and Stabilization Methods*, Wiley, ISBN 0471106224.

Allen, T., Nowak, A., and Bathurst, R., 2005, Calibration to Determine Load and Resistance Factors for Geotechnical and Structural Design. TRB Circular E-C079, 83 pp.

Santamarina, J. C., Altschaeffl, A. G., and Chameau, J. L., 1992, “Reliability of Slopes: Incorporating Qualitative Information,” Transportation Research Board, TRR 1343, Washington, D.C., pp. 1-5.

US Department of Defense, 2005, *Soil Mechanics*, Unified Facilities Criteria (UFC), UFC 3-220-10N,

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).

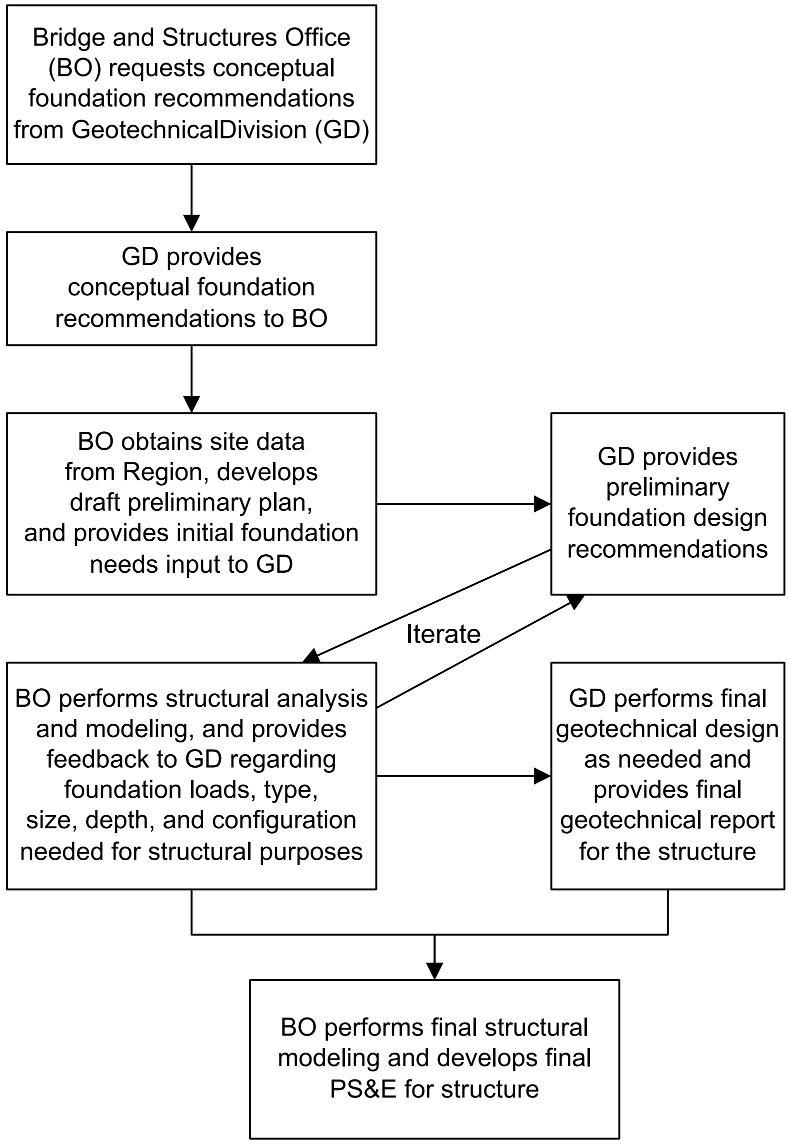
All structure foundations within WSDOT Right of Way or whose construction is administered by WSDOT shall be designed in accordance with the *Geotechnical Design Manual* (GDM) and the following documents:

- *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:

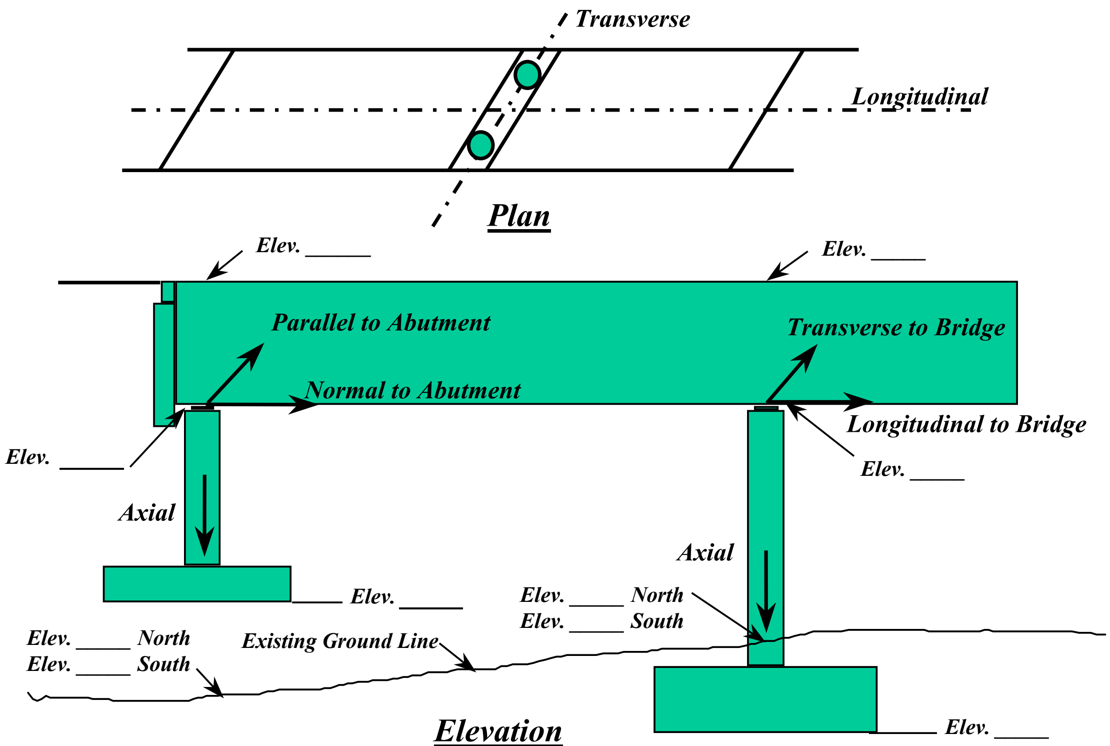
$$\Sigma \eta_i \gamma_i Q_i \leq \phi R_n \qquad (8\text{-}1)$$

Where:

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

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

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



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

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

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

8.6.1 Foundation Analysis to Establish Load Distribution for Structure

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

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

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

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

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

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

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

8.6.2 Downdrag Loads

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

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

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

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

Strength Limit State Downdrag Load Factors
Table 8-3

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

8.6.3 Uplift Loads due to Expansive Soils

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

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

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

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

8.6.4 Soil Loads on Buried Structures

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

8.6.5 Service Limit States

Foundation design at the service limit state shall include:

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

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

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

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

8.6.5.1 Tolerable Movements

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

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

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

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

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

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

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

Settlement Criteria for Bridges
Table 8-4

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

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

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

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

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

Settlement Criteria for Flexible Culverts
Table 8-6

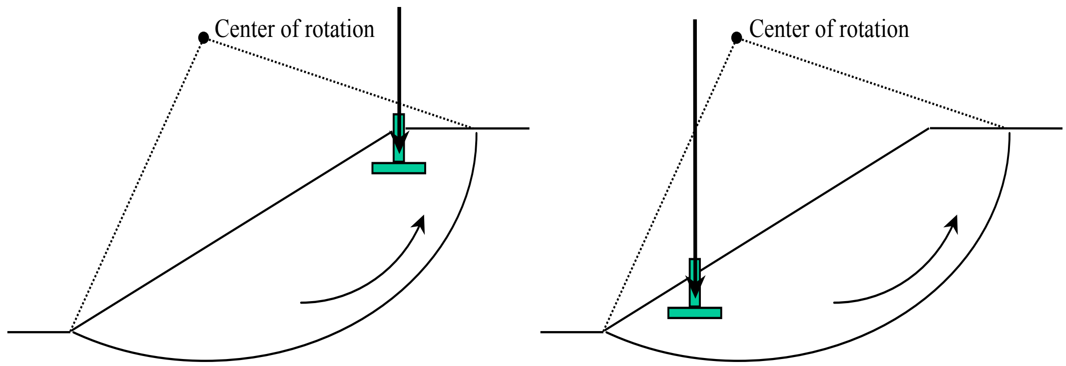
Rotation movements should be evaluated at the top of the substructure unit (in plan location) and at the deck elevation.

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

8.6.5.2 Overall Stability

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

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



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

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

8.6.5.3 Abutment Transitions

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

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

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

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

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

8.6.6 Strength Limit States

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

8.6.7 Extreme Event Limit States

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

8.7 Resistance Factors for Foundation Design – Design Parameters

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

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

8.8 Resistance Factors for Foundation Design – Service Limit States

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

8.9 Resistance Factors for Foundation Design – Strength Limit States

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

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

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

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

8.10 Resistance Factors for Foundation Design – Extreme Event Limit States

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

8.10.1 Scour

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

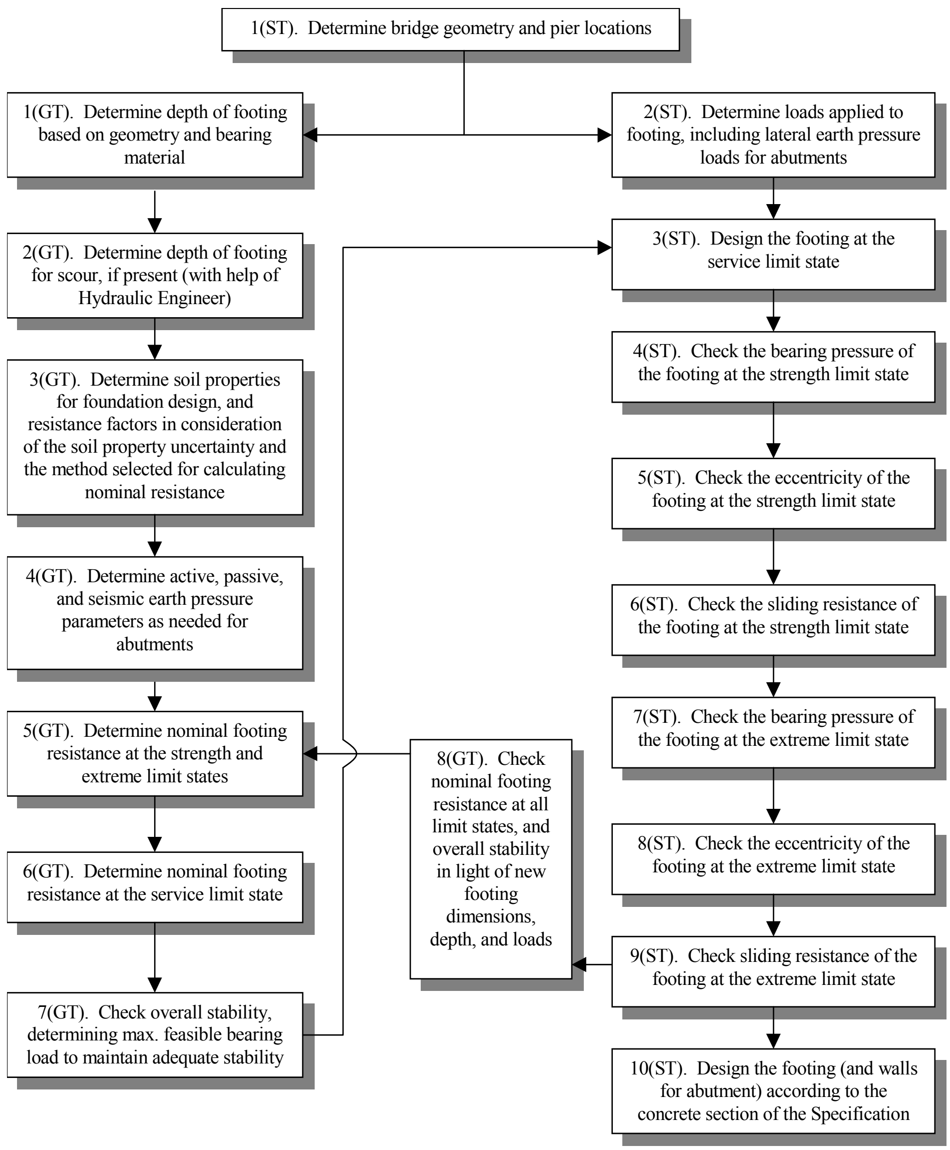
8.10.2 Other Extreme Event Limit States

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

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

8.11 Spread Footing Design

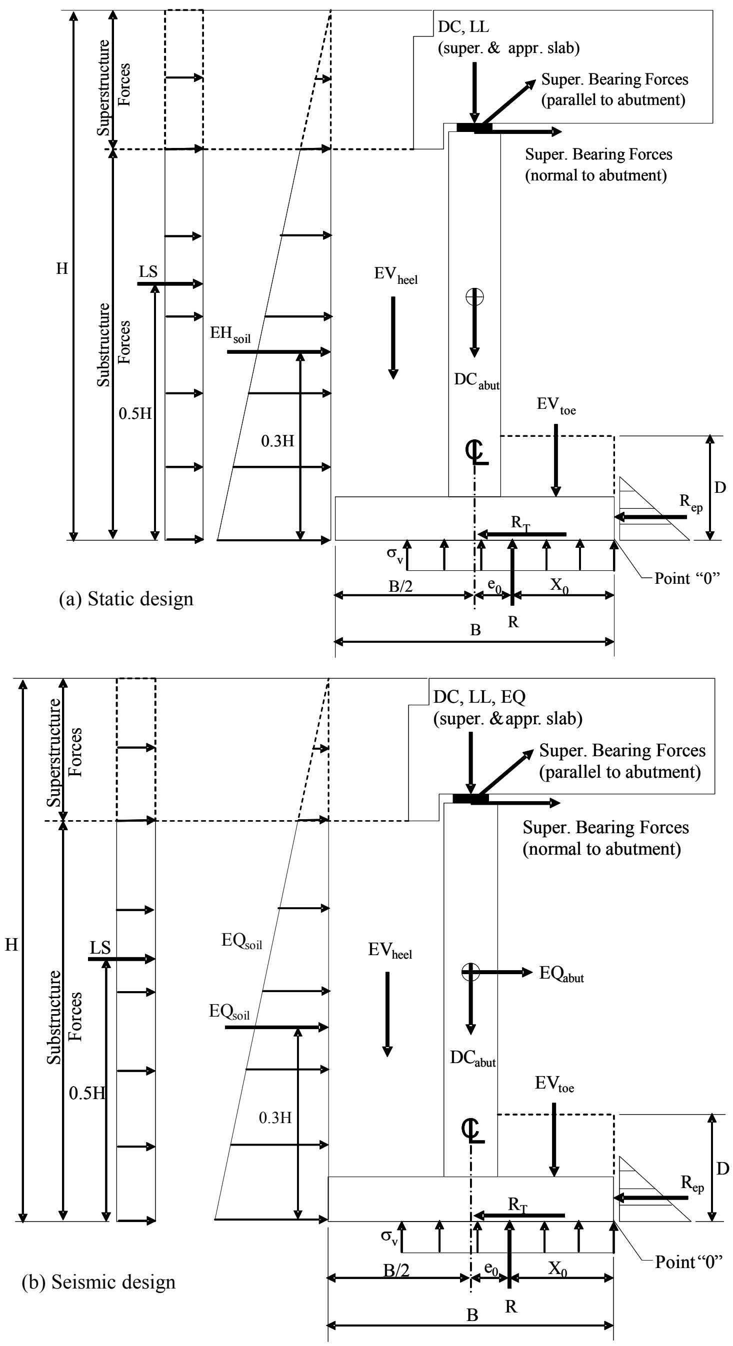
Figure 8-4 provides a flowchart that illustrates the design process, and interaction required between structural and geotechnical engineers, needed to complete a spread footing design. ST denotes steps usually completed by the Structural Designer, while GT denotes those steps normally completed by the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



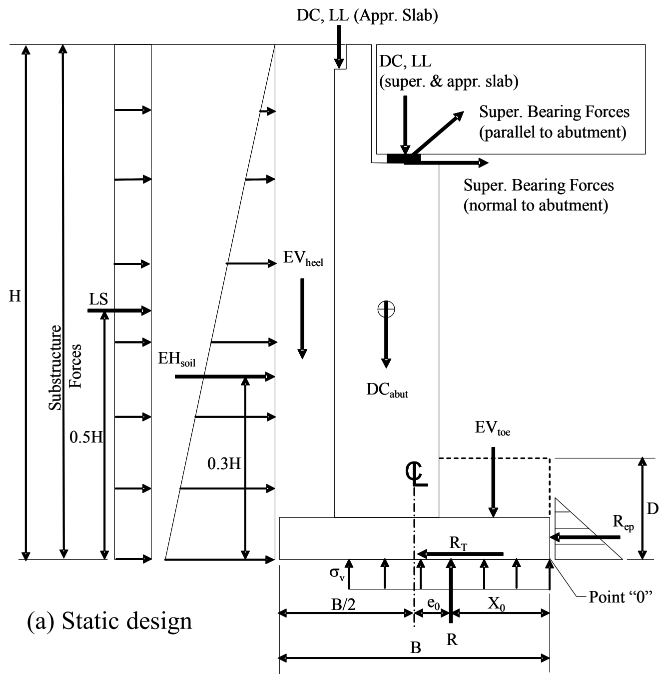
Flowchart for LRFD Spread Footing Design
Figure 8-4

8.11.1 Loads and Load Factor Application to Footing Design

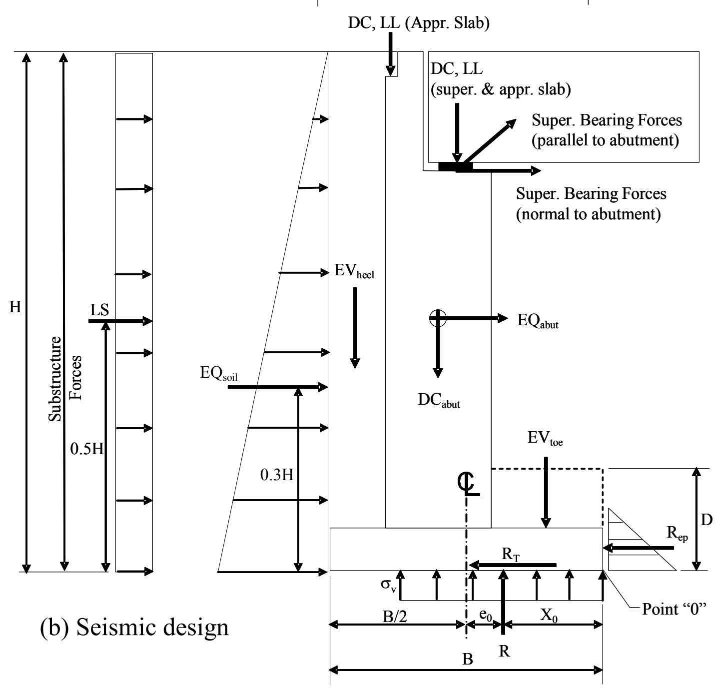
Figures 8-5 and 8-6 provide definitions and locations of the forces and moments that act on structural footings. Note that the eccentricity used to calculate the bearing stress in geotechnical practice typically is referenced to the centerline of the footing, whereas the eccentricity used to evaluate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



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



(a) Static design



(b) Seismic design

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

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

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

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

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

8.11.2 Footing Foundation Design

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

8.11.2.1 Footing Bearing Depth

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

8.11.2.2 Nearby Structures

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

8.11.2.3 Service Limit State Design of Footings

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

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

8.11.2.3.1 Settlement of Footings on Cohesionless Soils

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

8.11.2.3.2 Settlement of Footings on Rock

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

8.11.2.3.3 Bearing Resistance at the Service Limit State Using Presumptive Values

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

8.11.2.4 Strength Limit State Design of Footings

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

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

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

8.11.2.4.1 Theoretical Estimation of Bearing Resistance

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

8.11.2.4.2 Plate Load Tests for Determination of Bearing Resistance in Soil

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

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

8.11.2.4.3 Bearing Resistance of Footings on Rock

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

8.11.2.5 Extreme Event Limit State Design of Footings

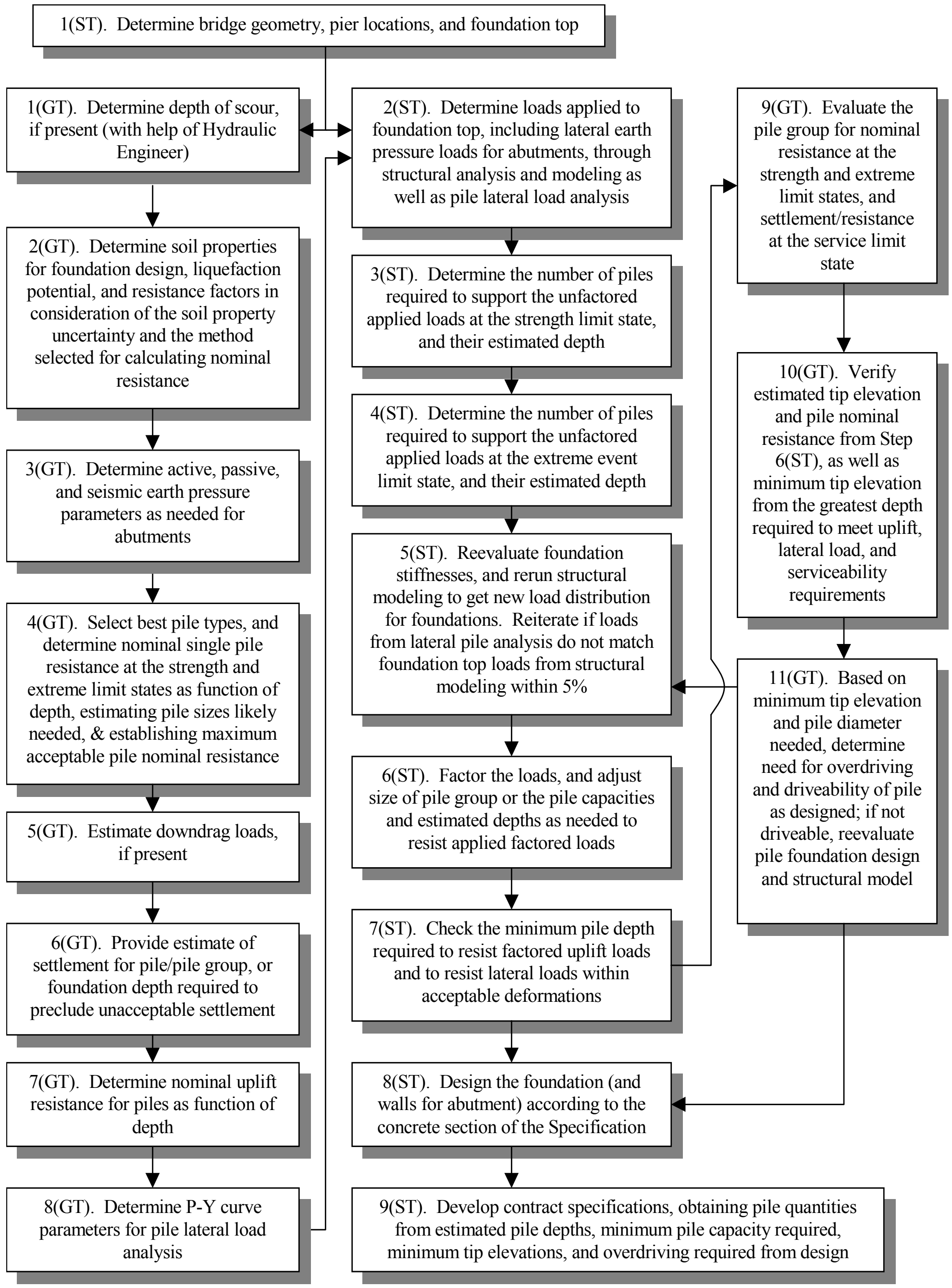
Footings shall not be located on or within liquefiable soil. Footings may be located on liquefiable soils that have been improved through densification or other means so that they do not liquefy. Footings may also be located above liquefiable soil in a non-liquefiable layer if the footing is designed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

