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Chapter 19
SEISMIC DESIGN

19.1 GENERAL

19.1.1 Overview

MDT projects will typically need to consider the potential for seismic loading during design. Western Montana is more susceptible to strong ground shaking than eastern Montana. The primary source of ground shaking has been along the Intermountain Seismic Belt that extends from the northwest corner of Montana to the Yellowstone Park region. As shown in Figure 19.1-A, small earthquakes have also occurred in eastern Montana, although the rate and sizes of seismic events in this part of the State are generally much smaller.

The largest historic earthquake in Montana was the August 18, 1959, Hebgen Lake earthquake with a magnitude of 7.3. The United States Geological Survey (USGS) earthquake catalogue for Montana identifies seven earthquakes with a magnitude of 5.5 or more have occurred since 1925. The most recent of these occurred on July 25, 2005, approximately 14 miles (23 km) north of Dillon, Montana with a magnitude of 5.6 and caused minor damage to structures in the general area. Small earthquakes occur in Montana at the rate of 7 to 10 events per day, according to the Montana Bureau of Mines and Geology website. In view of this historic seismicity in Montana, the large number of small earthquakes that occur on a daily basis and the potential for larger earthquakes, seismic loading must be considered.

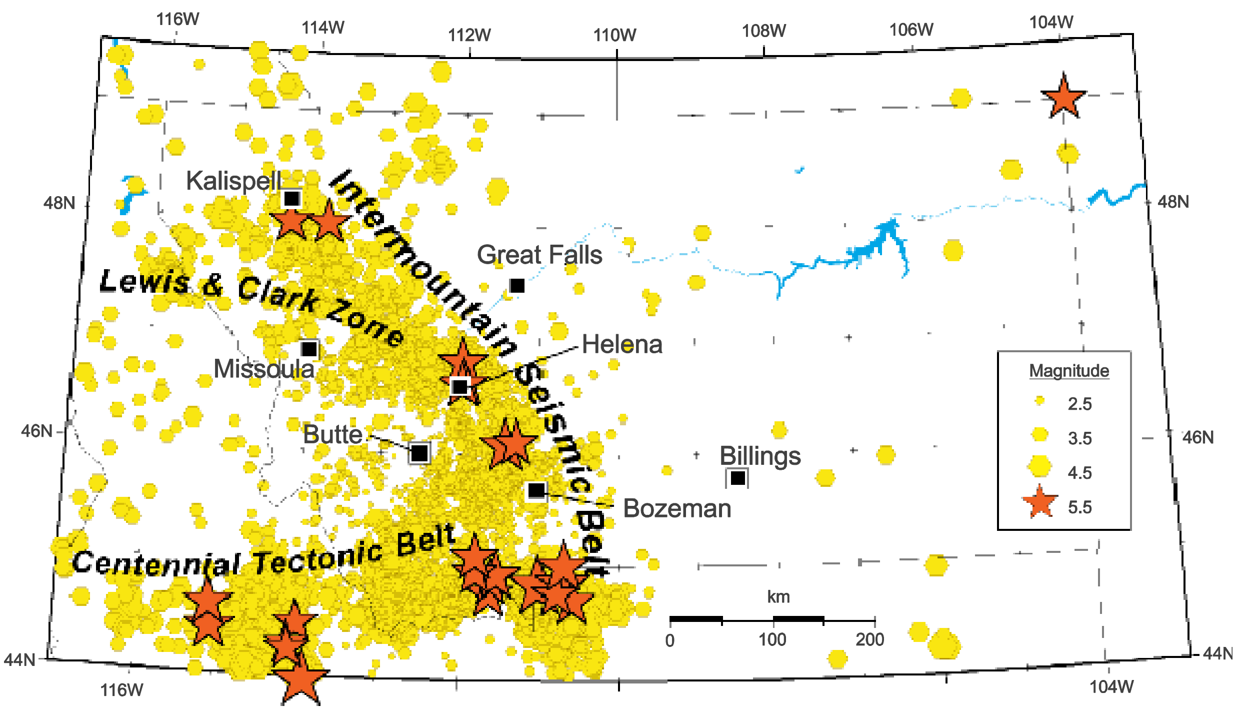

Figure 19.1-A — SEISMICITY OF MONTANA

The potential for seismic loading is important to roadways, bridges and other related transportation facilities because of its consequences, which include transient ground movement from seismic wave propagation and permanent ground movement. Ground shaking can result in inertial forces in bridge structures and slopes; increased earth pressures on retaining walls; soil liquefaction in saturated, loose cohesionless soils; and cyclic softening in clay soils. Permanent ground movement can be the result of fault displacements, slope instabilities, settlement from liquefaction or densification of ground.

Transient and permanent ground movements can cause damage to structures, retaining walls, embankments and natural slopes. Damage from earthquakes is often referred to as being either primary or secondary:

1. Primary. Primary damage is a direct result of strong shaking or fault rupture and can include partial or total collapse of a structure. The magnitude of the damage due to strong shaking will depend on the intensity of the motion, the frequency of the motion and the design of the structure.

2. Secondary. Secondary damage is a consequence of primary damage. For example, strong shaking may cause a landslide that damages a bridge or roadway. In saturated sands strong shaking may cause loss of soil strength in level ground that leads to settlement or lateral spreading of soil. This loss of strength is referred to as liquefaction. The consequences of liquefaction may include bearing capacity failure, excessive settlement, lateral spreading and slope instability. Structures located in the path of the moving soil will undergo increased loads and slopes of earth embankments can fail, resulting in loss of the roadway or damage to a bridge abutment.

In general, MDT adheres to the provisions in the latest edition or interim revision of AASHTO *LRFD Bridge Design Specifications* for seismic design. The performance objectives for seismic design in the *LRFD Specifications* require protection of the public from loss of life and serious injury due to structure collapse, as practical and economically feasible. With this philosophy, the structure may suffer damage and may need to be replaced after a design seismic event; however, it should be designed for non-collapse to meet the life safety requirement.

For critical structures, MDT may design to a more stringent seismic level than that defined by AASHTO. This represents a strategic decision to maintain the operation of a critical bridge, often referred to as a lifeline transportation route, during the design seismic event. This operational state could require determination of seismic ground motions that have a longer return period (i.e., lower chance of occurrence) than the design basis in the *LRFD Specifications*.

19.1.2 Responsibilities

The following identifies the basic responsibilities of the MDT Units for seismic design.

19.1.2.1 Geotechnical Section

The Geotechnical Section has a significant role in the assessment of seismic hazards, from defining ground motions to evaluating the potential for and consequences of earthquake-related geologic hazards. The Geotechnical Section is responsible for the following activities related to seismic design:

- defining ground motions at the rock/stiff soil interface either by using AASHTO hazard maps or by conducting site-specific probabilistic seismic hazard analyses (PSHA);
- determining site amplification/deamplification factors by using the AASHTO Site Class descriptions or by conducting site-specific, dynamic ground response studies;
- evaluating the potential for active faults at the site and, if found, the likelihood and amount of possible movements;
- assessing the geologic hazards resulting from seismic ground shaking (e.g., liquefaction, slope instabilities, lateral spreading, settlement, downdrag, dynamic earth pressures); and
- working with the Bridge Bureau to develop soil foundation spring constants for use in soil-structure interaction analyses.

19.1.2.2 Bridge Bureau

The Bridge Bureau evaluates the response of bridge structures, retaining walls and other structural facilities to seismic load based on the AASHTO *LRFD Bridge Design Specifications* and based on relevant site information provided by the Geotechnical Section. The relevant information includes Site Classes for use in determining design response spectra and the potential for liquefaction.

During design, the Bridge Bureau consults with the Geotechnical Section on the expected performance of the foundation during seismic loading or on how to mitigate poor performance. Mitigation measures typically involve some type of ground improvement to lower the seismic demand or to increase foundation capacity during seismic loading.

Some seismic hazards (e.g., liquefaction, lateral spreading) can have a significant effect on bridge-type selection and, therefore, the Geotechnical Section should notify the Bridge Bureau early in the design process if such conditions are anticipated.

