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Remarks:
The *Geotechnical Design Manual M 46-03*, Chapter 6, has been revised in its entirety. Please remove and recycle Chapter 6, Seismic Design, in any printed manuals. Changes are effective November 3, 2008. The electronic version of this manual is available at:

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6.1 Seismic Design Responsibility and Policy

6.1.1 Responsibility of the Geotechnical Designer

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

6.1.2 Geotechnical Seismic Design Policies

6.1.2.1 Seismic Performance Objectives

In general, the AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications shall be followed for structure classification of bridges. Critical, essential, and other structures are defined in AASHTO LRFD Bridge Design Specifications. In the current inventory, most structures are considered "other" with a few being "essential" or "critical". In keeping with the current seismic design approaches employed both nationally and internationally, geotechnical seismic design shall be consistent with the philosophy for structure design that loss of life and serious injury due to structure collapse are minimized, to the extent possible and economically feasible. This performance objective shall be achieved at a seismic risk level that is consistent with the seismic risk level required in the AASHTO specifications (e.g., 7 percent probability of exceedance in 75 years for other structures, or lower probability of exceedance such as 2 percent in 50 years for critical or essential bridges, as determined by the State Bridge Engineer – see ***Geotechnical Design Manual* M 46-03 (GDM) Section 6.3.1**). The definition of structure collapse is provided in the WSDOT LRFD *Bridge Design Manual* M 23-50 (BDM). Bridges, regardless of their AASHTO classification, may suffer damage and may need to be replaced after a design seismic event, but they are designed for non-collapse due to earthquake shaking and geologic hazards associated with a design seismic event.

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

foundations should also be designed to be adequately stable with regard to liquefaction, lateral flow, and other seismic effects to prevent bridge collapse.

All retaining walls and abutment walls shall be evaluated and designed for seismic stability internally and externally (i.e. sliding and overturning). With regard to overall seismic slope stability (often referred to as global stability) involving a retaining wall, with or without liquefaction, the geotechnical designer shall evaluate the impacts of failure due to seismic loading, if failure is predicted to occur. If collapse of the wall is likely during the design seismic event (i.e., does not meet minimum slope stability level of safety requirements during seismic loading in accordance with **WSDOT GDM Section 6.4.3.1**), and if that collapse is likely to cause loss of life or severe injury to the traveling public, the stability of the wall shall be improved such that the life safety of the traveling public during the design seismic event is preserved. As a general guide, walls that are less than 10 ft in height, or walls that are well away from the traveled way, are not likely to cause loss of life or severe injury to the traveling public. Therefore, the wall design may allow these lower height walls, or walls that are well away from the traveled way, to deform, translate, or rotate during a seismic event and overall stability of these lower height walls may be compromised.

Note that the policy to stabilize retaining walls for overall stability due to design seismic events may not be practical for walls placed on or near large marginally stable landslide areas or otherwise marginally stable slopes. In general, if the placement of a wall within a marginally stable slope (i.e., marginally stable for static conditions) has only a minor effect on the seismic stability of the landslide or slope, or if the wall has a relatively low risk of causing loss of life or severe injury to the traveling public if wall collapse occurs, the requirement of the wall and slope to meet minimum seismic overall stability requirements may be waived, subject to the approval of the State Geotechnical Engineer.

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 if failure due to seismic loading occurs should be stabilized.

6.1.2.2 Liquefaction Mitigation for Bridge Widenings

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

bridge, programming the implementation of the liquefaction mitigation of the existing bridge as part of the overall WSDOT seismic retrofit program. See WSDOT Instructional Letter IL4074.00 for the specific policy regarding this issue.

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

6.1.2.3 Maximum Considered Depth for Liquefaction

When evaluating liquefaction potential and its impacts to transportation facilities, the maximum considered liquefaction depth below the natural ground surface shall be limited to 80 ft. 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 with the approval of the State Geotechnical Engineer 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 ft). 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, that is highly variable at depth. For example, at shallow depth (15 ft), 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 ft. The uncertainty regarding the coefficient r_d and lack of verification of the simplified procedures used to predict liquefaction at depth, as well as some of the simplifying assumptions and empiricism within the simplified method with regard to the calculation of liquefaction resistance (i.e., the cyclic resistance ratio CRR), limit the depth at which these simplified procedures should be used. Therefore, simplified empirical methods should not be used to predict liquefaction at depths greater than 50 to 60 ft, and shall not be used at depths of greater than 80 ft.

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 specific approval from the State Geotechnical Engineer as well as independent peer review by expert(s) in the use of such methods for liquefaction analysis.

Decreasing Impact with Depth. Observation and analysis of damage in past earthquakes suggests that the damaging effects of liquefaction generally decrease as the depth of a liquefiable layer increases. This reduction in damage is largely attributed to decreased levels of relative displacement and the need for potential failure surfaces to extend down to the liquefying layer. The effects of a 10 ft thick soil layer liquefying between depths of 80 and 90 ft will generally be much less severe than those of a layer between the depths of 10 and 20 ft. 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.

The most current version of the AASHTO Guide Specifications for LRFD Seismic Bridge Design and AASHTO LRFD Bridge Design Specifications shall be used for geotechnical seismic design, in addition to the WSDOT BDM and GDM. 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. The BDM and GDM provide specific application of the AASHTO specifications to WSDOT design policy and practice.

For seismic design of new buildings and non-roadway infrastructure, the 2006 International Building Code (IBC) (International Code Council, 2006) should be used.

In addition to the above mentioned design specifications, geotechnical designers may utilize other resources that are available for geotechnical earthquake engineering to provide more detailed guidance in seismic design for design issues and areas not addressed in detail in the AASHTO specifications or herein. A brief description of four of these additional references is as follows:

FHWA Geotechnical Engineering Circular No. 3 (Kavazanjian, et al., 1997). 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.

NCHRP Report 472 (ATC-MCEER Joint Venture, 2001 and 2002). The National Cooperative Highway Research Program Report 472 (2002), "Comprehensive specifications for the Seismic Design of Bridges", is a report containing the findings of a study completed to develop recommended specifications for seismic design of highway bridges. The report covers topics including design earthquakes and performance objectives, foundation design, liquefaction hazard assessment and design, and seismic hazard representation. Of particular interest, this document contains a case-study on liquefaction assessment of a hypothetical bridge in Washington State including estimating lateral spread induced loads on the bridge.

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

The results of the hazards analysis using the 2006 USGS website hazard model at a return period of 5 percent in 50 years are the same as those from the AASHTO hazard analysis maps. However, the USGS is currently updating their hazards maps (see USGS website for 2008 update). Once the USGS update has been adopted, the hazard results from the USGS could differ somewhat from the results from the AASHTO hazards maps for the same location. In this case the AASHTO hazard maps should be used as the basis for design unless specific approval is received from the State Bridge Engineer and State Geotechnical Engineer.

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

Geotechnical seismic design is a rapidly developing sub-discipline within the broader context of the geotechnical engineering discipline, and new resources such as technical journal articles, as well as academic and government agency research reports, are becoming available to the geotechnical engineer. It is important when using these other resources, as well as those noted above, that a review be performed to confirm that the guidance represents the current state of knowledge and that the methods have received adequate independent review. Where new methods not given in the AASHTO Specifications or herein (i.e., **WSDOT GDM 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 (**WSDOT GDM Section 6.3.1**) with specification/code based ground motion response (**WSDOT GDM Section 6.3.2**)
- Use specification/code based hazard (**WSDOT GDM Section 6.3.1**) with site specific ground motion response (**WSDOT GDM Appendix 6-A**)
- Use site specific hazard (**WSDOT GDM Appendix 6-A**) with specification/code based ground motionresponse (**WSDOT GDM Section 6.3.2**)
- Use site specific hazard (**WSDOT GDM Appendix 6-A**) with site specific ground motion response (**WSDOT GDM Appendix 6-A**)

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

6.2.2 Site Characterization and Development of Seismic Design Parameters

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

- Identify, in coordination with the structural designer, structural characteristics (e.g., fundamental frequency/period), anticipated method(s) of structural analysis, performance criteria (e.g., collapse prevention, allowable horizontal displacements, limiting settlements, target load and resistance factors, components requiring seismic design, etc.) and design hazard levels (e.g., 7 percent PE in 75 years).
- Identify, in coordination with the structural engineer, what type of ground motion parameters are required for design (e.g., response spectra or time histories), and their point of application (e.g., mudline, bottom of pile cap, or depth of pile fixity).
- Identify, in coordination with the structural engineer, how foundation stiffness will be modeled and provide appropriate soil stiffness properties or soil/foundation springs.
- Identify potential geologic hazards, areas of concern (e.g. soft soils), and potential variability of local geology.

- Identify potential for large scale site effects (e.g., basin, topographic, and near fault effects).
- Identify, in coordination with the structural designer, the method by which risk-compatible ground motion parameters will be established (specification/code, deterministic, probabilistic, or a hybrid).
- Identify engineering analyses to be performed (e.g. site specific seismic response analysis, liquefaction susceptibility, lateral spreading/slope stability assessments).
- Identify engineering properties required for these analyses.
- Determine methods to obtain parameters and assess the validity of such methods for the material type.
- Determine the number of tests/samples needed and appropriate locations to obtain them.

It is assumed that the basic geotechnical investigations required for nonseismic (gravity load) design have been or will be conducted as described in **WSDOT GDM Chapters 2, 5** and the individual project element chapters (e.g., **WSDOT GDM Chapter 8** for foundations, **WSDOT GDM 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 should be given to drill rigs furnished with energy calibrated automatic SPT hammers. 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.

Geotechnical Issues	Engineering Evaluations	Required Information for Analyses	Field Testing	Laboratory Testing
Site Response	• source characterization and ground motion attenuation • site response spectra • time history	• subsurface profile (soil, groundwater, depth to rock) • shear wave velocity • shear modulus for low strains • relationship of shear modulus with increasing shear strain, OCR, and PI • equivalent viscous damping ratio with increasing shear strain, OCR, and PI • Poisson's ratio • unit weight • relative density • seismicity (design earthquakes - source, distance, magnitude, recurrence)	• SPT • CPT • seismic cone • geophysical testing (shear wave velocity) • piezometer	• Atterberg limits • grain size distribution • specific gravity • moisture content • unit weight • resonant column • cyclic direct simple shear test • torsional simple shear test • cyclic triaxial tests
Geologic Hazards Evaluation (e.g., liquefaction, lateral spreading, slope stability, faulting)	• liquefaction susceptibility • liquefaction triggering • liquefaction induced settlement • settlement of dry sands • lateral spreading flow failure • slope stability and deformations	• • subsurface profile (soil, groundwater, rock) • shear strength (peak and residual) • unit weights • grain size distribution • plasticity characteristics • relative density • penetration resistance • shear wave velocity • seismicity (PGA, design earthquakes, deaggregation data, ground motion time histories) • site topography	• SPT • CPT • seismic cone • Becker penetration test • vane shear test • piezometers • geophysical testing (shear wave velocity)	• grain size distribution • Atterberg Limits • specific gravity • organic content • moisture content • unit weight • soil shear strength tests (static and cyclic) • post-cyclic volumetric strain

Geotechnical Issues	Engineering Evaluations	Required Information for Analyses	Field Testing	Laboratory Testing
Input for Structural Design	• soil stiffness for shallow foundations (e.g., springs) • P-Y data for deep foundations • down-drag on deep foundations • residual strength • lateral earth pressures • lateral spreading/slope movement loading • post earthquake settlement • Kenematic soil-structure interaction	• subsurface profile (soil, groundwater, rock) • shear strength (peak and residual) • coefficient of horizontal subgrade reaction • seismic horizontal earth pressure coefficients • shear modulus for low strains or shear wave velocity • relationship of shear modulus with increasing shear strain • unit weight • Poisson's ratio • seismicity (PGA, design earthquake, response spectrum, ground motion time histories) • site topography • Interface strength	• CPT • SPT • seismic cone • piezometers • geophysical testing (shear wave velocity, resistivity, natural gamma) • vane shear test • pressuremete	• grain size distribution • Atterberg limits • specific gravity • moisture content • unit weight • resonant column • cyclic direct simple shear test • triaxial tests (static and cyclic) • torsional shear test • direct shear interface tests

Summary of site characterization needs and testing considerations for seismic design (adapted from Sabatini, et al., 2002).
Table 6-1

WSDOT GDM 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.

