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# A Screen Analysis Procedure for Seismic Slope Stability

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Site-specific seismic slope stability analyses are required in California by the 1990 California Seismic Hazards Mapping Act for sites located within mapped hazard zones and scheduled for development with more than four single-family dwellings. A screen analysis is performed to distinguish sites for which only small ground deformations are likely from sites for which larger, more damaging landslide movements could occur. No additional analyses are required for sites that pass the screen, whereas relatively detailed analyses are required for sites that fail the screen. We present a screen analysis procedure that is based on a calibrated pseudo-static representation of seismic slope stability. The novel feature of the present screen procedure is that it accounts not only for the effects of ground motion amplitude on slope displacement, but also accounts for duration effects indirectly via the site seismicity. This formulation enables a more site-specific screen analysis than previous formulations that made *a priori* assumptions of seismicity/duration. This screen procedure has recently been adopted by the Landslide Hazard Implementation Committee for implementation by practicing engineers and engineering geologists in southern California. [DOI: 10.1193/1.1597877]

## INTRODUCTION

The 1990 California Seismic Hazards Mapping Act called upon the California Geological Survey (CGS) to map geographic areas considered to be potentially susceptible to earthquake-induced liquefaction or landslides. For developments located in these "Special Studies Zones" that include more than four single-family dwellings, engineers must perform site-specific studies to evaluate whether the mapped hazard actually exists. If a hazard is identified, appropriate remedial measures must be taken.

Working with the CGS, a number of southern California municipal and county agencies formed committees of experts charged with developing detailed guidelines for implementation of the Hazard Act's liquefaction and landslide components. The liquefaction guidelines (Martin and Lew 1999) largely follow the recommendations developed by a separate international committee of experts (Youd et al. 2001). No such consensus document exists for seismic slope stability, however, so the Landslide Hazards Implementation Committee (i.e., "the Committee") has developed, over the course of about four years, an original guidelines document (Blake et al. 2002). This guidelines

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document addresses a suite of issues, including drilling and sampling techniques, shear strength evaluation, evaluation of static slope stability, evaluation of seismic slope stability, and mitigation of slope stability hazards. For the most part, the Committee drew upon existing research and experience to draft guidelines on these topics. However, the topic of seismic slope stability required the Committee to customize analysis procedures originally developed for earth dams and landfills for application to residential and commercial construction.

There are two principal components to the guidelines on seismic slope stability—a screen analysis to determine if a seismic stability hazard is likely to exist at the site, and a formal displacement analysis for sites that fail the screen. The objective of this paper is to document the screen analysis procedure developed by the Committee and the process by which it was formulated. The displacement analysis procedures recommended by the Committee have been published previously, and hence are not discussed in detail herein.

Although not the focus of this paper, engineers should recognize the critical link between soil shear strength evaluation and the results of stability analyses such as those described in this paper. The shear strength evaluation procedures that are intended for use with the present procedure follow established standards, and are thoroughly documented in the guidelines document (Blake et al. 2002). These procedures encourage high-quality sampling that minimizes sample disturbance; careful testing that reproduces the appropriate drainage condition, overburden stress, and strain rate in the tested specimen; and test result interpretation that utilizes the appropriate portion of the stress-deformation curve for a particular application (i.e., peak vs. residual or ultimate strength). The guidelines document allows, in certain cases, departures from optimal sampling and testing protocols that are consistent with procedures commonly used in practice (e.g., undrained testing at low strain rates instead of the rapid rates that would be present during an earthquake). However, such departures are only allowed when they lead to under-prediction of shear strength, which produces conservative assessments of slope stability hazards. We wish to emphasize that the screen analysis procedure described herein should not be used with strength evaluation procedures that depart from the recommendations contained in Blake et al. (2002).