19.1.3 References

For further guidance on seismic design, the project geotechnical specialist should review the following:

- *Geotechnical Engineering Circular No. 3,* "Design Guidance: Geotechnical Earthquake Engineering for Highways," FWHA-SA-97-076;
- *Geotechnical Earthquake Engineering, NHI Course No. 13239 — Module 9*, FHWA HI-99-012, December 1998;
- *Geotechnical Earthquake Engineering*, S. L. Kramer, Prentice-Hall;

- *Seismic Design Manual for Segmental Retaining Walls*, National Concrete Masonry Association;
- *AASHTO LRFD Bridge Design Specifications*;
- USGS website;
- DNRC Dam Safety Group website;
- Montana Bureau of Mines and Geology website;
- ATC/MCEER (2003), "NCHRP 12-49 Recommended LRFD Guidelines for the Seismic Design of Highway Bridges, Parts I and II: Specifications and Commentary and Appendices," 2003;
- Boulanger, R.W. and I.M. Idriss, "Liquefaction Susceptibility Criteria for Silts and Clays," *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 132, No. 11, pp. 1413-1426, November 2006;
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- Cetin, K.O., Seed, R.B., Der Kiureghian, A., Tokimatsu, K. Harder, L.F., Kayen, R.E., and R.E.S. Moss, "Standard Penetration Test-Based Probabilistic and Deterministic Assessment of Seismic Soil Liquefaction Potential," *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 130, No. 12, pp. 1314-1340, December 2004;
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- MCEER, *Seismic Retrofitting Manual for Highway Structures: Part 1 – Bridges*, MCEER-06-SP10, University of Buffalo, NY, 2006;
- Moss, R.E.S., Seed, R.B., Kayen, R.E., Stewart, J.P., Der Kiureghian, A. and K.O. Cetin, "CPT-Based Probabilistic and Deterministic Assessment of In-Situ Seismic Soil

Liquefaction Potential," *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 132, No. 8, pp. 1032-1051, August 2006;

- Olson, S.M. and T.D. Stark, "Liquefied Strength Ratio from Liquefaction Flow Failure Case Histories," *Canadian Geotechnical Journal*, Vol. 39, pp. 629-647, 2002;
- SCEC, "Recommended Procedures for Implementation of DMG Special Publication 117," *Guidelines for Analyzing and Mitigating Liquefaction Hazards in California*, Southern California Earthquake Center (SCEC), 1999;
- Seed, R.B. and L.F. Harder, "SPT-Based Analysis of Cyclic Pore Pressure Generation and Undrained Residual Strength," Proceedings, H.B. Seed Memorial Symposium, Bi-Tech Publishing, Vol. 2, pp 351-376, 1990;
- Shamsabdi, A., Rollins, K.M., and M. Kapuska, "Nonlinear Soil-Abutment-Bridge Structure Interaction for Seismic Performance-Based Design," *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 133, No. 6, pp. 707-720, June 2007; and
- Youd et al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 127, No. 10, pp. 817-833, October 2001.

19.2 GROUND MOTIONS

19.2.1 General

There are two types of ground movement to consider during seismic design: (1) seismic waves that propagate through the earth, and (2) fault-displacement involving rupture of the ground. Seismic waves cause vibratory loading to the soil and to structures supported on or in the soil. This loading lasts for a minute or more. Seismic waves can originate hundreds of miles from the site. Fault displacement is a permanent shift in the ground that is located along the fault trace. Most of the movement associated with faulting will be within a few tens of feet (meters) of the fault trace. Generally, the design emphasis involves ensuring that the structure or soil can withstand the inertial loading from the seismic wave. However, for structures or roadways crossing a fault, the effects of permanent displacement of the earth must also be evaluated.

A number of terms are used when characterizing seismic ground motions:

1. Peak Ground Acceleration (PGA). This is the peak ground motion that occurs during the seismic event. Geotechnical engineers use the PGA during the assessment of liquefaction potential, seismic slope stability, seismic-induced settlements and seismic earth pressures. The PGA is a transient condition. Average ground motions will range from two-thirds of the peak to less than one-half of the peak, depending on the magnitude and location of the earthquake source. The PGA will vary with the earthquake location, magnitude and type of rupture.

2. Earthquake Magnitude. The earthquake magnitude quantifies the size or energy of the earthquake. The magnitude will depend on the amount of area or length that ruptures during the event; small magnitudes are related to small rupture areas or lengths. The magnitude, and how it is measured (e.g., Richter, moment), is important for liquefaction and some slope stability analyses, because magnitude is correlated to the number of loading cycles. Large magnitude earthquakes have many cycles of loading, which result in greater potential for liquefaction and greater accumulation of slope displacement.

3. Response Spectrum. The response spectrum is a plot showing the peak response of a single degree of freedom (SDOF) system to ground motion. The peak response can be in terms of acceleration, velocity or displacement. When in terms of acceleration, the acceleration is referred to as the spectral acceleration. Peak response is defined at different periods to develop the response spectrum. The Bridge Bureau uses the response spectra to determine seismic forces that will develop during the earthquake. This determination uses the spectral acceleration for the predominant period of the structure, which is directly related to the mass and stiffness of the structure.

In 2007, AASHTO adopted a 3-point method of defining the response spectrum; see Figure 19.2-A. Further discussion on the ground motion determination is provided in the following Sections.

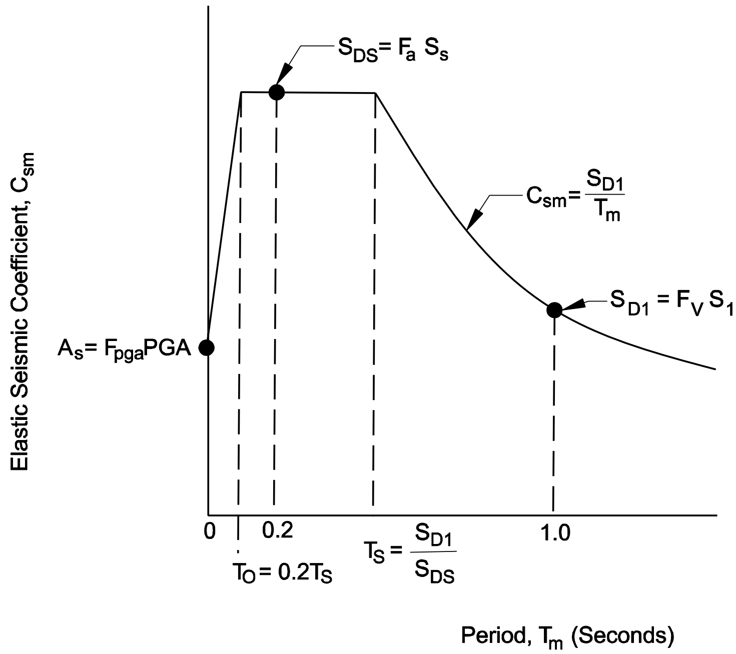


Figure 19.2-A — DESIGN RESPONSE SPECTRUM

19.2.2 <u>Determination of Seismic Ground Motions</u>

Predicted seismic ground motions can be estimated at a site using three primary methods — AASHTO ground motion maps, site-specific probabilistic methods or site-specific deterministic methods. Other maps (e.g., USGS) or detailed site-specific maps might also be available for certain areas. A brief description of each method is provided in the following Sections. Each method typically involves determining the ground motion on rock and then adjusting this motion for propagation through the soil, if soil occurs above the rock.

19.2.2.1 AASHTO Ground Motion Maps for Rock

For most MDT projects, seismic ground motion parameters are obtained from the AASHTO nationwide seismic ground motion hazard maps. The USGS developed these maps for AASHTO in 2006 based on probabilistic methods. Previously, the AASHTO seismic hazard maps had been based on an earthquake with a 10% probability of exceedance in a 50-year period. This earthquake has an average return period of 475 years; i.e., an earthquake producing the ground motion parameter (e.g., PGA, spectral acceleration) obtained from the map occurs on average once every 475 years.

In 2007, AASHTO revised the basis of design to 7% in 75 years. This earthquake has an average return period of 975 years, and is nominally referred to as the 1000-year earthquake. This change in the basis of seismic design adopted by AASHTO recognizes that most bridges are designed to be in service for at least 75 years. The longer average return period provides more conservatism in seismic design, especially for areas having low seismicity. In Montana, the level of design ground motion in the western part of the State remained the same or decreased somewhat, even though the return period increased from 475 to 975 years. The similarity to previous ground motion estimates reflects improvements in the seismic hazard model used for the new 975-year return period versus the old 475-year return period.

AASHTO has distributed a CD that determines the peak ground acceleration and spectral accelerations for a specified latitude and longitude. This is similar to the USGS National Hazard website for the continental United States, Hawaii, Alaska and other areas where earthquake ground motions can be determined for probabilities of 2% to 10% in 50 years. The USGS website also allows the determination of peak ground accelerations and spectral accelerations for a nominal 1000-year return period (i.e., 5% in 50 years) through the use of deaggregation methods. For MDT projects, the AASHTO CD should be used for determination of the PGA and spectral accelerations. The AASHTO hazard CD does not provide any specific information about the magnitude of earthquakes contributing to the hazard; therefore, the deaggregation feature in the USGS website will need to be used to obtain information about the mean magnitude associated with the 975-year earthquake.

The values of ground motion on the AASHTO CD are for rock conditions. For soil sites, the rock motion is modified as it propagates through the soil. The amount of modification depends on both the level of ground shaking and the site characteristics. Section 19.2.3 discusses methods for modifying the rock motions for soil conditions.

Although the AASHTO basis of seismic design is the 1000-year earthquake, MDT may design critical structures for longer return periods. A longer return period is equivalent to a less frequent probability of exceedance, which may be appropriate for a critical lifeline bridge whose failure could have significant economic consequences. If a longer return period is desired, one option is to use the USGS hazard website to determine the ground motions associated with a 2% in 50-year earthquake, which has a return period of approximately 2500 years. Another alternative is to conduct a site-specific probabilistic hazard analysis, as described in the following Section.

