Chapter

6

Seismic Design

6.1 General

This chapter describes ODOT's standards and policies regarding the geotechnical aspects of the seismic design of ODOT projects. The purpose is to provide geotechnical engineers and engineering geologists with specific seismic design guidance and recommendations not found in other standard design documents used for ODOT projects. Complete design procedures (equations, charts, graphs, etc.) are usually not provided unless necessary to supply, or supplement, specific design information, or if they are different from standards described in other references. This chapter also describes what seismic recommendations should typically be provided by the geotechnical engineer in the Geotechnical Report.

6.1.1 Seismic Design Standards

The seismic design of ODOT bridges shall follow methods described in the most current edition (including the latest interims) of the "*AASHTO Guide Specifications for LRFD Seismic Bridge Design"* (AASHTO, 2011), the "*AASHTO LRFD Bridge Design Specifications" (AASHTO, 2014)*, the *"ODOT Bridge Design and Drafting Manual"* (BDDM) and the recommendations supplied in this chapter. Refer to the *ODOT BDDM* for additional design criteria and guidance regarding the use of the AASHTO Guide Specifications on bridge projects. The term "AASHTO" as used in this chapter refers to AASHTO LRFD design methodology. For seismic design of new buildings the requirements prescribed by the Oregon Structural Specialty Code (Oregon Building Codes Division, 2014), with reference to the International Building Code (International Code Council, 2012), shall be used. Unless otherwise noted, the standards and policies described in this chapter supersede those described in the referenced documents.

In addition to these standards, the following document should be referenced for additional design guidance in seismic design for issues and areas not addressed in detail in the AASHTO specifications or this chapter:

> "*LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations"*, Geotechnical Engineering Circular No. 3. (Kavazanjian, et al. 2011).

This FHWA document provides design guidance on earthquake engineering fundamentals, seismic hazard analysis, ground motion characterization, site characterization, seismic site response analysis, seismic slope stability, liquefaction analysis, and soil-foundation-structure interaction for use in the seismic design of structure foundations and retaining walls.

Additional reference documents for use in design are as follows:

- NCHRP Report 611 (Anderson et. al., 2008): *"Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes, and Embankments",* is a research project that developed analysis and design methods, and recommended load and resistance factor design (LRFD) specifications, for the seismic design of retaining walls, slopes, embankments, and buried structures. Example problems for the design of retaining walls, slopes and embankments, and buried structures using LRFD methods are included in the report.
- Report No. FHWA-NHI-11-075 (Kavazanjian et al, 2011): *"LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations, Design Examples"*, is a supplement document to GEC-3 document (NHI Course #13094) containing useful examples problems demonstrating the use of LRFD seismic design principals in practice.
- NCHRP Report 472 (ATC-MCEER Joint Venture, 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.
- Oregon Department of Transportation, **Seismic web page**

  This site provides the maps of 2014 USGS Probabilistic Seismic Hazard Analyses (PSHA) in the form of the Uniform Seismic Hazard, which reflects the contribution of all seismic sources in the region on the ground motion parameters. The ground motion parameters (Peak Ground Acceleration (PGA), and acceleration response spectral ordinates at 0.2 and 1.0 seconds for Site Class B rock for 500-year, 1,000-year return periods, specified as a percentage probability of exceedance in a given exposure interval, in years. This website also provides the seismic hazard maps for the Cascadia Subduction Zone Earthquake (CSZE).
- Report No. FHWA-NHI-11-030 (Marsh et. al., 2011): *"LRFD Seismic Analysis and Design of Bridges, Reference Manual"*, is the reference manual for a comprehensive NHI training course that addresses the requirements and recommendations of the seismic provisions in both the AASHTO LRFD Bridge Design Specifications and the AASHTO Guide Specifications for LRFD Seismic Bridge Design. Topics include force- and displacement-based design methodologies, the principles of capacity demand, methods for modeling and analyzing bridges subjected to earthquake motions, base isolation design and seismic retrofit strategies.
- Report No. FHWA-HRT-06-032 (Buckle et al., 2006): "Seismic Retrofitting Manual for Highway Structures: Part 1 – Bridges."
- United States Geological Survey; National Seismic Hazard Mapping Project.

  In the past the USGS National Seismic Hazard Maps website has been used for characterizing the seismic hazard for a specific site. However, in an effort to make the 2014 USGS National Seismic Hazard Maps static the maps will be hosted at a different location which is not known at this time.
- **WSDOT Geotechnical Design Manual, M46-03.11, 2015.**

The following two ODOT documents are available on the ODOT Geo-Environmental website for general reference. Note that aspects of the analyses procedures outlined in these archival documents have subsequently been updated and refined. The example problems included in these documents, demonstrating the application of selected seismic design procedures, are considered useful for general guidance; however, practitioners should make use of the most current procedures.

- "*Assessment and Mitigation of Liquefaction Hazards to Bridge Approach Embankments in Oregon*", Dickenson, S., et al., Oregon State University, Department of Civil, Construction and Environmental Engineering, SPR Project 361, November, 2002.
- "*Recommended Guidelines For Liquefaction Evaluations Using Ground Motions From Probabilistic Seismic Hazard Analysis*", Dickenson, S., Oregon State University, Department of Civil, Construction and Environmental Engineering, Report to ODOT, June, 2005.

6.1.2 Background

In light of the complexity of seismic design of transportation facilities, continuous enhancements to analytical and empirical methods of evaluation are being made as more field performance data is collected and research advances the state of knowledge. New methods of analysis and design are continuously being developed and therefore it is considered prudent to not be overly prescriptive in defining specific design methods for use in the seismic design process. However, a standard of practice needs to be established within the geotechnical community regarding minimum required design criteria for seismic design. It is well recognized that these standards are subject to change in the future as a result of further research and studies. This chapter will be updated as more information is obtained, new design codes are approved and better design methods become available.

Significant engineering judgment is required throughout the entire seismic design process. The recommendations provided herein assume the geotechnical designer has a sound education and background in basic earthquake engineering principles. These recommendations are not intended to be construed as complete or absolute. Each project is different and requires important decisions and judgments be made at key stages throughout the design process. The applicability of these recommended procedures should be continually evaluated throughout the design process. Peer review may be required to assist the design team in various aspects of the seismic hazard and earthquake-resistant design process.

Earthquakes often result in large axial and lateral loads being transferred from above ground structures into the structure foundations. At the same time, foundation soils may liquefy, resulting in a loss of soil strength and foundation capacity. Under this extreme event condition it is common practice to allow the foundations to be loaded up to the nominal (ultimate) foundation resistances (allowing resistance factors as high as 1.0). This design practice requires an increased emphasis on quality control during the construction of bridge foundations since we are now often relying on the full, un-factored nominal resistance of each foundation element to support the bridge during the design seismic event.

In addition to seismic foundation analysis, seismic structural design also involves an analysis of the soil-structure interaction between foundation materials and foundation structure elements. Soil-structure interaction is typically performed in bridge design by modeling the foundation elements using equivalent linear springs. Some of the recommendations presented herein relate to bridge foundation modeling requirements and the geotechnical information the structural designer needs in order to do this analysis. Refer to *Section 1.10.4* of the "*ODOT Bridge Design and Drafting Manual*" *(BDDM)* for more information on bridge foundation modeling procedures.

6.1.3 Responsibility of the Geotechnical Designer

The geotechnical designer is responsible for providing geotechnical/seismic recommendations and input parameters to the structural engineers for their use in design of the transportation infrastructure.

Specific elements to be addressed by the geotechnical designer include the following: design ground motion parameters, dynamic 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. Refer to Chapter 21 for geotechnical seismic design reporting requirements.

The seismic geologic hazards to be evaluated include fault rupture, liquefaction, ground failure including flow slides and lateral spreading, ground settlement, and instability of natural slopes and earth structures. The seismic performance of tunnels is a specialized area of geotechnical earthquake engineering not specifically addressed in this guidance document; however, the ground motion parameters determined in the seismic hazard analyses outlined herein may form the basis for tunnel stability analyses (e.g., rock fall adjacent to portals and in unlined tunnels, performance of tunnel lining). The risk associated with seismic geologic hazards shall be evaluated by the geotechnical designer following the methods described in this chapter.

6.2 Seismic Design Performance Requirements

6.2.1 New Bridges

Design new bridges on or West of US97 for a two-level seismic design criteria; Life Safety and Operational. Bridges east of US97 will be designed using the Life Safety seismic design criteria. Seismic Design Criteria for Life Safety and Operational performance are described below.

The ODOT Seismic website, listed below, should be referrenced to obtain the earthquake hazards and design tools associated with the Life Safety and Operational design criteria. **http://www.oregon.gov/ODOT/HWY/BRIDGE/Pages/seismic.aspx**

"Life-Safety" Design Criteria:

Under this level of shaking, the bridge and approach structures, foundation and approach fills must be able to withstand the design forces and displacements without collapse of any portion of the structure and also be consistent with the Life Safety seismic design criteria described below and in the current ODOT BDDM. In general, bridges that are properly designed and detailed for seismic loads can accommodate relatively large deflections without the danger of collapse.

If large embankment displacements (lateral spread) or overall slope failure of the end fills are predicted, the impacts on the bridge end bent, abutment walls and interior piers should be evaluated to see if the impacts could potentially result in collapse of any part of the structure. Slopes adjacent to a bridge or tunnel should be evaluated if their failure could result in collapse of a portion or all of the structure.

Report ground motions having an average return period of 1000 years (7% probability of exceedance in 75 years). Ground motion parameters shall be based on the 2014 USGS seismic hazard maps (Peterson, M.D., et. al., 2014). The probabilistic hazard maps for the 1,000-year and 500-year return periods are available at ODOT Seismic website listed above.

To aid in consistency and efficiency, Bridge Section has developed an excel application, ODOT_ARS.v. 2014.16, for constructing the probabilistic design response spectrum using the general procedure (three-point curve) for the 2014 data. ODOT_ARS.v. 2014.16 has been released to incorporate the updated site coefficients associated with the 2014 hazard maps. The necessary inputs to generate a three point response spectra include latitude, longitude, and site class. The tables below replace Tables 3.4.2.3-1, and 3.4.2.3-2 in the AASHTO Guide Specifications for LRFD Seismic Bridge Design.

Replace *AASHTO Guide Spec Table 3.4.2.3-1* with tables 6.2-A and 6.2-B:

Table 6.2-A

Values of Site Factor, F_{pga}, at Zero-Period on Acceleration Spectrum

Site Class	Mapped Peak Ground Acceleration Coefficient (PGA)[1]					
	PGA ≤ 0.1	PGA = 0.2	PGA = 0.3	PGA = 0.4	PGA = 0.5	PGA ≥ 0.6
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.9	0.9	0.9	0.9	0.9	0.9
C	1.3	1.2	1.2	1.2	1.2	1.2
D	1.6	1.4	1.3	1.2	1.1	1.1
E	2.4	1.9	1.6	1.4	1.2	1.1
F[2]	*	*	*	*	*	*

Table 6.2-B

Values of Site Factor, F_a, for Short-Period Range of Acceleration Spectrum

Site Class	Mapped Spectral Acceleration Coefficient at Period 0.2 sec (S_S)[1]					
	$S_S \le 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S = 1.25$	$S_S \ge 1.5$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.9	0.9	0.9	0.9	0.9	0.9
C	1.3	1.3	1.2	1.2	1.2	1.2
D	1.6	1.4	1.2	1.1	1.0	1.0
E[3]	2.4	1.7	1.3	*	*	*
F[2]	*	*	*	*	*	*

Replace *AASHTO Guide Spec Table 3.4.2.3-2* with following table:

Table 6.2-C

Values of Site Factor, F_v, for Long-Period Range of Acceleration Spectrum

Site Class	Mapped Spectral Response Acceleration Coefficient at Period 1.0 sec (S_1)[1]					
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 = 0.5$	$S_1 \geq 0.6$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.8	0.8	0.8	0.8	0.8	0.8
C	1.5	1.5	1.5	1.5	1.5	1.4
D[4]	2.4	2.2[4]	2.0[4]	1.9[4]	1.8[4]	1.7[4]
E[4]	4.2	3.3[4]	2.8[4]	2.4[4]	2.2[4]	2.0[4]
F[2]	*	*	*	*	*	*

Notes:

[1] – Use straight-line interpolation for intermediate values of PGA, S_S, or S_1.

[2] – Perform a site-specific geotechnical investigation and dynamic site response analysis for all sites in Site Class F.

[3] – Perform a ground motion hazard analysis for structures on Site Class E sites with S_S greater than or equal to 1.0.

[4] – Perform a ground motion hazard analysis for structures on Site Class D and E sites with S_1 greater than or equal to 0.2.

“Operational” Design Criteria:

In addition to the “Life Safety” performance design criteria, all bridges on and west of US Hwy 97 shall be designed to remain in service following a level of ground shaking associated with a full-rupture Cascadia Subduction Zone Earthquake (CSZE). Seismic hazard maps and spectral accelerations of CSZE have been developed based on the full-rupture CSZE event. A summary of this work is provided the 2016 final report to ODOT titled “*Impact of Cascadia Subduction Zone Earthquake on the Evaluation Criteria of Bridges”*.These maps are available on the **ODOT Seismic** web page. Also available on the web page, is a program developed by Portland State University (PSU) to generate a deterministic (eighteen points) response spectra. A link to PSU’s program is located on the ODOT Seismic web page and is titled ***CSZE_ARS*** .

For the Operational performance level, bridges and approach fills are designed to remain in service shortly after the event (after the bridge has been properly inspected) to provide access for emergency vehicles. Some structural damage is anticipated but the damage should be repairable and the bridge should be able to carry emergency vehicles immediately following the earthquake. This holds true for the approach fills leading up to the bridge.

Approach fill settlement and lateral displacements should be minimal to provide for immediate emergency vehicle access for at least one travel lane. For mitigation purposes approach fills are defined as shown in Figure 6-15. As a general rule of thumb, an estimated lateral embankment displacement of up to 1 foot is considered acceptable in many cases as long as the “operational” performance criteria described above can be met and the structure foundations are adequately designed to withstand the soil loads resulting from the lateral displacements. Vertical settlements on the order of 6” to 12” may be acceptable depending on the roadway geometry, anticipated performance of the bridge end panels and the ability of bridge foundation elements to withstand any imposed downdrag loads. Bridge end panels are required on all state highway bridge projects (per *BDDM*) and should be evaluated for their ability to withstand the anticipated embankment displacements and settlement and still provide the required level of performance. These displacement criteria are to serve as general guidelines only and engineering judgment is required to determine the final amounts of acceptable displacement that will meet the desired criteria. It should be noted that these estimated displacements are not at all precise values and may easily vary by factors of 2 to 3 depending on the analysis method(s) used. The amounts of allowable vertical and horizontal displacements should be decided on a case-by-case basis, based on discussions and consensus between the bridge designer and the geotechnical designer and other appropriate project personnel.

In addition to bridge and approach fill performance, embankments through which cut-and-cover tunnels are constructed should be designed to remain stable during the design seismic event because of the potential for damage or possible collapse of the structure should they fail.

Approach embankments and structure foundations should be designed to meet the above performance requirements. Unstable slopes such as active or potential landslides and other seismic hazards such as liquefaction, lateral spread, post-earthquake settlement and downdrag may require mitigation measures to ensure that the structure meets these performance requirements. Refer to Chapter 11 for guidance on approved ground improvement techniques to use in mitigating these hazards.

6.2.2 Bridge Widenings

For the case where an existing bridge is to be widened and new foundation support is required, the seismic foundation designs for the widened bridge should be designed using the same seismic design criteria as "New Bridges". Consult with the bridge designer to determine the design and performance requirements for all new foundations required for bridge widening projects and/or the need for any Phase 2 retrofit design work.

If Phase 2 foundation retrofit or liquefaction mitigation is necessary to meet the performance criteria, these designs shall be reviewed and approved by the HQ Bridge Section.

6.2.3 Bridge Abutments and Retaining Walls

Seismic design performance objectives for bridge abutments shall be consistent with the design requirements for the supported bridge. Seismic design performance objectives for retaining walls depend on the function of the retaining wall and the potential consequences of failure. There are four retaining wall categories, as defined in Chapter 15. The seismic design performance objectives for these four categories are listed below. Refer to AASHTO, (2014) Article 11.5.4 for seismic design requirements for retaining walls under the Extreme Event Limit State condition. The Extreme Event I "no analysis" provisions of AASHTO Section 11 shall not apply to "Bridge Abutment Walls" or "Bridge Retaining Walls".

- **Bridge Abutments**: Bridge Abutments are considered to be part of the bridge, and shall meet the seismic design performance objectives for the bridge see Section 6.2.1**.**
- **Bridge Retaining Walls**: Design all Bridge Retaining Walls for 1000-year return period ground motions under the "Life Safety" bridge criteria. Under this level of shaking, the Bridge Retaining Wall must be able to withstand seismic forces and displacements without failure of any part of the wall or collapse of any part of the bridge which it supports. Bridge Retaining Walls shall be designed for overall stability under these seismic loading conditions, including anticipated displacements associated with liquefaction. Mitigation to achieve overall stability may be required.

In addition, design all Bridge Retaining Walls for the ground motions described under the "Operational" bridge criteria. Under this level of shaking, Bridge Retaining Wall movement must not result in unacceptable performance of the bridge or bridge approach fill, as described under the "Operational" criteria in Section 6.2.1.

- **<u>Highway Retaining Walls</u>**: Highway Retaining Walls should be designed for 1000-year return period ground motions unless the "No Analysis" option, as described in Article 11.5.4 of AASHTO (2014), is applicable. Under this level of shaking, the Highway Retaining Wall must be able to withstand seismic forces and displacements without failure of any part of the Highway Retaining Wall. Highway Retaining Walls shall be designed for overall stability under these seismic loading conditions, including anticipated displacements associated with liquefaction. Mitigation to achieve overall stability may be required
- **<u>Minor Retaining Walls</u>**: Minor Retaining Wall systems have no seismic design requirements.

The policy to design all Highway Retaining Walls to meet overall stability requirements for seismic design may not be practical at all wall locations. Where it is not practical to design a Highway Retaining Wall for overall stability under seismic loading, and where a failure of this type would not endanger the public, impede emergency and response vehicles along essential lifelines, or have an adverse impact on another structure, the local Region Tech Center should evaluate practicable alternatives for improving the seismic resistance and performance of the retaining wall.

In general, retaining walls and bridge abutments should not be built on or near landslides or other areas that are marginally stable under static conditions. However, if site conditions, project constraints (cost), prohibit an effective technical alternative, the local Region Tech Center will evaluate, on a case-by-case basis, the possible placement of these structures in these locations, as well as requirements for global (overall) instability of the landslide during the design seismic event.

6.2.4 Bridge Approach Embankments, General Embankments and Cut Slopes

Bridge approach embankments should be evaluated for seismic slope stability and settlement in all areas where the ground surface acceleration coefficient (As) is ≥ 0.15g., especially if they are relied upon to provide passive soil resistance behind the abutment (Earthquake-Resisting System). Bridge approach embankments (with or without retaining walls) should be designed to meet the operational and life safety performance requirements described in Section 6.2.1 and in accordance with all other applicable sections of this chapter.

