ODOT SEISMIC FOUNDATION DESIGN PRACTICE

ODOT BRIDGE ENGINEERING SECTION
October, 2005

1.0 Introduction

Like many of the geosciences, seismic foundation design is an evolving field. New design methods and techniques are continuously developing based on various research projects and activities. In this climate, it is often difficult to define specific design methods for use in the seismic foundation design process. However, a standard of practice needs to be established among foundation designers regarding seismic foundation design practice. It is well recognized that these standards are subject to change in the future as a result of further research and studies. AASHTO Guidelines for the Seismic Design of Highway Bridges are in final review and may be approved and implemented in the near future. This document is therefore a working document by necessity, and will be continually updated as new design code is approved and better design methods are developed.

The intent of this document is to provide foundation designers with specific design details, guidance, recommendations and policies not provided in other standard design documents. Complete design procedures (equations, charts, graphs, etc.) are not usually provided unless necessary to supply, or supplement, specific design information, or if they are different from standards described in other references. It is a place to document all relevant information regarding seismic foundation design practices within ODOT and also describe what seismic recommendations should typically be provided by the Foundation Designer to the Bridge Designer. References are provided at the end of this document.

It must be understood that a large amount of engineering judgment is required throughout the entire seismic design process. The recommendations provided herein are intended to provide a basis for standardization of design practices. These recommendations are not intended to be construed as complete or absolute. Each project is different in some way 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.

Earthquakes often result in the transfer of large axial and lateral loads from the bridge superstructure into the 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 design to ultimate foundation capacities and provide for a factor of safety as low as 1.0. This design practice results in an increased emphasis on quality control during the construction of bridge foundations since we are now relying on the full, unfactored resistance of each foundation element to support the bridge during the design seismic event.

In addition to seismic foundation capacity analysis, seismic structure 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 to do this analysis. Refer to Section 1.1.4 of the ODOT Bridge Design and Drafting Manual (BDDM) (ref. 2) for more information on bridge foundation modeling procedures.

2.0 General Design Guidelines & Policy

In general, seismic foundation design of ODOT structures will follow methods described in the AASHTO Standard Specifications for Highway Bridges (ref. 1), Division I-A, supplemented by the recommendations supplied in this document. The FHWA design manuals: "Design Guidance: Geotechnical Earthquake Engineering For Highways, Volumes I & II" (ref. 4), are also used for reference in seismic design. For liquefaction analysis, embankment deformation estimates and bridge damage assessment the following two documents should be referenced:

1) "Assessment and Mitigation of Liquefaction Hazards to Bridge Approach Embankments in Oregon", OSU Research Report (ref. 8),

2) "Recommended Guidelines For Liquefaction Evaluations Using Ground Motions From Probabilistic Seismic Hazard Analysis", OSU Paper (ref. 10)

These last two documents are available (October, 2005) on the ODOT Bridge Engineering web page under the heading: "Bridge Standards and Manuals". The web address is:

http://www.oregon.gov/ODOT/HWY/BRIDGE/standards_manuals.shtml

<u>Seismic Performance Requirements</u>
A two level approach is used in ODOT for the seismic design of all replacement bridges. The seismic design of ODOT bridges is evaluated in terms of performance requirements for both the 500-year and the 1000-year return events. The seismic foundation design requirements should be consistent with meeting the current ODOT Bridge Engineering Section seismic design criteria which is summarized as follows from the BDDM:

*<u>1000-year "No Collapse" Criteria:</u> Design all bridges for a 1000-year return period under a "no collapse" criteria. To satisfy the "no collapse" criteria, use Repose Modification Factors from Table 3.10.7.1-1 of the <u>AASHTO LRFD Bridge Design Specifications</u> using an importance category of **"other"**. Contrary to 3.10.2 in the <u>AASHTO LRFD Bridge Design Specifications</u>, use the bedrock acceleration coefficient from the ODOT 1000-year PGA map (see Figure 1.1.10.1A). When requested in writing by the local agency, the bedrock acceleration coefficient for local agency bridges may be taken from the ODOT 500-year PGA map.*

500-year "Serviceable" Criteria: In addition to the 1000-year "no collapse" criteria, design all bridges to remain "serviceable" after a 500-year return period event. To satisfy the "serviceable" criteria, use Response Modification Factors from Table 3.10.7.1-1 of the AASHTO LRFD Bridge Design Specifications using an importance category of ***"essential"****. Contrary to 3.10.2 in the AASHTO LRFD Bridge Design Specifications, use the bedrock acceleration coefficient from the ODOT 500-year PGA map. When requested in writing by the local agency, the "serviceable" criteria for local agency bridges may be waived.*

500-year Event Criteria

Under this level of shaking the bridge, and approach fills leading up to the bridge, are designed to provide access for emergency vehicles immediately following the event. This is consistent with the AASHTO definition of an "essential" bridge. In order to do so, the bridge is design to respond semi-elastically under seismic loads with minimal damage. Some structure 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 vehicle access for at least one travel lane. Approach fills are defined as the roadway embankment within 60 to 100 feet of the bridge end. As a general rule of thumb, an estimated lateral embankment displacement of up to 1 foot is considered acceptable in many cases. Allowable vertical settlements may be on the order of 6" to 12" depending on the anticipated performance of the impact panel. These allowable displacements are to serve as general guidelines only and a good deal of engineering judgment is required to determine the final allowable displacements that will meet the desired criteria. It should be noted that the estimation of lateral embankment displacement is far from an exact science and estimates may easily vary by an order of magnitude or more depending on the method(s) used. The actual amounts of allowable vertical and horizontal displacements should be decided on a case-by-case basis, based on discussions between the bridge designer and the foundation designer and perhaps other project personnel. If liquefaction mitigation is required to meet the required criteria refer to Section 7.

1000-year Event Criteria

Under this level of shaking the bridge, bridge foundation and approach fills must be able to withstand the forces and displacements without collapse of any portion of the structure. In general, bridges that are properly designed and detailed for seismic loads can accommodate relatively large seismic loads and deflections without collapse. If large embankment displacements (lateral spread) or global failure of the end fills are predicted, the impacts on the bridge and adjacent interior piers should be evaluated to see if the impacts could potentially result in collapse of any part of the structure.

Factors of Safety (FOS)

For seismic loading conditions (Extreme Event I) the AASHTO Standard Specifications for Highway Bridges allows ultimate capacities (FOS = 1.0) to be used for all foundation types and ODOT has generally adopted this policy. However, ODOT design practice includes a small factor of safety for pile/shaft uplift capacity due to the reduced overburden stress in the upper soil layers. The FOS for pile & shaft uplift capacity should be 1.10.

The seismic design of pile foundations often relies on the ultimate axial capacity of the piles (both in compression and tension). The ODOT Gates Equation and the Wave Equation are the most commonly used methods to develop pile driving resistance criteria. However, different factors of safety (FOS) are used with each of these methods. For a given allowable capacity, this will result in different ultimate capacities being required for each method. The higher FOS of 3.0 used with the Gates Equation is because it is typically considered a poorer predictor of capacity. This higher (Gates Equation) ultimate capacity should not be used for seismic design since it is really required due to the poor prediction capability of the equation. The Wave Equation method should be used for predicting ultimate pile capacity in areas where the pile design may be controlled by seismic loads. As a general rule of thumb, this is in areas where the site bedrock PGA is greater than 0.20g.