19.2.2.2 Site-Specific Probabilistic Seismic Hazard Analysis for Rock

A site-specific probabilistic seismic hazard analysis (PSHA) involves the development of a seismic hazard model for the area of interest, essentially repeating the USGS nationwide activities, but setting up the model on a more localized basis. PSHA requires a detailed understanding of seismic source characterization, ground motion attenuation relationships and methods of treating uncertainty in the probabilistic evaluation. As a benefit, this model allows the introduction or evaluation of new information (e.g., new potential seismic sources, new ground motion attenuation relationships). Typically, site-specific hazard studies are conducted by specialty consultants.

The Geotechnical Section must review and approve the use of site-specific procedures on all projects, because of the expertise required to conduct hazard analyses. If a site-specific analysis is conducted, a good practice is to assemble a peer review team to review the assumptions and methodologies followed during the study. It is also important to compare the results to the AASHTO map or USGS values. Unless special circumstances exist, the site-specific response spectrum based on a 7% probability of exceedance in 75 years should be no less than 2/3rds of the spectrum derived from the AASHTO maps.

19.2.2.3 Site-Specific Deterministic Seismic Hazard Analysis

Occasionally, the determination of design ground motions will be made using deterministic methods. This approach involves:

- identifying potential sources of ground motion (e.g., active faults);
- determining the magnitude associated with the fault if it ruptures; and
- estimating ground motions at a site based on the distance between the fault and the site.

A number of ground motion attenuation models are available for estimating the ground motion at a site, given the location and size of the fault rupture. Seismologists and earthquake engineers have developed these models by correlating measured ground motions during earthquakes to earthquake source characteristics. These relationships usually depend on the length and location of the fault, the distance to the site and the style of rupture (e.g., normal, reverse, strike slip). The relationships can be for sites comprised of rock or soil.

It is important to use an appropriate fault model when evaluating the level of ground motion that could occur, as the attenuation relationships vary with faulting style. This usually requires a study of the fault type and activity. When evaluating these sources in a deterministic analysis, it is desirable to use several of the available attenuation relationships to account for the uncertainties in each model.

Deterministic procedures are typically not used in the design of highway projects, primarily because the deterministic method does not give any consideration to the likelihood of occurrence. Rather, it is assumed that the fault will rupture during the design life of the structure. Usually, the ground motion from the deterministic analysis will be greater than the probabilistic method. Deterministic procedure might be required occasionally, for instance, where a roadway crosses over a high-hazard dam.

If a deterministic procedure is used, compare the ground motions values (i.e., PGA and spectral accelerations) to that obtained from the AASHTO seismic hazard maps. For MDT projects, the design ground motions obtained from a deterministic evaluation should be no less than the ground motions from the AASHTO seismic hazard maps.

19.2.3 <u>Adjustments for Local Site Conditions</u>

Two methods can be used to adjust for local soil conditions. One involves the use of charts in Section 3 of the *LRFD Specifications*. The other involves conducting site-specific, dynamic ground response analyses.

19.2.3.1 AASHTO Simplified Charts

The AASHTO simplified procedure involves the use of Site Factors (F_{pga}, F_a and F_v) that are based on the soil characteristics at the site and the level of ground motion. Site conditions are characterized in terms of a weighted average of the shear wave velocity, SPT N-value or shear strength of the soil or rock in the upper 100 ft (30 m) of profile. Of the various methods for defining Site Class, the shear wave velocity is the preferred, as it is consistent with the original studies used to define the site class. With the weighted average shear wave velocity, N-value or shear strength and the ground motion parameters on rock from the AASHTO seismic hazard maps or CD (i.e., PGA, S_s and S_1), the Site Factors (F_{pga}, F_a and F_v) can be derived. Linear interpolation is used for intermediate values of PGA, S_s and S_1.

The Site Factors represent the ratio of input motions at rock to output motion at the ground surface. These factors can result in either amplification or deamplification of rock motions. The Site Factor is multiplied by the rock response spectrum to obtain the response spectrum at the ground surface. Amplification typically occurs for lower levels of ground shaking where soils are relatively linear in their response. At high shaking levels, deamplification can occur for softer soils as they yield under the seismic loading. The amount of amplification of the PGA and S_s for low levels of ground shaking is as high as 2.5; the amount of deamplification for hard rock or for high levels of ground shaking in soil can be as low as 0.8. For longer period ground motions as represented by F_v, amplification and deamplification range from 0.8 to 3.5, depending on the level of ground shaking on rock.

When determining the Site Class, consider the following:

- The Site Class is determined for the soil profile existing at the project location following construction. If the soil conditions are significantly different on either side of a structure, then it is often best to define the Site Class for each location.
- It is important to use the weighting procedure given in the *LRFD Specifications* and not simply take the mean of the soil property within the upper 100 ft (30 m), as the two results can differ in some situations.
- For locations characterized by rock within the upper 100 ft (30 m), a shear wave velocity should be assigned to the rock.
- Where the depth to rock is relatively shallow, say less than 50 ft (15 m), it may be desirable to conduct site-specific ground response studies, as the simplified AASHTO site factors can be unconservative, particularly if the bridge planned for the site has a relatively low fundamental period (e.g., less than 0.5 sec).
- For sites characterized by Site Class F, conduct a site-specific ground response analysis as discussed in Section 19.2.3.2.

While the determination of the Site Class following the method in the *LRFD Specifications* appears to be relatively straightforward, a number of questions often occur when these methods are used. The following provides the answers to these questions that have not always been discussed in the *LRFD Specifications*:

- One common question is where to define the ground motions if the bridge is located on drilled shafts at a site where a thin layer of alluvium occurs over rock. In this case the response of the foundation is primarily determined by the ground motion in rock, and the soil has little effect on bridge response.

- A similar situation occurs if a thin, soft-soil layer occurs over stiff or dense soil. Typically, response of the foundation is determined by ground motions at the depth of pile fixity, which is typically located between 5 and 10 pile diameters below the ground surface. One approach that has been used for this situation is to ignore the soil above the depth of fixity and determine the Site Class based on the average soil conditions to 100 ft (30 m) below the depth of fixity. A consensus on how to treat these conditions is not currently available.

- Another question deals with whether to define the ground motion at the top of the approach fill or at the original ground surface. This answer will depend on the design of the bridge and whether ground motions at the approach fill will "drive" the seismic response of the bridge (e.g., as may occur with an integral abutment) or response is determined by response of center piers. In this case, determine the Site Class at the location that contributes most to the seismic response of the bridge.

Generally, the project geotechnical specialist will often need to use judgment when deciding how to apply the AASHTO Site Class information. Detailed guidance is not always available, and specifics of the particular site and bridge will often determine the most appropriate approach. If there are uncertainties on the appropriate use of the AASHTO simplified method and these uncertainties are having a significant effect on the design of the bridge, it may be appropriate to conduct site-specific dynamic ground response analyses or more complete soil-structure interaction analyses. For these cases, detailed discussions should take place with the Bridge Bureau to understand the importance of the uncertainties and to decide whether alternative methods are appropriate.

19.2.3.2 Site-Specific Dynamic Ground Response Evaluations Using Computer Modeling

In some cases, a more detailed evaluation of the soil effects might be required than can be obtained from the simplified AASHTO charts. This can occur at variable site conditions or where the thickness of the soil layer above rock is limited (e.g., less than 50 ft (15 m)). In this case, one-dimensional, equivalent-linear modeling using the computer program SHAKE or a non-linear program (e.g., DMOD) can be used to evaluate the site amplification or deamplification.

The site-specific, dynamic ground response analysis requires determination of dynamic soil response in two areas: (1) the shear wave velocity of the soil to define the low-strain shear modulus and (2) the effects of shearing strain on the modulus and material damping properties of the soil. This information is typically obtained by conducting field investigations and relying on published correlations:

- The preferred approach for defining the low-strain shear modulus is to measure the shear wave velocity using seismic geophysical methods; see Chapter 8 on Subsurface Investigation/Field Tests. Alternatively, empirical equations that relate shear wave velocity to factors such as SPT N-values can be used to estimate the low-strain shear wave velocity.

- The effects of shearing strain on soil shear modulus and on material damping are normally obtained from published modulus ratio and damping curves. Section 19.5.2 provides typical curves that are often used for this type of analysis. For special projects, dynamic laboratory tests might be considered; see Chapter 8.

A series of earthquake time histories are used as inputs to the site-specific, dynamic ground response model. These earthquake records are selected from the available large database of records (e.g., Pacific Earthquake Engineering Research Center) to represent the likely motions that could occur at the site. Selecting the earthquake record requires careful evaluation to obtain motions that are consistent with the size, location and style of earthquake source. Typically, either three or seven records are used, as follows:

1. Three Records. If three records are used, the model provides three sets of results showing response versus structural period. Draw an envelope around the response that encompasses the highest response at each of the structural periods.

2. Seven Records. If seven records are used, the model provides seven sets of results as a function of the structural period. In this case, use the mean of seven data points at each of the periods. The seven records will usually result in a lower response.

