

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 EARTH PRESSURES**

Summarized below are procedures that are commonly used for evaluating potential site geologic hazards and seismic lateral earth pressures due to earthquakes. The geologic hazards include slope instability, liquefaction, ground displacement, and surface fault rupture. Geologic hazards evaluations should be carried out by qualified geotechnical professionals and documented in a report. Reporting requirements are given in Provisions Section 11.8. Seismic lateral earth pressure discussions consider both yielding and nonyielding walls.

GEOLOGIC HAZARDS

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 geologic 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, ground displacement, 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).

Slope Instability Hazard

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

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

13 In the pseudo static analysis, inertial forces generated by earthquake shaking are represented by an
14 equivalent static horizontal force acting on the slope. The seismic coefficient for this analysis is generally
15 taken as proportional to the site geomean peak ground acceleration, a_{max} (see details in Stewart et al.,
16 2003). The vertical component of ground acceleration is normally assumed to be zero during this
17 representation. The factor of safety for a given seismic coefficient can be estimated by using traditional
18 slope stability calculation methods. A factor of safety greater than 1.0 indicates that the slope is stable for
19 the given lateral force level and further analysis is not required. A factor of safety of less than 1.0
20 indicates that the slope will yield and slope deformation can be expected, and a deformational analysis
21 should be made using the techniques discussed below.

22 A common practice when using the pseudo static method is to reduce the peak ground acceleration by a
23 factor to account for the transitory nature of the ground motions. The factor used in this reduction is often
24 selected as 0.5 but lower reduction factors have also been used. For these analyses the acceptable factor
25 of safety is often taken as 1.1 to 1.3. Implicit within this approach is that deformation of the slope is
26 acceptable. The amount of deformation can range from a few inches to several feet (Blake et al., 2002).
27 Movements of this magnitude are not normally acceptable for building design. To limit permanent
28 displacements to less than 2 inches (5 cm), the recommended approach in this guideline is to use the peak
29 ground acceleration multiplied by f_{eq} , as discussed in Blake et al. (2002) and Stewart et al. (2003), in the
30 pseudo static analysis, and then if the resulting factor of safety is less than 1.0, a deformational analysis is
31 conducted.

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

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

46 For sites where soils could liquefy or where sensitive soils are known to occur, special studies will be
47 required. If liquefaction is predicted under the design seismic event, the strength of the soil in a liquefied
48 state should be used in the pseudo static stability analyses. Additional discussions of the strength of

1 liquefied soils are presented later in the liquefaction hazard section. If sensitive clayey soils exist, special
2 laboratory tests may be required to establish the amount of degradation in soil strength that will occur
3 with cyclic loading.

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

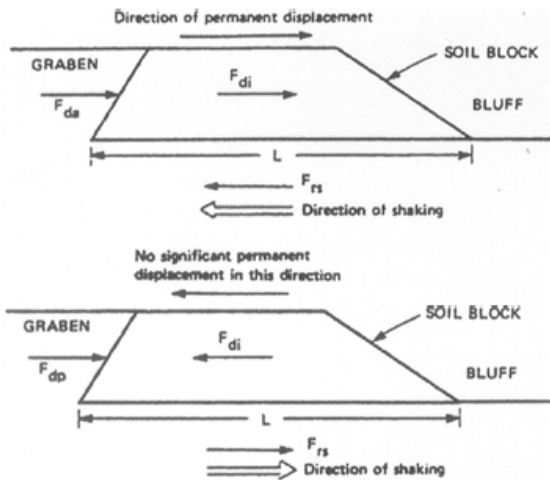
13 The yield acceleration depends primarily on the strength of the soil and the gradient and height and other
14 geometric attributes of the slope. The same comments on the characterization of the slope and soil
15 strength given above for the pseudo static analysis apply for the deformational analysis,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

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

30 An acceptable alternative method for the determination of displacements on many projects involves the
31 use of charts or simplified equations that show or estimate displacements for different acceleration ratios,
32 where the acceleration ratio is defined as the ratio of yield acceleration to the maximum horizontal
33 equivalent acceleration (MHEA) in the slide mass. These charts and equations have been developed by
34 calculating the cumulative displacement following the Newmark method for large sets of earthquake
35 records. The charts include those by Franklin and Chang (1977), Makdisi and Seed (1978), Wong and
36 Whitman (1982), Hynes and Franklin (1984), Martin and Qiu (1994); Bray and Rathje (1998), Bray et al.
37 (1998), and Jibson (2007). Simple equations include those by Bray and Travararou (2007), Jibson
38 (2007), Saygili and Rathje (2008), and Rathje and Saygili. (2008). Figure 11.8-3 shows the simplified
39 chart from Bray et al. (1998) that was recommended for use by Blake et al. (2002). The $D_{5,95}$ term in this
40 figure is the significant duration of shaking – with its relationship differing depending on whether the site
41 is within or greater than 10 km from the earthquake source. The selection between the different charts
42 should be made on the basis of the type of slope and the degree of conservatism necessary for the project.
43 It is important to recognize that when using one of the charts, or the Newmark method in general, a
44 number of simplifying assumptions are made regarding the relationship between the MHEA and the peak
45 acceleration at the site as well as other factors. These assumptions may limit the accuracy to which the
46 deformations can be estimated.

47 Slope deformations can also be estimated by using more rigorous two-dimensional computer modeling
48 methods. The computer codes FLAC (Itasca, 1997) and PLAXIS (Plaxis, 2008) are perhaps the most
49 common of the programs being used by practitioners for evaluating the response of slopes to seismic

1 loads. These computer programs allow various soil geometries, soil layering, and groundwater conditions
2 to be modeled. Earthquake records representative of the design seismic event are used to conduct the
3 time history analysis. Results provide an understanding of the development of deformations with time,
4 the location of critical surfaces of deformation, and the effects of pore water pressure buildup on slope
5 movement. As with any rigorous model, the accuracy of the deformation estimate is critically dependent
6 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 landslides for calculating permanent lateral displacements from earthquake ground motions (National Research Council, 1985; from Idriss, 1985)

