PROPOSAL 3-6 (2009) — Part 3

SCOPE: New Part 3 Proposal for the *2009 Provisions*

PROPOSAL FOR CHANGE:

Add the following new Section in Part 3:

EVALUATION OF GEOLOGIC HAZARDS AND DETERMINATION OF SEISMIC LATERAL PRESSURES

Summarized below are procedures that are commonly used for evaluating potential site geologic hazards due to earthquakes, including slope instability, liquefaction, differential ground settlement, and surface fault rupture. Evaluation of the hazard of differential ground settlement is discussed in the section on liquefaction hazard. Geologic hazards evaluations should be carried out by qualified geotechnical professionals and documented in a report.

Screening evaluation Evaluation of a seismically-induced geologic hazard may initially consist of a screening evaluation. Although a screening evaluation typically does not require use of detailed analytical procedures, it should be based on detailed site information, including topography, geology, groundwater conditions, subsurface soil and rock stratigraphy and engineering properties, and level of ground shaking. The potential for changes in site conditions over time or as part of site development should be considered. If the findings of a screening evaluation clearly demonstrate the absence of a hazard, then more detailed evaluations, using procedures such as those described in the following sections, need not be conducted. If a screening evaluation cannot demonstrate the absence of a hazard, then the more comprehensive quantitative evaluations described below for the hazards of slope instability, liquefaction, differential settlement, and surface fault rupture should be conducted.

Reference to the following publications is suggested for guidelines on screening evaluations:

- Slope instability: California Geological Survey (1997); Blake et al. (2002); Stewart et al. (2003); U.S. Army Corps of Engineers (2005).
- Liquefaction: Martin and Lew (1999). However, as summarized later in this section under “Recent Updates to the SPT Procedure,”, the “Chinese Criteria” for identifying clayey soils susceptible to liquefaction should be abandoned in favor of more recent research.
- Differential Settlement: In the absence of liquefaction, landsliding, or surface fault rupture, differential settlement is generally not a significant hazard except at sites underlain by poorly compacted fills or loose young alluvium.
- Surface fault rupture: U.S. Army Corps of Engineers (2005).

Earthquake ground motions for geologic hazards evaluations The earthquake ground motion parameter generally required for evaluation of the hazards of slope instability, liquefaction, and differential ground settlement is peak ground acceleration, a_{max}. Peak ground acceleration can be determined using the ground motion maps in Chapter 22 and the procedures described in Sections 11.4.1

through 11.4.6. Because peak ground acceleration is equal to the zero-period acceleration of a response spectrum, a design value of a_{max} is approximated as 0.4 x SDS from Eq. 11.4-5. The value of a_{max} can be determined more accurately for a probability of exceedance of 2% in 50 years by using the USGS national ground motion map website (http://earthquake.usgs.gov/research/hazmaps/) and using the option to obtain values of peak ground acceleration for specified latitudes and longitudes. The value of a_{max} so obtained should be adjusted for site class by multiplying by the site coefficient F_a (Section 11.4.3) and then multiplying by a factor of 2/3 to convert from the maximum considered earthquake (MCE) level to the design earthquake level. The procedure using the USGS web site is not applicable, however, to locations in the U.S. near highly active faults where probabilistic ground motions have been deterministically bounded to lower values as described in Commentary Appendix A of the 2003 NEHRP Provisions.Alternatively, peak ground acceleration can be obtained from a site-specific study conforming to the requirements of Section 11.4.7 and Chapter 21.

Although the basic evaluations for geologic hazards may be conducted for design earthquake ground motions, it is recommended that evaluations also be conducted for maximum considered earthquake (MCE) ground motions (equal to 1.5 times design ground motions) for use in checking for effects that may result in structure collapse. In addition, Seismic Use Group III structures should be checked for this required post-earthquake condition. If slope instability, liquefaction, or large settlements are predicted for the MCE, the hazard should be brought to the attention of the designer and owner, and a decision made as to the risk of the hazard and the need for mitigation following the methods discussed below.

Slope instability hazard When subjected to earthquake-induced ground shaking, sloping ground can pose a hazard to structures located on or in proximity to a slope. The potential severity of the hazard depends on the steepness of the slope, soil and groundwater conditions within the slope, the strength and duration of ground shaking, and the potential consequences of slope movement. In some situations acceptable slope movement can be on the order of feet, whereas in other situations – particularly where buildings are involved – movements of more than a few inches may be unacceptable. A critical first step in the assessment of the slope instability hazard is, therefore, to establish the performance criteria for the structure. Normally this requires detailed discussions between the geotechnical engineer and the structural designer and with the project owner.

Pseudo Static Method of Analysis The stability of slopes composed of dense (nonliquefiable) or nonsaturated sandy soils or nonsensitive clayey soils can be determined using either pseudo static- or deformation-based procedures. For initial evaluations, the pseudo static analysis may be used, although the deformational analysis described in the next section is now preferred.

In the pseudostatic analysis, inertial forces generated by earthquake shaking are represented by an equivalent static horizontal force acting on the slope. The seismic coefficient for this analysis should be the site peak ground acceleration, a_{max}. The vertical component of ground acceleration is normally assumed to be zero during this representation. The factor of safety for a given seismic coefficient can be estimated by using traditional slope stability calculation methods. A factor of safety greater than one indicates that the slope is stable for the given lateral force level and further analysis is not required. A factor of safety of less than one indicates that the slope will yield and slope deformation can be expected, and a deformational analysis should be made using the techniques discussed below.

A common practice when using the pseudo static method is to reduce the peak ground acceleration by a factor to account for the transitory nature of the ground motions. The factor used in this reduction is often selected as 0.5 but lower reduction factors have also been used. For these analyses the acceptable factor of safety is often taken as 1.1 to 1.3. Implicit within this approach is that deformation of the slope is acceptable. The amount of deformation can range from a few inches to several feet (Blake et al., 2002). Movements of this magnitude are not normally acceptable for building design. For this reason the recommended approach in this guideline is to use the full peak ground acceleration in the pseudo static analysis, and then if the resulting factor of safety is less than 1.0, a deformational analysis is conducted.

When conducting a pseudo static stability analysis, two key assessments must be made by the designer during the set-up of the stability model:

- An accurate characterization of the site must be developed. This characterization needs to consider the final slope geometry, the soil types and layering within the slope, and groundwater conditions likely to exist during the seismic event. The existence of thin soil layers that could serve as slip planes is particularly important in the characterization process.
- The appropriate soil strength to use for the seismic analyses must also be selected. This determination will depend on various factors, including whether the soil is fine- or coarse-grained alluvium, the effective stress conditions, the degree of saturation of the material, and the stress history for the soil. In most situations the undrained strength of the soil is appropriate because of the short duration of seismic loading. Blake et al. (2002) provide important guidance on the use of drained or undrained soil properties, the appropriate type of testing, the use of peak versus residual strengths, and whether reductions in strength are appropriate to account for the effects of loading rate and repeated cycles of load.

For sites where soils could liquefy or where sensitive soils are known to occur, special studies will be required. If liquefaction is predicted under the design seismic event, the residual strength of the soil should normally be used in the pseudo static stability analyses. Additional discussions of residual strength are presented later in the liquefaction hazard section. If sensitive clayey soils exist, special laboratory tests may be required to establish the amount of degradation in soil strength that will occur with cyclic loading.

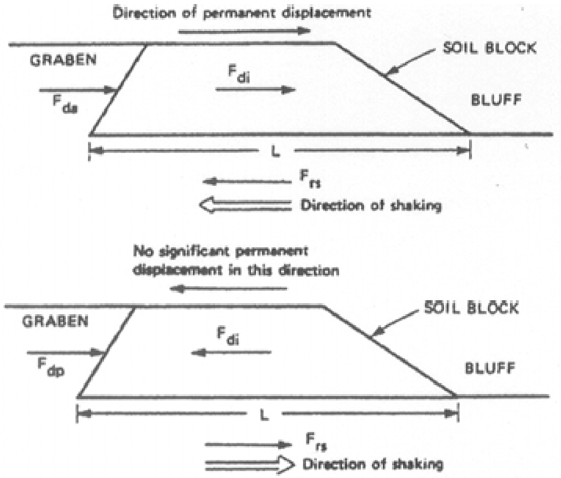
Deformational Methods of Analysis Deformational analyses resulting in estimates of slope displacement are now accepted practice. The most common analysis, termed a Newmark analysis (Newmark, 1965), uses the concept of a frictional block sliding on a sloping plane or arc. In this analysis, seismic inertial forces are calculated using a time history of horizontal acceleration as the input motion. Slope movement occurs when the driving forces (gravitational plus inertial) exceed the resisting forces. This approach estimates the cumulative displacement of the sliding mass by integrating increments of movement that occur during periods of time when the driving forces exceed the resisting forces. Displacement or yield occurs when the earthquake ground accelerations exceed the acceleration required to initiate slope movement or yield acceleration.

The yield acceleration depends primarily on the strength of the soil and the gradient and height and other geometric attributes of the slope. The same comments on the characterization of the slope and soil strength given above for the pseudo static analysis apply for the deformational analysis, though consideration can be given to the modification of strength with cycles of earthquake loading. See Figure C11.8-1 for forces and equations used in analysis and Figure C11.8-2 for a schematic illustration for a calculation of the displacement of a soil block toward a bluff.

Two methods are commonly used to estimate slope displacements by the Newmark method. The more rigorous approach involves use of earthquake records that will be representative of expected ground shaking at the site during the design seismic event. These records need to be scaled to be consistent with the design response spectra adjusted for site response effects. If more than one characteristic source mechanism contributes to the earthquake hazard, it may be necessary to select sets of records that are characteristic of each source mechanism. In this case multiple potential sources are considered because of the dependency of slope displacement on earthquake magnitude or duration; i.e., a large distant earthquake may result in lower peak ground acceleration but longer duration of shaking, which potentially could result in more cumulative deformation than a nearby earthquake of higher peak ground acceleration but short duration. Either computer programs (e.g., Jibson, 1993) or spreadsheet methods can be used to determine the cumulative displacement from the earthquake records.

An acceptable alternative method for the determination of displacements on many projects involves the use of charts that show displacements for different acceleration ratios, where the acceleration ratio is defined as the ratio of yield acceleration to peak ground acceleration. These charts have been developed by calculating the cumulative displacement following the Newmark method for large sets of earthquake records. The charts include those by Franklin and Chang (1977), Makdisi and Seed (1978), Wong and Whitman (1982), Hynes and Franklin (1984), Martin and Qiu (1994); Bray and Rathje (1998), and Blake et al. (2002). Figure 11.8-3 shows the simplified chart from Blake et al. (2002). The $D_{5\text{-}95}$ term in this figure is the significant duration of shaking – with its relationship differing depending on whether the site is within or greater than 10 km from the earthquake source. The selection between the different charts should be made on the basis of the type of slope and the degree of conservatism necessary for the project. It is important to recognize that when using one of the charts, or the Newmark method in general, a number of simplifying assumptions are made regarding the behavior of the soil during seismic loading. These assumptions limit the accuracy to which the deformations can be estimated. Generally, the method does not justify estimating deformations to less than a few inches.

Slope deformations can also be estimated by using more rigorous two-dimensional computer modeling methods. The computer code FLAC (Itasca, 1997) is perhaps the most common of the programs being used by practitioners for evaluating the response of slopes to seismic loads. This computer program allows various soil geometries, soil layering, and groundwater conditions to be modeled. Earthquake records representative of the design seismic event are used to conduct the time history analysis. Results provide an understanding of the development of deformations with time, the location of critical surfaces of deformation, and the effects of pore water pressure buildup on slope movement. As with any rigorous model, the accuracy of the deformation estimate is critically dependent on the properties and geometry of the model, as well as the earthquake record selection.