Most site-specific ground response analyses in the past were conducted using the computer program SHAKE. This program uses equivalent linear procedures to model the nonlinear response of the soil during seismic loading. Changes in soil stiffness and damping from pore water pressure effects are not accounted for in the model and, therefore, the approach has limited validity where liquefaction within saturated cohesionless soil layers occurs. The equivalent linear procedure can also introduce uncertainties in ground response predictions at soft soil sites when very high levels of ground shaking occur. The preferred approach for liquefiable sites or for soft soil sites with large ground motions is to use a computer program that more directly accounts for the nonlinearity of the soil during the earthquake loading sequence, including build-up and dissipation of porewater pressures at liquefiable sites. These programs are referred to as effective stress methods. By including the porewater pressure build-up, the effects of reduced soil stiffness and strength on the ground motions are obtained. For liquefiable sites and for deep soft soil sites, the nonlinear effective stress method will often result in lower values of PGA and spectral acceleration.

Nonlinear effective stress computer programs can now be purchased from computer software vendors and, therefore, the use of this approach will become more common in the future. Considerable expertise is required when conducting analyses using nonlinear, effective stress methods. If these methods are being proposed on an MDT project by a consultant, the Geotechnical Section will need to review and approve the use of this method. This approval will be based upon a review of the qualifications and experience of the consultant in conducting this type of analysis. It is also very desirable to include an independent peer reviewer with expertise in these analyses to review the assumptions made and the properties being used in the analysis.

19.3 FAULT LOCATION AND ACTIVITY

The project geotechnical specialist should consider the potential for active faults along the project alignment during design. Displacements associated with active faults can range from a few inches (millimeters) to multiple feet (meters). The direction of the movement can be lateral, vertical or some combination of the two. The consequences of this movement can be very serious to structures or roadways located over the fault, potentially resulting in structure collapse or large offsets in the road surface.

This is not usually a critical design concern because of the limited number of active faults in Montana. The recommended approach is to perform a check of available fault maps to confirm that the project alignment is not located over an active fault. Use the USGS and Montana Department of Natural Resources maps to identify the locations of active faults.

Various methods can be used to determine whether faults are located along an alignment if uncertainty exists from the published information. One procedure involves the use of LiDAR methods (Light Detection and Ranging) to obtain a visual image of the ground surface. Other methods include aero-magnetic procedures, which are used to identify likely fault traces. Trenching procedures are then used to confirm that a fault exists and to investigate the activity of the fault. An inactive fault is usually assumed to have a low likelihood of faulting. Faults are generally assumed to be inactive if they have not moved in the past 10,000 to 15,000 years.

If an active fault is identified for a project site, the preferred alternative is to modify the alignment away from the fault trace. If there is uncertainty in its location, it may be necessary to trench across the suspected fault and evaluate where evidence of recent faulting has occurred. The evidence of faulting could include displaced bedding planes or evidence of sand boils. Carbon dating can be used to determine the fault activity and the last date of movement.

19.4 GEOTECHNICAL HAZARD EVALUATIONS

19.4.1 General

A number of geotechnical hazards can result from strong ground shaking including, but not limited to:

- liquefaction of saturated granular soil;
- settlement, bearing failure, downdrag and lateral flow associated with liquefaction;
- slope failures; and
- additional earth pressures on retaining walls.

The Geotechnical Section is responsible for conducting the investigations and evaluations necessary to assess the likelihood of these occurrences and, as appropriate, to identify mitigation methods to eliminate or reduce the consequences.

19.4.2 Liquefaction Potential

Liquefaction occurs when loose, cohesionless soils located below the groundwater table undergo strong vibratory loading. Porewater pressures within the soil increase as the loose material tends to densify. Soil liquefaction occurs when the increase in porewater pressure equals the effective stress in the soil. In this state, the soil loses shearing strength, potentially leading to bearing failures or slope instability. After the earthquake ground shaking stops, the excess pore pressures dissipate, resulting in settlement. The settlement can effect roadways, retaining walls, bridges, culverts, spread footings and potentially cause downdrag on piles located in the settling soil.

Liquefaction analysis usually begins with a preliminary screening that evaluates three factors to rule out liquefaction. A detailed evaluation of liquefaction potential is not required if one or more of the following conditions occurs at the site:

- The estimated maximum groundwater level at the site is determined to be deeper than 75 ft (25 m) below the existing ground surface or proposed finished grade, whichever is deeper.

- The subsurface profile is characterized as having a minimum SPT resistance, corrected for overburden depth and hammer energy $(N_1)_{60}$, of 30 blows/ft (30 blows/0.3 m), or a cone tip resistance q_c of more than 160 tsf (15 MPa), or if the bedrock is present to the ground surface.

- The soil is clayey, as defined by the recommendations given by Idriss and Boulanger (2006) or Bray and Sancio (2006).

19.4.2.1 Field Methods

The potential for liquefaction is usually evaluated using empirical relationships between the liquefaction strength of the soil and the Standard Penetration Test (SPT) N-value or other soil characterization based on in-situ testing. These relationships are described in Youd et al.,

2001. Since the consensus paper by Youd et al. (2001), various other studies have been conducted on estimating liquefaction potential. For example, see papers by Cetin et al. (2004), Moss et al. (2006), and Idriss and Boulanger (2006). These papers update previous interpretations of liquefaction potential, resulting in slightly different charts and equations for evaluating liquefaction potential. Until a consensus exists on the adoption of the updates, the prudent approach is to perform an independent check using an alternative liquefaction analysis method.

The following additional factors must be considered when conducting liquefaction analyses:

1. Hammer Energy. For the SPT method, it is critical that the hammer energy be consistent and quantifiable. The preferred hammer source is the auto-hammer, because it is much less sensitive to operator procedures than a safety or donut hammer. The hammer energy for the auto-hammer is also relatively constant from blow to blow and from project to project. Nevertheless, it is still advisable to periodically calibrate the energy using ASTM procedures. If a safety or donut hammer is used, the energy should be calibrated more frequently to account for variations from equipment and operators.

2. Liquefiable Soils. Another key consideration during the liquefaction analysis is the determination of soils that are liquefiable. In the past, the "Chinese Criteria" was used to establish whether soils were liquefiable. MDT strongly recommends against the sole use of the Chinese criteria. Boulanger and Idriss (2006) suggest that soils with a PI ≥ 7 be considered clay-like in behavior. Although clay-like soils can undergo cyclic softening, they will not liquefy. Bray and Sancio (2006) suggest defining a soil as being liquefaction susceptible if the PI < 12 and the moisture content to liquid limit ratio (wc/LL) > 0.85. Again, no consensus exists within the profession at this time regarding the preferred method; therefore, the prudent approach is to check both of the above methods.

3. Liquefaction Depth. The maximum depth of liquefaction is another issue that must be considered by the project geotechnical specialist. In most cases, liquefiable soils are found close to the ground surface but, on occasion, the depth of loose saturated sands can be 75 ft (25 m) or more. The current consensus is that liquefaction analyses should be conducted to a depth of at least 75 ft (25 m). If liquefaction susceptible material exists below this depth, the potential consequences of deeper liquefaction should be considered. At a minimum, piles should be driven deep enough to reach a non-liquefiable bearing layer.

Three other field methods are also used for the assessment of liquefaction potential — the cone penetrometer test (CPT), shear wave velocity (SWV) test and Becker hammer test (BHT). The paper by Youd et al. (2001) provides a discussion of methods for evaluating liquefaction using SPT, CPT, BHT or SWV methods.

Each of these methods has relative advantages and disadvantages. For example, the CPT method is appropriate for identifying and evaluating thin liquefiable layers that could serve as sliding surfaces; the SWV method can be performed without mobilizing a drill rig to the site; and the BHT method is suitable if gravelly soils are present. Neither the SWV nor CPT method provides a sample of soil, which is a primary limitation of these methods. Often, a combination of methods is preferred.

When performing the liquefaction analysis, the project geotechnical specialist must estimate the magnitude of the earthquake. The magnitude is then used in the empirical liquefaction estimate (e.g., SPT, CPT, SWV or BHT method) to adjust the liquefaction strength to be consistent with the likely duration (or number of cycles) of earthquake loading. Charts and equations within the methodology facilitate the liquefaction strength adjustment; however, the designer must select the earthquake magnitude. Two approaches are typically used to make this estimate:

1. Experience based on the size of faults in the vicinity of the site. Empirical equations are available for estimating magnitude based on the length of faults. With this approach, the magnitude can be correlated to the level of ground motion likely to occur at the site.

2. Information in the USGS hazard website. Tabulations and plots of mean and modal magnitude can be obtained for the PGA, given latitude and longitude of the site and the return period of the earthquake (usually 975 years). Tabulated results also indicate the distance-magnitude combinations contributing most to the hazard. Typically, the mean and modal magnitudes are similar and are sufficient for use in the liquefaction analysis.

