5ICEGE
5th International Conference on Earthquake Geotechnical Engineering
January 2011, 10-13 | Santiago, Chile
Paper No. Theme Lecture 1

PSEUDOSTATIC SLOPE STABILITY PROCEDURE

Jonathan D. BRAY [1] **and Thaleia TRAVASAROU**[2]

ABSTRACT

Pseudostatic slope stability procedures can be employed in a straightforward manner, and thus, their use in engineering practice is appealing. The magnitude of the seismic coefficient that is applied to the potential sliding mass to represent the destabilizing effect of the earthquake shaking is a critical component of the procedure. It is often selected based on precedence, regulatory design guidance, and engineering judgment. However, the selection of the design value of the seismic coefficient employed in pseudostatic slope stability analysis should be based on the seismic hazard and the amount of seismic displacement that constitutes satisfactory performance for the project. The seismic coefficient should have a rational basis that depends on the seismic hazard and the allowable amount of calculated seismically induced permanent displacement. The recommended pseudostatic slope stability procedure requires that the engineer develops the project-specific allowable level of seismic displacement. The site-dependent seismic demand is characterized by the 5% damped elastic design spectral acceleration at the degraded period of the potential sliding mass as well as other key parameters. The level of uncertainty in the estimates of the seismic demand and displacement can be handled through the use of different percentile estimates of these values. Thus, the engineer can properly incorporate the amount of seismic displacement judged to be allowable and the seismic hazard at the site in the selection of the seismic coefficient.

Keywords: Dam; Earthquake; Permanent Displacements; Reliability; Seismic Slope Stability

INTRODUCTION

Pseudostatic slope stability procedures are often used in engineering practice to evaluate the seismic performance of earth structures and natural slopes. Although these procedures have the advantage of being relatively easy to use, they cannot offer a reliable assessment of the potential seismic performance of an earth structure or natural slope unless the parameters utilized in the analysis accurately reflect the potential seismic demand and its impact on the seismic performance of the system. The seismic coefficient that is employed in a pseudostatic slope stability analysis should be selected in a rational manner, because its value is critically important.

After discussing the pseudostatic slope stability procedure, a design procedure is recommended that takes advantage of recent work on probabilistic seismic slope displacement procedures. This design procedure calculates a value of the seismic coefficient that is directly a function of the allowable seismic displacement and the seismic hazard at the project site. The manner in which this design approach can be used in engineering practice is also discussed.

[1] Professor, Dept. of Civil & Environ. Engineering, Univ. of California, Berkeley, CA; e-mail: bray@ce.berkeley.edu
[2] Senior Engineer, Fugro Inc., 1000 Broadway, Suite 440, Oakland, CA.

PSEUDOSTATIC SLOPE STABILITY PROCEDURE

In a pseudostatic slope stability analysis, a factor of safety (*FS*) is computed using a static limit equilibrium method in which a horizontal earthquake-induced inertial force is applied to the potential sliding mass. This pseudostatic force represents the destabilizing effects of the earthquake shaking. The horizontal inertial force is expressed as the product of a seismic coefficient (*k*) and the weight (*W*) of the potential sliding mass (see Figure 1). The pseudostatic analysis also requires the use of appropriate material dynamic strengths (*S*). If the calculated *FS* is above a specified minimum value, the earth structure or natural slope is considered to be safe. The limit equilibrium method should satisfy all three conditions of equilibrium to ensure that a reliable *FS* is calculated (Duncan and Wright 2005).

The design seismic coefficient and acceptable minimum factor of safety are often selected based on precedence. Several combinations of seismic coefficient and factor of safety as reported in works of the late Professor H. Bolton Seed and his co-workers are listed in Table 1. Seismic coefficient values often range from 0.1 to 0.2 with factors of safety from 1.0 to 1.5. The basis for these combinations of values is not always clear.

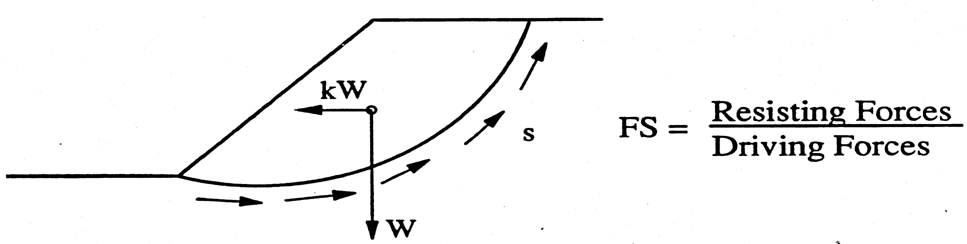

Figure 1. Pseudostatic slope stability analysis

Table 1. Seismic coefficients used previously (e.g., Seed and Martin 1966, Seed et al. 1978, Seed 1979, Seed 1981, Seed 1983)