F_{da} = driving force due to active soil pressure

F_{di} = driving force due to earthquake inertia

F_{rs} = resisting force due to soil shear strength

F_{dp} = resisting force due to passive soil pressure

$$F_{di} = K_{\max} W$$

where K_{max} = maximum seismic coefficient and W = weight of soil block

$$F_{rs} = S_u L$$

where S_u = average undrained shear strength of soil and L = length of soil block

Yield seismic coefficient:

$$K_y = \frac{F_{rs} - F_{da}}{W}$$

Figure 11.8-1 Forces and equations used in analysis of translatory landsildes for calculating permanent lateral displacements from earthquake ground motions (National Research Council, 1985; from Idriss, 1985)

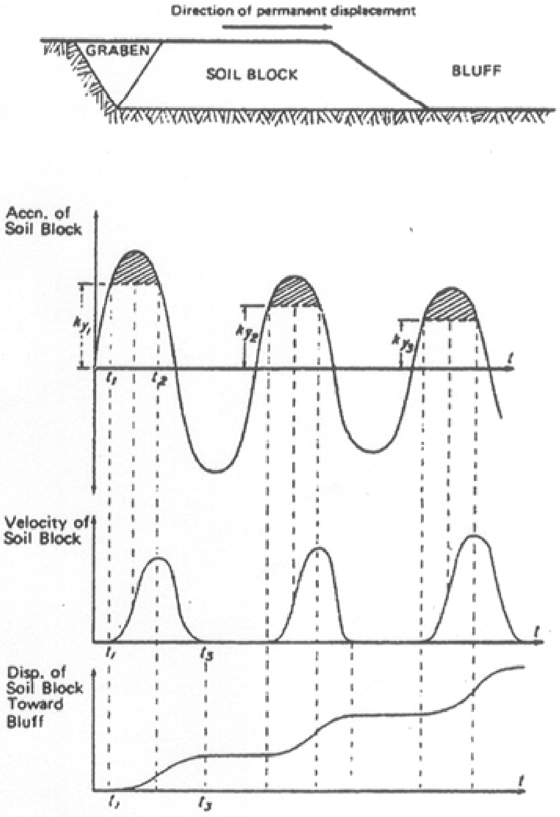


Figure 11.8-2 Schematic illustration for calculating displacement of soil block toward the bluff (National Research Council, 1985; from Idriss, 1985, adapted from Goodman and Seed, 1966)

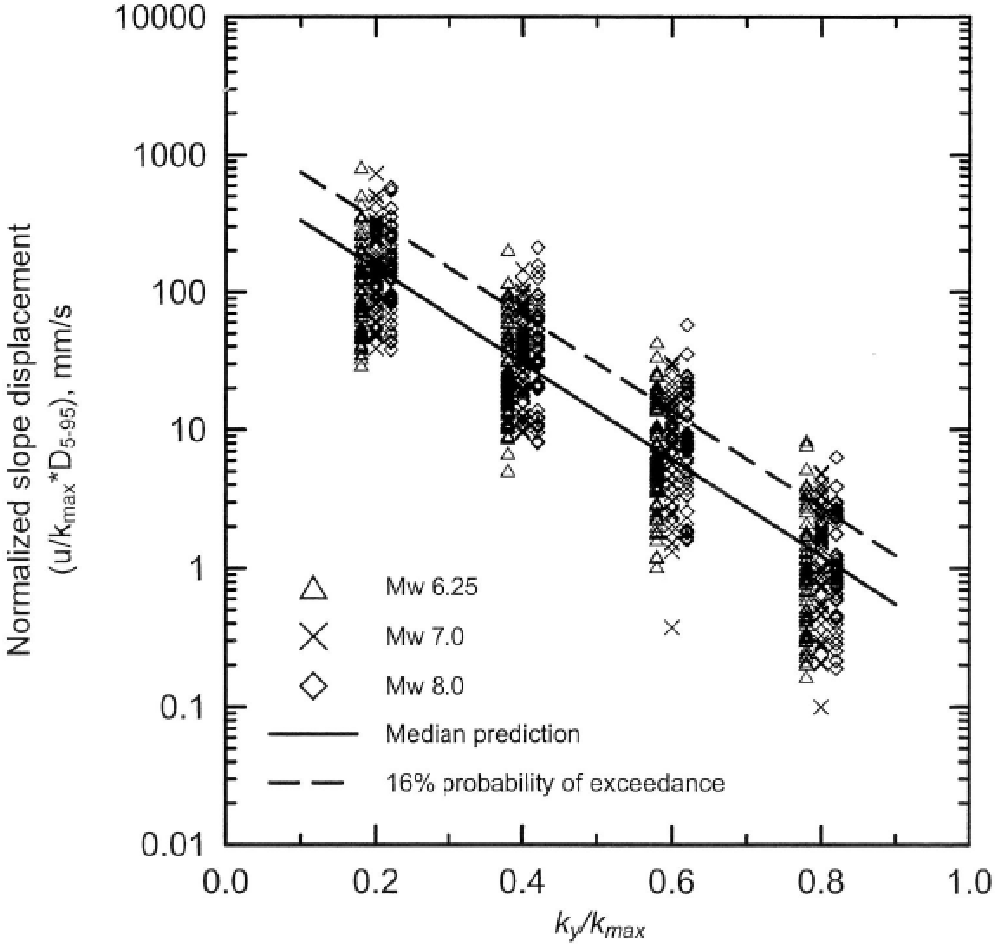


Figure C11.8-3 Normalized Sliding Displacement (Blake et al., 2002).

Mitigation of Slope Instability Hazard Three general mitigative measures might be considered for locations where slope instability is determined to represent a hazard: (1) design the structure to resist the hazard, (2) stabilize the site to reduce the hazard, or (3) choose an alternative site. Ground displacements generated by slope instability are similar in destructive character to fault displacements generating similar senses of movement: compression, shear, extension or vertical. Thus, the general comments on structural design to prevent damage given under mitigation of fault displacement apply equally to slope displacement.

Techniques to stabilize a site include reducing the driving forces by grading and drainage of slopes and increasing the resisting forces by subsurface drainage, buttresses, retaining walls, ground anchors, reaction piles or shafts, ground improvement using densification or soil mixing methods, or chemical treatment. Addition details for these mitigation methods can be found in various reports, including Blake et al. (2002).

Liquefaction hazard Liquefaction forms the second and, perhaps, the most widely known geologic hazard that must be considered at a building site. This hazards occurs when earthquake-induced ground shaking results in loss of strength within water-saturated, loose granular soils. The consequence of this strength loss relative to a building can be reduction in bearing strength, differential settlement, and horizontal ground displacement from lateral spreading or flow failures within the ground. In this section, the hazard of differential settlement, whether due to liquefaction of water-saturated soils or compaction of non-saturated soils, is addressed.

Design to prevent damage due to liquefaction consists of three parts: evaluation of liquefaction hazard, evaluation of potential ground displacement, and where necessary mitigation of the hazard by either designing to resist ground displacement or strength loss, by reducing the potential for liquefaction, or by choosing an alternative site with less hazard. Before providing guidance in these areas, the following subsections provide a summary of the methods that are used to evaluate the liquefaction hazard and a discussion of recent updates to the most commonly used method of assessing a liquefaction hazard – the empirical Standard Penetration Test (SPT) procedure.

Methods of liquefaction hazard evaluation Liquefaction hazard at a site is commonly expressed in terms of a factor of safety. This factor is defined as the ratio between the available liquefaction resistance, expressed in terms of the cyclic stresses required to cause liquefaction, and the cyclic stresses generated by the design earthquake. Both of these stress parameters are commonly normalized with respect to the effective overburden stress at the depth in question to define a cyclic resistance ratio (CRR) and a cyclic stress ratio induced by the earthquake (CSR).

Three different methods have been proposed and are used to various extents for evaluating liquefaction potential: empirical methods; analytical methods; and physical modeling.

1. Empirical methods are the most widely used methods in practice. These procedures rely on correlations between observed cases of liquefaction and measurements made in the field with conventional exploration methods. Seed and Idriss (1971) first published the widely used “simplified procedure” utilizing the Standard Penetration Test (SPT). Since then, field test methods in addition to the SPT have been utilized in similar simplified procedures. These methods include cone penetrometer tests (CPTs), Becker hammer tests (BHTs), and shear wave velocity tests (SVTs). These empirical procedures are summarized in the proceedings from a workshop (referred to as the Liquefaction Workshop) held in 1996 (NCEER, 1997; Youd et al., 2001). Martin and Lew (1999) provide additional details on the implementation of these procedures relative to engineering practice.

2. Analytical methods are used less frequently to evaluate liquefaction potential – though they may be required for special projects or where soil conditions are not amenable to the empirical method. Analytical methods will also likely continue to gain prominence with time as numerical methods and soil models improve and are increasingly validated. Originally (circa 1970s) the analytical method involved determination of the induced shearing stresses with a program such as SHAKE and comparing these stresses to results of cyclic triaxial or cyclic simple shear tests. Now the analytical method usually refers to a computer code that incorporates a soil model that calculates the buildup in pore water pressure. These more rigorous numerical methods include one-dimensional, nonlinear effective stress codes such as DESRA, SUMDES, and TESS and two dimensional, nonlinear effective stress codes such as FLAC, TARA, and DYNAFLOW. This new generation of analytical methods has soil models that are fit to or derived from laboratory data or from liquefaction curves developed from SPT information. These methods are limited by the ability to represent the soil model from either the laboratory or field measurements and by the complexity of the wave propagation mechanisms, including the ability to select appropriate earthquake records to use in the analyses.

3. Physical modeling originally involved the use of centrifuges or relatively small-scale shaking tables to simulate seismic loading under well-defined boundary conditions. Physical model testing also now includes large laminar boxes mounted on very large shake tables (e.g., Kagawa et al., 2004) and full-scale field blast loading tests (e.g., Ashford et al. 2004; Ashford et al., 2006). This type of modeling is one of the main focus areas of the 2004-2014 Network for Earthquake Engineering Simulation (NEES) supported by the National Science Foundation. Soil used in the small-scale and laminar box

models is reconstituted to represent different density and geometrical conditions. Because of difficulties in precisely modeling in situ conditions at liquefiable sites, small-scale and laminar box models have seldom been used in design studies for specific sites. However, physical models are valuable for analyzing and understanding generalized soil behavior and for evaluating the validity of constitutive models under well-defined boundary conditions. Blast loading tests have been conducted to capture the in situ characteristics of the soil for research and design purposes (e.g., Treasure Island, California; Cooper River Bridge in South Carolina, and in Japan). However, the cost and safety issues of blasting methods limits its use to only special design or research projects.

Most liquefaction hazards assessments for buildings will involve use of the SPT empirical method – partly because of the wide acceptance of this approach and also because this approach can be easily integrated into the geotechnical investigations normally performed during building design. The SPT method is based on recommendations developed at the Liquefaction Workshop as described in NCEER (1997) and Youd et al. (2001) or on one of the updates to this methodology as discussed below.

Although the SPT empirical method is the most commonly used of the empirical approaches, it is important to recognize that for certain site conditions alternate empirical methods, such as the CPT, BHT, and SVT, are acceptable and even preferred. This is particularly the case with the CPT method. Advantages of the CPT method compared to the SPT method are the ability of this method to detect thin liquefiable layers that could serve as sliding surfaces and the greater standardization of the method – though this approach has the disadvantage that soil samples are not obtained. Where possible a combination of procedures is recommended to take advantage of the best features of each.

Recent Updates to the SPT Procedure The methods presented in the Liquefaction Workshop and summarized in the following section represent a consensus-based approach for the onset or triggering of liquefaction; however, the consensus workshop occurred nearly 10 years ago. A number of significant modifications to the methods presented in the Liquefaction Workshop have been recommended over the past 10 years. These modifications include changes to the stress reduction coefficient (r_d), modifications to the magnitude scaling factor (MSF) [also referred to as the duration weighting factor (DWF)], revisions to the overburden correction term (K_σ) and the fines correction (FC), refinements to the overburden correction (C_N), and finally changes to the relationship between the curve relating cyclic stress ratio causing liquefaction and the normalized N value; i.e., the fundamental liquefaction strength curve such as shown in Figure C11.8-4. These modifications are discussed in detail in papers by Idriss and Boulanger (2004, 2006) and Cetin et al. (2004), and each set of recommended revisions resulted after detailed study of the database of case histories upon which the original blowcount relationships were developed.