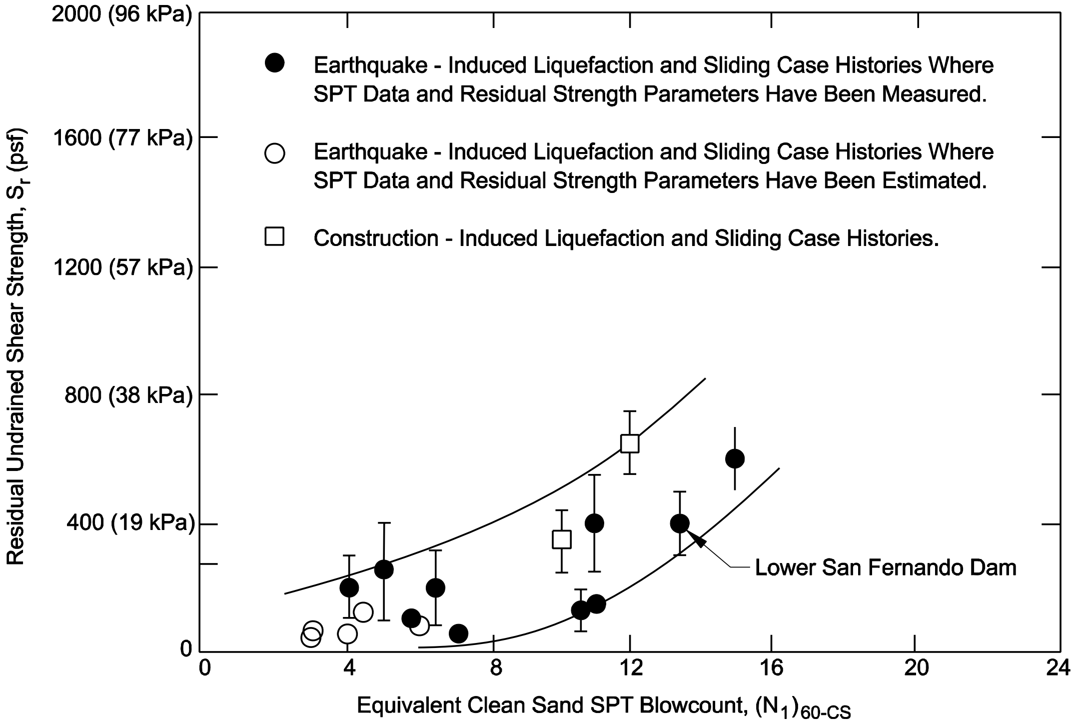
19.4.2.2 Consequences of Liquefaction

The potential consequences of liquefaction will depend on the thickness of the zone that has liquefied, the presence of any structures, the geometry of the site and if the area is determined to be in a critical location. As the soil liquefies, it loses strength. When fully liquefied, the soil is characterized by a residual strength.

A number of relationships have been developed during the past 20 years to determine the residual strength of liquefied soils. One of the most commonly used methods was developed by Seed and Harder (1990) and is shown in Figure 19.4-A. There is a relatively large range in values shown in Figure 19.4-A. To be conservative a value at the mean or at the lower third is often used for design. Newer methods include those by Olson and Stark (2002) and Idriss and Boulanger (2007). The equations developed by Olson and Stark and by Idriss and Boulanger are based on either SPT or CPT data, and are related to the initial effective confining pressure. There currently is no consensus on which of these methods to use; therefore, it is advisable to use two or more of the methods.

The increase in porewater pressure and reduction in strength leads to the following consequences:

1. <u>Loss in Soil Bearing Capacity</u>. This could affect a footing or embankment located on a liquefied layer. If the material liquefies, the soil strength could be reduced to the residual strength. Equations for cohesive bearing capacity are used in a liquefaction bearing capacity analysis, with the cohesion defined by the residual strength of the liquefied soil. If the load exceeds the capacity, large settlement, tilting, or overturning of the foundation might be expected.



Recommended Fines Correction for S_R Evaluation Using SPT Data

Percent Fines	N_{corr} (blows/ft) (blows/0.3 m)
10%	1
20%	2
50%	4
75%	5

Figure 19.4-A — RELATIONSHIP BETWEEN RESIDUAL STRENGTH (S_r) AND CLEAN SAND SPT BLOWCOUNT $(N_1)_{60}$ (Seed and Harder, 1990)

2. Flow Failures and Slope Instability. If the soil liquefies beneath a slope, there is a potential for either flow failures or slope instability. For flow failures, the strength of the liquefied soil is not sufficient to support the gravity loads after the earthquake has ceased. These failures can occur on very flat slopes, even less than 10°, and are frequently observed in areas adjacent to rivers. Slope instabilities can occur either during the earthquake from the added inertial forces in combination with reduced soil strength, or after the earthquake when porewater accumulations eventually lead to sufficient loss in strength and gravity loads are no longer supported.

3. Post-Earthquake Settlement. Liquefied soil undergoes a decrease in volume as pore pressures dissipate. This decrease can be as much as several percent of the liquefied layer thickness. As the soil settles, downdrag forces can be imposed on piles extending through the liquefied soil. If these conditions are possible, then consider the secondary effects of drag loads on the pile foundation. Post earthquake settlement may also occur in embankments and earth retaining structures.

4. Reduced Lateral Pile Response. With the reduction in soil strength, the lateral resistance of deep foundations will decrease. The typical approach for addressing this reduction is to conduct lateral analyses with softened P-y curves using the residual strength of the liquefied soil or to neglect the layer altogether. Common practice has been to use the soft clay soil model when conducting these analyses. Results from recent full-scale field tests on piles subjected to lateral loading in liquefied soil suggest that the deformation required to mobilize the residual strength may be greater than results if the soft clay model is used and alternate models should be considered. The L-PILE software and the DF-SAP software (see Chapter 13) include these alternative models.

Although the current edition of the *LRFD Specifications* does not provide guidance on evaluating the above consequences, various evaluation and design methods are available from ATC/MCEER (2003), MCEER (2006) and SCEC (1999).

19.4.2.3 Mitigation of Liquefiable Conditions

It may be necessary to mitigate the potential for liquefaction if liquefaction is predicted for the design seismic event and if the implications of liquefaction have significant life-safety, economic or environmental consequences. Various mitigation methods are available. These methods involve either ground improvement or the use of structural systems.

The most common ground improvement methods are:

1. Remove and Recompact. It is often possible to excavate and recompact the loose, cohesionless soil if the depth of liquefaction is relatively shallow. This approach is only suitable for relatively shallow depths, less than 10 ft to 15 ft (3 m to 5 m) and where dewatering is feasible.

2. Vibro-Densification. This approach involves densifying the soil in place with a down-the-hole vibrating tool. The probe is jetted to the depth of interest and then vibrated incrementally upward in lifts as the tool is removed. Additional backfill is added to help in the densification process. Probe columns are generally located from 5 ft to 10 ft (1.5 m to 3 m) center to center.

3. Stone Columns. This method involves a procedure similar to vibro-densification using columns of stone (or gravel) formed on 5 ft to 10 ft (1.5 m to 3 m) centers. After drilling to the maximum depth of improvement, stone is densified in place as the tool is removed. The diameter of the stone columns will usually range from 30 in to 36 in (750 mm to 900 mm).

4. In-Place Soil Mixing. In-place soil mixing usually involves mixing cement with the native soil to form columns or cells of cemented soil. Various methods are used to mix the soil, including jet grouting procedures and large-paddle augers for cement deep soil mixing (CDSM). The depth of improvement can be 100 ft (30 m) or more.

Mitigation using structural systems incorporates a structural element that provides additional shear capacity to the soil. Examples of this approach include:

1. Displacement Piles. Piles are driven at relatively close spacing, 5 ft to 10 ft (1.5 m to 3 m), and the soil is subsequently strengthened by a combination of densification from the displacement of the pile during installation and from additional shear capacity from the structural element. The displacement piles can be concrete, timber or steel. The piles are not tied together with a pile cap.

2. Micropiles. These drilled piles consist of a high-strength, small-diameter (< 12 in (300 mm)) pipe section grouted into a borehole. The micropiles are located at relatively close spacing, 5 ft (1.5 m) on center or less, and splayed in two directions. A cap foundation is located at the ground surface to tie the system together. The soil is strengthened by the combination of structural compression and pullout of the micropile and the bending stiffness of the piles.

3. Sheetpiles. Steel sheetpiles have been used at the toe of an embankment to limit sliding failure of the embankment slopes. The sheetpiles are designed to interrupt the failure plane that develops during instability.

19.4.3 Seismic Slope Stability

The stability of slopes during seismic loading should be assessed as part of the geotechnical evaluation at a site. Slope instability can be a significant concern, especially if failure of the slope could pose a risk to the public, or if the slope could damage an existing bridge or close a critical transportation route. In many cases, however, the cost of repair after failure of the slope is not overly expensive or time consuming relative to the cost of mitigation to prevent seismic-induced slope instability. In these cases, MDT may decide to accept the risk of slope failure. The likelihood for seismic-induced slope instability should be documented in the Geotechnical Report.

19.4.3.1 Pseudo-Static Method of Analysis

For most evaluations, pseudo-static methods of analysis can be used to evaluate the effects of earthquake ground shaking on existing or new slopes. Most conventional slope stability programs are capable of pseudo-static analyses through assignment of a seismic coefficient. The seismic coefficient is applied to the soil mass within the critical slope surface, resulting in a large horizontal inertial force. If the forces and moments under this load exceed the shear and moment capacity of the soil, a factor of safety (FS) of less than 1.0 results.

A number of factors must be considered when conducting a pseudo-static analysis:

1. Seismic Coefficient. The seismic coefficient used in the analysis is usually a fraction of the peak ground acceleration determined for the soil mass (i.e., PGA on rock adjusted for site effects). If several inches (millimeters) of permanent ground displacement are acceptable, then a seismic coefficient of 50% of the PGA can usually be safely assumed. If larger displacements are acceptable, the reduction can be even greater. However, in this case an estimate of the ground displacement should be made using, for example, the Newmark method. Typically, the seismic coefficient in the vertical direction is assumed to be zero during the analysis.