DAM	COUNTRY	SEISMIC COEFFICIENT	FACTOR OF SAFETY
Aviemore	New Zealand	0.10	1.50
Bersemisnoi	Canada	0.10	1.25
Digma	Chile	0.10	1.15
Globocica	Yugoslavia	0.10	1.00
Karamauri	Turkey	0.10	1.20
Kisenyama	Japan	0.12	1.15
Mica	Canada	0.10	1.25
Netzahualcovote	Mexico	0.15	1.36
Oroville	United States	0.10	1.20
Paloma	Chile	0.12 & 0.20	1.25 & 1.10
Ramganga	India	0.12	1.20
Tercan	Turkey	0.15	1.20
Yeso	Chile	0.12	1.50

POTENTIAL FOR MATERIAL STRENGTH LOSS

Before discussing the pseudostatic slope stability procedure further, it is imperative that the most critical design issue be addressed. In the evaluation of the seismic performance of an earth structure, the engineer must first access if there are materials in the earth structure or its foundation that can lose significant strength as a result of cyclic loading (e.g., soil liquefaction). If so, this should be the primary focus of the evaluation, because large displacement flow slides could result. In fact, Terzaghi (1950) warned:

> *Theoretically a value of FS = 1 would mean a slide but in reality a slope may remain stable in spite of FS being smaller than unity and it may fail at a value of FS > 1, depending on the character of the slope forming materials… The most stable materials are clays with a low degree of sensitivity, in a plastic state, dense sand either above or below the water table, and loose sand above the water table. The most sensitive materials are slightly cemented grain aggregates such as loess and submerged or partly submerged loose sand …*

Thus, the pseudostatic procedure can provide completely misleading results if the cyclic response characteristics of the earth materials in or below the slope are not accessed properly. If pre-seismic static strength values are utilized in the pseudostatic slope stability analysis and these materials lose significant strength as a result of earthquake shaking, the calculated factor of safety is meaningless. The potential for significant strength loss either due to liquefaction of sandy or silty materials or cyclic softening of sensitive clayey soils must be evaluated first. If it is likely that these materials will undergo strength loss as a result of earthquake shaking, their post-shaking reduced shear strength values should be employed in the seismic stability evaluation. If the post-seismic factor of safety approaches or falls below unity, potentially catastrophic flow slides with large displacements are possible.

The soil liquefaction evaluation procedures in Youd et al. (2001) have been primarily used in engineering practice. However, recent studies have identified deficiencies in some of these procedures, and there are several updates now available (e.g., Seed et al. 2003, and Idriss and Boulanger 2008). Importantly, the Chinese criteria should no longer be used to assess the liquefaction susceptibility of fine-grained soils. Instead, other criteria such as those developed by Bray and Sancio (2006), which are based on soil plasticity ($PI \leq 12$) and sensitivity ($w_c/LL \geq 0.85$) should be used. Additionally, soils with $PI > 12$ should be evaluated carefully, as sensitive clay flow slides have occurred, such as those in Anchorage during the 1964 Alaskan earthquake (e.g., Idriss 1985, Olsen 1989, & Stark and Contreras 1998). The potential for flow slides resulting from severe strength loss due to liquefaction of sands and silts or post-peak strength reduction in sensitive clays should be evaluated carefully before employing a limit equilibrium stability procedure that implicitly assumes a rigid-perfectly plastic material response during shear.

PSEUDOSTATIC COEFFICIENT SELECTION

The pseudostatic slope stability method has been shown to provide reasonable results for earth structures and natural slopes composed of and built atop materials that do not undergo severe strength loss as a result of earthquake shaking. Thus, the pseudostatic slope stability procedure is a potentially attractive design tool. However, its use must be calibrated with observations from case histories or with reliable results from more advanced analyses. A properly calibrated pseudostatic slope stability procedure requires a compatible selection of the seismic coefficient, material dynamic strengths, and an acceptable factor of safety. All three of these critical design parameters must be specified and be shown to provide reliable results.

The horizontal force *kW* in the pseudostatic slope stability analysis represents the dynamic effects of the design earthquake. However, unlike the earthquake's actual dynamic effects, this pseudostatic force acts as a constant force in only one direction. In limit equilibrium analysis, the *FS* = 1.0 when the activating forces (or moments) equal the resisting forces (or moments). Hence, a condition of incipient motion is only considered. With dynamically applied earthquake-induced loads, the force may act in one direction for only a few tenths of one second before its direction is reversed. As noted by Newmark (1965), the results of these transient forces will be a series of displacement pulses. Satisfactory performance is determined by whether or not the accumulated seismically induced permanent displacement is tolerable. It is reasonable to select a seismic coefficient that is some fraction of the maximum seismic demand, because exceeding the maximum seismic resistance for a few instances will only lead to minor accumulated seismic displacement. Thus, the proper seismic performance goal when employing a pseudostatic approach is still in terms of the likely seismic displacement (Bray 2007).