Another important observation that has been made over the past 10 years involves the fines criteria used to judge whether or not a soil is liquefiable. Originally, the “Chinese Criteria” was accepted as the method to determine whether or not a cohesionless soil was liquefiable. However, recent work summarized in Boulanger and Idriss (2004) and Bray et al. (2004) indicate that the Chinese Criteria will be unconservative in some situations, and alternate methods of assessing whether a soil with cohesive fines will be susceptible to liquefaction need to be considered.

Methods are also now available for treating the probability of liquefaction, given a certain design ground motion and SPT blowcount. Cetin et al. (2004) presents the latest comprehensive treatment of liquefaction probability. These researchers suggest that following the deterministic approach for estimating liquefaction potential results in approximately 15 percent probability of liquefaction. The approach presented by Cetin et al. (2004) allows limiting SPT blowcounts to be determined for alternate probabilities, or the probability associated with a given set of blowcounts and ground motions (in terms of CSR) to be defined. This probabilistic framework forms an important basis for performance-based design methods that are currently being developed.

Despite these many important modifications to the general approach for assessing liquefaction hazards over the past 10 years, the profession has not developed a consensus on which of the modifications should be used as a baseline for evaluating liquefaction hazard – similar to the recommendations in NCEER (1997) and Youd et al. (2001) based on the Liquefaction Workshop. Procedures suggested by Idriss and Boulanger (2004), as well as those developed by Cetin et al. (2004), present important changes to the liquefaction hazard analysis. However, until a consensus is reached or an adequate period of vetting occurs, it is difficult to recommend between the two. .

In using the more recent methods, it is important that these methods be used consistently. In other words the Idriss and Boulanger method should be used with the various improvements recommended by Idriss and Boulanger, including the revised liquefaction strength plot. Likewise, if the Cetin et al. method is going to be used, it should be used in its entirety. It is also important to use these new methods with some caution, particularly at the limits of the procedure (e.g., at higher blowcounts, deeper depths, and higher CSR values). If the more recent methods are used, the prudent approach will be to check the liquefaction hazard with an alternate method, such as the procedure discussed below. Differences between the hazard estimates resulting from different methods could reflect a real uncertainty in the prediction, and this uncertainty would need to be considered when judging the hazard at a site.

Empirical SPT method for evaluating liquefaction hazard Procedures for evaluating the liquefaction hazard using the Liquefaction Workshop methodology are summarized below. As discussed above, the recent changes in the methodology proposed by Idriss and Boulanger (2004) and Cetin et al. (2004) offer an updated alternative to this approach, but need to be used with some caution in the absence of a consensus from the profession that these newer methods are acceptable for generalized use.

1. The first step in the liquefaction hazard evaluation using the empirical SPT approach is usually to define the normalized cyclic shear stress ratio (CSR) from the peak horizontal ground acceleration expected at the site. This evaluation is made using the following equation:

   $$CSR = 0.65\left(a_{\max}/g\right)\left(\sigma_0/\sigma_0'\right)r_d \qquad \text{(C11.8-1)}$$

   Where:

   (a_{max}/g) = peak horizontal acceleration at ground surface expressed as a decimal fraction of gravity, σ_0 = the vertical total stress in the soil at the depth in question, σ_0' = the vertical effective stress at the same depth, and r_d = deformation-related stress reduction factor. The peak ground acceleration, a_{max}, commonly used in liquefaction analysis is that which would occur at the site in the absence of liquefaction. Thus, the a_{max} used in Eq. C11.8-1 is the estimated rock acceleration corrected for soil site response but with neglect of excess pore-water pressures that might develop.

   The stress reduction factor, r_d, used in Eq. C11.8-1 was originally determined using a plot developed by Seed and Idriss (1971) showing the reduction factor versus depth. The consensus from the Liquefaction Workshop was to represent r_d by the following equations:

   $$r_d = 1.0 - 0.00765z \quad \text{for } z \le 9.15\text{ m} \qquad \text{(C11.8-2a)}$$

   $$r_d = 1.174 - 0.267z \quad \text{for } 9.15\ m < z \le 23\text{ m} \qquad \text{(C11.8-2b)}$$

   It should be noted that because nearly all the field data used to develop the simplified procedure are for depths less than 15 m, there is greater uncertainty in the use of the SPT empirical approach at greater depths. Common practice is to use the SPT method to depths of 25 m. In some locations deep deposits of low blowcount or low CPT end resistance values occur, such as in the Puget Sound area and along the Columbia River. It is still prudent to consider these low blowcount materials as susceptible to liquefaction even if they are located at depths greater than 25 m. For these sites it may be appropriate to use strain-based procedures (Dobry et al., 1982) or one-dimensional, effective stress modeling methods such as are summarized in Commentary to Chapter 21.

2. The second step in the liquefaction hazard evaluation using the empirical approach usually involves

determination of the normalized cyclic resistance ratio (CRR). The most commonly used empirical relationship compares CRR with corrected Standard Penetration Test (SPT) resistance, $(N_1)_{60}$, from sites where liquefaction did or did not develop during past earthquakes. Figure C11.8-4 shows this relationship for Magnitude 7.5 earthquakes, with an adjustment at low values of CRR recommended by the Liquefaction Workshop. Similar relationships have been developed for determining CRR from CPT soundings, from BHT blowcounts, and from shear wave velocity data, as discussed by Youd et al. (2001) and as presented in detail in NCEER (1997).

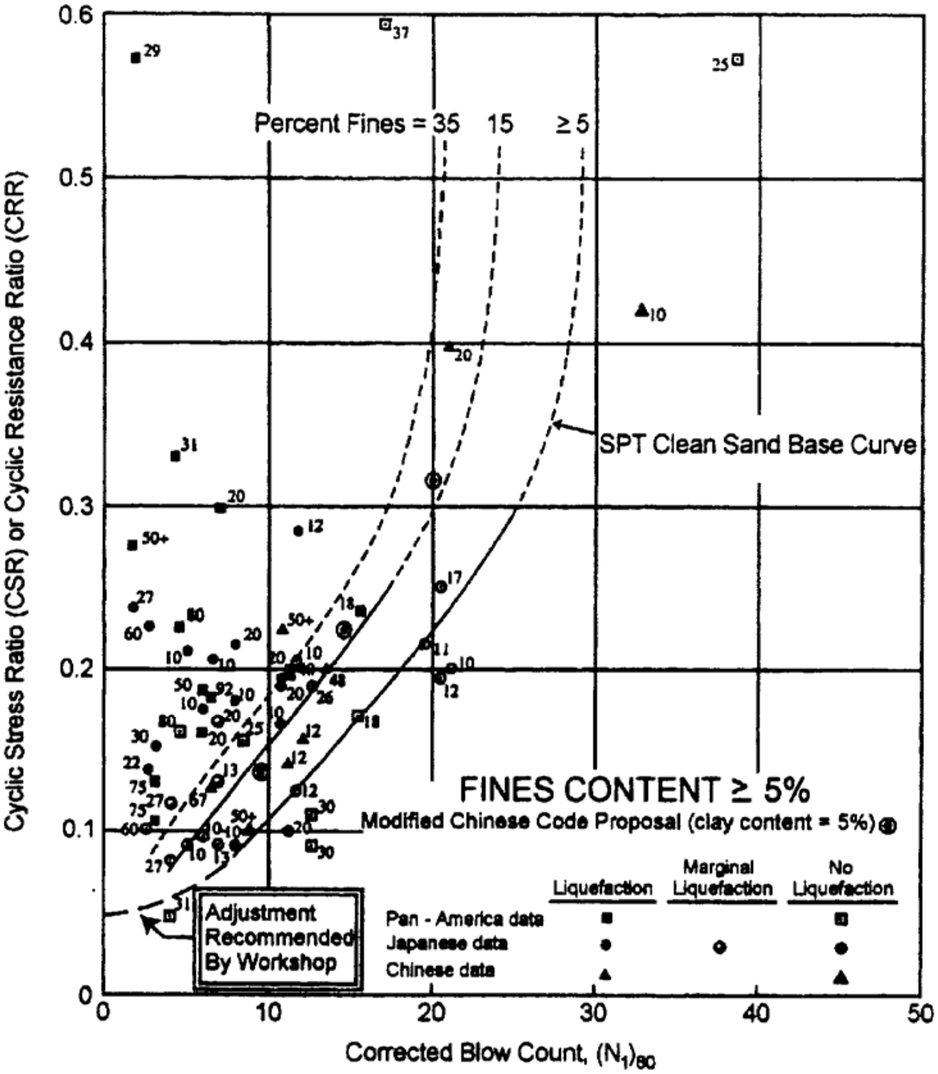


Figure C11.8-4. SPT clean sand base curve for magnitude 7.5 earthquakes with data from liquefaction case histories. (Modified from Seed et al., 1985). (NCEER, 1997; Youd et al., 2001).

In Figure C11.8-4, CRRs calculated for various sites are plotted against $(N_1)_{60}$, where $(N_1)_{60}$ is the SPT blowcount normalized for an overburden stress of 100 kPa and for an energy ratio of 60 percent. Solid symbols represent sites where liquefaction occurred and open symbols represent sites where surface evidence of liquefaction was not found. Curves were drawn through the data to separate regions where liquefaction did and did not develop. As shown, curves are given for soils with fines contents (FC) ranging from less than 5 to 35 percent.

The $(N_1)_{60}$ in Figure C11.8-4 is adjusted for various factors before its use, as recommended by the Liquefaction Workshop and discussed by Youd et al. (2001). These include an adjustment for fines, such that only the clean sand curve in Figure C11.8-4 is used, as well as adjustments for a number of other testing related parameters. These adjustments are not repeated in this guideline as they are all in conventional use by the profession and can readily be found in references by Martin and Lew (1999) and by Youd et al. (2001).

It is very important that the engineer consider these correction factors when conducting the liquefaction analyses. Failure to consider these corrections can result in inaccurate liquefaction estimates – leading to either excessive cost to mitigate the liquefaction concern or excessive risk of poor performance during a design event – potentially resulting in unacceptable damage.

Special mention needs to be made of the energy calibration term, C_E. This correction has a very significant effect on the $(N_1)_{60}$ used to compute CRR. The value of this correction factor can vary greatly depending on the SPT hammer system used in the field and on site conditions. The automatic hammer used to conduct SPTs in modern-day explorations avoids much of the uncertainty in energy; however even it should be periodically calibrated. These calibration measurements are relatively inexpensive and represent a small increase in overall field exploration costs. Many drilling contractors in areas that are seismically active provide calibrated equipment as part of their routine service.

Before computing the factor of safety from liquefaction, the CRR result obtained from Figure C11.8-4 (using the corrected SPT blow count identified in the equation for $(N_1)_{60}$) must be corrected for earthquake magnitude M if the magnitude differs from 7.5. The magnitude correction factor is shown in Figure C11.8-5. This plot was developed during the Liquefaction Workshop on the basis of input from experts attending the workshop. The range shown in Figure C11.8-5 is used because of uncertainties. The user should select a value consistent with the project risk. For M greater than 7.5 the factors recommended by Idriss (second from highest) should be used.