2. Factor of Safety. If the seismic coefficient used during the seismic analysis is reduced by 50%, MDT requires that the resulting Factor of Safety (FS) be greater than or equal to 1.1 for acceptable performance. At critical locations higher factors of safety may be required.

3. Soil Strengths. Soil strengths used in seismic analysis should be based on undrained strengths or total stress parameters for most cases, even for silty sands. Undrained strengths are used because of the rapid application of stress cycles in combination with the short duration of loading. For most soils, the rate and duration of loading are such that drainage will not occur, thereby necessitating the use of undrained strength parameters.

4. Stability Analysis. When conducting a seismic stability analysis, the slope stability computer program should be allowed to "search" for the most critical sliding surface, rather than fixing the analysis for the critical surface determined during static analysis. During seismic loading, the inertial force from the earthquake results in a flattening of the failure surface; consequently, use of the static surface will be unconservative.

The stability assessment should carefully consider the occurrence of liquefiable layers because liquefaction within a saturated, loose sand layer can result in a significant reduction in strength. If these layers are identified during the site exploration program and found to be liquefiable during engineering analyses, then use the residual strength of the soil in the stability analysis.

19.4.3.2 Simplified Displacement Analysis Methods

In some cases, it may be necessary to estimate permanent displacements of the slope during seismic loading. For example, if the FS from the pseudo-static analysis is less than 1.0, a deformation analysis can be conducted to estimate the displacements associated with the low FS. Alternatively, if displacement-sensitive pipelines or structures are located in or on a slope, it may be necessary to evaluate the permanent displacement that could occur during an earthquake.

The most common method of evaluating permanent deformations involves the use of the Newmark-sliding block method. In this method, the ratio of yield acceleration to peak ground acceleration is related to permanent displacements. Various charts and equations have been developed to show this relationship (e.g., Makdisi and Seed, 1978; Bray and Rathje, 1998; ATC/MCEER, 2003). These methods were developed using a combination of simplified Newmark and more rigorous methods. They often differ in terms of the databases that were used to develop the correlation between peak ground acceleration, yield acceleration and ground displacement. The yield acceleration for this analysis is the seismic coefficient that results in a factor of safety of 1.0 in the pseudo-static slope stability analysis.

The generalized steps involved in the Newmark analysis are as follows:

1. Determine the PGA on rock from the AASHTO hazard maps. Modify the PGA for site effects using the AASHTO site factors.

2. Determine the yield acceleration by conducting a pseudo-static slope stability analysis. Vary the seismic coefficient until the FS = 1.0 to define the yield acceleration.

3. Determine the acceleration ratio by dividing the yield acceleration by the PGA adjusted for site effects.

4. Use one of the available charts (i.e., Makdisi and Seed, 1978; Bray and Rathje, 1998; ATC/MCEER, 2003) to estimate displacements. *Note that the 2008 Interim Revisions to the 4th Edition of the LRFD Specifications includes a chart in Appendix A to Section 11 showing the estimated displacement for different acceleration ratios (i.e., yield acceleration to peak ground acceleration). The curves in this appendix are generally considered to be too conservative, particularly when the acceleration ratio is less than 0.5.*

Usually, the peak undrained strength is used for the stability analysis. However, if large deformations are predicted (e.g., 12 in (300 mm) or more), it is advisable to estimate the displacements using the residual strength of the soil. If liquefaction is predicted to occur, use the residual strength of liquefied soil.

19.4.3.3 Numerical Modeling

For special projects, the use of numerical models in advanced computer programs may be appropriate. These computer programs use finite element or finite difference methods to evaluate two-dimensional geometries under static and dynamic loading. The programs provide estimates of stresses and deformations in the soil as a function of various loading conditions, and they allow structural elements (e.g., walls, piles) to be included in the evaluation. The effects of pore-pressure buildup on stability can also be determined.

A number of important modeling requirements must be considered when using these programs:

1. <u>Earthquake Records</u>. The earthquake records must be selected and scaled to be consistent with ground motions expected for the site. Normally, three to seven records should be used to adequately capture the variability in response to changes in earthquake loading.

2. <u>Boundary Conditions/Soil Profile</u>. The boundary conditions and the soil profile must be carefully defined. Note that, for stability analyses, thin layers can serve as sliding surfaces. Therefore, it is important to define the finite element or finite difference mesh size appropriately.

3. <u>Soil Strengths/Drainage Conditions</u>. Soil strengths and drainage conditions that will result in appropriate drainage conditions during loading must be specified. Generally, for cohesive soils, use the total stress or undrained strengths. Cohesionless soils should consider the potential for porewater buildup and redistribution.

Considerable experience is required when using these numerical modeling methods. Subtle changes in boundary conditions or material property characterization can have a significant effect on response predictions, making it difficult to know whether the analysis is providing reasonable results without a detailed parametric evaluation and review. Numerical modeling results must be checked with simplified methods to determine reasonableness of the model. If an outside consultant is proposing to use advanced numerical modeling methods, the Geotechnical Section will carefully review the qualifications of the outside consultant to confirm that they have appropriate experience in using the numerical method, and that their quality assurance includes checking of input-output information, simplified reality checks and a senior review by someone with expertise in the area of modeling. An independent peer review of this type of modeling will most likely be performed.

19.4.4 Dynamic Earth Pressures

Dynamic earth pressures must be estimated for free-standing retaining walls and for bridge abutments and footings. Seismic active earth pressures result from the inertial response of the soil mass behind the wall. Seismic passive pressures are developed where the footing moves into the soil, as might occur for bridge footings and abutments.

19.4.4.1 Mononobe-Okabe Equations

The most common method of estimating seismic active and passive earth pressures is the Mononobe-Okabe (M-O) equations given in *LRFD Specifications* Appendix A11. The M-O equations are based on Coulomb wedge theory for homogeneous, dry, cohesionless soil. As such, there are significant limitations associated with these equations if soil conditions depart from this assumption, which is often the case. Experience has shown that the M-O equation for seismic active earth pressure can result in unreasonably high pressures for some combinations of ground motion and backslope condition. Section 19.4.4.2 presents an alternative method for estimating seismic active earth pressures that avoids some of these limitations.

When using the M-O equation for estimating the seismic active earth pressure, AASHTO allows the seismic coefficient to be reduced by 50%, resulting in a 50% reduction in the design seismic earth pressure. This reduction is based on the assumption that the wall can undergo some permanent displacement during the earthquake. The amount of displacement is usually assumed to be approximately 4 in to 6 in (100 mm to 150 mm). Recent work suggests that the displacement at 50% of the seismic coefficient could be on the order of 1 in to 2 in (25 mm to 50 mm) (ATC/MCEER, 2003). For walls that rely on gravity for stability (e.g., CIP standard cantilever, MSE, prefabricated modular, soil nail walls), this displacement allowance is acceptable. However, for walls supporting critical facilities or utilities, the permanent displacement of several inches (millimeters) may not be acceptable. In this case, the peak ground acceleration may need to be used for design. For the same reason, nongravity cantilever and anchored walls may not be able to tolerate or develop this amount of movement, and these walls may also need to be designed with the peak ground acceleration.

An M-O equation for passive earth pressure determination is also presented in *LRFD Specifications* Appendix A11. This equation is generally not recommended for use, despite its simplicity. In addition to being applicable for only homogeneous, cohesionless soils, the equation is based on Coulomb wedge theory. Various studies have shown that Coulomb theory can overestimate the passive earth pressure relative to the more accurate log-spiral method. The paper “Nonlinear Soil-Abutment-Bridge Structure Interaction for Seismic Performance-Based Design” (Shamsabadi, et al., 2007) presents a log-spiral method for estimating seismic passive pressure with seismic loading effects. This approach offers perhaps the best method of defining earth pressures for nongravity cantilever walls. For shallow embedment, such as might occur for a CIP standard cantilever wall or for a bridge abutment, the static passive earth pressure coefficients given in Section 3 of the *LRFD Specifications* provide a reasonable approach for defining the resistance for the passive pressure case. Use of the static pressure for the abutment is based on the relatively slow rate of loading resulting from the inertial response of the bridge deck. It is important to include wall friction in the passive pressure estimate. Typically, for seismic loading the interface friction will range from half of the soil friction angle to the full friction angle, depending on the method of constructing the footing.

19.4.4.2 Generalized Limit Equilibrium Method for Estimating Active Earth Pressures

An alternative to the M-O equation for determining seismic active earth pressures is to use a slope stability computer program. The seismic active pressure can be obtained by specifying an external force at the back of the wall, and then varying this external force until FS = 1.0 under the specified seismic coefficient. The external force at equilibrium is equal to the seismic active earth pressure.

This approach is relatively simple and has a number of significant benefits. Most importantly, the soil conditions behind the wall can be modeled correctly for seismic loading. This is especially important for cut slopes where cohesive layers occur in the backfill. It can also be important if the cut slope behind the wall is steep (e.g., greater than 1H:1V) and the soil characterizing the cut material is very strong relative to the backfill, as is the case for rock or a till. In this case, the failure surface for an M-O analysis may not develop, resulting in an overestimation of the seismic active earth pressure. The generalized limit equilibrium method can account for this effect. The generalized approach can also include the effects of a phreatic surface within the failure wedge.