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

For footings, whether on soil or on rock, the eccentricity of loading at the extreme limit state shall not exceed one-third (0.33) of the corresponding footing dimension, B or L, for $\gamma_{EQ} = 0.0$ and shall not exceed four-tenths (0.40) of the corresponding footing dimension, B or L, for $\gamma_{EQ} = 1.0$. 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

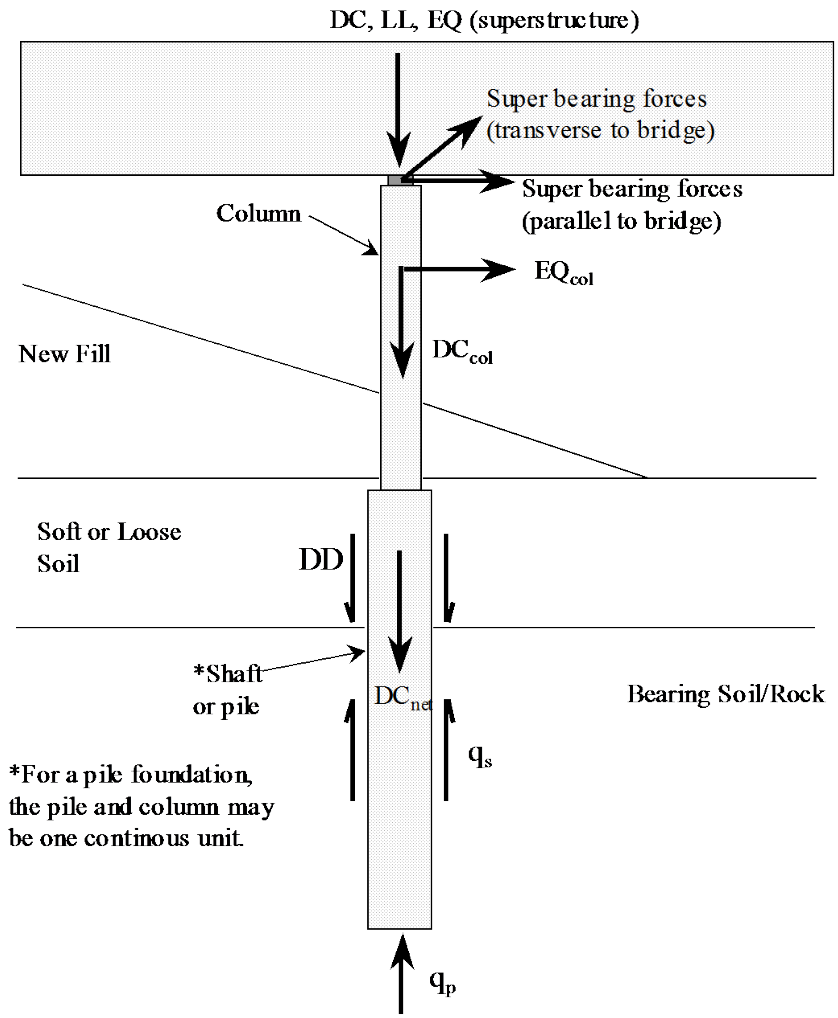
Figure 8-7 provides a flowchart that illustrates the design process, and interaction required between structural and geotechnical engineers, needed to complete a driven pile foundation design. ST denotes steps usually completed by the Structural Designer, while GT denotes those steps normally completed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



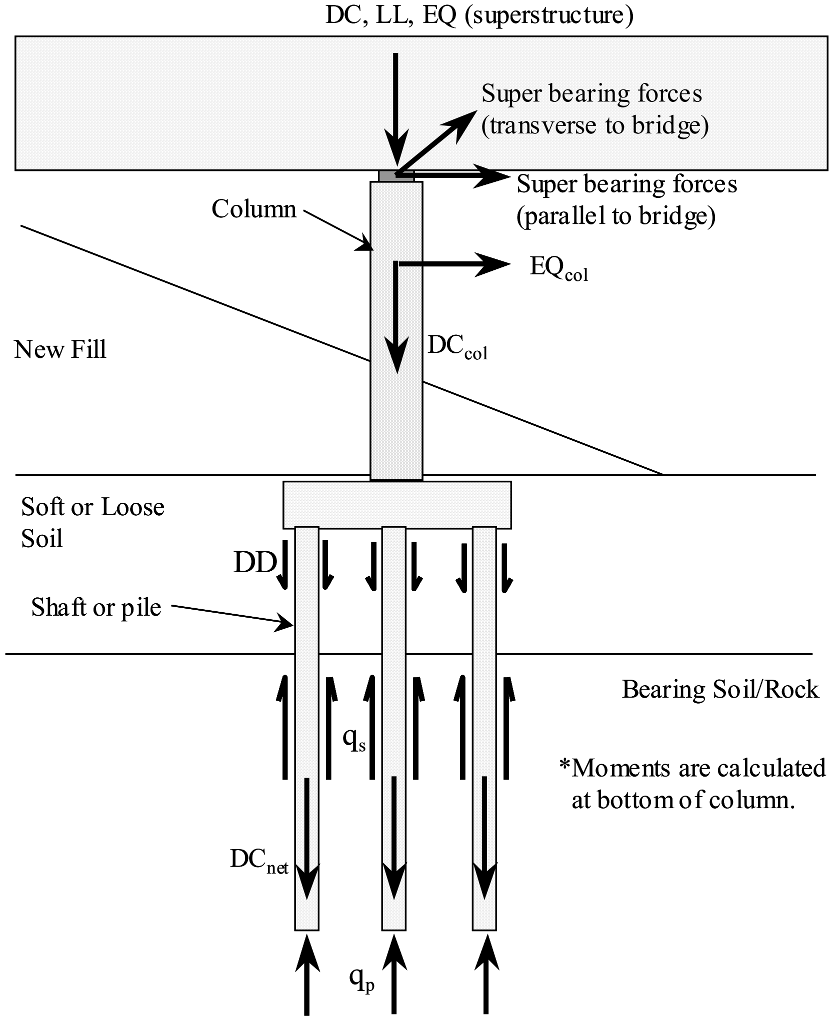
Design Flowchart for Pile Foundation Design
Figure 8-7

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})

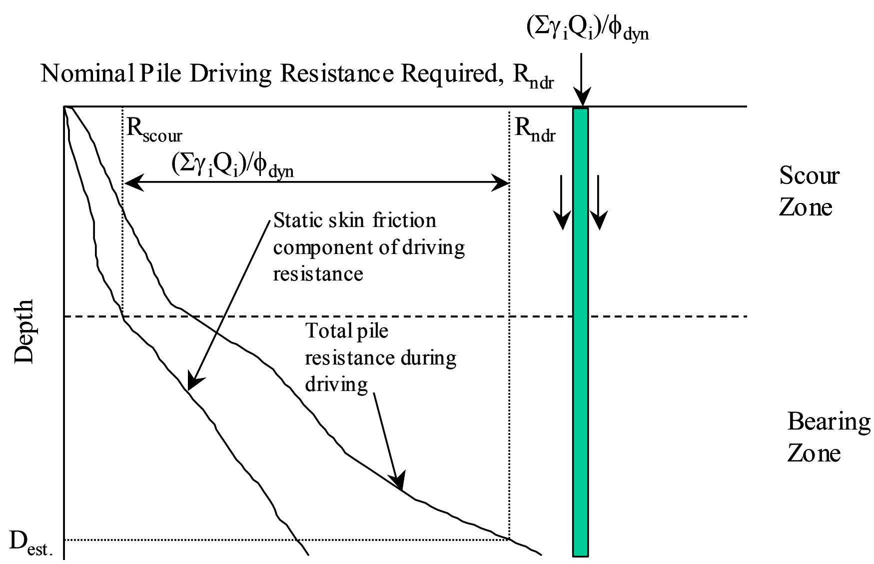
From Equation 8-1, the summation of the factored loads ($\Sigma\gamma_i Q_i$) must be less than or equal to the factored resistance (φR_n). Therefore, the nominal resistance R_n must be greater than or equal to the sum of the factored loads divided by the resistance factor φ. Hence, the nominal bearing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

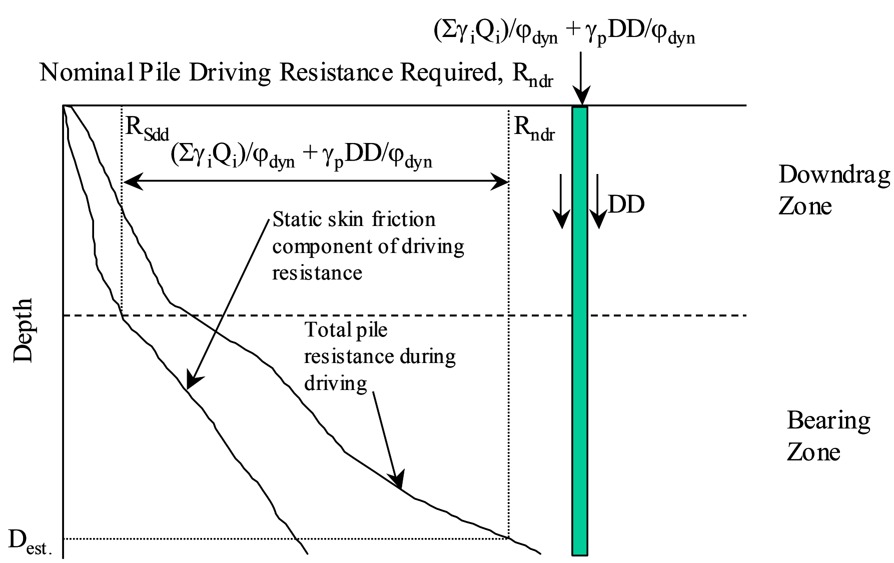
Similar to the derivation of Equation 8-2, the nominal bearing resistance of the pile needed to resist the factored loads, including downdrag, is therefore,

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

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:

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

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 $0.6f'_c$ 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

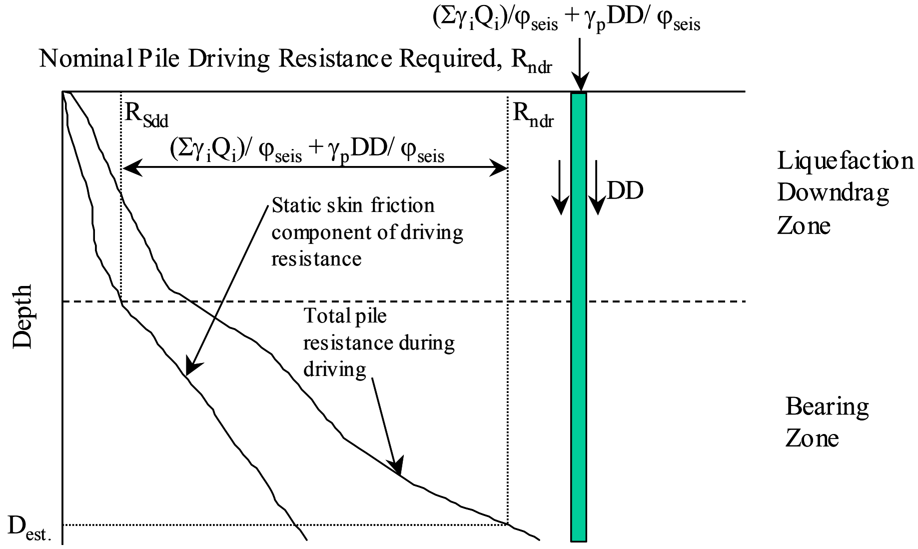
The nominal bearing resistance of the pile needed to resist the factored loads, including downdrag, is therefore,

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

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.

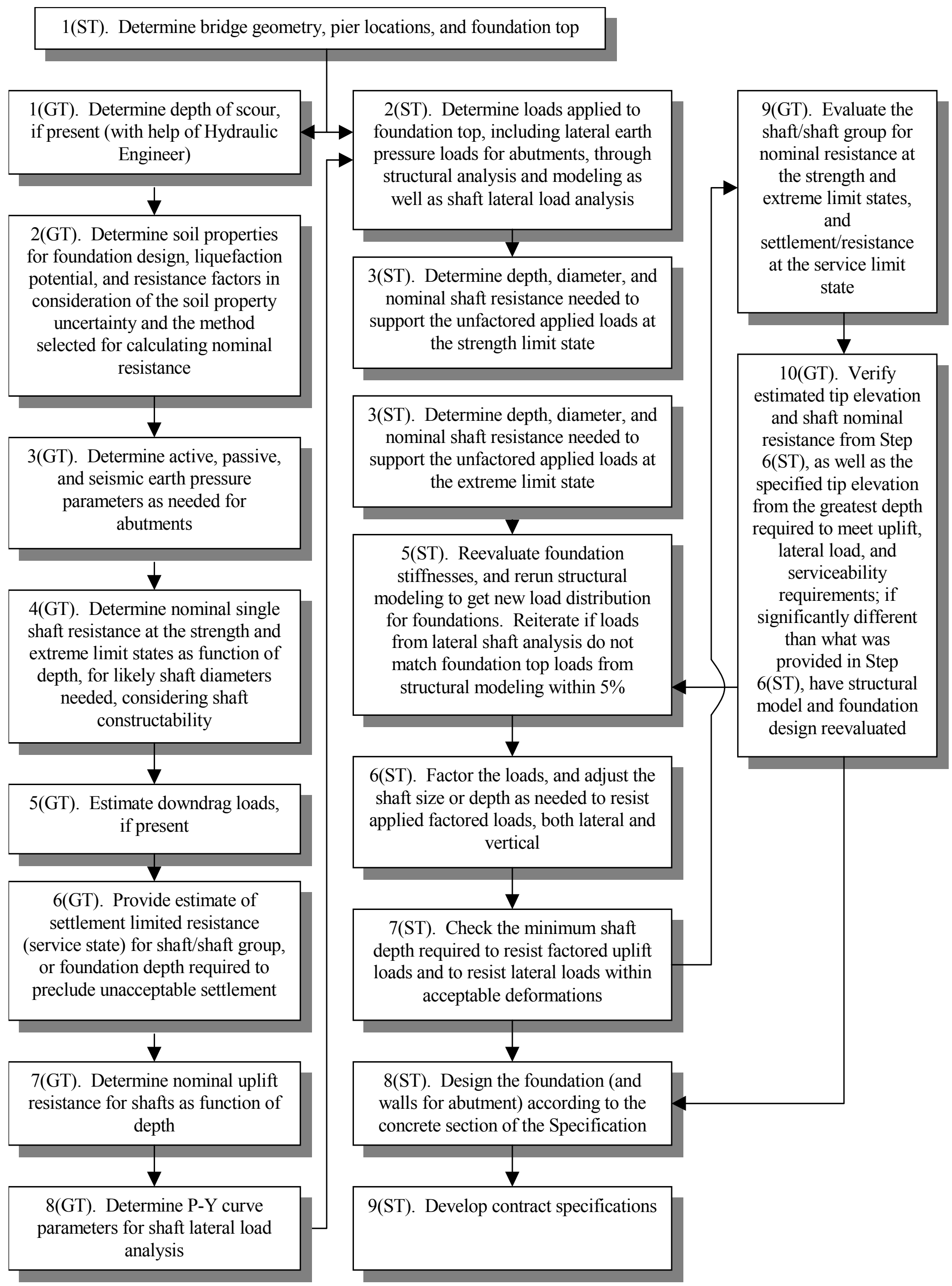
Regarding the reduction of P-Y soil strength and stiffness parameters to account for liquefaction, see Section 6.5.1.2.

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 <u>(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)</u>, 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 <u>and Appendix 8-B</u> 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) <u>may</u> be used to estimate the lateral resistance of the shafts <u>in lieu of P-Y methods.</u>

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).

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Appendix 8-A Approved Proprietary Foundation Systems (Vacant)

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

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

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

1

ATTACHMENT A — 2013 AGENDA ITEM __ - T-15

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

10.2—DEFINITIONS

ONE ADDITION BELOW – THE REMAINDER STAYS THE SAME

GSI—Geologic Strength Index

10.3—NOTATION

ONE ADDITION BELOW – THE REMAINDER STAYS THE SAME

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

10.4—SOIL AND ROCK PROPERTIES

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

10.4.2—Subsurface Exploration

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

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

C10.4.2

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

10.4.6—Selection of Design Properties

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

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

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

10.4.6.4—Rock Mass Strength

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

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

C10.4.6.4

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

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

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

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

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

~~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~~

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

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

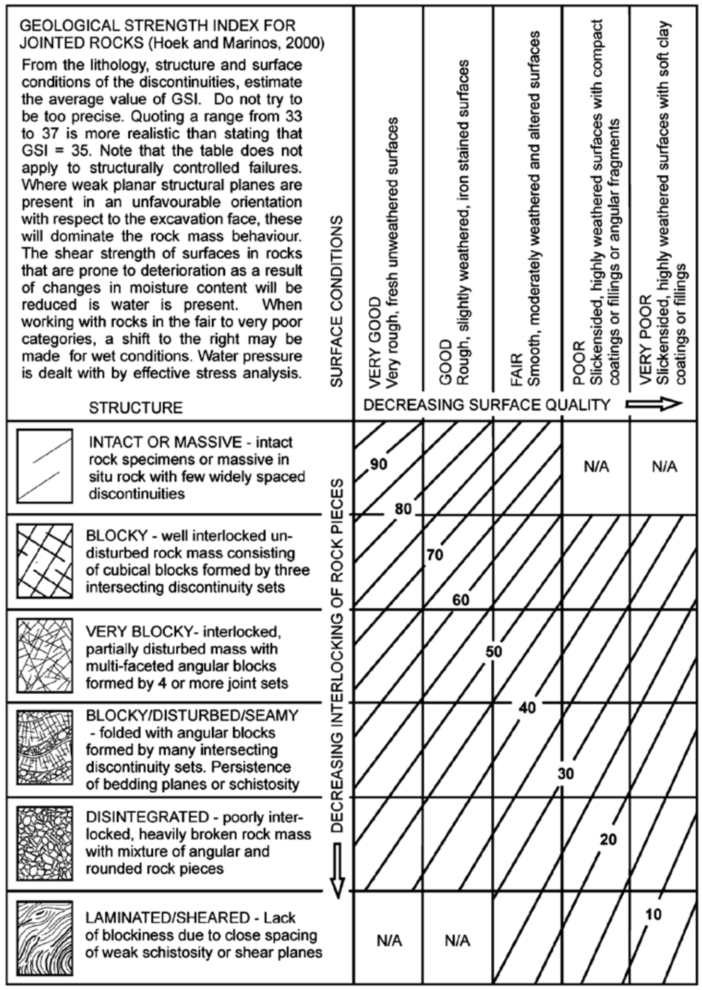

Figure 10.4.6.4-1—Determination of GSI for Jointed Rock Mass (Hoek and Marinos, 2000)

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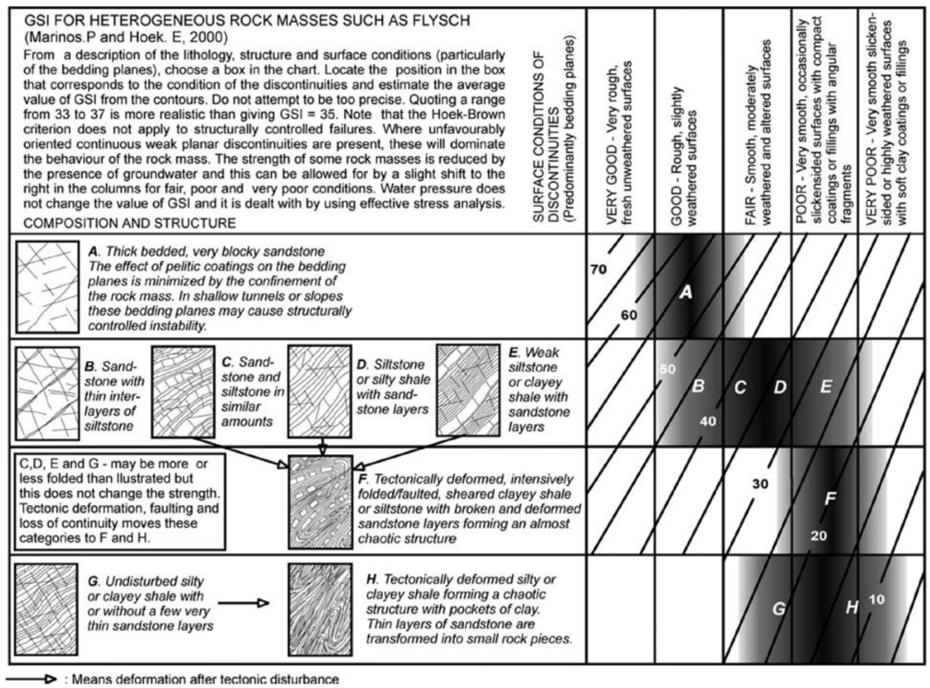

Figure 10.4.6.4-2—Determination of GSI for Tectonically Deformed Heterogeneous Rock Masses (Marinos and Hoek 2000)

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

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

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

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

$$\sigma'_1 = \sigma'_3 + q_u\left(m_b\frac{\sigma'_3}{q_u} + s\right)^a \quad (10.4.6.4\text{-}1)$$

in which:

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

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

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:

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

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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 polyminerallic 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:

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

~~where:~~

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

~~$E_m \leq E_i$~~

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

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

~~or~~

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

~~where:~~

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

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

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

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

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

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

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

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

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

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

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

Table 10.4.6.5-1—Estimation of E_m Based on GSI

Expression	Notes/Remarks	Reference
$E_m(GPa) = \sqrt{\frac{q_u}{100}}10^{\frac{GSI-10}{40}}$ for $q_u \le 100$ 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 ×10^3)			Standard Deviation (ksi × 10^3)
			Maximum	Minimum	Mean	
Granite	26	26	14.5	0.93	7.64	3.55
Diorite	3	3	16.2	2.48	7.45	6.19
Gabbro	3	3	12.2	9.8	11.0	0.97
Diabase	7	7	15.1	10.0	12.8	1.78
Basalt	12	12	12.2	4.20	8.14	2.60
Quartzite	7	7	12.8	5.29	9.59	2.32
Marble	14	13	10.7	0.58	6.18	2.49
Gneiss	13	13	11.9	4.13	8.86	2.31
Slate	11	2	3.79	0.35	1.39	0.96
Schist	13	12	10.0	0.86	4.97	3.18
Phyllite	3	3	2.51	1.25	1.71	0.57
Sandstone	27	19	5.68	0.09	2.13	1.19
Siltstone	5	5	4.76	0.38	2.39	1.65
Shale	30	14	5.60	0.001	1.42	1.45
Limestone	30	30	13.0	0.65	5.7	3.73
Dolostone	17	16	11.4	0.83	4.22	3.44

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

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

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

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

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.

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

Method/Soil/Condition			Resistance Factor
Nominal Axial Compressive Resistance of Single-Drilled Shafts, φ_{stat}	Side resistance in clay	α-method (~~O'Neill and Reese, 1999~~ 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.