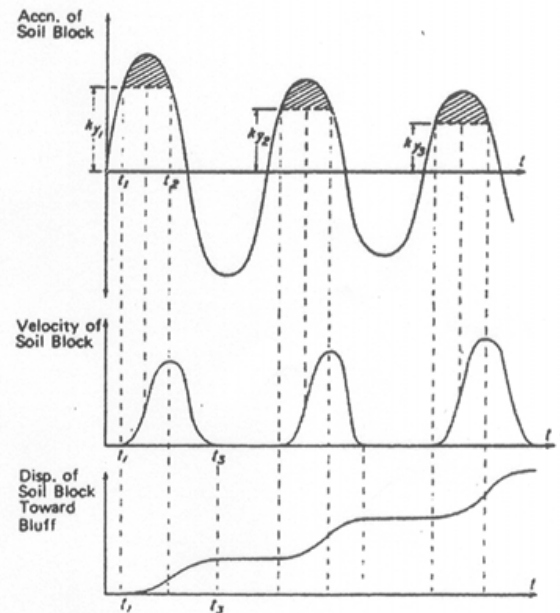
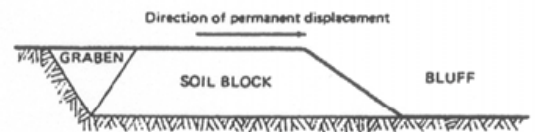
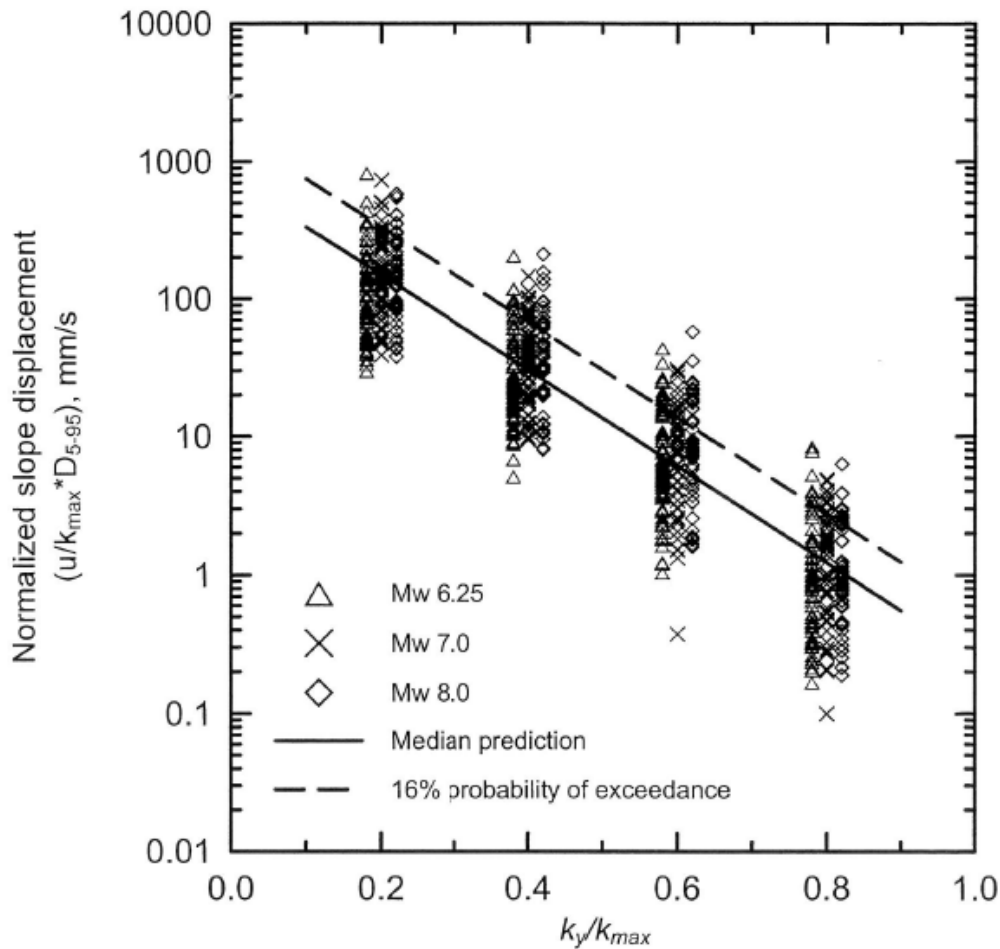


Figure 11.8-2 Schematic illustration for calculating displacement of soil block toward the bluff (National Research Council, 1985; from Idriss, 1985, adapted from Goodman and Seed, 1966)

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4 **Figure C11.8-3 Normalized Sliding Displacement (Bray et al., 1998) as recommended by Blake et**
 5 **al. (2002).**

6

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

14 Techniques to stabilize a site include increasing the resistance of the soil to displacement by subsurface
 15 drainage, buttresses, retaining walls, ground anchors, reaction piles or shafts, ground improvement using
 16 densification or soil mixing methods, or chemical treatment. Addition details for these mitigation
 17 methods can be found in various reports, including Blake et al. (2002).

18 **Liquefaction Hazard**

19 Liquefaction forms the second and, perhaps, the most widely known geologic hazard that must be
 20 considered at a building site. This hazard occurs when earthquake-induced ground shaking results in loss

1 of strength within water-saturated, loose granular soils. The consequence of this strength loss relative to a
2 building can be reduction in bearing capacity, total and differential settlement, and horizontal ground
3 displacement from lateral spreading or flow failures within the ground. In this section, the hazard of
4 differential settlement, whether due to liquefaction of water-saturated soils or compaction of non-
5 saturated soils, is addressed.

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

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

19
20 Three different methods have been proposed and are used to various extents for evaluating liquefaction
21 potential: empirical methods; analytical methods; and physical modeling.

- 22 1. Empirical methods are the most widely used methods in practice. These procedures rely on
23 correlations between observed cases of liquefaction/non-liquefaction and measurements made in the
24 field with conventional exploration methods. Seed and Idriss (1971) first published the widely used
25 “simplified procedure” utilizing the Standard Penetration Test (SPT). Since then, field test methods
26 in addition to the SPT have been utilized in similar simplified procedures. These methods include
27 cone penetrometer tests (CPTs), Becker hammer tests (BHTs), and shear wave velocity tests (SVTs).
28 These empirical procedures are summarized in the proceedings from a workshop (referred to as the
29 Liquefaction Workshop) held in 1996 (NCEER, 1997; Youd et al., 2001). Martin and Lew (1999)
30 provide additional details on the implementation of these procedures relative to engineering practice;
31 Idriss and Boulanger (2008) update the original Seed and Idriss (1982) monograph covering methods
32 for evaluating liquefaction during earthquakes.
- 33 2. Analytical methods are used less frequently to evaluate liquefaction potential – though they may be
34 required for special projects or where soil conditions are not amenable to the empirical method.
35 Analytical methods will also likely continue to gain prominence with time as numerical methods and
36 soil models improve and are increasingly validated. Originally (circa 1970s) the analytical method
37 involved determination of the induced shearing stresses with a program such as SHAKE and
38 comparing these stresses to results of cyclic triaxial or cyclic simple shear tests. Now the analytical
39 method usually refers to a computer code that incorporates a soil model that calculates the buildup in
40 pore water pressure. These more rigorous numerical methods include one-dimensional, nonlinear
41 effective stress codes such as DESRA, DMOD, SUMDES, and TESS and two dimensional, nonlinear
42 effective stress codes such as FLAC, TARA, and DYNFLOW. This new generation of analytical
43 methods has soil models that are fit to or derived from laboratory data or from liquefaction curves
44 developed from SPT or other field information. These methods are limited by the ability to represent
45 the soil model from either the laboratory or field measurements and by the complexity of the wave
46 propagation mechanisms, including the ability to select appropriate earthquake records to use in the
47 analyses.
- 48 3. Physical modeling originally involved the use of centrifuges or relatively small-scale shaking tables
49 to simulate seismic loading under well-defined boundary conditions. Physical model testing also now

1 includes large laminar boxes mounted on very large shake tables (e.g., Kagawa et al., 2004) and full-
2 scale field blast loading tests (e.g., Ashford et al. 2004; Ashford et al., 2006). This type of modeling
3 is one of the main focus areas of the 2004-2014 Network for Earthquake Engineering Simulation
4 (NEES) supported by the National Science Foundation. Soil used in the small-scale and laminar box
5 models is reconstituted to represent different density and geometrical conditions. Because of
6 difficulties in precisely modeling in situ conditions at liquefiable sites, small-scale and laminar box
7 models have seldom been used in design studies for specific sites. However, physical models are
8 valuable for analyzing and understanding generalized soil behavior and for evaluating the validity of
9 constitutive models under well-defined boundary conditions. Blast loading tests have been conducted
10 to capture the in situ characteristics of the soil for research and design purposes (e.g., Treasure Island,
11 California; Cooper River Bridge in South Carolina, and in Japan). However, the cost and safety
12 issues of blasting methods limit its use to only special design or research projects.