If the generalized equilibrium method is used to estimate the seismic active earth pressure, the recommended approach is to distribute the resulting force as a uniform load at the back of the wall. Various studies have been performed to evaluate the shape of the seismic pressure distribution. The results of these studies do not provide a clear consensus on the distribution; for simplicity, the uniform distribution is currently considered appropriate. Note that the common procedure of subtracting the static earth pressure from the seismic earth pressure is not appropriate unless the static earth pressure is also determined using undrained or total stress strength parameters. The guidelines in the *LRFD Specifications* for static earth pressure distribution are for long-term drained loading, which is not usually appropriate for seismic loading.

19.4.4.3 Nonyielding Walls

Most freestanding retaining walls, including anchored walls, will be flexible enough to develop the active earth pressure. Only small deformations are necessary for active pressure development. However, in some cases, the wall is assumed to be nonyielding under gravity loads, in which an at-rest (k_o) earth pressure coefficient should be used to calculate lateral soil loads. Common practice is to apply an incremental pressure increase on the static at-rest k_o pressure condition for seismic loading.

Usually, the incremental increase in stress above the at-rest case will not develop for seismic loading because of the small movements that occur either to the structure or the ground during seismic loading. One approach for the non-yield wall case is to use the higher of the static at-rest earth pressure or seismic active earth pressure for design. This approach is different than given in the *LRFD Specifications*, which suggests using the M-O equation but increasing the seismic PGA acceleration used in the equation by a factor of 1.5 for abutment walls restrained by tiebacks or batter piles. This is judged to be too conservative, because even tiebacks and battered piles will usually move enough to develop active earth pressures. Therefore, increasing the PGA by a factor of 1.5 would not be consistent with expected performance.

19.5 INPUT TO BRIDGE BUREAU

For many projects, particularly in western Montana, the Geotechnical Section is expected to provide the Bridge Bureau with information needed for the seismic design of the bridge. This information could include:

- ground motion parameters (e.g., PGA, S_s, S_1) from either the AASHTO seismic hazard CD or a site-specific PSHA;
- identification of Site Class using either the AASHTO simplified method or site-specific ground motion;
- seismic earth pressure information; and
- potential for liquefaction.

The Geotechnical Section may also need to provide information that can assist the Bridge Bureau in developing foundation springs, or stiffness values, for use in structural seismic loading analyses. The following sections provide an overview of these requirements. Discussions should be held with the Bridge Bureau before any seismic studies are performed to confirm the Bridge Bureau's requirements for the particular project.

19.5.1 <u>Ground Motion Information</u>

The ground motion information will usually be limited to the Site Class for the project site. The Bridge Bureau will determine PGA, S_s, S_1 using the latitude and longitude for the site. The Site Class should be determined following the methods given in the *LRFD Specifications*, unless site-specific ground motion studies are conducted.

If the Site Class is expected to differ at each bridge abutment, provide values for each abutment. Also, indicate whether there is a potential for liquefaction at the site, as the Bridge Bureau will have to conduct separate analyses for the liquefied and non-liquefied cases. If the Site Class is borderline between, for example, Site Classes D and E, the Bridge Bureau should be advised of the potential for either class being appropriate. Similarly, if the depth to bedrock is less than 50 ft (15 m), advise the Bridge Bureau that there is a potential for higher spectral accelerations at periods less than 0.5 seconds and that a site-specific determination of ground motions may be desirable to refine the spectral accelerations if the bridge has a predominant period in this range.

19.5.2 <u>Shear Modulus, Material Damping and Poisson's Ratio</u>

If the use of a shallow foundation is anticipated, the Bridge Bureau may request that the Geotechnical Section provide recommendations on the appropriate shear modulus, material damping and Poisson's ratios to use when estimating spring constants required for seismic design. On some occasions, the Bridge Bureau may also request a profile showing the shear wave velocity variation with depth. This information is usually needed to a depth of at least twice the minimum foundation width. If the foundation width has not been finalized, then a contingency should be used when selecting the depth to which the information should be provided.

19.5.2.1 Shear Modulus Determination

The shear modulus of the soil is determined either from the shear wave velocity of the soil, or empirical correlations that relate shear modulus to either soil classification properties or other in-situ measurements. At locations where critical structures are present, or where empirical correlations are questionable, measuring the shear wave velocity may be justified. The preferred approach for most sites is to use measurements of the shear wave velocity. Shear wave velocities are converted to the low-strain shear modulus using the following equation:

$$G_{max} = \rho V_s^2$$

Where ρ is the mass density of the soil (= γ/g), and V_s is the shear wave velocity determined by field or laboratory testing methods or by using empirical equations.

The shear wave velocity can be determined using one of the following methods:

1. In-Situ Geophysical Methods. As discussed in Chapter 8, the downhole or seismic cone methods are generally the most cost effective. The SASW method allows a profile to be obtained without borings and, therefore, avoids some field costs. However, the resolution of this method is usually not as good as the borehole methods, particularly at deeper depths.

2. Laboratory Testing Using Resonant Column or Torsional Shear Testing Equipment. High-quality soil samples may be tested if the foundation will be located on native soils. This approach is not commonly used because of the complexity of the testing methods and the adjustments that are required to correct the laboratory velocity to field conditions (e.g., age-related effects).

3. Empirical Relationships. Various empirical equations are available for estimating the shear modulus from classification properties of the soil or from other in-situ test information. *Geotechnical Earthquake Engineering* (Kramer, 1996) provides empirical relationships based on a number of field testing methods, including the Standard Penetration Test (SPT), the Cone Penetrometer Test (CPT), the dilatometer test (DMT) and the Pressuremeter Test (PMT). The empirical methods involve considerable uncertainty and, therefore, it is usually advisable to define a range of values. A variation of plus or minus 25% of the velocity would not be unusual.

The shear modulus determined from shear wave velocity measurements or using empirical relationships will usually be defined at very low shearing strain levels, typically 0.0001% or less. The shear modulus used in the calculation of foundation spring constants will normally be at some higher shear strain level. The shear modulus is inversely related to the shear strain; consequently, higher strain levels result in lower modulus values. Figure 19.5-A (from FEMA 356 *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*) is a guide to relate the G/G_{max} to Site Class. Figures 19.5-B and 19.5-C illustrate the typical reduction in modulus as a function of shearing strain for cohesionless and cohesive soils.

AASHTO Site Class	Peak Ground Acceleration (g)		
	≤ 0.1	0.4	≥ 0.8
A	1.00	1.00	1.00
B	1.00	0.95	0.90
C	0.95	0.75	0.60
D	0.90	0.50	0.10
E	0.60	0.05	*

*Note: *Should be evaluated from site-specific analysis.*

Figure 19.5-A — G/G_{max} VALUES
(As a Function of AASHTO Site Class and PGA)

19.5.2.2 Material Damping Determination

Figures 19.5-B and 19.5-C can be used to estimate the material damping characteristics of the soil. Another alternative is to measure the material damping properties as part of a laboratory testing program being conducted to determine the shear wave velocity or shear modulus of the soil. In-situ methods are currently not available for estimating the shear modulus of soils.

19.5.2.3 Poisson's Ratio Determination

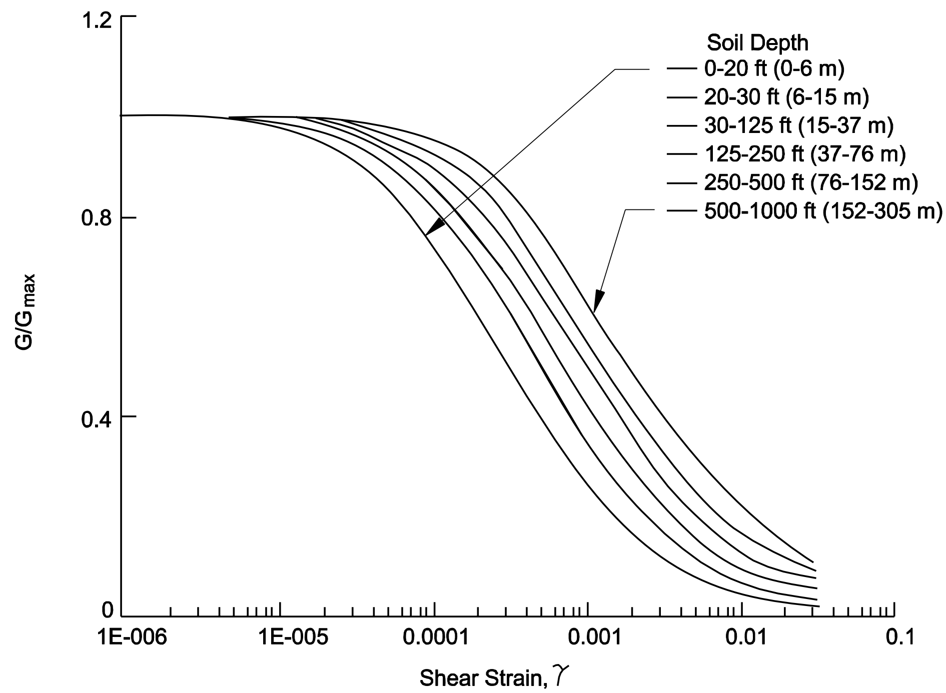
Poisson's ratio can be obtained from field geophysical testing based on the ratio of compression wave velocity to shear wave velocity of the soil or rock. Alternatively, values of Poisson's ratio are commonly published in textbooks. Typically, the Poisson's ratio for sand will be approximately 0.3; for a saturated clay, it will be approximately 0.5.