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

- Table 6-2, which presents correlations for estimating initial shear modulus based on relative density, penetration resistance or void ratio.
- Shear modulus reduction and equivalent viscous damping ratio equations by Darendelli (2001), applicable to all soils, as provided below, or Figure 6-1, which presents shear modulus reduction curves and equivalent viscous damping ratio for sands as a function of shear strain and depth, and, Figures 6-2 and 6-3, which present shear modulus reduction curves and equivalent viscous damping ratio, respectively, as a function of cyclic shear strain and plasticity index for fine grained soils.
- Figures 6-4 through 6-7, which present charts for estimating equivalent undrained residual shear strength for liquefied soils as a function of SPT blowcounts. It is recommended that all these figures be checked to estimate residual strength and averaged using a weighting scheme. Table 6-3 presents an example of a weighting scheme as recommended by Kramer (2008). 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.

Other property value correlations may be used, subject to the approval of the State Geotechnical Engineer. Designers are encouraged to develop region or project specific correlations for these seismic design properties.

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

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

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

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

Reference	Correlation	Units(1)	Limitations
Seed et al. (1984)	$G_{max} = 220\ (K_2)_{max}\ (\sigma'_m)^{½}$ $(K_2)_{max} = 20(N_1)_{60}^{1/3}$	kPa	$(K_2)_{max}$ is about 30 for very loose sands and 75 for very dense sands; about 80 to 180 for dense well graded gravels; Limited to cohesionless soils
Imai and Tonouchi (1982)	$G_{max} = 15{,}560\ N_{60}^{0.68}$	kPa	Limited to cohesionless soils
Ohta and Goto (1976)	$G_{max} = 20000(N_1)_{60}^{⅓}(\sigma'_{60})^{½}$	psf	Limited to cohesionless soils
Mayne and Rix (1993)	$G_{max} = 99.5(P_a)^{0.305}(q_c)^{0.695}/(e_0)^{1.13}$	kPa(2)	Limited to cohesive soils; P_a = atmospheric pressure
Notes: (1) 1 kPa = 20.885 psf (2) P_a and q_c in kPa			

Correlations for estimating initial shear modulus (adapted from Kavazanjian, et al., 1997).
Table 6-2

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

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

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:

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

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

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

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

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

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

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:

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

where

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

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

where,

freq = frequency of loading, in Hz

N = number of loading cycles

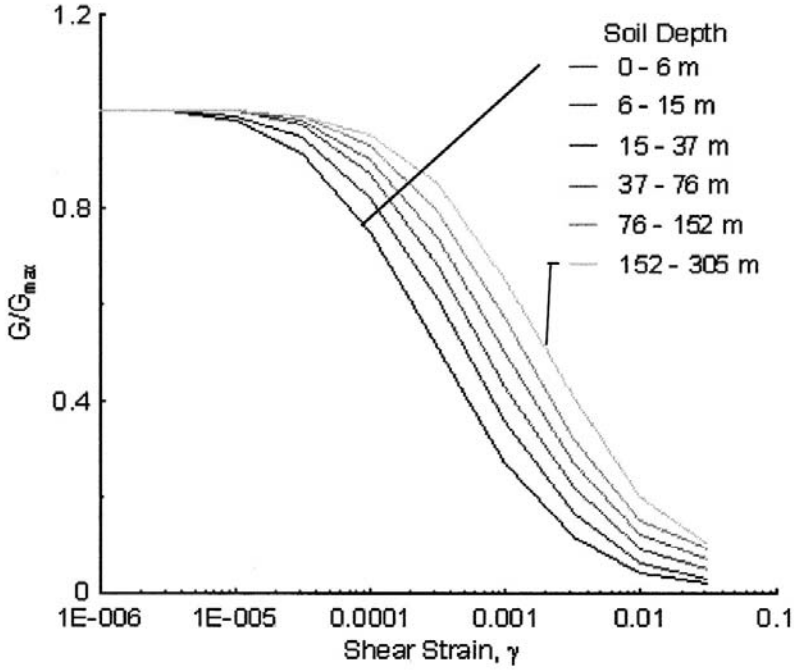
ϕ_6 = 0.8005; ϕ_7 = 0.0129; ϕ_8 = -0.1069;

ϕ_9 = -0.2889; ϕ_{10}= 0.2919; ϕ_{11}= 0.6329; ϕ_{12} = -0.0057

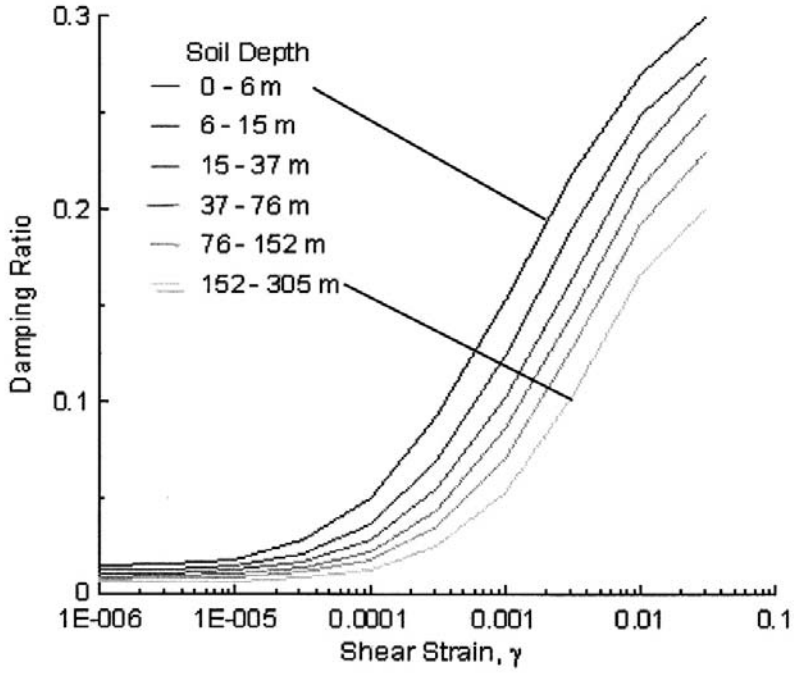
Model	Weighting Factor
Idriss	0.2
Olson-Stark	0.2
Idriss-Boulanger	0.2
Hybrid	0.4

Weighting factors for residual strength estimation (Kramer, 2008).

Table 6-3

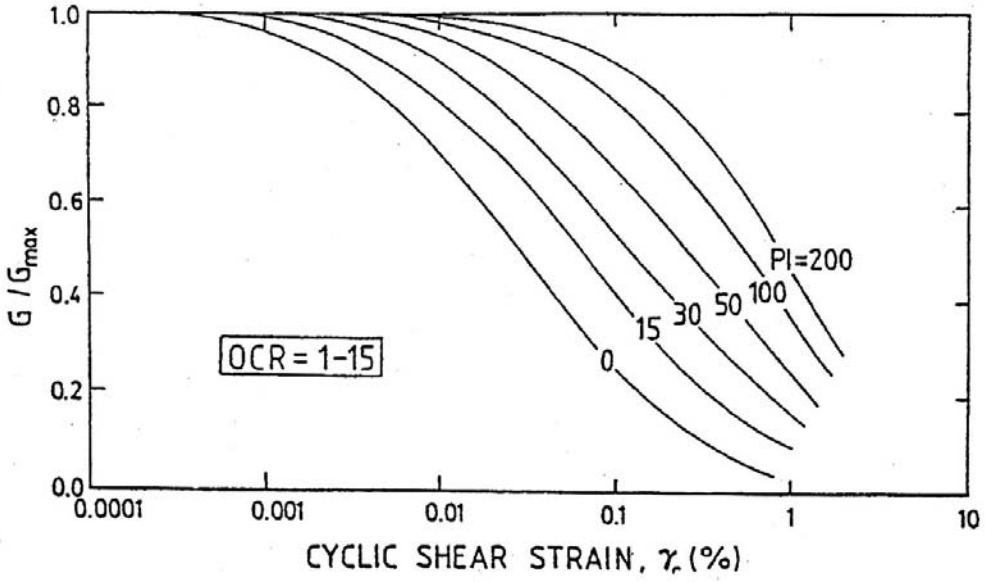


Shear Modulus Reduction Curves

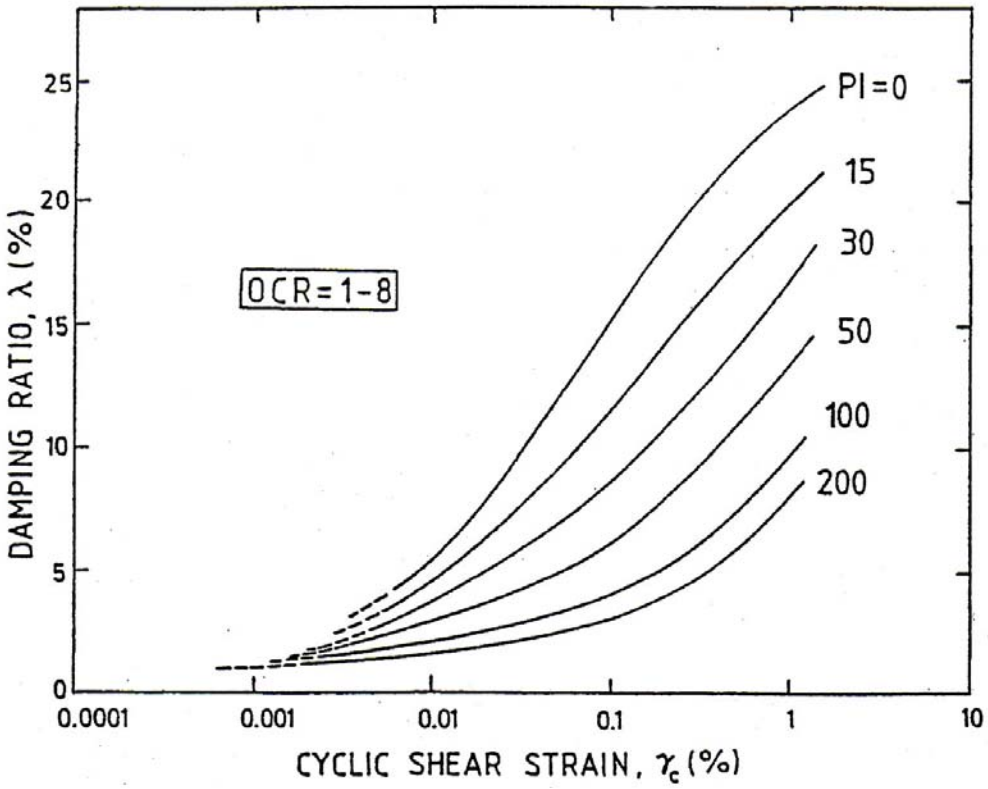


Damping Ratio Curves

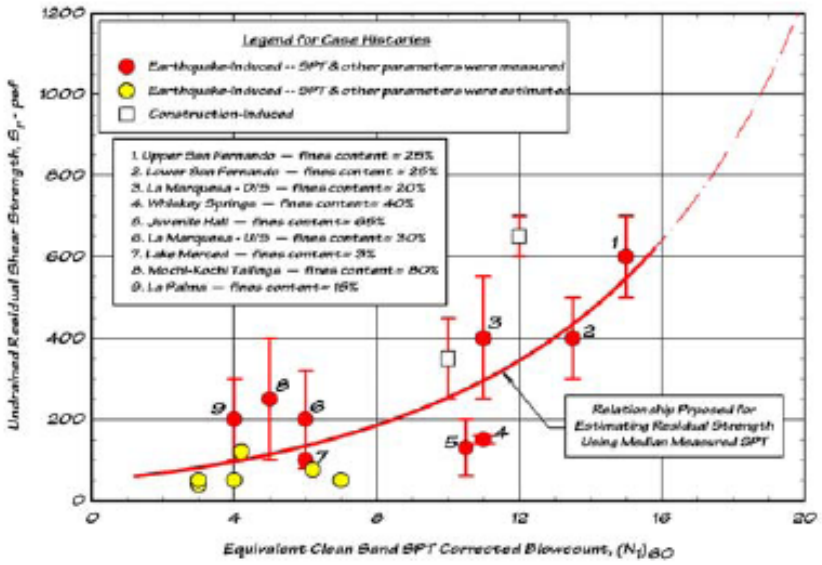
Shear modulus reduction and damping ratio curves for sand (EPRI, 1993).
Figure 6-1