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

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

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

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

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

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

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

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

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

10.8—DRILLED SHAFTS

10.8.1—General

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

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

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

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

10.8.1.5—Drilled Shaft Resistance

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

C10.8.1.5

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

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

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

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

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

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

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

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

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

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

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

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

10.8.1.6—Determination of Shaft Loads

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

10.8.1.6.2—Downdrag

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

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

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

C10.8.1.6.2

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

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

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

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

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

10.8.2—Service Limit State Design

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

10.8.2.2—Settlement

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

SHOWN

10.8.2.2.2—Settlement of Single-Drilled Shaft

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

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

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

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

C10.8.2.2.2

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

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

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

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

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

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

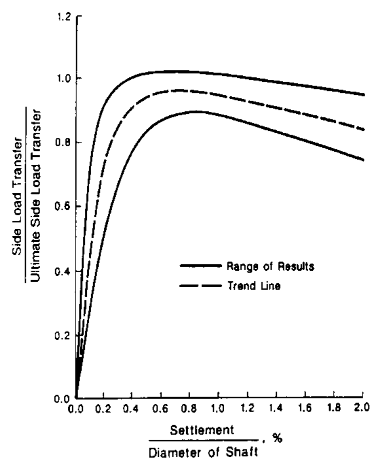


Figure 10.8.2.2.2-1 Normalized Load Transfer in Side Resistance versus Settlement in Cohesive Soils (from O'Neill and Reese, 1999)

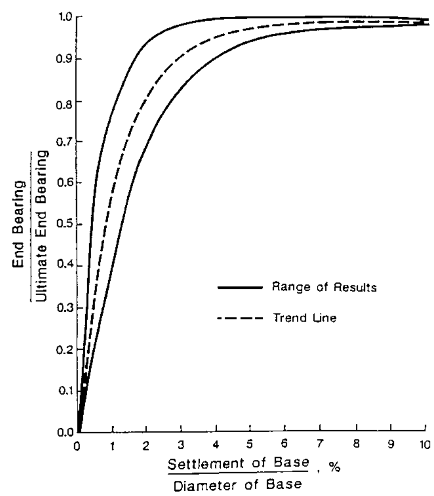


Figure 10.8.2.2.2-2—Normalized Load Transfer in End Bearing versus Settlement in Cohesive Soils (from O'Neill and Reese, 1999)

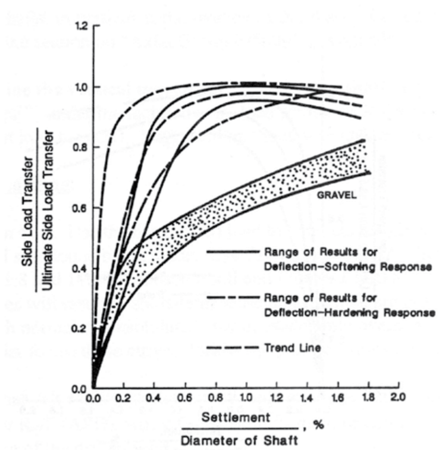


Figure 10.8.2.2.2-3—Normalized Load Transfer in Side Resistance versus Settlement in Cohesionless Soils (from O'Neill and Reese, 1999)

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

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

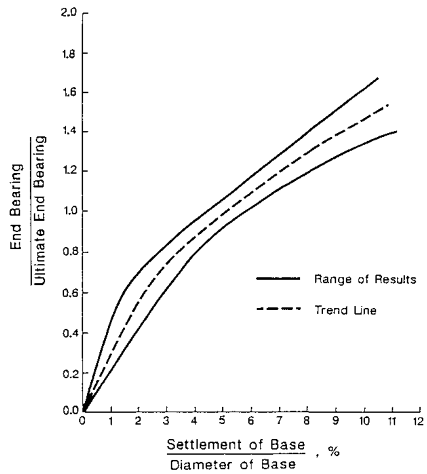


Figure 10.8.2.2.2-4—Normalized Load Transfer in End Bearing versus Settlement in Cohesionless Soils (from O'Neill and Reese, 1999)

10.8.2.2.3—Intermediate Geomaterials (IGMs)

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

C10.8.2.2.3

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

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

10.8.2.2.4—Group Settlement

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

C10.8.2.2.4

See commentary to Article 10.7.2.3.

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

10.8.2.3—Horizontal Movement of Shafts and Shaft Groups

The provisions of Articles 10.5.2.1 and 10.7.2.4 shall apply.

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

C10.8.2.3

See commentary to Articles 10.5.2.1 and 10.7.2.4.

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

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

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

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

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

10.8.3—Strength Limit State Design

10.8.3.1—General - *NO CHANGES – NOT SHOWN*

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

10.8.3.3—Scour - *NO CHANGES – NOT SHOWN*

10.8.3.4—Downdrag

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

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

C10.8.3.4

~~See commentary to Article 10.7.3.7.~~

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

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

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

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

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

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

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

10.8.3.5.1—Estimation of Drilled Shaft Resistance in Cohesive Soils

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

10.8.3.5.1b—Side Resistance

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

C10.8.3.5.1b

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

The adhesion factor is an empirical factor used to

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

in which:

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

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

where:

S_u = undrained shear strength (ksf)

α = adhesion factor (dim)

p_a = atmospheric pressure (= 2.12 ksf)

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

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

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

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

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

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

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

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

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

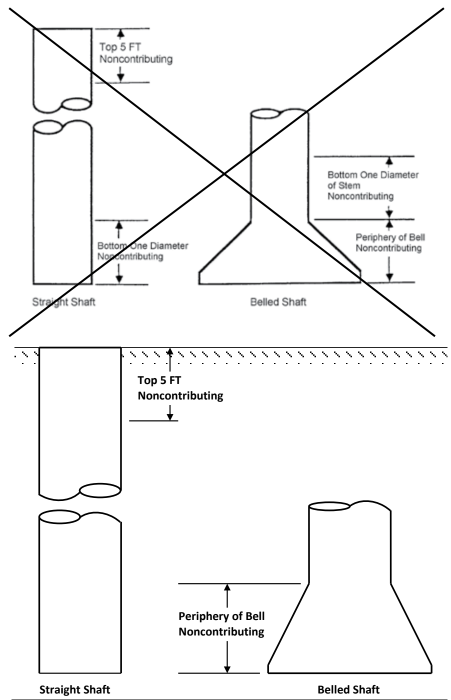


Figure 10.8.3.5.1b-1—Explanation of Portions of Drilled Shafts Not Considered in Computing Side Resistance (~~O'Neill and Reese, 1999~~ Brown et al., 2010)

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

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

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

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

10.8.3.5.1c—Tip Resistance

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

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

in which:

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

where:

D = diameter of drilled shaft (ft)

Z = penetration of shaft (ft)

C10.8.3.5.1c

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

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

S_u = undrained shear strength (ksf)

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

10.8.3.5.2—Estimation of Drilled Shaft Resistance in Cohesionless Soils

10.8.3.5.2a—General

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

C10.8.3.5.2a

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

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

10.8.3.5.2b—Side Resistance

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

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

~~in which, for sandy soils:~~

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

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

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

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

~~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.~~

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

$q_s = \beta\, \sigma'_v$ (10.8.3.5.2b-1)

in which:

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

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:

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

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:

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

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

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:

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

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

where:

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

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

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

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

~~where:~~

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

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

C10.8.3.5.2c

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

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

~~the shaft (ksf)~~

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

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

10.8.3.5.4—Estimation of Drilled Shaft Resistance in Rock

10.8.3.5.4a—General

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

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

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

C10.8.3.5.4a

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

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

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

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

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

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):~~

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

~~where:~~

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

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

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

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

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

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

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.~~

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

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

where:

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 2.0B is either intact or tightly jointed, i.e., no compressible material or gouge-filled seams, and the depth of the socket is greater than 1.5B ~~(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.

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

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

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

~~where:~~

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

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

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

In which:

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

where:

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

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

q_u = uniaxial compressive strength of intact rock

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

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

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

10.8.3.5.4d—Combined Side and Tip Resistance

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

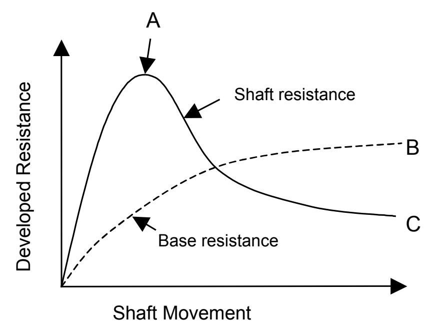
C10.8.3.5.4d

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

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

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

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



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

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

<u>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</u>

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*

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

Chapter 9 Embankments

9.1 Overview and Data Needed

This chapter addresses the design and construction of rock embankments, bridge approach embankments, earth embankments, and light weight fills. Static loading as well as seismic loading conditions are covered, though for a more detailed assessment of seismic loading on embankment performance, see Chapter 6. The primary geotechnical issues that impact embankment performance are overall stability, internal stability, settlement, materials, and construction.

For the purposes of this chapter embankments include the following:

- Rock embankments, defined as fills in which the material in all or any part of an embankment contains 25 percent or more, by volume, gravel or stone 4 inches or more in diameter.
- Bridge approach embankments, defined as fill beneath a bridge structure and extending 100 feet beyond a structure's end at subgrade elevation for the full embankment width, plus an access ramp on a 10H:1V slope from subgrade down to the original ground elevation. The bridge approach embankment also includes any embankment that replaces unsuitable foundation soil beneath the bridge approach embankment.
- Earth embankments are fills that are not classified as rock or bridge approach embankments, but that are constructed out of soil.
- Lightweight fills contain lightweight fill or recycled materials as a significant portion of the embankment volume, and the embankment construction is usually by special provision. Lightweight fills are most often used as a portion of the bridge approach embankment to mitigate settlement or in landslide repairs to reestablish roadways.

9.1.1 Site Reconnaissance

General requirements for site reconnaissance are given in Chapter 2.

The key geotechnical issues for design and construction of embankments include stability and settlement of the underlying soils, the impact of the stability and settlement on the construction staging and time requirements, and the impact to adjacent and nearby structures, such as buildings, bridge foundations, and utilities. Therefore, the geotechnical designer should perform a detailed site reconnaissance of the proposed construction. This should include a detailed site review outside the proposed embankment footprint in addition to within the embankment footprint. This reconnaissance should extend at least two to three times the width of the embankment on either side of the embankment and to the top or bottom of slopes adjacent to the embankment. Furthermore, areas below proposed embankments should be fully explored if any existing landslide activity is suspected.

9.1.2 Field Exploration and Laboratory Testing Requirements

General requirements for the development of the field exploration and laboratory testing plans are provided in Chapter 2. The expected project requirements and subsurface conditions should be analyzed to determine the type and quantity of information to be obtained during the geotechnical investigation. During this phase it is necessary to:

- Identify performance criteria (e.g. allowable settlement, time available for construction, seismic design requirements, etc.).
- Identify potential geologic hazards, areas of concern (e.g. soft soils), and potential variability of local geology.
- Identify engineering analyses to be performed (e.g. limit equilibrium slope stability analyses, liquefaction susceptibility, lateral spreading/slope stability deformations, settlement evaluations).
- 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 for them.

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

- Total stress and effective stress strength parameters;
- Unit weight;
- Compression indexes (primary, secondary and recompression); and
- Coefficient of consolidation).

Table 9-1 provides a summary of site characterization needs and field and laboratory testing considerations for embankment design.

Geotechnical Issues	Engineering Evaluations	Required Information for Analyses	Field Testing	Laboratory Testing
Embankments and Embankment Foundations	• settlement (magnitude & rate) • bearing capacity • slope stability • lateral pressure • internal stability • borrow source evaluation (available quantity and quality of borrow soil) • required reinforcement • liquefaction • delineation of soft soil deposits • potential for subsidence (karst, mining, etc.) • constructability	• subsurface profile (soil, ground water, rock) • compressibility parameters • shear strength parameters • unit weights • time-rate consolidation parameters • horizontal earth pressure coefficients • interface friction parameters • pullout resistance • geologic mapping including orientation and characteristics of rock discontinuities • shrink/swell/ degradation of soil and rock fill	• nuclear density • plate load test • test fill • CPT (w/ pore pressure measurement) • SPT • PMT • dilatometer • vane shear • rock coring (RQD) • geophysical testing • piezometers • settlement plates • slope inclinometers	• 1-D Oedometer • triaxial tests • unconfined compression • direct shear tests • grain size distribution • Atterberg Limits • specific gravity • organic content • moisture-density relationship • hydraulic conductivity • geosynthetic/soil testing • shrink/swell • slake durability • unit weight • relative density

Summary of Information Needs and Testing Considerations for Embankments (Adapted From Sabatini, Et Al., 2002)
Table 9-1

9.1.3 Soil Sampling and Stratigraphy

The size, complexity and extent of the soil sampling program will depend primarily on the type, height and size of embankment project as well as the expected soil conditions.

Generally, embankments 10 feet or less in height, constructed over average to good soil conditions (e.g., non-liquefiable, medium dense to very dense sand, silt or gravel, with no signs of previous instability) will require only a basic level of site investigation. A geologic site reconnaissance (see Chapter 2), combined with widely spaced test pits, hand holes, or a few shallow borings to verify field observations and the anticipated site geology may be sufficient, especially if the geology of the area is well known, or if there is some prior experience in the area.

For larger embankments, or for any embankment to be placed over soft or potentially unstable ground, geotechnical explorations should in general be spaced no more than 500 feet apart for uniform conditions. In non-uniform soil conditions, spacing should be decreased to 100 to 300 foot intervals with at least one boring in each major landform or geologic unit. A key to the establishment of exploration frequency for embankments is the potential for the subsurface conditions to impact the construction of the embankment, the construction contract in general, and the long-term performance of the finished project. The exploration program should be developed and conducted in a manner that these potential problems, in terms of cost, time, and performance, are reduced to an acceptable level. The boring frequency described above may need to be adjusted by the geotechnical designer to address the risk of such problems for the specific project.

All embankments over 10 feet in height, embankments over soft soils, or those that could impact adjacent structures (bridge abutments, buildings etc.), will generally require geotechnical borings for the design. The more critical areas for stability of a large embankment are between the top and bottom of the slopes. This is where base stability is of most concern and where a majority of the borings should be located, particularly if the near-surface soils are expected to consist of soft fine-grained deposits. At critical locations, (e.g., maximum embankment heights, maximum depths of soft strata), a minimum of two exploration points in the transverse direction to define the existing subsurface conditions for stability analyses should be obtained. More exploration points to define the subsurface stratigraphy, including the conditions within and below existing fill, may be necessary for very large fills or very erratic soil conditions.

Embankment widening projects will require careful consideration of exploration locations. Borings near the toe of the existing fill are needed to evaluate the present condition of the underlying soils, particularly if the soils are fine-grained. In addition, borings through the existing fill into the underlying consolidated soft soil, or, if overexcavation of the soft soil had been done during the initial fill construction, borings to define the extent of removal, should be obtained to define conditions below the existing fill.

In some cases, the stability and/or durability of the existing embankment fill may be questionable because the fill materials are suspect or because slope instability in the form of raveling, downslope lobes, or slope failures have been observed during the site reconnaissance phase. Some embankments constructed of material that is susceptible to accelerated weathering may require additional borings through the core of the embankment to sample and test the present condition of the existing fill.

Borings are also needed near existing or planned structures that could be impacted by new fill placement. Soil sampling and testing will be useful for evaluating the potential settlement of the existing structure foundations as the new fill is placed.

The depth of borings, test pits, and hand holes will generally be determined by the expected soil conditions and the depth of influence of the new embankment. Explorations will need to be sufficiently deep to penetrate through surficial problem soils such as loose sand, soft silt and clay and organic materials, and at least 10 feet into competent soil conditions. In general, all geotechnical borings should be drilled to a minimum depth of twice the planned embankment height.

Understanding of the underlying soil conditions requires appropriate sampling intervals and methods. As for most engineering problems, testing for strength and compression in fine-grained soils requires the need for undisturbed samples. The SPT is useful in cohesionless soil where it is not practical or possible to obtain undisturbed samples for laboratory engineering tests. SPT sampling is recommended at wet sand sites where liquefaction is a key engineering concern.

On larger projects, cone penetration test (CPT) probes can be used to supplement conventional borings. Besides being significantly less expensive, CPT probes allow the nearly continuous evaluation of soil properties with depth. They can detect thin layers of soil, such as a sand lens in clay that would greatly reduce consolidation time that may be missed in a conventional boring. In addition, CPT probes can measure pore pressure dissipation responses, which can be used to evaluate relative soil permeability and consolidation rates. Because there are no samples obtained, CPT probes shall be used in conjunction with a standard boring program. Smaller projects that require only a few borings generally do not warrant an integrated CPT/boring field program.

9.1.4 Groundwater

At least one piezometer should be installed in borings drilled in each major fill zone where stability analysis will be required and groundwater is anticipated. Water levels measured during drilling are often not adequate for performing stability analysis. This is particularly true where drilling is in fine-grained soils that can take many days or more for the water level to equalize after drilling (see Chapter 2). Even in more permeable coarse grained soils, the drilling mud used to drill the boring can obscure detection of the groundwater level. Notwithstanding, water levels should be recorded during drilling in all borings or test pits. Information regarding the time and date of the reading and any fluctuations that might be seen during drilling should be included on the field logs.

For embankment widening projects, piezometers are generally more useful in borings located at or near the toe of an existing embankment, rather than in the fill itself. Exceptions are when the existing fill is along a hillside or if seepage is present on the face of the embankment slope.

The groundwater levels should be monitored periodically to provide useful information regarding variation in levels over time. This can be important when evaluating base stability, consolidation settlement or liquefaction. As a minimum, the monitoring should be accomplished several times during the wet season (October through April) to assess the likely highest groundwater levels that could affect engineering analyses. If practical, a series of year-round readings taken at 1 to 2 month intervals should be accomplished in all piezometers.

The location of the groundwater table is particularly important during stability and settlement analyses. High groundwater tables result in lower effective stress in the soil affecting both the shear strength characteristics or the soil and its consolidation behavior under loading. The geotechnical designer should identify the location of the groundwater table and determine the range in seasonal fluctuation.

If there is a potential for a significant groundwater gradient beneath an embankment or surface water levels are significantly higher on one side of the embankment than the other, the effect of reduced soil strength caused by water seepage should be evaluated. In this case, more than one piezometer should be installed to estimate the gradient. Also, seepage effects must be considered when an embankment is placed on or near the top of a slope that has known or potential seepage through it. A flow net or a computer model (such as MODFLOW) may be used to estimate seepage velocity and forces in the soil. This information may then be used into the stability analysis to model pore pressures.

9.2 Design Considerations

9.2.1 Typical Embankment Materials and Compaction

General instructions for embankment construction are discussed in the WSDOT *Construction Manual* Section 2.3.3, and specific construction specifications for embankment construction are provided in WSDOT Construction Specifications Section 2-03. The geotechnical designer should determine during the exploration program if any of the material from planned earthwork will be suitable for embankment construction (see Chapter 10). Consideration should be given to whether the material is moisture sensitive and difficult to compact during wet weather.

9.2.1.1 Rock Embankments

The WSDOT *Standard Specifications* define rock embankment as "all or any part of an embankment in which the material contains 25 percent or more by volume of gravel or stone 4 inches or greater in diameter." Compaction tests cannot be applied to coarse material with any degree of accuracy; therefore, a given amount of compactive effort is specified for rock embankments, as described in *Standard Specifications* Section 2-03.3(14)A.

Special consideration should be given to the type of material that will be used in rock embankments. In some areas of the state, moderately weathered or very soft rock may be encountered in cuts and used as embankment fill. On projects located in southwestern Washington, degradable fine grained sandstone and siltstone are often encountered in the cuts. The use of this material in embankments can result in significant long term settlement and stability problems as the rock degrades, unless properly compacted with heavy tamping foot rollers (Machan, et al., 1989).

The rock should be tested by the Washington Degradation Test (WSDOT Test Method 113) and the slake durability test (see Chapter 5) if there is suspicion that the geologic nature of the rock source proposed indicates that poor durability rock is likely to be encountered. When the rock is found to be non-durable, it should be physically broken down and compacted as earth embankment provided the material meets or exceeds common borrow requirements. Special compaction requirements may be needed for these materials. In general, tamping foot rollers work best for breaking down the rock fragments. The minimum size roller should be about 30 tons. Specifications should include the maximum size of the rock fragments and maximum lift thickness. These requirements will depend on the hardness of the rock, and a test section should be incorporated into the contract to verify that the Contractor's methods will achieve compaction and successfully break down the material. In general, both the particle size and lift thickness should be limited to 12 inches.

9.2.1.2 Earth Embankments and Bridge Approach Embankments

Three types of materials are commonly used in WSDOT earth embankments, including common, select, and gravel borrow. Bridge approach embankments should be constructed from select or gravel borrow, although common borrow may be used in the drier parts of the State, provided it is not placed below a structure foundation or immediately behind an abutment wall. Common borrow is not intended for use as foundation material beneath structures or as wall backfill due to its tendency to be more compressible and due to its poor drainage characteristics.

Requirements for common, select and gravel borrow are in Section 9-03.14 of the WSDOT *Standard Specifications*. The suggested range of soil properties for each material type to be used in design is discussed in Chapter 5. The common and select borrow specifications are intended for use where it is not necessary to strictly control the strength properties of the embankment material and where all weather construction is not required.

Procedures for constructing earth embankments are described in Section 2-03.3(14) B of the *Standard Specifications*. Compaction is specified in accordance with Method A, Method B, or Method C. Method A consists of routing hauling equipment over the embankment and is not normally used on WSDOT projects. Method B limits the thickness of the lifts to 8 inches and requires that 90 percent of maximum dry density be achieved in all but the upper 2 feet of the embankment. In the upper two feet of the embankment the lift thickness is limited to 4 inches and the required compaction is 95 percent of maximum dry density. Method B is used on all embankments on WSDOT projects unless another method is specified.

Method C differs from Method B in that the entire embankment must be compacted to 95 percent of maximum dry density. Method C is required when the structural quality of the embankment is essential. Method C is required in bridge approach embankments as defined in Section 1-01.3 of the WSDOT *Standard Specifications*. Method C shall also be required on any foundation material beneath structures. Because foundation stresses are transferred outward as well as downward into the bearing soils, the limits of the foundation material should extend horizontally outward from each edge of the footing a distance equal to the thickness of the fill below the foundation.

The maximum density and optimum moisture content for soil placed in earth embankments are determined by testing in accordance with WSDOT Test Method No. 606 (Method of Test for Compaction Control of Granular Materials) or AASHTO T 99 Method A (standard Proctor) as prescribed in Section 2-03.3(14)D of the *Standard Specifications*. Test method 606 is used if 30 percent or more of the material consists of gravel size particles (retained on the No. 4 sieve).

9.2.1.3 Fill Placement Below Water

If material will be placed below the water table, material that does not require compaction such as Quarry Spalls, Foundation Material Class B, Shoulder Ballast, or light loose rip rap should specified. Once above the water table, other borrow materials should be used. Quarry spalls and rip rap should be choked with Shoulder Ballast or Foundation Material Class A or B before placement of borrow. Alternately, construction geosynthetic for soil stabilization may be used to prevent migration of the finer borrow into the voids spaces of the coarser underlying material.

9.2.2 Embankments for Detention/Retention Facilities

Embankments for detention/retention facilities impounding over 10 acre-feet of water come under the jurisdiction of the Dam Safety Office (DSO) of the Washington State Department of Ecology and shall be designed as a small dam in accordance with DSO requirements.

Embankments for detention/retention facilities impounding 10 acre feet of water or less are not regulated by the DSO, but they should be designed using the DSO guidelines as the basis for design. Unlined drainage facilities shall be analyzed for seepage and piping through the embankment fill and underlying soils. Stability of the fill and underlying soils subjected to seepage forces shall have a minimum safety factor of 1.5. Furthermore, the minimum safety factor for piping stability analysis shall be 1.5.