The Seed (1979) pseudostatic slope stability method is one of the most commonly used pseudostatic slope stability procedures. It was developed for earth dams with materials that do not undergo severe strength loss that have crest accelerations less than 0.75 *g*. Using *k* = 0.15 with appropriate dynamic strengths for the critical earth materials, performance is judged to be acceptable by this method if *FS* > 1.15. This approach involves several simplifying assumptions and importantly, is calibrated for evaluating earth dams wherein Seed (1979) assumed that a seismic displacement of one meter constituted acceptable performance. Moreover, the characteristics of the earthquake ground motion and the dynamic response of the potential slide mass to the earthquake shaking are represented by *k* = 0.15 for all cases. The value of the seismic coefficient should depend on the source-to-site distance (*R*) and moment magnitude (*M*) of the design earthquake event. For example, the seismic hazard differs greatly for a project site that is 20 km from an active fault that is capable of producing a *M* = 9 event and another project site that is located 50 km from a fault with a relatively slow slip-rate that could produce at most a *M* = 7 event. The design value of *k* should incorporate the specific seismic hazard at a project site.

Additionally, the use of a minimum acceptable factor of safety other than unity is problematic. The yield coefficient (k_y) is calculated as the seismic coefficient that results in a FS = 1.00 in the pseudostatic stability analysis. It represents the dynamic resistance of the earth dam. The use of *FS* > 1.15, as recommended by Seed (1979) for example, ensures that the yield coefficient will be greater than 0.15 by an unknown amount. Thus, the amount of conservatism involved in the Seed (1979) procedure and the expected seismic performance is unknown. In summary, an earth structure that satisfies the Seed (1979) recommended combination of *k*, *FS*, and *S* may displace an unknown amount that is up to 1 m. This procedure does not guarantee that an earth structure will be necessarily safe for other levels of performance. Lastly, the procedure does not capture the anticipated effects of different seismic hazard levels for particular project sites.

The Hynes-Griffin and Franklin (1984) pseudostatic screening procedure is based on a conservative interpretation of the dynamic response of an earth dam to a large number of earthquake ground motions. Based on their assumption that one meter of seismic displacement is acceptable for most earth dams, they recommended the use of *k* = half the peak ground acceleration (*PGA*) on rock at the site, *S* = 80% of the static strength of the materials that are not susceptible to severe strength loss, and $FS \geq 1.0$. If these criteria are met, then the dam is judged to be safe, but again, safe has been defined in terms of 1 m of seismic displacement being tolerable. This method should not be applied to cases where seismically induced permanent displacements of up to 1 m are not acceptable, which is the case for many projects, such as for compacted earth fills for industrial facilities.

There are several methods currently used to determine an appropriate value for the seismic coefficient in pseudostatic slope stability analysis. In the past, engineering judgment and precedence have largely guided the selection of the seismic coefficient. However, with the advent of performance-based earthquake engineering (PBEE) and a greater sensitivity to the spatial distribution of seismic hazard, the use of a prescribed seismic coefficient value across a geographic region just because it has been used previously is not rational. Instead, the seismic coefficient should be selected based on the unique seismic hazard present at a site and the level of seismically induced permanent displacement that constitutes an acceptable level of seismic performance of that particular system.

PBEE SEISMIC SLOPE DISPLACEMENT MODEL

The recommended procedure for selecting the design seismic coefficient is based on the work of Bray and Travasarou (2009), which extended the probabilistic seismic slope displacement model developed by Bray and Travasarou (2007). Their procedure utilizes 688 recorded ground motions from 41 different earthquakes, uses a nonlinear coupled stick-slip sliding model, and rigorously accounts for the uncertainty in the estimate of seismic displacement. The model and its limitations are described fully in Bray and Travasarou (2007).

The Bray and Travasarou (2007) seismic slope displacement model allows the engineer to estimate the potential seismic displacement (*D*) using two equations that calculate: (1) the probability of negligible ("zero") displacement (i.e., $D \leq 1$ cm), and (2) the likely amount of "nonzero" displacement. The first equation can be neglected in this application, because it does not have a noticeable effect on k_y for a median seismic displacement value larger than 5 cm. For smaller displacements, neglecting the first equation yields conservative results (i.e., the true displacement will be less than or equal to the target displacement). The amount of the "nonzero" seismic displacement (*D*) in centimeters is estimated as:

$$\begin{aligned} \ln(D) &= -1.10 - 2.83\ln(k_y) - 0.333\left(\ln(k_y)\right)^2 + 0.566\ln(k_y)\ln(S_a) + 3.04\ln(S_a) \\ &- 0.244(\ln(S_a))^2 + 1.5T_s + 0.278(M-7) \pm \varepsilon \end{aligned} \qquad (1)$$

where k_y is the yield coefficient, T_s is the initial fundamental period of the sliding mass in seconds (in most cases, $T_s = 4H/V_s$, where H = representative height and V_s = average shear wave velocity of the sliding mass), S_a is the 5% damped elastic spectral acceleration of the site's design ground motion at a period of $1.5T_s$ in the unit of g (the site is defined by the earth materials below the sliding mass, and if shallow sliding is being evaluated, topographic amplification may need to be included; Bray 2007), M is the earthquake's moment magnitude, and ε is a normally-distributed random variable with zero mean and standard deviation (σ) of 0.66. To eliminate the bias in the model when $T_s < 0.05$ s, the first term (i.e., the constant of -1.10) of Eq. (1) should be replaced with -0.22.