Cut slopes, fill slopes, and embankments that are not bridge approach embankments are generally not evaluated for seismic instability unless they directly affect a bridge, highway retaining wall or other structure. Seismic instability associated with routine cuts and fills are typically not mitigated due to the high cost of applying such a design policy uniformly to all slopes statewide. If failure and displacement of existing slopes, embankments or cut slopes, due to seismic loading, could adversely impact an adjacent structure or facility, these areas should be considered for stabilization. Such impacts should be evaluated in terms of meeting the performance criteria described in Section 6.2.

6.3 Ground Motion Parameters

The ground motion parameters for the Life Safety design criteria are based on the 2014 USGS National Seismic Hazard Mapping Project. These maps provide the results of probabilistic seismic hazard analysis (PSHA) at the regional scale. Ground motion maps and design parameters for the Life Safety (1000-year PSHA) design criteria are available on the ODOT Seismic web page. The designer should review the basis of these hazard maps and have a thorough understanding of the data they represent and the methods used for their development.

The USGS Open-File Report 2014-1091 (Petersen et al., 2014) should be referenced for important information on the development of these seismic hazard maps.

The seismic hazard maps on the ODOT Seismic web page provide Peak Ground Acceleration (PGA), 0.20 sec. and 1.0 sec. spectral accelerations scaled in contour intervals of 0.01g. The PGA and spectral accelerations can be obtained by entering the latitude and longitude of the site and the desired probability of exceedance (i.e., 7% in 75 years for the 1000 year return event). It should be noted that the PGA obtained from these maps is actually the Peak "Bedrock" Acceleration (i.e., Site Class B), and does not include, or take into account, any local soil amplification effects. See Section 6.5.1 for the development of design ground motion data.

The ground motion parameters for the Operational design criteria are based on the report titled "Impact of Cascadia Subduction Zone Earthquake on the Evaluation Criteria of Bridges" by Portland State University. The Operational design criteria maps are the result using three different full rupture locations and depths with associated moment magnitude values (Chen, Frankel and Peterson 2014) and four weighted ground motion prediction equations (Atkinson & Boore 2003, Atkinson & Macias 2009, Zhao et. al 2006, and BC Hydro 2012). The ground motion parameter maps for the CSZE scenario are available on the ODOT Seismic web page. The designer should review the basis of these hazard maps and have a thorough understanding of the data they represent and the methods used for their development.

6.3.1 Site Specific Probabilistic Seismic Hazard Analysis

Ground motion parameters are also sometimes determined from a site specific Probabilistic Seismic Hazard Analysis (PSHA). A site specific probabilistic hazard analysis focuses on the spatial and temporal occurrence of earthquakes, and evaluates all of the possible earthquake sources contributing to the seismic hazard at a site with the purpose of developing ground motion data consistent with a specified uniform hazard level. The analysis takes into account all seismic sources that may affect the site and quantifies the uncertainties associated with the seismic hazard, including the location of the source, extent and geometry, maximum earthquake magnitudes, rate of seismicity, and estimated ground-motion parameters. The result of the analysis is a uniform hazard acceleration response spectrum that is based on a specified uniform hazard level or probability of exceedance within a specified time period (i.e., 7% probability of exceedance in 75 years). The PSHA is usually performed to yield ground motion parameters for bedrock (Site Class B) sites. The influence of the soil deposits at the site on the ground motion characteristics is subsequently evaluated using the results of the PSHA for bedrock conditions. The bedrock response spectra developed from the probabilistic hazard analysis can also be used as the basis for matching or scaling time histories for use in a site-specific ground response analysis.

A site specific probabilistic hazard analysis is typically not performed on routine ODOT projects. If such an analysis is desired for the design of ODOT bridge projects the HQ Bridge Section must approve the justification and procedures for conducting the analysis and the analysis must be reviewed by an independent source approved by the HQ Bridge Section. Review and approval of all PSHAs will be coordinated with the region geotechnical engineer.

6.3.2 Magnitude and PGA for Liquefaction Analysis

Earthquake engineering evaluations that address repeated (cyclic) loading and failure of soils must include estimates of the intensity and duration of the earthquake motions. In soils, liquefaction and cyclic degradation of soil stiffness/strength represent fatigue failures that often impact bridge structures. In practice-oriented liquefaction analysis, the intensity of the cyclic loading is related to the PGA and/or cyclic stress ratio, and the duration of the motions is correlated to the magnitude of the causative event. The PGA and magnitude values selected for the analysis should represent realistic ground motions associated with specific, credible scenario earthquakes. The PGA values obtained from the USGS web site represent the “mean” values of all of the sources contributing to the hazard at the site for a particular recurrence interval. These “mean” PGA values should not typically be used for liquefaction analysis unless the ground motions at the site are dominated by a single source, as demonstrated in the PSHA deaggregation. Otherwise, the “mean” PGA values may not represent realistic ground motions resulting from known sources affecting the site. Additionally, the mean magnitude provided by PSHA should not be used as the causative event as this often averages the magnitude of large Cascadia Subduction Zone earthquakes and the magnitude of the smaller, local crustal events with a resulting magnitude that is not representative of any seismic source in the region. For this reason the modal event(s), designated as Magnitude and Distance (M-R) pairs, should typically be evaluated individually along with other M-R pairs that contribute significantly to the hazard.

6.3.3 Deaggregation of Seismic Hazard

For evaluation of the seismic hazard at sites using uniform hazard-based ground motions a deaggregation of the total seismic hazard should be performed to find the principal individual sources contributing to the seismic hazard at the site. The relative contribution of all considered sources, in terms of magnitude and distance, on PGA and on spectral accelerations can be readily evaluated using the results of the USGS seismic hazard mapping tools and deaggregation capabilities available through the USGS seismic hazard web site. In general, sources that contribute more than about 5% to the hazard should be considered for evaluation. However, sources that contribute less than 5% may also be sources to consider since they may still significantly affect the liquefaction analysis or influence portions of the site’s response spectra.

It is recommended that the relative contributions of all of the following sources be considered when performing liquefaction and ground deformation hazards:

1. Cascadia Subduction Zone – mega-thrust earthquakes,
2. Deep, Intraslab Benioff Zone earthquakes such as the 1949 and 1965 Puget Sound, and 2001 Nisqually earthquakes,
3. Shallow crustal earthquakes associated with mapped faults,
4. Regional background seismicity and ‘randomly” occurring earthquakes that are not associated with mapped faults (gridded seismicity).

A deaggregation of the seismic hazard will provide the mean and modal values of Magnitude (M) and Distance (R) and also a table of M-R pairs associated with each source contributing to the hazard at the site. The mean deaggregation provides the weighted mean values of M and R for all sources that contribute to the hazard. The modal value(s) yields the M and R pair(s) having the largest contribution in the hazard deaggregation of each grid location. The modal pairs represent the primary sources that should be considered in subsequent liquefaction and ground hazard analysis. For areas in the state where there are more than one significant seismic source the modal values are much more representative of the primary sources, and mean values of M and R are not recommended for use in liquefaction hazard analyses. In some areas of the state where the seismic hazard is derived mostly from a single primary source the mean values may be very representative of the site. In addition to consideration of mean and modal pairs, other individual M-R pairs listed in the deaggregation table that represent significant contributions to the hazard may be considered to supplement the modal (or mean) pairs. Sound engineering judgment is required throughout this process to decide which, if any, of these additional M-R pairs warrant consideration.

The M-R pairs selected from this process represent the primary sources and can then be utilized with ground motion prediction equations (GMPEs) to obtain bedrock PGA values at the site. It is recommended that more than one GMPE be used to estimate ground motion parameters for each of the primary seismic sources in Oregon (i.e., Cascadia Subduction Zone events, and shallow crustal events). The use of three to four GMPEs is common in practice.

In order to be consistent with the 2014 USGS seismic hazard maps, the same GMPEs and weighting factors that were used in developing the 2014 USGS seismic hazard maps would need to be used. Refer to the USGS Open-File Report 2014-1091 (Petersen et. al., 2014) for important information on how these GMPEs were used in developing the 2014 USGS Seismic Hazard maps.

The source distances for the subduction zone events reported from the USGS deaggregation web site are the closest distances to the fault or slab (R_{rup}).

There are various definitions of the source-to-site distance to faults, depending on the GMPE selected. The source-to-site distance used in any given prediction calculation should be consistent with the source-to-site distance definition described in the documentation for that particular GMPE.

Figure 6.1 depicts most of the typical distance definitions used in these prediction equations.

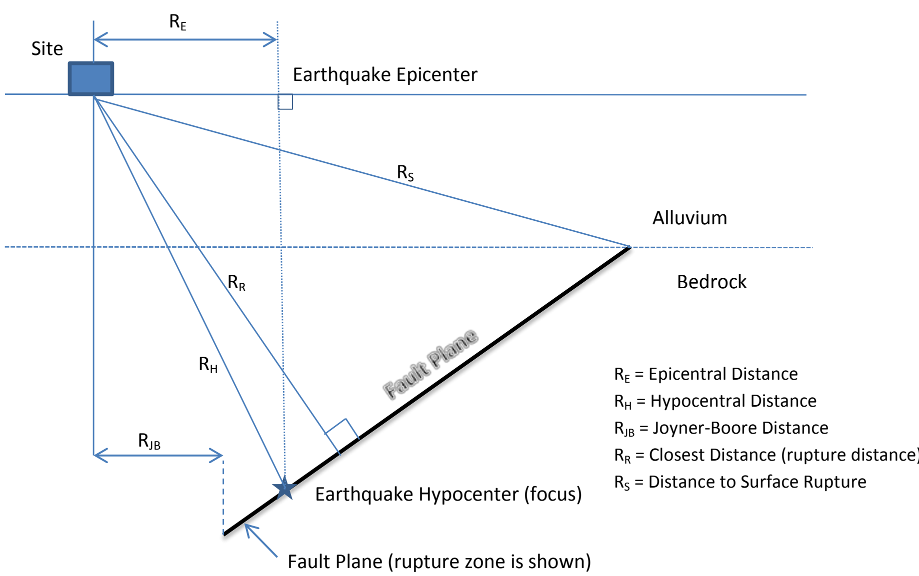


Figure 6.1: Typical Source to Site distance definitions

It is important to note that the ground motion values (PGA, $S_{0.2}$, $S_{1.0}$) obtained for the primary M-R pairs obtained in this fashion will not likely be the same as the "mean" values developed for the Uniform Seismic Hazard (USH), which are used as the basis for structural analysis. Also, it is likely that the average value of a specific ground motion parameter obtained for the principal M-R pairs will also vary from the mean value provided by the USGS USH. The difference will reflect the number M-R pairs considered and the relative contributions of the sources to the overall hazard.

This deaggregation process will likely yield more than one M-R pair, and therefore more than one magnitude and peak ground acceleration, for liquefaction analysis in some areas of the state where the hazard is dominated by two or more seismic sources. In most of western Oregon, this will include both shallow crustal sources and the Cascadia Subduction Zone. In this case, each M-R (i.e., M-PGA) pair should be evaluated individually in a liquefaction analysis. If liquefaction is estimated for any given M-PGA pair, the evaluation of that pair is continued through the slope stability and lateral deformation evaluation processes.

In some areas in the state where the seismic hazard is dominated by a single source, such as the Cascadia Subduction Zone along parts of the Oregon coast, a single pair of M-R values (largest magnitude (M) and closest distance (R)) may be appropriate for defining and assessing the worst case liquefaction condition. In this area of the state, where the seismic hazard is dominated by the CSZ, the PGA calculated from the M-R pair for the 1000-yr return event (Life Safety criteria) may be roughly equivalent to the PGA obtained from the deterministic CSZ hazard maps, used for the Operational performance level. In that case the larger PGA value of the two should be used in the liquefaction (and subsequent) analysis for both the Life Safety evaluations.

Refer to Dickenson (2005), for a practice-oriented approach for incorporating deaggregation results into liquefaction hazard assessment. A simplified approach applying the results of the deaggregation process, and examples for several locations in Oregon, is provided. This document is provided as an example and not intended to be a standard procedure or guideline.

A recommended procedure for estimating lateral embankment deformations is also included in this document, along with a flow chart describing the overall process for the evaluation of liquefaction hazard and ground deformation at bridge sites. This flow chart is provided in Appendix 6-A.

6.4 Site Characterization for Seismic Design

The geotechnical site investigation should identify and characterize the subsurface conditions and all geologic hazards that may affect the seismic analysis and design of the proposed structures or features. The goal of the site characterization for seismic design is to develop the subsurface profile and soil property information needed for seismic analyses. The geotechnical designer should review and discuss the project objectives with the project engineering geologist and the structural designer, 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 potential geologic hazards, areas of concern (e.g., deep soft soils or liquefiable soils), and potential variability of local geology,
- Identify engineering analyses to be performed (e.g., ground response analysis, liquefaction susceptibility, lateral spreading/slope stability assessments, seismic-induced settlement/ downdrag, dynamic earth pressures),
- Identify engineering properties required for these analyses,
- Determine methods to obtain the required design parameters and assess the validity of such methods for the soil and rock material types.

Develop an integrated investigation of in-situ testing, soil sampling, and laboratory testing. This includes determining the number of tests/samples needed and appropriate locations to obtain them.

6.4.1 Subsurface Investigation for Seismic Design

Refer to Section 6.0 of AASHTO, 2014, for guidance regarding subsurface investigation and site characterization for seismic foundation design. With the possible exception of geophysical explorations associated with obtaining seismic shear wave velocities in soil and rock units, the subsurface data required for seismic design is typically obtained concurrently with the data required for static design of the project (i.e., additional exploration for seismic design over and above what is required for foundation design is typically not necessary). However, the exploration program may need to be adjusted to obtain the necessary parameters for seismic design. For example, the use of the seismic cone penetration test, SCPT, is recommended in order to supplement tip resistance and friction data with shear wave velocity. Also, for Site Class determination, subsurface investigations must extend to a depth of at least 100 feet unless bedrock is encountered before reaching that depth.

The selection of field drilling equipment and sampling methods will reflect the goals of the investigation. If liquefaction potential is a significant issue, mud rotary drilling with SPT sampling, combined with seismic piezocone penetrometer testing, are the preferred methods of investigation. The SPT methods described in ASTM D6066-11 should be used in addition to those described in ASTM D1586-11, to obtain the best quality SPT results for use in liquefiable soils. While mud-rotary drilling methods are preferred, hollow-stem auger (HSA) drilling may be utilized for SPT sampling and testing if precautionary measures are taken. Soil heaving and disturbance in HSA borings can lead to unreliable SPT “N” values. Therefore care must be taken if using HSA methods to maintain an adequate water head in the boring at all times and to use drilling techniques that minimize soil disturbance. Non-standard samplers shall not be used to collect data used in liquefaction analysis and mitigation design.

In addition to standard subsurface investigation methods, the following equipment calibration, soil testing, and/or sampling should be considered depending upon site conditions.

- **SPT Hammer Energy:** This value (usually termed hammer efficiency) should be noted on the boring logs or in the Geotechnical Report. The hammer efficiency should be obtained from the hammer manufacturer, preferably through field testing of the hammer system used to conduct the test. This is needed to determine the hammer energy correction factor, C_{er}, for liquefaction analysis.
- **Soil Samples for Gradation Testing:** Used for determining the amount (percentage) of fines in the soil for liquefaction analysis. Also useful for scour estimates.
- **Undisturbed Samples:** Laboratory testing for parameters such as S_u, $e_{50,}$ E, G, OCR, and other parameters for both foundation modeling and seismic design.
- **Shear Wave Velocity Measurements:** For use in determining soil Site Class. Also used to develop a shear wave velocity profile of the soil column and to obtain low strain shear modulus values to use in analyses such as dynamic soil response.
- **Seismic Piezocone Penetrometer:** For use in determining soil Site Class. Also used to develop a shear wave velocity profile and obtain low strain shear modulus values to use in a ground response analysis.
- **Piezocone Penetrometer Test:** Used for liquefaction analysis and is even preferred in some locations due to potential difficulties in obtaining good quality SPT results. Pore pressure measurements and other parameters can be obtained for use in foundation design and modeling. Also useful in establishing the pre-construction subsurface soil

conditions prior to conducting ground improvement techniques and the post-construction condition after ground improvement.

- **Depth to Bedrock:** If a ground response analysis is to be performed, the depth to bedrock must be known or reasonably estimated based on local data. "Bedrock" material for this purpose is defined as a material unit with a shear wave velocity of at least 2500 ft./sec.
- **Pressuremeter Testing:** For development of p-y curves if soils cannot be adequately characterized using the default relationships supplied in the LPile, GROUP, DFSAP or other soil-structure interaction programs. Testing is typically performed in soft clays, organic soils, very soft or decomposed rock and for unusual soil or rock materials. The shear modulus, G, for shallow foundation modeling and design can also be obtained.

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

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

Geotechnical Issues	Engineering Evaluations	Required Information For Analyses	Field Testing	Laboratory Testing
Site Response	• source characterization and attenuation • site response spectra • time history	• subsurface profile (soil, groundwater, depth to rock) • shear wave velocity • bulk shear modulus for low strains • relationship of shear modulus with increasing shear strain • equivalent viscous damping ratio with increasing shear strain • Poisson’s ratio • unit weight • relative density • seismicity (PGA, design earthquakes)	• SPT • CPT • seismic one • geophysical testing (shear wave velocity) • piezometer	• cyclic triaxial tests • Atterberg Limits • specific gravity • moisture content • unit weight • resonant column • cyclic direct simple shear test • torsional simple shear test
Geologic Hazards Evaluation (e.g. liquefaction, lateral spreading, slope stability)	• liquefaction susceptibility • liquefaction induced settlement • settlement of dry sands • lateral spreading • 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) • site topography	• SPT • CPT • seismic cone • Becker penetration test • vane shear test • piezometers • geophysical testing (shear wave velocity)	• soil shear tests • triaxial tests (including cyclic) • grain size distribution • Atterberg Limits • specific gravity • organic content • moisture content • unit weight

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

Geotechnical Issues	Engineering Evaluations	Required Information For Analyses	Field Testing	Laboratory Testing
Input for Structural Design	• shallow foundation 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	• subsurface profile (soil, groundwater, rock) • shear strength (peak and residual) • 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) • site topography	• CPT • SPT • seismic cone • piezometers • geophysical testing (shear wave velocity) • vane shear test	• triaxial tests • soil shear tests • unconfined compression • grain size distribution • Atterberg Limits • specific gravity • moisture content • unit weight • resonant column • cyclic direct simple shear test • torsional simple shear test

For analysis and design of standard bridges, in-situ or laboratory testing for parameters such as the dynamic shear modulus at small strains, equivalent viscous damping, shear modulus and damping ratio versus shear strain, and residual shear strength are generally not directly obtained. Instead, index properties and correlations based on in-situ field measurements (such as the SPT and CPT) are generally 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, the following correlations are recommended. Other acceptable correlations can be found in Wair et al. (2012), Dickenson et al. (2002), Kramer (1996), Mayne (2007) and other technical references. Region and site-specific correlations developed by practitioners are acceptable with adequate supporting documentation and approval by ODOT. The use of multiple, applicable correlations, followed by weighted averaging of the computed soil parameter, is recommended. Figures 6-2, 6-3 and 6-4 are provided as examples for shear modulus reduction and damping curves for soil types typically encountered. The formulations presented by Darendeli (2001) are also acceptable for use in developing shear modulus reduction and damping curves. Other alternative correlations may be necessary for unusual soils conditions such as organic soils (peats), diatomaceous soils, sawdust or highly weathered rock.