## EXISTING SCREEN PROCEDURES FOR SEISMIC SLOPE STABILITY

Screen analysis procedures for seismic slope stability have been adopted by a number of U.S. agencies with jurisdiction over hillside residential construction, earth dams, and solid-waste landfills. These procedures generally utilize a pseudo-static representation of seismic demand in which a destabilizing horizontal seismic coefficient ($k$) is utilized within a conventional limit equilibrium slope stability calculation. The seismic coefficient represents the fraction of the weight of the sliding mass that is applied as an equivalent horizontal force acting through the centroid of the slide mass. The factor of safety against shear failure is checked with the equivalent horizontal force applied to the slope, and the slope passes the screen if the factor of safety exceeds a specified minimum value. For the sake of convenience, notation for two types of seismic coefficients is introduced here for later reference. The first is the seismic coefficient that reduces the pseudo-static factor of safety (FS) for a given slope to unity, and is referred to as the yield coefficient, $k_y$. The second is the peak value of spatially averaged horizontal acceleration (normalized by $g$) within the slide mass, and is denoted $k_{\max}$.

One widely used screen analysis procedure was developed by Seed (1979) for application to earth dams. The procedure calls for $k=0.1$ or 0.15 to be applied for $M=6.5$ and 8.25 earthquakes, respectively. The screen is passed if the factor of safety, FS, exceeds 1.15. A slightly modified version of that procedure, in which $k=0.15$ and FS $\geqslant 1.1$ regardless of local seismicity, was adopted in 1978 by Los Angeles County for application to hillside residential construction and has been used since that time. Seed (1979) recommended that his procedure only be applied for cases where the earth materials do not undergo significant strength loss upon cyclic loading (i.e., strength loss $<15\%$) and where several feet of crest displacement was deemed "acceptable performance," as is the case for many earth dams (e.g., 0.9 m displacement for $M=8.25$ and crest acceleration$=0.75g$).

An important feature of the Seed (1979) procedure is its calibration to a particular slope performance level, which is represented by the displacement of a rigid block on an inclined plane (i.e., a Newmark-type displacement analysis, Newmark 1965). Seed (1979) calibrated his pseudo-static approach using Newmark displacements calculated with simplified methods (e.g., Makdisi and Seed 1978). The Makdisi and Seed simplified procedure, in turn, is based on a limited number of calculations that were used to relate Newmark displacement to earthquake magnitude and $k_y/k_{\max}$ (e.g., five calculations for $M=6.5$, two for $M=7.5$, and two for $M=8.25$). Seed's (1979) recommendations are an important milestone, as they represent perhaps the first calibration of a pseudo-static method to a particular level of slope performance as indexed by displacement. This concept underlies other widely used screen analysis procedures that have been developed to date, and is retained as well in the present work.

Since the Seed (1979) work, additional screen analysis procedures have been developed for application to earth dams and solid waste landfills. A procedure for earth dams was developed by Hynes-Griffin and Franklin (1984) based on (1) calculations of shaking within embankment sections using a linear elastic shear beam model by Sarma (1979) and (2) calculations of Newmark displacement from time histories using the analysis approach of Franklin and Chang (1977). Those calculations resulted in statistical relationships between the amplification of shaking within embankments (i.e., ratio of $k_{\max}\times g$ to maximum horizontal acceleration of base rock, $\mathrm{MHA_r}$) and the depth of the sliding surface, as well as between Newmark displacement and $k_y/k_{\max}$. Hynes-Griffin and Franklin (1984) developed their pseudo-static procedure using approximately a 95th-percentile value of amplification for deep sliding surfaces along with the upper-bound value of $k_y/k_{\max}$ that produces 1.0 m of displacement. In the resulting procedure, $k$ is taken as $0.5\times\mathrm{MHA_r}$, and the screen is passed if $\mathrm{FS}\geqslant 1.0$. The procedure is intended for use with 80% of the shear strength in nondegrading materials. The method is not recommended for areas subject to large earthquakes, embankments constructed of or on liquefiable soils, or embankments for which small displacements are intolerable.

Bray et al. (1998) used a similar procedure to that of Hynes-Griffin and Franklin (1984) to develop a screen procedure for solid-waste landfills. As with the earlier procedure, two suites of statistical results underlie the procedure. One relates the peak acceleration of the slide mass ($k_{max} \times g$) to $MHA_r$, the other relates displacement for a given $k_y/k_{max}$ to the amplitude and duration of shaking. A large number of calculations were performed by Bray et al. (1998) to establish these relationships, which are discussed in more detail below. The screen procedure was developed using nearly upper bound amplification factors (i.e., $k_{max} \times g/MHA_r$) and tolerable displacements of about 0.15–0.3 m. The resulting procedure calls for $k$ to be taken as $0.75 \times MHA_r$, and the screen is passed if $k > k_y$ (which is analogous to having $FS \geqslant 1$ when $k$ is applied in a pseudo-static analysis).