9.2.3 Stability Assessment

In general, embankments 10 feet or less in height with 2H:1V or flatter side slopes, may be designed based on past precedence and engineering judgment provided there are no known problem soil conditions such as liquefiable sands, organic soils, soft/loose soils, or potentially unstable soils such as Seattle clay, estuarine deposits, or peat. Embankments over 10 feet in height or any embankment on soft soils, in unstable areas/soils, or those comprised of light weight fill require more in depth stability analyses, as do any embankments with side slope inclinations steeper than 2H:1V. Moreover, any fill placed near or against a bridge abutment or foundation, or that can impact a nearby buried or above-ground structure, will likewise require stability analyses by the geotechnical designer. Slope stability analysis shall be conducted in accordance with Chapter 7.

Prior to the start of the stability analysis, the geotechnical designer should determine key issues that need to be addressed. These include:

- Is the site underlain by soft silt, clay or peat? If so, a staged stability analysis may be required.
- Are site constraints such that slopes steeper than 2H:1V are required? If so, a detailed slope stability assessment is needed to evaluate the various alternatives.
- Is the embankment temporary or permanent? Factors of safety for temporary embankments may be lower than for permanent ones, depending on the site conditions and the potential for variability.
- Will the new embankment impact nearby structures or bridge abutments? If so, more elaborate sampling, testing and analysis are required.
- Are there potentially liquefiable soils at the site? If soil, seismic analysis to evaluate this may be warranted (see Chapter 6) and ground improvement may be needed.

Several methodologies for analyzing the stability of slopes are detailed or identified by reference in Chapter 7 and are directly applicable to earth embankments.

9.2.3.1 Safety Factors

Embankments that support structure foundations or walls or that could potentially impact such structures should be designed in accordance with the AASHTO LRFD Bridge Design Specifications and Chapters 8 and 15. If an LRFD design is required, a resistance factor is used in lieu of a safety factor. However, since slope stability in the AASHTO LRFD Bridge Design Specifications is assessed only for the service and extreme event (seismic) limit states, the load factors are equal to 1.0, and the resistance factor is simply the inverse of the factor of safety (i.e., 1/FS) that is calculated in most slope stability analysis procedures and computer programs. The resistance factors and safety factors for overall stability under static conditions are as follows:

- All embankments not supporting or potentially impacting structures shall have a minimum safety factor of 1.25.
- Embankments supporting or potentially impacting non-critical structures shall have a resistance factor for overall stability of 0.75 (i.e., a safety factor of 1.3).
- All Bridge Approach Embankments and embankments supporting critical structures shall have a resistance factor of 0.65 (i.e., a safety factor of 1.5). Critical structures are those for which failure would result in a life threatening safety hazard for the public, or for which failure and subsequent replacement or repair would be an intolerable financial burden to the citizens of Washington State.

Under seismic conditions, only those portions of the new embankment that could impact an adjacent structure such as bridge abutments and foundations or nearby buildings require seismic analyses and an adequate overall stability resistance factor (i.e., a maximum resistance factor of 0.9 or a minimum factor of safety of 1.1). See Chapter 6 for specific requirements regarding seismic design of embankments.

9.2.3.2 Strength Parameters

Strength parameters are required for any stability analysis. Strength parameters appropriate for the different types of stability analyses shall be determined based on Chapter 5 and by reference to FHWA Geotechnical Engineering Circular No. 5 (Sabatini, et al., 2002).

If the critical stability is under drained conditions, such as in sand or gravel, then effective stress analysis using a peak friction angle is appropriate and should be used for stability assessment. In the case of over-consolidated fine grained soils, a friction angle based on residual strength may be appropriate. This is especially true for soils that exhibit strain softening or are particularly sensitive to shear strain such as Seattle Clay.

If the critical stability is under undrained conditions, such as in most clays and silts, a total stress analysis using the undrained cohesion value with no friction is appropriate and should be used for stability assessment.

For staged construction, both short (undrained) and long term (drained) stability need to be assessed. At the start of a stage the input strength parameter is the undrained cohesion. The total shear strength of the fine-grained soil increases with time as the excessive pore water dissipates, and friction starts to contribute to the strength. A more detailed discussion regarding strength gain is presented in Section 9.3.1.

9.2.4 Embankment Settlement Assessment

New embankments, as is true of almost any new construction, will add load to the underlying soils and cause those soils to settle. As discussed in Section 8.11.3.2, the total settlement has up to three potential components: 1) immediate settlement, 2) consolidation settlement, and 3) secondary compression.

Settlement shall be assessed for all embankments. Even if the embankment has an adequate overall stability factor of safety, the performance of a highway embankment can be adversely affected by excessive differential settlement at the road surface.

Settlement analyses for embankments over soft soils require the compression index parameters for input. These parameters are typically obtained from standard one-dimensional oedometer tests of the fine-grained soils (see Chapter 5 for additional information). For granular soils, these parameters can be estimated empirically (see Section 8.11.3.2). Oedometer tests should be completed to at least twice the preconsolidation pressure with at least three, and preferably four, points on the virgin consolidation curve (i.e., at stresses higher than the preconsolidation pressure). The coefficient of consolidation value for the virgin curve can be ten times higher than that for the test results below the preconsolidation pressure.

9.2.4.1 Settlement Impacts

Because primary consolidation and secondary compression can continue to occur long after the embankment is constructed (post construction settlement), they represent the major settlement concerns for embankment design and construction. Post construction settlement can damage structures and utilities located within the embankment, especially if those facilities are also supported by adjacent soils or foundations that do not settle appreciably, leading to differential settlements. Embankment settlement near an abutment could create an unwanted dip in the roadway surface, or downdrag and lateral squeeze forces on the foundations. See Chapter 8 for more information regarding the use of bridge approach slabs to minimize the effects of differential settlement at the abutment, and the methodology to estimate downdrag loads on foundations.

If the primary consolidation is allowed to occur prior to placing utilities or building structures that would otherwise be impacted by the settlement, the impact is essentially mitigated. However, it can take weeks to years for primary settlement to be essentially complete, and significant secondary compression of organic soils can continue for decades. Many construction projects cannot absorb the scheduling impacts associated with waiting for primary consolidation and/or secondary compression to occur. Therefore, estimating the time rate of settlement is often as important as estimating the magnitude of settlement.

To establish the target settlement criteria, the tolerance of structures or utilities to differential settlement that will be impacted by the embankment settlement shall be determined. Lateral movement (i.e., lateral squeeze) caused by the embankment settlement and its effect on adjacent structures, including light, overhead sign, and signal foundations, shall also be considered. If structures or utilities are not impacted by the embankment settlement, settlement criteria are likely governed by the long-term maintenance needs of the roadway surfacing. In that case, the target settlement criteria shall be established with consideration of the effect differential settlement will have on the pavement life and surface smoothness.

9.2.4.2 Settlement Analysis

9.2.4.2.1 Primary Consolidation

The key parameters for evaluating the amount of settlement below an embankment include knowledge of:

- The subsurface profile including soil types, layering, groundwater level and unit weights;
- The compression indexes for primary, rebound and secondary compression from laboratory test data, correlations from index properties, and results from settlement monitoring programs completed for the site or nearby sites with similar soil conditions. See Chapters 5 and 8 for additional information regarding selection of design parameters for settlement analysis.
- The geometry of the proposed fill embankment, including the unit weight of fill materials and any long term surcharge loads.

The detailed methodology to estimate primary consolidation settlement is provided in Section 8.11.3.2, except that the stress distribution below the embankment should be calculated as described in Section 9.2.4.3. The soil profile is typically divided into layers for analysis, with each layer reflecting changes in soils properties. In addition, thick layers with similar properties are often subdivided for refinement of the analysis since the settlement calculations are based on the stress conditions at the midpoint of the layer (i.e. it is typically preferable to evaluate a near-surface, 20-foot thick layer as two 10-foot thick layers as opposed to one 20-foot thick layer). The total settlement is the sum of the settlement from each of the compressible layers.

If the pre-consolidation pressure of any of the soil layers being evaluated is greater than its current initial effective vertical stress, the settlement will follow its rebound compression curve rather than its virgin compression curve (represented by C_c). In this case $C_{r\varepsilon,}$ the recompression index, should be used instead of $C_{c\varepsilon}$ in Equation 8-8 up to the point where the initial effective stress plus the change in effective stress imposed by the embankment surpasses the pre-consolidation pressure. Pre-consolidation pressures in excess of the current vertical effective stress occur in soils that have been overconsolidated, such as from glacial loading, preloading, or desiccation.

9.2.4.2.2 Secondary Compression

For organic soils and highly plastic soils determined to have an appreciable secondary settlement component, the secondary compression should be determined as described in Section 8.11.3.2.2, Equation 8-13. Note the secondary compression is in general independent of the stress state and theoretically is a function only of the secondary compression index and time.

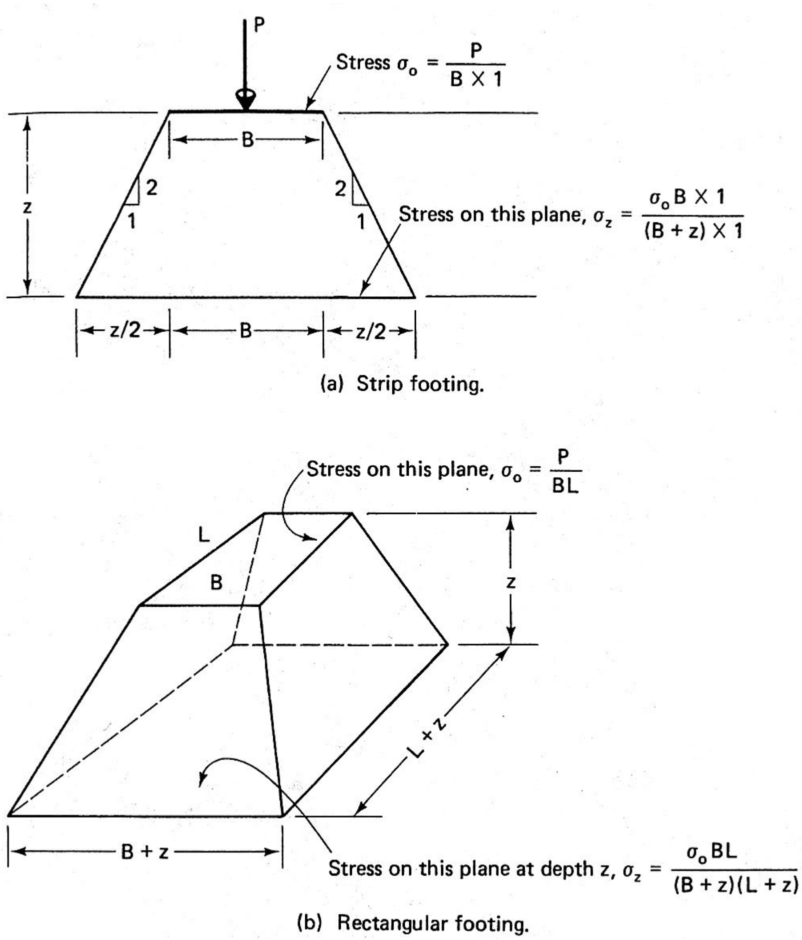
Similar to estimating the total primary consolidation, the contribution from the individual layers are summed to estimate the total secondary compression. Since secondary compression is not a function of the stress state in the soil but rather how the soil breaks down over time, techniques such as surcharging to pre-induce the secondary settlement are sometimes only partially effective at mitigating the secondary compression. Often the owner must accept the risks and maintenance costs associated with secondary compression if a cost/benefit analysis indicates that mitigation techniques such as using lightweight fills or overexcavating and replacing the highly compressible soils are too costly.

9.2.4.3 Stress Distribution

One of the primary input parameters for settlement analysis is the increase in vertical stress at the midpoint of the layer being evaluated caused by the embankment or other imposed loads. It is generally quite conservative to assume the increase in vertical stress at depth is equal to the bearing pressure exerted by the embankment at the ground surface. In addition to the bearing pressure exerted at the ground surface, other factors influencing the stress distribution at depth include the geometry (length and width) of the embankment, inclination of the embankment side slopes, depth below the ground surface to the layer being evaluated, and horizontal distance from the center of the load to the point in question. Several methods are available to estimate the stress distribution.

9.2.4.3.1 Simple 2V:1H Method

Perhaps the simplest approach to estimate stress distribution at depth is using the 2V:1H (vertical to horizontal) method. This empirical approach is based on the assumption that the area the load acts over increases geometrically with depth as depicted in Figure 9-1. Since the same vertical load is spread over a much larger area at depth, the unit stress decreases.



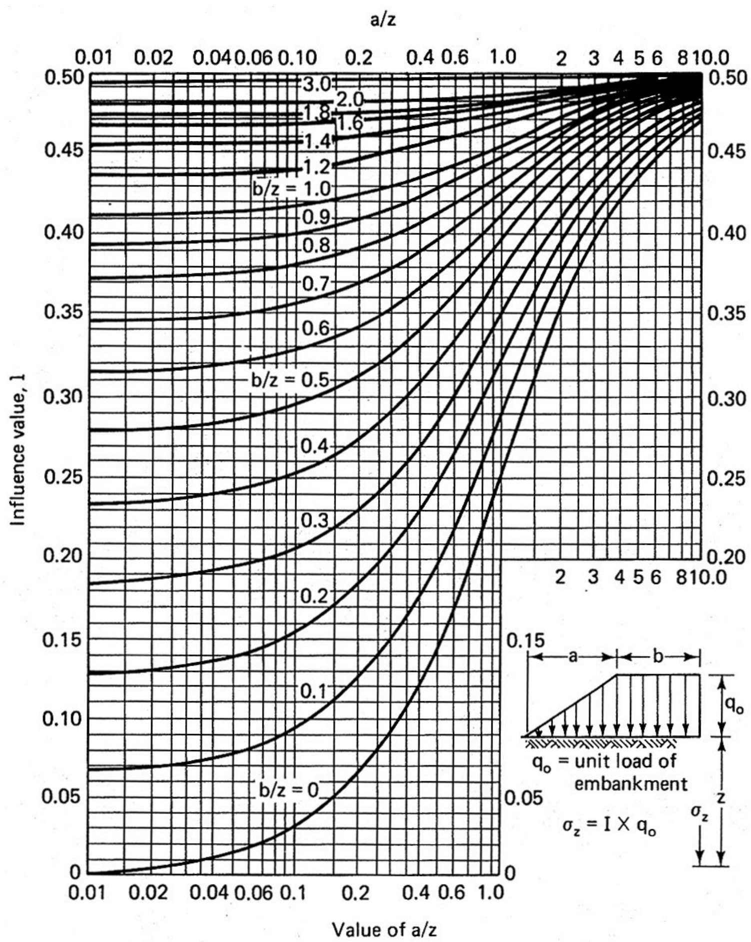
2V:1H Method to Estimate Vertical Stress Increase as a Function of Depth Below Ground (After Holtz and Kovacs, 1981)
Figure 9-1

9.2.4.3.2 Theory of Elasticity

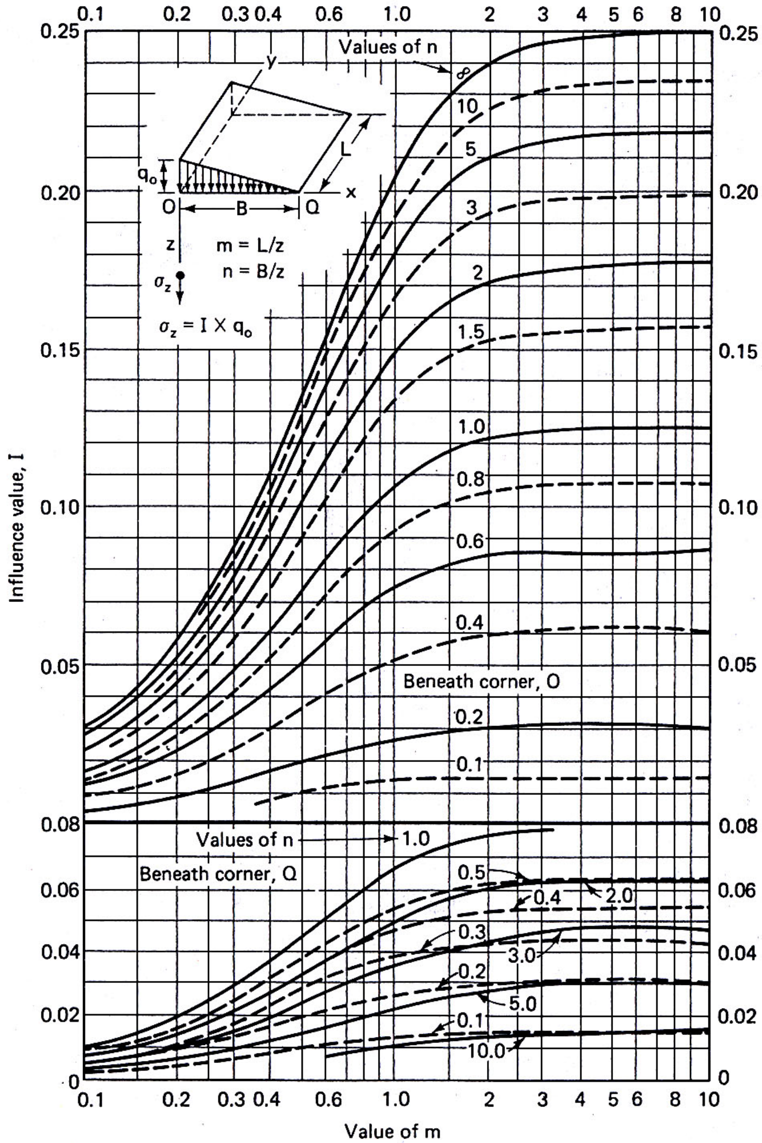
Boussinesq (1885) developed equations for evaluating the stress state in a homogenous, isotropic, linearly elastic half-space for a point load acting perpendicular to the surface. Elasticity based methods should be used to estimate the vertical stress increase in subsurface strata due to an embankment loading, or embankment load in combination with other surcharge loads. While most soils are not elastic materials, the theory of elasticity is the most widely used methodology to estimate the stress distribution in a soil deposit from a surface load. Most simplifying charts and the subroutines in programs such as SAF-1 and EMBANK are based on the theory of elasticity. Some are based on Boussinesq theory and some on Westergaard's equations (Westegaard, 1938), which also include Poisson's ratio (relates the ratio of strain applied in one direction to strain induced in an orthogonal direction).

9.2.4.3.3 Empirical Charts

The equations for the theory of elasticity have been incorporated into design charts and tables for typical loading scenarios, such as below a foundation or an embankment. Almost all foundation engineering textbooks include these charts. For convenience, charts to evaluate embankment loading are included as Figures 9-2 and 9-3.



Influence Factors for Vertical Stress Under a Very Long Embankment
(After NAVFAC, 1971 as Reported in Holtz and Kovacs, 1981)
Figure 9-2



Influence Values for Vertical Stress Under the Corners of a Triangular Load of Limited Length (after NAVFAC, 1971 as reported in Holtz and Kovacs, 1981)
Figure 9-3

9.2.4.3.4 Rate of Settlement

The time rate of primary consolidation is typically estimated using equations based on Terzaghi's one-dimensional consolidation theory. The time rate of primary consolidation shall be estimated as described in Section 8.11.3.2.

The value of C_v should be determined from the laboratory test results, piezocone testing, and/or back-calculation from settlement monitoring data obtained at the site or from a nearby site with similar geologic and soil conditions.

The length of the drainage path is perhaps the most critical parameter because the time to achieve a certain percentage of consolidation is a function of the square of the drainage path length. This is where incorporating CPTs into the exploration program can be beneficial, as they provide a nearly continuous evaluation of the soil profile, including thin sand layers that can easily be missed in a typical boring exploration program. The thin sand lenses can significantly reduce the drainage path length.

It is important to note some of the assumptions used by Terzaghi's theory to understand some of its limitations. The theory assumes small strains such that the coefficient of compressibility of the soil and the coefficient of permeability remain essentially constant. The theory also assumes there is no secondary compression. Both of these assumptions are not completely valid for extremely compressible soils such as organic deposits and some clays. Therefore, considerable judgment is required to when using Terzaghi's theory to evaluate the time rate of settlement for these types of soil. In these instances, or when the consolidation process is very long, it may be beneficial to complete a preload test at the site with sufficient monitoring to assess both the magnitude and time rate of settlement for the site.

9.2.4.4 Analytical Tools

The primary consolidation and secondary settlement can be calculated by hand or by using computer programs such as SAF-1 (Prototype Engineering Inc., 1993) or EMBANK (FHWA, 1993). Alternatively, spreadsheet solutions can be easily developed. The advantage of computer programs such as SAF-1 and EMBANK are that multiple runs can be made quickly, and they include subroutines to estimate the increased vertical effective stress caused by the embankment or other loading conditions.

9.3 Stability Mitigation

A variety of techniques are available to mitigate inadequate slope stability for new embankments or embankment widenings. These techniques include staged construction to allow for the underlying soils to gain strength, base reinforcement, ground improvement, use of lightweight fill, and construction of toe berms and shear keys. A summary of these instability mitigation techniques is presented below along with the key design considerations.

9.3.1 Staged Construction

Where soft compressible soils are present below a new embankment location and it is not economical to remove and replace these soils with compacted fill, the embankment can be constructed in stages to allow the strength of the compressible soils to increase under the weight of new fill. Construction of the second and subsequent stages commences when the strength of the compressible soils is sufficient to maintain stability. In order to define the allowable height of fill for each stage and maximum rate of construction, detailed geotechnical analysis is required. This analysis typically requires consolidated undrained (CU), consolidated drained (CD) or consolidated undrained with pore pressure measurements (CU_p), and initial undrained (UU) shear strength parameters for the foundation soils along with the at-rest earth pressure coefficient (K_o), soil unit weights, and the coefficient of consolidation (Cv).

The analysis to define the height of fill placed during each stage and the rate at which the fill is placed is typically completed using a limit equilibrium slope stability program along with time rate of settlement analysis to estimate the percent consolidation required for stability. Alternatively, numerical modeling programs, such as FLAC and PLAXIS, can be used to assess staged construction, subject to the approval of the WSDOT State Geotechnical Engineer. Numerical modeling has some advantages over limit equilibrium approaches in that both the consolidation and stability can be evaluated concurrently. The disadvantages of numerical modeling include the lack of available field verification of modeling results, and most geotechnical engineers are more familiar with limit equilibrium approaches than numerical modeling. The accuracy of the input parameters can be critical to the accuracy of numerical approaches. Steps for using a limit equilibrium approach to evaluate staged construction are presented below.

For staged construction, two general approaches to assessing the criteria used during construction to control the rate of embankment fill placement to allow the necessary strength gain to occur in the soft subsoils are available. The two approaches are total stress analysis and effective stress analysis:

- For the total stress approach, the rate of embankment construction is controlled through development of a schedule of maximum fill lift heights and intermediate fill construction delay periods. During these delay periods the fill lift that was placed is allowed to settle until an adequate amount of consolidation of the soft subsoil can occur. Once the desired amount of consolidation has occurs, placement of the next lift of fill can begin. These maximum fill lift thicknesses and intermediate delay periods are estimated during design. For this approach, field measurements such as the rate of settlement or the rate of pore pressure decrease should be obtained to verify that the design assumptions regarding rate of consolidation are correct. However, if only a small amount of consolidation is required (e.g., 20 to 40% consolidation), it may not be feasible to determine of the desired amount of consolidation has occurred, since the rate of consolidation may still be on the linear portion of the curve at this point. Another approach may be to determine if the magnitude of settlement expected at that stage, considering the degree of consolidation desired, has been achieved. In either case, some judgment will need to be applied when interpreting such data and deciding whether or not to reduce or extend the estimated delay period during fill construction.
- For the effective stress approach, the pore pressure increase beneath the embankment in the soft subsoil is monitored and used to control the rate of embankment construction. During construction, the pore pressure increase is not allowed to exceed a critical amount to insure embankment stability. The critical amount is generally controlled in the contract by use of the pore pressure ratio (ru), which is the ratio of pore pressure to total overburden stress. To accomplish this pore pressure measurement, pore pressure transducers are typically located at key locations beneath the embankment to capture the pore pressure increase caused by consolidation stress. As is true of the total stress approach, some judgment will need to be applied when interpreting such data and deciding whether or not to reduce or extend the estimated delay period during fill construction, as the estimate of the key parameters may vary from the actual values of the key parameters in the field. Also, this approach may not be feasible if the soil contains a high percentage of organic material and trapped gases, causing the pore pressure readings to be too high and not drop off as consolidation occurs.