- Table 6-2, which presents correlations for estimating initial shear modulus (G_{max}) based on relative density, penetration resistance, void ratio, OCR or cone resistance.

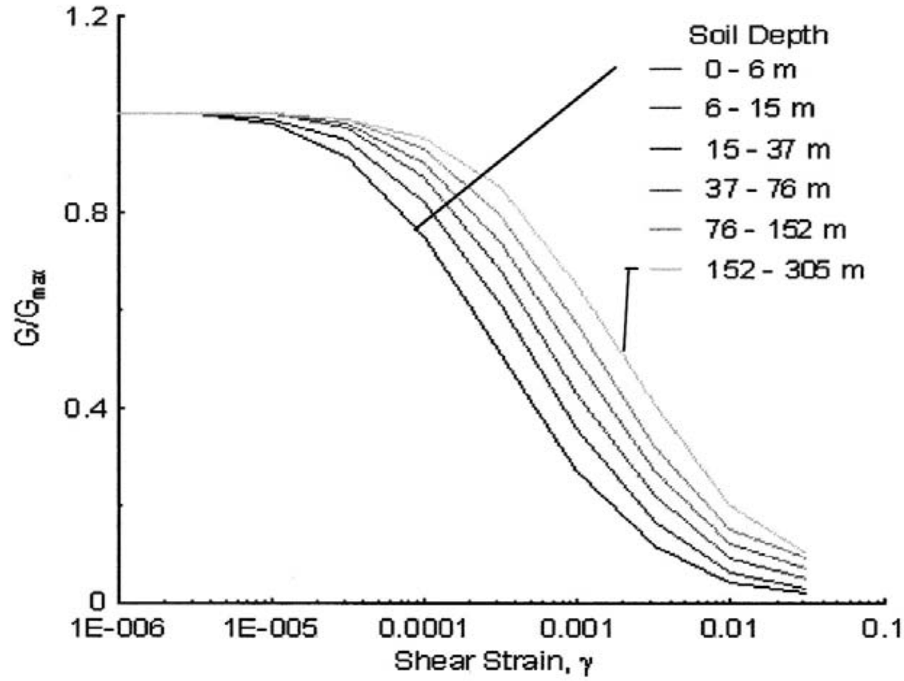
- Figure 6-2, which presents shear modulus reduction curves and equivalent viscous damping ratio for cohesionless soils (sands) as a function of shear strain and depth.
- Figure 6-3 and Figure 6-4, which present shear modulus reduction curves and equivalent viscous damping ratio, respectively, as a function of cyclic shear strain and plasticity index for fine grained (cohesive) soils.
- Figure 6-5, Figure 6-6, Figure 6-7 and Figure 6-8 which presents charts for estimating undrained residual shear strength for liquefied soils as a function of SPT blow counts (N'_{60}), CPT (q_{cl}) and vertical effective stress.

Table 6-2. Correlations for Estimating Initial Shear Modulus (SCDOT, 2010).

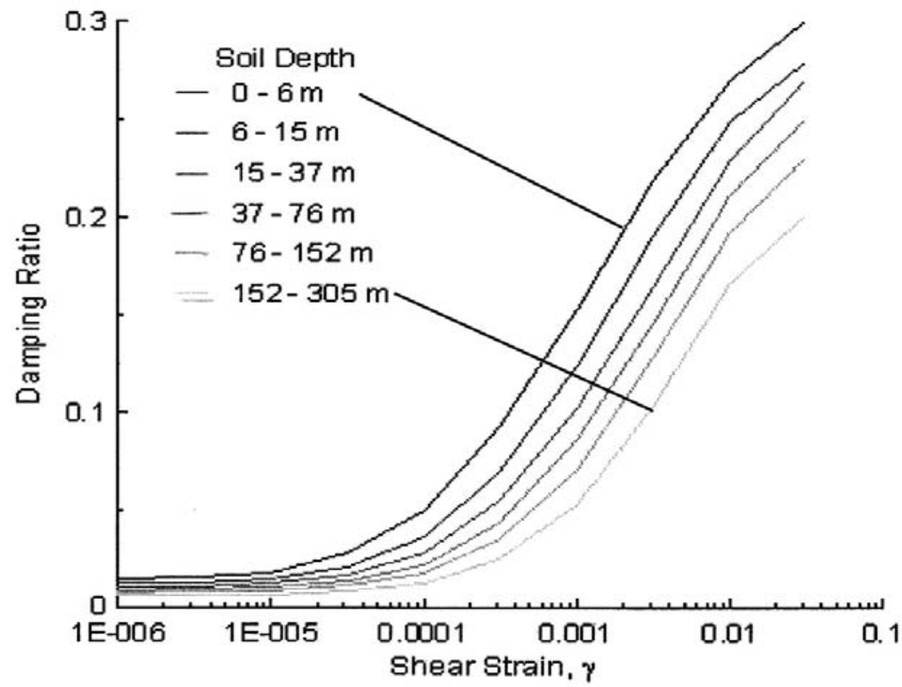
Reference	Correlation Equation	Units	Comments
Seed, et al. (1984)	$G_{max} = 220(K_2)_{max}(\sigma'_m)^{0.5}$ $(K_2)_{max} \approx 20(N_1)_{60}^{1/3}$	kPa	$(K_2)_{max} \approx$ 30 for loose sands and 75 for very dense sands; ≈80-180 for dense well graded gravels; Limited to cohesionless soils
Imai and Tonouchi (1982)	$G_{max} = 15{,}560(N_{60})^{0.68}$	kPa	Limited to cohesionless soils
Hardin (1978)	$G_{max} = \frac{625}{(0.3+0.7e_o^2)}(P_a\sigma'_m)^{0.5}OCR^k$	kPa [(1)]	Limited to cohesive soils P_a = atmospheric pressure P_a and σ'_m in kPa
Jamiolkowski, et al. (1991)	$G_{max} = \frac{625}{e_o^{1.3}}(P_a\sigma'_m)^{0.5}OCR^k$	kPa [(1)]	Limited to cohesive soils P_a and σ'_m in kPa
Mayne and Rix (1993)	$G_{max} = 99.5(P_a)^{0.305}\frac{(q_c)^{0.695}}{(e_o)^{1.13}}$	kPa	Limited to cohesive soils P_a and q_c in kPa

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

PI	k	PI	k
0	0.00	60	0.41
20	0.18	80	0.48
40	0.30	>100	0.50



Shear Modulus Reduction Curves



Damping Ratio Curves

Figure 6-2. Shear modulus reduction and damping ratio curves for sand (EPRI, 1993).

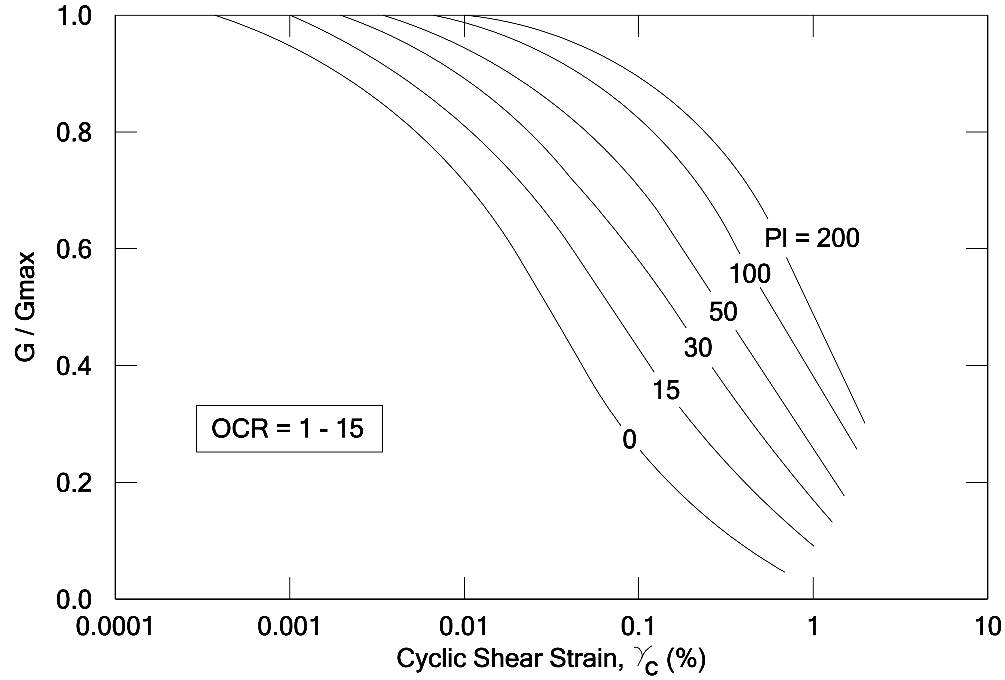


Figure 6-3. Variation of G/Gmax vs. cyclic shear strain for fine grained soils (redrafted from Vucetic and Dobry, 1991).

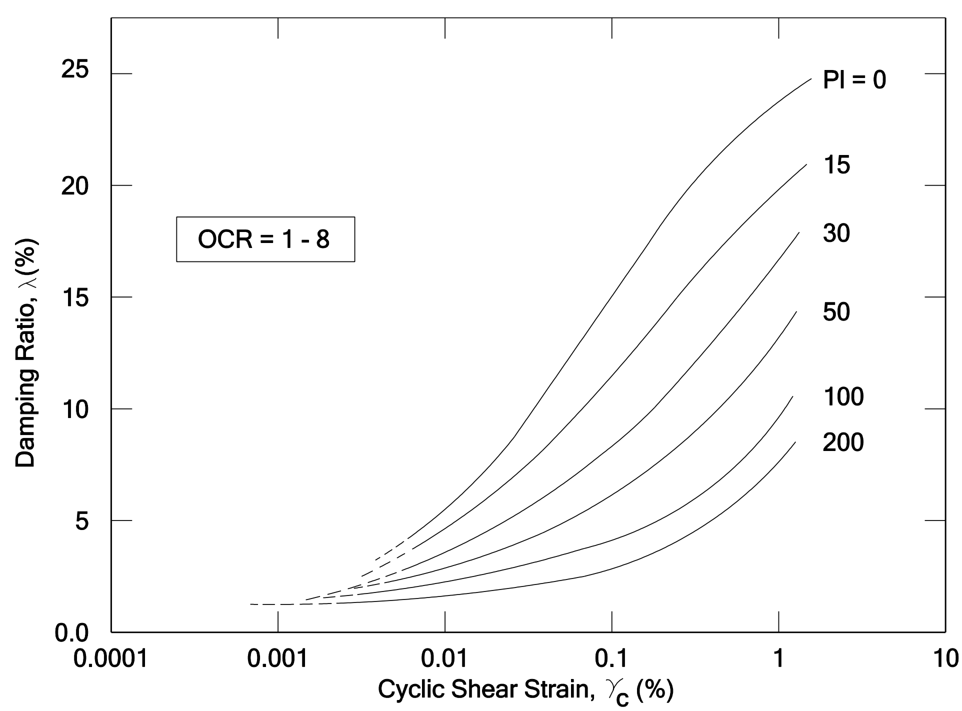


Figure 6-4. Equivalent viscous damping ratio vs. cyclic shear strain for fine grained soils (redrafted from Vucetic and Dobry, 1991).

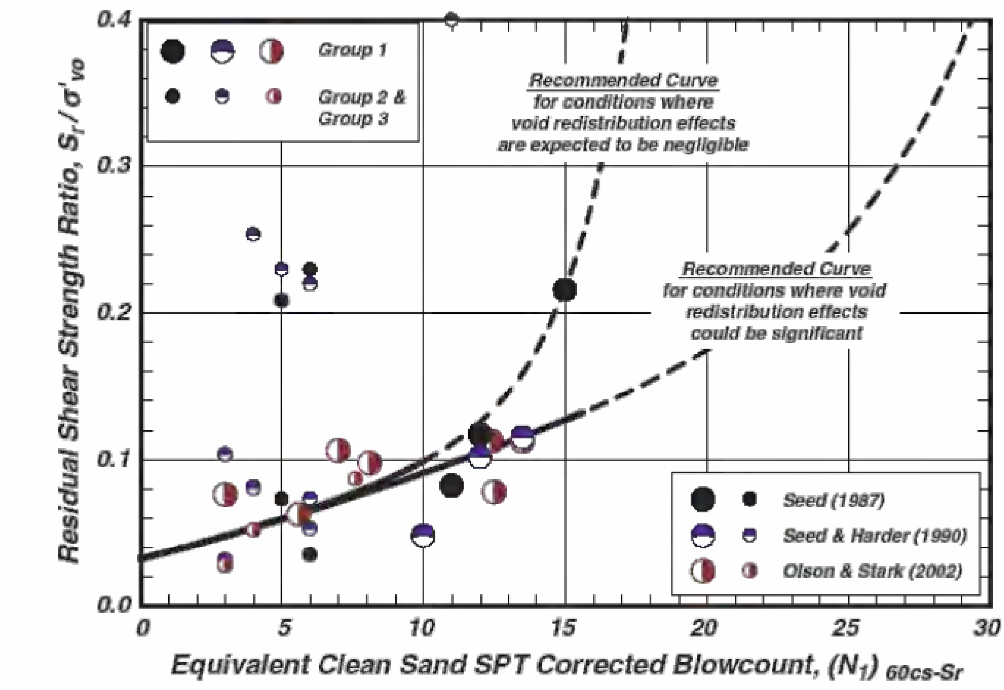
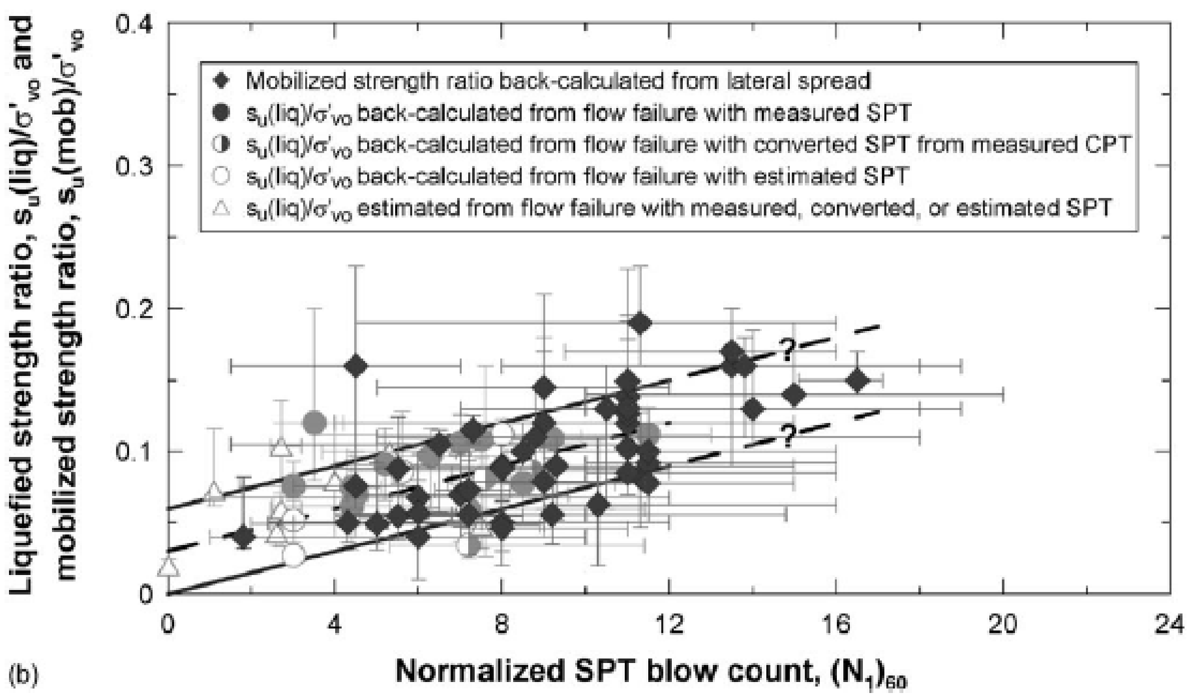


Figure 6-5. Correlation between the Residual Undrained Strength Ratio, S_r/σ'_{vo} and equivalent clean sand SPT blow count, $(N1)_{60\text{-}CS}$ (Idriss and Boulanger, 2007).



(b)

Figure 6-6. Correlation between Undrained Residual Strength Ratio (S_r/σ'_{vo}) and Normalized SPT Resistance ($(N_1)_{60}$) (Olson and Johnson, 2008).

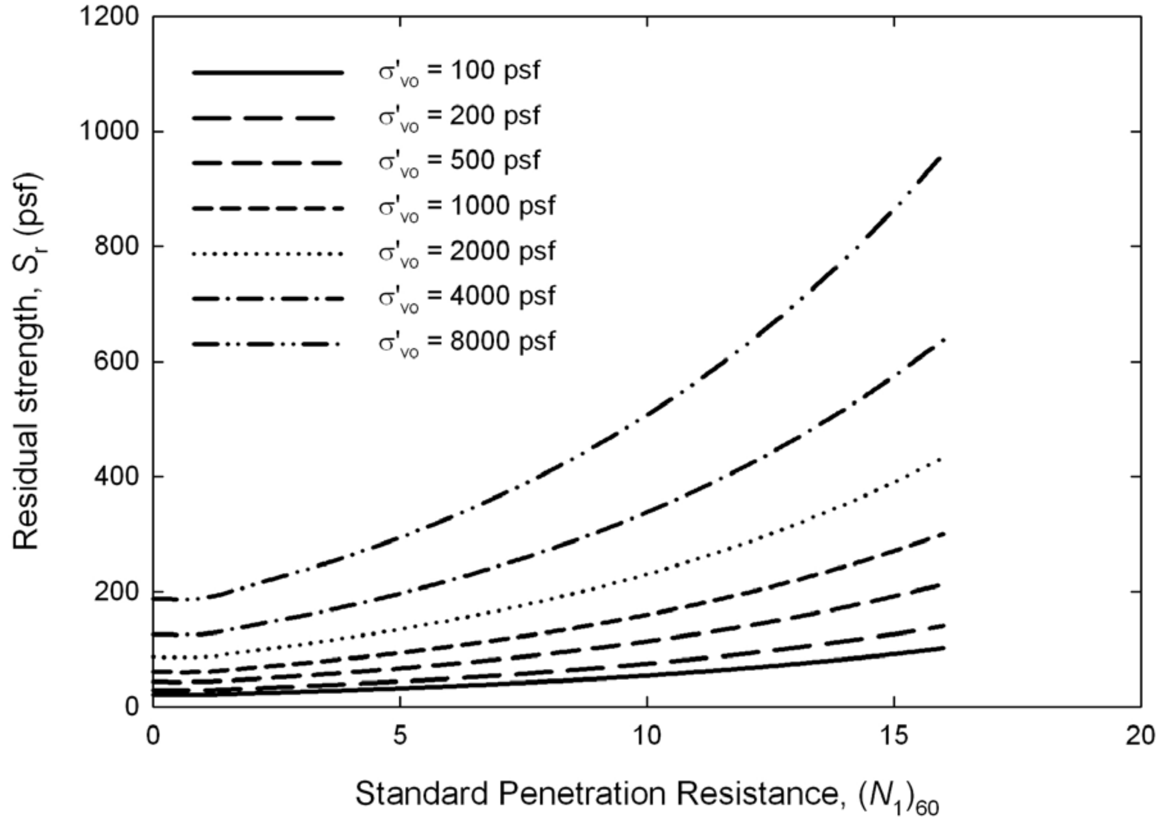


Figure 6-7. Variation of residual strength ratio with SPT resistance and initial vertical effective stress using Kramer-Wang model (Kramer, 2008).