The above is not a comprehensive review of all screen procedures developed to date for seismic slope stability. Rather, our intent is to illustrate the principal steps taken in the development of commonly used, rational screen procedures, and the conditions for which these procedures are intended to be applicable. Three important conditions underlie the screen procedures: (1) the level of displacement considered tolerable for a specific application, (2) the earthquake magnitude associated with the time histories used to calculate displacements, and (3) the level of conservatism employed in the interpretation of statistical distributions of results. Discussion on these three points is provided below:

- The limiting displacements used by Seed (1979) and Hynes-Griffin and Franklin (1984) for earth dams were on the order of 100 cm. The limiting displacements used by Bray et al. (1998) for landfills were 15 to 30 cm, which is similar to an earlier 15 cm value recommended by Seed and Bonaparte (1992).
- The earthquake magnitude used by Seed (1979) in developing the criteria subsequently adopted by L.A. County is 8.25. The time histories used by Hynes-Griffin and Franklin (1984) are from magnitudes that range from 3.8 to 7.7, with most being 6.6 (San Fernando earthquake). Bray et al. (1998) did not use magnitude directly, but instead used duration, which is strongly correlated to magnitude. The durations used by Bray et al. are consistent with earthquake magnitudes of about 7 to 8, with most being closer to 8 (Bray 2002).
- Seed (1979) exercised conservatism by using upper-bound values of displacement for a given $k_y/k_{max}$. Hynes-Griffin and Franklin (1984) were highly conservative through their use of 95th-percentile amplification levels coupled with upper-bound displacements for a given $k_y/k_{max}$. Bray et al. (1998) exercised conservatism by using nearly upper-bound amplification levels and 84th-percentile displacements.

The screen analysis procedure developed herein is intended principally for application to hillside residential and commercial developments. For construction of this type, small ground deformations can cause collateral loss that is considered unacceptable by owners, insurers, and regulatory agencies. Accordingly, the limiting displacements used in existing screen procedures for earth dams and landfills are considered to be too large for application to hillside construction. Another problem with the existing procedures is the level of conservatism employed in their development. For example, the existing methods apply for specific ranges of earthquake magnitude (which are high for the Seed and Bray et al. methods), and may not pass otherwise safe sites for which the design magnitude is smaller than that used in the development of the screen. Moreover, the conservative interpretation of amplification and displacement distributions used in the development of existing schemes likely makes the level of risk associated with the slope performance differ significantly from that associated with the ground motions. In other words, if the ground motion is evaluated with probabilistic hazard analysis for a given return period, and the slope displacement conditioned on that ground motion is extreme (i.e., a rare realization), the resulting slope design is based on displacements having a much longer return period than the design-basis ground motions.

Given these shortcomings, the Committee has developed a new screen procedure tailored to the needs of hillside residential and commercial construction (in terms of displacement) and which accounts for site-specific seismicity. The screen procedure was also developed so as to limit the level of conservatism in order to maintain a reasonable return period on the expected slope performance. The remainder of this paper describes the development of the procedure.

## DEVELOPMENT OF SCREEN ANALYSIS PROCEDURE

### INTRODUCTION

The purpose of screen investigations for sites within zones of required study is to filter out sites that have no potential or low potential for earthquake-induced landslide development. No additional seismic stability analysis is required for a site that passes the screen, whereas further quantitative evaluation of landslide hazard potential (and possibly mitigation) is required for sites that fail the screen.

Like other screen procedures described in the previous section, ours is based on a pseudo-static representation of seismic slope stability. The procedure is implemented by entering a de-stabilizing horizontal seismic coefficient ($k$) into a conventional slope stability analysis. The seismic coefficient represents the fraction of the weight of the sliding mass that is applied as an equivalent horizontal force acting through the centroid of the mass. If the factor of safety is greater than one (FS$>$1), the site passes the screen, and the site fails if FS$<$1.