Since both approaches have limitations and uncertainties, it is generally desirable to analyze the embankment using both approaches, to have available a backup plan to control the rate of fill placement, if the field data proves difficult to interpret. Furthermore, if the effective stress method is used, a total stress analysis should in general always be conducted to obtain an estimate of the time required to build the fill for contract bidding purposes.

Detailed procedures for both approaches are provided in the sections that follow. These procedures have been developed based on information provided in Ladd (1991), Symons (1976), Skempton and Bishop (1955), R. D. Holtz (personal communication, 1993), S. Sharma (personal communication, 1993), and R. Cheney (personal communication, 1993). Examples of the application of these procedures are provided in Appendix 9-A.

9.3.1.1 Design Parameters

First, define the problem in terms of embankment geometry, soil stratigraphy, and water table information.

The geotechnical designer must make some basic assumptions regarding the fill properties. Typically, the designer assumes presumptive values for the embankment fill, since the specific source of the fill material is usually not known at the time of design. However, specialized soils laboratory tests should be performed for the soft underlying soils. From undisturbed samples, the geotechnical designer should obtain Unconsolidated Undrained (UU) triaxial tests and Consolidated Undrained (CU) triaxial tests with pore pressure measurements. These tests should be used to determine the initial undrained shear strength available. The CU test with pore pressure measurements should also be used to determine the shear strength envelope needed for total or effective stress analyses. In addition, the geotechnical designer should obtain consolidation test data to determine compressibility of the soft underlying soils as well as the rate of consolidation for the compressible strata (C_v). C_v will be an important parameter for determining the amount of time required during consolidation to gain the soil shear strength needed.

In general triaxial tests should be performed at the initial confining stress (P_o’) for the sample as determined from the unit weight and the depth that the sample was obtained.

$$P_o' = D\gamma' \qquad (9\text{-}1)$$

Where:
D = Sample Depth in feet
γ’ = Effective Unit Weight (pcf)

The third point in the triaxial test is usually performed at $4P_o$’. During the triaxial testing it is important to monitor pore pressure to determine the pore pressure parameters A and B. Note that A and B are not constant but change with the stress path of the soil. These parameters are defined as follows:

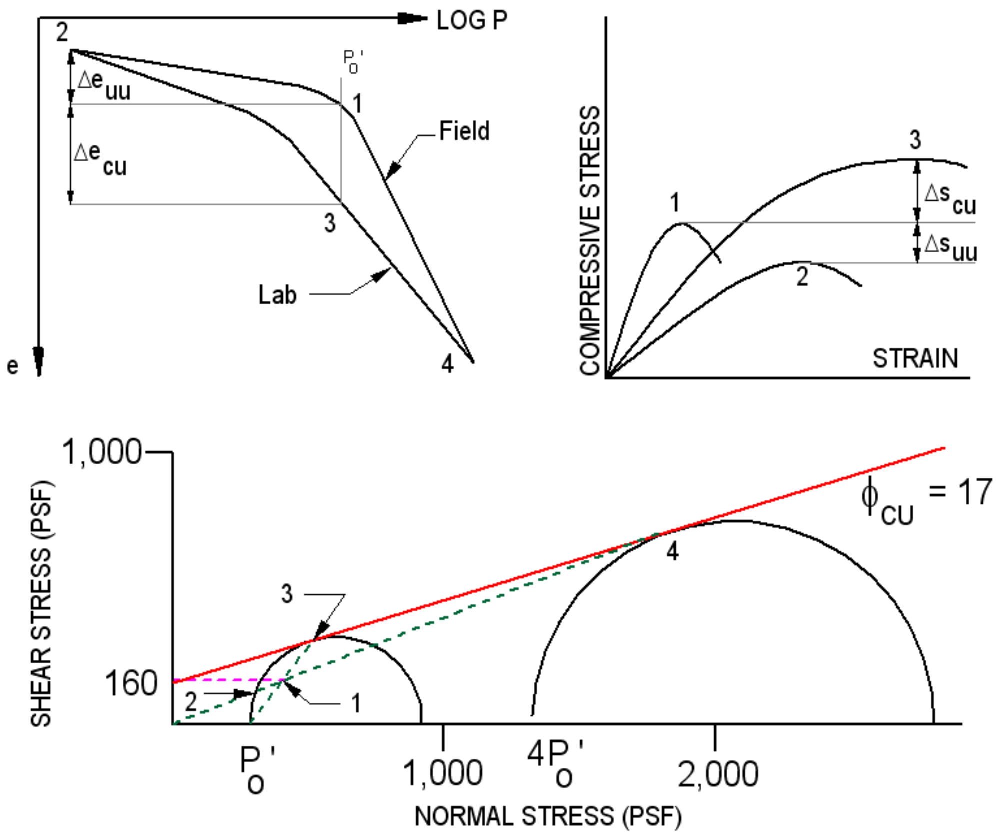
$$A = \Delta U / \Delta\sigma_1 \qquad (9\text{-}2)$$

$$B = \Delta U / \Delta\sigma_3 \qquad (9\text{-}3)$$

9.3.1.2 In-Situ Shear Strength and Determination of Stability Assuming Undrained Loading

The first step in any embankment design over soft cohesive soils is to assess its stability assuming undrained conditions throughout the entire fill construction period. If the stability of the embankment is adequate assuming undrained conditions, there is no need to perform a staged construction design. The UU shear strength data, as well as the initial shear strength from CU tests, can be used for this assessment.

The geotechnical designer should be aware that sample disturbance can result in incorrect values of strength for normally consolidated fine grained soils. Figure 9-4 shows how to correctly obtain the cohesive strength for short term, undrained loading.



Determination of Short Term Cohesive Shear Strength From the CU Envelope
Figure 9-4

When a normally consolidated sample is obtained, the initial effective stress (P_O’) and void ratio correspond to position 1 on the e - Log P curve shown in Figure 9-4. As the stress changes, the sample will undergo some rebound effects and will move towards point 2 on the e – Log P curve. Generally, when a UU test is performed, the sample state corresponds to position 2 on the e – Log P curve. Samples that are reconsolidated to the initial effective stress (P_O’) during CU testing undergo a void ratio change and will generally be at point 3 on the e – Log P curve after reconsolidation to the initial effective stress. It is generally assumed that consolidating the sample to 4 times the initial effective stress prior to testing will result in the sample closely approximating the field “virgin” curve behavior.

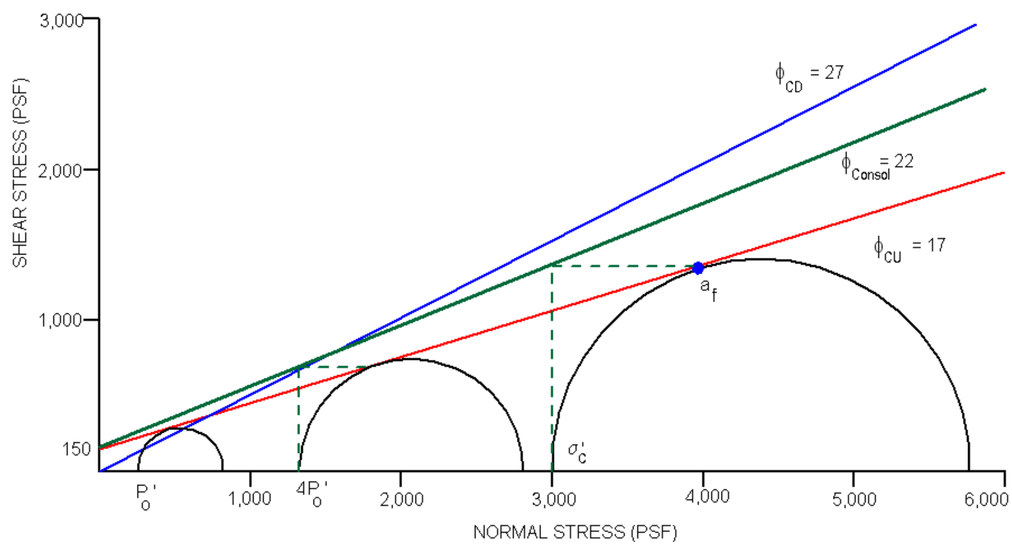
To determine the correct shear strength for analysis, perform a CU triaxial test at the initial effective stress (P_O') and as close as practical to $4P_O$'. On the Mohr diagram draw a line from the ordinate to point 4, and draw a second line from P_O' to point 3. Where the two lines intersect, draw a line to the shear stress axis to estimate the correct shear strength for analysis. In Figure 9-4, the cohesion intercept for the CU strength envelope (solid line) is 150 psf. The corrected strength based on the construction procedure in Figure 9-4 would be 160 psf. While the difference is slight in this example, it may be significant for other projects.

Once the correct shear strength data has been obtained, the embankment stability can be assessed. If the embankment stability is inadequate, proceed to performing a total stress or effective stress analysis, or both.

9.3.1.3 Total Stress Analysis

The CU triaxial test is ideally suited to staged fill construction analysis when considering undrained strengths. A CU test is simply a series of UU tests performed at different confining pressures. In the staged construction technique, each embankment stage is placed under undrained conditions (i.e., "U" conditions). Then the soil beneath the embankment stage is allowed to consolidate under drained conditions, which allows the pore pressure to dissipate and the soil strength to increase (i.e., "C" conditions).

In most cases, the CU envelope cannot be used directly to determine the strength increase due to the consolidation stress placed on the weak subsoil. The stress increase from the embankment fill is a consolidation stress, not necessarily the normal stress on potential failure planes in the soft soil, and with staged construction excess pore pressures due to overburden increases are allowed to partially dissipate. Figure 9-5 illustrates how to determine the correct strength due to consolidation and partial pore pressure dissipation.



Consolidated Strength Construction From Triaxial Data
Figure 9-5

To correct φ_{cu} for the effects of consolidation use the following (see Ladd, 1991):

$$af/\sigma'c = \tan \varphi_{consol} \tag{9-4}$$

$$\tan \varphi_{consol} = \sin \varphi_{cu}/(1-\sin \varphi_{cu}) \tag{9-5}$$

Determine the strength gain (ΔCuu) by multiplying the consolidation stress increase ($\Delta\sigma_v$) by the tangent of φ_{consol}. The consolidation stress increase is the increased effective stress in the soft subsoil caused by the embankment fill.

$$\Delta Cuu = \Delta\sigma_v \tan\varphi_{consol} \tag{9-6}$$

This is an undrained strength and it is based on 100% consolidation. When constructing embankments over soft ground using staged construction practices, it is often not practical to allow each stage to consolidate to 100%. Therefore, the strengths used in the stability analysis need to be adjusted for the consolidation stress applied and the degree of consolidation achieved in the soft soils within the delay period between fill stages. The strength at any degree of consolidation can be estimated using:

$$Cuu_{\,u\%} = Cuu_i + U(Cuu) = Cuu_i + U\Delta\sigma_v \tan\varphi_{consol} \tag{9-7}$$

The consolidation is dependent upon the time (t), drainage path length (H), coefficient of consolidation (C_v), and the Time Factor (T). From Holtz and Kovacs (1981), the following approximation equations are presented for consolidation theory:

$$T = tC_v/H^2 \tag{9-8}$$

Where:

$$T = 0.25\pi U^2;\ \text{for } U < 60\% \tag{9-9}$$

and,

$$T = 1.781 - 0.933\log(100 - U\%);\ \text{for } U > 60\% \tag{9-10}$$

The geotechnical designer should use these equations along with specific construction delay periods (t) to determine how much consolidation occurs by inputting a time (t), calculating a Time Factor (T), and then using the Time Factor (T) to estimate the degree of consolidation (U).

Once all of the design parameters are available, the first step in a total stress staged fill construction analysis is to use the initial undrained shear strength of the soft subsoil to determine the maximum height to which the fill can be built without causing the slope stability safety factor to drop below the critical value. See Section 9.3.1.1.2 for determination of the undrained shear strength needed for this initial analysis.

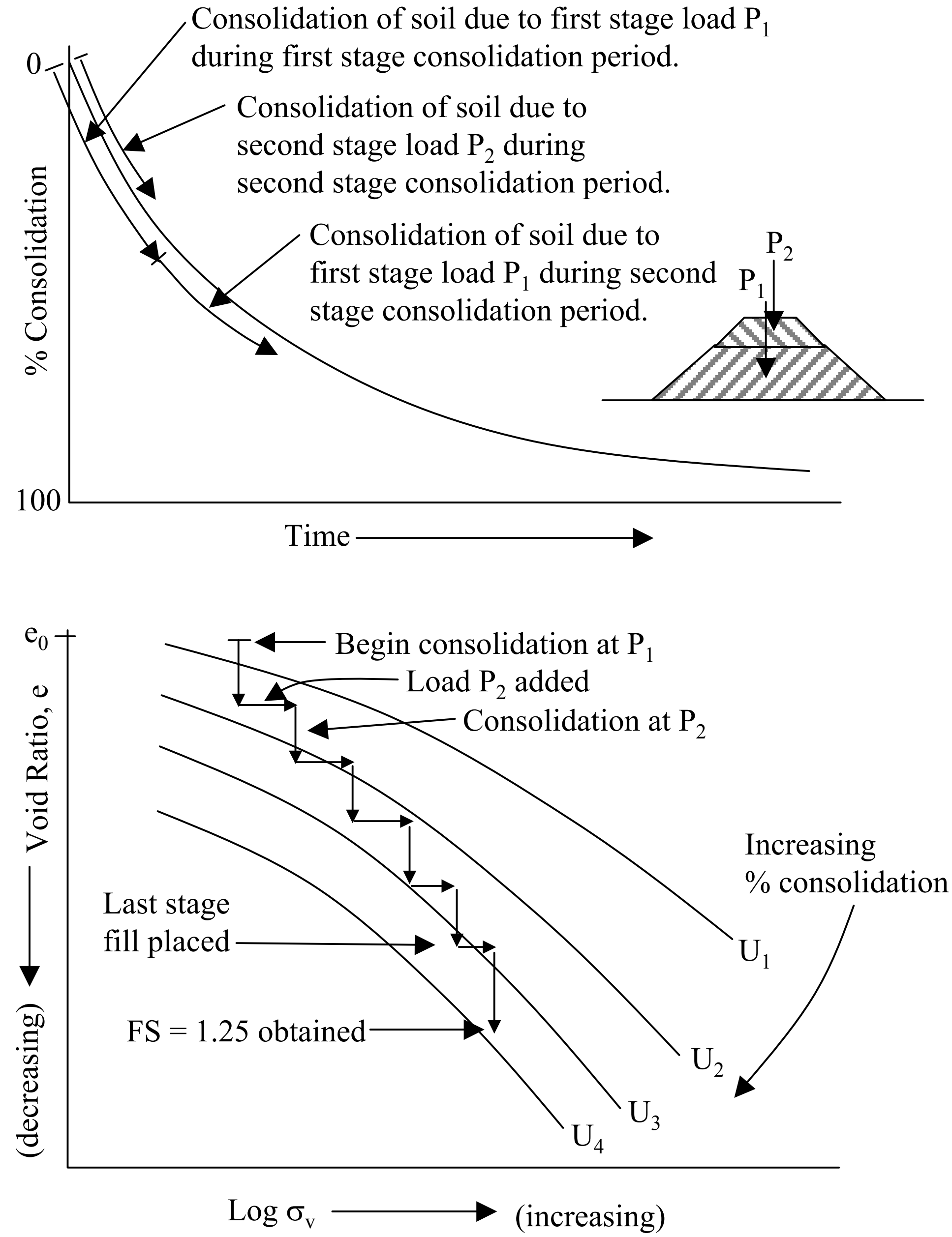
In no case shall the interim factor of safety at any stage in the fill construction be allowed to drop below 1.15. A higher critical value should be used (i.e., 1.2 or 1.25) if uncertainty in the parameters is high, or if the soft subsoil is highly organic. At the end of the final stage, determine the time required to achieve enough consolidation to obtain the minimum long-term safety factor (or resistance factor if structures are involved) required, as specified in Section 9.2.3.1. This final consolidation time will determine at what point the embankment is considered to have adequate long-term stability such that final paving (assuming that long-term settlement has been reduced during that time period to an acceptable level) and other final construction activities can be completed. In general, this final consolidation/strength gain period should be on the order of a few months or less.

Once the maximum safe initial fill stage height is determined, calculate the stress increase resulting from the placement of the first embankment stage using the Boussinesq equation (e.g., see Figures 9-2 and 9-3). Note that because the stress increase due to the embankment load decreases with depth, the strength gain also decreases with depth. To properly account for this, the soft subsoil should be broken up into layers for analysis just as is done for calculating settlement. Furthermore, the stress increase decreases as one moves toward the toe of the embankment. Therefore, the soft subsoil may need to be broken up into vertical sections as well.

Determine the strength gain in each layer/section of soft subsoil by multiplying the consolidation stress increase by the tangent of φ_{consol} (see Equation 9-6), where φ_{consol} is determined as shown in Figure 9-5 and Equation 9-5. This will be an undrained strength. Multiply this UU strength by the percent consolidation that has occurred beneath the embankment up to the point in time selected for the fill stage analysis using Equations 9-7, 9-8, and 9-9 or 9-10. This will be the strength increase that has occurred up to that point in time. Add to this the UU soil strength existing before placement of the first embankment stage to obtain the total UU strength existing after the selected consolidation period is complete. Then perform a slope stability analysis to determine how much additional fill can be added with consideration to the new consolidated shear strength to obtain the minimum acceptable interim factor of safety.

Once the second embankment stage is placed, calculation of the percent consolidation and the strength gain gets more complicated, as the stress increase due to the new fill placed is just starting the consolidation process, while the soft subsoil has already had time to react to the stress increase due to the previous fill stage. Furthermore, the soft subsoil will still be consolidating under the weight of the earlier fill stage. This is illustrated in Figure 9-6. For simplicity, a weighted average of the percent consolidation that has occurred for each stage up to the point in time in question should be used to determine the average percent consolidation of the subsoil due to the total weight of the fill.

Continue this calculation process until the fill is full height. It is generally best to choose as small a fill height and delay period increment as practical, as the conservatism in the consolidation time estimate increases as the fill height and delay time increment increases. Typical fill height increments range from 2 to 4 feet, and delay period increments range from 10 to 30 days.



Concepts Regarding the Percent Consolidation Resulting From Placement of Multiple Fill Stages
Figure 9-6

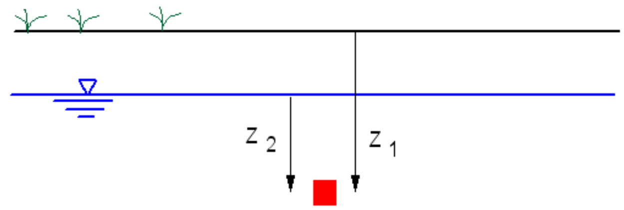
9.3.1.4 Effective Stress Analysis

In this approach, the drained soil strength, or φ_{CD}, is used to characterize the strength of the subsoil. Of course, the use of this soil strength will likely indicate that the embankment is stable, whereas the UU strength data would indicate that the embankment is unstable (in this example). It is the buildup of pore pressure during embankment placement that causes the embankment to become unstable. The amount of pore pressure buildup is dependent on how rapidly the embankment load is placed. Given enough time, the pore pressure buildup will dissipate and the soil will regain its effective strength, depending on the permeability and compressibility of the soil.

The key to this approach is to determine the amount of pore pressure buildup that can be tolerated before the embankment safety factor drops to a critical level, using φ_{CD} for the soil strength and conducting a slope stability analysis (see Chapter 7). A slope stability computer program such as XSTABL can be used to determine the critical pore pressure increase directly. This pore pressure increase can then be used to determine the pore pressure ratio, ru, which is often used to compare with in-situ pore pressure measurements. The pore pressure ratio, r_u, is defined as shown in Figure 9-7.

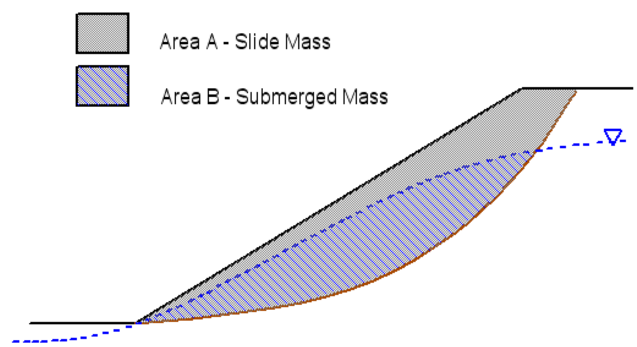
Horizontal Surface

$$r_u = \frac{\gamma_w \cdot Z_2}{\gamma_{total} \cdot Z_1}$$



Sloping Ground or Phreatic Surface

$$r_u = \frac{\text{Area B}}{\text{Area A}} \cdot \frac{\gamma'}{\gamma_{total}}$$



Pore Pressure Ratio Concepts
Figure 9-7

For XSTABL, the critical pore pressure increase is input into the program as a "pore pressure constant" for each defined soil unit in the soil property input menu of the program. This pore pressure is in addition to the pore pressure created by the static water table. Therefore, a water table should also be included in the analysis. Other slope stability programs have similar pore pressure features that can be utilized.

To determine the pore pressure increase in the soft subsoil to be input into the stability analysis, calculate the vertical stress increase created by the embankment at the original ground surface, for the embankment height at the construction stage being considered. Based on this, determine the vertical stress increase, $\Delta\sigma v$, using the Boussinesq stress distribution (e.g., Figures 9-2 and 9-3), at various depths below the ground surface, and distances horizontally from the embankment centerline, in each soil unit which pore pressure buildup is expected (i.e., the soft silt or clay strata which are causing the stability problem). Based on this, and using Ko, the at rest earth pressure coefficient, to estimate the horizontal stress caused by the vertical stress increase, determine the pore pressure increase, Δu_p, based on the calculated vertical stress increase, $\Delta\sigma v$, as follows:

$$\Delta u_p = B(\Delta\sigma_{oct} + a\Delta\tau_{oct})(1\text{-}U) \qquad (9\text{-}11)$$

The octahedral consolidation stress increase at the point in question, $\Delta\sigma_{oct}$. is determined as follows:

$$\Delta\sigma_{oct}. = (\Delta\sigma_1 + \Delta\sigma_2 + \Delta\sigma_3)/3 = (\Delta\sigma_v + K_0\Delta\sigma_v + K_0\Delta\sigma_v)/3 = (1 + 2K_0)\Delta\sigma v/3 \qquad (9\text{-}12)$$

Where:

B = pore pressure parameter which is dependent on the degree of saturation and the compressibility of the soil skeleton. B is approximately equal to 1.0 for saturated normally consolidated silts and clays.

$\Delta\sigma_{oct}$ = the change in octahedral consolidation stress at the point in the soil stratum in question due to the embankment loading,

a = Henkel pore pressure parameter that reflects the pore pressure increase during shearing. “a” is typically small and can be neglected unless right at failure. If necessary, “a” can be determined from triaxial tests and plotted as a function of strain or deviator stress to check if neglecting “a” is an acceptable assumption.