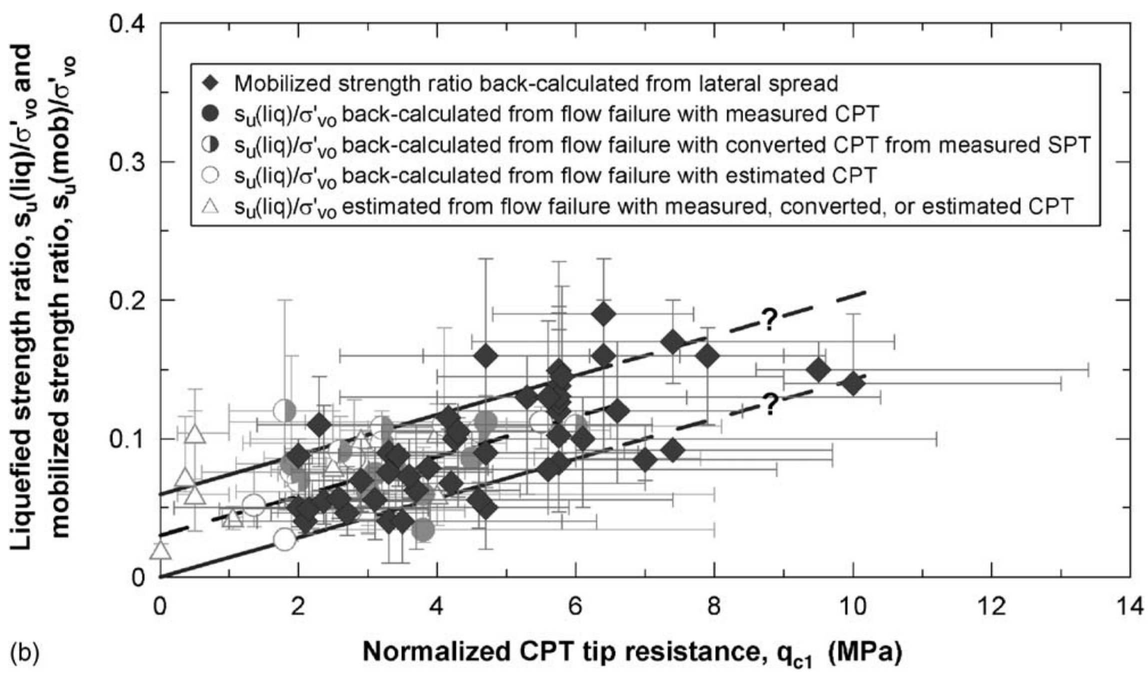


Figure 6-8. Correlation between the Undrained Residual Strength Ratio, S_r/σ'_{vo} , and normalized CPT tip resistance, q_{c1} (Olson and Johnson, 2008)

6.5 Geotechnical Seismic Design Procedures

The geotechnical designer shall evaluate the site and subsurface conditions to the extent necessary to provide the following assessments and recommendations:

- An assessment of the seismic hazard,
- Determination of design ground motion values,
- Site characterization,
- Seismic analysis of the foundation materials, and
- An assessment of the effects of the foundation response on the proposed structure.

Specific aspects of seismic foundation design generally consist of the following procedures:

- Determine the Peak Bedrock Acceleration (PGA), 0.2 and 1.0 second spectral accelerations for the bridge site from the 2014 USGS National Seismic Hazard Maps for the 1000-year return period and the 2014-CSZE Seismic Hazard map,
- Determine the Site Class and Site Coefficients based on the properties of the soil profile,
- Develop the Design Response Spectrum for the site per AASHTO (2011) or conduct ground response analysis if necessary,
- Determine the potential for loss of soil strength and degradation of stiffness of foundation soils,
- If significant cyclic degradation due to excess pore pressure generation (e.g., liquefaction of sand or silt, sensitive fine-grained soil) is predicted:
    - Estimate embankment deformations due to slope instability and lateral spreading and evaluate the impacts of embankment deformations in terms of bridge damage potential and approach fill performance for both the 1000-year event and the CSZE (if appropriate),
    - Estimate embankment settlement due to seismic loading and the potential for any resulting downdrag loading and potential bridge damage,
    - Determine soil properties for both the liquefied and non-liquefied soil conditions for use in the lateral load analysis and modeling of deep foundations,
    - Determine reduced foundation resistances and their effects on proposed bridge foundation elements.
- Evaluate seismic-induced slope stability and settlement for non-liquefied soil conditions,
- Evaluate impacts of seismic-induced loads and deformations on bridge foundations,
- Develop values for nonlinear soil stiffness (e.g., foundation springs) for use in modeling dynamic loading (liquefied and non-liquefied soil conditions). Also provide recommendations regarding lateral springs for use in modeling abutment backfill soil resistance,
- Determine earthquake induced earth pressures (active and passive) and provide stiffness values for equivalent soil springs (if required) for retaining structures and below grade walls,

- Evaluate options to mitigate seismic geologic hazards, such as ground improvement, if appropriate.

Note that separate analysis and recommendations will be required for the 2014-CSZE and 1000-year seismic design ground motions. A general design procedure is described in the flow chart shown in Figure 6-9 along with the information that should be supplied in the final geotechnical report.

STEP 1; Ground Motion Data

- Identify the seismic sources in the region affecting the site for the Life Safety and Opertaion criteria:
- Determine Peak Ground Accelerations (PGA), S_s, and S_1, from the 2014 USGS Seismic Hazard Maps or the deterministic CSZE hazard maps, for bedrock (Site Class B) conditions
- Life Safety: Determine Site Class and Site Coefficients, then develop the Design Response Spectrum representing the Uniform Seismic Hazard

Operational: Using the CSZE_ARS program develop the Design response Spectrum representing the Operational Uniform Seismic Hazard

STEP 2; Site Response Analysis

- Decide whether a site response analysis is warranted. If not, use standard AASHTO General Procedure to develop Design Response Spectra (Step 1).
- If so, perform deaggregation of seismic hazard to determine principal contributing M & R pairs.
- Use GMPE's to obtain bedrock/ground surface PGA and spectral acceleration values for each contributing M&R pair
- Select appropriate acceleration time histories and establish scaling factors or perform spectral matching (for each principal contributing M-R pair).
- Generate the following using dynamic site response analysis (for each principal contributing M-R pair):
    - PGA and 5% damped smoothed response spectra at the depth(s) of interest (e.g., ground surface, depth of pile/pier fixity).

STEP 3; Evaluate Liquefaction Potential & Effects (As≥0.15g)

- Estimate the cyclic resistance of the soils as a function of depth from in situ and/or lab data.
- Specify the cyclic loading at each depth from either a site response analysis or using simplified methods.
- Using the ratio of the cyclic resistance to the cyclic loading, determine the potential for significant excess pore pressure generation and cyclic degradation of soil stiffness and strength.

Liquefaction Potential

STEP 3a; For foundation soils susceptible to liquefaction:

- Estimate post-liquefaction soil strengths
- Evaluate embankment stability and estimate deformations
- Evaluate effects of embankment deformations on structure foundations and bridge performance
- Develop mitigation designs if required
- Provide reduced foundation resistances under liquefied soil conditions.

No Liquefaction Potential

STEP 3a; Evaluate Non-liquefied Soil Response

- Dynamic settlement of foundation soils and downdrag potential
- Evaluate approach fill slope stability
- Estimate lateral approach fill displacements

STEP 4; Provide seismic foundation modeling parameters as appropriate (see Section 1.10.4 of BDDM):

Spread Footings

- Effective shear modulus (as per Section 6.6.1.1). A ground response analysis may also be conducted to determine the appropriate shear strain value to use.
- Poisons ratio, ν
- Kp, Su, μ, γ

Piles

- p-y curve and other soils data for modeling non-liquefied and liquefied soils
- p-y multipliers
- Designation as "end bearing" or "friction" piles for modeling axial stiffness

Shafts

- p-y curve and other soils data for modeling non-liquefied and liquefied soils
- p-y multipliers

Figure 6-9. General Geotechnical Seismic Design Procedures

6.5.1 Design Ground Motion Data

6.5.1.1 Development of Design Ground Motion Data

The geotechnical engineer is responsible for developing and providing the design response spectra for the project.

With the implementation of the CSZE scenario, the design response spectrum generated by the **CSZE_ARS** will be used to meet the Operational Design Criteria. If a site specific ground motion response analysis is required, the **CSZE_ARS** at V_{s30}=760-m/s response spectrum will be used as the target spectrum which the earthquake records should be scaled.

For Life Safety, there are two options for the development of design ground motion parameters (response spectral ordinates) for seismic design. These are described as follows:

> **AASHTO General Procedure:** Use ground motion values for the 2014 USGS Seismic Hazard Maps, as appropriate, combined with site coefficients in Tables 6.2-A, 6.2-B, and 6.2-C of this manual.
>
> **Site Specific Ground Motion Response:** Use ground motion values for the 2014 USGS Seismic Hazard Maps, as appropriate, with site specific ground response analysis.

Both methods take local site effects into account. For most routine structures at sites with competent soils (i.e., no liquefiable, sensitive, or weak soils), the first method (General Procedure), described in Article 3.4 of the "*AASHTO Guide Specification for LRFD Seismic Bridge Design"*, is sufficient to account for site effects. However, the importance of the structure, the ground motion levels and the soil and geological conditions of a site may dictate the need for a Site Specific Ground Response Analysis.

At some bridge sites, the subsurface conditions (soil profile) may change dramatically along the length of the bridge and more than one response spectrum may be required to represent segments of the bridge with different soil profiles. If the site conditions dictate the need for more than one response spectrum for the bridge, the design response spectrum may be developed by combining the individual spectra into a composite spectrum that envelope the spectral acceleration values of the individual spectra.

6.5.1.2 AASHTO General Procedure

The standard method of developing the acceleration response spectrum is described in AASHTO, 2014. First, the peak ground acceleration (PGA), the short-period spectral acceleration (S_s) and the long-period spectral acceleration (S_1) are obtained for both the 1000-year return period (Life Safety evaluation). Then the soil profile is classified as one of six different site classes (A through F) based on the time-averaged shear wave velocity in the upper 30 meters of soil $(Vs)_{30}$. This Site Class designation is then used to determine the "Site Coefficients", F_{pga}, F_a and F_v**,** except for sites classified as Site Class F, which required a site-specific ground response analysis (see Section 6.5.1.4)**.** These site coefficients are then multiplied by the peak ground acceleration (***Fpga*** x PGA), the short-period spectral acceleration (***Fa*** x Ss) and the long period spectral acceleration (***Fv*** x S_1) respectively and the resulting values are used to develop the site response spectrum. A program, **ODOT_ARS_v2014.16**, to develop the response spectra using the general procedure has been developed by the ODOT Bridge Section and can be accessed through the ODOT Bridge Section seismic web page.

In addition to the Site Class F soils, the standard Site Class designations may not be appropriate for other subsurface conditions. Sites with significant contrasts in the shear wave velocity among layers within 200 ft of the ground surface (i.e. strong impedance contrasts) do not conform to the model used to develop the AASHTO site coefficients. A site specific ground response analysis should be conducted to develop the design response spectrum in these cases.

Also, sites with deep soil columns, e.g. soil columns in excess of 500 ft, should also be considered candidates for a site-specific seismic response analysis, as the differences in the soil profile at these types of sites, compared to the profiles used to develop the AASHTO site coefficients, may create significant differences in site response compared to that predicted using the AASHTO site factors.

Sites with shallow bedrock conditions (less than 100 feet to bedrock) require special consideration. The AASHTO site coefficients were developed by modeling soil profiles representing each of the Site Classes that were at least 100 feet (30 meters) in depth. Where bedrock (defined as a material unit with a shear wave velocity ≥ 2500 fps) is less than 100 feet deep the standard methods described in AASHTO for characterizing site class are not applicable and currently there is no consensus about how to adjust site class parameters for shallow bedrock conditions. Shear wave velocities, or SPT "N", values, obtained in bedrock that is within 100 feet of the ground surface should not be included in the calculation for determining the average shear wave velocity ($V_{s(30)}$) used in site class designation. In these conditions the following guidance is recommended:

- If the depth to Site Class B bedrock is greater than 80 feet, then the AASHTO site coefficients are considered acceptable for use. As an approximation the $V_{s(30)}$ value should be computed assuming that the soil extends to a depth of 100 feet (30 m) and extrapolating the profile of Vs in the soil to that depth.
- If Site Class B bedrock is within 10 feet of the ground surface, or the base of the foundation footing or pile cap, assume Site Class B conditions.
- If the depth to Site Class B bedrock is between 10 ft and 80 ft, develop the Site Class based on the average shear wave velocity obtained from only the soil layers above the bedrock. Adjust the site class obtained from this procedure upwards to a higher site class if necessary based on engineering judgment.

At these locations, a site-specific seismic ground response analysis may also be considered. However such an analysis may lead to unrealistically amplified ground motions at the predominant period of the soil deposit. This effect should be critically reviewed and evaluated in light of the influence on ground motions in the structural period range of interest for the project.

6.5.1.3 Response Spectra and Analysis for Liquefied Soil Sites

Site coefficients have not been developed for liquefied soil conditions. For this case site-specific analysis is required to estimate ground motion characteristics. The "*AASHTO Guide Specifications for LRFD Seismic Bridge Design"* (2011) states that at sites where soils are predicted to liquefy the bridge shall be analyzed and designed under two configurations, the non-liquefied condition and liquefied soil condition described as follows:

- **Nonliquefied Configuration:** The structure is analyzed and designed, assuming no liquefaction occurs by using ground response spectrum and soil design parameters based on non-liquefied soil conditions,

- **Liquefied Configuration:** The structure is reanalyzed and designed under liquefied soil conditions assuming 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 non-liquefied configuration.

A site-specific response spectrum may be developed for the "Liquefied Configuration" based on a ground response analysis that utilizes non-linear, effective stress methods, which properly account for pore pressure buildup and stiffness degradation of the liquefiable soil layers see Section 6.5.1.4. The decision to complete a ground response analysis where liquefaction is anticipated should be made by the geotechnical designer based on the site geology and characteristics of the bridge being designed. The design response spectrum resulting from the ground response analyses shall not be less than two-thirds of the spectrum developed using the general procedure for the non-liquefied soil condition.

6.5.1.4 Site Specific Ground Motion Response

For most projects, the General Procedure as described in Article 3.4.1 of the "*AASHTO Guide Specifications for LRFD Seismic Bridge Design"* (2011) is appropriate and sufficient for determining the seismic hazard and site response spectrum. However, it may be appropriate to perform a site-specific evaluation for cases involving special aspects of seismic hazard (e.g., near fault conditions, high ground motion values, coastal sites located in relatively close proximity to the CSZ source), specific soil profiles, and essential bridges. The results of the site-specific response analysis may be used as justification for a reduction in the spectral response ordinates determined using the standard AASHTO design spectrum (General Procedure) representing the Uniform Seismic Hazard.

Site specific ground response analyses (GRA) are required for Site Class "F" soil profiles, and may be warranted for other site conditions or project requirements. Site Class "F" soils are defined as follows:

- Peat or highly organic clays, greater than 10 ft in thickness,
- Very high plasticity clays (H > 25 ft with PI > 75),
- Very thick soft/medium stiff clays (H >120 ft).

Other conditions under which a ground response analysis should be considered are listed below:

- Very important or critical structures or facilities,
- Liquefiable Soil Conditions. For liquefiable soil sites, it may be desirable to develop response spectra that take into account increases in pore water pressure and soil softening. This analysis results in a response spectra that is generally lower than the nonliquefied response spectra in the short-period range (approximately < 1.0 sec). A nonlinear effective stress analysis may also be necessary to refine the standard liquefaction analysis based on the simplified empirical method (Youd et. al., 2001) with information from a GRA. This is especially true if liquefaction mitigation designs are proposed. The cost of liquefaction mitigation is sometimes very large and a more detailed analysis to verify the potential, and extent, of liquefaction is usually warranted,
- Very deep soil deposits, thin soil layers (<50') over bedrock and profiles with high Impedance contrasts (i.e. large, abrupt changes in V_s),

- To obtain better information for evaluating lateral deformations, near surface soil shear strain levels or deep foundation performance,
- To obtain ground surface PGA values for abutment wall or other design.

Procedures for conducting a site specific ground response analysis are described in Article 3.4.3. of AASHTO (2011) and in Chapter 5 of Kavazanjian, et al. (2011).

A ground response analysis simulates the response of a layered soil deposit subjected to earthquake motions. One-dimensional, equivalent-linear models are commonly utilized in practice. This model uses an iterative total stress approach to estimate the nonlinear elastic behavior of soils. Modified versions of the numerical model SHAKE (e.g., ProSHAKE, SHAKE91, SHAKE2000) and other models (e.g., DMOD, DEEPSOIL) are routinely used to simulate the propagation of seismic waves through the soil column and generate output consisting of ground motion time histories at selected locations in the soil profile, plots of ground motion parameters with depth (e.g., PGA, cyclic shear stress, cyclic shear strain), and acceleration response spectra at depths of interest. The program calculates the induced cyclic shear stresses in individual soil layers which may be used in liquefaction analysis.

The equivalent linear model provides reasonable results for small to moderate cyclic shear strains (less than about 1 to 2 percent) and modest accelerations (less than about 0.3 to 0.4g) (Kramer and Paulsen, 2004). Equivalent linear analysis cannot be used where large strain incompatibilities are present, to estimate permanent displacements, or to model development of pore water pressures in a coupled manner. Computer programs capable of modeling non-linear, effective stress soil behavior are recommended for sites where high ground motion levels are indicated and it is anticipated that moderate to large shear strains will be mobilized. These are typically sites with soft to medium stiff fine-grained soils or saturated deposits of loose to medium dense cohesionless soils.

Input parameters required for site specific ground response analysis include soil layering (thickness), standard geotechnical index properties for the soils, dynamic soil properties for each soil layer, the depth to bedrock or firm soil interface, and a set of ground motion time histories representative of the primary seismic hazards in the region. Dynamic soil parameters for the equivalent linear models include the shear wave velocity, or initial (small strain) shear modulus, the unit weight for each soil layer and curves relating the shear modulus and damping ratio as a function of shear strain (see Section 6.4.1 and Figure 6-2, Figure 6-3 and Figure 6-4 for examples).

Nonlinear effective stress analysis methods such as D-MOD2000, DESRA and others may also be used to develop response spectra, especially at sites where liquefaction of foundation soils is likely (see Section 6.5.2.2). All non-linear, effective stress modeling and analysis will require an independent peer reviewer with expertise in this type of analysis.

The results of the dynamic ground response modeling should be presented in the form of a standard response spectrum graph showing the ”average” soil response spectrum from all of the output response spectra. Site-specific response spectra may be used for design; however the spectral ordinates shall be no less than 2/3rd of the spectral ordinates for the AASHTO response spectrum using the General Procedure. The standard AASHTO response spectrum and the “2/3 AASHTO” response spectrum should both be plotted on the same graph as the response spectrum from the site response analysis for comparison purposes. A “smoothed” response spectra may be obtained following procedures outlined in AASHTO.