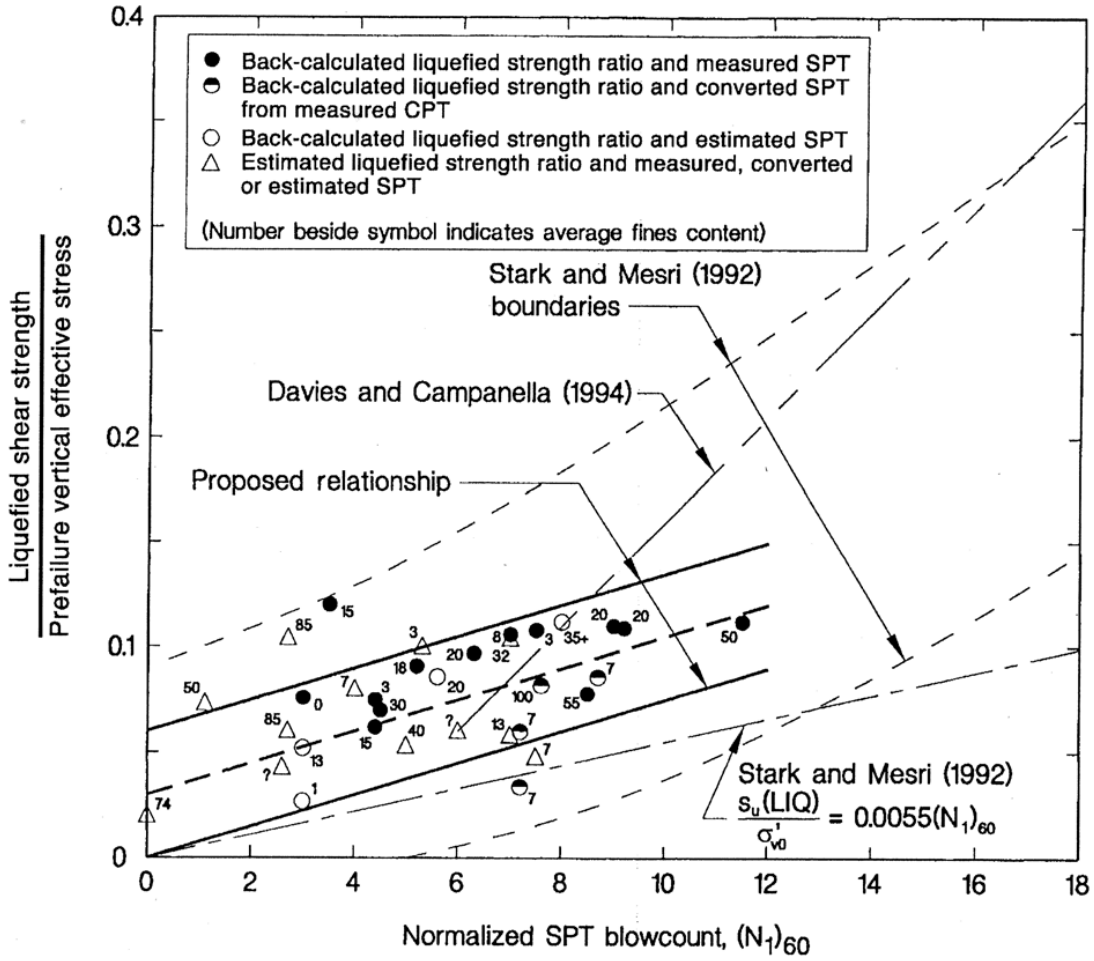
Shear modulus reduction curves for fine grained soils (Vucetic and Dobry, 1991).
Figure 6-2



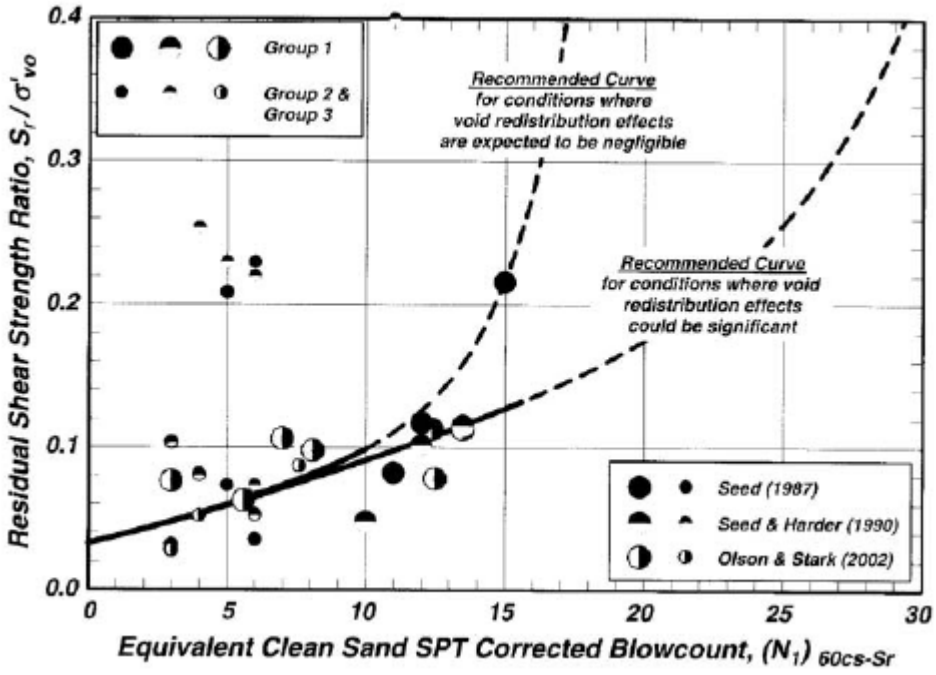
Equivalent viscous damping ratio for fine grained soils (Vucetic and Dobry, 1991).
Figure 6-3



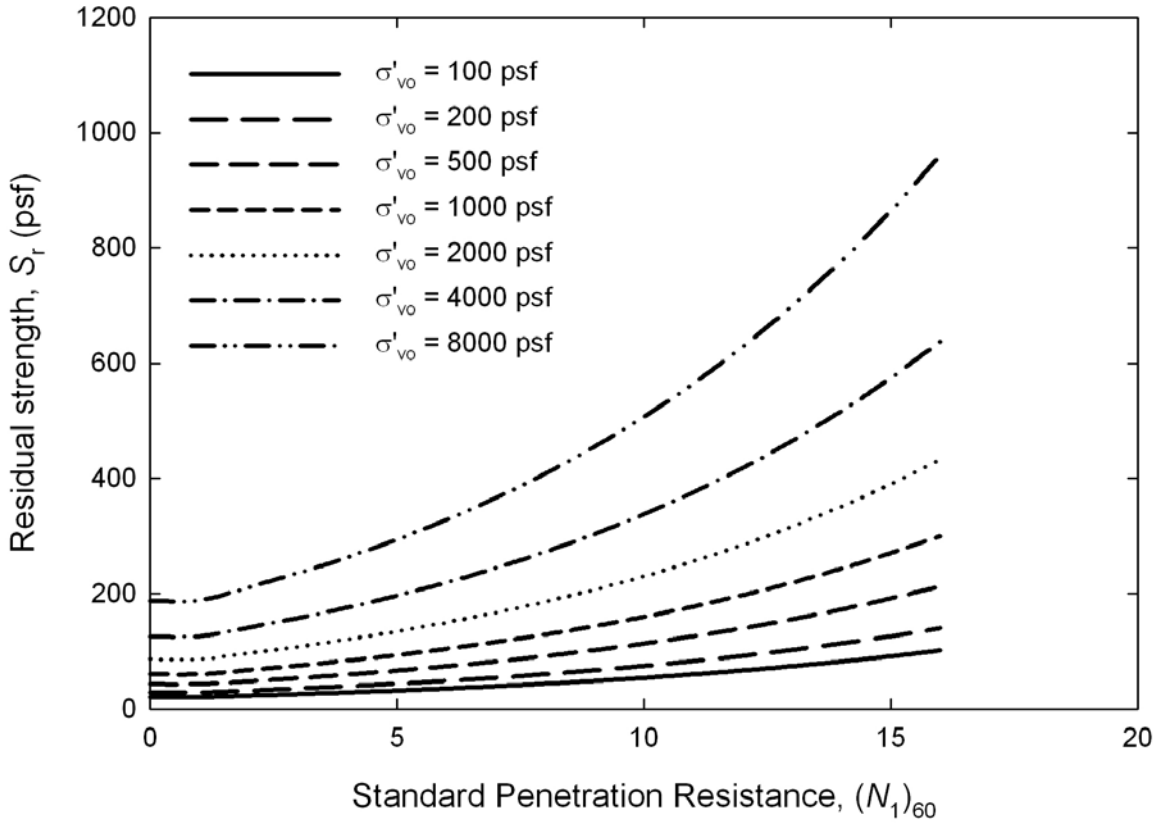
Estimation of residual strength from SPT resistance (Idriss and Boulanger, 2007).
Figure 6-4



Estimation of residual strength ratio from SPT resistance (Olson and Stark, 2002).
Figure 6-5



Estimation of residual strength ratio from SPT resistance (Idriss and Boulanger, 2007).
Figure 6-6



Variation of residual strength ratio with SPT resistance and initial vertical effective stress using Kramer-Wang model (Kramer, 2008).
Figure 6-7

6.2.3 Information for Structural Design

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

- Foundation spring values for dynamic loading (lateral and vertical), as well as geotechnical parameters for evaluation of sliding resistance applicable to the foundation design. If liquefaction is possible, spring values for liquefied 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 should be used. However, a site specific hazard analysis should be considered in the following situations:

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

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

A site specific ground motion response analysis should 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, or
- Site subsurface conditions are classified as Site Class F.

A site specific ground motion response analysis should also be considered, subject to the approval of the State Bridge Engineer and State Geotechnical Engineer, for sites where the effects of liquefaction on the ground motion could cause the ground motion response to be overly conservative or unconservative, or where the AASHTO or IBC site classes do not fit the subsurface conditions adequately.

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

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

- For liquefiable sites, use the specification based site coefficient for soil conditions without any modifications for liquefaction. This approach is believed to be conservative for higher frequency motions (i.e., $T_F < 1.0$ sec). If a site specific ground response analysis is conducted, the response spectrum should be no lower than two-thirds of the non-liquefied specification based spectrum, unless specifically approved by the State Bridge and Geotechnical Engineers to go lower. However, when accepting a spectrum lower than two-thirds of the specification based spectrum, the uncertainties in the analysis method should be carefully reviewed, particularly for longer periods (i.e., $T > 1.0$ sec.) where increases in the spectral ordinate may occur. Because of this, for structures that are characterized as having a fundamental period, T_F, greater than 1.0 sec., a site specific ground response analysis should be considered if liquefiable soils are determined to be present.
- For Site Class F soils, conduct a site specific ground response analysis. In previous guidance documents, the suggestion was made to use a Site Class E site coefficient for Site Class F soils. Use of F_{pga}, F_a and F_v from Site Class E for Site Class F soils appears to be overly conservative and is not recommended.

If a site specific analysis to establish a response spectrum that is lower than two-thirds of the specification based spectrum is approved by the State Geotechnical and Bridge Engineers, the site specific analysis should be independently peer reviewed by someone with expertise in the site specific ground response analysis technique used to conduct the analysis.

6.3.1 Determination of Seismic Hazard Level

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

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

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

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

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

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

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

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

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

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

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

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

For buildings, restrooms, shelters, and covered walkways, specification based seismic design parameters required by the 2006 IBC should be used. The seismic design requirements of the 2006 IBC are based on a risk level of 2 percent PE in 50 years. The 2 percent PE in 50 years risk level corresponds to the maximum considered earthquake (MCE). The 2006 IBC identifies procedures to develop a maximum considered earthquake acceleration response spectrum, at the ground surface by adjusting Site Class B spectra for local site conditions, similar to the methods used by AASHTO except that the probability of exceedance is lower (i.e., 2 percent in 50 years versus 7 percent in 75 years). However, the 2006 IBC defines the design response spectrum as two-thirds of the value of the maximum considered earthquake acceleration response spectrum. The site factors used in IBC 2006 are the same as used by AASHTO for modifying the Site Class B spectrum for local site effects. As is true for transportation structures, for critical or unique structures, for sites characterized as soil profile Type F (thick sequence of soft soils 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 **WSDOT GDM Appendix 6-A**.

Peak horizontal acceleration (%g) for 7% probability of exceedance in 75 years for Site Class B (adapted from AASHTO 2007).
Figure 6-8

Horizontal spectral acceleration at 0.2 second period (%g) for 7% probability of exceedance in 75 years with 5% of critical damping for Site Class B (adapted from AASHTO 2007).