13 Most liquefaction hazards assessments for buildings will involve use of the SPT empirical method –
14 partly because of the wide acceptance of this approach and also because this approach can be easily
15 integrated into the geotechnical investigations normally performed during building design. The SPT
16 method is based on recommendations developed at the Liquefaction Workshop as described in NCEER
17 (1997) and Youd et al. (2001) or on one of the updates to this methodology as discussed below.

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

25 **Recent Updates to the SPT and CPT Procedures.** The methods presented in the Liquefaction
26 Workshop and summarized in the following section represent a consensus-based approach for the onset or
27 triggering of liquefaction; however, the consensus workshop occurred over 10 years ago. A number of
28 significant modifications to the methods presented in the Liquefaction Workshop have been
29 recommended over the past 10 years. These modifications include changes to the stress reduction
30 coefficient (r_d), modifications to the magnitude scaling factor (MSF) [also referred to as the duration
31 weighting factor (DWF)], revisions to the overburden correction term for CRR (K_σ) and the fines
32 correction (FC), refinements to the overburden correction for penetration resistance (C_N), and finally
33 changes to the relationship between cyclic stress ratio causing liquefaction and the normalized penetration
34 resistance; that is, the fundamental liquefaction strength curve such as shown in Figure C11.8-4. These
35 modifications are discussed in detail in papers by Cetin et al. (2004), Idriss and Boulanger (2004, 2006,
36 2008), and Moss et al. (2006); each set of recommended revisions resulted after detailed study and
37 supplementation of the databases of case histories upon which the original relationships were developed.

38 Another important observation that has been made over the past 10 years involves the fines criteria used
39 to judge whether or not a soil is liquefiable. Originally, the “Chinese Criteria” was accepted as the
40 method to determine whether or not a cohesionless soil was liquefiable. However, recent work
41 summarized in Idriss and Boulanger (2008), Boulanger and Idriss (2006), and Bray and Sancio (2006)
42 indicate that the Chinese Criteria will be unconservative in some situations, and alternate methods of
43 assessing whether a soil with cohesive fines will be susceptible to liquefaction or cyclic strength reduction
44 need to be considered. The methods recommended by Boulanger and Idriss (2006) and Bray and Sancio
45 (2006) also establish whether the simplified empirical field methods described previously should be used
46 to estimate liquefaction potential or whether other methods, such as laboratory testing, may be more
47 suitable for evaluating the effects of cyclic loading on soil strength.

48 Methods are also now available for treating the probability of liquefaction, given a certain design ground
49 motion and SPT blowcount. Cetin et al. (2004) and Moss et al. (2006) present a comprehensive treatment

of liquefaction probability. These researchers suggest that following the deterministic approach for estimating liquefaction potential, as discussed above, 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. Kramer and Mayfield (2007) show how the probability of ground shaking can be combined with the probability of liquefaction in a performance-based approach to evaluating liquefaction potential. 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, 2006, 2008), as well as those developed by Cetin et al. (2004) and Moss et al. (2006), 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 these methods.

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, 2006, 2008), Cetin et al. (2004), and Moss et al. (2006) offer an updated alternative to this approach.

1. The first step in the liquefaction hazard evaluation using the empirical SPT approach is 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:

$$CSR = 0.65(a_{max}/g)(\sigma_0/\sigma'_0)r_d \quad (C11.8-1)$$

Where:

(a_{max}/g) = geomean 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:

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

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

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

It should be noted that because nearly all the field data used to develop the simplified procedure are for depths less than 50 ft, there is greater uncertainty in the use of empirical approaches at greater depths. Common practice is to use the SPT or CPT method to depths of 75 ft. 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 75 ft. For these sites, it is important to correct the CRR with an overburden correction factor (K_σ). Alternatively, 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.

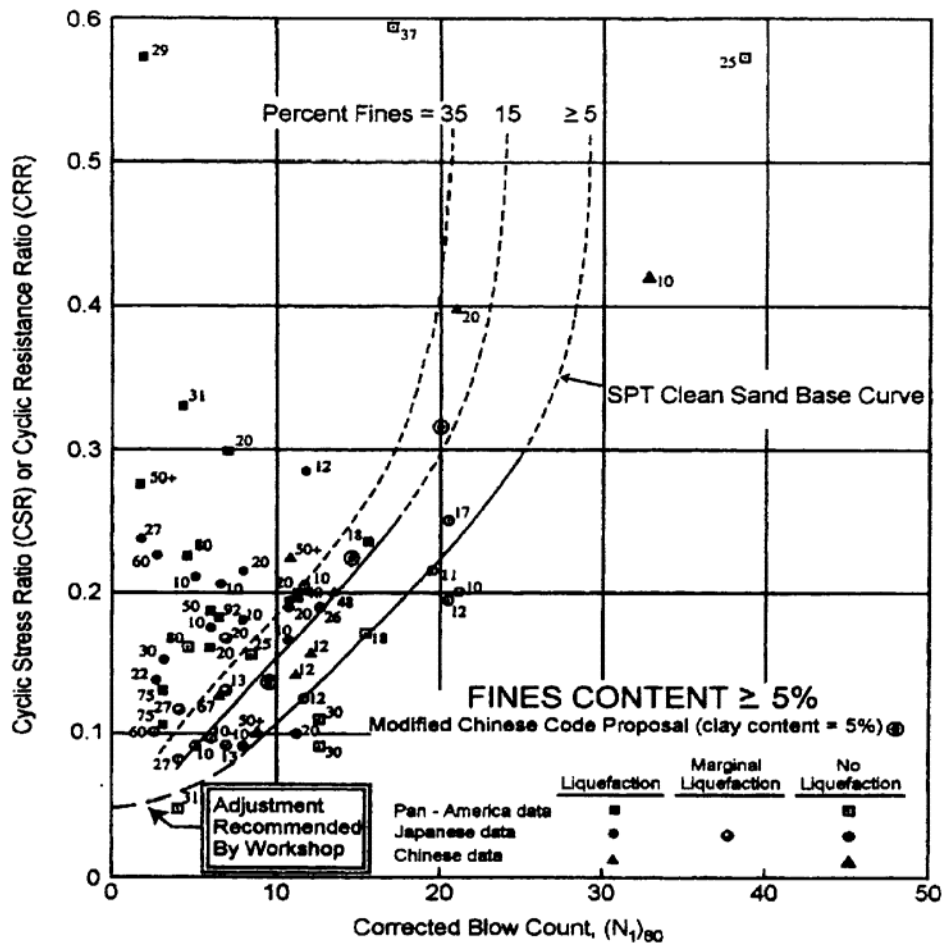


Figure C11.8-4. SPT clean sand base curve for magnitude 7.5 earthquakes with data from liquefaction case histories. (Modified from Seed et al., 1985). (NCEER, 1997; Youd et al., 2001).

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.

1 Solid symbols represent sites where liquefaction occurred and open symbols represent sites where
2 surface evidence of liquefaction was not found. Curves were drawn through the data to separate
3 regions where liquefaction did and did not develop. As shown, curves are given for soils with fines
4 contents (FC) ranging from less than 5 to 35 percent.