SELECTION OF THE SEISMIC COEFFICIENT

The Bray and Travasarou (2007) procedure has been reworked by Bray and Travasarou (2009) to provide a rational basis for selecting a seismic coefficient that is based on the expected seismic demand at the project site and the desired level of seismic performance for the earth structure or natural slope being evaluated. Instead of calculating the seismic slope displacement (*D*) as a function of k_y, T_s, S_a, and M, Eq. (1) can be reworked to solve for k_y as a function of *D* and best estimates of the other parameters. If this value of k is

used in a pseudostatic slope stability analysis as the seismic coefficient and the calculated $FS \geq 1.0$, then the selected percentile estimate of the seismic displacement will be less than or equal to the allowable seismic displacement (D_a).
The Bray and Travasarou (2009) site-dependent and allowable displacement-dependent value of the seismic coefficient (k) is:

$$k = \exp\left[\frac{-a+\sqrt{b}}{0.665}\right] \tag{2a}$$

where

$$a = 2.83 - 0.566\ln(S_a) \tag{2b}$$

$$b = a^2 - 1.33\cdot\left[\ln(D_a) + 1.10 - 3.04\ln(S_a) + 0.244(\ln(S_a))^2 - 1.5T_s - 0.278(M-7) - \varepsilon\right] \tag{2c}$$

When $T_s < 0.05$ s (i.e., essentially a rigid block), the term 1.10 of Eq. (2c) should be replaced with 0.22.

The steps required to perform the pseudostatic slope stability analysis with a seismic hazard-dependent and project performance-dependent design seismic coefficient are:

1. Evaluate if there are materials that could lose significant strength as a result of earthquake shaking. If such materials are present, focus on evaluating their cyclic shear strength. Use post-shaking reduced strengths, if appropriate. Generally, use best-estimate dynamic shear strength values for critical soil strata that do not lose significant strength.

2. Develop the allowable seismic displacement (D_a in cm) and the percent exceedance of this displacement threshold (i.e., 50% exceedance level is achieved using $\varepsilon = 0$) through consultation with the owner while considering the consequences of unsatisfactory performance at displacement levels greater than this threshold.

   [As the calculated seismic displacement is merely an index of the system's potential seismic performance and the seismic hazard assessment typically includes some level of conservatism, the median seismic displacement estimate at the 50% exceedance level is often appropriate (i.e., use $\varepsilon = 0$ in Eqn. 2c). If a greater level of confidence is desired, then the 16% exceedance level for the seismic displacement estimate can be used with $\varepsilon = 1\sigma = 0.66$. The 16% displacement level is about half of the median displacement level, and if selected, it would lead to a higher seismic coefficient.]

3. The initial period of the potential sliding mass (T_s) is then estimated using $T_s = 4H/V_s$, where H = representative height of the potential sliding mass and V_s = average shear wave velocity of the sliding mass. Generally, median or best estimate values of the soil's shear wave velocity and the representative height of the potential sliding mass assuming it is 1D should be used.

4. The seismic demand is then defined in terms of the design spectral acceleration for the site conditions below the sliding mass (which may include topographic amplification) at the degraded

period of the sliding mass (i.e., $S_a(1.5T_s)$) and the design moment magnitude (M) of the controlling earthquake event.

[The design spectral acceleration can be estimated using ground motion models in a deterministic seismic hazard assessment (DSHA) or estimated using a probabilistic seismic hazard assessment (PSHA) at the selected return period. The design spectral acceleration will vary for each project and depend on important seismic factors such as source-to-site distance, earthquake magnitude, site conditions, topographic effects, slip-rate, etc. The ground motion hazard level (e.g., whether a median or 84% value is used from the DSHA or whether the 475-year or 975-year return period value is used from the PSHA) should be established based on the uncertainty in the seismic hazard characterization, the criticality of the project, and the consequences of poor performance.]

5. Calculate the seismic coefficient using Eqn. (2), and apply this seismic coefficient in a pseudostatic slope stability analysis that satisfies all three conditions of equilibrium. If the resulting *FS* is greater or equal to one, the system is judged to perform satisfactorily, because the selected percentile estimate of the calculated seismically induced permanent displacement will be less than D_a.