Figure 6-9

Horizontal spectral acceleration at 1.0 second period (%g) for 7% probability of exceedance in 75 years with 5% of critical damping for Site Class B (adapted from AASHTO 2007).

Figure 6-10

6.3.2 Site Ground Motion Response Analysis

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

The AASHTO LRFD Bridge Design Specifications do not specifically require that a site specific seismic ground response analyses be completed for sites where liquefaction is anticipated during a design earthquake. The AASHTO Guide Specifications for LRFD Bridge Seismic Design require that the specification based ground motion spectral response for nonliquefied conditions be used unless a site specific ground motion response analysis is conducted. However, as discussed at the beginning of Section 6.3 herein, for structures with a fundamental period, T_F, greater than 1.0 sec., a site specific response analysis should be considered if the soils at the site are potentially liquefiable.

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

If a site specific ground motion response analysis is conducted, it shall be done in accordance with the AASHTO Guide Specifications for LRFD Bridge Seismic Design and **WSDOT GDM Appendix 6A**.

6.3.3 2006 IBC for Site Response

The 2006 IBC, Sections 1613 through 1615, provides procedures to estimate the earthquake loads for the design of buildings and similar structures. Earthquake loads per the 2006 IBC are defined by acceleration response spectra, which can be determined through the use of the 2006 IBC general response spectrum procedures or through site-specific procedures. The intent of the 2006 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 2006 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 2006 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 should 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 2006 IBC requires that a site-specific geotechnical investigation and dynamic site response analysis be completed, see **WSDOT GDM Appendix 6-A**. Dynamic site response analysis may not be required for liquefiable soil sites for structures with predominant periods of vibration less than 0.5 seconds.

6.3.4 Adjusting Ground Surface Acceleration to Other Site Classes

The site coefficient F_{pga} to account for the difference in ground response between Class B soil/rock conditions to other site classes with regard to the estimation of acceleration A_s are directly incorporated into the development of the standard response spectra for structural design of bridges and similar structures in the AASHTO LRFD Bridge Design Specifications and for the structural design of buildings and non-transportation related structures in the 2003 IBC. However, the PGA should also be multiplied by F_{pga} to account for the site class when assessing the potential for liquefaction and for the estimation of seismic earth pressures and inertial forces for retaining wall and slope design. For liquefaction assessment and retaining wall and slope design, the site coefficient presented in the AASHTO Guide Specifications for LRFD Bridge Seismic Design should be used, unless a site specific evaluation of ground response conducted in accordance with these AASHTO Guide specifications and **GDM Section 6.3** and **Appendix 6-A** is performed. Note that the site class should be determined considering the soils up to the ground surface, not just soil below the foundations.

6.3.5 Earthquake Magnitude

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

6.4 Seismic Geologic Hazards

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

6.4.1 Fault Rupture

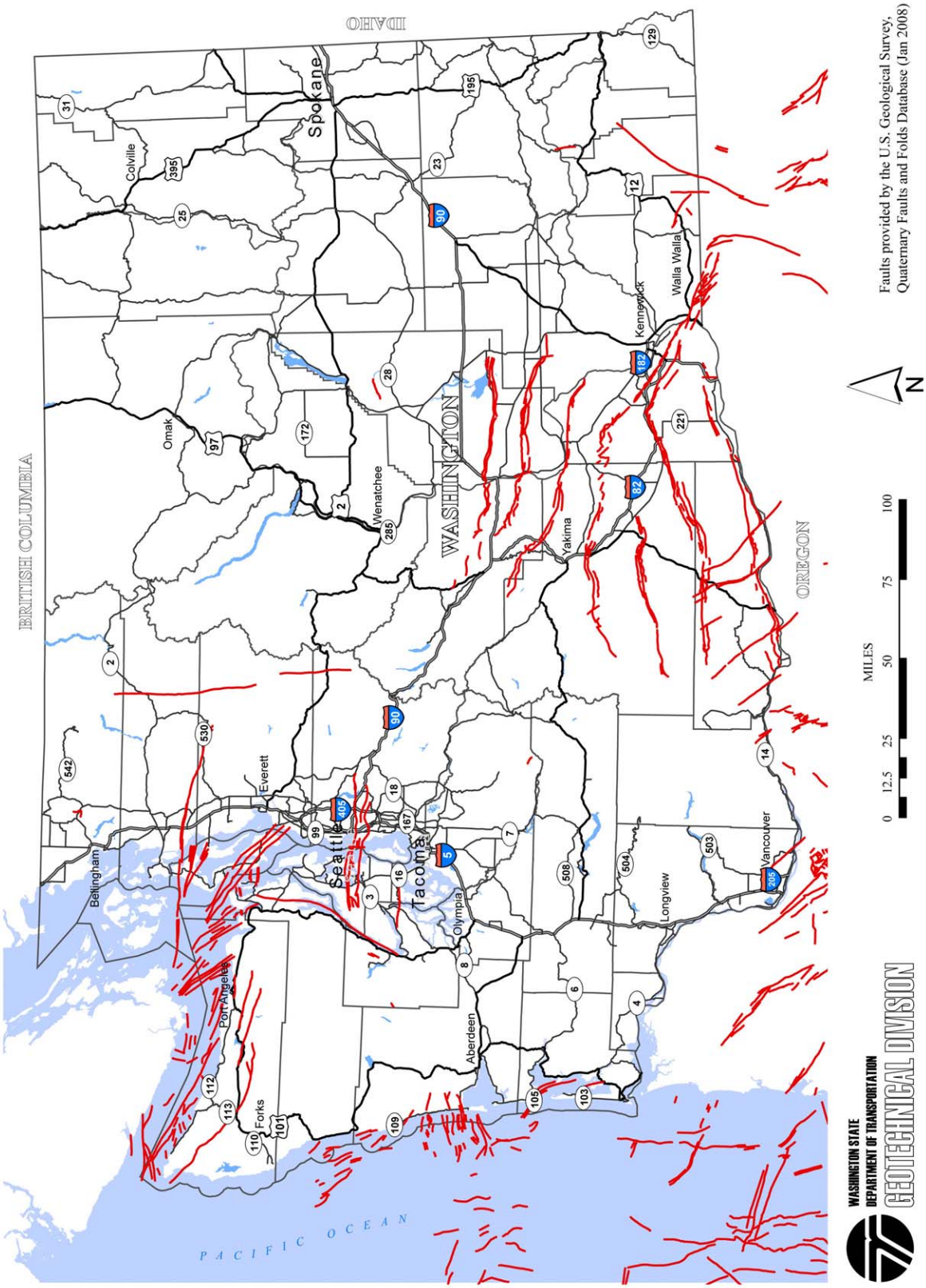
Washington State is recognized as a seismically active region; however, only a relatively small number of active faults have been identified within the state. Thick sequences of recent geologic deposits, heavy vegetation, and the limited amount of instrumentally recorded events on identified faults are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

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

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

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

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



Earthquake Faults in Washington State (adapted from USGS, 2002).
Figure 6-11

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 **WSDOT GDM 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
- Flow failures, lateral spreading, and slope instability.

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 failure, 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 should be assessed. If the soil liquefies, the stability is determined by the residual strength of the soil. The residual strength of liquefied soils can be estimated using empirical methods. Loss of lateral resistance can allow abutment soils to move laterally, resulting in bridge substructure distortion and unacceptable deformations and moments in the superstructure.
- **Reduced foundation bearing resistance**. The residual strength of liquefied soil is often a fraction of nonliquefied strength. This loss in strength can result in large displacements or bearing failure. For this reason spread footing foundations are not recommended where liquefiable soils exist unless the spread footing is located below the maximum depth of liquefaction or soil improvement techniques are used to mitigate the effects of liquefaction.
- **Reduced soil stiffness and loss of lateral support for deep foundations**. This loss in strength can change the lateral response characteristics of piles and shafts under lateral load.
- **Vertical ground settlement 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 should be assessed as specified in **WSDOT GDM Sections 6.5.3 and 8.12.2.7**, and in Article 3.11.8 in the AASHTO LRFD Bridge Design Specifications.

The effects of liquefaction will depend in large part on the amount of soil that liquefies and the location of the liquefied soil with respect to the foundation. On sloping ground, lateral flow, spreading, and slope instability can occur even on gentle slopes on relatively thin layers of liquefiable soils, whereas the effects of thin liquefied layer on the lateral response of piles or shafts (without lateral ground movement) may be negligible. Likewise, a thin liquefied layer at

the ground surface results in essentially no downdrag loads, whereas the same liquefied layer deeper in the soil profile could result in large downdrag loads. Given these potential variations, the site investigation techniques that can identify relatively thin layers are a fundamental part of the liquefaction assessment.

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

6.4.2.1 Methods to Evaluate Potential Susceptibility of Soil to Liquefaction

Evaluation of liquefaction potential should 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); however, these methods are not preferred and are used less frequently than SPT or CPT methods. V_s and BPT testing may be appropriate in soils difficult to test using SPT and CPT methods, such as gravelly soils. If the CPT method is used, SPT sampling and soil gradation testing shall still be conducted to obtain direct information on soil gradation parameters for liquefaction susceptibility assessment and to provide a comparison to CPT based analysis.

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

If a more refined analysis of liquefaction potential is needed, laboratory cyclic triaxial shear or cyclic simple shear testing may be used to evaluate liquefaction susceptibility and initiation in lieu of empirical soil gradation/PI/density criteria, in accordance with WSDOT **GDM 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 ft of the existing ground surface or proposed finished grade, which ever is lower.
- The subsurface profile is characterized in the upper 75 ft as having low plasticity silts or sands with a measured SPT resistance, corrected for overburden depth and hammer energy ($(N1)_{60}$), of 25 blows/ft, or a cone tip resistance q_{ciN} of 150, or a geologic unit is present at the site that has been observed to liquefy in past earthquakes. For low plasticity silts and clays, the soil is considered liquefiable as defined by the Bray and Sancio (2006) or Boulanger and Idriss (2006) criteria.

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

extent to impact bridge stability and A_s is greater than or equal to 0.15. These loose to very loose sands are likely to be present in hydraulically placed fills and alluvial or estuarine deposits near rivers and waterfronts.

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

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

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

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

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

6.4.2.2 Assessment of Liquefaction Potential

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 should 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.

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

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

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

$$CSR = 0.65 \frac{A_{max}}{g} \frac{\sigma_o}{\sigma_o'} \frac{r_d}{MSF} \quad \textbf{(6-8)}$$

Where

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

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

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

$$FS_{liq} = CRR/CSR \quad \textbf{(6-9)}$$

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 **WSDOT GDM Chapter 3**. In addition:

- Correction factors for borehole diameter, rod length, hammer type, and sampler liners should 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 **WSDOT GDM 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 ft. 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 ft 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 **WSDOT GDM Section 6.1.2.2**.

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

Nonlinear Effective Stress Methods. An alternative to the simplified procedures for evaluating liquefaction susceptibility is to complete a nonlinear, effective stress site response analysis utilizing a computer code capable of modeling pore water pressure generation and dissipation, such as D-MOD2000 (Matasović, et al., 2007). 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 ft, 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 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 ft. This limitation is partly the result of the very few well documented sites with measurement of pore-water pressure measurements during liquefaction, either at shallow or deep depths. For this reason greater reliance must be placed on observed response from laboratory testing or centrifuge modeling when developing the soil 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.

It should 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 and evaluate the sensitivity of its results to any uncertain parameters or modeling assumptions. 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.

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 (2008). Performance based techniques can be accomplished using the WSLIQ software (Kramer, 2008). The hazard level used for this analysis shall be consistent with the hazard level selected for the structure for which the liquefaction analysis is being conducted (typically, a probability of exceedance of 7 percent in 75 years in accordance with the AASHTO Guide Specifications for LRFD Seismic Bridge Design). Liquefaction hazards to be assessed include settlement and related effects, and liquefaction induced instability (e.g., flow failure or lateral spreading), and the effects of liquefaction on foundations.