3.0 Ground Motion Data

The ground motion values to be used in design are based on the 2002 USGS Seismic Hazard maps for the Pacific Northwest region. These maps are available in the ODOT Bridge Design and Drafting Manual (BDDM) and also available on the USGS Seismic Hazard Maps internet web page at:

http://earthquake.usgs.gov/hazmaps/

The BDDM maps are contour maps for Peak Ground Acceleration (PGA), 0.20 sec. and 1.0 sec. spectral accelerations scaled in contour intervals of 0.01g. In some areas of the state such as the southern Oregon coast the contours may be too close together to read and interpret. In these cases the ground motion values should be obtained from the USGS web page by selecting the "Custom Mapping and Analysis Tools" link and then the "Interactive Deaggregation, 2002" link. The PGA (or spectral accelerations) can then be obtained by entering the latitude and longitude of the site and the desired probability of exceedance (i.e. 5% in 50 years for the 1000 year return event).

Magnitude and PGA for Liquefaction Analysis

For liquefaction analysis, an earthquake magnitude is needed, to use in combination with the PGA or cyclic shear stress ratios to perform the analysis. The PGA and magnitude values selected for the analysis should represent realistic ground motions that could actually occur at the site due to known active faults in the area. 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 totally dominated by a single source. Otherwise the "mean" PGA values may not represent realistic ground motions resulting from known sources affecting the site. Typically, a deaggregation of the total seismic hazard should be performed to find the individual sources contributing the most to the seismic hazard of the site. As a general rule of thumb, all sources that contribute more than about 5% to the hazard should be evaluated. 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.

A deaggregation of the seismic hazard will provide the Magnitude (M) and the Distance (R) of each source contributing hazard to the site. These M & R values are then used along with attenuation relationships to obtain bedrock PGA values for the site, which are then used in the liquefaction analysis. It is important to note that the PGA values obtained from this procedure will not necessarily be the same as the "mean" PGA values used in the structural analysis. This deaggregation process will likely yield more than one M-PGA pair for liquefaction analysis in some areas of the state where there are significant crustal sources and also significant influence from the Cascadia Subduction Zone (CSZ) event. Each 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.

The steps involved in the deaggregation process and liquefaction analysis are outlined in the OSU paper titled: "Recommended Guidelines For Liquefaction Evaluations Using Ground Motions From Probabilistic Seismic Hazard Analysis"(ref. 10). Four example problems are provided in this paper for different areas of the state, demonstrating the deaggregation procedure. A recommended procedure for estimating lateral embankment deformations is also included in this paper along with two example problems. A flow chart of this process, copied from the OSU paper, is attached in the appendix of this document.

4.0 Site Investigations for Seismic Designs

In addition to the standard subsurface investigation methods described in the AASHTO Manual on Subsurface Investigations, 1988, (ref. 3), the following soil testing and/or sampling should be conducted depending upon site conditions. Refer to the FHWA manual "Geotechnical Engineering Circular No. 3; "Design Guidance: Geotechnical Earthquake Engineering for Highways", Volume I, Chapter 5 (ref. 4) for additional guidance.

- SPT Hammer Energy – This value (usually termed hammer efficiency) should be noted on the boring logs or in the Foundation Report. This is needed to determine the hammer energy correction factor, Cer, 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 Su, e_{50}, E, G, and other parameters for both foundation modeling and seismic design.

- Pressuremeter Testing - For development of p-y curves if soils cannot be adequately characterized using standard COM624P or LPILE parameters. 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.

- Shear Wave Velocity Measurements - Used to develop a shear wave velocity profile of the soil column and to obtain low strain shear modulus values to use in a ground response (SHAKE) analysis. Also for use in determining soil amplification factors. For downhole measurements, a PVC pipe may be installed in exploratory bore holes for later testing if necessary.

- Seismic Piezocone Penetrometer - Used to develop a shear wave velocity profile and obtain low strain shear modulus values to use in a ground response (SHAKE) analysis.

- Cone Penetrometer Testing (CPT) - 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.

- Depth to Bedrock – If a site specific analysis is to be performed, the depth to bedrock must be known. “Bedrock” material for this purpose is defined as a material unit with a shear wave velocity of at least 2500 ft./sec.

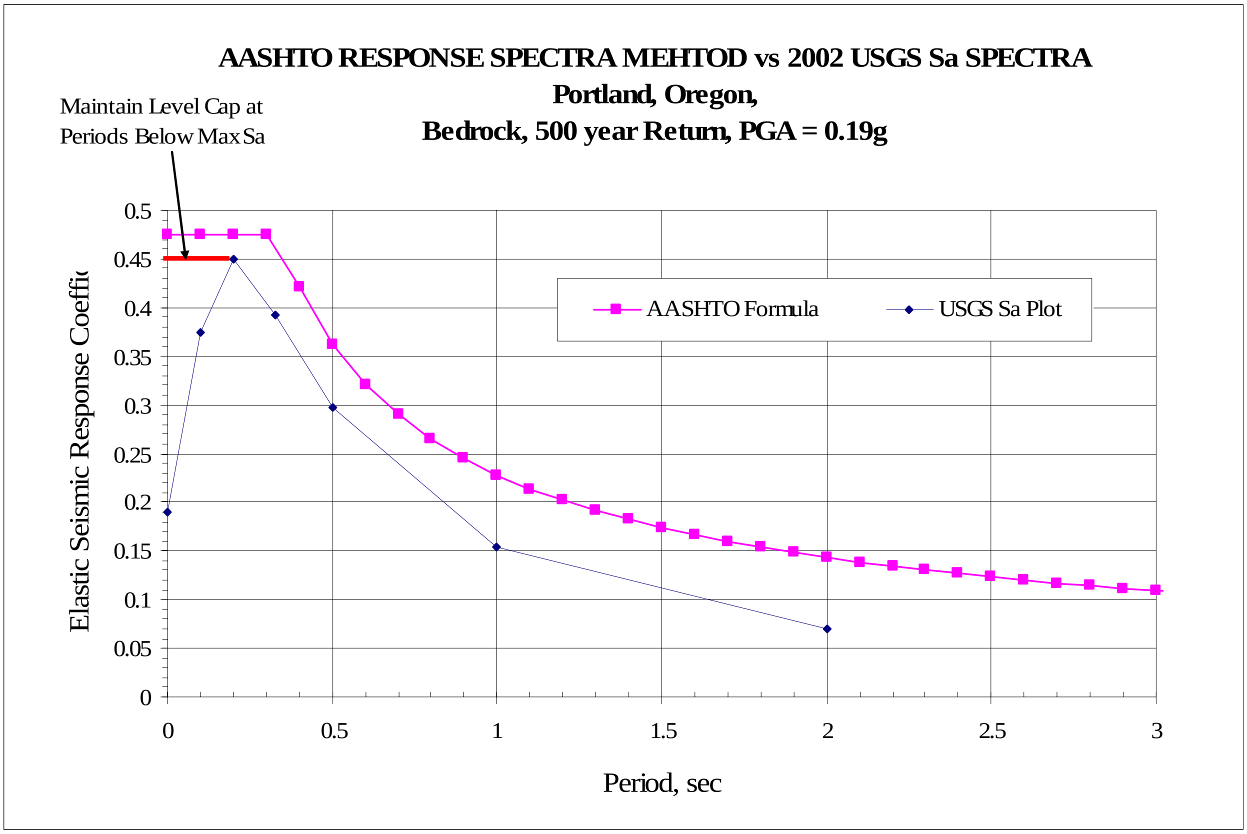
5.0 Response Spectra Development

Response spectra may be developed from one of the following three methods:

1. Standard AASHTO formula
2. Using 2002 USGS Spectra Acceleration values
3. A site specific response analysis (SHAKE)

For most sites the standard AASHTO formula for calculating the Elastic Seismic Response Coefficient, can be used (Article 3.6, ref. 1). This formula simply uses the bedrock PGA at the site (from the USGS maps) and the soil site coefficient.

As an alternative to the standard AASHTO formula it is also acceptable to use spectra acceleration values (Sa) from the USGS web site to generate a more “site specific” bedrock response spectra. An example is provided below. Spectra accelerations for periods of 0.20 sec. and 1.0 sec. are provided in the BDDM and additional Sa values may be obtained from the USGS web site. The spectra produced from the USGS Sa values is for bedrock and AASHTO soil coefficients (amplification factors) must be applied to these spectra to account for site effects.



The third method is a site specific response analysis which evaluates the response of a layered soil deposit subjected to earthquake motion. In general, the Equivalent-Linear One-Dimensional method is the preferred method of choice if the site conditions fit the program model. Typically the program SHAKE91 is used to generate response spectra, peak ground surface acceleration and other information for use in design. The program calculates the induced cyclic shear stresses in individual soil layers for use in liquefaction analysis. The program SHAKE2000 is available for use in running the SHAKE91 program and contains additional post processing analysis tools for plotting response spectra, computing average response spectra, calculating liquefaction potential and estimating lateral ground deformation.

The procedure for conducting a site specific response analysis is described in the flow chart on page 10. For more details regarding the methods available for conducting this analysis, refer to Chapter 4 of FHWA Publication FHWA-SA-97-076 titled: “Design Guidance: Geotechnical Earthquake Engineering For Highways” (ref. 4).

A site response analysis (SRA) may be warranted at a site due to several contributing factors. Engineering judgment is a key element in determining whether or not a SRA should be conducted. Factors to consider in determining whether or not a SRA should be conducted include the following:

- Relatively High Seismic Hazard Area (PGA > 0.30g).
- Very deep, unusual or highly variable soil conditions. These are sites where the foundation soils do not fit the standard AASHTO soil profile types and stratigraphy. Examples are high plasticity clays (>25' thick), >125' of soft-medium stiff clays, highly organic clays and peat or thick diatomaceous soils.
- Marginal liquefaction conditions. It may be necessary to refine the standard liquefaction analysis based on Seed's simplified (SPT) method (or others) with information from a SRA. 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.

Some additional reasons for performing a site specific SRA is to:

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

At least 3 time histories should be used for each SHAKE analysis. In areas where the hazard has a significant contribution from both the Cascadia Subduction Zone (CSZ) and from crustal sources (i.e. Portland and much of the Western part of the state) both earthquake sources need to be included in the development of a site specific response spectra. This is because the short period spectral accelerations are strongly influenced by the crustal sources and the long period response is more heavily influenced by the CSZ.

For a CSZ analysis, the PGA should be determined from the M-R values obtained from the USGS web site and attenuation relationships for the CSZ event. Time histories from subduction zone events should then be selected and scaled to this PGA. Do not use the "mean" PGA from the USGS web site to scale CSZ time histories unless the site is on the coast and the "mean" PGA is completely dominated by the CSZ. Likewise, in areas where crustal sources dominate the hazard (areas other than the coast), the "mean" PGA value from the USGS maps can typically be used to scale the time histories. Time histories for crustal earthquakes should be selected that reflect magnitude and distance values that are as close as possible to the M-R values from the USGS deaggregation tables. The top 2 or 3 M-R pairs from the deaggregation tables should be evaluated to see if the range of M and R values is small enough that perhaps only one M-R pair is really needed for selecting time histories. The time histories selected should be from earthquakes that are similar to the major contributing sources identified in the deaggregation analysis in terms of magnitude, type of faulting, PGA and geology. Five percent (5%) damping is typically used in all site specific analysis.

Geologic conditions such as basin effects, near-source effects ("pulsing"), wave propagation direction, irregular formation discontinuities, stratigraphic profile and other conditions should be considered in this analysis. All of these geologic and seismic conditions are difficult to characterize for seismic foundation design but should be taken into account when developing the final site specific, smoothed response spectra. The need for a more in-depth analysis involving additional site investigation work and computer modeling techniques that take these conditions into account should be weighed against the potential benefits.

Output from the SHAKE2000 program can include the mean, average and 85th percentile curves from all the output response spectra. A “smoothed” response spectra is obtained from the SHAKE2000 program by drawing a horizontal line through the peaks of the 85% percentile response curve, using engineering judgment. The 85th percentile curve does not have to be totally capped by the horizontal line. The spectrum decay curve is typically a function of 1/T, again roughly capped through the peaks of the 85% percentile spectrum. An example response spectrum is attached in the Appendix.

The SHAKE program may overemphasize spectral response where the predominant period of the soil profile closely matches the predominant period of the bedrock motion. Try and use a range of earthquake records that do not result in an extreme bias in the computed results. Compare the smoothed response spectra to spectra developed from the other methods available in the SHAKE2000 program, such as AASHTO, IBC, and NEHRP methods. Further adjust the final smoothed spectra as necessary based on engineering judgment.

The peaks of the individual response spectra are very important, especially as related to the structure’s predominant period. The average predominant period of the site should be reported as well as the smoothed response spectrum.

Response spectra developed using SHAKE, or other, ground response analysis programs may be used for design regardless of whether it is higher or lower than the response spectra developed using the standard AASHTO criteria. There is no upper limit recommended, however, a lower limit of no less than 2/3 of the AASHTO spectra is recommended.

The subsurface conditions (soil profile) may change dramatically in some cases along the length of a bridge and more than one response spectrum may be required to represent segments of the bridge with different soil profiles.