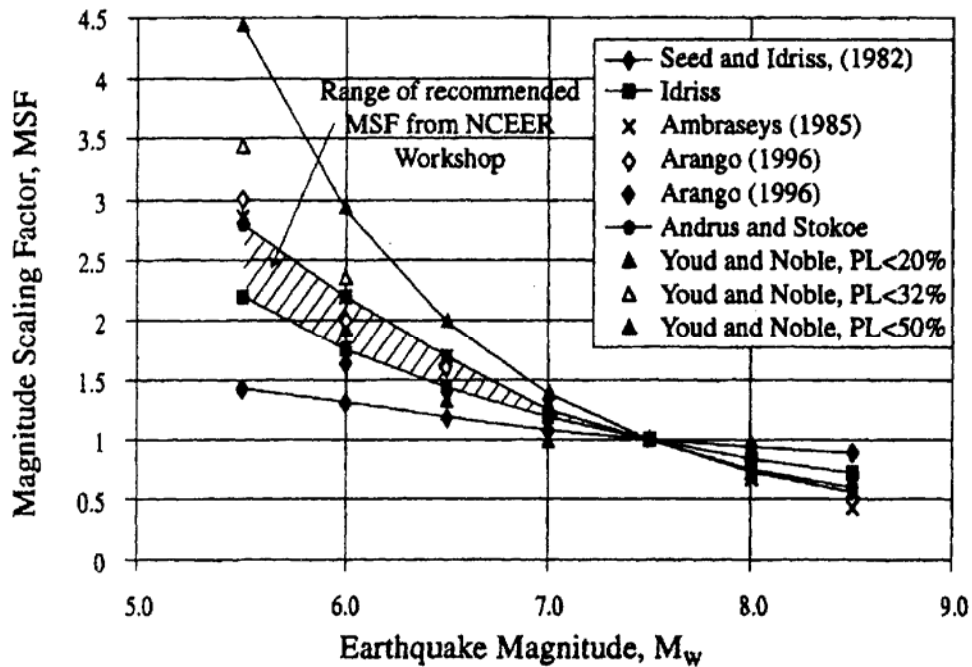
5 The $(N_I)_{60}$ in Figure C11.8-4 is adjusted for various factors before its use, as recommended by the
6 Liquefaction Workshop and discussed by Youd et al. (2001). These include an adjustment for fines,
7 such that only the clean sand curve in Figure C11.8-4 is used, as well as adjustments for a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

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

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

23 Before computing the factor of safety from liquefaction, the CRR result obtained from Figure C11.8-
24 4 (using the corrected SPT blow count identified in the equation for $(N_I)_{60}$) must be corrected for
25 earthquake magnitude M if the magnitude differs from 7.5. The magnitude correction factor is shown
26 in Figure C11.8-5. This plot was developed during the Liquefaction Workshop on the basis of input
27 from experts attending the workshop. The range shown in Figure C11.8-5 is used because of
28 uncertainties. Research conducted since the Liquefaction Workshop has shown that magnitude
29 scaling factors near the lower limit of the recommended range are appropriate for $M < 7.5$ (Liu et al.,
30 2001; Cetin et al., 2004; Boulanger and Idriss, 2006).

31



1
2 **Figure C11.8-5 Magnitude scaling factors derived by various investigators. (NCEER, 1997;**
3 **Youd et al., 2001).**

4 The magnitude, M , needed to determine a magnitude scaling factor from Figure C11.8-5 should
5 correspond to the Maximum Considered Earthquake (MCE). Where the general procedure for ground
6 motion estimation is used (Sections 11.4.1 - 11.4-6) and the MCE is determined probabilistically, the
7 magnitude used in these evaluations can be obtained as the dominant magnitude(s) determined from
8 deaggregation information available by latitude and longitude from the USGS website
9 (<http://earthquake.usgs.gov/research/hazmaps/>). Where the general procedure is used and the MCE is
10 bounded deterministically near known active fault sources, the magnitude of the MCE should be the
11 characteristic maximum magnitude assigned to the fault in the construction of the MCE ground
12 motion maps. Where the site-specific procedure for ground motion estimation is used (Sections
13 11.4.7 and Chapter 21), the magnitude of the MCE should be similarly determined from the site-
14 specific analysis. In all cases, it should be remembered that the likelihood of liquefaction at the site
15 (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
16 by a_{max} alone. Because of the longer duration of strong ground-shaking, large distant earthquakes
17 may in some cases generate liquefaction at a site while smaller nearby earthquakes may not generate
18 liquefaction even though a_{max} of the nearer events is larger than that from the more distant events.

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

21
$$F_L = CRR/CSR \tag{C11.8-3}$$

22 If F_L is greater than 1.0, then liquefaction should not develop. If at any depth in the sediment profile,
23 F_L is equal to or less than 1.0, then there is a liquefaction hazard. Although the curves shown in
24 Figure C11.8-4 envelop the plotted data, it is possible that liquefaction may have occurred beyond the
25 enveloped data and was not detected at ground surface. For this reason a factor of safety of 1.1 to 1.3
26 is usually appropriate for building sites – with the actual factor selected on the basis of the importance
27 of the structure and the potential for ground displacement at the site.

28 Additional guidance on the selection of the appropriate factor of safety is provided by Martin and
29 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 blowcounts 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	$(N_1)_{60}cs$	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 Loss of Ground Support, Increased Loads, and Ground Displacements.

Liquefaction by itself may or may not be of engineering significance. Only when liquefaction is accompanied by loss of ground support, increased soil loads, and/or ground deformation does this phenomenon become important to structural design. Surface manifestations, loss of bearing capacity,

1 increased lateral earth pressures, ground settlement, flow failure, and lateral spread are ground failure
2 mechanisms that have caused structural damage during past earthquakes. These types of ground failure
3 are described in Martin and Lew (1999), U.S. Army Corps of Engineers (2005), and National Research
4 Council (1985) and are discussed below. The type of failure and amount of ground displacement are a
5 function of several parameters including the looseness of the liquefied soil layer, the thickness and extent
6 of the liquefied layer, the thickness and permeability of unliquefied material overlying the liquefied layer,
7 the ground slope, and the nearness of a free face.

8 *Surface Manifestations.* Surface manifestations refer to sand boils and ground fissures on level ground
9 sites. For structures supported on shallow foundations, the effects of surface manifestations on the
10 structure could be tilting or cracking. Criteria are given by Ishihara (1985) and Youd and Garris (1995)
11 for evaluating the influence of thickness of layers on surface manifestation of liquefaction effects for level
12 sites. These criteria may be used for noncritical or nonessential structures on level sites not subject to
13 lateral spreads (see later in this section). Additional analysis should be performed for critical or essential
14 structures.

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

23 *Increased lateral earth pressure.* Another possible consequence of liquefaction is increased lateral
24 pressures against basement and retaining walls. A common procedure used in design for such increased
25 pressures is to assume that the liquefied material acts as a dense fluid having a unit weight of the liquefied
26 soil. The wall then is designed assuming that hydrostatic pressure for the dense fluid acts against the
27 wall. If unsaturated soil is present above the liquefied soil, it is treated as a surcharge that increases the
28 fluid pressure within the underlying liquefied soil by an amount equal to the thickness times the total unit
29 weight of the surcharge soil. The procedure applies equivalent horizontal earth pressures that are greater
30 than typical at-rest earth pressures but less than passive earth pressures. As a final consideration, to
31 prevent buoyant rise of a structure as a consequence of liquefaction, the total weight of the structure
32 should be greater than the volume of the basement or other cavity times the unit weight of liquefied soil.
33 (Note that structures with insufficient weight to counterbalance buoyant effects could differentially rise
34 during an earthquake.)