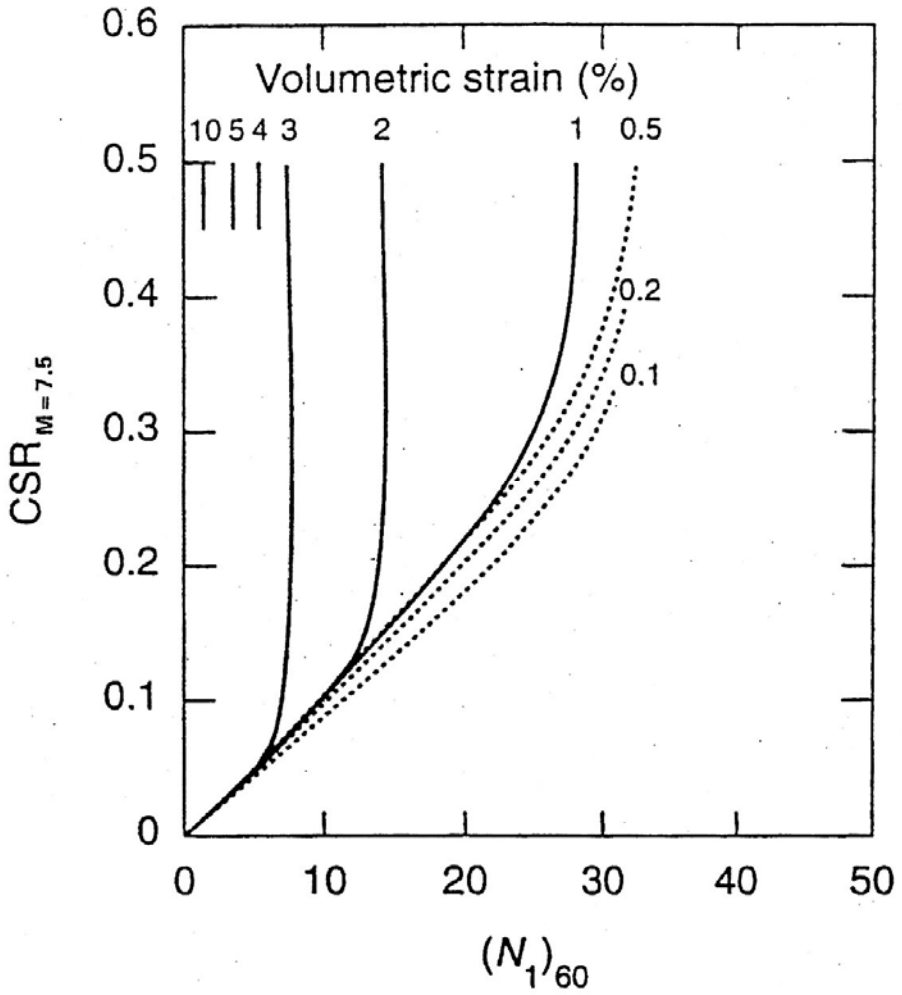
6.4.2.4 Liquefaction Induced Settlement

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

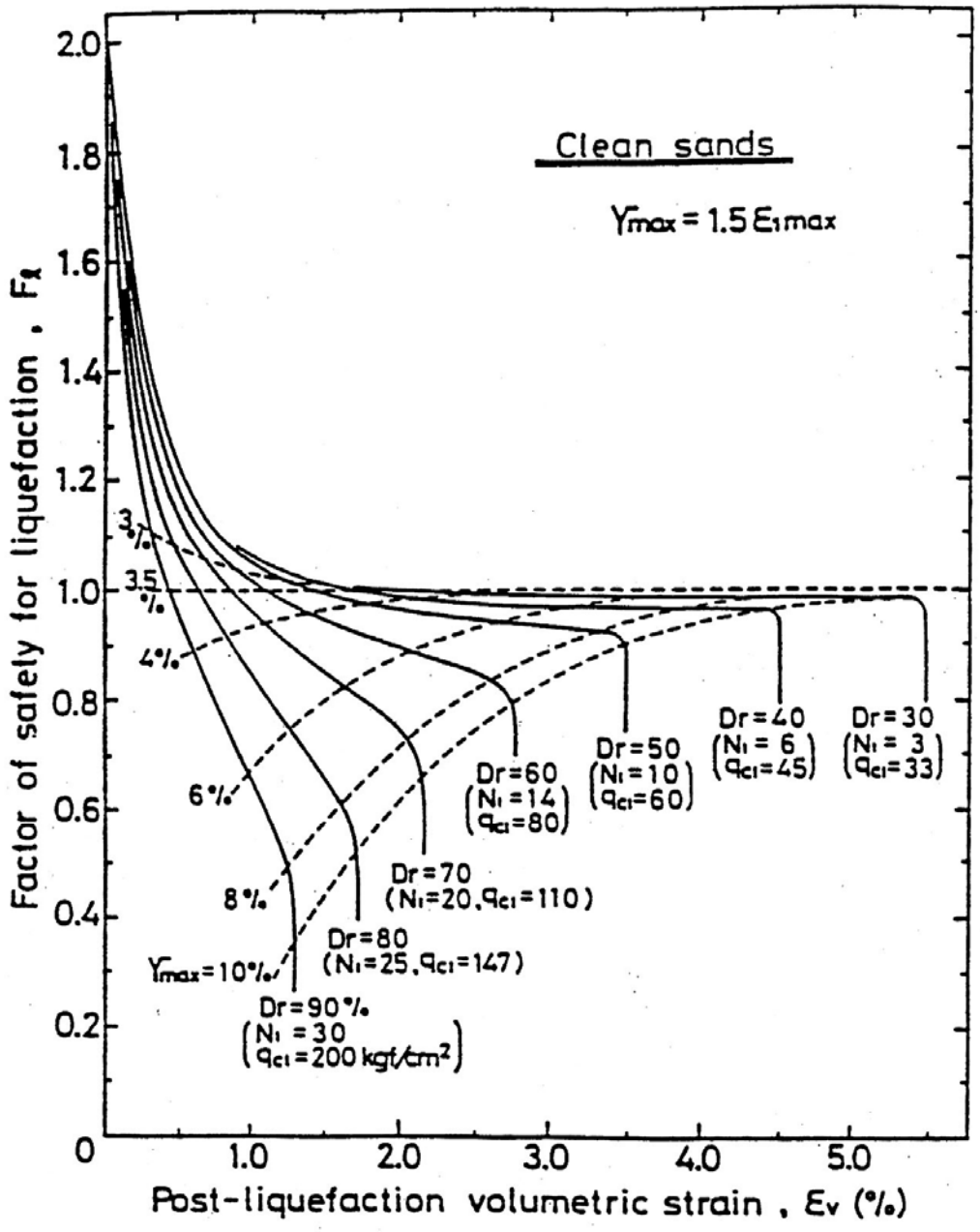
If the Simplified Procedure is used to evaluate liquefaction potential, liquefaction induced ground settlement of saturated granular deposits should be estimated using the procedures by Tokimatsu and Seed (1987) or Ishihara and Yoshimine (1992). The Tokimatsu and Seed (1987) procedure estimates the volumetric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

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

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



Liquefaction induced settlement estimated using the Tokimatsu & Seed procedure (Tokimatsu and Seed, 1987).
Figure 6-12



Liquefaction induced settlement estimated using the Ishihara and Yoshimine procedure (Ishihara and Yoshimine, 1992)

Figure 6-13

6.4.2.5 Residual Strength Parameters

Liquefaction induced instability is strongly influenced by the residual strength of the liquefied soil. Instability occurs when the shear stresses required to maintain equilibrium exceed the residual strength of the soil deposit. Evaluation of residual strength of a liquefied soil deposit is one of the most difficult problems in geotechnical practice (Kramer, 1996). A variety of empirical methods are available to estimate the residual strength of liquefied soils. The empirical relationships provided in Figures 6-4 through 6-7 and Table 6-3 should 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 accounted for in design and evaluation calculations.

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

The number of cycles, and either the cyclic stress ratios (stress-controlled testing) or cyclic shear strain (strain-controlled testing) used during the cyclic testing to liquefy or to attempt to liquefy the soil, should cover the range of the number of cycles and cyclic loading anticipated for the earthquake/ground motion being modeled. Testing to more than one stress or strain ratio should be done to fully capture the range of stress or strain ratios that could occur. Preliminary calculations or computer analyses to estimate the likely cyclic stresses and/or strains anticipated should be conducted to help provide a basis for selection of the cyclic loading levels to be used

for the testing. The vertical confining stress should be consistent with the in-situ vertical effective stress estimated at the location where the soil sample was obtained. Therefore K_{o}-consolidation is required in triaxial tests.

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

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

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

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

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

To obtain the liquefied residual strength, after the cyclic shearing is completed, the drain lines in the test should be left closed, and the sample sheared statically. If the test results are to be used with simplified ground motion modeling techniques (e.g., specification based ground response analysis or total stress site specific ground motion analysis), residual strength should be measured only for fully liquefied conditions. If effective stress ground motion analysis (e.g., D-MOD) is conducted, residual shear strength testing should be conducted at the cyclic stress

ratio and number of loading cycles predicted by the effective stress analysis for the earthquake being modeled at the location in the soil profile being modeled, whether or not that combination results in a fully liquefied state.

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

6.4.2.7 Weakening Instability Due to Liquefaction

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 residual value, two types of slope instability can occur: flow failure, and lateral spreading.

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 **WSDOT GDM 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 1.0, 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 1.0 for liquefied conditions, deformations should be estimated using a lateral spreading analysis (see the subsection “Lateral Spreading,” below, 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 **WSDOT GDM 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 **WSDOT GDM 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.

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.

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 1.0 or greater, a lateral spreading/deformation analysis shall be conducted. This analysis 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 should be evaluated using empirical relationships such as the procedure of Youd et al. (2002) or Kramer (2008). The Youd, et al. and Kramer procedures use empirical relationships based on case histories of lateral spreading. 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.

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.

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

Simplified charts based on Newmark-type analyses (see **WSDOT GDM Section 6.4.3.2**) may also 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, these simplified, empirically modified Newmark analyses may not be directly applicable to lateral spreading, as they were not developed for soil that weakens during earthquake shaking, as is the case for soil liquefaction. Therefore, as is true of Newmark time history analyses, these simplified Newmark based charts should be used cautiously, especially with regard to the selection of a yield acceleration to be used to enter these design charts or the equations upon which they are based.

There is no consensus on the best approach to use to estimate lateral spreading deformations. If the more rigorous approaches are used, the empirically based analyses shall still be conducted to provide a baseline of comparison, to qualitatively check the reasonableness of the estimates from the more rigorous procedures, using the more rigorous approaches to evaluate the effect of various input parameters on deformation. See Youd, et al. (2002), Kramer (1996, 2008), Seed, et al. (2003), and Dickenson, et al. (2002) for additional discussion on the assessment of slope deformations resulting from lateral spreading.

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

6.4.2.8 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) and/or very loose soils, liquefaction may be triggered early in the motion, and the structure may be subjected to strong shaking while the soil is in a liquefied state. In this case, coupled estimation of the inertial demands and liquefaction-induced deformation demands should be considered.

With regard to flow failure prediction, even though there is a possibility that seismic inertial forces may be concurrent with the liquefied conditions, 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 should be conducted without slope and structure seismic inertial forces (i.e., k_h and k_v are set equal to zero).

With regard to lateral spreading prediction, the timing of the onset of liquefaction relative to the timing and magnitude of the soil inertial forces caused by the earthquake ground motion is at least indirectly considered if the empirical methods (e.g., Youd, et al. 2002, Kramer 1996, 2008 and Seed, et al. 2003), or the simplified Newmark analyses described in **WSDOT GDM Section 6.4.3.2** (e.g., Bray and Travasarou, 2007), are used. These empirical methods to estimate displacements caused by lateral spreading should be considered approximate.

There is currently no consensus on how to specifically address this issue of timing of seismic acceleration and the development ofinitial 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 should be used to review the specific methods used, the development of the input data, how the methods are applied, and the resulting impacts.

Since seismic inertial forces are acting on the soil during the development of lateral spreading, 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 affect 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 any 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, if the foundation is located near the toe of the lateral spread, and if minimal plastic hinging of the column is anticipated.

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.

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

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

6.4.3 Slope Instability Due to Inertial Effects

Slope instability can occur during earthquakes due to inertial effects associated with ground accelerations. 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.

Slope instability can also be initiated during a seismic event due to the weakening of the soil caused by earthquake shaking induced soil strain. For example, sensitive fine grained soils can loose strength due to earthquake shaking and the soil deformation it causes. For earthquake induced slope instability, with or without soil strength loss resulting from deformation induced by earthquake shaking, the target factor of safety or resistance factor are as specified in **WSDOT GDM Section 6.4.3.1**. However, if liquefaction is the cause of the slope instability, it shall be evaluated as specified in **WSDOT GDM Section 6.4.2.7**.