SITE RESPONSE ANALYSIS
(Development of Average Response Spectra, ARS)

STEP 1

Identify the seismic sources in the region affecting the site for the given return period (500 or 1000 yrs):
- Use the 2002 USGS Seismic Hazard Maps
- CSZ (Inter-plate), Deep Intra-plate, Shallow Crustal Earthquakes

STEP 2

Determine the Magnitude and Source-to-Site Distance for each of the relevant sources
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STEP 2a

CSZ Inter-plate Earthquake
- Specify M given Return Period
- Determine source-to-site distance
- Estimate PGA using several attenuation relationships

STEP 2b

Deep Intra-plate Earthquake
- Specify M given Return Period
- Determine source-to-site distance
- Estimate PGA using several attenuation relationships

STEP 2c

Crustal, Areal, or "Gridded" Seismicity
- Specify the M values for all regional sources
- Use the "mean" PGA value*
- Account for fault-specific sources and random, areal sources

The magnitude(s) can be determined from the seismic hazard de-aggregation tables available from the USGS Seismic Hazard Program web site. A selection criterion should be established for specifying the M-R pairs that require evaluation. Each M-R pair should be considered in terms of the percentage contribution that it makes to the cumulative seismic hazard. The percentage contribution must be evaluated for the PGA and spectral components.

**Appropriate for all regions of Oregon except along the coast where CSZ earthquakes dominate the hazard with respect to PGA*.

STEP 3

Select Appropriate Acceleration Time Histories
- Consider type of source, magnitude, duration, frequency content, and energy
- Select at least 3 records per M-R pair

STEP 4

Perform Dynamic Soil Response Analysis
- Develop soil profile for SHAKE
- Scale time histories to appropriate PGA values
- Compute suite of Acceleration Response Spectra (ARS) for ground surface motions using all the selected time histories
- Define recommended design ARS using established criteria for determining the SA at each period given the collection of computed ARS (*e.g., mean + 1σ SA determined for the entire suite of computed ARS*)

STEP 5

Compare Computed ARS to Applicable Code-Base Spectra (AASHTO, NEHRP, IBC)
- Modify ARS for design as needed

STEP 6

Establish Final Design ARS

6.0 Liquefaction Analysis

All new replacement bridges in areas with seismic acceleration coefficients greater than or equal to 0.10g should be evaluated for liquefaction potential. Bridges scheduled for seismic retrofit should also be evaluated for liquefaction potential if they are in a seismic zone with an acceleration coefficient ≥ 0.10g.

Liquefaction analysis is typically performed using the “Simplified” method developed by Seed and Idriss (ref. 5) and described in the AASHTO Division I-A, Section 6 Commentary (ref. 1) and the FHWA Earthquake Engineering manual (ref. 4). This method is based on empirical correlations to SPT $(N_1)_{60}$ values. The EXCEL spreadsheet “Liquefy_301” is available (internal to ODOT) to perform these calculations. Liquefaction analysis based on CPT data is also acceptable.

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. 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.

If liquefaction is predicted, and results in mitigation being required, a more thorough site specific analysis, utilizing the SHAKE91, or other, computer program, is recommended to substantiate the predicted, induced ground motions. This procedure is especially recommended for sites where liquefaction potential is marginal ($0.9 < FS_L < 1.10$).

If a site specific response analysis is not performed, the peak ground surface acceleration can be very approximately estimated from graphs in the FHWA Earthquake Engineering manual (ref. 4) or the OSU report (ref. 8), based on the peak bedrock acceleration.

All field SPT “N” values should be multiplied by the corresponding SPT hammer energy correction factors (Cer) to obtain N_{60} and then further corrected for overburden pressure. Hammer energy correction factors for some ODOT drill rigs may be obtained from the GRL report (ref. 6) for the particular drill rig and SPT hammer used. Otherwise the hammer efficiency should be obtained from the hammer manufacturer, preferably through field testing of the hammer system used to conduct the test.

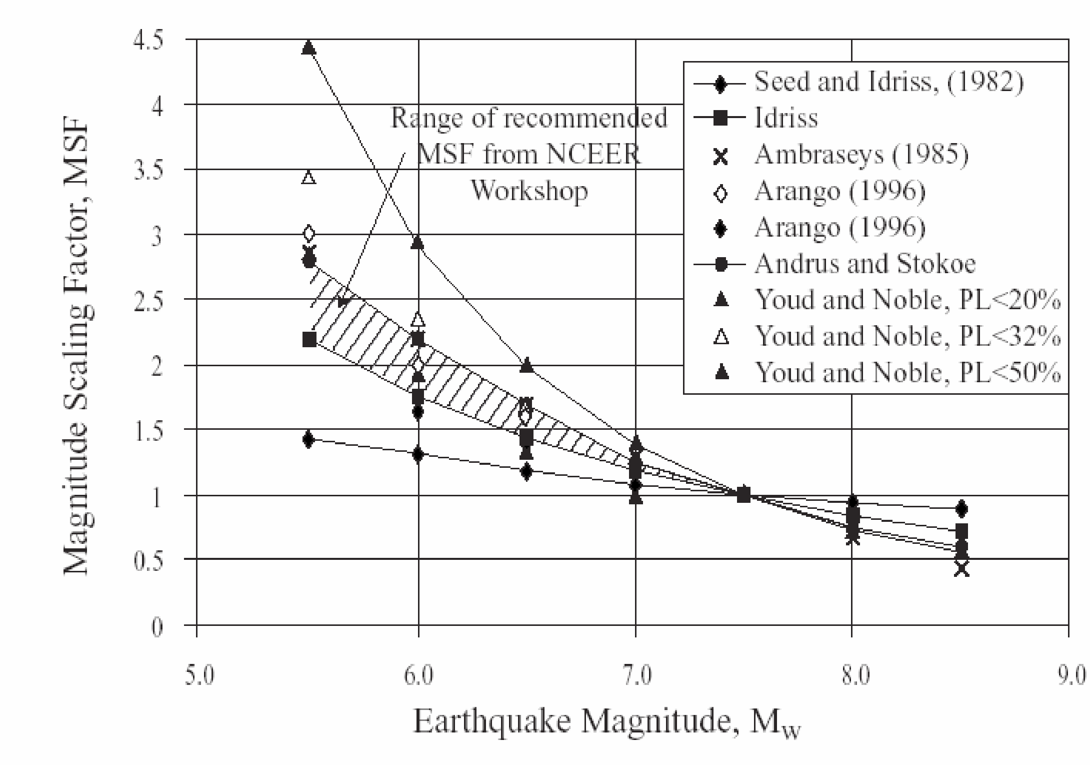
The groundwater level to use in the liquefaction analysis should be determined as follows:

- Static Groundwater Condition - Use the estimated, average annual groundwater level. Perched water tables should only be used if water is present in these zones more than 50% of the time.
- 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: Groundwater levels measured in drill holes advanced using drill water may not be indicative of true static groundwater levels. Water in these holes should be allowed to stabilize over a period of time to insure measured levels reflect true static groundwater levels.

Magnitude scaling factors (MSF) are required to adjust the critical stress ratio (CSR) obtained from the standard Seed & Idriss method (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 (ref. 7) 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 below and the equation of the curve is: $MSF = 10^{2.24} / M^{2.56}$.



Magnitude Scaling Factors Derived by Various Investigators
(1996 NCEER Workshop Summary Report, ref. 7)

Determine whether the liquefied soil layer is stratigraphically continuous and oriented in a manner that will result in lateral spread or other adverse impact to the bridge. If liquefaction is predicted at the site, use the recommendations provided in the OSU references (ref. 8 and 10) to assess embankment deformations, the potential for damage to the proposed bridge approaches, abutments and/or piers and mitigation strategies.