$\Delta\tau_{oct}$ = the change in octahedral shear stress at the point in the soil stratum in question due to the embankment loading,

U = the percent consolidation, expressed as a decimal, under the embankment load in question.

$$\Delta\tau_{oct} = [(\Delta\sigma_1 - \Delta\sigma_2)^2 + (\Delta\sigma_2 - \Delta\sigma_3)^2 + (\Delta\sigma_3 - \Delta\sigma_1)^2]^{1/2} \qquad (9\text{-}13)$$

In terms of vertical stress, before failure, this equation simplifies to:

$$\Delta\tau_{oct} = 1.414\Delta\sigma_v(1 - K_0) \qquad (9\text{-}14)$$

In this analysis, since only consolidation stresses are assumed to govern pore pressure increase, and strength gain as pore pressure dissipates (i.e., the calculation method is set up to not allow failure to occur), it can be assumed that “a” is equal to zero. Therefore, Equation 9-11 simplifies to:

$$\Delta u_p = B[(1 + 2K_0)/3]\Delta\sigma_v(1\text{-}U) \qquad (9\text{-}15)$$

where, $K_0 = 1 - \sin\varphi_{CD}$ for normally consolidated silts and clays.

Estimate the slope stability factor of safety, determining Δu_p at various percent consolidations (i.e., iterate) to determine the maximum value of Δu_p that does not cause the slope stability interim safety factor to drop below the critical value (see Section 9.3.1.3).

Now determine ru as follows:

$$r_u = \Delta u_p/\Delta\sigma_v. = B[(1 + 2K_0)/3]\Delta\sigma_v(1\text{-}U)/\Delta\sigma_v = B[(1 + 2K_0)/3](1\text{-}U) \qquad (9\text{-}16)$$

The pore pressures measured by the piezometers in the field during embankment construction are the result of vertical consolidation stresses only (Boussinesq distribution). Most experts on this subject feel that pore pressure increase due to undrained shearing along the potential failure surface does not occur until failure is actually in progress and may be highly localized at the failure surface. Because of this, it is highly unlikely that one will be able to measure pore pressure increase due to shearing along the failure surface using piezometers installed below the embankment unless one is lucky enough to have installed a piezometer in the right location and happens to be taking a reading as the embankment is failing. Therefore, the pore pressure increase measured by the piezometers will be strictly due to consolidation stresses.

Note that ru will vary depending on the embankment height analyzed. ru will be lowest at the maximum embankment height, and will be highest at the initial stages of fill construction. Therefore, r_u should be determined at several embankment heights.

9.3.2 Base reinforcement

Base reinforcement may be used to increase the factor of safety against slope failure. Base reinforcement typically consists of placing a geotextile or geogrid at the base of an embankment prior to constructing the embankment. Base reinforcement is particularly effective where soft/weak soils are present below a planned embankment location. The base reinforcement can be designed for either temporary or permanent applications. Most base reinforcement applications are temporary, in that the reinforcement is needed only until the underlying soil's shear strength has increased sufficiently as a result of consolidation under the weight of the embankment (see Section 9.3.1). Therefore, the base reinforcement does not need to meet the same design requirements as permanent base reinforcement regarding creep and durability. For example, if it is anticipated that the soil will gain adequate strength to meet stability requirements without the base reinforcement within 6 months, then the creep reduction factor determined per WSDOT Standard Practice T925 could be based on, say, a minimum 1 year life, assuming deformation design requirements are met. Other than this, only installation damage would need to be addressed, unless unusual chemical conditions exist that could cause rapid strength degradation. Alternatively, the values of T_{al} provided in the WSDOT Qualified Products List (QPL) could be used, but will be conservative for this application. However, if it is anticipated that the soil will never gain enough strength to cause the embankment to have the desired level of stability without the base reinforcement, the long-term design strengths provided in the QPL or as otherwise determined using T925 for a minimum 75 year life shall be used.

The design of base reinforcement is similar to the design of a reinforced slope in that limit equilibrium slope stability methods are used to determine the strength required to obtain the desired safety factor (see Chapter 15). The detailed design procedures provided by Holtz, et al. (1995) should be used for embankments utilizing base reinforcement.

Base reinforcement materials should be placed in continuous longitudinal strips in the direction of main reinforcement. Joints between pieces of geotextile or geogrid in the strength direction (perpendicular to the slope) should be avoided. All seams in the geotextiles should be sewn and not lapped. Likewise, geogrids should be linked with mechanical fasteners or pins and not simply overlapped. Where base reinforcement is used, the use of gravel borrow, instead of common or select borrow, may also be appropriate in order to increase the embankment shear strength.

9.3.3 Ground Improvement

Ground improvement can be used to mitigate inadequate slope stability for both new and existing embankments, as well as reduce settlement. The primary ground improvement techniques to mitigate slope stability fall into two general categories, namely densification and altering the soil composition. Chapter 11 Ground Improvement, should be reviewed for a more detailed discussion and key references regarding the advantages and disadvantages of these techniques, applicability for the prevailing subsurface conditions, construction considerations, and costs. In addition to the two general categories of ground improvement identified above, wick drains (discussed in Chapter 11 and Section 9.4.1) may be used in combination with staged embankment construction to accelerate strength gain and improve stability, in addition to accelerating long-term settlement. The wick drains in effect drastically reduce the drainage path length, thereby accelerating the rate of strength gain. Other ground improvement techniques such as stone columns can function to accelerate strength gain in the same way as wick drains, though the stone columns also reduce the stress applied to the soil, thereby reducing the total strength gain obtained. See Chapter 11 for additional guidance and references to use if this technique is to be implemented.

9.3.4 Lightweight Fills

Lightweight embankment fill is another means of improving embankment stability. Lightweight fills are generally used for two conditions: the reduction of the driving forces contributing to instability, and reduction of potential settlement resulting from consolidation of compressible foundation soils. Situations where lightweight fill may be appropriate include conditions where the construction schedule does not allow the use of staged construction, where existing utilities or adjacent structures are present that cannot tolerate the magnitude of settlement induced by placement of typical fill, and at locations where post-construction settlements may be excessive under conventional fills.

Lightweight fill can consist of a variety of materials including polystyrene blocks (geofoam), light weight aggregates (rhyolite, expanded shale, blast furnace slag, fly ash), wood fiber, shredded rubber tires, and other materials. Lightweight fills are infrequently used due to either high costs or other disadvantages with using these materials.

9.3.4.1 Geofoam

Geofoam is approximately 1/100th the weight of conventional soil fill and, as a result, is particularly effective at reducing driving forces or settlement potential. Typical geofoam embankments consist of the foundation soils, the geofoam fill, and a pavement system designed to transfer loads to the geofoam. Geofoam dissolves readily in gasoline and other organic fluids/vapors and therefore must be encapsulated where such fluids can potentially reach the geofoam. Other design considerations for geofoam include creep, flammability, buoyancy, moisture absorption, photo-degradation, and differential icing of pavement constructed over geofoam. Furthermore, geofoam should not be used where the water table could rise and cause buoyancy problems, as geofaom will float. Design guidelines for geofoam embankments are provided in the NCHRP document titled *Geofoam Applications in the Design and Construction of Highway Embankments* (Stark et al., 2004). Additional information on the design properties and testing requirements are provided in Chapter 5.

9.3.4.2 Lightweight Aggregates

Mineral aggregates, such as expanded shales, rhyolite, fly ash, or blast furnace slags, can also be used as lightweight fill materials. Expanded shales and rhyolite materials consist of inert mineral aggregates that have similar shear strengths to many conventional fill materials, but weigh roughly half as much. The primary disadvantage with expanded shales and rhyolite is that these materials are expensive. Fly ash can also be used for lightweight fill; however, fly ash is difficult to place and properly control the moisture condition. Blast furnace slag is another waste material sometimes used for lightweight fill. Due to the weight of blast furnace slag, it is not as effective as other lightweight fill materials. Also, slag materials have been documented to swell when hydrated, potentially damaging improvements founded above the slag. The chemical composition of fly ash and blast furnace slag should be investigated to confirm that high levels of contaminants are not present. Due to the potential durability and chemical issues associated with some light weight aggregates, approval from the State Geotechnical Engineer is required before such materials may be considered for use in embankments.

9.3.4.3 Wood Fiber

Wood fibers may also be used for lightweight fill. For permanent applications, only fresh wood fiber should be used to prolong the life of the fill. Wood fiber fills typically have unit weights between about 35 to 55 pcf. To mitigate the effects of leachate, the amount of water entering the wood should be minimized. Wood fiber fill will experience creep settlement for several years and some pavement distress should be expected during that period. See Chapter 5 for more information regarding wood fiber fills.

9.3.4.4 Scrap (Rubber) Tires

In 1996, a moratorium on the use of scrap tires as embankment fill was put into effect due to several instances where the tire fills caught fire due to some type of exothermic reaction which has yet to be fully defined. A report to the Washington State legislature was published in 2003 to address whether or not, and under what circumstances, the moratorium on the use of scrap tires as fill should be lifted (Baker, et al., 2003). Based on that report, scrap tire fills up to 10 feet in thickness may be considered, provided that they are designed and specified as described in Baker, et al. (2003).

9.3.4.5 Light Weight Cellular Concrete

Large quantities of air can be entrained into concrete to produce a very light weight porous concrete that can be poured in place of soil to reduce the driving force to improve stability or reduce settlement. Typical unit weights feasible range from 20 to 80 pcf, and relative to soil, its shear strength is fairly high. However, if significant differential settlement is still anticipated in spite of the use of the light weight concrete, due to its relatively brittle nature, the concrete could crack, losing much of its shear strength. This should be considered if using light weight cellular concrete. Its cost can be quite high, being among the most expensive of the light weight fill materials mentioned herein.

9.3.4.6 Toe Berms and Shear keys

Toe berms and shear keys are each methods to improve the stability of an embankment by increasing the resistance along potential failure surfaces. Toe berms are typically constructed of granular materials that can be placed quickly, do not require much compaction, but have relatively high shear strength. As implied by the name, toe berms are constructed near the toe of the embankment slopes where stability is a concern. The toe berms are often inclined flatter than the fill embankment side slopes, but the berm itself should be checked for stability. The use of berms may increase the magnitude of settlements as a consequence of the increased size of the loaded area.

Toe berms increase the shearing resistance by:

- Adding weight, and thus increasing the shear resistance of granular soils below the toe area of the embankment;
- Adding high strength materials for additional resistance along potential failure surfaces that pass through the toe berm; and
- Creating a longer failure surface, thus more shear resistance, as the failure surface now must pass below the toe berm if it does not pass through the berm.

Shear keys function in a manner similar to toe berms, except instead of being adjacent to and incorporating the toe of the fill embankment, the shear key is placed under the fill embankment—frequently below the toe of the embankment. Shear keys are best suited to conditions where they key can be embedded into a stronger underlying formation. Shear keys typically range from 5 to 15 feet in width and extend 4 to 10 feet below the ground surface. They are typically backfilled with quarry spalls or similar materials that are relatively easy to place below the groundwater level, require minimal compaction, but still have high internal shear strength. Like toe berms, shear keys improve the stability of the embankment by forcing the potential failure surface through the strong shear key material or along a much longer path below the shear key.

9.4 Settlement Mitigation

9.4.1 Acceleration Using Wick Drains

Wick drains, or prefabricated drains, are in essence vertical drainage paths that can be installed into compressible soils to decrease the overall time required for completion of primary consolidation. Wick drains typically consist of a long plastic core surrounded by a geotextile. The geotextile functions as a separator and a filter to keep holes in the plastic core from being plugged by the adjacent soil, and the plastic core provides a means for the excess pore water pressures to dissipate. A drainage blanket is typically placed across the ground surface prior to installing the wick drains and provides a drainage path beneath the embankment for water flowing from the wick drains.

The drains are typically band-shaped (rectangular) measuring a few inches wide in plan dimension. They are attached to a mandrel and are usually driven/pushed into place using either static or vibratory force. After the wick drains are installed, the fill embankment and possibly surcharge fill are placed above the drainage blanket. A key consideration for the use of wick drains is the site conditions. If obstructions or a very dense or stiff soil layer is located above the compressible layer, pre-drilling may be necessary. The use of wick drains to depths over about 60 feet require specialized equipment.

The primary function of a wick drain is to reduce the drainage path in a thick compressible soil deposit. As noted in Section 9.3.3, a significant factor controlling the time rate of settlement is the length of the drainage path. Since the time required for a given percentage consolidation completion is related to the square of the drainage path, cutting the drainage path in half would reduce the consolidation time to one-fourth the initial time, all other parameters held constant. However, the process of installing the wick drains creates a smear zone that can impede the drainage. The key design issue is maximizing the efficiency of the spacing of the drains, and one of the primary construction issues is minimizing the smear zone around the drains. A full description of wick drains, design considerations, example designs, guideline specifications, and installation considerations are provided by reference in Chapter 11. Section 2-03.3(14) H of the WSDOT *Standard Specifications* addresses installation of prefabricated vertical drains.

9.4.2 Acceleration Using Surcharges

Surcharge loads are additional loads placed on the fill embankment above and beyond the design height. The primary purpose of a surcharge is to speed up the consolidation process. The surcharges speed up the consolidation process because the percentage of consolidation required under a surcharge will be less than the complete consolidation under the design load. As noted previously, it is customary to assume consolidation is essentially complete at the theoretical 90% completion stage, where $T = 0.848$. In comparison, $T = 0.197$ for 50% consolidation. Therefore it takes less than one-fourth the time to achieve an average of 50% consolidation in a soil layer than it does to achieve 90%. In this example, the objective would be to place a surcharge sufficiently large such that 50% of the total settlement estimated from the fill embankment and the surcharge is equal to or greater than 100 percent of the settlement estimated under the fill embankment alone at its design height. Based on previous experience, the surcharge fill needs to be at least one-third the design height of the embankment to provide any significant time savings.

In addition to decreasing the time to reach the target settlement, surcharges can also be used to reduce the impact of secondary settlement. Similar to the example presented above, the intent is to use the surcharge to pre-induce the settlement estimated to occur from primary consolidation and secondary compression due to the embankment load. For example, if the estimated primary consolidation under an embankment is 18 inches and secondary compression is estimated at an additional 6 inches over the next 25 years, then the surcharge would be designed to achieve 24 inches of settlement or greater under primary consolidation only. The principles of the design of surcharges to mitigate long-term settlement provided by Cotton, et al. (1987) should be followed.

Using a surcharge typically will not completely eliminate secondary compression, but it has been successfully used to reduce the magnitude of secondary settlement. However, for highly organic soils or peats where secondary compression is expected to be high, the success of a surcharge to reduce secondary compression may be quite limited. Other more positive means may be needed to address the secondary compression in this case, such as removal.

Two significant design and construction considerations for using surcharges include embankment stability and re-use of the additional fill materials. New fill embankments over soft soils can result in stability problems as discussed in Section 9.3. Adding additional surcharge fill would only exacerbate the stability problem. Furthermore, after the settlement objectives have been met, the surcharge will need to be removed. If the surcharge material cannot be moved to another part of the project site for use as site fill or as another surcharge, it often not economical to bring the extra surcharge fill to the site only to haul it away again. Also, when fill soils must be handled multiple times (such as with a "rolling" surcharge), it is advantageous to use gravel borrow to reduce workability issues during wet weather conditions.

9.4.3 Lightweight Fills

Lightweight fills can also be used to mitigate settlement issues as indicated in Section 9.3.4. Lightweight fills reduce the new loads imposed on the underlying compressible soils, thereby reducing the magnitude of the settlement. See Chapter 5 and Section 9.3.4 for additional information on light weight fill.

9.4.4 Over-excavation

Over-excavation simply refers to excavating the soft compressible soils from below the embankment footprint and replacing these materials with higher quality, less compressible soil. Because of the high costs associated with excavating and disposing of unsuitable soils as well as the difficulties associated with excavating below the water table, over-excavation and replacement typically only makes economic sense under certain conditions. Some of these conditions include, but are not limited to:

- The area requiring overexcavation is limited;
- The unsuitable soils are near the ground surface and do not extend very deep (typically, even in the most favorable of construction conditions, over-excavation depths greater than about 10 feet are in general not economical);
- Temporary shoring and dewatering are not required to support or facilitate the excavation;
- The unsuitable soils can be wasted on site; and
- Suitable excess fill materials are readily available to replace the over-excavated unsuitable soils.

9.5 Construction Considerations and PS&E Development

Consideration should be given to the time of year that construction will likely occur. If unsuitable soil was encountered during the field investigation, the depth and station limits for removal should be provided on the plans. Chapter 530 of the WSDOT *Design Manual* provides guidance for the use of geotextile for separation or soil stabilization (see also Chapter 16). Note that for extremely soft and wet soil, a site specific design should be performed for the geotextile.

Hillside Terracing is specified in Section 2-03.3(14) of the WSDOT *Standard Specifications*. Where embankments are built on existing hillsides or existing embankment slopes, the existing surface soil may form a plane of weakness unless the slope is terraced or stepped. Terracing breaks up the plane, increasing the strength of the entire system. Generally slopes that are 3H:1V or steeper should be terraced to improve stability. However there may be specific cases where terracing may be waived during design, such as when the existing slope is steeper than 1H:1V and benching would destabilize the existing slope.

The compaction requirements in the WSDOT *Standard Specifications* apply to the entire embankment, including near the sloping face of the embankment. For embankment slopes of 2H:1V or steeper, depending on the embankment soil properties, getting good compaction out to the embankment face can be difficult to achieve, and possibly even unsafe for those operating the compaction equipment. The consequences of poor compaction at the sloping face of the embankment include increased risk of erosion and even surficial slope instability. This issue becomes especially problematic as the embankment slope steepness approaches 1.5H:1V. Surficial stability of embankments (See Chapter 7) should be evaluated during design for embankment slopes of 2H:1V or steeper. The embankment design shall include the use of techniques that will improve embankment face slope stability for embankment slopes steeper than 1.7H:1V, and should consider the use of such techniques for slopes of 2H:1V or steeper.

Approaches typically used to address compaction and surficial stability of embankment slopes include:

- Over-build the embankment laterally at the slope face approximately 2 feet, compact the soil, and then trim off the outer 2 feet of the embankment to produce a well compacted slope face.
- Use strips of geosynthetic placed in horizontal layers at the slope face as a compaction and surficial stability aid (see Elias, et al., 2001). The strips should generally be a minimum of 4 feet wide (horizontally into the slope) and spaced vertically at 1 to 1.5 feet (1.5 feet maximum). The specific reinforcement width and vertical spacing will depend on the soil type. The reinforcement strength required depends on the coarseness and angularity of the backfill material and the susceptibility of the geosynthetic to damage during placement and compaction. See Elias, et al. (2001) for specific guidance on the design of geosynthetic layers as a compaction and surficial stability aid.

Even if good compaction can be obtained using one of these techniques, the potential for erosion and surficial instability should be addressed through appropriate use of slope vegetation techniques such as seeding and mulching, temporary or permanent turf reinforcement mats, or for deeper surficial stability problems, bioengineering. Note that if geosynthetic layers are placed in the soil as a compaction aid or to improve overall embankment slope stability, the typical practice of cultivating the upper 1 feet of the soil per the WSDOT *Standard Specifications*, Section 8-02, should not be conducted. Instead, the landscape architect who is developing the slope vegetation plan should consult with the HQ Geotechnical Division to insure that the slope vegetation plan (either per the WSDOT *Standard Specifications* or any special provisions developed) does not conflict with the slope geosynthetic reinforcement and the need for good compaction out to the slope face.

9.5.1 Settlement and Pore Pressure Monitoring

If settlement is expected to continue after embankment construction, some type of monitoring program should be provided. Settlement should be monitored, if post construction settlement will affect pavement performance or a settlement sensitive structure will be constructed on the embankment. The type of monitoring will depend on the magnitude and time frame of the settlement. For many monitoring programs, use of survey hubs or monuments and routine surveying methods are adequate. These methods are commonly used if paving should be delayed until embankment settlement is nearly complete. The geotechnical report should include the time period that the settlement should be monitored and the frequency of observations.

Settlement estimates provided in the contract should be conservative. Therefore, if another construction operation must be delayed until the settlement of the embankment is nearly complete, the time estimate should be the longest length of time that is likely to be necessary; then the contractor will not be delayed longer than anticipated.

As discussed in Section 9.3.1, embankments constructed over soft ground may require the use of staged construction to ensure the stability of the embankment. Geotechnical instrumentation is a vital part of construction to monitor field performance and provide information relevant to decisions regarding the rate of construction. The principal parameters monitored during embankment construction are pore water pressure and displacement, both vertical and lateral.

As discussed previously, in relatively impermeable, soft, saturated soil, the applied load from embankment construction increases the pore water pressure. With time, the excess pore water pressure will dissipate and the shear strength will increase. It is important to measure the pore water pressure to determine when it is safe to proceed with additional embankment construction. In such cases it is also useful to measure vertical deformation to assist in the interpretation of the data to assess the rate at which embankment construction should proceed.

9.5.2 Instrumentation

The following discussion of monitoring equipment typically used for embankment construction monitoring provides an overview of the typical equipment available. A more comprehensive discussion of monitoring techniques is available in *Geotechnical Instrumentation for Monitoring Field Performance* (Dunnicliff, 1993) and *Geotechnical Instrumentation Reference Manual*, NHI Course No. 13241 FHWA-HI-98-034 (Dunnicliff, 1998). Additional information on WSDOT policies regarding instrumentation installation and standards is provided in Chapter 3.

9.5.2.1 Piezometers

Three types of piezometers are commonly used to monitor embankment construction: open standpipe, pneumatic and vibrating wire. Each type of piezometer has advantages and disadvantages. The sections below describe the various piezometer types.

Open Standpipe Piezometers – These piezometers are installed in a drilled borehole. A porous zone or screen is installed in the soil layer of interest. For embankment settlement purposes it is necessary to completely seal the porous zone against the inflow of water from shallower zones. Open standpipe piezometers are relatively simple to install and the water level readings are easy to obtain. However, standpipes may interfere with or be damaged by construction activities and the response time for changes in water pore pressure in low permeability soils is slow. This type of piezometer is generally not very useful for monitoring the pore pressure increase and subsequent decrease due to consolidation in staged construction applications.

Pneumatic Piezometers – Pneumatic piezometers are usually installed in drilled boreholes in a manner similar to standpipe piezometers, but they can be sealed so that increases in pore water pressure result in a smaller volume change and a more rapid response in instrument measurement. Pneumatic piezometers do not need open standpipes. However, crimping or rupture of the tubes due to settlement of the embankment can cause failure.

Vibrating Wire Piezometers – Vibrating wire piezometers are usually installed in drilled boreholes; although, models are available for pushing into place in soft soils. The cables can be routed long distances and they are easily connected to automatic data acquisition systems.

9.5.2.2 Instrumentation for Settlement

9.5.2.2.1 Settlement Plates

Settlement plates are used to monitor settlement at the interface between native ground and the overlying fill. They consist of a steel plate welded to a steel pipe. An outer pipe consisting of steel or PVC pipe is placed around the pipe and the embankment is built up around it. Both pipes are extended to the completed surface. The outer pipe isolates the inner pipe from contact with the fill. As the embankment and soil surface settle, the top of the inner pipe can be monitored with standard survey equipment. These devices are simple to use, but provide data at only one point and are subject to damage during construction.

9.5.2.2.2 Pneumatic Settlement Cells

These cells are generally placed at the interface between the embankment fill and native ground. A flexible tube is routed to a reservoir, which must be located away from the settlement area. The reservoir must be kept at a constant elevation. The precision of the cells is about 0.75 inches.

9.5.2.2.3 Sondex System

The Sondex System can be used for monitoring settlement at several points at depth. The system is installed in a borehole and consists of a series of stainless steel wire loops on a plastic corrugated pipe. The plastic pipe is placed over an access casing and grouted in the borehole. The locations of the stainless steel loops are determined by electrical induction measurements from a readout unit. The loops can be located to about 0.05 inches and displacements of up to 2 inches can be measured. Accurate measurement of settlement depends on the compatibility of the soil and grout. Therefore, if the grout mix has a higher strength than the surrounding soil, not all the settlement will be measured.