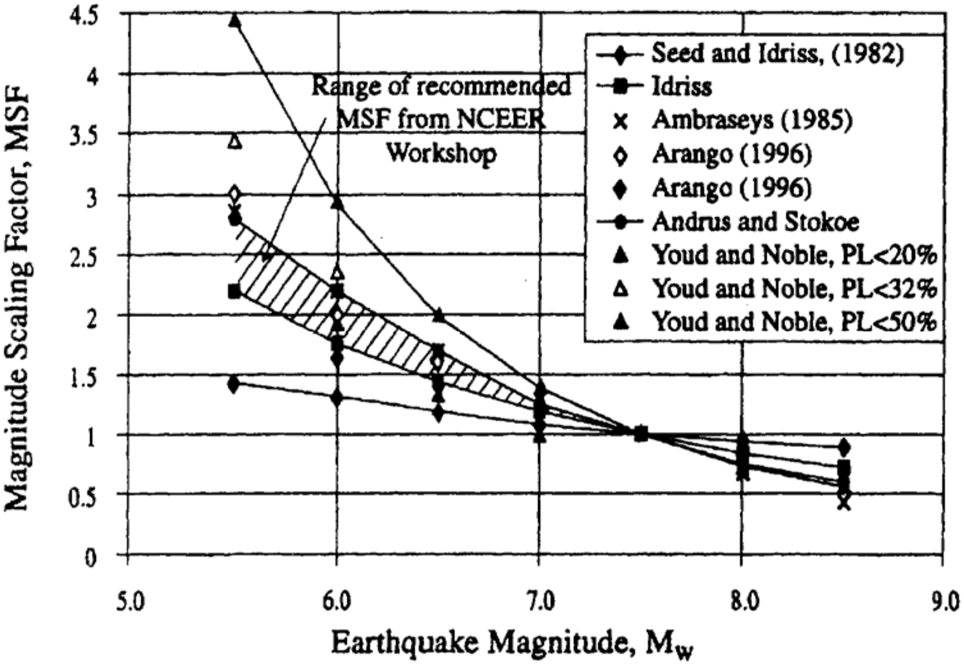


Figure C11.8-5. Magnitude scaling factors derived by various investigators. (NCEER, 1997; Youd et al., 2001).

The magnitude, M, needed to determine a magnitude scaling factor from Figure C11.8-5 should correspond to the Maximum Considered Earthquake (MCE). Where the general procedure for ground motion estimation is used (Sections 11.4.1 - 11.4-6) and the MCE is determined probabilistically, the magnitude used in these evaluations can be obtained as the dominant magnitude(s) determined from deaggregation information available by latitude and longitude from the USGS website (http://earthquake.usgs.gov/research/hazmaps/). Where the general procedure is used and the MCE is bounded deterministically near known active fault sources, the magnitude of the MCE should be the characteristic maximum magnitude assigned to the fault in the construction of the MCE ground motion maps. Where the site-specific procedure for ground motion estimation is used (Sections 11.4.7 and Chapter 21), the magnitude of the MCE should be similarly determined from the site-specific analysis. In all cases, it should be remembered that the likelihood of liquefaction at the site (as defined later by the factor of safety F_L in Eq. C11.8-3) is determined jointly by a_{max} and M and not by a_{max} alone. Because of the longer duration of strong ground-shaking, large distant earthquakes may in some cases generate liquefaction at a site while smaller nearby earthquakes may not generate liquefaction even though a_{max} of the nearer events is larger than that from the more distant events.

3. The final step in the liquefaction hazard evaluation using the empirical SPT approach involves the computation of the factor of safety (F_L) against liquefaction using the equation:

$$F_L = \text{CRR/CSR} \qquad \text{(C11.8-3)}$$

If F_L is greater than one, then liquefaction should not develop. If at any depth in the sediment profile, F_L is equal to or less than one, then there is a liquefaction hazard. Although the curves shown in Figure C11.8-4 envelop the plotted data, it is possible that liquefaction may have occurred beyond the enveloped data and was not detected at ground surface. For this reason a factor of safety of 1.2 to 1.5 is usually appropriate for building sites – with the actual factor selected on the basis of the importance of the structure and the potential for ground displacement at the site.

Additional guidance on the selection of the appropriate factor of safety is provided by Martin and Lew (1999). They suggest that the following factors be considered when selecting the factor of safety:

- The type of structure and its vulnerability to damage.
- Levels of risk accepted by the owner or governmental regulations with questions related to design for life safety, limited structural damage, or essentially no damage.
- Damage potential associated with the particular liquefaction hazard. Flow failures or major lateral spreads pose more damage potential than differential settlement. Hence factors of safety could be adjusted accordingly.
- Damage potential associated with design earthquake magnitude. A magnitude 7.5 event is potentially more damaging than a 6.5 event.
- Damage potential associated with SPT values; low blow counts have a greater cyclic strain potential than higher blowcounts.
- Uncertainty in SPT- or CPT- derived liquefaction strengths used for evaluations. Note that a change in silt content from 5 to 15 percent could change a factor of safety from, say, 1.0 to 1.25.
- For high levels of design ground motion, factors of safety may be indeterminate. For example, if $(N_1)_{60}) = 20$, $M = 7.5$, and fines content = 35 percent, liquefaction strengths cannot be accurately defined due to the vertical asymptote on the empirical strength curve.

Martin and Lew (1999) indicate that the final choice of an appropriate factor of safety must reflect the particular conditions associated with the specific site and the vulnerability of site-related structures. Table C11.8-1 summarizes factors of safety suggested by Martin and Lew.

Table C11.8-1. Factors of safety for liquefaction hazard assessment (from Martin and Lew, 1999).

Consequences of Liquefaction	$(N_1)_{60cs}$	Factor of Safety
Settlement	≤ 15	1.1
	≥ 30	1.0
Surface Manifestations	≤ 15	1.2
	≥ 30	1.0
Lateral Spread	≤ 15	1.3
	≥ 30	1.0

As a final comment on the assessment of liquefaction hazards, it is important to note that soils composed of sands, silts, and gravels are most susceptible to liquefaction while clay soils generally are not susceptible to this phenomenon. The curves in Figure C11.8-4 are valid for soils composed primarily of sand. The curves should be used with caution for soils with substantial amounts of gravel. Verified corrections for gravel content have not been developed; a geotechnical engineer, experienced in liquefaction hazard evaluation, should be consulted when gravelly soils are encountered.

Evaluation of potential for ground displacements Liquefaction by itself may or may not be of engineering significance. Only when liquefaction is accompanied by loss of ground support and/or ground deformation does this phenomenon become important to structural design. Surface manifestations, loss of bearing strength, ground settlement, flow failure and lateral spread are ground failure mechanisms that have caused structural damage during past earthquakes. These types of ground failure are described in Martin and Lew (1999), U.S. Army Corps of Engineers (2005) and National Research Council (1985) and are discussed below. The type of failure and amount of ground displacement are a function of several parameters including the looseness of the liquefied soil layer, the thickness and extent of the liquefied layer, the thickness and permeability of unliquefied material overlying the liquefied layer, the ground slope, and the nearness of a free face.

*Surface Manifestations***.** Surface manifestations refer to sand boils and ground fissures on level ground sites. For structures supported on shallow foundations, the effects of surface manifestations on the structure could be tilting or cracking. Criteria are given by Ishihara (1985) for evaluating the influence of thickness of layers on surface manifestation of liquefaction effects for level sites. These criteria may be used for noncritical or nonessential structures on level sites not subject to lateral spreads (see later in this section). Additional analysis should be performed for critical or essential structures.

Loss of bearing strength. Loss of bearing strength can occur if the foundation is located within or above the liquefiable layer. The consequence of bearing failure could be settlement or tilting of the structure. Usually, loss of bearing strength is not likely for light structures with shallow footings founded on stable, nonliquefiable materials overlying deeply buried liquefiable layers, particularly if the liquefiable layers are relatively thin. Simple guidance for how deep or how thin the layers must be has not yet been developed. Martin and Lew (1999) provide some preliminary guidance based on the Ishihara (1985) method. Final evaluation of the potential for loss of bearing strength should be made by a geotechnical engineer experienced in liquefaction hazard assessment

Ground settlement. For saturated or dry granular soils in a loose condition, the amount of ground settlement can approach 3 to 4 percent of the thickness of the loose soil layer in some cases. This amount of settlement could cause tilting or cracking of a building, and therefore, it is usually important to evaluate the potential for ground settlement during earthquakes.

Tokimatsu and Seed (1987) published an empirical procedure for estimating ground settlement. It is beyond the scope of this commentary to outline that procedure which, although explicit, has several rather complex steps. The Tokimatsu and Seed procedure can be applied whether liquefaction does or does not occur. For dry cohesionless soils, the settlement estimate from Tokimatsu and Seed should be multiplied by a factor of 2 to account for multi-directional shaking effects as discussed by Martin and Lew (1999). An alternate approach is that proposed by Ishihara and Yoshimine (1992). It also can be used for saturated and unsaturated soils.

*Flow failures***.** Flow failures or flow slides are the most catastrophic form of ground failure that may be triggered when liquefaction occurs. They may displace large masses of soils tens of meters. Flow slides occur when the average static shear stresses on potential failure surfaces are less than the average shear strengths of liquefied soil on these surfaces. Standard limit equilibrium static slope stability analyses may be used to assess flow failure potential with the residual strength of liquefied soil used as the strength parameter in the analyses.

The determination of residual strengths is very inexact, and consensus as to the most appropriate approach has not been reached to date. Two relationships for residual strength of liquefied soil that are often used in practice are those of Seed and Harder (1990) and Stark and Mesri (1992). A more complete discussion and references on this topic may be found in Martin and Lew (1999).

Lateral spreads. Lateral spreads are ground-failure phenomena that can occur on gently sloping ground underlain by liquefied soil. They may result in lateral movements in the range of a few centimeters to several meters. Earthquake ground-shaking affects the stability of gently sloping ground containing liquefiable materials by seismic inertial forces combined with static gravity forces within the slope and by shaking-induced strength reductions in the liquefiable materials. Temporary instability due to seismic inertial forces are manifested by lateral "downslope" movement. For the duration of ground shaking associated with moderate-to large-magnitude earthquakes, there could be many such occurrences of temporary instability during earthquake shaking, producing an accumulation of "downslope" movement.

Various analytical and empirical techniques have been developed to date to estimate lateral spread ground displacement; however, no single technique has been widely accepted or verified for engineering design. Three approaches are used depending on the requirements of the project: empirical procedures; simplified analytical methods; and more rigorous computer modeling.

1. Empirical procedures use correlations between past ground displacement and site conditions under which those displacements occurred. Youd et al. (2002) present an empirical method that provides an estimate of lateral spread displacements as a function of earthquake magnitude, distance, topographic conditions, and soil deposit characteristics. As shown in Figure C11.8-6, the displacements estimated by the Youd et al. (2002) method are generally within a factor of two of the observed displacements. Bardet et al. (2002) present an empirical method having a formulation similar to that of Youd et al. (2002) but using fewer parameters to describe the soil deposit. The Bardet et al. (2002) model was developed to assess lateral spread displacements at a regional scale rather than for site-specific applications. Various other empirical methods are also available, including an alternate SPT method by Rausch and Martin (2000) and both a SPT and CPT-based method by Zhang et al. (2004). These methods can result in large differences in predicted displacement and therefore, it is usually best to use several methods when estimating displacement. Because of the uncertainty in results, these methods are best used for preliminary or comparative evaluations.

2 Simplified analytical techniques generally apply some form of Newmark's analysis of a rigid body sliding on an infinite or circular failure surface with ultimate shear resistance estimated from the residual strength of the deforming soil. Additional discussion of the simplified Newmark method is

provided in the discussion of slope instability hazard.

3. More rigorous computer modeling typically involves use of nonlinear finite element or finite difference methods to predict deformations, such as with the computer code FLAC. Both the simplified Newmark method and the rigorous computer codes require considerable experience to obtain meaningful results. For example, the soil model within the nonlinear computer codes is often calibrated for only specific conditions. If the site is not characterized by these conditions, errors in estimating the displacement by a factor of two or more can easily occur.