A spreadsheet has been developed, which is available from the authors, to simplify the calculation of the design seismic coefficient using the Bray and Travasarou (2009) procedure. As stated previously, the median displacement level should typically be used, but a lower percentile displacement level could be used for critical projects (i.e., $\varepsilon > 0$). However, it is difficult to track the overall performance level of the system when, for example, a 84% spectral acceleration value is used to represent the design earthquake in combination with the 16% displacement level. Hence, it is preferred to use median estimates throughout the calculation and possibly select a different exceedance level only for the seismic displacement random error variable (i.e., ε). Lastly, the minimum value of the acceptable *FS* should not be set to be greater than 1.0, because *FS* varies nonlinearly as a function of the reliability of the system. The effect of achieving a *FS* greater than one cannot be reliably assessed in terms of potential seismic performance.

Representative results that show the relationship between the seismic coefficient corresponding to a specified allowable displacement level and key seismic slope stability parameters are provided in Figure 2. Allowable displacement values of 5, 15, and 30 cm were used to illustrate the dependence of the seismic coefficient on the selected level of allowable displacement. Results are also provided for the 15 cm displacement level for two magnitude scenarios representing a M = 6 event and a M = 7.5 event. For a specific displacement target and slope, the seismic coefficient increases with increasing ground motion intensity. It also increases as the magnitude (i.e., duration) of the earthquake event increases. The seismic coefficient decreases significantly as allowable displacement level increases. The seismic coefficient is shown to vary systematically in a rational manner as the allowable displacement threshold and design ground shaking level vary.

The seismic coefficient also depends on the potential sliding mass's fundamental period. Relatively stiff slopes that have short fundamental periods (i.e., 0.1 s < T_s < 0.3 s) tend to displace more because of resonance with the ground shaking. More flexible slopes (i.e., T_s > 0.5 s) have relatively less potential for seismic displacement. This effect can be seen through examination of a particular earthquake scenario. For the representative scenario specified in Figure 3 (i.e., soft rock site at R = 12 km from a shallow crustal strike-slip M = 7.2 event along a plate margin), the seismic coefficient initially increases as the slope's period increases from zero (i.e., the rigid sliding mass case) until a peak value is reached and then it decreases progressively as the slope's period moves away from resonance. Thus, as noted also by

Kramer and Smith (1997) and Rathje and Bray (2000), the seismic coefficient required to limit a specified allowable seismic displacement threshold is larger for shallow stiff sliding masses than for deep flexible sliding masses.

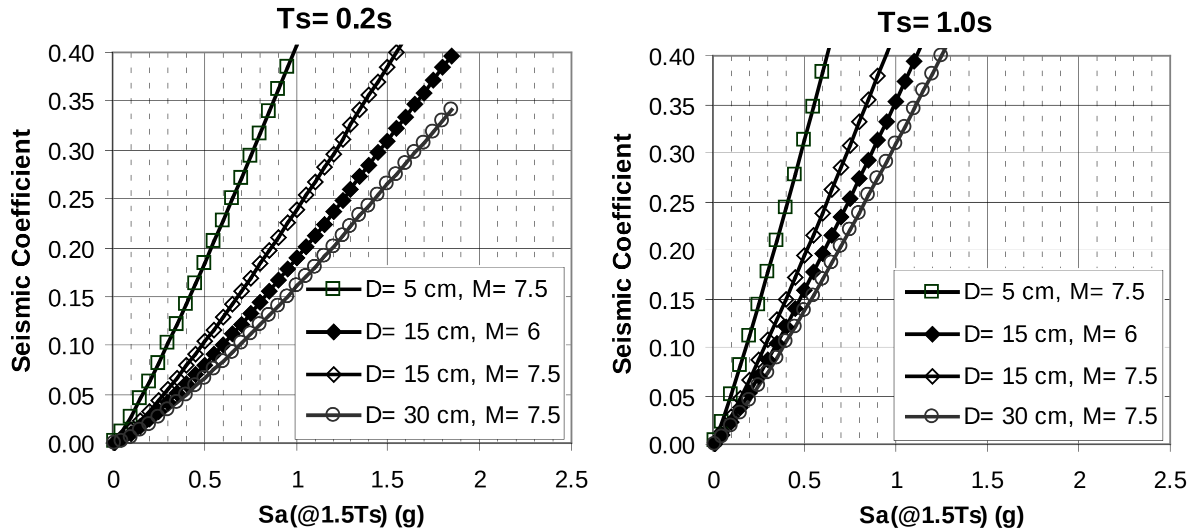

Figure 2. Variation of the seismic coefficient as a function of key seismic slope stability parameters (from Bray and Travasarou 2009)