7.0 Liquefaction Effects on Foundation Design

If liquefaction is predicted under either the 500 or 1000 year return events, the effects of liquefaction on foundation design and performance must be evaluated. For design purposes, liquefaction is assumed to occur concurrent with the peak loads in the structure (i.e. no reduction in the transfer of seismic energy due to liquefaction and soil softening). Liquefaction effects include:

- reduced axial and lateral capacities and stiffness in deep foundations,
- lateral spread and global instabilities of embankments,
- ground settlement and possible downdrag effects

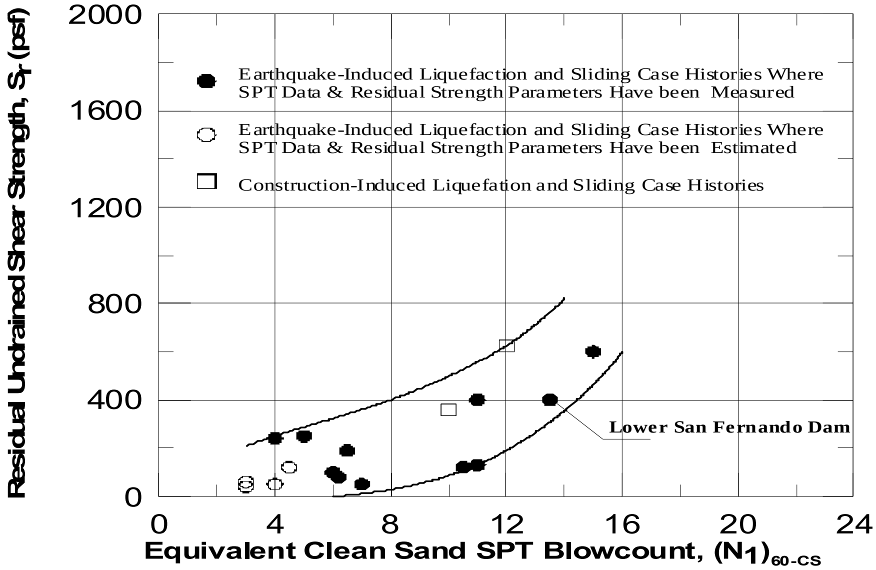
The following office practice is recommended:

Spread Footings - Spread footings are not recommended for bridge or abutment wall foundation support over liquefiable soils unless ground improvement techniques are employed that eliminate the liquefaction condition.

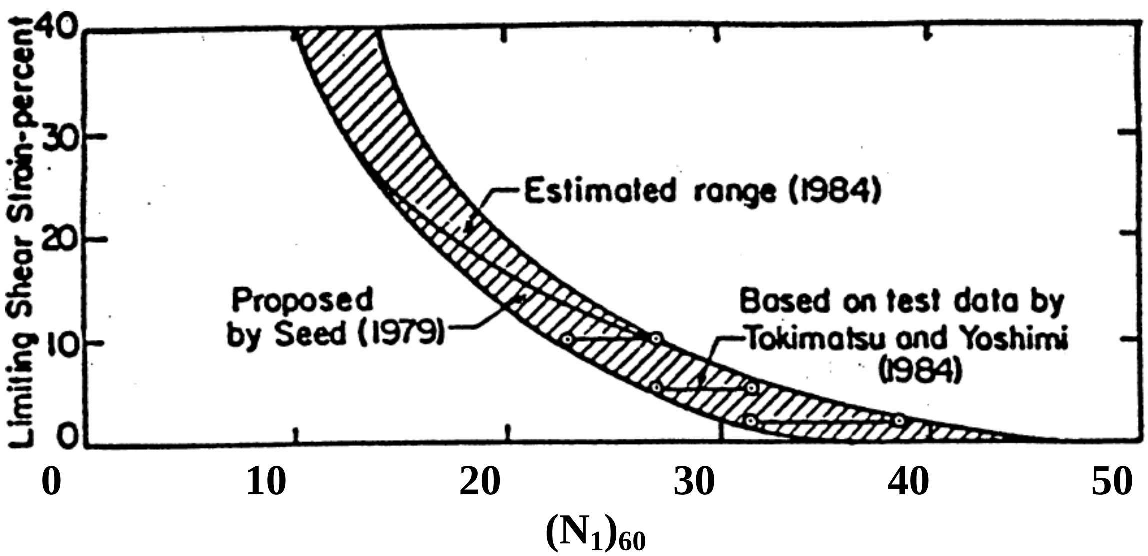
Piles and Drilled Shafts - Friction resistance from liquefied soils should not be included in either compression or uplift capacity recommendations for the seismic loading condition. As stated in Section 5.0, liquefaction of foundation soils, and the accompanying lose of soil strength, is assumed to 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 in the Seed and Idriss paper (ref. 5) and the ODOT Research Report (ref. 8).

Liquefied p-y Curves: Studies have shown that liquefied soils retain a reduced (residual) shear strength and this shear strength may be used in evaluating the lateral capacity of foundation soils. The following is recommended:

1) Soft Clay Criteria: Use the “soft clay” soil type for p-y curve generation in the LPILE or COM624P analysis along with e_{50} = 1/3 * (limiting strain), and residual shear strength from the Seed and Harder paper (ref. 9) shown in the graph below. Use static loading since cyclic loading is already accounted for using these parameters.



Corrected SPT Blow Count vs. Residual Strength
(Seed and Harder, 1990)



Relationship Between Corrected SPT Blow Count vs. Limiting Strains For Natural Deposits of Clean Sand (Seed, Tokimatsu, Harder and Chung, Harder, 1985)

2) Additional liquefied p-y curve recommendations are provided in the research report titled: "TILT: The Treasure Island Liquefaction Test: Final

Report" (ref.11) available from the Bridge Section. This full scale study produced liquefied soil p-y curves for sand that are fundamentally different than those derived from the "soft clay" or any other standard p-y criteria. The results of this study should be used with caution until further studies are completed and a consensus is reached on the standard of practice for p-y curves to use in modeling liquefied soils. These liquefied p-y curves are also available in Version 5 of the LPILE computer program.

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

1) For the PL/AE method, if the liquefied zone reduces total pile skin friction to less than 50% of ultimate bearing capacity, use "end bearing" condition (i.e. full length of pile) in stiffness calculations. Otherwise use "friction" pile condition.

2) For the APILE program, assume clay layer for liquefied zone with modified soil input parameters similar to methods for p-y curve development (i.e. residual shear strength and e_{50} values).

<u>Pile Design Alternatives</u>: Obtaining adequate lateral pile capacity is generally the main concern at pier locations where liquefaction is predicted. Battered piles are not recommended. 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 recommended to provide the most flexible, ductile foundation system. Steel pipe piles are preferred over H-piles due to their uniform section properties, versatility in driving either closed or open end and their potential for filling with reinforced concrete. For a given pile group loading, the following design alternatives should be considered for increasing group capacity 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.

<u>Group Effects</u>: Use the same group effects (p-y multipliers) as for the nonliquefied condition.