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

39 Tokimatsu and Seed (1987) published an empirical procedure for estimating ground settlement. It is
40 beyond the scope of this discussion to outline that procedure which, although explicit, has several rather
41 complex steps. The Tokimatsu and Seed procedure can be applied whether liquefaction does or does not
42 occur. For dry cohesionless soils, the settlement estimate from Tokimatsu and Seed should be multiplied
43 by a factor of 2 to account for multi-directional shaking effects as discussed by Martin and Lew (1999).
44 Duku et al. (2008) present updated relationships for the calculation of settlement of unsaturated clean
45 sands. An alternate approach for settlement of liquefiable soils is that proposed by Ishihara and
46 Yoshimine (1992).

47 *Flow failures.* Flow failures or flow slides are the most catastrophic form of ground failure that may be
48 triggered when liquefaction occurs. They may displace large masses of soils tens of feet. Flow slides

1 occur when the average static shearing stresses on potential failure surfaces are less than the average shear
2 strengths of liquefied soil on these surfaces. Standard limit equilibrium static slope stability analyses may
3 be used to assess flow failure potential with the residual strength of liquefied soil used as the strength
4 parameter in the analyses.

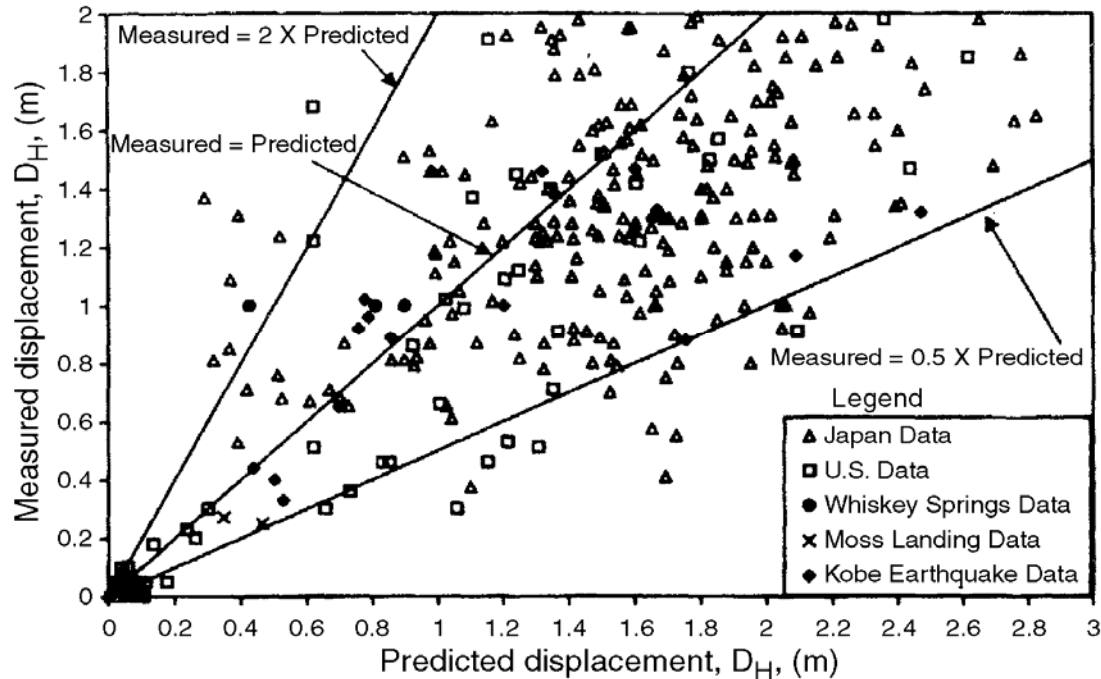
5 The determination of residual strengths is very inexact, and consensus as to the most appropriate approach
6 has not been reached to date. Relationships for residual strength of liquefied soil that are often used in
7 practice are those of Seed and Harder (1990), Olson and Stark (2002), and Idriss and Boulanger (2007).
8 These strengths have been empirically determined from back analyses of flow failures.

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

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

- 21 1. Empirical procedures use correlations between past ground displacement and site conditions under
22 which those displacements occurred. Youd et al. (2002) present an empirical method that provides an
23 estimate of lateral spread displacements as a function of earthquake magnitude, distance, topographic
24 conditions, and soil deposit characteristics. As shown in Figure C11.8-6, the displacements estimated
25 by the Youd et al. (2002) method are generally within a factor of two of the observed displacements.
26 Bardet et al. (2002) present an empirical method having a formulation similar to that of Youd et al.
27 (2002) but using fewer parameters to describe the soil deposit. The Bardet et al. (2002) model was
28 developed to assess lateral spread displacements at a regional scale rather than for site-specific
29 applications. Various other empirical methods are also available, including an alternate SPT method
30 by Rausch and Martin (2000) and both SPT and CPT-based methods by Zhang et al. (2004). These
31 methods can result in large differences in predicted displacement and therefore, it is usually best to
32 use several methods when estimating displacement. Because of the uncertainty in results, these
33 methods are normally used for preliminary screening or comparative evaluations.
- 34 2. Simplified analytical techniques generally apply some form of Newmark’s analysis of a rigid body
35 sliding on an infinite or circular failure surface with ultimate shear resistance estimated from the
36 strength of the liquefied soil. Additional discussion of the simplified Newmark method is provided in
37 the discussion of slope instability hazard. A key question for this approach is the method of defining
38 the strength of the liquefied soil. The same residual strength as used for flow failure assessments has
39 often been used for the spreading analyses. However, many researchers will argue that lateral spreads
40 do not involve the same boundary conditions as occur for lateral flows, and specifically that the
41 ratcheting mechanism of loading with dilation at larger strains is not properly considered. No
42 consensus currently exists on the most suitable method for obtaining the liquefied strength for lateral
43 spreading analyses, though the use of the residual strength from flow failures is thought to be
44 conservative for most lateral spreading analyses. Work by Olson and Johnson (2008) appear to
45 support the acceptability of use of the residual strength. In view of the current uncertainties, a
46 cautious approach must be taken when estimating deformations for cases involving liquefaction.
- 47 3. More rigorous computer modeling typically involves use of nonlinear finite element or finite
48 difference methods to predict deformations, such as with the computer codes FLAC and PLAXIS. As

1 noted previously, the accuracy of this approach is critically dependent on the properties and geometry
 2 of the model, as well as the earthquake record selection. Of particular importance for the liquefaction
 3 problem is the completeness of the pore pressure model, and its ability to handle various soil
 4 conditions. For example, the soil model within the nonlinear computer codes is often calibrated for
 5 only specific conditions. If the site is not characterized by these conditions, errors in estimating the
 6 displacement by a factor of two or more can easily occur.