6.4.3.1 Pseudo-Static Analysis

Pseudo-static slope stability analyses should 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 **WSDOT GDM 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 on pseudo-static analysis.

A horizontal pseudo-static coefficient, k_h, of $0.5A_s$ and a vertical pseudo-static coefficient, k_v, equal to zero should be used when seismic stability of slopes is evaluated, not considering liquefaction. For these conditions, the target factor of safety 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 the use of a reduced acceleration coefficient considers the ability of the slope to displace laterally based on a Newmark type analysis, thereby reducing the acceleration that can be experienced by the slope failure mass. See Anderson, et al. (2008) for additional guidance on this issue. For soils that exhibit a significant drop in strength to a residual value, the soil strength loss caused by the deformation should be considered in the slope design. In this case, the slope stability should be evaluated using residual shear strengths but without seismic inertial forces, as the drop in shear strength will not be complete until after seismic shaking is complete.

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 of $0.5A_s$ 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 $0.5A_s$ used for the stability assessment due to wave scattering effects. See Anderson, et al. (2008) for guidance on how to address this issue.

6.4.3.2 Deformations

Deformation analyses should be employed where an estimate of the magnitude of seismically induced slope deformation is required, and the slope stability factor of safety under the peak horizontal seismic coefficient (A_s) is less than 1.0. Acceptable methods of estimating the magnitude of seismically induced slope deformation include Newmark sliding block (time history) analysis, simplified displacement charts and equations based on Newmark-type analyses (Saygili and Rathje, 2008; and Rathje and Saygili, 2008; Bray and Travasarou, 2007), or dynamic stress-deformation models. These methods should not be employed to estimate displacements if the post earthquake shaking (i.e., assume k_h and k_v are zero) slope stability factor of safety is less than 1.0, as the slope will be unstable during gravity (static) loading.

More detailed descriptions of these analysis procedures are provided as follows:

Newmark Time History Analysis. Newmark (1965) proposed a seismic slope stability analysis that provides an estimate of seismically induced slope deformation. The advantage of the Newmark analysis over pseudo-static analysis is that it provides an index of permanent deformation. The term "index" is used to indicate that the estimate of deformation has a number of simplifying assumptions, and these assumptions limit the precision to which the displacement estimate can be made. For example, estimates of deformation to less than the nearest inch and perhaps the nearest several inches are very difficult to justify.

The Newmark time history analysis treats the unstable soil mass as a rigid block on an inclined plane. The procedure for the Newmark time history analysis consists of three steps that can generally be described as follows:

- Identify the yield acceleration of the slope by completing limit equilibrium stability analyses. The yield acceleration is the horizontal pseudo-static coefficient, k_h, required to bring the factor of safety to unity. Note that wave scattering effects may need to be considered (see **WSDOT GDM Section 6.4.3.1**) to insure consistency between the critical pseudo-static failure surface and the assumed pseudo-static seismic coefficient used.
- Select earthquake time histories representative of the design earthquake as described in **WSDOT GDM Appendix 6-A**. A minimum of three time histories representative of the predominant earthquake source zone(s) should be selected for this analysis. These time histories should be response spectrum compatible and may need to be propagated through the soil column to the base of the sliding soil mass to adjust for local site effects. For time histories representing distinctly different source zones (e.g., shallow crustal versus subduction zone), the time histories may be spectrally matched or scaled to an appropriate source-specific spectra as discussed in **WSDOT GDM Section 6.3.1** and **Appendix 6-A**. However, the different source-specific spectra in aggregate should envelope the design (target) spectrum.
- Double integrate all relative accelerations (i.e., the difference between acceleration and yield acceleration) while velocities are not zero in the earthquake time histories.

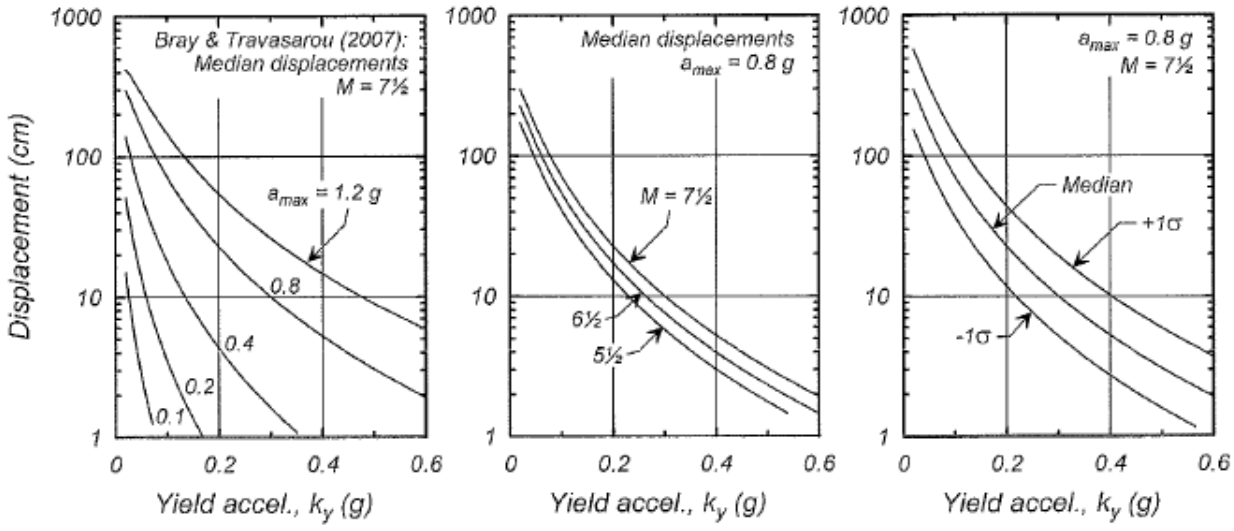
See Kramer (1996) for additional details regarding the Newmark time history analysis and its application.

A number of commercially available computer programs are available to complete Newmark analysis, such as Shake 2000 (Ordoñez, 2000) or Java Program for using Newmark Method and Simplified Decoupled Analysis to Model Slope Deformation During Earthquakes (Jibson and Jibson, 2003).

Newmark Based Displacement Charts. Bray and Rathje (1998) developed an approach to estimate permanent base sliding deformation for solid waste landfills. The method was based on the Newmark sliding block model, and is similar to the Makdisi and Seed (1978) approach. Although the charts were developed for solid waste landfills, the methodology is suitable for use at soil and rock sites, since the deformation relationship estimate is based on the ratio of yield acceleration to peak ground acceleration, and this ratio is relatively independent of the material type. The Bray-Rathje charts are based on significantly more analyses and a wider range of earthquake magnitudes, peak ground accelerations and frequency content than the Makdisi-Seed charts, and may be more reliable.

Recent work by Saygili and Rathje (2008) and Rathje and Saygili (2008) have extended the work by Bray and Rathje (1998) to provide equations for estimating displacements from seismic loading. Both deterministic and probabilistic methods are discussed.

A Bray-Travasarou chart, as developed by Idriss and Boulanger (2008) based on the method developed by Bray and Travasarou (2007), depicting permanent base deformation as a function of yield acceleration (K_y) acting on the slide mass is presented in Figure 6-14. See Bray and Travasarou (2007) and Idriss and Boulanger (2008) for additional discussion regarding the determination of deformation from this method. These more recent methods (i.e., Bray and Travasarou 2007; Saygili and Rathje 2008; and Rathje and Saygili 2008) should be used if Newmark based displacement charts are used.



Permanent Base Sliding Block Displacements as a Function of Yield Acceleration (after Idriss and Boulanger, 2008).
Figure 6-14

Dynamic Stress-Deformation Models. Seismically induced slope deformations 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. 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. As the quality of the constitutive models used in dynamic stress-deformation models improves, the accuracy of these methods will improve. A key benefit of these models is their ability to illustrate mechanisms of deformation, which can provide useful insight into the proper input for simplified analyses.

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. Use of dynamic stress-deformation models for design on WSDOT projects shall be approved by the State Geotechnical Engineer.

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 presented in Section 8.5 of Geotechnical Engineering Circular No. 3 (Kavazanjian, et al., 1997).

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:

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

where:

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

The maximum dynamic shear modulus is associated with small shear strains (typically less than 0.0001 percent). As the seismic ground motion level increases, the shear strain level increases, and dynamic shear modulus decreases. If the specification based general procedure described in **WSDOT GDM 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 for convenience. Note that $S_{XS}/2.5$ in the table is essentially equivalent to A_s (i.e., $PGAxF_{pga}$). This table reflects the dependence of G on both the shear strain induced by the ground motion and on the soil type (i.e., G drops off more rapidly as shear strain increases for softer or looser soils).

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

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

Notes: Use straight-line interpolation for intermediate values of $S_{xs}/2.5$.
* Site-specific geotechnical investigation and dynamic site response analyses shall be performed.

Effective Shear Modulus Ratio (G/G_0)
Table 4-7

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 **WSDOT GDM 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

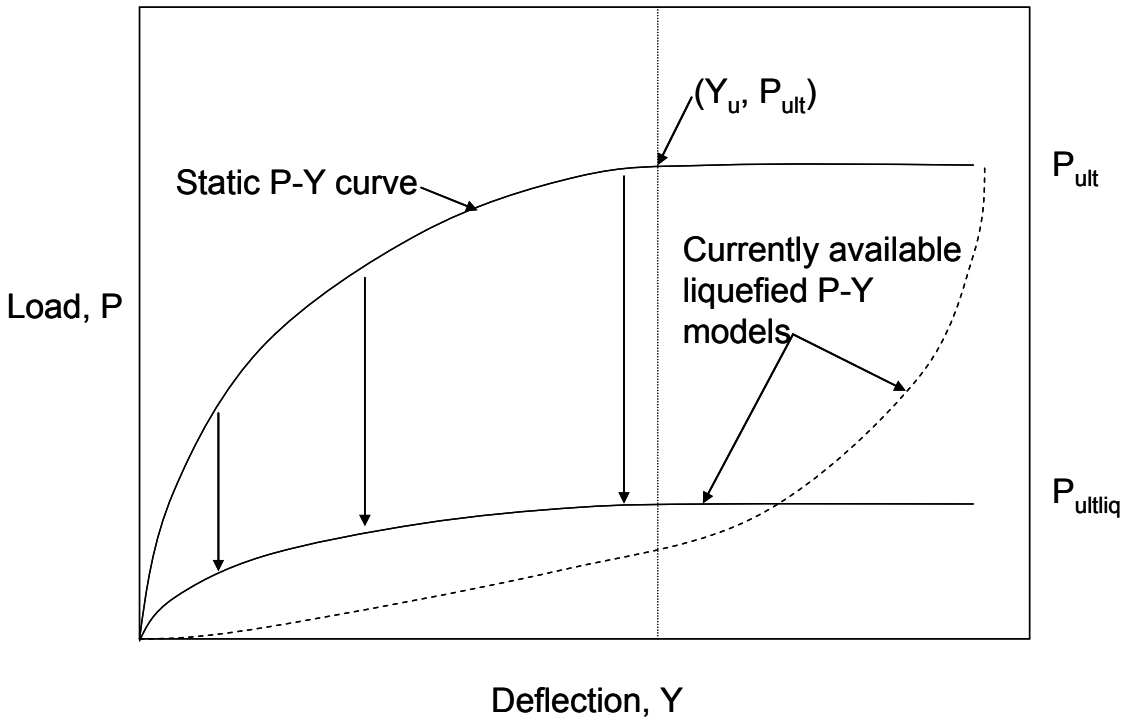
Lateral soil springs for deep foundations shall be determined in accordance with **WSDOT GDM Chapter 8**.

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

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

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

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



Conceptual P-Y curve model for liquefied conditions.
Figure 6-15

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

1. Use the static sand model and the P-multiplier approach as provided by Brandenberg, et al. (2007b) and Boulanger, et al. (2003) to reduce P_{ult} calculated for the static P-Y curve to a liquefied value. This approach is illustrated conceptually in Figure 6-15. The p-multiplier, m_p, used to reduce the static curve to a liquefied curve is determined from Figure 6-16. The p-multiplier approach is primarily applicable to use in L-Pile or a similar computer program.
2. Use the static sand model, using the residual strength and the overburden stress at the depth at which the residual strength was calculated to estimate a reduced soil friction value. The reduced soil friction angle is calculated using the inverse tangent of the residual undrained shear strength divided by the effective vertical stress at which the residual shear strength was determined or measured , i.e., $\phi_{reduced} = \tan^{-1}(S_r/\sigma'_{vo})$, where S_r is the residual shear strength and σ'_{v0} is the effective vertical stress. Use the reduced soil friction angle (i.e., for liquefied conditions) to generate the liquefied P-Y curves. This approach is applicable to both the strain wedge (DFSAP computer program) and L-Pile computer program methods. The entire static curve needs to be reduced from static to liquefied conditions, as illustrated in Figure 6-15. Parameters representing the initial stiffness of the P-Y curves may also need to be reduced in a manner similar to the reduction applied to obtain P_{ultliq}. For the DFSAP computer program, this adjustment to liquefied conditions would be applied to E_{50}. For L-Pile, this adjustment would be applied to the modulus of subgrade reaction, k. For both approaches, the soil unit weight should not be adjusted for liquefied conditions.