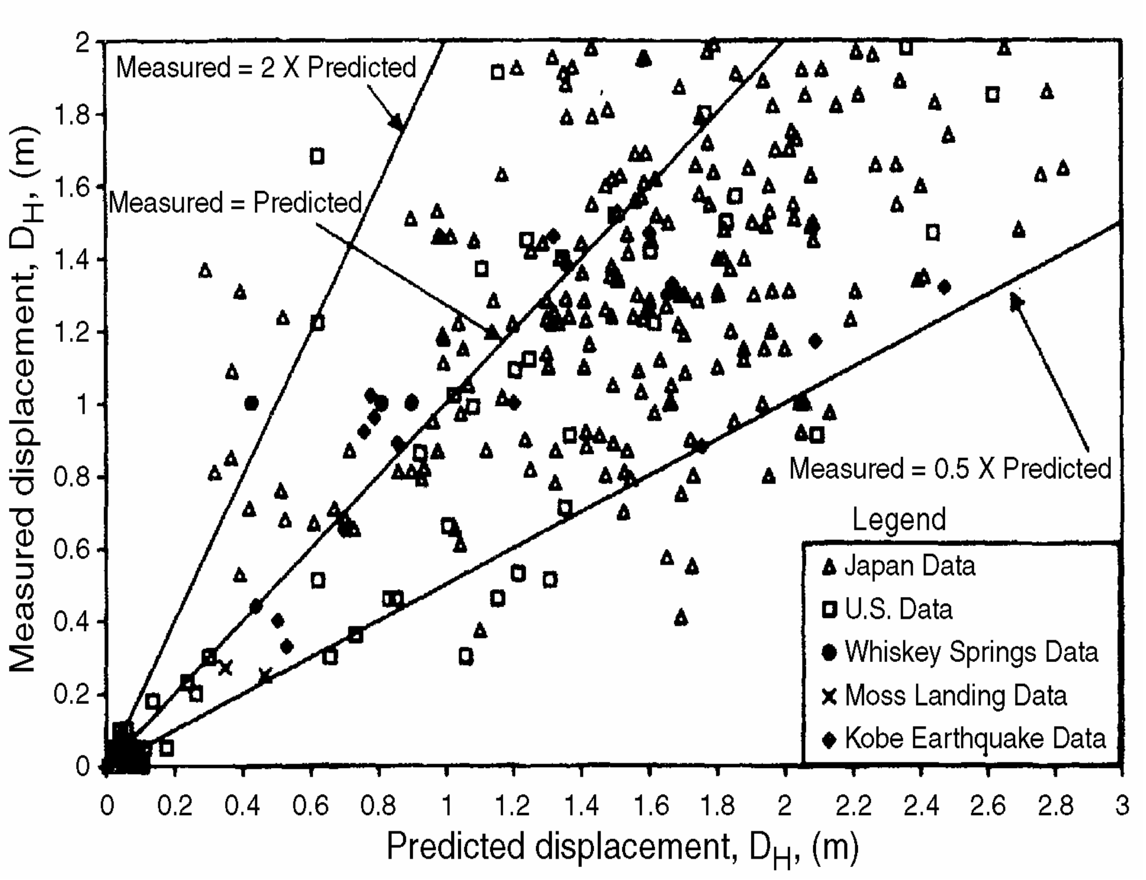


Figure C11.8-6. Measured versus predicted displacements for displacements up to 2 meters. (Youd et al., 2002).

Mitigation of liquefaction hazard Three general measures might be considered for mitigation of liquefaction hazards: (1) design the structure to resist the hazard, (2) stabilize the site to reduce the hazard, or (3) choose an alternative site. Structural measures that are used to reduce the hazard include deep foundations, mat foundations, or footings interconnected with ties. Deep foundations have performed well at level sites of liquefaction where effects were limited to ground settlement and ground oscillation with no more than a few inches of lateral displacement. Deep foundations, such as piles, may receive reduced soil support through the liquefied layer and may be subjected to transient lateral displacements across the layer. Well reinforced mat foundations also have performed well at localities where ground displacements were less than 1 foot, although re-leveling of the structure has been required in some instances (Youd, 1989). Strong ties between footings also should provide increased resistance to damage where differential ground displacements are less than a foot.

Evaluations of structural performance following two Japanese earthquakes, 1993 Hokkaido Nansei-Oki and 1995 (Kobe) Hyogo-Ken Nanbu, indicate that small structures on shallow foundations performed well in liquefaction areas. Sand boil eruptions and open ground fissures in these areas indicate minor effects of liquefaction, including ground oscillation and up to several tenths of a meter of lateral spread displacement. Many small structures (mostly houses, shops, schools, etc.) were structurally undamaged although a few tilted slightly. Foundations for these structures consist of reinforced concrete perimeter wall footings with reinforced concrete interior wall footings tied into the perimeter walls at intersections. These foundations acted as diaphragms causing the soil to yield beneath the foundation which prevented fracture of foundations and propagation of differential displacements into the superstructure.

Similarly, well reinforced foundations that would not fail could be used in U.S. practice as a mitigative measure to reduce structural damage in areas subject to liquefaction but with limited potential for lateral (< 0.3 m) or vertical (< 0.05 m) ground displacements. Such strengthening also would serve as an effective mitigation measure against damage from other sources of limited ground displacement including fault zones, landslides, and cut fill boundaries. Where slab-on-grade or basement slabs are used as foundation elements, these slabs should be reinforced and tied to the foundation walls to give the structure adequate strength to resist ground displacement. Although strengthening of foundations, as noted above, would largely mitigate damage to the structure, utility connections may be adversely affected unless special flexibility is built into these nonstructural components.

Another possible consequence of liquefaction to structures is increased lateral pressures against basement walls as discussed in Section 11.8-3. A common procedure used in design for such increased pressures is to assume that the liquefied material acts as a dense fluid having a unit weight of the liquefied soil. The wall then is designed assuming that hydrostatic pressure for the dense fluid acts along the total subsurface height of the wall. The procedure applies equivalent horizontal earth pressures that are greater than typical at-rest earth pressures but less than passive earth pressures. As a final consideration, to prevent buoyant rise as a consequence of liquefaction, the total weight of the structure should be greater than the volume of the basement or other cavity times the unit weight of liquefied soil. (Note that structures with insufficient weight to counterbalance buoyant effects could differentially rise during an earthquake.)

At sites where expected ground displacements are unacceptably large, ground modification to lessen the liquefaction or ground failure hazard or selection of an alternative site may be required. Techniques for ground stabilization to prevent liquefaction of potentially unstable soils include removal and replacement of soil; compaction of soil in place using vibrations, heavy tamping, compaction piles, or compaction grouting; buttressing; chemical stabilization with grout; and installation of drains. Further discussion of mitigation methods is given by the National Research Council (1985) and Martin and Lew (1999).

Surface fault rupture hazard. Fault ruptures during past earthquakes have led to large surface displacements that are potentially destructive to engineered construction. Displacements, which range from a fraction of an inch to tens of feet, may occur along traces of active faults. The sense of displacement ranges from horizontal strike-slip to vertical dip-slip to many combinations of these components. The following commentary summarizes procedures to follow or consider when assessing the hazard of surface fault rupture. Sources of detailed information for evaluating the hazard of surface fault rupture include Slemmons and dePolo (1986), the Utah Section of the Association of Engineering Geologists (1987), Swan et al. (1991), Hart and Bryant (1997), Hanson et al. (1999), and California Geological Survey (2002). Other beneficial references are given in the bibliographies of these publications.

Assessment of surface faulting hazard The evaluation of surface fault rupture hazard at a given site is based extensively on the concepts of recency and recurrence of faulting along existing faults. The magnitude, sense, and frequency of fault rupture vary for different faults or even along different segments of the same fault. Even so, future faulting generally is expected to recur along pre-existing active faults. The development of a new fault or reactivation of a long inactive fault is relatively uncommon and generally need not be a concern. For most engineering applications related to foundation design, a sufficient definition of an active fault is given in CDMG Special Publication 42 (Hart and Bryant, 1997): “An active fault has had displacement in Holocene time (last 11,000 years).”

As a practical matter, fault investigations should be conducted by qualified geologists and directed at the problem of locating faults and evaluating recency of activity, fault length, the amount and character of past displacements, and the expected amount and potential of future displacement. Identification and characterization studies should incorporate evaluation of regional fault patterns as well as detailed study of fault features at and in the near vicinity (within a few hundred yards to a mile) of the site. Detailed studies often include trenching to accurately locate, document, and date fault features.

Suggested approach for assessing surface faulting hazard. The following approach should be used, or at least considered, in fault hazard assessment. Some of the investigative methods outlined below should be carried out beyond the site being investigated. However, it is not expected that all of the following methods would be used in a single investigation:

1. A review should be made of the published and unpublished geologic literature from the region along with records concerning geologic units, faults, ground-water barriers, etc.
2. A stereoscopic study of aerial photographs and other remotely sensed images should be made to detect fault-related topography/geomorphic features, vegetation and soil contrasts, and other lineaments of possible fault origin. The study of predevelopment aerial photographs is often essential to the detection of fault features. Recently, the use of LiDAR (LIght Detection And Ranging) has been found to provide excellent identification of fault traces in areas where tree growth and vegetation would normally obscure evidence of faulting from the air.
3. A field reconnaissance study generally is required and should include observation and mapping of bedrock and soil units and structures, geomorphic surfaces, fault-related geomorphic features, springs, and deformation of man-made structures due to fault creep. Field study should be detailed within the site with less detailed reconnaissance of an area within a mile or so of the site. Evidence from prehistoric liquefaction (paleoliquefaction) can also provide important information regarding the magnitude and timing of fault movement in the site area or region.
4. Subsurface investigations commonly are needed to evaluate location and activity of fault traces. These investigations may include trenches, test pits, and/or boreholes to permit detailed and direct observation of geologic units and faults.
5. The geometry of faults may be further defined by geophysical investigations including seismic refraction, seismic reflection, gravity, magnetic intensity, resistivity, ground penetrating radar, etc. These indirect methods require a knowledge of specific geologic conditions for reliable interpretation. Geophysical methods alone never prove the absence of a fault and they typically do not identify the recency of activity.
6. More sophisticated and more costly studies may provide valuable data where geological special conditions exist or where requirements for critical structures demand a more intensive investigation. These methods might involve repeated geodetic surveys, strain measurements, or monitoring of microseismicity and radiometric analysis (C^{14}, K-Ar), stratigraphic correlation (fossils, mineralology) soil profile development, paleomagnetism (magnetostratigraphy), or other dating techniques (thermoluminescense, cosmogenic isotopes) to evaluate the age of faulted or unfaulted units or surfaces.

The following information should be developed to provide documented support for conclusions relative to location and magnitude of faulting hazards:

1. Maps should be prepared showing the existence (or absence) and location of hazardous faults on or near the site. The distribution of primary and secondary faulting (fault zone width) and fault-related surface deformation should be shown.

2. The type, amount, and sense of displacement of past surface faulting episodes should be documented, if possible.
3. From this documentation, estimates of location, magnitude, and likelihood or relative potential for future fault displacement can be made, preferably from measurements of past surface faulting events at the site, using the premise that the general pattern of past activity will repeat in the future. Estimates also may be made from published empirical correlations between fault displacement and fault length or earthquake magnitude (e.g., Wells and Coppersmith, 1994). Where fault segment length and sense of displacement are defined, these correlations may provide an estimate of future fault displacement (either the maximum or the average to be expected). Probabilistic studies may be considered to evaluate the probability of fault displacement (e.g., Youngs et al., 2003).
4. The degree of confidence and limitations of the data should be addressed.

There are no codified procedures for estimating the amount or probability of future fault displacements. Estimates may be made, however, by qualified earth scientists using techniques described above. Because techniques for making these estimates are not standardized, peer review of reports is useful to verify the adequacy of the methods used and the estimates reports, to aid the evaluation by the permitting agency, and to facilitate discussion between specialists that could lead to the development of standards.