9.5.2.2.4 Horizontal Inclinometer

Horizontal inclinometers are used to measure vertical deflections in a grooved guide casing, placed horizontally beneath the embankment. The probe is pulled through the casing and readings of inclination relative to horizontal are obtained. The inclinometer is a highly accurate system for obtaining settlement data. Because the length of the inclinometer probe is typically about 2 feet, large displacements of the casing caused by settlement may stop passage of the probe.

9.5.3 PS&E Considerations

Specifications for monitoring equipment that will be supplied by the contractor should ensure that the equipment is compatible with the read out equipment that will be used during construction. The specifications should also make clear who will provide the monitoring and analyze the data. If the contractor's survey crew will collect the settlement data, it should be indicated in the special provisions. It is also important to stipulate who will analyze the data and provide the final determination on when settlement is complete or when additional fill can be placed. In general, the geotechnical designer should analyze and interpret the data.

9.5.4 PS&E Checklist

The following issues should be addressed in the PS&E regarding embankments:

- Slope inclination required for stability
- Embankment foundation preparation requirements, overexcavation limits shown on plans
- Plan details for special drainage requirements such as lined ditches, interceptor trenches, drainage blankets, etc.
- Hillside terracing requirements
- Evaluation of on-site materials
- Special embankment material requirements
- Special treatment required for fill placement such as non-durable rock, plastic soil, or lightweight fill
- Magnitude and time for settlement
- Settlement waiting period estimated in the Special Provisions (SP)
- Size and limits of surcharge
- Special monitoring needs
- If instrumentation is required to control the rate of fill placement, do the SP's clearly spell out how this will be done and how the readings will be used to control the contractor's operation
- SP's clearly state that any instrumentation damaged by contractor personnel will be repaired or replaced at no cost to the state
- Settlement issues with adjacent structures, should construction of structures be delayed during embankment settlement period
- Monitoring of adjacent structures

9.5.5 Requirements for Temporary Fills for Construction Facilitation

Temporary fills for haul roads, construction equipment access, and other temporary construction activities shall be designed in accordance with this GDM, in particular this chapter (Chapter 9), except as noted in the following subsections.

9.5.5.1 Design Requirements

The design of the temporary fill/fill slope shall address the stability and settlement of the temporary fill itself as well as the impact of the temporary fill on the global stability and deformation of the of the overall slope on which the fill is located. The stability and movement of any temporary structures and construction equipment (e.g., cranes, compaction equipment, etc.) placed on the temporary fill shall also be addressed in the design. Temporary fills and fill slopes shall be designed such that the risk to health and safety of workers and the public is kept to an acceptable level and that adjacent facilities are not damaged. Seismic design of temporary fills and fill slopes is not required.

If temporary fills are placed on or adjacent to permanent or temporary structures, the impact of the temporary fill on those structures, both with regard to stability and lateral and vertical movements, shall be assessed. The functioning and design life of those structures shall not be compromised by the placement of the temporary fill.

If temporary walls are used to support the temporary fill, the impact of the temporary fill on the wall stability and deformations shall be addressed, and the design of the temporary wall shall meet the requirements in Chapter 15 and the AASHTO LRFD Bridge Design Specifications.

As a minimum, the design of temporary fill slopes for stability by or under the supervision of a registered professional engineer shall include geotechnical calculations to address slope stability (i.e., Chapter 7). If the fill is placed over relatively soft to very soft ground, the deformation of the fill shall also be determined through engineering calculations (i.e., Chapter 9) that are based on a knowledge of the subsurface conditions present and engineering data that can be used to estimate soil and rock properties. Such calculations shall also address the effect of ground water conditions and the loading conditions on or above the slope that could affect its stability and deformation. The design shall be conducted in accordance with the requirements in this GDM and referenced documents. Engineering recommendations based upon field observations alone shall not be considered to be an engineering design, unless the fill is a low height (less than 10 feet high) granular, cohesionless well-compacted fill without concentrated loads from large equipment or structure supports, and the fill is placed over dense to very dense soil or rock, in which the supporting soil or rock is not affected by fissures, slickensides, or other localized weaknesses.

9.5.5.2 Safety Factors and Design Life Considerations

For temporary fill slopes, the safety factors specified in Section 9.2.3.1 are applicable. If the soil properties are well defined and shown to have low variability, a lower factor of safety may be justified through the use of the Monte Carlo simulation feature available in slope stability analysis computer programs. In this case, a probability of failure of 0.01 or smaller shall be targeted (Santamarina, et al., 1992). However, even with this additional analysis, in no case shall a slope stability safety factor less than 1.2 be used for design of the temporary fill slope.

9.5.5.3 Design Loads

The design of temporary fills and fill slopes shall address the actual construction-related loads that could be imposed on the temporary fill. As a minimum, the temporary fill shall be designed for a live load surcharge of 250 psf to address routine construction equipment traffic on the fill. For unusual temporary loadings resulting from large cranes or other large equipment placed on the fill, the loading imposed by the equipment shall be specifically assessed and taken into account in the design of the fill. For the case where large or unusual construction equipment loads will be applied to the fill, the construction equipment loads shall still be considered to be a live load, unless the dynamic and transient forces caused by use of the construction equipment can be separated from the construction equipment weight as a dead load, in which case, only the dynamic or transient loads carried or created by the use of the construction equipment need to be considered live load.

If temporary structures (e.g., false work and formwork support) are placed on or adjacent to the temporary fill, the temporary fill shall be designed to carry the loads resulting from the temporary structures and to meet the stability and deformation requirements of those structures.

9.5.5.4 Design Property Selection

In addition to the requirements in Chapter 9 for determination of design properties, the requirements for design property selection for temporary cuts and shoring in Chapters 5 and 15 shall also be considered applicable to temporary fills and fill slopes.

9.5.5.5 Performance Requirements for Temporary Fills and Fill Slopes

Temporary fills and slopes shall be designed to prevent excessive deformation that could result in damage to adjacent facilities, both during fill construction and during the life of the temporary fill. An estimate of expected displacements or vibrations, threshold limits that would trigger remedial actions, and a list of potential remedial actions if thresholds are exceeded should be developed. Thresholds shall be established to prevent damage to adjacent facilities, as well as degradation of the soil properties due to deformation.

The removal of the temporary fill shall not adversely impact adjacent structures and facilities.

9.5.5.6 Temporary Fill Submittal and Submittal Review Requirements

Temporary Fill submittals shall generally meet the requirements in Section 2-09.3(3)B of the *Standard Specifications* M 41-10.

When performing a geotechnical review of a contractor temporary fill submittal, the following items should be specifically evaluated:

1. Performance objectives for the temporary fill
   a. Is the anticipated length of time the temporary fill will be in place provided?
   b. Are objectives regarding anticipated and allowed deformations of the fill and adjacent and supported structures provided?
   c. Are the performance objectives compatible and consistent with contract and GDM/BDM requirements?
2. Subsurface conditions
   a. Is the soil/rock stratigraphy consistent with the subsurface geotechnical data provided in the contract boring logs?
   b. Did the contractor/fill designer obtain the additional subsurface data needed to meet the geotechnical exploration requirements fills and temporary fill walls as identified in Chapters 9 and 15, respectively?
   c. Was justification for the soil, rock, and other material properties used for the design of the temporary fill provided, and is that justification, and the final values selected, consistent with Chapter 5 and the subsurface field and lab data obtained at the fill site?
   d. Were ground water conditions adequately assessed through field measurements combined with the site stratigraphy to identify zones of ground water, aquitards and aquicludes, artesian conditions, and perched zones of ground water that could impact the stability and deformation of the fill and adjacent facilities that may be impacted by the presence of the temporary fill?
3. Temporary fill loading
   a. Have the anticipated loads on or caused by the temporary fill been correctly identified, considering all applicable limit states?
   b. If construction or public traffic near or on the temporary fill, has a minimum traffic live load surcharge of 250 psf been applied?
   c. If larger construction equipment such as cranes will be placed on the temporary fill, have the loads from that equipment been correctly determined and included in the temporary fill design?

4. Temporary fill design
    a. Have the correct design procedures been used (i.e., the GDM and referenced design specifications and manuals)?
    b. Have all appropriate limit states been considered (e.g., global stability of slopes above and below wall, global stability of wall/slope combination, internal wall stability, external wall stability, bearing capacity, settlement, lateral deformation, piping or heaving due to differential water head, etc.)?
5. Are all safety factors, or load and resistance factors for LRFD temporary wall or structure design, identified, properly justified in a manner that is consistent with the GDM, and meet or exceed the minimum requirements of the GDM?
6. Have the effects of any construction activities adjacent to the temporary fill on the stability/performance of the fill been addressed in the shoring design (e.g., excavation or soil disturbance below the fill, excavation dewatering, vibrations and soil loosening due to soil modification/improvement activities, etc.)?
7. Temporary fill monitoring/testing
    a. Is a monitoring/testing plan provided to verify that the performance of the fill and the structures it supports or impacts is acceptable throughout the design life of the system?
    b. Have appropriate displacement or other performance triggers been provided that are consistent with the performance objectives of the fill and adjacent facilities?
8. Temporary fill removal
    a. Have any portions of the temporary fill (including temporary fill walls used to support the fill) to be left in place after construction of the permanent structure is complete been identified?
    b. Has a plan been provided regarding how to prevent the remaining portions of the temporary fill or walls from interfering with future construction and performance of the finished work (e.g., will the remaining portions impede flow of ground water, create a hard spot, create a surface of weakness regarding slope stability, etc.)?

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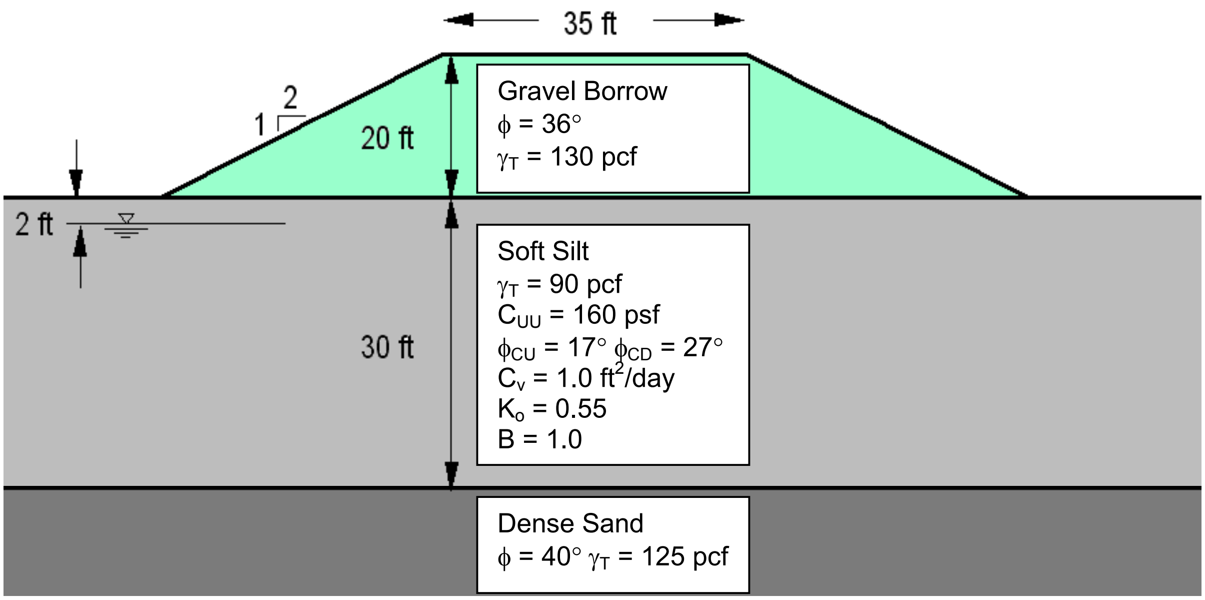
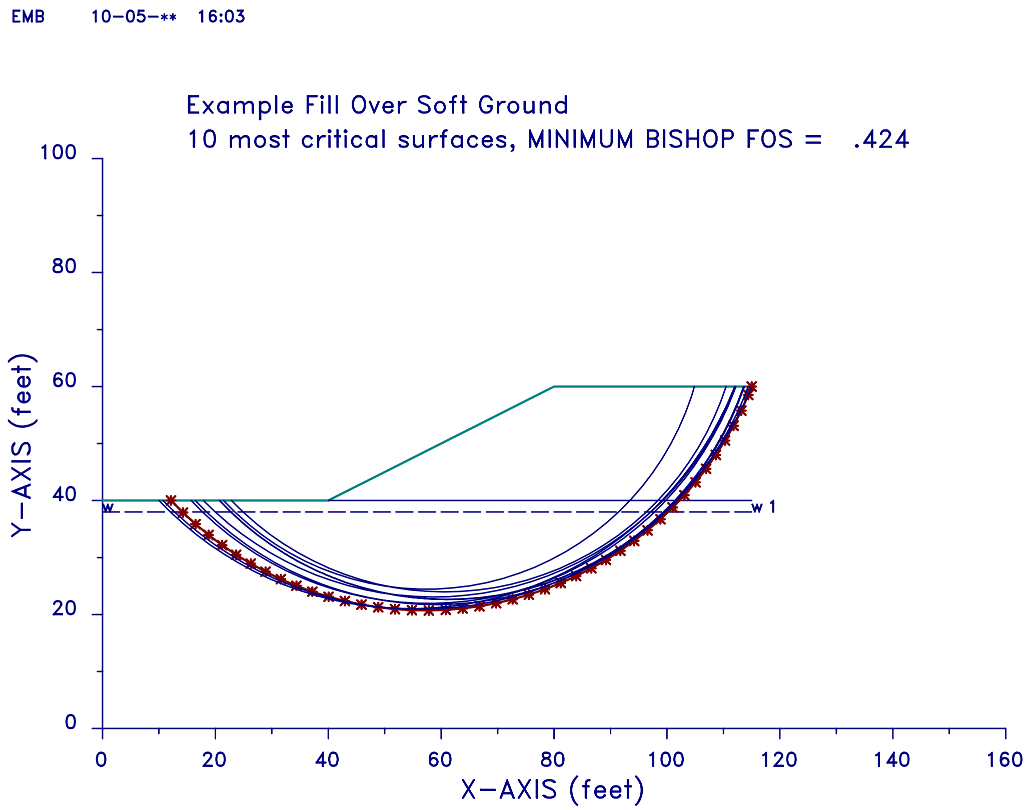
9-A.1 Problem Setup

First, the geotechnical designer should define the problem in terms of embankment geometry, soil stratigraphy, and water table information. For this example the proposed construction entails constructing a 20 feet thick earth embankment from Gravel Borrow with 2H:1V side slopes. The embankment will have a roadway width of 35 feet and will be constructed over soft silt. The soft silt is 30 feet thick and overlies dense sand. Ground water was observed 2 feet below the existing ground surface during the field exploration.



Embankment Geometry for Example
Figure 9-A-1

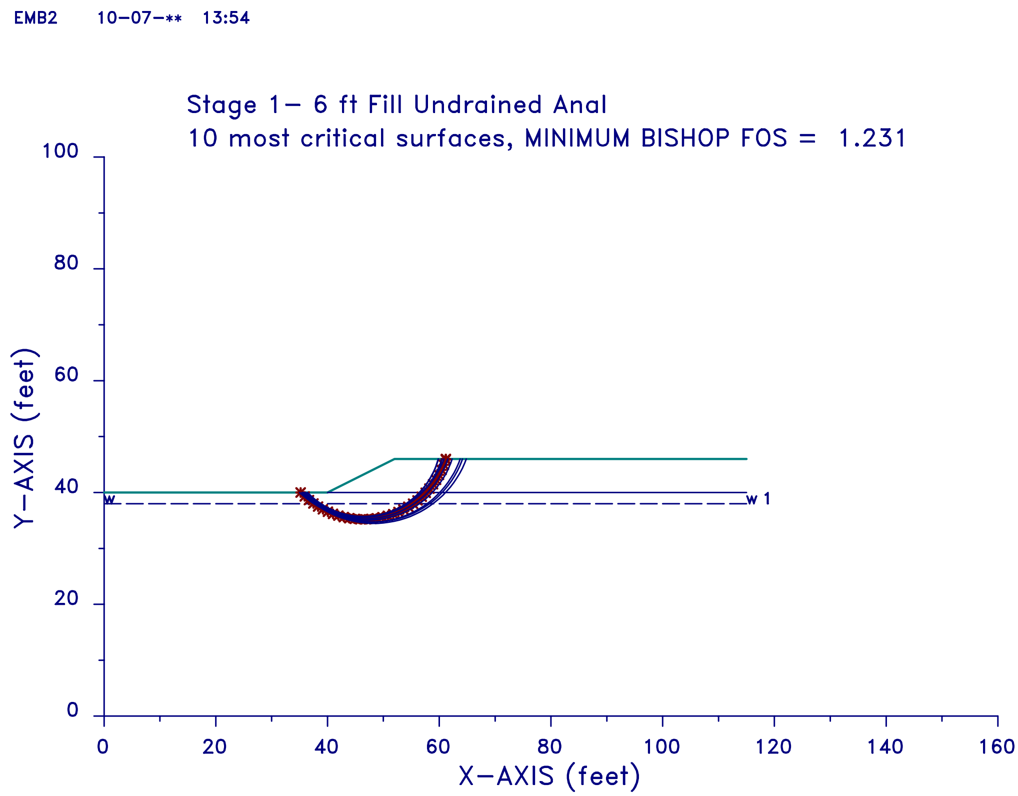
Using the test results, the geotechnical designer should first assess short term (undrained) strength of the embankment to determine if staged construction is required. For the example geometry, XSTABL was used to assess short-term (undrained) stability using C_{uu} = 160 psf (see Figures 9-4 and 9-5 for the specific strength envelopes used). Figure 9-A-2 provides the results of the stability analysis, and indicates that the factor of safety is well below the minimum long-term value of 1.25 required for an embankment without a structure. Therefore, staged construction or some other form or mitigation is required to construct the embankment. For this example, continue with a staged construction approach.



Undrained Stability for the Example Geometry
Figure 9-A-2

9-A.2 Determination of Maximum Stable First Stage Fill Height

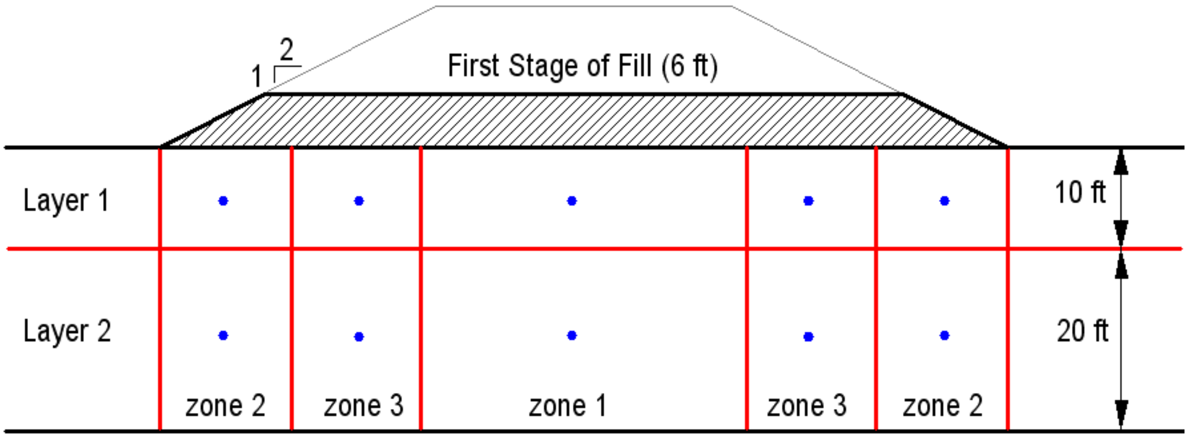
The analysis conducted in the previous section is conducted again, but this time limiting the fill height to that which has a factor of safety that is equal to or greater than the minimum acceptable interim value (use FS = 1.15 to 1.2 minimum for this example). As shown in Figure 9-A-3, the maximum initial fill height is 6 feet. This initial fill height is used as a starting point for both the total stress and the effective stress analyses.



Stage 1 Fill Stability, Assuming no Strength Gain and a Fill Height of 6 Feet
Figure 9-A-3

9-A.3 Total Stress Analysis Procedure Example

In this approach, the undrained soil strength envelope, or φ_{consol}, as determined in Figure 9-5, is used to characterize the strength of the subsoil. Next, the geotechnical designer determines how much strength gain can be obtained by allowing the first stage of fill to consolidate the underlying soft soils, using total stresses and undrained strengths after consolidation (see Section 9.3.1.3). The geotechnical designer calculates the stress increase resulting from the placement of the first embankment stage using the Boussinesq equation or those of Westergaard (see Figures 9-2 and 9-3). Note that because the stress increase due to the embankment load decreases with depth, the strength gain also decreases with depth. To properly account for this, the soft subsoil should be broken up into layers and zones for analysis just as is done for calculating settlement. For the example, the subsurface is divided into the layers and zones shown in Figure 9-A-4 to account for the differences in stress increase due to the embankment. The geotechnical designer will have to utilize judgment in determining the optimum number of layers and zones to use. If the division of zones is too coarse, the method may not properly model the field conditions during construction, and too fine of a division will result in excessive computational effort.