Engineering judgment will be required to account for possible limitations of the response modeling. For example, equivalent linear analysis methods may overemphasize spectral response where the predominant period of the soil profile closely matches the predominant period of the bedrock motion. Final modification of the design spectrum must provide representative constant velocity and constant displacement portions of the response.

6.5.1.5 Selection of Time Histories for Ground Response Analysis

AASHTO (2014) allows two options for the selection of time histories to use in ground response analysis. The two options are:

a) Use a suite of 3 response spectrum-compatible time histories representing the bedrock motions and then define the design response spectrum at the ground surface by enveloping the maximum computed response, or

b) Use at least 7 bedrock time histories and develop the design spectrum as the mean of the computed ground surface response spectra.

For both options, the time histories shall be developed from the representative recorded earthquake motions, or in special instances synthetic ground motions may be used with approval of ODOT. 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.

Analytical techniques used for spectral matching shall be demonstrated to be capable of achieving seismologically realistic time series. The time histories should be spectrally-matched to the bedrock spectrum of interest. Alternatively, if ground motion scaling is used to modify the bedrock motions the bedrock spectra should match the bedrock spectrum in the period range of significance (i.e., $0.5 < T < 2.0$, where "T" is the fundamental period of the structure). The predominant period of the soil profile should also be considered in the scaling process.

The procedures for selecting and adjusting time histories for use in ground motion response analysis can be summarized as follows:

1. Identify the target response spectra to be used to develop the time histories. The target spectra are obtained from the 2014 USGS Seismic Hazard Maps (for the Site Class B/C boundary) or the CSZE response spectra at V_{s30}=760-m/s, as appropriate. Two spectra may be required, one for the Operational performance level (the CSZ earthquake) and one for the Life Safety performance level (PSHA, 1000-yr event), depending on location within the state.

2. Identify the seismic sources that contribute to the seismic hazard for the site. For the Cascadia Subduction Zone event, selected subduction zone time histories that best represent and model the significant characteristics of the CSZ. For the PSHA sources use the deaggregation information for the 2014 USGS Seismic Hazard maps to obtain information on the primary sources that affect the site. Select time histories to be considered for the analysis, considering tectonic environment and style of faulting (subduction zone, Benioff zone, or shallow crustal faults), seismic source-to-site-distance, earthquake magnitude, duration of strong shaking, peak acceleration, site subsurface characteristics, predominant period, etc. In areas where the hazard has a significant contribution from both the Cascadia Subduction Zone (CSZ) and from crustal sources (e.g., Portland and much of the Western part of the state) both earthquake sources need to be included in the analysis and development of a site specific response spectra. In cases such as this, it is recommended that the ground response analysis be conducted using a collection of time histories that include at least 3 motions representative of subduction zone events and 3

motions appropriate for shallow crustal earthquakes with the design response spectrum developed considering the mean spectrum of each of these primary sources.

The adjusted time histories (either scaled or spectrally matched) must satisfy the following requirements:

1. Peak amplitudes are representative (PGA, PGV, PGD),
2. Frequency content is representative (spectral components; SA, SV, SD),
3. Duration is appropriate,
4. Energy is appropriate (e.g., Arias Intensity).

All 4 of these ground motion characteristics can be checked against up-to-date empirical relationships.

At sites where the uniform hazard is dominated by a single source, three (3) time histories, representing the seismic source characteristics, may be used and the design response spectrum determined by enveloping the caps of the resulting response spectra.

3. Scale the time histories to match the target spectrum as closely as possible in the period range of interest prior to spectral matching. Match the response spectra from the recorded earthquake time histories to the target spectra using methods that utilize either time series adjustments in the time domain or adjustments made in the frequency domain. See AASHTO (2011), Matasovic et. al., (2012) and Kramer (1996) for additional guidance on these techniques.

4. Once the time history(ies) have been spectrally matched, they can be used directly as input into the ground response analysis programs to develop response spectra and other seismic design parameters. Five percent (5%) damping is typically used in all site response analysis.

6.5.1.6 Near-Fault Effects on Ground Motions

For sites located within 6 miles of a known active fault capable of producing at least a magnitude 5 earthquake the near-field effects of the fault should be considered. If the fault is included in the USGS Seismic Hazard maps, then the higher ground motions due solely to the proximity of the fault are already accounted for in the spectral acceleration values. However, the near-fault ground motion effects of directivity and directionality were not explicitly modeled in the development of national ground motion maps, and the code/specification based hazard level may be significantly unconservative in this regard. These “near-fault” effects are normally only considered for essential or critical structures and are usually not considered for routine seismic design. Consult with the bridge designer to determine the importance of the structure and the need to consider near-fault effects.

6.5.1.7 Ground Motion Parameters for Other Structures

For buildings, restrooms, shelters, and other non-transportation structures, specification based seismic design parameters required by the Oregon Structural Specialty Code (OSSC) and previous of the International Building Code (ICC., 2012) should be used. The seismic design requirements of the OSSC 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. The OSSC identifies procedures to develop a maximum considered earthquake acceleration response spectrum.

Site response shall be in accordance with the OSSC. As is true for transportation structures, for critical or unique structures or for sites characterized as soil profile Type F (thick sequence of soft soils or liquefiable soils), site response analysis may be required.

6.5.1.8 Site Amplification Factors

Soil amplification factors that account for the presence of soil over bedrock, with regard to the estimation of peak ground acceleration (PGA), are directly incorporated into the development of the general procedure for developing response spectra for structural design of bridges and similar structures in AASHTO (2011, 2014) and also for the structural design of buildings and non-transportation related structures in the International Building Code (IBC, 2012). Amplification factors should be applied to the peak bedrock acceleration to determine the peak ground acceleration (PGA) for liquefaction assessment, such as for use with the Simplified Method Section 6.5.5.2 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 Factor (F_{pga}) presented in AASHTO (2014), Article 3.10.3.2 may be applied to the bedrock PGA used to determine the ground surface acceleration, unless a site specific evaluation of ground response is conducted. Refer to Anderson, et al. (2008) for additional guidance on the selection and use of site amplification values.

6.5.2 Liquefaction Analysis

Liquefaction has been one of the most significant causes of damage to bridge structures during earthquakes. Liquefaction can damage bridges, retaining walls and other transportation structures and facilities in many ways including:

- Bearing failure of shallow foundations founded above liquefied soil,
- Liquefaction induced ground settlement,
- Lateral spreading or flow failures of liquefied ground,
- Large transient displacements associated with low frequency ground motion,
- Increased active earth pressures on subsurface structures,
- Reduced passive resistance for anchors, piles, and walls,
- 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, cohesionless soils. Liquefaction can occur in sand and non-plastic to low plasticity silt-rich soils, and in confined gravel layers; however, it is most common in sands and silty sands. For a detailed discussion of the effects of liquefaction, including the types of liquefaction phenomena, liquefaction-induced bridge damage, evaluation of liquefaction susceptibility, post liquefaction soil behavior, deformation analysis and liquefaction mitigation techniques refer to Kramer (2008), Caltrans (2013) and Dickenson, et al. (2002).

Liquefaction hazard assessment includes identifying soils susceptible to liquefaction on the basis of composition and cyclic resistance, evaluating whether the design earthquake loading will initiate liquefaction or significant cyclic degradation, and estimating the potential effects on the planned facility.

Potential effects of soil liquefaction on structure foundations include the following:

- Loss of strength in the liquefied layer(s); resulting in reduced foundation stiffness and resistance to foundation loading,
- Liquefaction-induced ground settlement; resulting in downdrag loads on deep foundations,

- Slope instability due to flow failures or lateral spreading; resulting in large embankment displacements and deep foundation loads.

Due to the high cost of liquefaction mitigation measures, it is important to identify liquefiable soils and the potential need for mitigation measures early on in the design process (during the DAP (TS&L) phase) so that appropriate and adequate funding decisions are made. The following sections provide ODOT's policies regarding liquefaction and a general overview of liquefaction hazard assessment and its mitigation.

6.5.2.1 Liquefaction Design Policies

All new bridges, bridge widening projects and retaining walls in areas with a ground surface seismic acceleration coefficient, As, greater than or equal to 0.15g should be evaluated for liquefaction potential.

The maximum considered depth of influence of liquefaction-related effects on surface structures shall be limited to 75 feet. The potential for strength and stiffness reductions due to increased seismically-induced pore pressures may be considered below this depth for specific projects (e.g., deep foundations, buried structures or utilities) based of cyclic laboratory test data and/or the use of non-linear, effective stress analysis techniques. All non-linear, effective stress modeling and analysis will require an independent peer reviewer with expertise in this type of analysis.

Bridges scheduled for Phase 2 seismic retrofits should also be evaluated for liquefaction potential if they are in a seismic zone with an acceleration coefficient, (As), ≥ 0.15g.

In general, liquefaction is conservatively predicted to occur when the factor of safety against liquefaction (FS_L) is less than 1.1. A factor of safety against liquefaction of 1.1 or less also indicates the potential for liquefaction-induced ground movement (lateral spread and settlement). Soil layers with FS_L between 1.1 and 1.4 will have reduced soil shear strengths due to excess pore pressure generation. For soil layers with FS_L greater than 1.4, excess pore pressure generation is considered negligible and the soil does not experience appreciable reduction in shear strength.

Groundwater: The groundwater level to use in the liquefaction analysis should be determined as follows:

- **Static Groundwater Condition:** Use the estimated, average annual groundwater level. Perched water tables should only be used if water is estimated to be present in these zones more than 50% of the year,
- **Tidal Areas:** Use the mean high tide elevation,
- **Adjacent Stream, Lake or Standing Water Influence:** Use the estimated, annual, average elevation for the wettest (6 month) seasonal period.

Note that groundwater levels measured in borings advanced using water or other drilling fluids may not be indicative of true static groundwater levels. Water in these borings should be allowed to stabilize over a period of time to insure measured levels reflect true static groundwater levels. Groundwater levels are preferably measured and monitored using piezometers, taking measurements throughout the climate year to establish reliable static groundwater levels taking seasonal effects into account.

6.5.2.2 Methods to Evaluate Liquefaction Potential

Evaluation of liquefaction potential should be based on soil characterization using in-situ testing methods 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 considered supplementary unless the soil profile includes clean gravels and adjacent soil layers that may impede the rapid dissipation of excess pore water pressure during cyclic loading. V_s and BPT testing may be appropriate in soils difficult to test using SPT and CPT methods such as gravelly soils though, in the absence of fine grained soil layers that may act as poorly drained boundaries, these soils often have a low susceptibility to liquefaction potential due to high permeability and rapid drainage. If the CPT method is used, SPT sampling and soil gradation testing shall still be conducted to obtain direct information on soil type and gradation parameters for use in liquefaction susceptibility assessment.

Preliminary Screening: A detailed evaluation of liquefaction potential is not required if any of the following conditions are met:

- The peak ground acceleration coefficient, As, is less than 0.15g,
- The ground water table is more than 75 feet below the ground surface,
- The soils in the upper 75 feet of the profile are low plasticity silts, sand, or gravelly sand having a minimum SPT resistance, corrected for overburden depth and hammer energy ($N1_{60}$), of 25 blows/ft., a cone tip resistance q_{ciN} of 150 tsf or a minimum shear wave velocity of 800 feet/sec.
- All soils in the upper 75 feet have a P1>12 and a water content (Wc) to liquid limit (LL) ratio of less than 0.85. Note that cohesive soils with P1>12 may still be very soft or exhibit sensitive behavior and could therefore undergo significant strength loss under earthquake shaking. This criterion should be used with care and good engineering judgment. Refer to Bray and Sancio, (2006) and Boulanger and Idriss, (2006) for additional information regarding the evaluation of fine-grained soils for strength loss during cyclic loading.

Simplified Procedures: Simplified Procedures should always be used to evaluate the liquefaction potential even if more rigorous methods are used to supplement or refine the analysis. The Simplified Procedure was originally developed by Seed and Idriss (1971) and has been periodically modified and improved since. It is routinely used to evaluate liquefaction resistance in geotechnical practice.

The paper titled "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils" (Youd et al., (2001) should be referenced for the Simplified Procedures to be used in the assessment of liquefaction susceptibility. This paper resulted from a 1996 workshop of liquefaction experts sponsored by the National Center for Earthquake Engineering Research and the National Science Foundation with the objective being to gain consensus on updates and augmentation of the Simplified Procedures. Youd et al. (2001) provide procedures for evaluating liquefaction susceptibility using SPT, CPT, V_s, and BPT criteria.

The Simplified Procedures are based on the evaluation of both the cyclic resistance ratio (CRR) of a soil layer (i.e., the cyclic shear stress required to cause liquefaction) and the earthquake induced cyclic shear stress ratio (CSR). The resistance value (CRR) is estimated 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. Youd et al. (2001) provide the empirical liquefaction resistance charts for both SPT and CPT data to be used with the simplified procedures. 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 basic form of the simplified procedures used to calculate the earthquake induced CSR for the Simplified Method is shown in the following equation:

Equation 6.1:

$$CSR_{eq} = \frac{\tau_{av}}{\sigma_{vo}'} = 0.65\left(\frac{a_{max}}{g}\right)\left(\frac{\sigma_{vo}}{\sigma_{vo}'}\right) r_d$$

Where: τ_{av} = average or uniform earthquake induced cyclic shear stress

a_{max} = peak horizontal acceleration at the ground surface accounting for site amplification effects (ft/sec^2)

g = acceleration due to gravity (ft/sec^2)

σ_o = initial total vertical stress at depth being evaluated (lb/ft^2)

σ_o' = initial effective vertical stress at depth being evaluated (lb/ft^2)

r_d = stress reduction coefficient

The factor of safety against liquefaction is defined by:

FS_{liq} = CRR/CSR

The use of the SPT for the Simplified Procedure has been most widely used and has the advantage of providing soil samples for fines content and gradation testing. The CPT provides the most detailed soil stratigraphy, is less expensive, can simultaneously 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 the most detailed liquefaction assessment for a site.

Where SPT data is used, the sampling and testing procedures should include:

- Documentation on the hammer efficiency (energy measurements) of the system used,
- Correction factors for borehole diameter, rod length 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 using non-standard samplers such as the Dames and Moore or modified California samplers shall not be used for liquefaction evaluations.

Liquefaction potential may also be evaluated using shear wave velocity (Vs) testing and Becker Penetration Tests (BPT); however, these methods are considered supplementary unless the soil profile includes clean gravels and adjacent soil layers that may impede the rapid dissipation of excess pore water pressure during cyclic loading. Vs and BPT testing may be appropriate in soils difficult to test using SPT and CPT methods such as gravelly soils though, in the absence of fine grained soil layers that may act as poorly drained boundaries, these soils often have a low susceptibility to liquefaction potential due to high permeability and rapid drainage. The Becker Penetration Test (BPT) is often used for major projects involving gravelly foundation soils. Recent investigations of the BPT have highlighted the strengths and limitations of the methods, as well as demonstrated the need for energy measurements in order to convert BPT blow counts to equivalent SPT N_{60} values (Ghafghazi et al, 2014).

If liquefaction is predicted based on the Simplified Method Section 6.5.2.2, and the effects of liquefaction require mitigation measures, a more thorough ground response analysis (e.g. SHAKE, DMOD) should be considered to verify and substantiate the predicted, induced ground motions. This procedure is especially recommended for sites where liquefaction potential is marginal ($0.9 < FS_L < 1.10$). It is also important to determine whether the liquefied soil layer is stratigraphically (laterally) continuous and oriented in a manner that will result in lateral spread or other adverse impact to the structure or facility.

Limitations of the Simplified Procedures: The limitations of the Simplified Procedures should be recognized. The Simplified Procedures were developed from empirical evaluations of field observations of ground surface evidence for the occurrence or non-occurrence of liquefaction at depth. Most of the case history data was collected from level to gently sloping terrain underlain by Holocene-age alluvial or fluvial sediment at depths less than 50 feet. Therefore, the Simplified Procedures are applicable to only these site conditions. Caution should be used for evaluating liquefaction potential at depths greater than 50 feet using the Simplified Procedure. In addition, the Simplified Procedures estimate the trend of earthquake induced cyclic shear stress ratio with depth based on a coefficient, r_d, which becomes highly variable at depths below about 40 feet.

As an alternative to the Simplified Procedures, one dimensional ground response analyses should be used to better determine the maximum earthquake induced shear stresses at depths greater than about 50 feet. Equivalent linear or nonlinear, total stress computer programs (e.g Shake2000, ProShake, DEEPSOIL, DMOD) may be used for this purpose.

Magnitude and PGA for Liquefaction Analysis: The procedures described in Section 6.3.2 and Section 6.3.3 should be used to determine the appropriate earthquake magnitude and peak ground surface acceleration to use in the simplified procedure for liquefaction analysis. If a site specific ground response analysis is used to determine the peak ground surface acceleration(s) for use in liquefaction analyses, this value should be representative of the cyclic loading induced by the M-R pair(s) of interest. It is anticipated that PGA values obtained from site-specific ground response analysis will often differ from the PGA determined by the AASHTO General Procedure for the uniform seismic hazard. The PGA and magnitude values used in the liquefaction hazard analysis shall be tabulated for all considered seismic sources.

Magnitude Scaling Factors (MSF): Magnitude scaling factors are required to adjust the cyclic stress ratios (either CRR or CSR) obtained from the Simplified Method (based on M = 7.5) to other magnitude earthquakes. The range of Magnitude Scaling Factors recommended in the 1996 NCEER Workshop on Evaluation of Liquefaction Resistance of Soils (Youd, et. al., 2001) is recommended. Below magnitude 7.5, a range is provided and engineering judgment is required for selection of the MSF. Factors more in line with the lower bound range of the curve are recommended. Above magnitude 7.5 the factors recommended by Idriss are recommended. This relationship is presented in the graph (Figure 6-10) and the equation of the curve is: $MSF = 10^{2.24} / M^{2.5}$.