The following guidelines are given for safe siting of engineered construction in areas crossed by active faults:

1. Where ordinances have been developed that specify safe setback distances from traces of active faults or active fault zones, those distances must be complied with and accepted as the minimum for safe siting of buildings. For example, the general setback requirement in California is a minimum of 50 ft from a well-defined zone containing the traces of an active fault. That setback distance is mandated as a minimum for structures near faults unless a site-specific special geologic investigation shows that a lesser distance could be safety applied (*California Code of Regulations*, Title 14, Division 2, Sec. 3603(a)).
2. In general, safe setback distances may be determined from geologic studies and analyses as noted above. Setback requirements for a site should be developed by the site engineers and geologists in consultation with professionals from the building and planning departments of the jurisdiction involved. Where sufficient geologic data have been developed to accurately locate the zone containing active fault traces and the zone is not complex, a 50-ft setback distance may be specified. For complex fault zones, greater setback distances may be required. Dip-slip faults, with either normal or reverse motion, typically produce multiple fractures within rather wide and irregular fault zones. These zones generally are confined to the hanging-wall side of the fault leaving the footwall side little disturbed. Setback requirements for such faults may be rather narrow on the footwall side, depending on the quality of the data available, and larger on the hanging wall side of the zone. Some fault zones may contain broad deformational features such as pressure ridges and sags rather than clearly defined fault scarps or shear zones. Nonessential structures may be sited in these zones provided structural mitigative measures are applied as noted below. Studies by qualified geologists and engineers are required for such zones to assure that building foundations can withstand probable ground deformations in such zones.

Mitigation of surface faulting hazards There is no mitigative technology that can be used to prevent fault rupture from occurring. Thus, sites with unacceptable faulting hazard must either be avoided or structures designed to withstand ground deformation or surface fault rupture. In general practice, it is economically impractical to design a structure to withstand more than a few inches of fault displacement. Some buildings with strong foundations, however, have successfully withstood or diverted a few inches of surface fault rupture without damage to the structure (Youd, 1989; Kelson et al., 2001). Well reinforced mat foundations and strongly inter-tied footings have been most effective. In general, less damage has been inflicted by compressional or shear displacement than by vertical or extensional displacements.

Determination of Lateral Pressures on Basement and Retaining Walls Due to Earthquake Motions

Paragraph 1 of Section 11.8.3 requires that seismic lateral pressures on basement walls and free-standing retaining walls be determined for structures on SDC D through F, but does not specify the methods for calculating these pressures. Discussion and guidance regarding different approaches for determining seismic lateral pressures is given below.

Waterfront structures often have performed poorly in major earthquake due to excess pore water pressure and liquefaction conditions developing in relatively loose, saturated granular soils. However, damage reports for structures away from waterfronts are generally limited with only a few cases of stability failures or large permanent movements (Whitman, 1991). Due to the apparent conservatism or overstrength in static design of most walls, the complexity of nonlinear dynamic soil-structure interaction, and the poor understanding of the behavior of retaining structures with cohesive or dense granular soils, Whitman (1991) recommends that "engineers must rely primarily on a sound understanding of fundamental principles and of general patterns of behavior."

Seismic pressures on retaining walls is discussed below for two categories of walls: "yielding" walls that can move sufficiently to develop minimum active earth pressures and "nonyielding" walls that do not satisfy this movement condition. The amount of movement to develop minimum active pressure is very small. A displacement at the top of the wall of 0.002 times the wall height is typically sufficient to develop the minimum active pressure state. Generally, free-standing gravity or cantilever walls are considered to be yielding walls (except massive gravity walls founded on rock), whereas building basement walls restrained at the top and bottom are considered to be nonyielding.

Yielding walls At the 1970 Specialty Conference on Lateral Stresses in the Ground and Design of Earth Retaining Structures, Seed and Whitman (1970) made a significant contribution by reintroducing and reformulating the Mononobe-Okabe (M-O) seismic coefficient analysis (Mononobe and Matsuo, 1929; Okabe, 1926), the earliest method for assessing the dynamic lateral pressures on a retaining wall. The M-O method is based on the key assumption that the wall displaces or rotates outward sufficiently to produce the minimum active earth pressure state. The M-O formulation is expressed as:

$$P_{AE} = (1/2)\gamma H^2 (1-k_v) K_{AE} \qquad \text{(C11.8-4)}$$

where:

P_{AE} is the total (static + dynamic) lateral thrust, γ is unit weight of backfill soil, H is height of backfill behind the wall, k_V is vertical ground acceleration divided by gravitational acceleration, and K_{AE} is the static plus dynamic lateral earth pressure coefficient which is dependent on (in its most general form) angle of friction of backfill, angle of wall friction, slope of backfill surface, and slope of back face of wall, as well as horizontal and vertical ground acceleration. The formulation for K_{AE} is given in textbooks on soil dynamics (Prakash, 1981; Das, 1983; Kramer, 1996) and discussed in detail by Ebeling and Morrison (1992).

Simplified formulation. Seed and Whitman (1970), as a convenience in design analysis, proposed to evaluate the total lateral thrust, P_{AE}, in terms of its static component (P_A) and dynamic incremental component (ΔP_{AE}):

$$P_{AE} = P_A + \Delta P_{AE} \qquad \text{(C11.8-5a)}$$

or

$$K_{AE} = K_A + \Delta K_{AE} \qquad \text{(C11.8-5b)}$$

or

$$\Delta P_{AE} = (1/2)\gamma H^2 \Delta K_{AE} \qquad \text{(C11.8-5c)}$$

Seed and Whitman (1970), based on a parametric sensitivity analysis, further proposed that for practical purposes:

$$\Delta K_{AE} = (3/4)K_h \qquad \text{(C11.8-6)}$$

$$\Delta P_{AE} = (1/2)\gamma H^2 (3/4)k_h = (3/8)k_h \gamma H^2 \qquad \text{(C11.8-7)}$$

Where:

k_h is horizontal ground acceleration divided by gravitational acceleration. Unless permanent displacement of the wall is acceptable, k_h should be taken equal to the site peak ground acceleration, a_{max}, that is consistent with design earthquake ground motions as determined in Section 11.8.2, subsection entitled earthquake ground motions for geologic hazards evaluations. Equation 11.8-7 generally is referred to as the simplified M-O formulation. For walls that are in excess of 15 feet in height, special studies can also be conducted to evaluate the coherency of ground motions behind the wall from which an average seismic coefficient can be developed. These special studies require consideration of the frequency characteristics of ground motion, as well as the stiffness of the soil and the wall height, and usually require use of a finite element or difference computer model.

Since its introduction, there has been a consensus in geotechnical engineering practice that the simplified M-O formulation reasonably represents the dynamic (seismic) lateral earth pressure increment for yielding retaining walls. For the distribution of the dynamic thrust, ΔP_{AE}, Seed and Whitman (1970) recommended that the resultant dynamic thrust act at $0.6H$ above the base of the wall (that is, inverted trapezoidal pressure distribution).

Displacement-based approach. Using the simplified M-O formulation, a yielding wall may be designed using either a limit-equilibrium force approach (conventional retaining wall design) or an approach that permits movement of the wall up to tolerable amounts. Richards and Elms (1979) introduced a method for seismic design analysis of yielding walls considering translational sliding as a failure mode and based on tolerable permanent displacements for the wall. Elms and Martin (1979) showed that the $k_h = a_{max}/2$ is adequate for design if the wall is allowed to slide up to $10a_{max}$, where a_{max} is the design ground motion and displacement is in "inches." The use of $k_h = a_{max}/2$ has now become a common design practice without regard for the displacement that is associated with this assumption.

Similar to evaluations of seismic slope stability described in Section 11.8.2, careful attention needs to be given to the characterization of soil conditions behind the wall when using the displacement-based approach. Both the geometry of fill and native deposits, as well as the strength of the soil under cyclic loading, must be considered. Initially, the peak strength of the soil can be used for the analysis; however, if significant deformations are predicted, it may be necessary to repeat the analysis using the residual strength of the soil. See discussions on site characterization within the seismic slope stability section for additional guidance on the selection of soil strengths.

There are a number of other empirical formulations for estimating permanent displacements under a translation mode of failure; these have been reviewed by Whitman and Liao (1985). Nadim (1980) and Nadim and Whitman (1984) incorporated the failure mode of wall tilting as well as sliding by employing coupled equations of motion, which were further formulated by Siddharthan et al. (1992) as a design method to predict the seismic performance of retaining walls taking into account both sliding and tilting. Alternatively, Prakash et al. (1995) described design procedures and presented design charts for estimating both sliding and rocking displacements of rigid retaining walls. These design charts are the results of analyses for which the backfill and foundation soils were modeled as nonlinear viscoelastic materials. A simplified method that considers rocking of a wall on a rigid foundation about the toe was described by Steedman and Zeng (1996) and allows the determination of the threshold acceleration beyond which the wall will rotate. A simplified procedure for evaluating the critical threshold accelerations for sliding and tilting was described by Richards et al. (1996).

Application of methods for evaluating tilting of yielding walls has been limited to a few case studies and back-calculation of laboratory test results. Evaluation of wall tilting requires considerable engineering judgment. Because the tilting mode of failure can lead to instability of a yielding retaining wall, it is suggested that this mode of failure be avoided in the design of new walls by proportioning the walls to prevent rotation in order to displace only in the sliding mode.

Limitations of M-O approach. Although the M-O approach is simple to use, certain designs become very difficult to solve with the standard M-O equations. These designs involve high ground accelerations, combinations of moderate-to-high ground accelerations and steep backslopes, and where mixed backfill conditions exist (i.e., either where c-ϕ soils occur or where only a thin zone of granular backfill is placed between the wall and a cohesive or rock condition). For these cases the M-O approach does not provide realistic answers. An acceptable alternative approach for these cases is to use a generalized limit equilibrium (slope stability) computer program. With this alternate approach appropriate soil properties and geometry can be modeled, and the seismic coefficient can be defined on the basis of the peak ground acceleration or a reduced seismic coefficient if displacement is acceptable. With this generalized limit equilibrium method, the external force required for stability is computed. This force represents the dynamic earth pressure on the wall.

Nonyielding walls Wood (1973) analyzed the response of a rigid nonyielding wall retaining a homogeneous linear elastic soil and connected to a rigid base. For such conditions, Wood established that the dynamic amplification was insignificant for relatively low-frequency ground motions (that is, motions at less than half of the natural frequency of the unconstrained backfill), which would include many earthquake problems.

For uniform, constant k_h applied throughout the elastic backfill, Wood (1973) developed the dynamic thrust, ΔP_E, acting on smooth rigid nonyielding walls as:

$$\Delta P_E = F k_h \gamma H^2 \qquad \text{(C11.8-8)}$$

The value of F is approximately equal to unity (Whitman, 1991) leading to the following approximate formulation for a rigid nonyielding wall on a rigid base:

$$\Delta P_E = k_h \gamma H^2 \qquad \text{(C11.8-9)}$$

As for yielding walls, the point of application of the dynamic thrust is taken typically at a height of $0.6H$ above the base of the wall.

It should be noted that the model used by Wood (1973) does not incorporate any effect on the pressures of the inertial response of a superstructure connected to the top of the wall. This effect may modify the interaction between the soil and the wall and thus modify the pressures from those calculated assuming a rigid wall on a rigid base.

Although the study performed by Wood included dynamic analysis of a rigid wall with fixed base condition, the solution commonly used and presented in Equations C11.8-8 and C11.8-9 are based on static “1 g” loading of the soil and wall and does not include the effects of the wave propagation in the soil. The subject of soil-wall interaction is addressed in the following sections. This section also provides further discussion on the applicability of the Wood and the M-O formulations.

Soil-structure-interaction approach and modeling for partially embedded structures Lam and Martin (1986), Soydemir and Celebi (1992), Veletsos and Younan (1994a and 1994b), and Ostadan (2005), among others, argue that the earth pressures acting on the walls of partially embedded structures (e.g., basement walls) during earthquakes are primarily governed by soil-structure interaction (SSI) and, thus, should be treated differently from the concept of limiting equilibrium (that is, M-O method). Soil-structure interaction includes both a kinematic component—the interaction of a massless rigid wall with the adjacent soil as modeled by Wood (1973) but including the wave propagation in the soil—and an inertial component—the interaction of the wall, connected to a responding superstructure, with the adjacent soil. Detailed SSI analyses incorporating kinematic and inertial interaction may be considered for the estimation of seismic earth pressures on critical walls. Computer programs that may be utilized for such analyses include FLUSH (Lysmer et. al, 1975) and SASSI2000 (Lysmer et al., 1999).