We formulate the seismic coefficient as the product of the maximum horizontal acceleration at the site for a rock site condition ($\mathrm{MHA_r}$) and a factor ($f_{eq}$) related to the seismicity of the site, the maximum tolerable slope displacement, and other factors:

$$k=f_{eq}\times MHA_r/g \tag{1}$$

where $g$ is the acceleration of gravity. The two key steps in the development of the screen procedure are therefore (1) rationale for the use of $\mathrm{MHA_r}$ to represent the amplitude of shaking within the slide mass, and (2) formulation of $f_{eq}$ to represent the effects of local seismicity and the maximum tolerable slope displacement. The following two subsections discuss these steps.

### AMPLITUDE OF SHAKING IN SLIDE MASS

Ideally, the $\mathrm{MHA_r}/g$ term in Equation 1 should represent the spatially averaged peak amplitude of shaking within the slide mass, which differs from the maximum horizontal acceleration at the base of the slide for a rock site condition ($\mathrm{MHA_r}$) as a result of ground response and topographic effects within the slide mass (which can amplify or

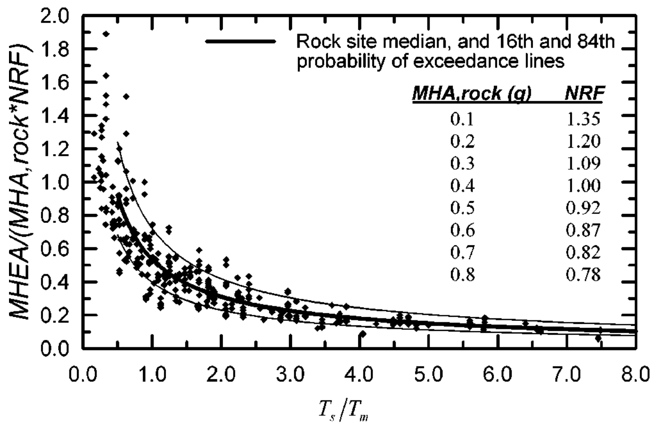


**Figure 1.** Normalized MHEA for deep-seated slide surface vs. normalized fundamental period of slide mass (after Bray et al., 1998).

de-amplify shaking) and vertical and lateral incoherence of ground motion within the slide mass (which tends to de-amplify shaking). Bray et al. (1998) define the spatially averaged peak acceleration of a slide mass as the maximum horizontal equivalent acceleration (MHEA). Parameter MHEA is a more direct indicator of shaking amplitude in a slide mass than $MHA_r$ and hence Equation 1 could be rewritten as

$$k = f_{eq}^{*} \times MHEA/g \tag{2}$$

where $f_{eq}^{*} = f_{eq} \times (MHA_r/MHEA)$. Bray et al. (1998) evaluated MHEA as a function of $MHA_r$ from calculations of wave propagation through an equivalent one-dimensional slide mass. As shown in Figure 1, Bray et al. normalize calculated MHEA in the slide mass by the product of $MHA_r$ and a nonlinear response factor (NRF), which accounts for nonlinear ground response effects as vertically propagating shear waves pass through the slide mass. Bray et al. use $MHA_r$ as the normalizing ground motion even for sites where the foundation materials are soil because their analyses did not indicate site condition as significantly affecting MHEA (except for deep soft clay sites such as NEHRP E sites, for which site specific analyses were recommended). The ratio MHEA/($MHA_r$ $\times$NRF) differs from one as a result of vertical ground motion incoherence within the slide mass, and is related in Figure 1 to the ratio of the small-strain period of the sliding mass ($T_s$) to the mean period of the input motion ($T_m$). The ratio MHEA/($MHA_r^*$NRF) is less than one for $T_s/T_m > \sim 0.5$, and is variable with an average of about 1.0 for $T_s/T_m < \sim 0.5$.

The magnitude and distance that control the peak acceleration hazard in much of urban California are moment magnitude 6.5 to 7.5 earthquakes at distances generally less than 10 km (Petersen et al. 1996). Parameter $T_m$ has a median value of about 0.5 s for those magnitude and distance ranges (Rathje et al. 1998). Parameter $T_s$ is calculated as