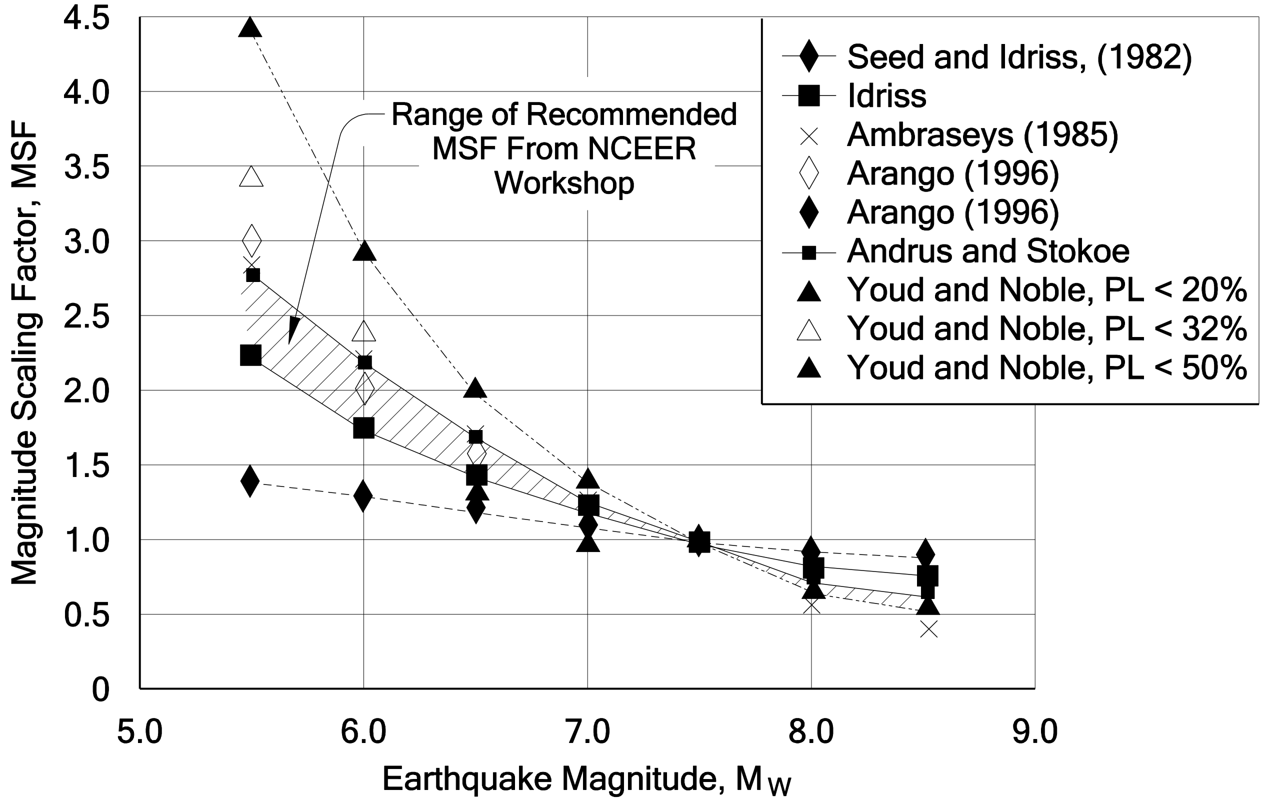


Figure 6-10. Magnitude Scaling Factors Derived by Various Investigators (redrafted from 1996 NCEER Workshop Summary Report)

It should be noted that the topic of Magnitude Scaling Factors has been the focus of considerable investigation over the past decade. Recent refinements to the MSF's have been made that account for soil density, soil-specific cyclic resistance (i.e., the slope of the cyclic resistance curve), and confining stress (e.g., Boulanger and Idriss, 2014). It is recommended that the most current procedures for evaluating soil liquefaction be considered for use on ODOT projects; however, refinements in one generation of the liquefaction triggering procedures should not be used with earlier methods, or with methods developed by different investigators. For example, the MSF's proposed by Bouldanger and Idriss (2014) should not be used with the liquefaction triggering procedure as presented by Youd et al (2001). The methods must be applied in a consistent manner following the procedures developed by the specific investigators.

<u>**Nonlinear Effective Stress Methods:**</u> An alternative to the simplified procedures for evaluating liquefaction susceptibility is to perform a nonlinear, effective stress site response analysis utilizing a computer code capable of modeling pore water pressure generation and dissipation (D-MOD2000, DESRA, FLAC). These are more rigorous analyses and they require additional soil parameters, validation by the practitioner, and additional specialization.

The advantages of this method of analysis include the ability to assess liquefaction at depths greater than 50 feet, the effects of liquefaction and large shear strains on the ground motion, and the effects of higher accelerations that can be more reliably evaluated. In addition, 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 can be used to estimate liquefaction susceptibility at depth. However, few of these programs are being used by geotechnical designers in routine practice at this time. In addition, there has been little verification of the ability of these programs to predict liquefaction at depths greater than 50 feet because there are few well documented sites of deep liquefaction. In addition, there is the potential for these programs to underestimate the liquefaction potential of near surface soils layers due to ground motion damping effects in underlying liquefied soil layers. This effect may be inherent in the program analysis and should be thoroughly evaluated.

Due to the highly specialized nature of these more sophisticated liquefaction assessment approaches, an independent peer review by an expert in this type of analysis is required to use nonlinear effective stress methods for liquefaction evaluation.

6.5.2.3 Liquefaction Induced Settlement

Both dry and saturated deposits of loose granular soils tend to densify and settle during earthquake shaking. Settlement of unsaturated (dry) granular deposits is discussed in Section 6.5.4. If the Simplified Procedure is used to evaluate liquefaction potential, liquefaction induced ground settlement of saturated sandy soils should be estimated using the procedures by Tokimatsu and Seed (1987), Ishihara and Yoshimine (1992) or more recent methods that have been documented in the technical literature (Zhang et al. 2002, Cetin et al, 2009, Tsukamoto and Ishihara, 2010). 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 in Figure 6-11 and Figure 6-12, respectively. Refer to Kavazanjian, et. al., (2011) for additional guidance on settlement analysis of liquefiable soils.

Non-plastic to low plasticity silts (PI ≤ 12) have also been found to be susceptible to volumetric strain following liquefaction. In cases where saturated silt is liquefiable the post-cyclic loading volumetric strain should be estimated from project-specific cyclic laboratory testing, or approximated from the relationships developed by Ishihara and Yoshimine.

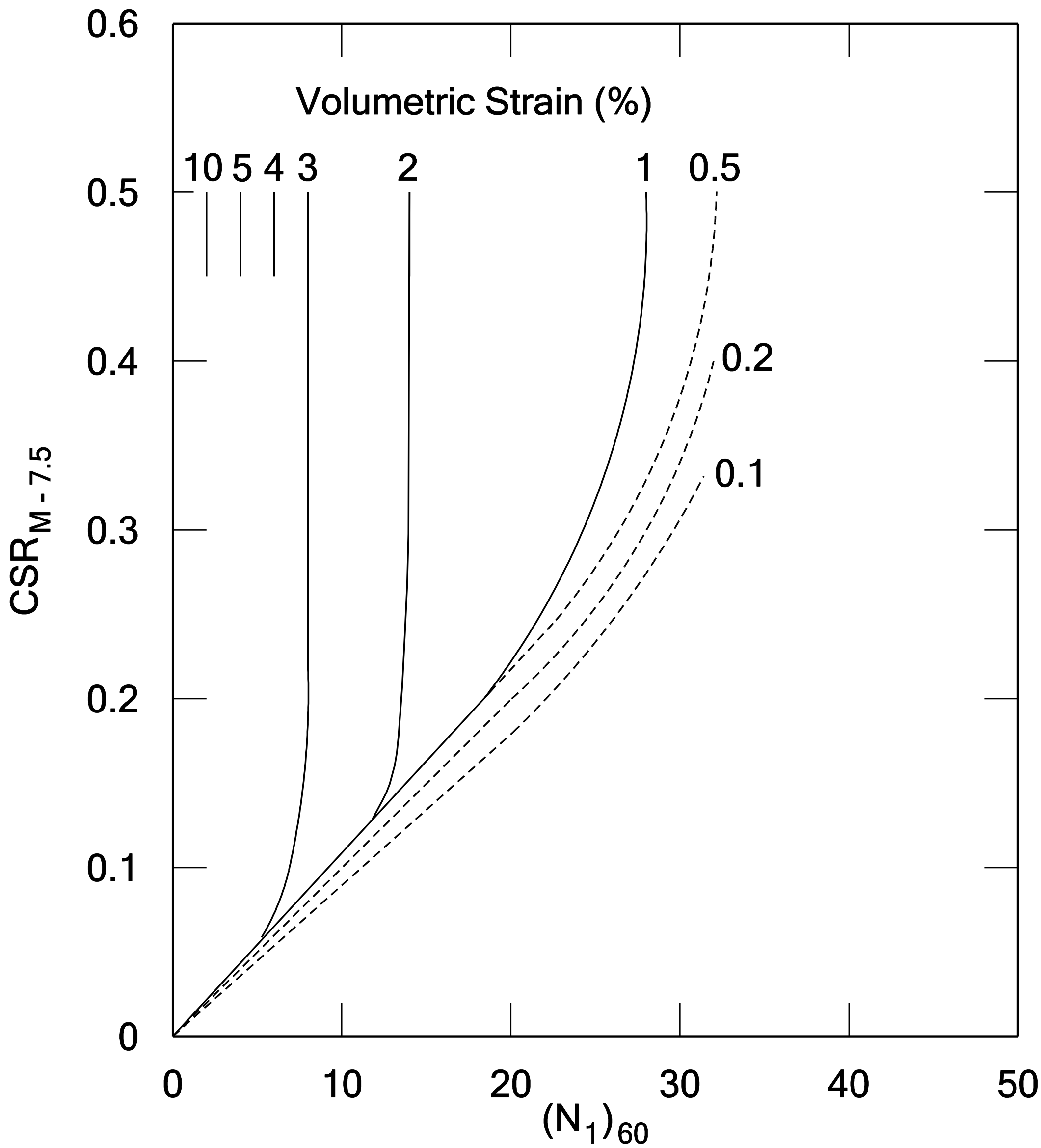


Figure 6-11. Post-liquefaction volumetric strain estimated using the Tokimatsu & Seed procedure (redrafted from Tokimatsu and Seed, 1987).

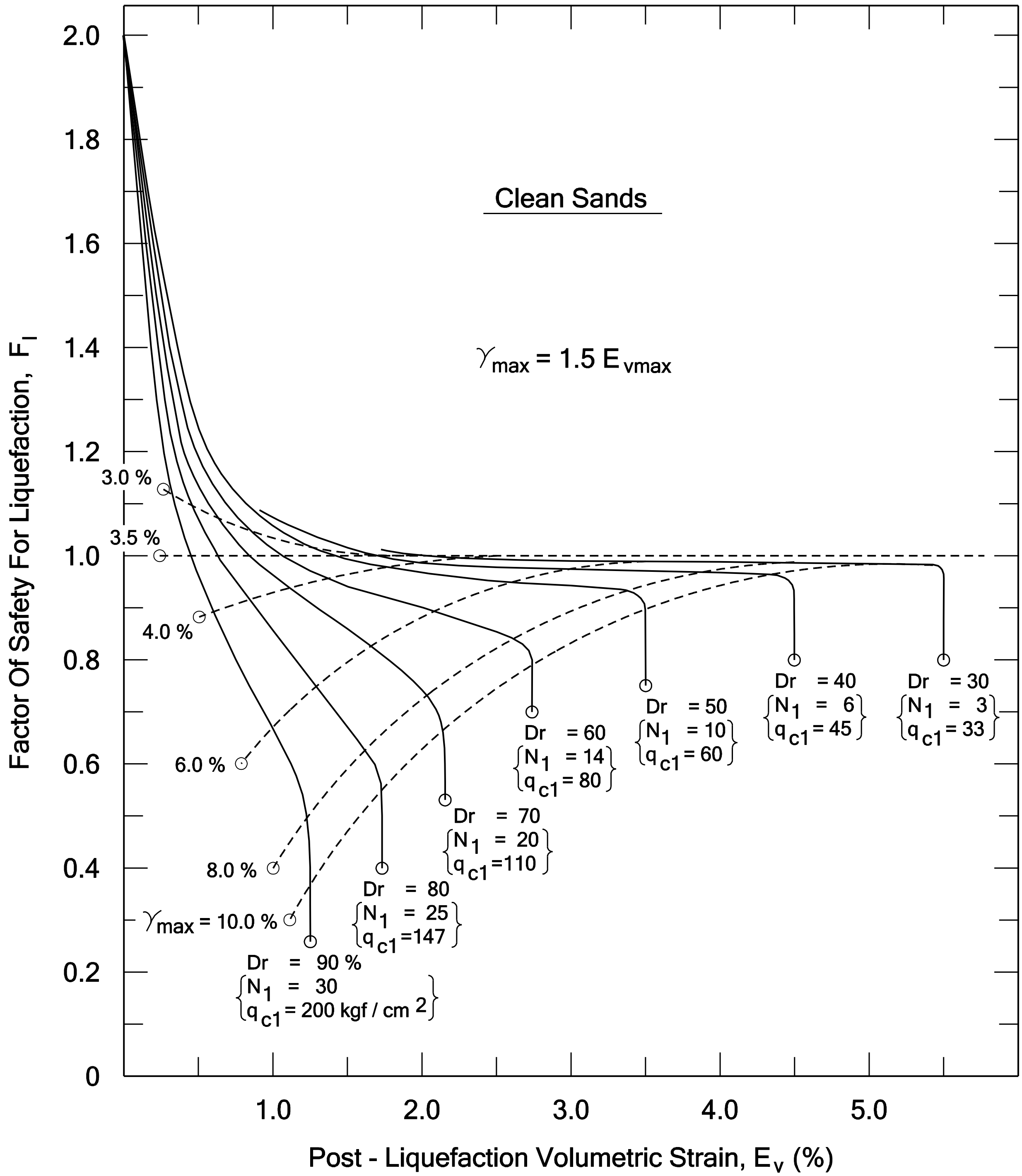


Figure 6-12. **Post-liquefaction volumetric strain estimated using the Ishihara and Yoshimine procedure. (redrafted from Ishihara and Yoshimine, 1992).**

6.5.2.4 Residual Strength Parameters

Liquefaction induced ground failure and foundation damage are 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. A variety of empirically methods based on back-calculated shear strengths from lateral spreads and flow failures are available to estimate the residual strength of liquefied sand. The procedures recommended in Section 6.4.1 should be used to estimate residual strength of liquefied sand. Other methods as described in Kramer (2008) may also be used.

All of these methods estimate the residual strength of a liquefied sand deposit based on an empirical relationship between residual undrained shear strength and equivalent clean sand SPT blowcounts or CPT q_{cl} values using the results of back-calculation of the apparent shear strengths from case histories, including flow slides. All of these methods should be used to calculate the residual undrained shear strength and an average value selected based on engineering judgment, taking into consideration the basis and limitations of each correlation method.

When laboratory residual shear strength test results are obtained and used for design, the empirically based analyses should still be conducted as a baseline evaluation to qualitatively check the reasonableness of the laboratory test results. The final residual shear strength value selected from the laboratory testing should also consider the amount of shear strain in the soil that can be tolerated by the structure or slope being impacted by the reduced shear strength (i.e., how much lateral deformation can the structure tolerate?).

6.5.3 Slope Stability and Deformation Analysis

Earthquake-induced ground motions imposed on sloping earth structures and native slopes can result in slope instability due to: 1) strength loss in the soil caused by increases in pore water pressures (cyclic degradation and/or full liquefaction), 2) inertial effects associated with ground accelerations, or 3) combinations of both. Inertial slope instability is caused by temporary exceedance of the soil strength by the combination of static shear stresses and the transient shear stresses imposed by the earthquake. In this case the soil strength remains generally unaffected by the earthquake shaking. In other cases the earthquake shaking results in the soil becoming progressively weaker to the point where the soil shear strength becomes insufficient to maintain a stable slope.

Seismic slope instability analysis is conducted to assess the impact of instability and slope deformation on structures such as bridges, tunnels, and walls. Slopes that do not impact such structures are generally not evaluated or mitigated for seismic slope instability.

The methods described in this section, in Kavazanjian et al., (2011) and in Anderson et al., (2008) should be used to assess seismic slope stability and for estimating ground displacements. The slopes and conditions requiring such assessments and analysis are described in Section 6.2.4.

6.5.3.1 Pseudo-static Analysis

A pseudo-static seismic slope stability analysis should be conducted at each bridge site regardless of whether or not liquefied soil conditions are predicted. The pseudo-static analysis shall consist of conventional limit equilibrium static slope stability analysis, using horizontal and vertical pseudo-static acceleration coefficients (k_h and k_v) as described in this section.

Pseudo-static analyses do not result in predictions of slope deformation and therefore are not sufficient for evaluation of bridge approach fill performance (such as meeting serviceability criteria) or for evaluating the effects of lateral embankment displacements on bridge foundations at the extreme limit state. The pseudo-static analysis is generally used to determine:

1) If the slope/embankment will be stable under the design seismic loading (i.e., there's a sufficient margin of safety against failure such that permanent deformations are likely within acceptable estimated deformations), in which case no further analysis will be necessary,
2) A yield acceleration for use in the Newmark (or other) analysis for estimating ground displacements, as described in Section 6.5.3.2, or
3) Whether or not a slope over liquefiable soils may fail in the form of a "flow failure" as described below.

Methods for conducting dynamic slope stability analysis under non-liquefied and liquefied conditions, and methods for determining embankment displacements under these conditions, are described in the following sections.

Non-liquefied Soil Conditions: If liquefaction of the foundation soils is not predicted, ground accelerations may still produce inertial forces within the slope or embankment that could exceed the strength of the foundation soils and result in slope failure and/or large displacements. At these sites a pseudo-static analysis, which includes earthquake induced inertia forces, is conducted to determine the general stability of the slope or embankment under these conditions. The pseudo-static analysis is also used to determine the yield acceleration for use in estimating slope or embankment displacements.

The soil inertia forces should be modeled using a horizontal pseudo-static coefficient, k_h, of 0.5As and a slope height reduction factor to account for wave scattering effects as described in Kavazanjian et al. (2011) and Anderson (2008). The vertical pseudo-static coefficient, k_v, should be equal to zero. For these conditions, the minimum allowable factor of safety (C/D ratio) is 1.1. Permanent seismic slope deformations of 1 to 2 inches can be anticipated under this condition. If the factor of safety is less than 1.1 but greater than 1.0, embankment displacements should be estimated using the Newmark methods described in Section 6.5.3.2 and the results evaluated in terms of meeting overall seismic performance requirements. For factors of safety equal to or less than 1.0, embankment stabilization measures should be designed and constructed to mitigate the condition and provide for a factor of safety of at least 1.1.

Liquefiable Soil Conditions: If soils vulnerable to cyclic degradation (liquefiable soils, sensitive soils, brittle soils) are present, slope instability may develop in the form of flow failures, lateral spreading or other large embankment deformations.

Flow failures are driven by large static stresses that lead to large deformations or flow following triggering of liquefaction. 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. For flow failures, both stability and deformation should be assessed and mitigated if stability failure or excessive deformation is predicted.

Conventional limit equilibrium slope stability analysis methods should be used to assess flow failure potential. Residual undrained shear strength parameters are used to model the strength of the liquefied soil. Under these liquefied soil conditions, slope stability is usually modeled in the "post-earthquake" condition without including any inertial force from the earthquake ground motions (a de-coupled analysis) and the horizontal and vertical pseudo-static coefficients, kh and kv, should both be set equal to zero.

Where the factor of safety is less than 1.05 flow failure shall be considered likely. In these instances, the magnitude of deformation is usually too large to be acceptable for design of bridges or structures, and some form of mitigation may be appropriate. The exception is where the liquefied material and crust flow past the structure and the structure can accommodate the imposed loads (see Section 6.5.5). Where the factor of safety is greater than 1.05, deformation and stability shall be evaluated using the lateral spread deformation analysis methods described in Section 6.5.3.2.

6.5.3.2 Deformation Analysis

Deformation analyses should be employed where estimates of the magnitude of seismically induced slope deformation are required. This is especially important for bridge approach fills where the deformation analysis is a crucial step in evaluating whether or not the bridge performance requirements described in Section 6.2 will be met.

Lateral spreading is the horizontal displacement that occurs on mostly level ground or gentle slopes (< 5 degrees) as a result of liquefaction of shallow sandy soil deposits. The soil can slide as intact blocks down the slope towards a free face such as an incised river channel. Lateral spreading, in contrast to flow failures, occurs 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. As a result of the slope instability, a failure surface resembling a sliding block typically develops along the liquefied soils and is subject to lateral displacements until equilibrium is restored. Lateral spreading at bridge approaches typically results in the horizontal displacement of the approach fill downslope or towards a free face. The resulting lateral movements can range in magnitude from inches to several feet and are typically accompanied by ground cracking with horizontal and vertical offsets. In contrast to flow failures, lateral spreading analysis is by definition a coupled analysis (i.e., directly considers the effect of seismic acceleration).