19.5.3 Foundation Spring Constants

The Bridge Bureau uses spring constants during the modeling of bridge piers and abutments for seismic loading. The Geotechnical Section supports the Bridge Bureau by providing the shear modulus, material damping and Poisson's ratio values. The Geotechnical Section also provides information for developing spring constants for pile and drilled shaft foundations, as discussed below. On some occasions, the Bridge Bureau may also request that the Geotechnical Section provide modulus of subgrade reaction values.

19.5.3.1 Spring Constants for Shallow Foundations

The Bridge Bureau will normally determine the spring constant for shallow foundations using equations for footings on an elastic material. The effects of soil nonlinearity are incorporated by using reduced shear modulus values as discussed above.



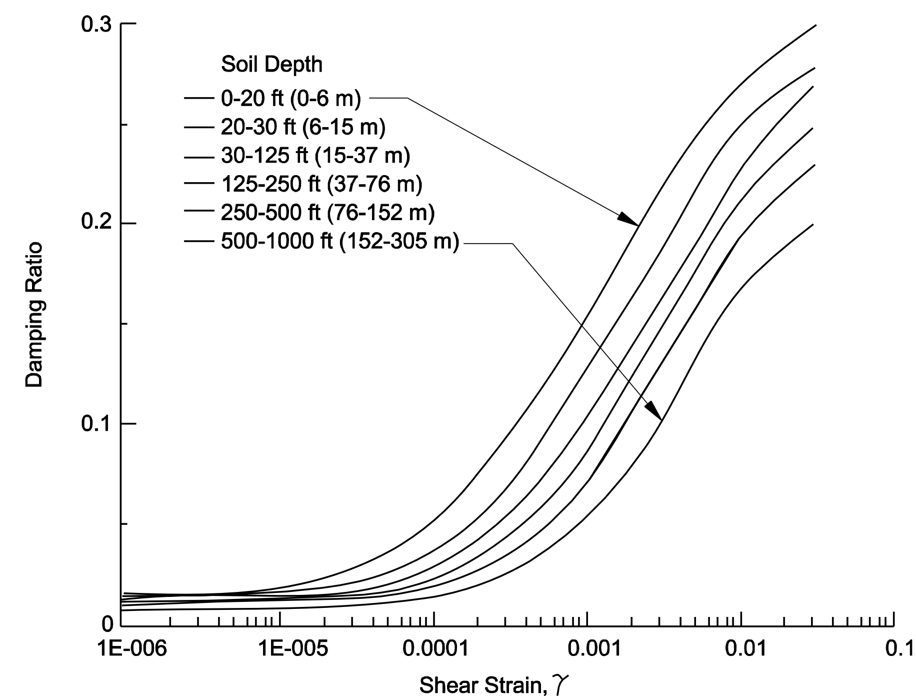
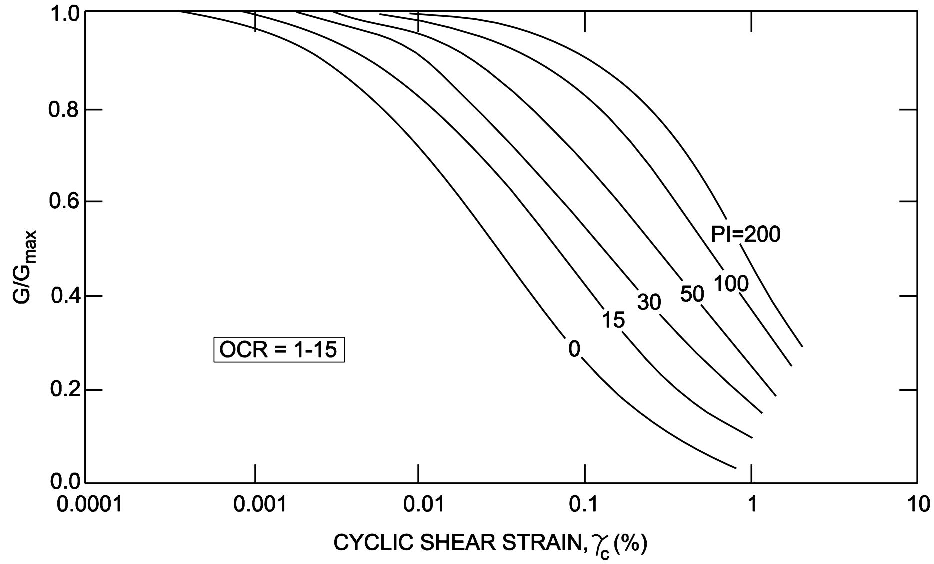


Figure 19.5-B — SHEAR MODULUS REDUCTION AND DAMPING CURVES FOR SAND (EPRI, 1993)



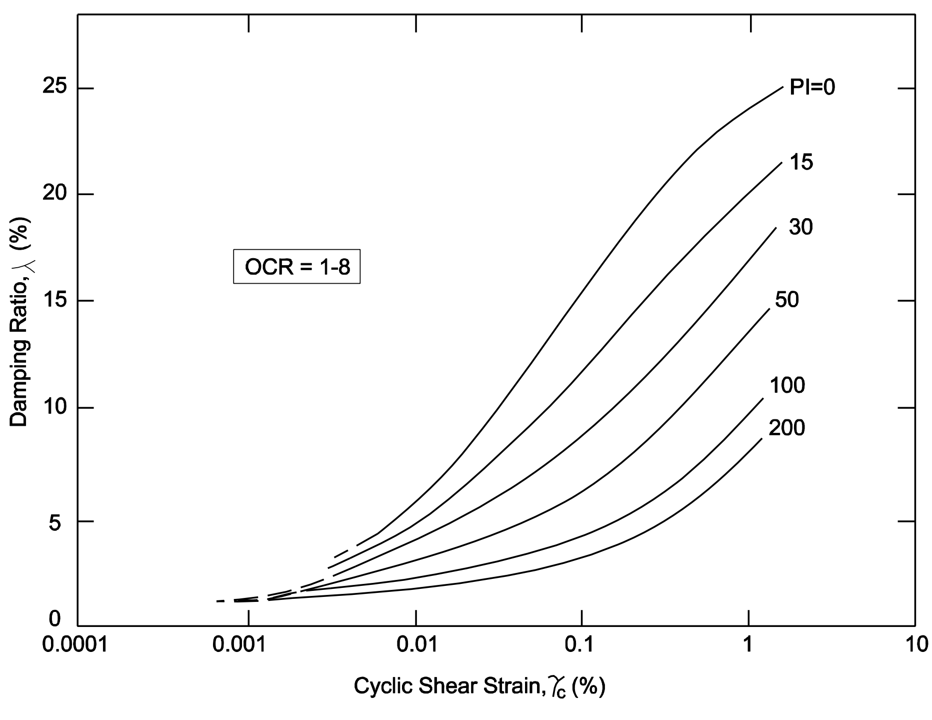


Figure 19.5-C — SHEAR MODULUS REDUCTION AND DAMPING CURVES FOR COHESIVE SOILS (Vucetic and Dobry, 1991)

19.5.3.2 Spring Constants for Pile and Drilled Shafts Foundations

For pile and drilled shaft foundations, the Geotechnical Section provides the Bridge Bureau with stiffness values for lateral loading based on L-PILE analyses. Stiffness values are determined as the slope of the load versus displacement relationship calculated using L-PILE. This information is typically presented in the form of an electronic file containing the load versus displacement plot and data tabulation.

When performing the L-PILE analyses, the appropriate pile-head fixity should be discussed with the Bridge Bureau before conducting the L-PILE analyses. If the degree of fixity is uncertain or has not been determined at the time of the analyses, it is best to provide results for both fixed- and free-head conditions. The potential for pile-group interaction effects should be included in the evaluation. The *LRFD Specifications* provide guidance on p-multipliers to use when accounting for group effects.

If layers of soil at the site are predicted to liquefy, then results should also be developed to determine the load-displacement relationship for the liquefied and nonliquefied states. Chapter 16 provides additional discussion on the design of piles and drilled shafts.

19.5.3.3 Subgrade Reaction Values

For shallow bearing foundations that are flexible relative to the supporting soil, the foundation stiffness can be calculated by a decoupled Winkler model using the unit subgrade spring coefficient. For flexible foundation systems, the unit subgrade spring coefficient is calculated by the following equation:

$$K_{sv} = \frac{1.3\,G}{B\,(1-\nu)}$$

Where:

G = the strain-adjusted shear modulus
B = the width of the footing
ν = Poisson’s ratio

For foundations that are rigid relative to the soil, the subgrade reaction value is typically increased by a factor of approximately 3.