$$T_s = \frac{4H}{V_s} \tag{3}$$

where $H$=thickness of slide mass and $V_s$=average shear wave velocity of slide mass. If $V_s$ is taken as 300 m/s (consistent with soft bedrock or compacted fill materials), the slide mass thickness would have to exceed about 20 m for $T_s/T_m > 0.5$. Thus, it was the Committee's judgment that MHEA/($\text{MHA}_r \times$NRF)=1.0 would be a reasonable assumption for sites with critical slip surfaces at depth $<\sim$20 m [since 1.0 is an average value of MHEA/($\text{MHA}_r \times$NRF) for this condition], and would be conservative for deeper-seated slip surfaces (depth $>\sim$20 m). Because parameter NRF is a function of $\text{MHA}_r$ (as shown in Figure 1) the assumption of MHEA/($\text{MHA}_r \times$NRF)=1.0 makes MHEA solely a function of $\text{MHA}_r$. Accordingly, Equation 2 can be rewritten as Equation 1 provided the effect of NRF is incorporated into factor $f_{eq}$, which is done in the next section.

Some additional comments on the use of the MHEA model in Figure 1 are appropriate at this point. First, while the model was developed from one-dimensional response analyses, it has been found to be conservative for deep-seated slide surfaces through two-dimensional slope geometries (Rathje and Bray 1999). Second, Rathje and Bray (1999) found the model to be unconservative for shallow surfaces near slope crests. Accordingly, we do not recommend use of the present screen analysis procedure for surficial stability problems such as raveling or rockfalls. Analysis procedures for such conditions are presented by Ashford and Sitar (2002).

## FORMULATION OF SEISMICITY FACTOR $f_{eq}$

For a given $\text{MHA}_r$, large magnitude earthquakes will tend to cause poorer slope performance than smaller magnitude earthquakes. One important reason for this is that duration increases with magnitude, and slope deformations increase with duration. Moreover, $T_m$ also increases with magnitude, which implies longer wavelengths and more vertical coherence of motion within the slide mass [although this wavelength effect is not critical for the present application since we have conservatively assumed no vertical incoherence, i.e., MHEA/($\text{MHA}_r \times$NRF)=1.0]. Accordingly, we incorporate the effects of magnitude on slope performance into parameter $f_{eq}$. Previous pseudo-static procedures for seismic slope stability have specified a single value for $f_{eq}$, and thus have made implicit, and usually very conservative, assumptions about the magnitude of earthquakes causing the design-basis $\text{MHA}_r$. The Committee sought to reduce that conservatism by developing a range of $f_{eq}$ values that are a function of magnitude as well as site-source distance.

Magnitude- and distance-dependent $f_{eq}$ values were developed using a statistical model that relates slope displacements from a Newmark-type analysis ($u$) to the amplitude of shaking in the slide mass ($k_{\max}$=MHEA/$g$), significant duration of shaking (measured as the time between 5–95% normalized Arias intensity, $D_{5\text{-}95}$), and the ratio $k_y/k_{\max}$. The statistical model employed here was developed by Bray and Rathje (1998) from regression analysis of 309 Newmark displacement values calculated from ground motion records from magnitude 6.25 to 8 earthquakes at each of four $k_y/k_{\max}$ ratios. The model and data from Bray and Rathje are shown in Figure 2, and indicate a lognormal

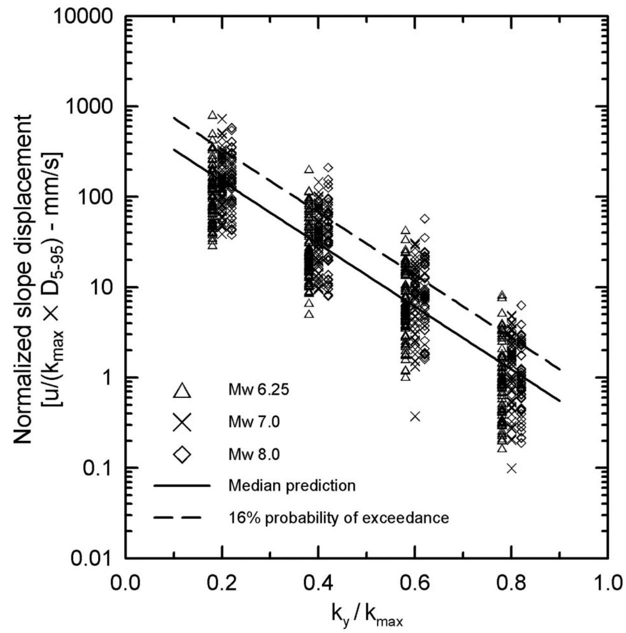


**Figure 2.** Normalized sliding displacement (modified from Bray and Rathje, 1998).

distribution of normalized displacement $u/(k_{\max}\cdot D_{5\text{-}95})$ for a given $k_y/k_{\max}$ ratio. Regression analyses indicate that the median of this lognormal distribution is described by

$$\log_{10}\left(\frac{u}{k_{\max}\cdot D_{5\text{-}95}}\right)=1.87-3.477\cdot\frac{k_y}{k_{\max}} \tag{4}$$

where $u$ is the median displacement in cm. The standard deviation is 0.35 in $\log_{10}$ units.