At sites where liquefaction is predicted, a lateral spreading/displacement analysis shall be conducted if the factor of safety for slope stability from a pseudo-static analysis, using post-earthquake soil strength parameters, is 1.05 or greater (no flow failure conditions). Lateral spread analysis does not need to be conducted if the depth below the natural ground surface to the upper boundary of the liquefied soil layers is greater than 50 ft.

Several approaches have been proposed for estimating lateral spreading displacements. Four of these approaches are described below for use in the assessment of lateral spread displacements. These four approaches are: 1) Empirical-based, 2) Semi-empirical based 3) Newmark-based and 4) Numerical Modeling methods. At sites where liquefaction is not predicted, lateral deformation analysis should be conducted using any of the Newmark based methods. For evaluation and estimates of lateral spread displacement a minimum of three methods, one taken from each approach, should be used to demonstrate a likely range of potential lateral displacements. This range of lateral displacements should then be used with engineering judgment to determine lateral spread displacement values to be used in the further assessment of bridge performance (i.e. foundation loading and meeting serviceability performance requirements).

Empirical-Based Approaches: Empirical models for lateral spreading displacements have been developed by using regression techniques with compiled data from lateral spreading case histories.

The following methods are recommended:

- Youd et al. (2002)
- Rauch & Martin (2000)

Input into the models include earthquake magnitude, source-to-site distance, and site geometry/slope, cumulative thickness of saturated soil layers and their characteristics (e.g. SPT "N" values, average fines content and average grain size). These methods are based on regression analysis of these input parameters, and other independent variables, correlated to field measurements of lateral spread. Therefore they are best applied to site conditions that fit within the range of variables used in the models. Care should be taken when applying these methods to sites with conditions outside the range of the model variables. These procedures provide a useful approximation of the potential magnitude of deformation that is calibrated against lateral spreading deformations observed in actual earthquakes. In addition to the cited references for each method, see Kramer (2008) for details on how to carry out these methods. These methods should be used primarily as a preliminary screening tool for assessing the general magnitude of lateral spread displacements. If the results of these methods indicate minimal lateral displacements which can be accommodated by the bridge foundation elements, and bridge design performance levels are satisfied, no further lateral spread analysis is required.

Semi-Empirical Approaches: Methods in this step include those that are semi-empirical in approach and more geomechanics based, requiring assessment of liquefaction potential and incorporating the results of laboratory testing into a cumulative strain model. Each method estimates the permanent shear strains that are expected within the liquefied zones (and nonliquefied zones, if warranted) and then integrates those shear strains over depth to obtain an estimate of the potential lateral displacement at the ground surface. The estimated lateral displacement may also be empirically adjusted on the basis of calibration to case history observations.

- Zhang et al. (2004)
- Idriss and Boulanger (2008)

Newmark-Based Analysis: The Newmark sliding-block approach consists of a seismic slope stability analysis that provides an estimate of seismically induced slope deformation (Jibson, 1993). In the Newmark time history analysis, lateral deformations are assumed to occur along a well-defined plane and the sliding mass is assumed to be a rigid block as shown in Figure 6-13. In this analysis, a standard slope stability analysis is first conducted, using the post-earthquake undrained residual shear strengths of the liquefied soil, to determine the yield acceleration of the slide mass (the pseudo-static acceleration that results in a factor of safety of 1.0). When the earthquake accelerations exceed this yield acceleration threshold, the sliding mass displaces. The total displacement is computed by double integrating the area of the accelerogram that lies above the yield acceleration line and summing these displacements for the duration of the earthquake.

Several analytical methods based on the Newmark sliding block model have been developed to estimate deformations induced by earthquake cyclic loadings. These Newmark-type methods typically fall into one of the following categories, simplified Newmark charts or Newmark Time-History Analysis.

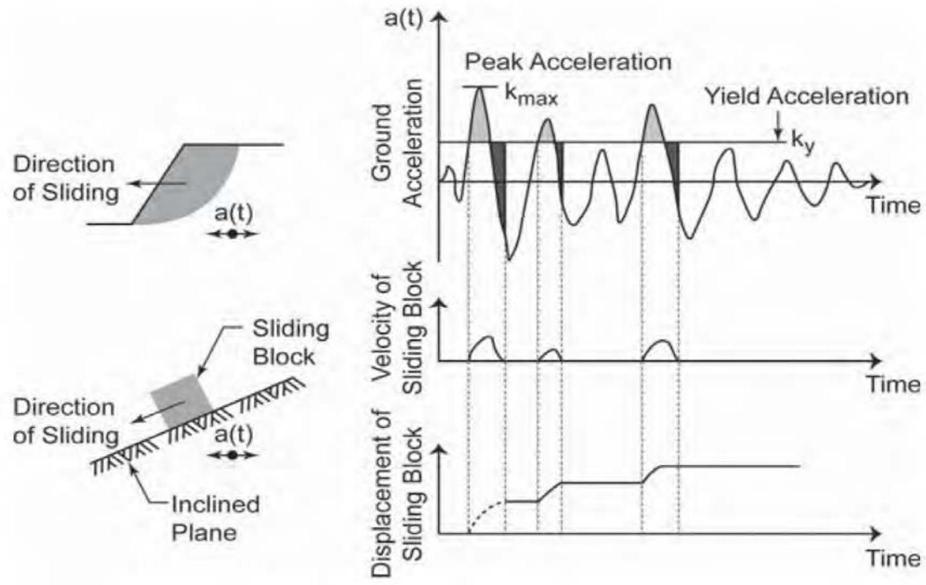


Figure 6-13. Newmark Sliding Block Concept for Slopes (Kavazanjian, et al. (2011).

Simplified Newmark Charts: Simplified Newmark charts were developed based on a large database of earthquake records and the Newmark Time History Analysis method. These charts relate an acceleration ratio (the ratio of the yield acceleration to the peak acceleration occurring at the base of the sliding mass) to horizontal ground displacement. The Newmark displacement method can also be performed using time history acceleration records if a site-specific seismic response is performed.

The simplified Newmark chart methods described in Anderson et al., (2008) and ATC-MCEER (2002) should be used for developing estimates of lateral spread displacements. These documents include worked examples and a discussion of which procedures are appropriate for specific conditions. Additional reference documents illustrating regional examples are provided in Dickenson et al., (2002) and Dickenson (2005).

The USGS computer program SLAMMER (Jibson, 2013), is also available to model slope performance during earthquakes using the Newmark method with various methods of analysis. This program allows for any combination of rigid-block, decoupled or fully coupled analysis to be conducted utilizing a large database of earthquake records. Simplified rigid-block analysis using empirical regression relationships to predict permanent displacements are also included.

The Newmark-based methods developed by Bray and Travasarou, (2007) and Saygili and Rathje, (2008) may also be used, are included in the SLAMMER program, and are described briefly below.

- **Bray and Travasarou, 2007:** This method is another modification, or enhancement, of the original Newmark sliding block model. It consists of a simplified, semi empirical approach for estimating permanent displacements due to earthquake-induced deviatoric deformations using a nonlinear, fully coupled, stick-slip sliding block model. In addition to estimating permanent displacements from rigid body slippage (basic Newmark approach) it also includes estimates of permanent displacement (deviatoric straining) from shearing within the sliding mass itself. The model can be used to predict the probability of exceeding certain permanent displacements or for estimating the displacement for a single deterministic event. This procedure is also available in EXCEL spreadsheet form.

- **Saygili and Rathje, 2008:** This method is another modification, or enhancement, of the original Newmark sliding block model, suitable for shallow sliding surfaces that can be approximated by a rigid sliding block. The model predicts displacements based on multiple ground motion parameters in an effort to reduce the standard deviation of the predicted displacements.

Newmark Time History Analysis: Newmark Time History Analysis is performed using the time history acceleration records developed form a site-specific ground response analysis. Note that in this type of analysis the yield acceleration is normally maintain at a constant value throughout the duration of the shaking. However, at sites with liquefiable soils the yield acceleration will be higher at the beginning of the analysis, before liquefaction has occurred, than at some time later in the record when cyclic degradation and strain softening has reduced the yield acceleration to lower values. In these cases, if the yield acceleration associated with partially, or fully, liquefied soil conditions is used throughout the analysis the resulting estimated displacements will be conservative.

The earthquake shaking that triggers the displacement is characterized by an acceleration record placed at the base of the sliding mass representing the design earthquake being evaluated. A minimum of seven independent earthquake records should be selected from a catalogue of earthquake records that are representative of the source mechanism, magnitude (Mw), and site-to-source distance (R). A sensitivity analysis of the input parameters used in the site-specific response analysis should be performed to evaluate its effect on the magnitude of the displacement computed. The results of the Newmark Time History Analyses should be compared with the results obtained using Simplified Newmark Charts.

The USGS computer program SLAMMER (Jibson, 2013), as described above, has the capability to perform time history Newmark analysis including decoupled and fully-coupled analysis of flexible sliding blocks.

Numerical Modeling of Dynamic Slope Deformation: Seismically induced slope deformations can also be estimated through a variety of dynamic stress-deformation computer models such as PLAXIS, DYNAFLOW, and FLAC. The accuracy of these models is highly dependent upon the quality of the input parameters. As the quality of the constitutive models used in dynamic stress-deformation models improves, the accuracy of these methods will improve. Another benefit of these models is their ability to illustrate mechanisms of deformation, which can provide useful insight into the proper input for simplified analyses.

Dynamic stress deformation models should not be used for routine design due to their complexity, and due to the sensitivity of the accuracy of deformation estimates from these models on the constitutive model selected and the accuracy of the input parameters. Use of dynamic stress-deformation computer models to evaluate seismically induced slope deformations requires the approval of the ODOT Bridge Section.

Numerical Modeling Correlations (GMI): In addition to the previously described empirical approaches, an additional simplified analysis method based on two dimensional numerical modeling of typical approach embankments using a finite difference computer code (FLAC) may be used as a screening and preliminary analysis tool for estimating lateral deformations of embankments over liquefied soils. This method, as presented in Dickenson et al. (2002), uses limit equilibrium methods to first calculate the post-earthquake factor of safety, using residual shear strengths in liquefied soils as appropriate. The resulting FOS is then used in combination with a Ground Motion Intensity (GMI = PGA/MSF) parameter to estimate embankment displacements. The GMI was developed to account for the intensity and duration of the ground motions used in the FLAC analysis. This procedure is also useful for estimating the amount, or area, of ground improvement needed to limit displacements to acceptable levels.

6.5.4 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'_{60} values. The step-by-step procedure is presented in Geotechnical Engineering Circular No. 3 (Kavazanjian, et al., 2011).

6.5.5 Liquefaction Effects on Structure Foundations

6.5.5.1 Bridge Approach Embankments

All bridge approach embankments should be assessed for the potential of excessive embankment deformation (lateral displacement and settlement) due to seismic loading and the effects of these displacements on the stability and functional performance requirements of the bridge. This is true whether liquefaction of the foundation soils is predicted or not. As a general rule, for the CSZE event (Operational Level), up to one (1) foot of lateral and 6 to 12 inches of vertical embankment displacement can be used as a general guideline for determining adequate performance of bridge approach embankments. This range of displacements should be considered only as a general guideline for evaluating the final condition of the roadway surface and the ability to provide a minimum of one-lane access to the bridge for emergency response vehicles following the earthquake. Always keep in mind the accuracy of the methods used to predict embankment deformations.

Bridge approach embankments are also commonly required to provide passive soil resistance to lateral loads that are transferred from the bridge superstructure to bridge abutments during earthquake events. This resistance is primarily provided by the backfill materials behind the abutments backwalls. This is the case for either seat-type abutments or for integral abutments. Liquefaction of foundation soils can result in settlement and/or lateral deformation of the backfill soils which can greatly reduce the ability of the backfill materials to provide the required passive soil resistance. The geotechnical engineer should evaluate the potential for this condition to occur, the possible design impacts, and consult with the bridge designer to determine the backfill passive resistance design requirements.

Lateral displacement and fill settlement will also produce loads on the bridge foundation elements which should also be evaluated in terms of providing the required overall bridge stability and performance. Specific embankment displacement limits are not provided for the 1000-year event since under this level of shaking the bridge and approach fills are evaluated only in terms of meeting the “No-Collapse” criteria.

6.5.5.2 General Liquefaction Policies Regarding Bridge Foundations

If liquefaction is predicted under either the 1000-year return or CSZE events, the effects of liquefaction on foundation design and performance must be evaluated. Soil liquefaction and the associated effects of liquefaction on foundation resistances and stiffness is generally assumed, in standard analyses, to be concurrent with the peak loads in the structure (i.e. no reduction in the transfer of seismic energy due to liquefaction and soil softening). This applies except for the case where a site-specific nonlinear effective stress ground response analysis is performed which takes into account pore water pressure increases (liquefaction) and soil softening.

Liquefaction effects include:

- Reduced axial and lateral capacities and stiffness in deep foundations,
- Lateral spread, global instabilities and displacements of slopes and embankments,
- Ground settlement and possible downdrag effects.

The following design practice, related to liquefied foundation conditions, should be followed:

- **Spread Footings:** Spread footings are not recommended for bridge or abutment wall foundations constructed over liquefiable soils unless ground improvement techniques are employed that eliminate the potential liquefaction condition,
- **Piles and Drilled Shafts:** The tips of piles and drilled shafts shall be located below the deepest liquefiable soil layer. Friction resistance from liquefied soils should not be included in either compression or uplift resistance recommendations for the Extreme Event Limit I state loading condition. As stated above, liquefaction of foundation soils, and the accompanying loss of soil strength, is assumed to be concurrent with the peak loads in the structure. If applicable, reduced frictional resistance should also be applied to partially liquefied soils either above or below the predicted liquefied layer. Methods for this procedure are presented Dickenson et al. (2002).

Pile Design Alternatives: Obtaining adequate lateral pile resistance is generally the main concern at pier locations where liquefaction is predicted. Battered piles have sometimes performed poorly at locations of lateral spreading and if considered the pile head connection must be designed for adequate ductility and to accommodate possible displacement demands. Prestressed concrete piles have not been recommended in the past due to problems with excessive bending stresses at the pile-footing connection. Vertical steel piles are generally recommended in high seismic areas to provide the most flexible, ductile and cost-effective pile foundation system. Steel pipe piles often are preferred over H-piles due to their uniform section properties, versatility in driving either closed or open ended and their potential for filling with reinforced concrete. The following design alternatives should be considered for increasing group resistance or stiffness and the most economical design selected:

- Increase pile size, wall thickness (section modulus) and/or strength,
- Increase numbers of piles,
- Increase pile spacing to reduce group efficiency effects,
- Deepen pile cap and/or specify high quality backfill around pile cap for increase capacity and stiffness,
- Design pile cap embedment for fixed conditions,
- Ground improvement techniques.

Liquefied P-y Curves: Studies have shown that liquefied soils retain a reduced, or residual, shear strength and this shear strength may be used in evaluating the lateral capacity of foundation soils. In light of the complexity of liquefied soil behavior (including progressive strength loss, strain mobilization, and possible dilation and associated increase in soil stiffness) computer programs

commonly used for modeling lateral pile performance under liquefied soil conditions often rely on simplified relationships for soil-pile interaction. At this time, no consensus exists within the professional community on the preferred approach to modeling lateral pile response in liquefied soil.

The following three options described below are recommended for modeling liquefied soils in lateral load (p-y) analysis. Refer to Rollins et al., (2005), Ashford, et al., (2012) and the other references provided for additional information on modeling liquefied or partial liquefied soil conditions.

1. **P-multiplier Approach**: This method uses a static sand model and the P-multiplier approach as presented in Caltrans (2013). In this approach, p-multipliers (m_p) are applied to the non-liquefied sand p-y curves to obtain the equivalent p-y curves for liquefied soil. Mid-range values of p-multipliers from the Brandenberg (2005) study, as shown on Figure 6-14, are recommended.
2. **Soft Clay Criteria**: This method, proposed by Wang and Reese (1998), utilizes p-y curves generated using the soft clay criteria (Matlock, 1970) with the undrained shear strength of the clay replaced by the residual shear strength of liquefied sand. It is recommended that ε_{50} = 0.05 be used when applying the soft clay procedure.
3. **Modified Sand Model**: This method modifies the static sand model(s) in the LPILE, or equivalent, program by using a reduced soil friction angle to represent the reduced, or residual shear strength of the liquefied soil. The reduced soil friction angle is calculated using the inverse tangent of the residual undrained shear strength divided by the effective vertical stress at the depth where the residual shear strength was determined or measured. The equation is:

   Equation 6.2:

   $$\varphi_{reduced} = \tan^{-1} (S_r/\sigma'_{vo}),$$

   Where S_r is the residual shear strength and σ'_{v0} is the effective vertical stress.

   Parameters representing the initial stiffness of the P-Y curves 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 the L-Pile and Group programs, 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.

Note that for partially liquefied conditions, the p-multipliers in Option 1 can be increased from those values shown in Figure 6-14, linearly interpolating between the values taken from the curves and 1.0, based on the pore pressure ratio, r_u, achieved during shaking (e.g., Dobry, et al., 1995). For Options 2 and 3, partially liquefied shear strengths may be used to calculate the reduced P_{ultliq} and corresponding p-y curves.

Other procedures can be used with approval by ODOT.

The modified soil parameters representing liquefied, or partially liquefied, soil conditions may be applied to either of the LPile GROUP, DFSAP or equivalent static soil models. 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, it should be noted 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 and is not recommended at this time. Liquefied sand p-y curves, based on full scale lateral load testing, are also available in the LPile and GROUP computer programs. This load test study (Ashford, et al., 2002) produced p-y curves for liquefied sand conditions that are fundamentally different than those derived from the standard static p-y curve models. The use of these liquefied p-y curves is not recommended at this time until further studies are completed and a consensus is reached on the use of these p-y curves in practice.

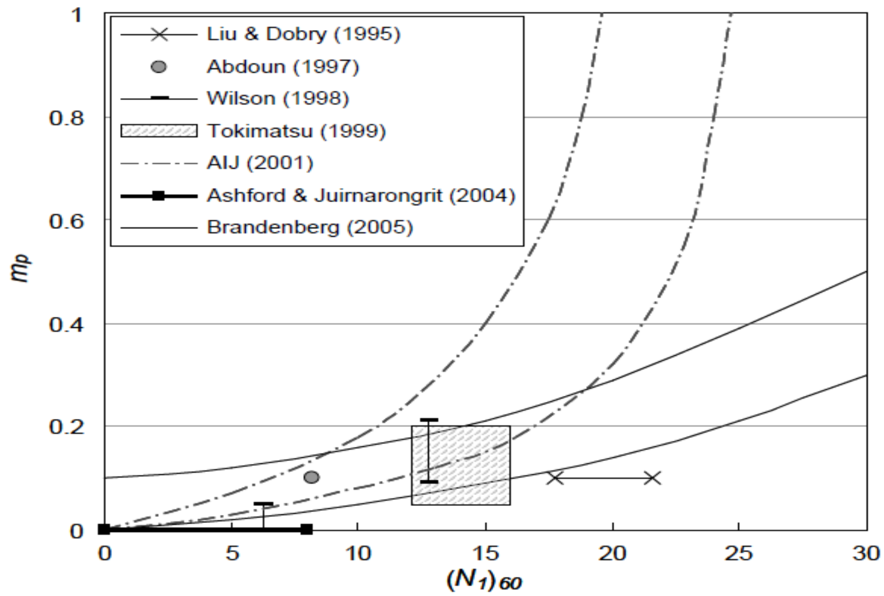


Figure 6-14, p-multiplier (m_p) vs. clean sand equivalent corrected blow count, $(N_1)_{60CS}$, from a variety of studies. (Ashford et al., 2012)

For pile or shaft groups within fully liquefied conditions, P-y curve group reduction factors may be set to 1.0. For partially liquefied conditions, the group reduction factors shall be consistent with the group reduction factors used for static loading.