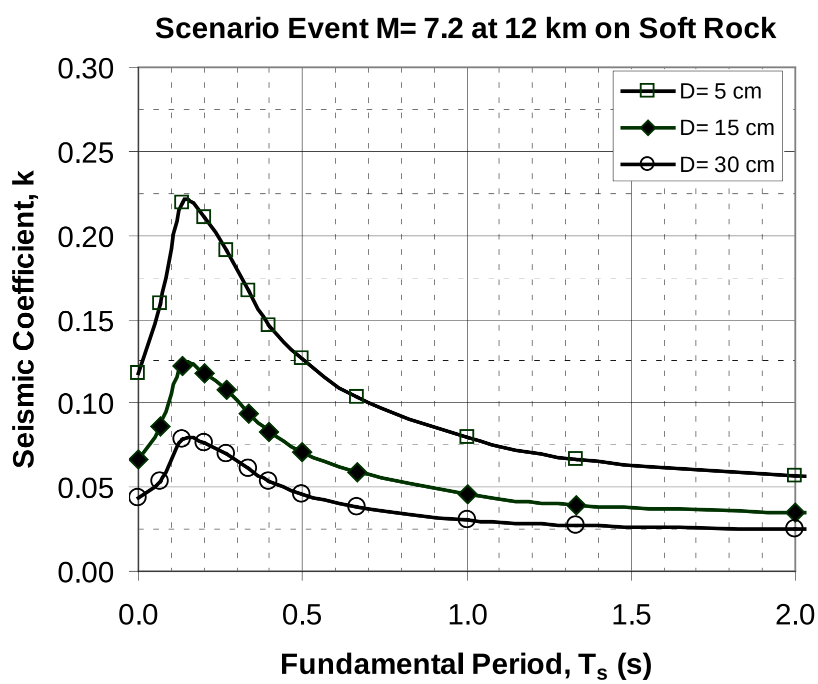

Figure 3. Dependence of the seismic coefficient on the fundamental period of the potential sliding mass for a scenario earthquake (from Bray and Travasarou 2009)

ILLUSTRATIVE PSEUDOSTATIC ANALYSIS

The seismic performance of a compacted earth embankment that is constructed as part of an industrial facility is evaluated using the recommended procedure to illustrate its use. The earth fill is 20 m high and with a side slope of 2H:1V with the shape shown in Figure 4. The embankment is located on a competent rock site at the IBC B/C boundary (i.e., V_{s30}= 760 m/s) at a source-to-site distance of 25 km from a M = 7.9 strike-slip event on the San Andreas fault in California. A deterministic pseudostatic slope stability analysis is performed to evaluate the potential of a deep slide through the base of the earth embankment.

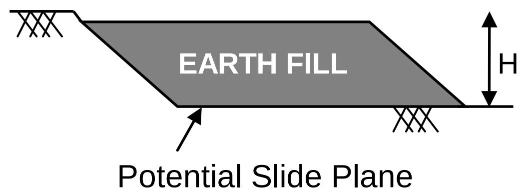

Figure 4. Illustrative seismic slope stability problem of a compacted earth fill atop a rock site

The recommended pseudostatic slope stability procedure is employed in these steps:

1. The compacted earth fill material, or its rock foundation, is not subject to severe strength loss as result of earthquake shaking. Its total stress undrained dynamic shear strength properties can be characterized as c = 5 kPa and $\phi = 35^o$.

2. In consultation with the owner, the allowable seismic displacement is set to be a median estimate of D_a = 15 cm for those areas of the compacted earth fill that support engineered structures on well-designed reinforced concrete mat foundations. At the 50% exceedance level, $\varepsilon = 0$.

3. The average shear wave velocity of the earth fill is estimated to be 270 m/s. For the case of base sliding at the maximum height of this trapezoidal-shaped potential sliding mass, the best estimate of its initial fundamental period is $T_s = 4H/V_s$ = (4)(20m)/(270 m/s) $\approx$ 0.3 s. The degraded period of the sliding mass is estimated to be $T_s' = 1.5T_s$ = 1.5 (0.3 s) = 0.45 s.

4. The best estimate of the spectral acceleration at the degraded period of the sliding mass can be computed as the mean of the median predictions from multiple ground motion models. Using the average of the median spectral acceleration estimates from the NGA models (Abrahamson et al. 2008) for the rock site condition for a strike-slip fault with M = 7.9 and R = 25 km, the design value of S_a at the degraded period of sliding mass is 0.28 g.

5. Using Eq. (2) with D_a = 15 cm, ε = 0, S_a = 0.28 g, T_s = 0.3 s, and M = 7.9, the design seismic coefficient is estimated as k = 0.06. Applying k = 0.06 in a pseudostatic slope stability program with the estimated dynamic strength properties produces a FS > 1, so the likely seismic performance earth embankment slope is judged to be acceptable for the conditions stated.