<u>Foundation Settlement</u>: Ground settlement due to the densification of liquefied soils or dynamic compaction generally does not result in significant damage to structures

because the downdrag shear stresses on the piles or shafts resulting from the settling liquefied soils is usually small relative to the ultimate capacity of the piles if the piles are end bearing or extend well below the liquefied zone. However, in cases where liquefiable layers are below nonliquefiable layers, the upper, nonliquefied soils may also settle and thereby transfer significant downdrag loads to the piles or shafts. In these cases, foundation settlements should be estimated and a downdrag analysis performed. Seismic downdrag loads should be reported in the Foundation Report since these loads also have to be included in the structure analysis. In addition, an Ultimate Capacity for seismic loading should be provided. The Ultimate Capacity for seismic loading is the ultimate pile capacity the pile is driven to minus the downdrag loads. A factor of safety of at least 1.0 should be maintained in the piles or shafts under this Extreme Event loading condition. If this is not possible due to extremely high seismic loads and friction pile conditions the bridge must be evaluated and designed to accommodate the estimated foundation settlements.

Embankment Stability and Displacement Estimates: Embankment stability should be evaluated using the slope stability programs XSTABL, Slope/W or other standard recognized slope stability program. The methods described in the OSU research paper: “Assessment and Mitigation of Liquefaction Hazards to Bridge Approach Embankments in Oregon”, (ref. 8), should be used to estimate embankment displacements under liquefied conditions. The Bracketed Intensity and Newmark methods are recommended to estimate lateral displacements. The paper titled: ”Recommended Guidelines For Liquefaction Evaluations Using Ground Motions From Probabilistic Seismic Hazard Analysis” (ref. 10) also contains examples of how to estimate embankment displacements under liquefied soil conditions.

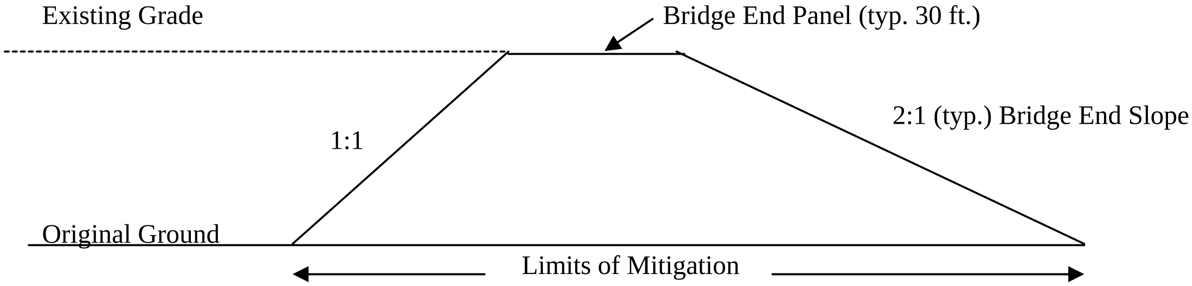
If lateral displacements result in large embankment displacements and the need for extensive mitigation or ground improvement, then a more detailed analysis, including finite element methods, may be warranted to verify the need for mitigation or to better define the extent of the mitigation area.

8.0 Liquefaction Mitigation

The need for liquefaction mitigation 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 guidelines previously described in Section 2 should be followed. Ground Improvement methods described in the FHWA Publication: “Ground Improvement Technical Summaries (ref. 10) should be referenced for guidance on acceptable ground improvement methods.

If, under the 500-year event, the estimated bridge damage is sufficient to render the bridge out of service for emergency traffic then mitigation measures should be undertaken. If, under the 1000-year event, estimated bridge damage results in the possible collapse of a portion or all of the structure then mitigation is recommended. A flow chart of the ODOT Liquefaction Mitigation Policy is attached in the Appendix.

Liquefaction mitigation is defined herein as ground improvement techniques that result in reducing estimated ground and embankment displacements to acceptable levels. Liquefaction mitigation of soils beneath approach fills should extend a distance away 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 end slope to a point where a 1:1 slope extending from the back of the bridge end panel intersects the original ground (see figure below). The final limits of the mitigation area required should be determined from a slope stability analysis and the methods described in the ODOT Research Report.



Liquefaction mitigation should also be considered as part of any Phase II 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.

9.0 Seismic Foundation Design Recommendations

Seismic foundation design will generally require an assessment of the seismic hazard, determination of design ground motion values, site characterization, and seismic analysis of the foundation materials and their effects on the proposed foundation system. Note that separate analysis and recommendations will be required for the 500 and 1000 year seismic design ground motions. If liquefaction potential exists at the site, pile and shaft capacities and stiffness should be reported for the non-liquefied and liquefied soil condition.

A general design procedure is described in the following flow chart along with the information that should be supplied in the Final Foundation Report.

SEISMIC FOUNDATION DESIGN PROCEDURE
(ODOT, October, 2005)

STEP 1; Ground Motion Data

- Identify the seismic sources in the region affecting the site for the given return period (500 and 1000 yrs):
- Determine Peak Ground Accelerations from the 2002 USGS Seismic Hazard Maps
- Determine Soil Profile Type and Site Coefficient (AASHTO Section 3.5)

STEP 2; Site Response Analysis

- Decide whether a site response analysis is warranted and if so provide:
  - 5% damped smoothed response spectra
  - Predominant period of ground motion
  - Ground surface PGA

STEP 3; Evaluate Liquefaction Potential & Effects ($PGA \geq 0.10g$)

- Perform Deaggregation of seismic hazard, determine M & R pairs
- Estimate PGA using several attenuation relationships
- Calculate liquefaction potential

Liquefaction Potential

STEP 3a; For foundation soils susceptible to liquefaction:

- estimate post-liquefaction soil strengths
- Evaluate embankment stability and est. deformations
- Develop mitigation designs if required
- Assess the effects of liquefaction on foundation capacities and provide reduced foundation capacities under liquefied soil conditions. (CHECK DOWNDRAG)

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 1.1.4 of BDDM):

Spread Footings

- Shear modulus); 'G' is dependent on the shear strain; generally a 'G' corresponding to a shear strain in the range of 0.20% to 0.02% is appropriate. For large magnitude events (M>7.5) and very high PGA (>0.6g), a 'G' corresponding to a shear strain of 1% is recommended. 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 data for non-liquefied and liquefied soils (see Table 6.1 below)
- p-y multipliers (see FHWA Pile Design Manual)
- Designation as "end bearing" or "friction" piles for modeling axial stiffness

Shafts

- p-y curve data for non-liquefied and liquefied soils (see Table 6.1 below)
- p-y multipliers (see FHWA Drilled Shaft Design Manual)

TABLE 6.0 (example)
AXIAL PILE CAPACITIES (1000 yr Return Period)

Bridge 12345		**Tip Elevations (feet)**		**Compression (kips)**			**Tension (uplift) (kips)**		
Bent	Pile Type	Est.	Req.	Allow.	Ultimate		Allow.	Ultimate	
					w/o Liq.	With Liq.	(FS=3.0)	w/o Liq.	With Liq.
2	PP12.75x0.375	85	100	180	450	380	80	240	170

Notes on Table 6.0:

- Capacities based on AASHTO Service Load Design.
- Ultimate capacity determined by Wave Equation construction control with FOS = 2.5 applied to determine Allowable Compression Capacity.
- Qult in compression w/ liquefaction should also be reduced by any seismic downdrag loads. Include estimated downdrag loads in the table if present.
- Allowable static uplift FOS = 3.0 is from AASHTO Section 4.5.6.6.1

TABLE 6.1 (example)
Soil Input for LPILE or COM624 Analysis
(Extreme Event I Limit State, 1000 yr return)

ELEVATION		**KSOIL**	**K (pci)**	**SOIL PROPERTIES**				**COMMENTS**
From	**To**			**γ,(pci)**	**c,(psi)**	**e_{50}**	**φ**	
200.0	185.0	1	N/A	0.03	2.5	.13	--	Liquefied fine Sand
185.0	165.0	4	60	0.05	-	-	36	Non-Liquefied Silty Sand (below water table)
165.0	145.0	4	125	0.07	-	-	38	V. Dense Sand & Gravel
145.0	100.0	4	130	0.08	-	-	42	V Dense Gravel

(COM624P or LPILE Computer Programs)

Note: KSOIL 1: Soft Clay criteria (COM624)
KSOIL 4: Sand criteria (COM624)

Report the results of the liquefaction analysis (factors of safety against liquefaction), embankment deformation estimates and any estimated damage potential for all bridges. This requires discussions with the structure designer to properly assess damage potential. If mitigation measures are required, provide the recommended design, estimated cost, plans and special provisions as necessary. For ground improvement designs, performance based specifications should be used and include a thorough field testing program (QA) of the improved soil volume for conformance to the specifications.

Some of the soil properties recommended above may be a function of the construction methods and backfill materials used by the contractor. If so, these materials and construction methods need to be specified in the contract documents to insure compliance with design assumptions.

10 Seismic Foundation Design of Retaining Walls

Seismic design is only required for walls supporting bridge abutments or walls supporting other critical structures or facilities. Critical wall applications are defined on a case-by-case basis through consultation with the ODOT Retaining Wall coordinator. Most retaining walls, especially MSE walls, have performed very well under earthquake loading with minimal damage. The foundations for bridge abutment walls should be evaluated under seismic loading conditions to estimate lateral wall displacements, settlement, liquefaction potential and calculate global stability factors of safety. The performance of abutment walls must meet the performance criteria described in Section 2.1.

AASHTO commentary C6.3.2 (A), C6.4.2 (A) and C6.5.2 (A) should be referred to for design. For walls requiring seismic design, the phi angle, (ϕ') and unit weight of the retained material (γ) are required to determine the seismic active and passive pressure coefficients. The vertical acceleration coefficient (k_v) is typically assumed to be zero (0).

References

(1) “**Standard Specifications For Highway Bridges”**, AASHTO, 16th Edition plus latest revisions.

(2) “**Bridge Design and Drafting Manual”,** Oregon Department of Transportation, October, 2004

(3) **“AASHTO Manual on Subsurface Investigations”,** 1988.

(4) **“Design Guidance: Geotechnical Earthquake Engineering for Highways”,** Geotechnical Engineering Circular No. 3; Volumes I & II, FHWA-SA-97-076/077

(5) “**Simplified Procedure for Evaluating Soil Liquefaction Potential**”, H. Bolton Seed & I. M. Idriss, Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 97, SM9, September, 1971.

(6) “**Energy Measurements On Standard Penetration Tests**”, Goble, Rausche, Likins & Associates, Inc., Report to ODOT Bridge Engineering Section, Foundation Design Unit, March, 1995.

(7) “**Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils”**, edited by T. L. Youd & I. M. Idriss; held at Temple Square, Salt Lake City UT, Jan. 5-6, 1996.

(8) “**Assessment and Mitigation of Liquefaction Hazards to Bridge Approach Embankments in Oregon**”, Oregon State University, Department of Civil, Construction and Environmental Engineering, SPR Project 361, November, 2002.

(9) “**SPT-Based Analysis of Cyclic Pore Pressure Generation and Undrained Residual Strength**”, Raymond Seed & Leslie Harder, Proceedings, H.B. Seed Memorial Symposium, BiTech Publishing, Vancouver, B.C., Canada, 2 (1990).

(10) “**Recommended Guidelines For Liquefaction Evaluations Using Ground Motions From Probabilistic Seismic Hazard Analysis”,** Oregon State University, Department of Civil, Construction and Environmental Engineering, June, 2005.

(11) “**Ground Improvement Technical Summaries, Volumes I & II”,** Demonstration Project 116, FHWA Publication No. FHWA-SA-98-086, March 2000.

(12) “**TILT: The Treasure Island Liquefaction Test: Final Report”,** Scott A. Ashford & Kyle M. Rollins, Dept. of Structural Engineering, Univ. of California, San Diego, Report No. SSRP-2001/17, January 2002.