7
 8 **Figure C11.8-6 Measured versus predicted displacements for displacements up to 2 meters. (Youd**
 9 **et al., 2002).**

10
 11 **Mitigation of Liquefaction Hazard.** Three general measures might be considered for mitigation of
 12 liquefaction hazards: (1) design the structure to resist the hazard, (2) stabilize the site to reduce the
 13 hazard, or (3) choose an alternative site. Structural measures that are used to reduce the hazard include
 14 deep foundations, mat foundations, or footings interconnected with ties. Deep foundations have
 15 performed well at level sites of liquefaction where effects were limited to ground settlement and ground
 16 oscillation with no more than a few inches of lateral displacement. Deep foundations, such as piles, must
 17 consider the potential for reduced soil support through the liquefied layer and may be subjected to lateral
 18 displacements across the layer. Well reinforced mat foundations also have performed well at localities
 19 where ground displacements were less than 1 foot, although re-leveling of the structure has been required
 20 in some instances (Youd, 1989). Strong ties between footings also should provide increased resistance to
 21 damage where differential ground displacements are less than a foot.

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

1 the superstructure.

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

8 **Surface Fault Rupture Hazard**

9 Fault ruptures during past earthquakes have led to large surface displacements that are potentially
10 destructive to engineered construction. Displacements, which range from a fraction of an inch to tens of
11 feet, may occur along traces of active faults. The sense of displacement ranges from horizontal strike-slip
12 to vertical dip-slip to many combinations of these components. The following paragraphs summarize
13 procedures to follow or consider when assessing the hazard of surface fault rupture. Sources of detailed
14 information for evaluating the hazard of surface fault rupture include Slemmons and dePolo (1986), the
15 Utah Section of the Association of Engineering Geologists (1987), Swan et al. (1991), Hart and Bryant
16 (1997), Hanson et al. (1999), and California Geological Survey (2002). Other beneficial references are
17 given in the bibliographies of these publications.

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

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

32 *Suggested approach for assessing surface faulting hazard.* The following approach should be used, or at
33 least considered, in fault hazard assessment. Some of the investigative methods outlined below should be
34 carried out beyond the site being investigated.

- 35 1. A review should be made of the published and unpublished geologic literature from the region along
36 with records concerning geologic units, faults, ground-water barriers, etc.
- 37 2. A stereoscopic study of aerial photographs and other remotely sensed images should be made to
38 detect fault-related topography/geomorphic features, vegetation and soil contrasts, and other
39 lineaments of possible fault origin. The study of predevelopment aerial photographs is often essential
40 to the detection of fault features. Recently, the use of LiDAR (Light Detection And Ranging) has
41 been found to provide excellent identification of fault traces in areas where tree growth and
42 vegetation would normally obscure evidence of faulting from the air.
- 43 3. A field reconnaissance study generally is required and should include observation and mapping of
44 features such as bedrock and soil units and structures, geomorphic surfaces, fault-related geomorphic
45 features, springs, and deformation of man-made structures due to fault creep. Field study should be
46 detailed within the site with less detailed reconnaissance of an area within a mile or so of the site.
47 Evidence from prehistoric liquefaction (paleoliquefaction) can also provide important information

1 regarding the magnitude and timing of fault movement in the site area or region.

- 2 4. Subsurface investigations may be necessary to evaluate location and activity of fault traces, where
3 uncertainty exists about the location or activity of a fault. These investigations may include trenches,
4 test pits, and/or boreholes to permit detailed and direct observation of geologic units and faults.
- 5 5. The geometry of faults may be further defined by geophysical investigations including seismic
6 refraction, seismic reflection, gravity, magnetic intensity, resistivity, ground penetrating radar, etc.
7 These indirect methods require knowledge of specific geologic conditions for reliable interpretation.
8 Geophysical methods alone never prove the absence of a fault and they typically do not identify the
9 recency of activity.
- 10 6. More sophisticated and more costly studies may provide valuable data where geological special
11 conditions exist or where requirements for critical structures demand a more intensive investigation.
12 These methods might involve repeated geodetic surveys, strain measurements, or monitoring of
13 microseismicity and radiometric analysis (C^{14} , K-Ar), stratigraphic correlation (fossils, mineralogy)
14 soil profile development, paleomagnetism (magnetostratigraphy), or other dating techniques
15 (thermoluminescence, cosmogenic isotopes) to evaluate the age of faulted or unfaulted units or
16 surfaces.

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

- 19 1. Maps should be prepared showing the existence (or absence) and location of active faults on or near
20 the site. The distribution of primary and secondary faulting (fault zone width) and fault-related
21 surface deformation should be shown.
- 22 2. The type, amount, and sense of displacement of past surface faulting episodes should be documented,
23 if possible.
- 24 3. From this documentation, estimates of location, magnitude, and likelihood or relative potential for
25 future fault displacement can be made, preferably from measurements of past surface faulting events
26 at the site, using the premise that the general pattern of past activity will repeat in the future.
27 Estimates also may be made from published empirical correlations between fault displacement and
28 fault length or earthquake magnitude (e.g., Wells and Coppersmith, 1994). Where fault segment
29 length and sense of displacement are defined, these correlations may provide an estimate of future
30 fault displacement (either the maximum or the average to be expected). Probabilistic studies may be
31 considered to evaluate the probability of fault displacement (e.g., Youngs et al., 2003).
- 32 4. The degree of confidence and limitations of the data should be addressed.

33 Both deterministic and probabilistic methods are available for estimating the amount or probability of
34 future fault displacement (e.g., Youngs et al., 2003). Because techniques for making these estimates are
35 not standardized, peer review of reports is useful to verify the adequacy of the methods used and the
36 estimated amount or frequency of movement, to aid the evaluation by the permitting agency, and to
37 facilitate discussion between specialists that could lead to the development of standards.

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

- 40 1. Where ordinances have been developed that specify safe setback distances from traces of active faults
41 or active fault zones, those distances must be complied with and accepted as the minimum for safe
42 siting of buildings.
 - 43 • Provisions Section 11.8 precludes structures in Seismic Design Category E or F from being sited
44 where there is a known potential for an active fault to cause rupture of the ground surface at the
45 structure.

- States may also adopt more definitive requirements. 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 safely 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 smaller 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 or even feet 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. Deep foundations such as driven piles or drilled shafts are not preferred. In general, less damage has been inflicted by compressional or shear displacement than by vertical or extensional displacements.

SEISMIC LATERAL EARTH PRESSURES

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 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 are 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 earth pressures on retaining walls are 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 often considered to be nonyielding.

Yielding Walls

Limit Equilibrium Force Approach. 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 a limit-equilibrium approach based on a Coulomb failure wedge with the assumption that the wall displaces or rotates outward sufficiently to produce the minimum active earth pressure state.

The M-O formulation is expressed as:

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

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).

The value of a_{max} used in the K_{AE} determination is for the design earthquake ground motion (that is, 2/3rd of the geomean MCE ground motion), as required by Section 11.8.3. The instantaneous peak acceleration (that is, a_{max} for the design earthquake ground motion) and not an average of the ground motion is used in this determination. In the past it has been common practice for geotechnical engineers to reduce the instantaneous peak by a factor from 0.5 to 0.7 to represent an average seismic coefficient for determining the seismic earth pressure on a wall. The reduction factor was introduced in a manner similar to the method used in a simplified liquefaction analyses to convert a random acceleration record to an equivalent average series of cyclic loads. This approach can result in confusion on the magnitude of the seismic active earth pressure and, therefore, is not recommended. The use of the design earthquake rather than the MCE in the determination of a_{max} already considers that a reserve of 1.5 exists within the structural design. Any further reduction to represent average rather than instantaneous peak loads is a structural decision and must be an informed decision made by the structural designer. As discussed within the section **Displacement-based Approach**, a reduction in a_{max} is, however, permitted if the wall can undergo permanent displacements.