PRELIMINARY SCREENING PROCEDURE

In many building code applications, it is useful to employ a simpler form of Eq. (2) that is tied to readily available spectral values. For example, the U.S. National Seismic Hazard Maps provide short period spectral acceleration values ($S_a(T=0.2s)$) at the 10% and 2% probability of exceedance in 50 year levels for a rock site (V_{s30} = 760 m/s). The site coefficient F_a and a 2D slope factor may be used to modify S_a for other site classes and for topographic amplification, respectively. Assuming that the initial site period of the slope is about 0.1 s (i.e., a relatively shallow stiff soil), the seismic coefficient for a median seismic displacement estimate of 15 cm at a rock site is:

$$k_{15cm} = (0.036M - 0.004)S_a - 0.030 > 0.0 \text{ ; for } S_a = S_a(T=0.2s) < 2.0\ g \quad (3)$$

If a more stringent seismic displacement criterion was required, the seismic coefficient for a median seismic displacement estimate of 5 cm is:

$$k_{5cm} = (0.040M + 0.120)S_a - 0.034 > 0.0 \text{ ; for } S_a = S_a(T=0.2s) < 2.0\ g \quad (4)$$

Many slopes have larger initial site periods, which are within the range from 0.2 s to 0.5 s, where T_s = 0.33 s is a more representative value, so that a better estimate of the spectral acceleration at the degraded period of the slope is $S_a(T=0.5s)$. Spectral acceleration values at a period of 0.5 s are also provided at the U.S. National Seismic Hazard Mapping project web site at several return periods for a rock site (http://earthquake.usgs.gov/research/hazmaps/). The simplified relationships for the seismic coefficient for 15 cm and 5 cm median seismic displacement thresholds site are:

$$k_{15cm} = (0.038M + 0.006)S_a - 0.026 > 0.0 \text{ ; for } S_a = S_a(T=0.5s) < 1.5\ g \quad (5)$$

$$k_{5cm} = (0.038M + 0.166)S_a - 0.027 > 0.0 \text{ ; for } S_a = S_a(T=0.5s) < 1.5\ g \quad (6)$$

Eq. (5) has been adopted by the Association of Professional Engineers and Geoscientists of British Columbia (BC) for use in applying the 2006 BC Building Code through a pseudostatic slope stability analysis at the code-required ground motions level at the 2% probability of exceedance in 50 years. Simplified relationships for estimating the seismic coefficient at other allowable displacement levels for slopes of different fundamental periods can be developed through manipulation of Eq.(2). It must be remembered that the seismic coefficient employed in a pseudostatic slope stability analysis should depend on the site-specific seismic hazard and the amount of seismic displacement that is judged to be acceptable for the project.

CONCLUSIONS

The seismic coefficient used in a pseudostatic slope stability analysis should be selected in a rational manner. It is not rational to select an arbitrary value of the seismic coefficient such as k = 0.15 without considering the site-specific seismic hazard and determining the threshold of allowable seismic displacement that constitutes satisfactory seismic performance for the project. In the recommended procedure for selecting the seismic coefficient, the engineer incorporates the consequences of different amounts of seismic displacement and different seismic exposures in the selection of the seismic coefficient in a rational manner. Importantly, the engineer considers explicitly the project performance criteria and the seismic hazard in the selection of the seismic coefficient.

The recommended procedure requires that the engineer develop the project-specific allowable level of seismic displacement in consultation with the owner. This is a critically important dialogue and is clearly project-specific. The robustness of the earth structure or the systems supported on it should be assessed, and the consequences of unsatisfactory performance should be considered. The site-dependent seismic demand is characterized by the 5% damped elastic design spectral acceleration at the degraded period of the potential sliding mass either through a probabilistic or deterministic seismic hazard assessment. The level of uncertainty in the estimate of the seismic demand can be addressed through the use of different return periods in the PSHA or different percentiles of the deterministic estimate of the design spectral acceleration in the DSHA. The uncertainty of the displacement estimate given the other parameters can be handled through the use of different percentiles of the estimate seismic displacement that is determined to be allowable.

The potential for strength loss due to cyclic loading of the earth materials within and below the earth structure or natural slope should be evaluated before employing the pseudostatic slope stability procedure in engineering practice. Assessing the strength loss potential of soil strata is critically important. Additionally, it is important to remember that seismic slope performance is evaluated in terms of the potential range of seismically induced permanent displacements, even when a pseudostatic slope stability procedure is employed. Calculated Newmark-type seismic displacements are merely an index of seismic performance. Thus, an appropriate level of engineering judgment should be exercised.

ACKNOWLEDGMENTS

Support for this work was provided by the Earthquake Engineering Research Centers Program of the National Science Foundation under award number EEC-2162000 through the Pacific Earthquake Engineering Research Center (PEER) under award numbers NC5216 and NC7236.