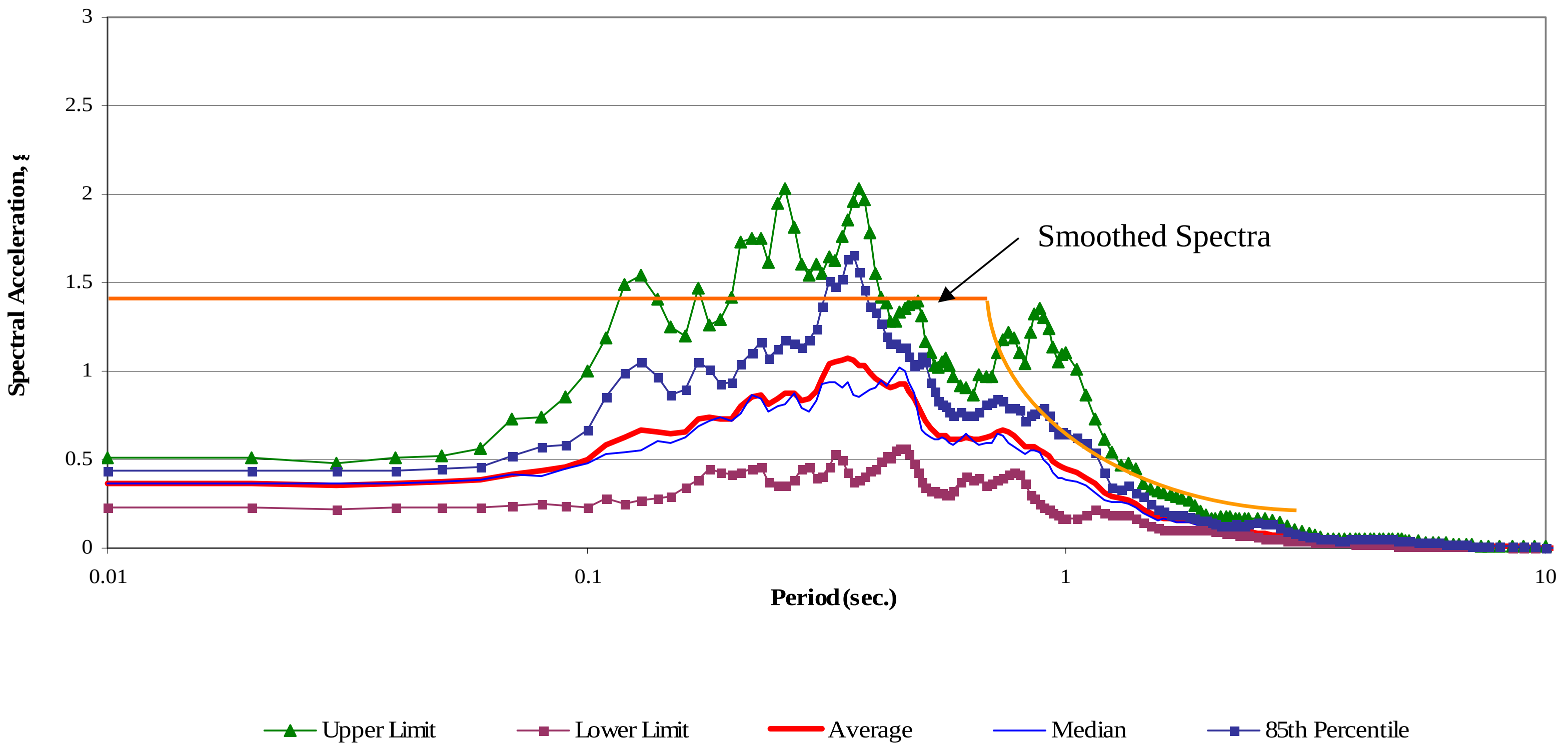
APPENDIX

Example of Smoothed Response Spectra

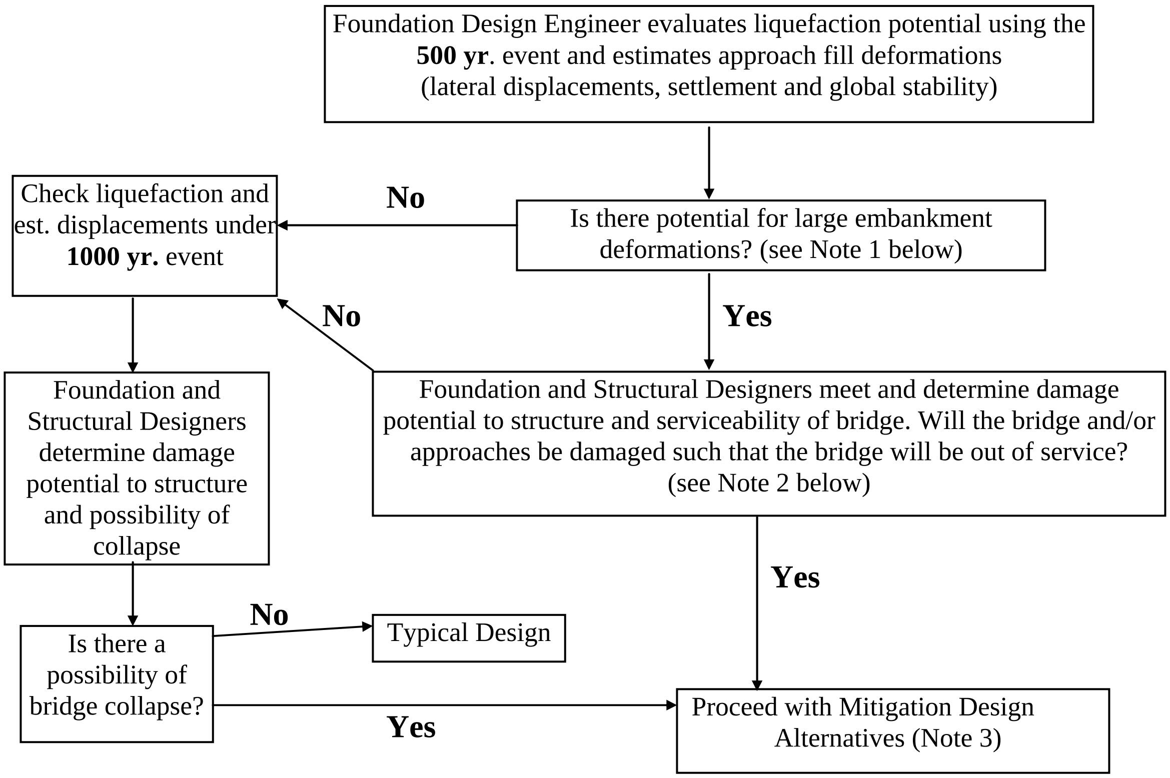
ODOT Liquefaction Mitigation Procedures

Flow Chart For Evaluation Of Liquefaction Hazard And Ground Deformation At Bridge Sites

Example Smoothed Response Spectra

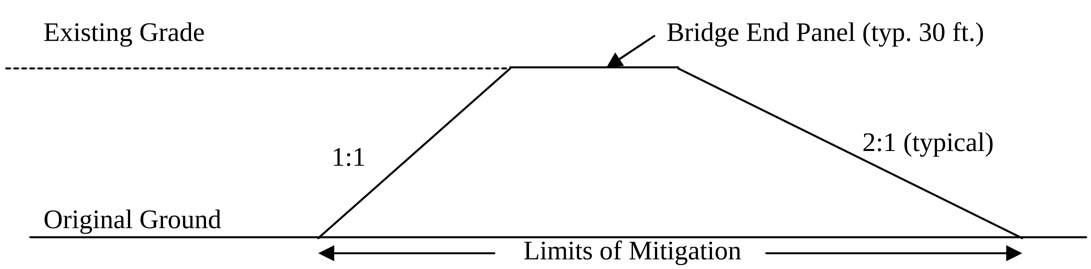


ODOT Liquefaction Mitigation Procedures



Note 1: Lateral deformations up to 12” are generally considered acceptable under most circumstances.
Note 2: The bridge should be open to emergency vehicles immediately after the 500-year design event (after 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 ODOT research report SPR Project 361: “Assessment and Mitigation of Liquefaction Hazards to Bridge Approach Embankments in Oregon”, Nov. 2002 and FHWA Demonstration Project 116; “Ground Improvement Technical Summaries, Volumes I & II”, (Pub. No. FHWA-SA-98-086) for mitigation alternatives and design procedures.

As a general guideline, along centerline, the foundation mitigation should extend from the toe of the end slope to a point that is located at the base of a 1:1 slope which starts at the end of the bridge end panel. In cross section, the mitigation should extend as needed to limit deformations to acceptable levels.



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 the 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 $FS_{liq} \geq 1.4$

Use drained shear strengths

If $1.4 > FS_{liq} > 1.0$

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

If $FS_{liq} \leq 1.0$

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

- Rigid-body, sliding block analysis (Newmark Method)
- Simplified chart solutions
- 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 repeat analysis incorporating the effects of remedial ground treatment
- Return to Step 4 if the soil improvement does not significantly change the anticipated dynamic response of the soil column (e.g., isolated soil improvement)
- Return to Step 3 if the ground treatment 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)
- A reduced number of input time histories are acceptable for each M-R pair (bracket the problem using trends from the initial analysis)