Ostadan (2005) observed that for partially embedded structures subjected to ground shaking, the characteristics of the dynamic earth pressure amplitudes versus frequency of the ground motion were those of a single-degree-of-freedom (SDOF) system and proposed a simplified method to estimate the magnitude and distribution of dynamic thrust. Results provided by Ostadan (2005) utilizing this simplified method, which were also confirmed by dynamic finite element analyses, indicate that, depending on the dynamic properties of the backfill as well as the frequency characteristics of the input ground motion, a range of dynamic earth pressure solutions would be obtained for which the M-O solution and the Wood (1973) solution represent a "lower" and an "upper" bound, respectively.

The solution by Ostadan considers the kinematic soil-structure interaction effects and is based on the dynamic soil properties and the design ground motion characteristics. This solution assumes a rigid wall on rigid foundation and does not include the effect of the superstructure and its inertia on seismic soil pressure. The 5-step method to compute the seismic soil pressure following Ostadan's method is as follows:

1. Perform free-field soil column analysis and obtain the ground response motion at the depth corresponding to the base of the wall in the free-field. The response motion in terms of acceleration response spectrum at 30 percent damping should be obtained. The free-field soil column analysis may be performed using the computer program SHAKE with input motion specified either at the ground surface or at the depth of the foundation basemat. The choice for location of control motion should be consistent with the development of the design motion.

2. Use Eq. (C11.8-10) to compute the total soil mass (m) using the Poisson's ratio (ν) and mass density of the soil.

$$m = 0.50\,(\rho)\,H^2\,(\Psi\nu) \qquad \text{(C11.8-10)}$$

where:

ρ = mass density of the soil (total weight density divided by acceleration of gravity)
H = height of the wall
$\Psi\nu$ = factor to account for the Poisson's ratio as defined by the following equation.

$$\Psi\nu = 2 / [(1-\nu)(2-\nu)]^{0.5} \qquad \text{(C11.8-11)}$$

3. Obtain the total lateral seismic force from the product of the total mass obtained in Step 2 and the acceleration spectral value of the free-field response at the soil column frequency obtained at the depth of the bottom of the wall (Step 1). The soil column frequency (f_s) is an output provided by SHAKE.

$$f_s = v_s/(4H) \qquad \text{(C11.8-12)}$$

where:

v_s = average strain-compatible shear wave velocity of the soil column over the height of the wall.

4. Obtain the maximum lateral seismic soil pressure at the ground surface level by dividing the lateral force obtained in Step 3 by the area under the normalized seismic soil pressure, 0.744 *H*.

5. Obtain the pressure profile by multiplying the peak pressure from Step 4 by the pressure distribution relationship given by Equation (C11.8-13 below).

$$p(y) = -0.0015+5.05y-15.84y^2+28.25y^3-24.59y^4+8.14y^5 \qquad \text{(C11.8-13)}$$

where:

y = normalized height ratio (Y/H) measured from the bottom of the wall and ranging from 0 at the bottom of the wall to 1 at the top of the wall.

Y = the distance of the point under consideration from the bottom of the wall.

The area under the seismic soil pressure curve can be obtained from integration of the pressure distribution over the height of the wall. The total area is $0.744H \times p_{\max}$ for a wall with the height of H and maximum pressure of $p_{\max}$ at the top.

With this method, the site specific dynamic soil properties, soil nonlinear effects and the characteristics of the design motion are considered in the computation of the seismic soil pressure. A complete verification of the 5-step method against finite element solutions and comparison with the Wood solution and the M-O method is presented by Ostadan (2005).

Observations and discussion for partially embedded structures Chang et al. (1990) have found that dynamic earth pressures recorded on the wall of a model nuclear reactor containment building were consistent with dynamic pressures predicted by the M-O solution. Analysis by Chang et al. indicated that the dynamic wall pressures were strongly correlated with the rocking response of the structure. Whitman (1991) has suggested that SSI effects on basement walls of buildings reduce dynamic earth pressures and that M-O pressures may be used in design except where structures are founded on rock or hard soil (that is, no significant rocking). In the latter case, the pressures given by the Ostadan (2005) method with the Wood (1973) formulation as the upper bound would appear to be more applicable. The effect of rocking in reducing the dynamic earth pressures on basement walls also has been suggested by Ostadan and White (1998). This condition may be explained if it is demonstrated that the dynamic displacements induced by kinematic and inertial components are out of phase.

Effect of saturated backfill on wall pressures The previous discussions are limited to backfills that are not water-saturated. In current (2006) practice, drains typically are incorporated in the design to prevent groundwater from building up within the backfill. This is not practical or feasible, however, for waterfront structures (such as quay walls) where most of the earthquake-induced failures have been reported (Seed and Whitman, 1970; Ebeling and Morrison, 1992; ASCE-TCLEE, 1998).

During ground shaking, the presence of water in the pores of a backfill can influence the seismic loads that act on the wall in three ways (Ebeling and Morrison, 1992; Kramer, 1996): (1) by altering the inertial forces within the backfill, (2) by developing hydrodynamic pressures within the backfill and (3) by generating excess porewater pressure due to cyclic straining. Effects of the presence of water in cohesionless soil backfill on seismic wall pressures can be estimated using formulations presented by Ebeling and Morrison (1992).

A soil liquefaction condition behind a wall may under the design earthquake have a pronounced effect on the wall pressures during and for some time after the earthquake. At present there is no general consensus established for estimating lateral earth pressures for liquefied backfill conditions. One simplified and probably somewhat conservative approach is to treat the liquefied backfill as a heavy viscous fluid exerting a hydrostatic pressure on the wall. In this case, the viscous fluid has the total unit weight of the liquefied soil. If unsaturated soil is present above the liquefied soil, it is treated as a surcharge that increases the fluid pressure within the underlying liquid soil by an amount equal to the thickness times the total unit weight of the surcharge soil. In addition to these "static" fluid pressures exerted by a liquefied backfill, hydrodynamic pressures can be exerted by the backfill. The magnitude of any such hydrodynamic pressures would depend on the level of shaking following liquefaction. Hydrodynamic effects may be estimated using formulations presented by Ebeling and Morrison (1992).

REFERENCES

American Society of Civil Engineers, Technical Committee on Lifeline Earthquake Engineering. 1998. *Seismic Guidelines for Ports*, edited by S. D. Werner, Monograph 12.

Ashford, S. A., Juirnarongrit, T. Sugano, T., and M. Hamada. 2006. "Soil-Pile Response to Blast-Induced Lateral Spreading. I. Field Test," Journal of Geotechnical and Geoenvironmental Engineering, Vol. 132, No. 2, February.

Ashford, S.A., Rollins, K.M., and J.D. Lane. 2004. "Blast-Induced Liquefaction for Full-Scale Foundation Testing," Journal of Geotechnical and Geoenvironmental Engineering, Vol. 130, No. 8, August.

Bardet, J.P., Tobita, T., Mace, N., and Hu, J., 2002." Regional modeling of liquefaction-induced ground deformation," Earthquake Spectra, 18(1), 19-46.

Blake, T.F., Hollingsworth, R.A., and Stewart, J.P., editors 2002. Recommended procedures for implementation of DMG Special Publication 117, Guidelines for analyzing and mitigating landslide hazards in California, Southern California Earthquake Center, University of Southern California, Los Angeles, California, 130 p.

Boulanger, R.W. and I.M. Idriss. 2004. Evaluating the Potential for Liquefaction or Cyclic Failure of Silts and Clays. University of California at Davis, Report No. UCD/CGM-04/01.

Bray, J.D., and Rathje, E.M. 1998. Earthquake-induced displacements of solid-waste landfills, *Journal of Geotechnical and Geoenvironmental Engrg.*, ASCE, 124, 242-253.

Bray, J.D., Sancio, R.B., Riemer, M.F. and T. Durgunoglu. 2004. "Liquefaction Susceptibility of Fine-grained Soils." 11th International Conference of Soil Dynamics and Earthquake Engineering and 3rd Internation Conference on Earthquake Geotechnical Engineering, D. Doolan et al. eds. Stallion Press.

California Geological Survey, 1997. Special Publication 117, Guidelines for Evaluating and Mitigating Seismic Hazards in California.

California Geological Survey, 2002. Guidelines for evaluating the hazard of surface fault rupture: California Geological Survey Note 49, 4 p.

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

Chang, C. Y., M. S. Power, C. M. Mok, Y. K. Tang, and H. T. Tang. 1990. "Analysis of Dynamic Lateral Earth Pressures Recorded on Lotung Reactor Containment Model Structure," in *Proceedings, Fourth U.S. National Conference on Earthquake Engineering*, pp. 643-652. EERI.

Das, B. M. 1983. *Fundamentals of Soil Dynamics*. New York: Elsvier.

Dobry, R, Ladd. R.S., Yokel, F.Y., Chung, R.M., and D. Powell. 1982. “Prediction of Pore Water Pressure Buildup and Liquefaction of Sands During Earthquakes by the Cyclic Strain Method,” National Bureau of Standards Building Science Series 138.

Ebeling, R. M., and E. E. Morrison. 1992. *The Seismic Design of Waterfront Retaining Structures*, Technical Report ITL-92-11. Vicksburg, Mississippi: Corps of Engineers Waterways Experiment Station.

Elms, D.A. and Martin, G.R. 1979. “Factors Involved in the Seismic Design of Bridge Abutments,” In *Proceedings of the Workshop on Seismic Problems Related to Bridges.* Applied Technology Council, Redwood City, CA.

Franklin, A.G. and Chang, F.K., 1977. "Earthquake Resistance of Earth and Rock-Fill Dams; Permanent Displacements of Earth Embankments by Newmark Sliding Block Analysis," Miscellaneous Paper S-71-17, Report 5, U.S. Army Waterways Experiment Station, CE, Vicksburg, MS.

Goodman, R.E. and Seed, H.B., 1966. Earthquake-induced displacements in Sand embankments, ASCE Journal of Soil Mechanics and Foundation, v. 92, no. SM2, p. 125-146.

Hart, E.W., and Bryant, W.A., 1997 (revised). Fault-rupture hazard zones in California: California Department of Conservation, Division of Mines and Geology (now California Geological Survey) Special Publication 42, 38p.

Hanson, K. L., Kelson, K. I., Angell, M. A., and Lettis, W. R., 1999, Techniques for Identifying Faults and Determining Their Origin. NUREG/CR-5503. U.S. Nuclear Regulatory Commission.

Hynes, M.E. and Franklin, A.G., 1984. "Rationalizing the Seismic Coefficient Method," Miscellaneous Paper GL-84-13, U.S. Army Waterways Experiment Stations, Vicksburg, MS, July, 21 p.

Idriss, I. M. 1985. “Evaluating Seismic Risk in Engineering Practice.” In *Proceedings of the 11th International Conference on Soil Mechanics and Foundation Engineering*, Vol. 1, p. 155.

Idriss, I.M. and Boulanger, R.W., 2004. “Semi-empirical procedures for evaluating liquefaction potential during earthquakes”, Proc. 11th International conference on soil dynamics and earthquake engineering, and 3rd International conference on earthquake geotechnical engineering, vol. 1 Stallion Press; 2004. p. 32-56.

Idriss, I. M. And Boulanger, R.W., 2006. ”Semi-empirical procedures for evaluating liquefaction potential during earthquakes,” Soil Dynamic and Earthquake Engineering 26, 115-130.

Ishihara, Kenji. 1985. “Stability of Natural Deposits During Earthquakes.” In *Proceedings of the 11th International Conference on Soil Mechanics and Foundation Engineering*, San Francisco, Vol. 1, pp. 321-376.

Ishihara, K. and Yoshimine, M., 1992 “Evaluation of Settlements in Sand Deposits Following Liquefaction During Earthquakes,” Soils and Foundations, JSSMFE, Vol. 32, No. 1.

Itasca Consulting Group. 1997. “User’s Manual for FLAC, V. 3.4,” Itasca, Minneapolis, MN.