REFERENCES

Abrahamson, N., Atkinson, G., Boore, D., Bozorgnia, Y., Campbell, K., Chiou, B., Idriss, I.M., Silva, W. and Youngs, R. (2008). “Comparisons of the NGA Ground-Motion Relations.” Earthquake Spectra, EERI, V. 24(1), pp. 45-66.

Bray, J.D. (2007). “Chapter 14: Simplified seismic slope displacement procedures.” Earthquake Geotechnical Engineering, 4th International Conference on Earthquake Geotechnical Engineering - Invited Lectures, in Geotechnical, Geological, And Earthquake Engineering Series, Vol. 6, Pitilakis, Kyriazis D., Ed., Springer, pp. 327-353.

Bray, J.D. and Rathje, E.R. (1998). “Earthquake-induced displacements of solid-waste landfills.” J. of Geotechnical and Geoenvironmental Engineering, ASCE, V. 124(3), pp. 242-253.

Bray, J.D. and Sancio, R.B. (2006). “Assessment of the Liquefaction Susceptibility of Fine-Grained Soils.” J. of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 132, No. 9, Sept., pp. 1165-1177.

Bray, J.D. and Travasarou, T. (2007). “Simplified procedure for estimating earthquake-induced deviatoric slope displacements.” J. of Geotechnical and Geoenvironmental Engineering, ASCE, V. 133(4), pp. 381-392.

Bray, J.D. and Travasarou, T. (2009). "Pseudostatic Coefficient for Use in Simplified Seismic Slope Stability Evaluation." J. of Geotechnical and Geoenvironmental Engineering, ASCE, 135(9), 2009, 1336-1340.

Duncan, J.M. and Wright, S.G. (2005) Soil Strength and Slope Stability. John Wiley & Sons, Inc., New Jersey.

Hynes-Griffin, M.E. and Franklin, A.G. (1984). "Rationalizing the seismic coefficient method." Misc. Paper No. GL-84-13. U.S. Army Engineer WES, Vicksburg, MS.

Idriss, I.M., 1985, "Evaluating seismic risk in engineering practice," Proc. Of the 11^{th} Intl. Conf. on SMFE, San Francisco, CA, V. 1, pp. 255-320.

Idriss, I.M., and Boulanger, R.W. (2008). Soil liquefaction during earthquakes. Earthquake Engineering Resesarch Institute, MNO-12, Oakland, CA.

Kramer, S. L., and Smith, M. W. (1997). "Modified Newmark model for seismic displacements of compliant slopes." J. Geot. and Geoenv. Engrg., ASCE, 123 (7), 635-644.

Newmark, N. M. (1965). "Effects of earthquakes on dams and embankments." *Geotechnique*, London, 15(2), 139-160.

Olsen, H.W. (1989) "Sensitive Strata in Bootlegger Cove Formation." J. of Geotech. Engrg., ASCE, V. 115(9), pp. 1239-1251.

Rathje EM, and Bray J.D. (2000). "Nonlinear coupled seismic sliding analysis of earth structures." Journal of Geotechnical and Geoenvironmental Engineering 2000; 126(11): 1002-1014.

Seed, H.B. (1979). "Considerations in the Earthquake-Resistant Design of Earth and Rockfill Dams." Geotechnique, V. 29(3), pp. 215-263.

Seed, H.B. (1981). "Earthquake-resistant design of earth dams." Proc. Inter. Conf. on Rec. Adv. in Geotech. EQ Engrg. and Soil Dyn., Univ. of Missouri-Rolla, MO, April 26-May 3, 17 pp.

Seed, H.B. (1983). "Earthquake-resistant design of earth dams." Proc. Symp. On Seismic Design of Embankments and Caverns, ASCE National Convention, Philadelphia, PA, May 16-20, pp. 41-64.

Seed, H.B. and Martin, G.R. (1966). "The seismic coefficient in earth dam design," J. Soil Mechanics and Foundation Div., ASCE, V. 92(3).

Seed, H.B., Makdisi, F.I., and DeAlba, P. (1978). "Performance of earth dams during earthquakes," J. Soil Mechanics and Foundation Div., ASCE, V. 104(7), pp. 967-994.

Seed, Seed, R.B., Cetin, K.O., Moss, R.E.S., Kammerer, A.M., Wu, J., Pestana, J.M., Riemer, M.F., Sancio, R.B., Bray, J.D., Kayen, R.E., and Faris, A. (2003). "Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework." 26^{th} Annual ASCE Los Angeles Geotechnical Spring Seminar, Keynote Presentation, Long Beach, Calif., April 30.

Stark, T.D. and Contreras, I.A. (1998). "Fourth Avenue Landslide During 1964 Alaskan Earthquake." J. Geotech. & Geoenviron. Engrg., V. 124(2), pp. 99-109.

Terzaghi, K. (1950). "Mechanisms of landslides." The Geological Survey of America, Engineering Geolgoy (Berkeley) Volume.

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