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

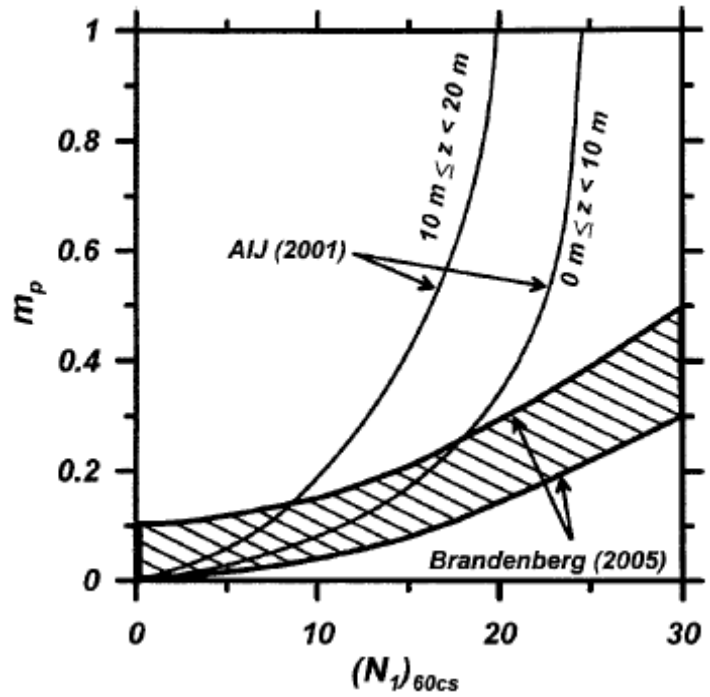
The p-multiplier values represent fully liquefied conditions. Note that for partially liquefied conditions, the p-multipliers can be increased from those values shown in the table, linearly interpolating between the tabulated values and 1.0 based on the pore pressure ratio, r_u, achieved during shaking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

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

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

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

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



Recommended p-multipliers for liquefied soil (after Brandenberg, et al., 2007b).
Figure 6-16

6.5.2 Earthquake Induced Earth Pressures on Retaining Structures

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

6.5.3 Downdrag Loads on Structures

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

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

6.5.4 Lateral Spread / Slope Failure Loads on Structures

In general, there are two different approaches to estimate the lateral spread/slope failure induced load on deep foundations systems—a displacement based method and a force based method. Displacement based methods are more prevalent in the United States. The 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 loads on deep foundation systems is presented in Boulanger, et al. (2003) and Brandenberg, et al. (2007a and b). The general procedure is as follows:

Deep foundations in liquefied, lateral spreading ground can be designed to resist lateral forces imposed on the pile by the lateral spreading ground. L-Pile or similar computer programs can be used to perform this analysis by incorporating the design steps below. The design steps that consider the kinematic loading from the liquefaction-induced lateral spreading ground are as follows (Boulanger et al. 2007a and b):

1. Estimate the free-field ground surface displacements caused by lateral spreading. Free-field soil displacement assumes no influence from the individual piles or pile group. Free-field soil displacements shall be estimated as specified in **WSDOT GDM Sections 6.4.2.7** and **6.4.3.2**. The free field displacement and its distribution (Step 2 below) is used as input into a lateral load analysis program such as L-Pile in combination with soil stiffness properties (Steps 3 and 4) to estimate load in the foundation.
2. Estimate the distribution of the lateral spreading displacement as a function of depth. Options include:
    a. **The shear strain profile approach described in Zhang et al. (2004) and illustrated by Idriss and Boulanger (2008).** The shear strain profile is estimated from SPT or CPT based liquefaction analyses to provide the distribution of lateral spreading displacement versus depth above a non-liquefied layer. The maximum shear strain is obtained using the

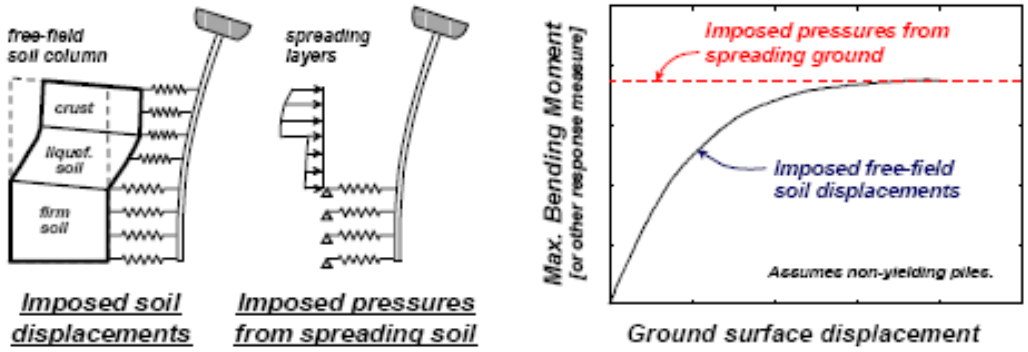
relationship between maximum shear strain, CSR, and the corrected SPT blowcount or normalized corrected CPT tip resistance. The lateral spreading displacement is obtained by integration of maximum shear strain for each depth increment. However, for cases of deep liquefaction the maximum depth of lateral spreading would have to be assumed. This maximum depth should consider the site topography and height of the free face. A lateral spread is unlikely to develop for depths greater than twice the height of a free face or a maximum of 50 ft.

b. **Non-linear effective stress dynamic finite-difference time history analyses.** Several input time histories would likely be run through the soil profile and a representative distribution of lateral spreading with depth would have to be selected from these outputs.

c. **A generic simplified soil profile that assumes constant ground surface displacement in a non-liquefied crust and a linear variation with depth across liquefied layers.** The constant ground surface displacement would be based on step 1 above. The linear variation would extend from the bottom of the non-liquefiable crust to the bottom of the lateral spreading layer. According to Boulanger et al. (2007a and b) piles that provide satisfactory performance and lateral stiffness are relatively insensitive to the assumed displacement profile. For flexible foundations, a range of soil displacement profiles may have to be assumed to capture the range of bending moment, shear, and pile displacements versus depth.

3. Lateral resistance of the liquefied soil springs shall be estimated as specified in **WSDOT GDM Section 6.5.1.2** and as discussed below. For soil layers that liquefy, scale the ultimate static P-Y resistance (P_{ult}) to an ultimate liquefied resistance (P_{ultliq}) as described in **WSDOT GDM Section 6.5.1.2**. The ultimate liquefied resistance should also be no greater than $0.6\sigma'_{vo}$ *b for this analysis, where σ'_{vo} is the effective overburden stress before seismic loading and "b" is pile width (Boulanger et al 2003). The P-Y curves for the nonliquefied layers within the soil profile for this analysis should be determined using the static analysis procedures (see **WSDOT GDM Chapter 8**).

4. To estimate kinematic loading effects on deep foundations from lateral spreading, the above lateral spreading displacement versus depth profile should be input directly as a free field displacement (soil movement) into a lateral load analysis program such as L-Pile. The modified P-Y curves calculated as described above should be used to characterize the soil in this analysis. The computer lateral load analysis program will determine the net foundation movement considering the structural stiffness of the foundation and the lateral resistance of the soil in front of the foundation. Conceptually, this is illustrated in Figure 6-17.



Methods for imposing kinematic loads from laterally spreading ground (after Boulanger, et al., 2003).
Figure 6-17

The estimated induced loads are then checked against the ability of the foundation system to resist those loads. The ultimate foundation resistance is based in part on the resistance provided by the portion of the pile/shaft embedded in non-liquefiable soils below the lateral spread zone and the structural capacity of the pile/shaft. Large pile deformations may result in plastic hinges forming in the pile/shaft. If foundation resistance is greater than that applied by the lateral spreading soil, the soil will flow around the structure. If the potential load applied by the soil is greater than the foundation system resistance, the pile/shaft is likely to move in concert with the soil. Also, the passive pressure and side friction generated on the pile cap by the spreading soil needs to be considered in the total load applied to the foundation system. Since large-scale structural deformations may be difficult and costly to accommodate in design, mitigation of foundation subsoils will likely be required.

Similar approaches to those outlined above can be used to estimate loads that other types of slope failure may have on the bridge foundation system.

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. The pressures on pile foundations are simply specified as follows:

- The liquefied soil exerts a pressure equal to 30 percent of the total overburden pressure (lateral earth pressure coefficient of 0.30 applied to the total vertical stress).
- Non-liquefied crustal layers exert full passive pressure on the foundation system.

Data from simulated earthquake loading of model piles in liquefiable sands in centrifuge tests indicate that the Japanese Force Method is an adequate design method (Finn and Fujita, 2004).

Another force-based approach to estimate lateral spreading induced foundation loads is to use a limit equilibrium slope stability program to determine the load the foundation must resist to achieve a target safety factor of 1.1. This force is distributed over the foundation in the liquefiable zone as a uniform stress. This approach may be utilized to estimate the forces that

foundation elements must withstand if they are to act as shear elements stabilizing the slope. See **WSDOT GDM Section 6.5.3** for specific stability analysis procedures.

6.5.4.3 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 **WSDOT GDM 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 ft 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. **WSDOT GDM 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.

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

Drainage Enhancements: By improving the drainage properties of soils susceptible to liquefaction, it may be possible to prevent the build-up of excess pore water pressures, and thus liquefaction. However, drainage improvement is not considered adequately reliable by WSDOT

to prevent excess pore water pressure buildup due to liquefaction due to drainage path time for pore pressure to dissipate, and due to the potential for drainage structures to become clogged during installation and in service. In addition, with drainage enhancements some settlement is still likely. Therefore, drainage enhancements shall not be used as a means to mitigate liquefaction. However, drainage enhancements may provide some potential benefits with densification and reinforcement techniques such as stone columns.

6.6 References

AASHTO, 2007. *AASHTO LRFD Bridge Design Specifications*, Fourth Edition.

AASHTO, 2008, *AASHTO Guide Specifications for LRFD Seismic Bridge Design*, First Edition.

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

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

ATC-MCEER Joint Venture, 2001, *Recommended LRFD Guidelines for the Seismic Design of Highway Bridges, Parts I and II,* National Cooperative Highway Research Program, NCHRP Project 12-49, Washington DC.

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

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

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

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

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

Brandenberg, S. J., 2005. *Behavior of Pile Foundations in Liquefied and Laterally Spreading Ground*, Ph.D. dissertation, Univ. of California at Davis, Davis, CA.

Brandenberg, S.J., Boulanger, R.W., Kutter, B.L., and Chang, D., 2007a. "Liquefaction Induced Softening of Load Transfer Between Pile Groups and Laterally Spreading Crusts." *ASCE Journal of. Geotechnical and Geoenvironmental Enginerring*, Vol. 133, No. 1, pp. 91-103.

Brandenberg, S. J., Boulanger, R. W., Kutter, B. L., and Chang, D., 2007b, "Static Pushover Analyses of Pile Groups in Liquefied and Laterally Spreading Ground in Centrifuge Tests,"

ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 133, No. 9, pp. 1055-1066.

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

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

Bray, J. and Rathje, E., 1998. "Earthquake Induced Displacements of Solid Waste Landfills." *ASCE Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 124, pp. 242-253.

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

Bray, J. D., and Travasarou, T., 2007, Simplified Procedure for Estimating Earthquake-Induced Deviatoric Slope Displacements, *ASCE Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 133, No. 4, pp. 381-392.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Kavazanjian, E., Matasovic, N., Hadj-Hamou, T. and Sabatini, P.J., 1997. *Geotechnical Engineering Circular #3, Design Guidance: Geotechnical Earthquake Engineering for Highways, Volume I Design Principles*. Report No. FHWA-SA-97-077. U.S. Department of Transportation, Federal Highway Administration, Washington, D.C.