The M-O equation makes several other very important assumptions, including that the soil behind the retaining wall is a uniform, cohesionless soil and that the groundwater elevation is below the base of the retaining wall. The implications of these assumptions are discussed later in this section.

Simplified M-O 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}):

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

1 or

$$2 \quad K_{AE} = K_A + \Delta K_{AE} \quad (C11.8-5b)$$

3 or

$$4 \quad \Delta P_{AE} = (1/2)\gamma H^2 \Delta K_{AE} \quad (C11.8-5c)$$

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

$$7 \quad \Delta K_{AE} = (3/4)K_h \quad (C11.8-6)$$

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

9 Where:

10 k_h is horizontal ground acceleration divided by gravitational acceleration. Unless permanent displacement
11 of the wall is acceptable, k_h should be taken equal to the site peak ground acceleration, a_{max} , that is
12 consistent with design earthquake ground motions. For the distribution of the dynamic thrust, ΔP_{AE} , Seed
13 and Whitman (1970) recommended that the resultant dynamic thrust act at $0.6H$ above the base of the
14 wall (that is, inverted trapezoidal pressure distribution). Note that this approach assumes dry,
15 cohesionless backfill material. If soil conditions behind the wall have a cohesive soil component (that is,
16 a $c-\phi$ soil), this simplified approach is no longer appropriate. Additional discussion of this issue is
17 included below in the subsection *Limitations of M-O approach*.

18 Equation 11.8-7 generally is referred to as the simplified M-O formulation and is not applicable for
19 sloping ground above the wall. For walls that are in excess of 15 ft in height, special studies can also be
20 conducted to evaluate the coherency of ground motions behind the wall from which an average seismic
21 coefficient can be developed. These special studies require consideration of the frequency characteristics
22 of ground motion, as well as the stiffness of the soil and the wall height, and usually require use of a finite
23 element or difference computer model.

24 Since its introduction, there has been a consensus in geotechnical engineering practice that the simplified
25 M-O formulation reasonably represents the dynamic (seismic) lateral earth pressure increment for
26 yielding retaining walls. However, there are limitations associated with the M-O approach, and these
27 limitations can have a significant effect on the magnitude of estimated seismic active earth pressure.

28 *Limitations of M-O approach.* Although the M-O approach is simple to use, certain designs become very
29 difficult to solve with the standard M-O equations. These designs involve high ground accelerations,
30 combinations of moderate-to-high ground accelerations and steep backslopes, and where mixed backfill
31 conditions exist (that is, either where $c-\phi$ soils occur or where only a thin zone of granular backfill is
32 placed between the wall and a cohesive or rock condition). For these cases the M-O approach does not
33 provide realistic answers.

34 An acceptable alternative approach for these cases is to use a generalized limit equilibrium (slope
35 stability) computer program. With this alternate approach appropriate soil properties and geometry can
36 be modeled, and the seismic coefficient can be defined on the basis of the peak ground acceleration or a
37 reduced seismic coefficient if displacement is acceptable. For most seismic loading cases the total stress
38 (undrained) $c-\phi$ parameters will be appropriate for design because of the rate of seismic loading. With
39 this generalized limit equilibrium method, the external force required for stability is computed. This force
40 represents the dynamic earth pressure on the wall. The total force can be distributed as a uniform seismic
41 pressure or the seismic increment can be determined and applied as an inverted triangle. Note that when
42 subtracting the static force from the total seismic earth pressure, it is necessary to determine the static
43 earth pressure under the same conditions as used during the pseudo-static seismic analysis. This could
44 mean determining the static earth pressure for the same $c-\phi$ combination as used for the seismic analysis.

1 **Displacement-based Approach.** The alternate approach for the design of yielding walls is to evaluate
2 the movement of the wall during seismic loading. Various methods are available for conducting
3 displacement-based analyses – ranging from extensions of the M-O formulations to two- and three-
4 dimensional computer modeling. One of the key requirements for the displacement-based approach is the
5 determination of the level of acceptable displacements. This determination will depend both on the wall
6 type and on the nature of facilities next to the wall. These nearby facilities can range from buildings to
7 buried utilities.

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

14 *Simplified M-O Approach.* Richards and Elms (1979) introduced a method for seismic design analysis of
15 yielding walls considering translational sliding as a failure mode and based on tolerable permanent
16 displacements for the wall. Elms and Martin (1979) showed that the $k_h = a_{max}/2$ is adequate for design if
17 the wall is allowed to slide up to $10a_{max}$, where a_{max} is the design ground motion and displacement is in
18 “inches.” For seismically active areas, the displacement associated with $10 a_{max}$ can be 4 to 6 inches.

19 In practice $k_h = a_{max}/2$ is often used without regard for the displacement that is associated with this
20 assumption. Clearly, several inches of movement can be tolerable for some types of yielding walls, but
21 not all. For example, a semi-gravity cantilever wall could be designed to slide several inches; however,
22 the anchors for a tieback wall would likely restrict this level of movement from occurring for a well
23 designed anchored wall. Use of $k_h = a_{max}/2$ requires that the designer check to confirm that deformations
24 will develop without damaging the wall or other nearby facilities. Various factors can limit
25 displacements, such as physical obstructions or underestimating the amount of soil strength that will be
26 mobilized. If movement cannot be tolerated or if the wall may not move enough to mobilize the yield
27 condition, then either the full a_{max} should be used for determination of seismic earth pressure or more
28 rigorous procedures such as described below should be used.

29 *Other simplified displacement methods.* There are a number of other empirical formulations for
30 estimating permanent displacements under a translation mode of failure; these have been reviewed by
31 Whitman and Liao (1985). Nadim (1980) and Nadim and Whitman (1984) incorporated the failure mode
32 of wall tilting as well as sliding by employing coupled equations of motion, which were further
33 formulated by Siddharthan et al. (1992) as a design method to predict the seismic performance of
34 retaining walls taking into account both sliding and tilting. Alternatively, Prakash et al. (1995) described
35 design procedures and presented design charts for estimating both sliding and rocking displacements of
36 rigid retaining walls. These design charts are the results of analyses for which the backfill and foundation
37 soils were modeled as nonlinear viscoelastic materials. A simplified method that considers rocking of a
38 wall on a rigid foundation about the toe was described by Steedman and Zeng (1996) and allows the
39 determination of the threshold acceleration beyond which the wall will rotate. A simplified procedure for
40 evaluating the critical threshold accelerations for sliding and tilting was described by Richards et al.
41 (1996).

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

47 *Computer Modeling.* An alternative displacement-based approach is the use of two-dimensional
48 computer codes such as FLAC and PLAXIS. These methods allow a more detailed evaluation of soil-

1 structure interaction for different wall geometries and external loads, different soil and structural
 2 properties, and different earthquake loading conditions. Results from these analyses can be particularly
 3 helpful in understanding the deformations that occur within and near the retaining wall, including the soil
 4 in front of and behind the wall. As noted previously, these methods require considerable expertise in
 5 terms of soil and structural modeling and selection of earthquake records and should be used with
 6 particular care. While results may appear very reasonable, small changes in model setup or input
 7 parameters selection can significantly affect the quality of results, potentially leading to unconservative
 8 design decisions.