Jibson, R.W. 1993. “Predicting Earthquake-Induced Landslide Displacements Using Newmark’s Sliding Block Analysis,” Transportation Research Record, No. 1411 – Earthquake-Induced Ground Failure Hazards, Transportation Research Board. National Research Council, Washington, DC.

Kagawa, T., Sata, M., Minowa, C., Abe, A., and T. Tazoh. 2004. “Centrifuge Simulations of Large-Scale Shaking Table Tests: Case Studies,” Journal of Geotechnical and Geoenvironmental Engineering, Vol. 130, No. 7, July.

Kelson, K.I., Kang, K.-H., Page, W.D., Lee, C.-T., and Cluff, L.S., 2001. “Representative styles of deformation along the Chelungpu fault from the 1999 Chi-Chi (Taiwan) Earthquake: Geomorphic characteristics and responses of man-made structures,” Bulletin of the Seismological Society of America,

v. 91, no. 5, p. 930-952.

Kramer, S.L. 1996. *Geotechnical Earthquake Engineering*. New Jersey: Prentice Hall.

Lam, I., and G. R. Martin. 1986. *Seismic Design of Highway Bridge Foundations—Design Procedures and Guidelines*, Report FHWA/RD-86/102. Washington, D.C.: FHWA.

Lysmer, J., F. Ostadan, C. Chin. 1999. *SASSI2000 A System for Analysis of Soil-Structure Interaction*, University of California, Berkeley, Department of Engineering.

Lysmer, J., T. Udka, C. P. Tsai, and H. B. Seed. 1975. *FLUSH: A Computer Program for Approximate 2-D Analysis of Soil-Structure Interaction Problems*, Report EERC 75-30. Berkeley: University of California, Earthquake Engineering Research Center.

Makdisi, F. T., and H. B. Seed. 1978. "Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations." *Journal of Geotechnical Engineering* ASCE 104(GT7):849-867.

Martin, G.R., and Qiu, P., 1994. "Effects of Liquefaction on Vulnerability Assessment," NCEER Highway Project on Seismic Vulnerability of New and Existing Highway Construction, One Year Research Tasks – Technical Research Papers, 1994.

Martin, G.R., and Lew, M. 1999. "Recommended Procedures for Implementation of DMG Special Technical Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California," Southern California Earthquake Center, University of Southern California, March, 63 p.

Monobe, N., and H. Matsuo. 1929. "On the Determination of Earth Pressures During Earthquakes." In *Proceedings, World Engineering Congress*.

Nadim, F. 1980. "Tilting and Sliding of Gravity Retaining Walls," MS thesis, Department of Civil Engineering, Massachusetts Institute of Technology, Cambridge.

Nadim, F., and R. V. Whitman. 1984. "Coupled Sliding and Tilting of Gravity Retaining Walls During Earthquakes." In *Proceedings of the Eighth World Conference on Earthquake Engineering*, pp. 477-484.
National Research Council, Committee on Earthquake Engineering. 1985. *Liquefaction of Soils During Earthquakes*. Washington, D.C.: National Academy Press.

National Center for Earthquake Engineering Research (NCEER). 1997. Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils, Technical Report NCEER-97-0022, 276 p.

National Research Council. 1985. Liquefaction of Soils During Earthquakes, National Academy Press, 240p.

Newmark, N.M. 1965. "Effects of earthquakes on dams and embankments," Geotechnique, London, England v.5, no.2.

Okabe, S. 1926. "General Theory of Earth Pressure," *Journal of the Japan Society of Civil Engineers* 12,1.

Ostadan, F. 2005. "Seismic soil pressure for building walls – an updated approach," Journal of Soil Dynamics and Earthquake Engineering, v.25, p. 785-793; also published in Proceedings of the 11th International Conference on Soil Dynamics and Earthquake Engineering and 3rd International Conference on Earthquake Geotechnical Engineering, University of California, Berkeley, 2004.

Ostadan, F., and W. H. White. 1998. Lateral Seismic Soil Pressure—An Updated Approach. In Preproceedings of UJNR Workshop on Soil-Structures Interaction, U.S. Geological Survey, Menlo Park, California.

Prakash, S. 1981. *Soil Dynamics*. New York: McGraw-Hill Book Co.

Prakash, S., Y. Wu, and E. A. Rafnsson. 1995. *Displacement Based Aseismic Design Charts for Rigid Walls*. Rolla, Missouri: Shamsher Prakash Foundation.

Rausch, A.F. and J.M. Martin. 2000. "EPOLLS Model for Predicting Average Displacement on Lateral Spreads," Journal of Geotechnical and Geoenvironmental Engineering, Vol. 126, No. 4, April.

Richards, R. J., and D. Elms. 1979. "Seismic Behavior of Gravity Retaining Walls." *Journal of the Geotechnical Engineering Divisions, ASCE* 105(GT4): 449-464.

Richards, R., K. L. Fishman, and R. Devito. 1996. "Threshold Accelerations for Rotation or Sliding of Bridge Abutments," *Journal of Geotechnical Engineering*, *ASCE* 105, GT4, 449-464.

Seed, H. B., and I. M. Idriss. 1971. "Simplified Procedure for Evaluating Soil Liquefaction Potential." *Journal of the ASCE Soil Mechanics and Foundations Division* 97(SM9):1249-1273.

Seed, H. B., K. Tokimatsu, L. F. Harder, and R. M. Chung. 1985. *The Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations*, Report UBC/EERC-84/15. Berkeley, California: Earthquake Engineering Research Center.

Seed, H. B., and I. M. Idriss. 1982. *Ground Motions and Soil Liquefaction During Earthquakes*, Vol. 5 of *Engineering Monographs on Earthquake Criteria, Structural Design, and Strong Motion Records*. El Cerrito, California: Earthquake Engineering Research Institute.

Seed, H. B., and R. V. Whitman. 1970. "Design of Earth Retaining Structures for Dynamic Loads," in *Proceedings, ASCE Specialty Conference on Lateral Stresses in the Ground and Design of Earth-Retaining Structures*, pp. 103-147. Ithaca, New York: Cornell University.

Seed, R. B., and L. F. Harder. 1990. "SPT-Based Analysis of Cyclic Pore Pressure Generation and Undrained Residual Strength." In *Proceedings of the H. Bolton Seed Memorial Symposium*, Vol. 2, pp. 351-376.

Siddarthan, R., S. Ara, and G. Norris. 1992. "Simple Rigid Plastic Model for Seismic Tilting of Rigid Walls," *Journal of Structural Engineering*, ASCE, 469-487.

Slemmons, D.B., and dePolo, C.M., 1986. Evaluation of active faulting and associated hazards, in Studies in Geophysics—Active Tectonics: National Academy Press, Washington, D.C., p. 45-62.

Soydemir, C., and M. Celebi. 1992. "Seismic Design of Buildings with Multi-Level Basements," in *Proceedings*, Tenth World Conference on Earthquake Engineering, Madrid Spain, Vol. 6, pp. 1731-1734.

Stark, T.D., and Mesri, G., 1992. "Undrained Shear Strength of Liquefied Sands for Stability Analysis", *Journal of Geotechnical Engineering,* American Society of Civil Engineers, v. 118, no. 11, p. 1727-1747.

Steedman, R. S., and X. Zeng. 1996. "Rotation of Large Gravity Retaining Walls on Rigid Foundations Under Seismic Loading." In *Analysis and Design of Retaining Walls Against Earthquakes*, ASCE Geotechnical Special Publication 60, edited by S. Prakash, pp. 38-56. New York: ASCE.

Stewart, J.P., Blake, T.M., and Hollingsworth, R.A., 2003, A screen analysis procedure for seismic slope stability: Earthquake Spectra, 19 (3).

Swan, F.H., Stepp, J.C., McGuire, R.K., 1991, Assessment of the potential for tectonic fault rupture for high-level nuclear waste repositories: Quarterly Journal of Engineering Geology, v. 24, p. 337-346.

Tokimatsu, K., and H. B. Seed. 1987. "Evaluation of Settlements in Sands Due to Earthquake Shaking." *Journal of Geotechnical Engineering* ASCE 113(8):861-878.

U.S. Army Corps of Engineers, 2005. Design: Seismic Design for Buildings, Unified Facilities Criteria, UFC 3-310-03A, formerly TI-809-04 (1998), Appendix F, Geologic Hazards Evaluations.

Utah Section of the Association of Engineering Geologists, 1987, Guidelines for evaluating surface fault rupture hazard in Utah: Utah Geological and Mineral Survey Miscellaneous Publication N, 2 p.

Veletsos, A., and A. H. Younan. 1994a. "Dynamic Soil Pressure on Rigid Vertical Walls," *Earthquake*

Engineering and Soil Dynamics, 23:275-301.

Veletsos, A. and Younan, A. H. 1994b. “Dynamic Modeling and Response of Soil-Wall Systems,” *Journal of Geotechnical Engineering*, ASCE, 120:12.

Wells, D.L., and Coppersmith, K.J., 1994. "New Empirical Relationships Among Magnitude, Rupture Area, and Surface Displacement," Bulletin of the Seismological Society of America, Vol, 84, p. 974-1002.

Whitman, R. V. 1991. “Seismic Design of Earth Retaining Structures,” in *Proceedings*, Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, Missouri.

Whitman, R. V., and S. Liao. 1985. *Seismic Design of Retaining Walls*, Miscellaneous Paper GL-85-1. Vicksburg, Mississippi: Corps of Engineers Waterways Experiment Station.

Wong, C.P. and Whitman, R.V., 1982. "Seismic Analysis and Improved Seismic Design Procedure for Gravity Retaining Walls," Research Report 82-83, Department of Civil Engineering, Massachusetts Institute of Technology, Cambridge, MA.

Wood, J. H. 1973. *Earthquake-Induced Soil Pressures on Structures*, Report EERL 73-05. Pasadena: California Institute of Technology.

Youd, T. L. 1989. “Ground Failure Damage to Buildings During Earthquakes.” In *Foundation Engineering—Current Principles and Practices*, Vol. 1, pp. 758-770. New York: American Society of Civil Engineers.

Youd, T.L., and Noble, S.K., 1997. Magnitude scaling factors, *Proc. NCEER Workshop on Evaluation of Liquefaction Resistance of Soils,* National Center for Earthquake Engineering Research, State University of New York at Buffalo, 149-165.

Youd, T.L., Hansen, C.M., and Bartlett, S.F., 2002. “Revised multilinear repression equations for prediction of lateral spread displacement,” ASCE Journal of Geotechnical and Geoenvironmental Engineering, 128:12, p. 1007-1017.

Youd, T.L., Idriss, I.M., Andrus, R.D., Arango, I., Castro, G., Christian, J.T., Dobry, R., Finn, W., D.L. Harder, L.F., Jr., Hynes, M.E., Ishihara, K., Koester, J.P., Liao, S.S.C., Marcuson, W. F., III, Martin, G.R. Mitchell, J.K., Moriwaki, Y., Power, M.S., Robertson, P.K., Seed, R.B., and Stokoe, K.H., II, 2001. 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 Geo-environmental Engineering, ASCE, 127 (10), p. 817-833.

Youngs, R.R., Arabasz, W.J., Anderson, R.E., Ramelli, A.R., Ake, J.P., Slemmons, D.B., McCalpin, J.P., Doser, D.I., Fridrich, C.J., Swan III, F.H., Rogers, A.M., Yount, J.C., Anderson, L.W., Smith, K.D., Bruhn, R.L., Kneupfer, P.L.K., Smith, R.B., dePolo, C.M., O’Leary, D.W., Coppersmith, K.J., Pezzopane, S.K., Schwartz, D.P., Whitney, J.W., Olig, S.S., and Toro, G.R., 2003. A methodology for probabilistic fault displacement hazard analysis (PFDHA): Earthquake Spectra, v. 19, no. 1, p. 191-219.

Zhang, G., Robertson, P.K., and R.W.I. Brachman. 2004. “Estimating Liquefaction-Induced lateral Displacements Using the Standard Penetration Test of Cone Penetration Test,” Journal of Geotechnical and Geoenvironmental Engineering, Vol. 130, No. 8, August.