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

Kramer, S. L., 2008. *Evaluation of Liquefaction Hazards in Washington State*, Washington State Department of Transportation, Report WA-RD _____, pp.

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

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

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

Makdisi, F.I. and Seed, H.B., 1978. "Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations." *ASCE Journal of the Geotechnical Engineering Division*, Vol. 104, No. GT7, July, 1978, pp. 849-867.

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

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

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

Newmark, N.M., 1965. "Effects of Earthquakes on Dams and Embankments." *Geotechnique*, 15(2), pp. 139-160.

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

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

Rathje, E. M., and Saygili, G., 2008, "Probabilistic Seismic Hazard Analysis for the Sliding Displacement of Slopes: Scalar and Vector Approaches," *ASCE Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 134, No. 6, pp. 804-814.

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

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

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

Saygili, G., and Rathje, E. M., 2008, "Empirical Predictive Models for Earthquake-Induced Sliding Displacements of Slopes," *ASCE Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 134, No. 6, pp. 790-803.

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

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

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

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

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

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

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

Washington State Department of Transportation, 2008, *LRFD Bridge Design Manual*. Publication Number M23-50.

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

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

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

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

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

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

Appendix 6-A Site Specific Seismic Hazard and Site Response

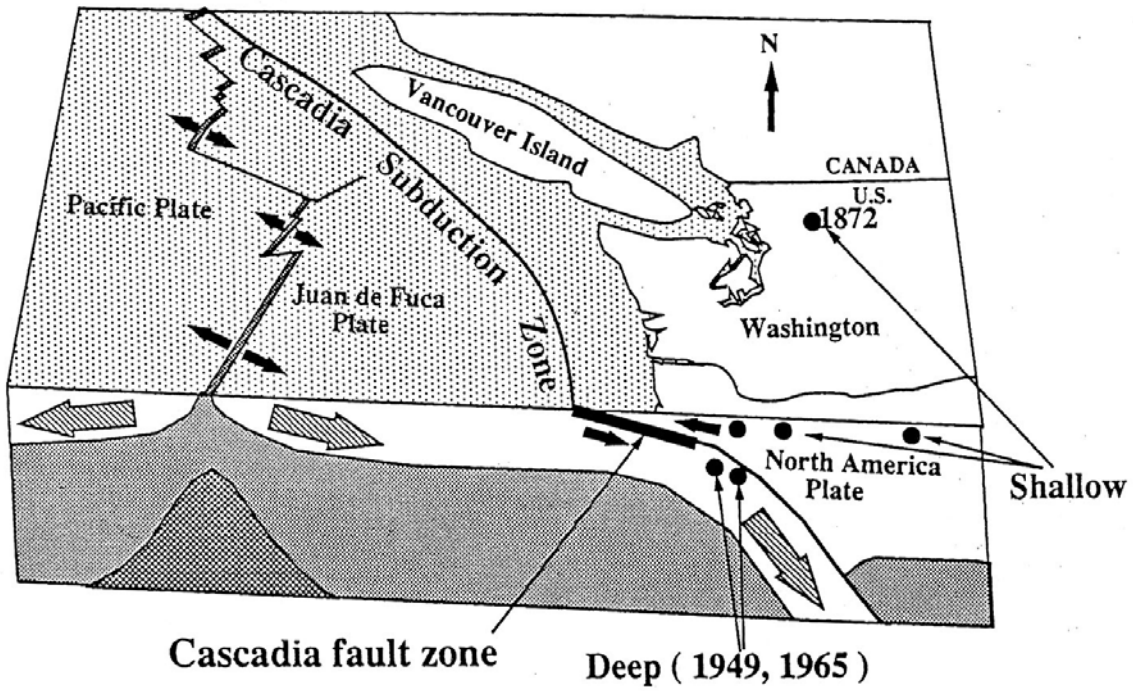
Site specific seismic hazard and response analyses shall be conducted in accordance with **WSDOT GDM Section 6.3** and the AASHTO Guide Specifications for LRFD Seismic Bridge Design. When site specific hazard characterization is conducted, it shall be conducted using the design risk levels specified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

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

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

6-A.1.1 Regional Tectonics

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



The three potential seismic source zones present in the Pacific Northwest (Yelin et al., 1994).
Figure 6-A-1

6-A.1.2 Seismic Source Zones

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

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

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

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

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

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

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

6-A.2 Design Earthquake Magnitude

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

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

- The design earthquake should consider hazard-compatible events occurring on crustal and subduction-related sources.
- More than one design earthquake may be appropriate depending upon the source zones thatcontribute to the site's seismic hazard and the impact that these earthquakes may have on site response.
- The design earthquake should be consistent with the design hazard level prescribed in **WSDOT GDM Section 6.3.1**.

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

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

1. 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 (2008). These techniques can be accomplished using the WSLIQ software (Kramer, 2008). 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).
2. If a single or a few larger magnitude earthquakes dominate the deaggregation, the magnitude of the single dominant earthquake or the mean of the few dominant earthquakes in the deaggregation should be used.
3. For routine design, a default moment magnitude of 7.0 should be used for western Washington and 6.0 for eastern Washington, except within 30 miles of the coast where Cascadia Subduction zone events contribute significantly to the seismic hazard. In that case, the geotechnical designer should use a moment magnitude of 8.0. These default magnitudes should not be used if they represent a smaller hazard than shown in the deaggregation data. Note that these default magnitudes are intended for use in simplified empirically based liquefaction and lateral spreading analysis only and should not be used for development of the design ground motion parameters.

6-A.3 Probabilistic and Deterministic Seismic Hazard Analyses

Probabilistic seismic hazard analysis (PSHA) and deterministic seismic hazard analysis (DSHA) can be completed to characterize the seismic hazard at a site. A DSHA consists of evaluating the seismic hazard at a site for an earthquake of a specific magnitude occurring at a specific location. A PSHA consists of completing numerous deterministic seismic hazard analyses for all feasible combinations of earthquake magnitude and source to site distance for each earthquake source zone. The result of a PSHA is a relationship of the mean annual rate of exceedance of the ground motion parameter of interest with each potential seismic source considered. Since the PSHA provides information on the aggregate risk from each potential source zone, it is more useful in characterizing the seismic hazard at a site if numerous potential sources could impact the site. The USGS 2002 probabilistic hazard maps on the USGS website and as published in AASHTO (2007) are based on PSHA.

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

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

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

- Cascadia subduction zone interpolate (interface) earthquake
- Cascadia subduction zone intraplate (Bennioff) earthquake
- Crustal earthquakes associated with non-specific or diffuse sources (potential sources follow). These sources will account for differing tectonic and seismic provinces and include seismic zones associated with Cascade volcanism
- Earthquakes on known and potentially active crustal faults. The following list of potential seismic sources may be used for hazard assessment and site response development. The applicability of these sources will depend on their proximity to the site.
  - Seattle Fault Zone
  - Southern Whidbey Island Fault
  - Utsalady Fault
  - Strawberry Point Fault
  - Devils Mountain Fault
  - Horse Heaven Hills Anticline
  - Rattlesnake-Wallula Fault System
  - Mill Creek Fault
  - Saddle Mountains Fault
  - Hite Fault System

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

- Overview of seismic sources considered in analysis
- Summary of seismic source parameters including length/boundaries, source type, slip rate, segmentation, maximum magnitude, recurrence models and relationships used, source depth and geometry. This summary should include the rationale behind selection of source parameters.
- Assumptions underlying the analysis should be summarized in either a table (DSHA) or in a logic tree (PSHA)

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

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

6-A.4 Selection of Attenuation Relationships

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

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

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

6-A.5 Site Specific Ground Response Analysis

6-A.5.1 Design/Computer Models

Site specific ground response analyses are most commonly done using one-dimensional equivalent-linear or non linear procedures. A one dimensional analysis is generally based on the assumption that soils and ground surface are laterally uniform and horizontal and that ground surface motions can be modeled by vertically propagating shear wave through laterally uniform soils. The influence of vertical motions, surface waves, laterally non-uniform soil conditions, incoherence and spatial variation of ground motions are not accounted for in conventional, one-dimensional analyses (Kavazanjian, et al., 1997). A variety of site response computer models are available to geotechnical designers for dynamic site response analyses. In general, there are three classes of dynamic ground response models: 1) one dimensional equivalent linear, 2) one dimensional nonlinear, and 3) multi-dimension models.

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

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

One-Dimensional Nonlinear Models. One-dimensional, nonlinear computer codes, such as D-MOD, or DESRA, use direct numerical integration of the 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.

Two and Three Dimensional Models. Two- and three-dimensional site response analyses can be performed using computer codes, such as QUAD4, PLAXIS, FLAC, and DYNAFLOW, and use both equivalent linear and nonlinear models. Many attributes of the two- and three-dimensional models are similar to those described above for the one-dimensional equivalent linear and nonlinear models. However, the two- and three-dimensional computer codes typically require significantly more model development and computational time than one-dimensional analyses. The important advantages of the two- and three-dimensional models include the ability to consider soil anisotropy, irregular soil stratigraphy, surface waves , irregular topography, and soil-structure interaction. Another advantage with the two- and three-dimensional models is that seismically induced permanent displacements can be estimated. Successful application of these codes requires considerable knowledge and experience. Expert review of the analysis is recommended.

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

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

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

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

A suite of ground motion time histories is required for both equivalent linear and nonlinear site response analyses as described in **WSDOT GDM 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 **WSDOT GDM Section 6-A.6**.

6-A.6 Analysis Using Acceleration-Time Histories

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

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

> *Earthquake acceleration time histories will be required for site-specific ground motion response evaluations and for nonlinear inelastic dynamic analysis of bridge structures. The time histories for these applications shall have characteristics that are representative of the seismic environment of the site and the local site conditions, including the response spectrum for the site.*
>
> *Response-spectrum-compatible time histories shall be developed from representative recorded earthquake motions. Analytical techniques used for spectrum matching shall be demonstrated to be capable of achieving seismologically realistic time series that are similar to the time series of the initial time histories selected for spectrum matching. The recorded time histories should be scaled to the approximate level of the design response spectrum in the period range of significance unless otherwise approved by the Owner. At least three response-spectrum-compatible time histories shall be used for representing the design earthquake (ground motions having 7 percent probability of exceedance in 75 years) when conducting dynamic ground motion response analyses or nonlinear inelastic modeling of bridges.*
>
> - *For site-specific ground motion response modeling single components of separate records shall be used in the response analysis. The target spectrum used to develop the time histories is defined at the base of the soil column. The target spectrum is obtained from the USGS/AASHTO Seismic Hazard Maps or from a site-specific hazard analysis as described in Article 3.4.3.1.*
> - *For nonlinear time history modeling of bridge structures, the target spectrum is usually located at or close to the ground surface, i.e., the rock spectrum has been modified for local site effects. Each component of motion shall be modeled. The issue of requiring all three orthogonal components (x, y, and z) of design motion to be input simultaneously shall be considered as a requirement when conducting a nonlinear time-history analysis. The design actions shall be taken as the maximum response calculated for the three ground motions in each principal direction.*
>
> *If a minimum of seven time histories are used for each component of motion, the design actions may be taken as the mean response calculated for each principal direction.*
>
> *For near-field sites (D < 6 miles) the recorded horizontal components of motion selected should represent a near-field condition and that they should be transformed into principal components before making them response-spectrum-*

compatible. The major principal component should then be used to represent motion in the fault-normal direction and the minor principal component should be used to represent motion in the fault-parallel direction.

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

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

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

Response spectrum matching approaches include methods in which time series adjustments are made in the time domain (Lilhanand and Tseng, 1988; Abrahamson, 1992) and those in which the adjustments are made in the frequency domain (Gasparini and Vanmarcke, 1976; Silva and Lee, 1987; Bolt and Gregor, 1993). Both of these approaches can be used to modify existing time-histories to achieve a close match to the design response spectrum while maintaining fairly

well the basic time-domain character of the recorded or simulated time-histories. To minimize changes to the time-domain characteristics, it is desirable that the overall shape of the spectrum of the recorded time-history not be greatly different from the shape of the design response spectrum and that the time-history initially be scaled so that its spectrum is at the approximate level of the design spectrum before spectrum matching.

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

- *Use of a higher scaling factor to meet the minimum aggregate match requirement for one component while exceeding it for the other two,*
- *Use of a scaling factor to meet the aggregate match for the most critical component with the match somewhat deficient for other components, and*
- *Compromising on the scaling by using different factors as required for different components of a time-history set.*

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

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

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

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

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

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

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

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

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