9 **Nonyielding Walls**

10 By definition nonyielding walls do not deform when subjected to seismic earth pressures. This type of
 11 response requires a very stiff wall in combination with a rigid base condition. Most nonyielding walls
 12 will be located on rock or very stiff soil. Even in this condition, wall flexibility can be sufficient to
 13 develop active seismic earth pressures, significantly reducing the loading on basement walls. Where the
 14 basement wall is located on rock or very stiff soil and where structural analyses determine that the wall
 15 flexibility is such that deformations will not develop seismic active earth pressures (that is, deformations
 16 $< 0.002H$ where H is the wall height), the wall should be designed as a nonyielding wall. The following
 17 discussions provide guidance on two methods for dealing with cases where rigid wall conditions occur.
 18 Also included is a discussion of soil-structure interaction methods for evaluating earth pressures on
 19 basement walls where uncertainties on the flexibility of the wall occur.

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

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

$$27 \quad \Delta P_E = Fk_h\gamma H^2 \quad (C11.8-8)$$

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

$$30 \quad \Delta P_E = k_h\gamma H^2 \quad (C11.8-9)$$

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

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

37 Although the study performed by Wood included dynamic analysis of a rigid wall with fixed base
 38 condition, the solution commonly used and presented in Equations C11.8-8 and C11.8-9 are based on
 39 static “1 g” loading of the soil and wall and does not include the effects of the wave propagation in the
 40 soil. The subject of soil-wall interaction is addressed in the following sections.

41 **Ostadan Rigid Wall Approach.** Ostadan (2005) observed that for partially embedded structures
 42 subjected to ground shaking, the characteristics of the dynamic earth pressure amplitudes versus
 43 frequency of the ground motion were those of a single-degree-of-freedom (SDOF) system and proposed a
 44 simplified method to estimate the magnitude and distribution of dynamic thrust. Results provided by
 45 Ostadan (2005) utilizing this simplified method, which were also confirmed by dynamic finite element

1 analyses, indicate that, depending on the dynamic properties of the backfill as well as the frequency
 2 characteristics of the input ground motion, a range of dynamic earth pressure solutions would be obtained
 3 for which the M-O solution and the Wood (1973) solution represent a “lower” and an “upper” bound,
 4 respectively.

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

- 10 | 1. Perform free-field soil column analysis and obtain the ground response motion at the depth
 11 | corresponding to the base of the wall in the free-field. The response motion in terms of
 12 | acceleration response spectrum at 30 percent damping should be obtained. The free-field soil
 13 | column analysis may be performed using the computer program SHAKE with input motion
 14 | specified either at the ground surface or at the depth of the foundation basemat. The choice for
 15 | location of control motion should be consistent with the development of the design motion.
- 16 | 2. Use Eq. (C11.8-10) to compute the total soil mass (m) using the Poisson’s ratio (ν) and mass
 17 | density of the soil.

$$18 \quad m = 0.50 (\rho) H^2 (\Psi\nu) \quad (C11.8-10)$$

21 Where:

22
 23 ρ is the mass density of the soil (total weight density divided by acceleration of gravity), H is the
 24 height of the wall, and $\Psi\nu$ is a factor to account for the Poisson’s ratio as defined by the
 25 following equation.

$$26 \quad \Psi\nu = 2 / [(1-\nu)(2-\nu)]^{0.5} \quad (C11.8-11)$$

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

$$33 \quad f_s = v_s / (4H) \quad (C11.8-12)$$

36 | Where:

37
 38 | v_s is the average strain-compatible shear wave velocity of the soil column over the height of the
 39 | wall.

- 41 | 4. Obtain the maximum lateral seismic soil pressure at the ground surface level by dividing the
 42 | lateral force obtained in Step 3 by the area under the normalized seismic soil pressure, $0.744 H$.
- 43
 44 | 5. Obtain the pressure profile by multiplying the peak pressure from Step 4 by the pressure
 45 | distribution relationship given by Equation (C11.8-13 below).

$$46 \quad p(y) = -0.0015 + 5.05y - 15.84y^2 + 28.25y^3 - 24.59y^4 + 8.14y^5 \quad (C11.8-13)$$

48
 49 Where:

1
2 y is the normalized height ratio (Y/H) measured from the bottom of the wall (ranging from 0 at the
3 bottom of the wall to 1 at the top of the wall), and Y is the distance of the point under
4 consideration from the bottom of the wall.

5
6 The area under the seismic soil pressure curve can be obtained from integration of the pressure
7 distribution over the height of the wall. The total area is $0.744H \times p_{\max}$ for a wall with the height of H
8 and maximum pressure of p_{\max} at the top.

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

13 **Soil-structure-interaction Approach and Modeling for Partially Embedded Structures.** Lam and
14 Martin (1986), Soydemir and Celebi (1992), Veletsos and Younan (1994a and 1994b), and Ostadan
15 (2005), among others, argue that the earth pressures acting on the walls of partially embedded structures
16 (e.g., basement walls) during earthquakes are primarily governed by soil-structure interaction (SSI) and,
17 thus, should not be treated as a nonyielding wall. Soil-structure interaction includes both a kinematic
18 component—the interaction of a massless rigid wall with the adjacent soil as modeled by Wood (1973)
19 but including the wave propagation in the soil—and an inertial component—the interaction of the wall,
20 connected to a responding superstructure, with the adjacent soil. Detailed SSI analyses incorporating
21 kinematic and inertial interaction may be considered for the estimation of seismic earth pressures on
22 critical walls.

23 Whitman (1991) has suggested that SSI effects on basement walls of buildings reduce dynamic earth
24 pressures and that M-O pressures may be used in design except where structures are founded on rock or
25 hard soil (that is, where no significant rocking occurs). In the latter case, the pressures given by the
26 Ostadan (2005) method with the Wood (1973) formulation as the upper bound would appear to be more
27 applicable. The effect of rocking in reducing the dynamic earth pressures on basement walls also has
28 been suggested by Ostadan and White (1998). This condition may be explained if it is demonstrated that
29 the dynamic displacements induced by kinematic and inertial components are out of phase. Chang et al.
30 (1990) have found that dynamic earth pressures recorded on the wall of a large-scale model nuclear
31 reactor containment building (e.g., 1/4 the size of a full-size power block) were consistent with dynamic
32 pressures predicted by the M-O solution. Analyses by Chang et al. indicate that the dynamic wall
33 pressures were strongly correlated with the rocking response of the structure.

34 **Effect of Saturated Backfill on Wall Pressures.**

35 The previous discussions on yielding and nonyielding walls are limited to backfills that are not water-
36 saturated. In current (2008) practice, drains typically are incorporated in the design to prevent
37 groundwater from building up within the backfill. This is not practical or feasible, however, for
38 waterfront structures (such as quay walls) where most of the earthquake-induced failures have been
39 reported (Seed and Whitman, 1970; Ebeling and Morrison, 1992; ASCE-TCLEE, 1998).

40 During ground shaking, the presence of water in the pores of a backfill can influence the seismic loads
41 that act on the wall in three ways (Ebeling and Morrison, 1992; Kramer, 1996): (1) by altering the
42 inertial forces within the backfill, (2) by developing hydrodynamic pressures within the backfill and (3)
43 by generating excess porewater pressure due to cyclic straining. Effects of the presence of water in
44 cohesionless soil backfill on seismic wall pressures can be estimated using formulations presented by
45 Ebeling and Morrison (1992) and Kramer (1996). The effects of soil liquefaction associated with excess
46 porewater pressure generation on wall pressures is discussed in the subsection entitled *Increased lateral*
47 *earth pressure* within the **Liquefaction Hazard** section.

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