Division of Subsurface for Estimating Strength Increase and Consolidation
Figure 9-A-4

For the example geometry model the embankment as a continuous strip with a width of 103 feet (B = 35’ + (4x20) – (2x6)). As zone 3 is located close to the center of the embankment the stress change in that zone will be close to that near the center of the embankment for the stage 1 loading. Therefore, zone 3 is not used in the analysis example yet. It will be used later in the example. The stress increases in the zones are as follows:

Zone	Layer	Z	Z/B	I	σ_v 6 feet × 130 pcf	$\Delta\sigma_v$ ($I \times \sigma_v$)
1	1	5 feet	0.049	0.98	780 psf	764 psf
	2	20 feet	0.190	0.93	780 psf	725 psf
2	1	5 feet	0.049	0.55	780 psf	429 psf
	2	20 feet	0.190	0.75	780 psf	585 psf

Once the geotechnical designer has the stress increase, the increase in strength due to consolidation can be estimated using Equations 9-6 and 9-7. However, the strength increase achieved will depend on the degree of consolidation that occurs. The consolidation is dependant upon the time (t), drainage path length (H), coefficient of consolidation (C_v), and the Time Factor (T). Using Equations 9-8 through 9-10, assuming the stage 1 fill is allowed to consolidate for 15 days and assuming the soft soil layer is doubly drained, the percent consolidation would be:

$T = tC_v/H^2$
$T = 15\text{ days}(1\text{ feet}^2/\text{Day})/(30\text{ feet}/2)^2$ (assumed double draining)
$T = 0.067 = 0.25\pi U^2$; for $U < 60\%$
$U = 0.292$ or 29%

Therefore, at 15 days and 29% consolidation, using Equation 9-7, the strength gain would be as follows:

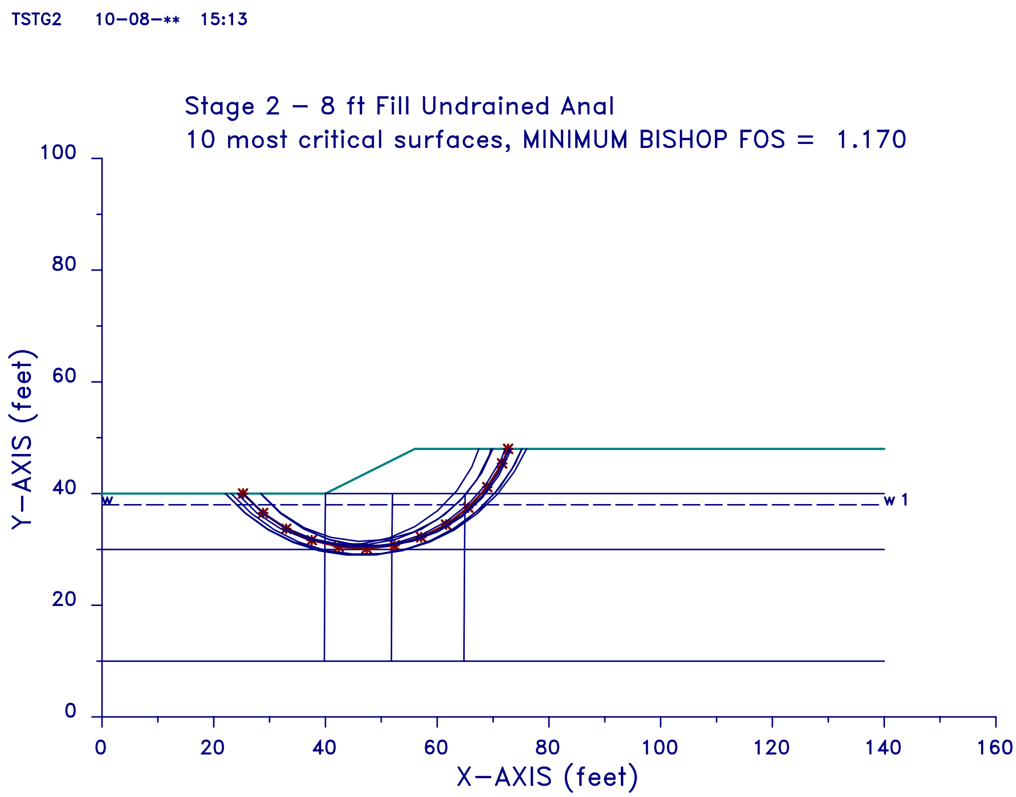
Zone	Layer	$\Delta\sigma_v$ ($I \times \sigma_v$)	C_{uui}	U	φ_{consol}	$C_{uu\ 29\%}$
1	1	764 psf	160 psf	0.29	22°	250 psf
	2	725 psf	160 psf	0.29	22°	245 psf
2	1	429 psf	160 psf	0.29	22°	210 psf
	2	585 psf	160 psf	0.29	22°	228 psf

Using the same procedure the strength gain at other time periods can be estimated. For example, at 60 days the percent consolidation would be 59%, and the strength gain would be as follows:

Zone	Layer	$\Delta\sigma_v$ ($I \times \sigma_v$)	Cuu_i	U	φ_{consol}	$C_{uu\ 59\%}$
1	1	764 psf	160 psf	0.59	22°	342 psf
	2	725 psf	160 psf	0.59	22°	333 psf
2	1	429 psf	160 psf	0.59	22°	262 psf
	2	585 psf	160 psf	0.59	22°	299 psf

The geotechnical designer should consider that as consolidation time increases the relative increase in strength becomes less as time continues to increase. Having a settlement delay period that would achieve 100% consolidation is probably not practical due to the excessive duration required. Delay period of more than 2 months are generally not practical. Continue the example assuming a 15 day settlement delay period will be required. Using the strength gained, the geotechnical designer determines how much additional fill can be placed.

Determine the height of the second stage fill that can be constructed by using $C_{uu\ 29\%}$ and increasing the fill height until the factor of safety is approximately 1.2 but not less than 1.15. As shown in Figure 9-A-5, the total fill height can be increased to 8 feet (2 feet of new fill is added) after the 15 day delay period.



Stage 2 Undrained Analysis, Assuming 15 Day Delay Period After Atage 1, and a Total Fill Height of 8 Feet
Figure 9-A-5

For the second stage of fill, the effective footing width changes as the fill becomes thicker. The equivalent footing width for use with the Boussinesq stress distribution will be 99 feet (B = 35' + (4 × 20) – (2 × 8)). As zone 3 is located close to the center of the embankment the stress change in that zone will be close to that near the center of the embankment for the stage 1 and stage 2 loading. Therefore, zone 3 is not used in the analysis example yet. It will be used later in the example. The stress increases in the zones are as follows:

Zone	**Layer**	**Z**	**Z/B**	**I**	σ_v **8 feet × 130 pcf**	$\Delta\sigma_v$ **(I × σ_v)**
1	1	5 feet	0.049	0.98	1040 psf	1019 psf
	2	20 feet	0.190	0.93	1040 psf	967 psf
2	1	5 feet	0.049	0.55	1040 psf	231 psf
	2	20 feet	0.190	0.75	1040 psf	315 psf

Once the geotechnical designer has the stress increase, the increase in strength due to consolidation can be estimated. The geotechnical designer must now begin to use weighted averaging to account for the difference in consolidation times (see Figure 9-6). The first stage of fill was allowed to settle for 15 days prior to placing the additional 2 feet of fill in the second stage, bringing the total fill height up to 8 feet. If the second lift of soil is allowed to consolidate for another 15 days, the soil will actually have been consolidating for 30 days total. For 30 days, the Time Factor (T). would be:

$T = tC_v/H^2$
$T = 30$ days(1 feet^2/Day)/(30 feet/2)^2 (assumed double draining)
$T = 0.133 = 0.25\pi U^2$; for $U < 60\%$
So, $U = 0.41$ or 41%

The average consolidation of the 15 + 15 day delay period will be:

[6 feet(0.41) + 2 feet(0.29)]/8 feet = 0.38 or 38%

The strength gain at 30 days and 38% average consolidation would be as follows:

Zone	Layer	$\Delta\sigma_v$ ($I \times \sigma_v$)	C_{uui}	U	φ_{consol}	$C_{uu\ 38\%}$
1	1	764 psf	160 psf	0.38	22°	317 psf
	2	725 psf	160 psf	0.38	22°	309 psf
2	1	429 psf	160 psf	0.38	22°	248 psf
	2	585 psf	160 psf	0.38	22°	280 psf

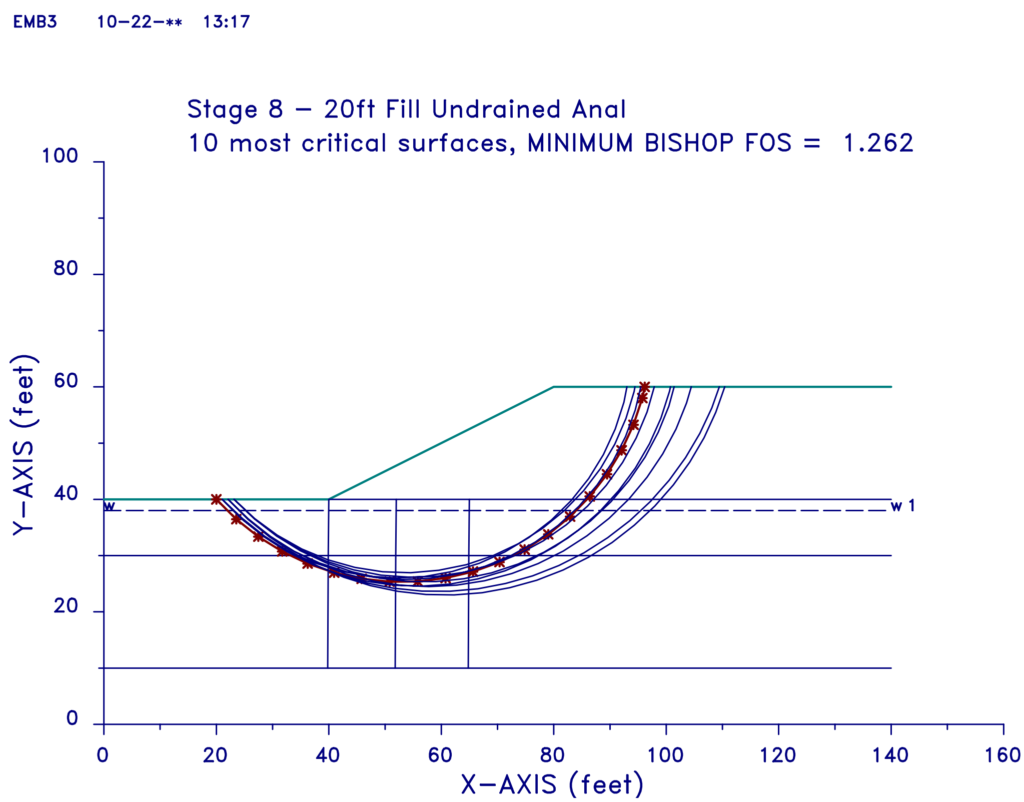
The geotechnical designer would continue this iterative process of adding fill, determining the weighted average consolidation, subsequent strength gain, and stability analysis to determine the next "safe" lift until the embankment is constructed full height.

Once the final stage fill is placed, it will continue to cause consolidation of the soft subsoil, increasing its strength. The calculations to determine the time required once the embankment is completed to cause the factor of safety to increase to the minimum long-term acceptable FS of 1.25 are summarized as follows:

Zone	Layer	$\Delta\sigma_v$ ($I \times \sigma_v$)	C_{uui}	U	φ_{consol}	$C_{uu\ 38\%}$
1	1	2509 psf	160 psf	0.71	22°	880 psf
	2	780 psf	160 psf	0.71	22°	384 psf
2	1	2314 psf	160 psf	0.71	22°	824 psf
	2	962 psf	160 psf	0.71	22°	436 psf
3	1	1430 psf	160psf	0.71	22°	570 psf
	2	1560 psf	160 psf	0.71	22°	608 psf

The calculations tabulated above assume that 25 days after the final fill layer is has elapsed, resulting in an average degree of consolidation of 71%.

The final stability analysis, using the undrained shear strengths tabulated above, is as shown in Figure 9-A-6.



Final Stage Undrained Analysis, Assuming 25 Days Have Expired Since Last Fill Increment Was Placed, and a Total Fill Height of 20 Feet
Figure 9-A-6

In summary, the fill increments and delay periods are as follows:

Stage	Fill Increment	Time Delay Prior to Next Stage
1	6 feet	15 days
2	2 feet	15 days
3	2 feet	15 days
4	2 feet	15 days
5	2 feet	30 days
6	2 feet	30 days
7	3 feet	10 days
8	1 feet	25 days to obtain FS = 1.25
TOTALS	20 feet	155 days

Fewer stages can be selected by the geotechnical designer, but longer delay periods are required to achieve more consolidation and the higher strength increases necessary to maintain stability. A comparable analysis using thicker fill stages and longer settlement delay periods yielded the following:

Stage	Fill Increment	Time Delay Prior to Next Stage
1	6 feet	60 days
2	4.5 feet	60 days
3	5.5 feet	40 days
4	4 feet	5 days to obtain FS = 1.25
TOTALS	20 feet	165 days

When using the total stress method of analysis it is often best to maximize the initial fill height. Doing this will produce the greatest amount of soil strength gain early in the construction of the fill. In addition, keeping the subsequent stages of fill as small as possible enables the fill to be constructed with the shortest total delay period, though in the end, the time required to achieve the final long-term safety factor is approximately the same for either approach.

9-A.4 Effective Stress Analysis Procedure Example

In this approach, the drained soil strength, or ϕ_{CD}, is used to characterize the strength of the subsoil. From Figure 9-5, ϕ_{CD} is 27°. However, it is the buildup of pore pressure during embankment placement that causes the embankment to become unstable. The amount of pore pressure buildup is dependent on how rapidly the embankment load is placed. Given enough time, the pore pressure buildup will dissipate and the soil will regain its effective strength, depending on the permeability and compressibility of the soil. The key to this approach is to determine the amount of pore pressure build up that can be tolerated before the embankment safety factor drops to a critical level when using ϕ_{CD} for the soil strength. A limit equilibrium stability program such as XSTABL should be used to determine the pore pressure increase that can be tolerated and result in the embankment having a safety factor of 1.15 to 1.2 during construction.

Many of the newer stability programs have the ability to accept r_u values directly or to calculate r_u. The geotechnical designer should be aware of how the stability program calculates r_u. When using XSTABL, the geotechnical designer should not input r_u directly. Instead, he should input excess pore pressures directly into the program and then run the stability analysis.

The rate of fill construction required to prevent r_u from being exceeded cannot be determined directly from the drained analysis, as embankment stability needs in addition to the subsoil consolidation rate affects the rate of construction. The total construction time cannot therefore be determined directly using C_v and the percent consolidation required for stability.

Using the example geometry shown in Figure 9-A-1, the geotechnical designer should divide the subsurface into layers and zones in a manner similar to that shown in Figure 9-A-4. The geotechnical designer then determines the stress increase due to the first stage of fill, 6 feet in this case.

The stress increases in the zones are as follows based on an equivalent strip footing width of 103 feet:

Zone	Layer	Z	Z/B	I	σ_v 6 feet × 130 pcf	$\Delta\sigma_v$ (I × σ_v)
1	1	5 feet	0.049	0.98	780 psf	764 psf
	2	20 feet	0.190	0.93	780 psf	725 psf
2	1	5 feet	0.049	0.55	780 psf	429 psf
	2	20 feet	0.190	0.75	780 psf	585 psf
3	1	5 feet	0.049	0.98	780 psf	764 psf
	2	20 feet	0.019	0.93	780 psf	725 psf

Note that Zone 3 has the same stress increase as Zone 1.

As discussed previously in Section 9.3.1.4, the pore pressure increase is dependent upon the load and the degree of consolidation. Using Equation 9-15 with an assumed percent consolidation, determine the pore pressure change to use in the stability analysis. It will be necessary to perform the analysis for several percent consolidations to determine what the critical pore pressure is for maintaining stability.

$K_0 = 1 - \sin \varphi_{CD} = 1 - \sin 27° = 0.55$

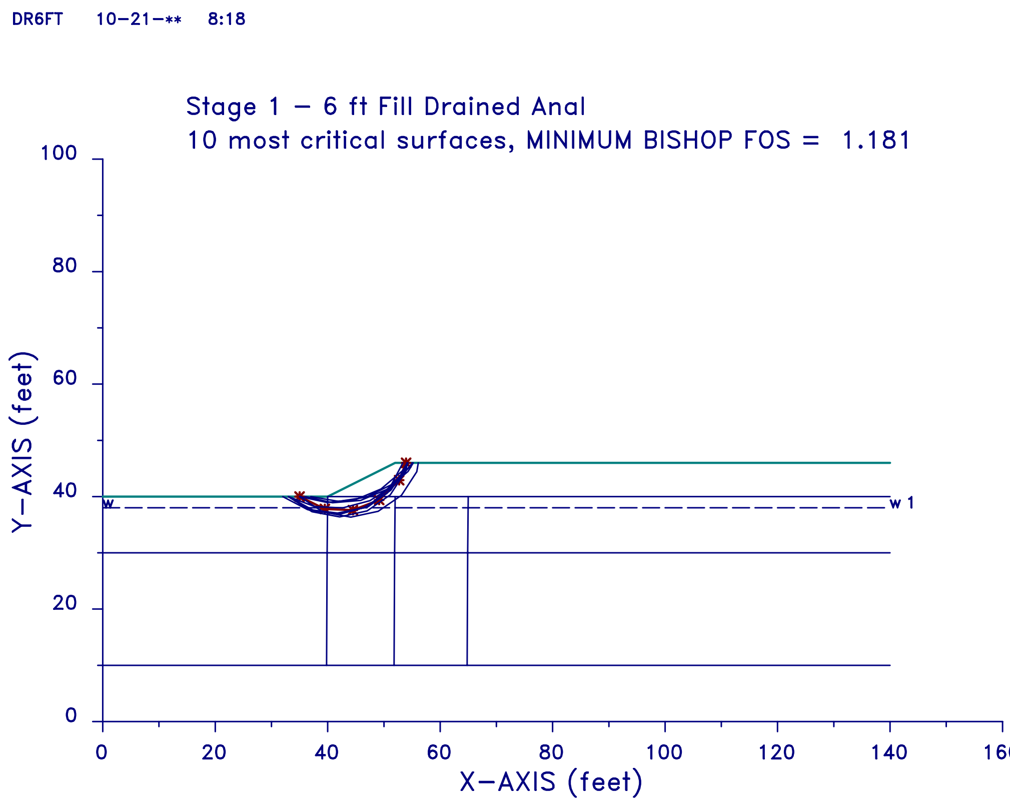
B = 1.0, assuming subsoil is fully saturated. For Layer 1, Zone 1, at 30% consolidation,

$\Delta u_p = B[(1 + 2K_0)/3]\Delta\sigma v(1\text{-}U) = 1.0[(1 + 2(0.55))/3](764 \text{ psf})(1\text{-}.30) = 374 \text{ psf}$

The remaining values are as follows:

Layer	Zone	$\Delta\sigma_v$ ($I \times \sigma_v$) (psf)	U (%)	$\Delta u_{p30\%}$ (psf)	U (%)	$\Delta u_{p35\%}$ (psf)	U (%)	$\Delta u_{p40\%}$ (psf)
1	1	764	30	374	35	346	40	320
	2	725	30	354	35	329	40	303
2	1	429	30	209	35	194	40	179
	2	585	30	286	35	265	40	245
3	1	764	30	373	35	346	40	320
	2	725	30	354	35	329	40	303

The slope stability results from XSTABL are provided in Figure 9-A-7. For the two subsoil layers, all zones, a drained friction angle, ϕ_{CD}, of 27° was used, and the pore pressure increases Δu_p from the tabulated summary of the calculations provided above were inserted into the soil zones shown in Figure 9-A-7 as pore pressure constants. The results shown in this figure are for a percent consolidation of 35%.



Stage 1 Drained Analysis at Percent Consolidation of 35% and a Fill Height of 6 Feet

Figure 9-A-7

Using Equation 9-16, r_u at this stage of the fill construction is determined as follows:

$$r_u = B[(1 + 2K_0)/3](1\text{-}U) = 1.0[(1 + 2(0.55))/3](1\text{-}0.35) = 0.45$$

Subsequent stages of fill construction are checked to determine the critical pore pressure ratio, up to the point where the fill is completed. The pore pressure ratio is evaluated at several fill heights, but not as many stages need to be analyzed as is the case for total stress analysis, as the rate of fill construction is not the focus of the drained analysis. All that needs to be achieved here is to adequately define the relationship between r_u and the fill height. Therefore, one intermediate fill height (13.5 feet) and the maximum fill height (20 feet) will be checked.

For a fill height of 13.5 feet, the stress increases in the zones are as follows based on an equivalent strip footing width of 88 feet:

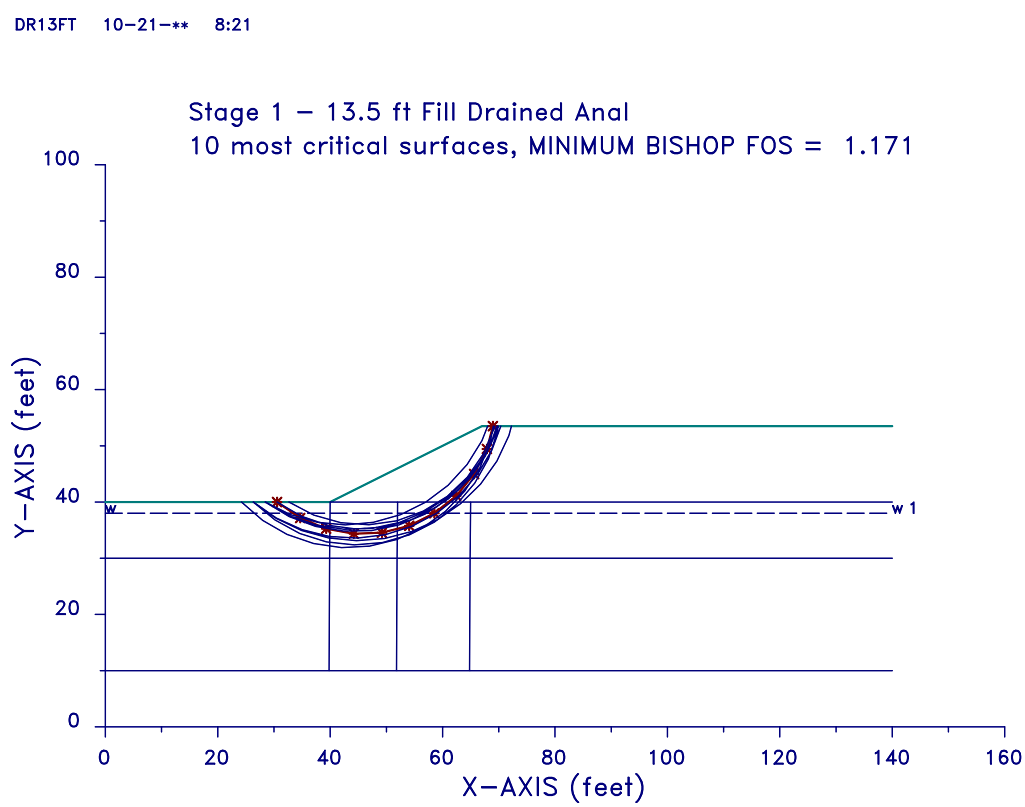
Zone	Layer	Z	Z/B	I	σ_v 13 feet × 130 pcf	$\Delta\sigma_v$ ($I \times \sigma_v$)
1	1	5 feet	0.049	0.97	1,690 psf	1,700 psf
	2	20 feet	0.190	0.90	1,690 psf	1,580 psf
2	1	5 feet	0.049	0.40	1,690 psf	702 psf
	2	20 feet	0.190	0.55	1,690 psf	965 psf
3	1	5 feet	0.049	0.75	1,690 psf	1,320 psf
	2	20 feet	0.019	0.70	1,690 psf	1,230 psf

Note that the stress increase in Zone 3 is now different than the stress increase in Zone 1, due to the fact that the embankment slope now is over the top of Zone 3.

The pore pressure increase resulting from a 13.5 feet high fill, assuming various percent consolidations, is recalculated using Equation 9-15 as illustrated earlier. The results of these calculations are as tabulated below:

Zone	Layer	$\Delta\sigma_v$ ($I \times \sigma_v$) (psf)	U (%)	$\Delta u_{p55\%}$ (psf)	U (%)	$\Delta u_{p60\%}$ (psf)	U (%)	$\Delta u_{p65\%}$ (psf)
1	1	1702	55	534	60	475	65	415
	2	1580	55	496	60	441	65	386
2	1	702	55	220	60	196	65	171
	2	695	55	218	60	194	65	170
3	1	1316	55	413	60	367	65	321
	2	1229	55	386	60	343	65	300

Note that higher percent consolidations are targeted, as a higher percent consolidation is likely to have occurred by the time the fill is 13.5 feet high. The slope stability results from XSTABL are provided in Figure 9-A-8. For the two subsoil layers, all zones, a drained friction angle, φ_{CD}, of 27° was used, and the pore pressure increases Δu_p from the tabulated summary of the calculations provided above were inserted into the soil zones shown in Figure 9-A-8 as pore pressure constants. The results shown in this figure are for a percent consolidation of 60%.



Stage 2 Drained Analysis at Percent Consolidation of 60% and a Fill Height of 13.5 Feet
Figure 9-A-8

Using Equation 9-16, r_u at this stage of the fill construction is determined as follows:

$$r_u = B[(1 + 2K_0)/3](1\text{-}U) = 1.0[(1 + 2(0.55))/3](1\text{-}0.60) = 0.28$$

Similarly, these calculations were conducted for the full fill height of 20 feet, and for a minimum FS = 1.15 to 1.2, r_u was determined to be 0.22 (U = 68%).

In summary, the pore pressure ratios that should not be exceeded during fill construction are as follows:

Total Fill Height (ft)	r_u
6	0.45
13.5	0.28
20	0.22

Values of r_u could be interpolated to estimate the critical r_u at other fill heights.
It should be assumed that if these values of r_u are used to control the rate of fill construction, the time required to build the fill will be approximately as determined from the total stress analysis provided in the previous section.