T-Z curves: Modify either the PL/AE method or APILE Plus program as follows:

- For the PL/AE method, if the liquefied zone reduces total pile skin friction to less than 50% of the nominal bearing resistance, use "end bearing" condition (i.e. full length of pile) in stiffness calculations. Otherwise use "friction" pile condition.
- For the APile program, use the methods described for P-y curves to develop t-z (axial) or q-z (tip) stiffness curves for liquefiable soil layers.

Settlement and Downdrag Loads: Settlement of foundation soils due to the liquefaction or dynamic densification of unsaturated cohesionless soils could result in downdrag loads on foundation piling or shafts. Refer to Section 3.11.8 of *AASHTO (2014)* for guidance on designing for liquefaction-induced downdrag loads. Refer to Chapter 8 for guidance on including seismic-induced settlement and downdrag loads on the seismic design of pile and shaft foundations.

6.5.5.3 Lateral Spread and Flow Failure Loads on Structures

In general, there are two different approaches to estimate the induced load on deep foundations systems due lateral spreading or flow failures— a displacement based approach and a force based approach. Displacement based approaches 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 in the following sections.

6.5.5.4 Displacement Based Approach

The recommended displacement based approach for evaluating the impact of liquefaction induced lateral spreading and flow failure loads on deep foundation systems is presented in the ODOT research report titled, "*Reducing Seismic Risk to Highway Mobility: Assessment and Design Examples for Pile Foundations Affected by Lateral Spreading",* (Ashford, et. al., 2012). This approach provides methods to evaluate deep foundation systems that partially restrain the ground movement caused by lateral spreading/flow failure, and those foundation systems in which the ground can freely flow around them. Additional guidance on these procedures, including step-by-step design examples, are presented in Caltrans (2013). To be consistent with the design provisions in this GDM, the procedures described in Ashford, et. al., (2012) shall be modified as follows:

- Evaluate the liquefaction potential and lateral spread foundation load effects for both the 1000-year return event and the CSZE (if appropriate),
- Assessment of liquefaction potential shall be in accordance with Section 6.5.2.2,
- Determination of liquefied residual strengths shall be in accordance with Section 6.5.2.4,
- Lateral spread deformations shall be estimated using methods provided in Section 6.5.3.2,
- Deep foundation springs shall be determined using Section 6.5.5.2,
- Foundation performance shall meet the requirements in Section 6.2,
- Foundation moment and displacement demands shall meet the requirements specified in the ODOT BDDM. In-ground hinging and plastic failure of piles or shafts due to lateral spread and slope failures is not permitted on ODOT bridge projects for either the Life Safety or Operational performance level evaluations..

In cases where a significant crust of non-liquefiable material may exist, the foundation is likely to continue to move with the soil. Since large-scale structural deformations may be difficult and costly to accommodate in design, mitigation of foundation sub-soils will likely be required.

6.5.5.5 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 Refer to Yokoyama, et al., (1997) for background on this method. The pressures on pile foundations are simply specified as follows:

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

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

6.5.6 Mitigation Alternatives

The two basic options to mitigate lateral spread or flow failure 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): Refer to Sections 6.5.5.4 (displacement based approach) and 6.5.5.5 (force based approach) for more details on the specific analysis procedures for structural design mitigation options. The results of either the displacement or force-based approaches should be used to determine if it is feasible and economical for the structure to accommodate the estimated forces and/or displacements and provide the required design performance. Multiple design iterations may be required in this assessment. It is sometimes cost prohibitive to design the bridge foundation system to resist the loads imposed by liquefaction induced lateral spreading, especially if the depth of liquefaction extends more than about 20 feet below the ground surface and if a non-liquefied crust is part of the failure surface. If an acceptable level of design performance is not achievable through the structural option, then ground improvement should be considered.

Ground Improvement: The need for ground improvement techniques to mitigate liquefaction effects depends, in part, upon the type and amount of anticipated damage to the structure and approach fills due to the effects of liquefaction and embankment deformation (both horizontal and vertical). The performance criteria described in Section 6.2 should be followed. Ground improvement methods are described in Elias et al. (2006) and Chapter 11. All ground improvement designs required to mitigate the effects of soil liquefaction shall be reviewed by the HQ Bridge Section.

If, under the Operational performance level evaluation, the estimated bridge damage, or the estimated bridge approach fill displacements, are sufficient to render the bridge out of service for one lane of emergency traffic then ground improvement measures should be considered. If, under the 1000-year event, estimated bridge damage results in the possible collapse of a portion or all of the structure then ground improvement is required. A flow chart of the ODOT Liquefaction Mitigation Procedures is provided in Appendix 6-B.

Ground improvement techniques should result in reducing estimated ground and embankment displacements to acceptable levels. Mitigation of liquefiable soils beneath approach fills should extend a distance away, in both longitudinal and transverse directions, from the bridge abutment sufficient enough to limit lateral embankment displacements to acceptable levels. As a general rule of thumb, foundation mitigation should extend at least from the toe of the bridge end slope (or face of abutment wall) to the point where a 1:1 slope extending from the back of the bridge end panel intersects the original ground (Figure 6-15). The final limits of the mitigation area required should be determined from iterative slope stability analysis and consideration of ground deformations.

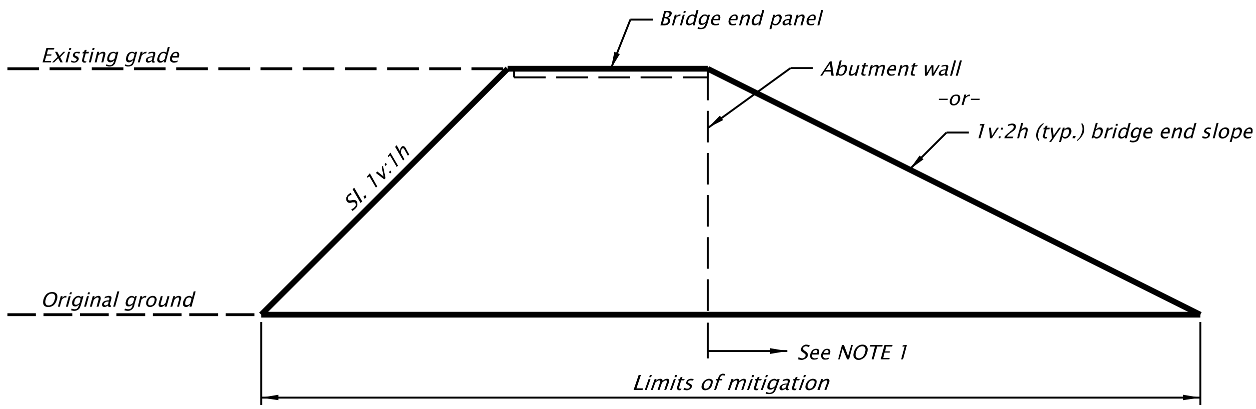


Figure 6-15. Extent of Ground Improvement for Liquefaction Mitigation

Ground improvement techniques should also be considered as part of any Phase II (substructure & foundation) seismic retrofit process. All Phase II retrofit structures should be evaluated for liquefaction potential and mitigation needs. The cost of liquefaction mitigation for retrofitted structures should be assessed relative to available funding.

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. Refer to GDM Chapter 11 for a more detailed discussion regarding the use and design of these and other ground improvement mitigation 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. Examples of 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 sandy soils susceptible to liquefaction, it may be possible to reduce the build-up of excess pore water pressures, and thus liquefaction during seismic loading. However, drainage improvement

is not considered adequately reliable by ODOT to prevent excess pore water pressure buildup due to the length of the drainage path, the time for pore pressure to dissipate, the

influence of fines on the permeability of the sand, 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 alone shall not be used as a means to mitigate liquefaction.

Geotechnical engineers are encouraged to work with ground treatment contractors having regional experience in the development of soil improvement strategies for mitigating hazards due to permanent ground deformation.

6.6 Input for Structural Design

6.6.1 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 placed in 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, design ground motions, and soil parameters such as dynamic soil shear modulus, Poisson's ratio, nominal bearing resistance, p-y curves and other parameters depending on foundation type. Refer to the *ODOT BDDM* for additional information on foundation modeling methods and the soil/rock design parameters required by the structural designer for the analysis. Additional guidance on the development of foundation springs can be found in Kavazanjian et. al., (2011) and Marsh, et. al., (2011) and their companion reports containing worked design examples.

6.6.1.1 Shallow Foundations

For evaluating shallow foundation springs, the structure designer generally 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 can be estimated using index properties and the correlations presented in Table 6-2. Alternatively, the maximum shear modulus can be calculated using Equation 6.3, if the shear wave velocity is known:

Equation 6.3:

$$G_{max} = \gamma /g(V_s)^2$$

Where:

G_{max} = maximum dynamic shear modulus

γ = soil unit weight

V_s = shear wave velocity

g = acceleration due to gravity

The maximum dynamic shear modulus (G_{max}) is associated with very small shear strains (less than 0.0001 percent). As the seismic ground motion level increases, the soil shear strain level increases and the dynamic shear modulus decreases. The effective shear modulus, G, to be used in developing shallow foundation springs, should be developed in accordance with AASHTO (2011) using the methods described in FEMA 356 (ASCE 2000). Table 4-7 in this document 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).

As an alternative, 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 ground response analysis. An effective shear strain, equal to 65 percent of the peak shear strain, should be used in this analysis. Laboratory test results may also be used to determine the relationship between G/G_{max} and shear strain.

Poisson's Ratio should be estimated based on soil type, relative density/consistency of the soils, and correlation charts such as those presented in Foundation Analysis and Design (Bowles, 1996).

6.6.1.2 Deep Foundations

Lateral soil springs for deep foundations shall be determined in accordance with Chapter 8. Refer to Section 6.5.5.2 for guidance on modifying t-z curves and the soil input required for P-y curves representing liquefied or partially liquefied soils.

6.6.1.3 Downdrag Loads on Structures

Downdrag loads on foundations shall be determined in accordance with Chapter 8.

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Appendix 6-A

FLOW CHART FOR EVALUATION OF LIQUEFACTION HAZARD AND GROUND DEFORMATION AT BRIDGE SITES

STEP 1

Identify Seismic Sources in the Region

CSZ interplate, deep intraplate, shallow crustal earthquakes refer to USGS Seismic Hazard Mapping Project Web Site
Obtain M-R pairs from de-aggregation tables for 475 and 975 mean return periods
Consider the following sources:

CSZ Interplate Earthquakes
M 8.3 and M 9.0
as defined by the USGS

Deep Intraplate Earthquake

- Very small contribution to PGA hazard in most of Oregon
- Confirm on De-Aggregation tables by checking for representative M-R pairs

Crustal, Areal, or "Gridded" Seismicity

- Obtain M-R pairs from USGS de-aggregation tables for all regional
- Define criteria for selecting all M-R pairs that significantly contribute to the overall seismic hazard

STEP 2

Select Appropriate Ground Motion Attenuation Relationships for each Source and Style of Faulting

Calculate the bedrock PGA values for each M-R pair

STEP 3

Select Appropriate Acceleration Time Histories for Bedrock Motions

- Three, or more, records from different earthquakes are recommended per M-R pair
- Consider style of faulting, magnitude, and the characteristics of the candidate motions (duration, frequency content, and energy)

STEP 4

Perform Dynamic Soil Response Analysis

- Develop profiles of cyclic stress ratio (CSR) versus depth for each M-R pair (3 or more time histories per M-R pair)
- Compute the average CSR profile with depth for each M-R pair
- Compute suite of Acceleration Response Spectra (ARS) if needed for structural engineering

STEP 5

Compute the Factor of Safety against Liquefaction for each M-R Pair

- Use the averaged CSR profile for each M-R pair
- Utilize standard methods for liquefaction susceptibility evaluation based on penetration resistance or shear wave velocity

STEP 6

Establish Post-Cyclic Loading Shear Strengths of Embankment and Foundation Soils

- This is performed for each M-R pair
- Focus on sensitive soils, weak fine-grained soils, loose to medium dense sandy soils *(potentially liquefiable soils are addressed as follows)*

If FSliq ≥ 1.4

Use drained shear strengths

If 1.4 > FSliq > 1.1

- Estimate the residual excess pore pressure
- Compute the equivalent friction angle

If FSliq ≤ 1.1

Estimate the residual undrained strength using two or more methods

STEP 7

Perform Slope Stability Analysis

- Static analysis using post-cyclic loading shear strengths for each M-R pair
- Calculate the FOS against sliding and determine the critical acceleration values for each M-R pair
- Focus trial slip surfaces on weak soil layers

STEP 8

Perform Deformation Analysis for each M-R pair

- Empirical-Based Approaches
- Semi-Empirical Approaches
- Simplified Newmark Chart Solutions
- Newmark Time-History Analysis
- Numerical modeling

STEP 9

Evaluate Computed Deformations in Terms of Tolerable Limits

Permanent Deformations are Acceptable

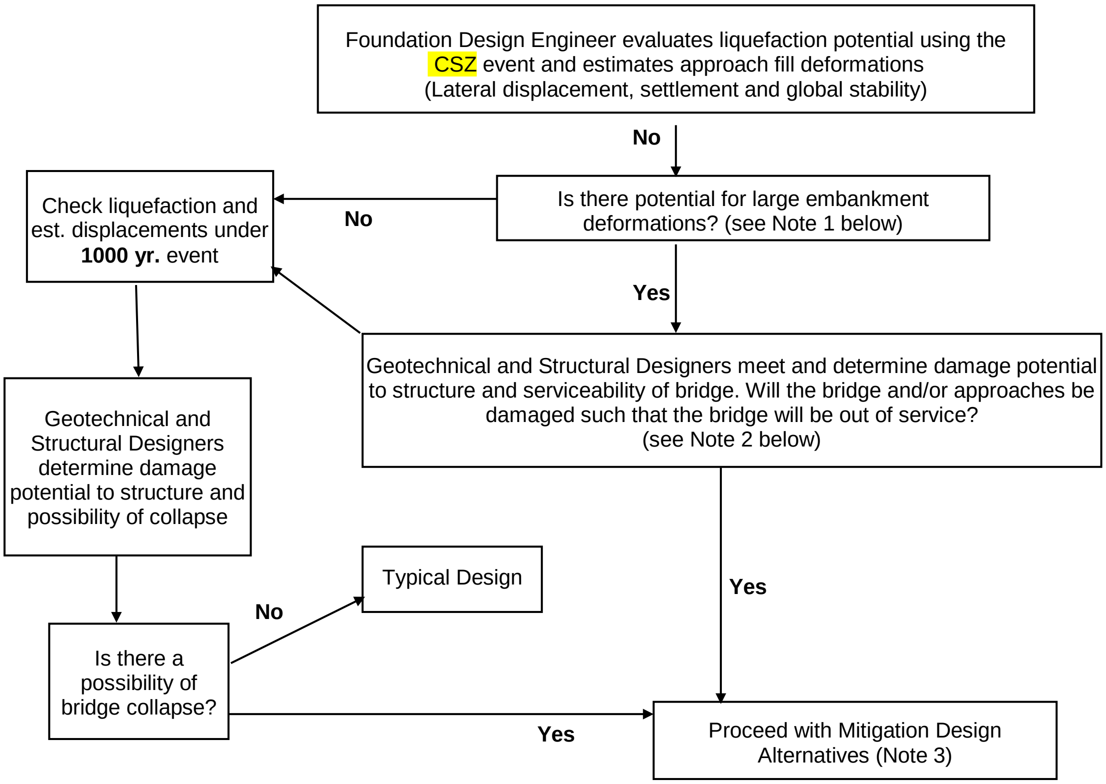
- Computed displacements are less than defined limits
- Continue with structural design

Permanent Deformations are Unacceptable

- Computed displacements exceed defined limits; Pursue mitigation alternatives using modified structure design option, ground improvement or combination of both. Perform analysis iterations until acceptable deformations are achieved and all structural design requirements are met.
- If ground improvement substantially alters the dynamic response of the site (e.g., extensive soil improvement in the vertical and lateral direction, extensive treatment including grouting or deep soil mixing) return to Step 4.
- A reduced number of input time histories are acceptable for each M-R pair (bracket the problem using trends from the initial analysis)

Appendix 6-B

ODOT Liquefaction Mitigation Procedures

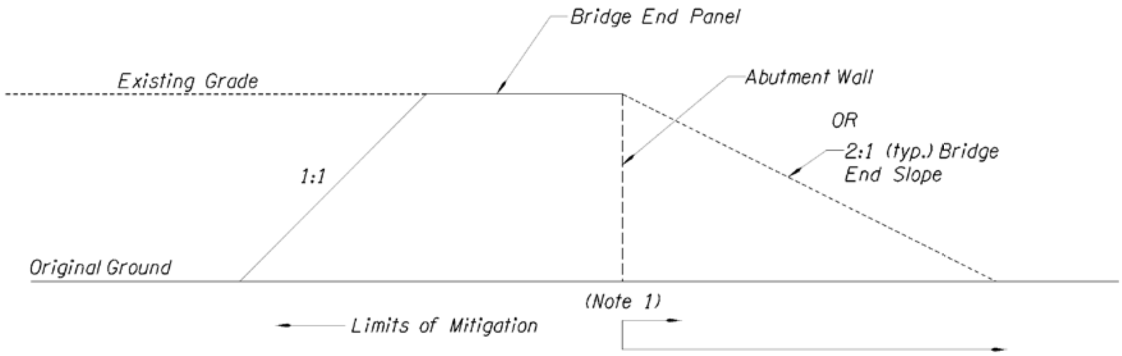


Note 1: For meeting the performance requirements of the CSZ event (Operational Level)), lateral deformation of approach fills of up to 12" are generally considered acceptable under most circumstances pending an evaluation of this amount of lateral deformation on abutment piling. Larger lateral deformations and settlements may be acceptable under the 1000 year event as long as the "no-collapse" criteria are met.

Note 2: The bridge should be open to emergency vehicles after the CSZ design event, following a thorough inspection. If the estimated embankment deformations (vertical or horizontal or both) are sufficient enough to cause concerns regarding the serviceability of the bridge, mitigation is recommended.

Note 3: Refer to GDM Section 6.5.6, ODOT research report SRS 500-300: "*Reducing Seismic Risk to Highway Mobility: Assessment and Design Examples for Pile Foundations Affected by Lateral Spreading*", December, 2012 and FHWA NHI-06-019 and 020 reports; "*Ground Improvement Methods, Volume I & II*" for mitigation alternatives and design procedures (Elias et al., 2006).

As a general guideline, the foundation mitigation should extend from the toe of the bridge end slope, or face of abutment wall, to the point that is located at the base of a 1:1 slope which starts at the end of the bridge end panel:



Note 1: Extend ground improvement beyond the abutment face as needed for design.