A relationship between magnitude, distance, $\mathrm{MHA_r}$, and $f_{eq}$ was established using the Bray and Rathje relationship (Equation 4) with the following assumptions and observations:

1. Factor $f_{eq}^{*}$ (Equation 2) was taken as equivalent to $k_y/k_{\max}$. The equivalency of $k_y/k_{\max}$ and $f_{eq}^{*}$ can be understood by recognizing that $k_y/k_{\max}$ simply represents the factor by which the actual ground shaking intensity ($k_{\max}$) needs to be reduced to render a seismic coefficient associated with FS=1 (i.e., $k_y=k_y/k_{\max}\times k_{\max}$). Referring to Equation 2, because our screen procedure is intended for use with FS=1, $f_{eq}^{*}$ represents the factor by which MHEA/$g$ needs to be reduced to yield a seismic coefficient associated with FS=1 (i.e., $k_y$). Accordingly, if $k_y$ is substituted for $k$ in Equation 2 (appropriate for FS=1) and $k_{\max}$ is substituted for MHEA/$g$ (by definition), it can be readily seen that $f_{eq}^{*}=k_y/k_{\max}$.
2. Parameter MHEA is inconvenient for use in a screen procedure because its relationship to $\mathrm{MHA_r}$ is affected by vertical ground motion incoherence effects

and nonlinear ground response effects. As described in the previous section, to simplify the analysis we assume $\mathrm{MHEA}/(\mathrm{MHA_r}\times\mathrm{NRF})=1.0$. From Equations 1 and 2, we see that $f_{eq}=f_{eq}^{*}\times\mathrm{MHEA}/\mathrm{MHA_r}$, which reduces to $f_{eq}^{*}\times\mathrm{NRF}$ with the above assumption. Because $f_{eq}^{*}=k_y/k_{\max}$, we calculate parameter $f_{eq}=k_y/k_{\max}\times\mathrm{NRF}$.

3. Two threshold levels of Newmark displacement were selected by the Committee, $u=5$ and 15 cm. It should be noted that the Newmark displacement parameter is merely an index of slope performance. The 5 cm threshold value likely distinguishes conditions for which very little displacement is likely from conditions for which moderate or higher displacements are likely. The 15 cm value likely distinguishes conditions in which small to moderate displacements are likely from conditions where large displacements are likely. It should be noted that these threshold displacement values are smaller than values used in the development of existing screen procedures for dams and landfills. The Committee's use of the smaller displacement values is driven by a concern on the part of owners, insurers, and regulatory agencies to minimize collateral loss from slope deformations in future earthquakes.
4. On the left-hand side of Equation 4, factor $k_{\max}$ is taken as $\mathrm{MHA_r}\times\mathrm{NRF}/g$. Parameter $D_{5\text{-}95}$ is a function of magnitude and distance, and can be estimated from available attenuation relationships.

Based on the above, calculations were performed to evaluate as a function of $f_{eq}$ the probability that seismic slope displacement $u>5$ cm conditional on $\mathrm{MHA_r}$, magnitude ($M$), and distance ($r$). This probability is calculated as

$$P(u>5\text{ cm}|MHA_r,M,r,f_{eq})=\int_{D_{5\text{-}95}} f(D_{5\text{-}95}|M,r)P(u>5\text{cm}|D_{5\text{-}95}(M,r),MHA_r,f_{eq})d(D_{5\text{-}95}) \tag{5}$$

where $d(D_{5\text{-}95})$ represents a differential duration; $f(D_{5\text{-}95}|M,r)$ represents a log-normal probability density function described by an attenuation relationship for duration; the probability term is evaluated from the cumulative distribution function described by the median and standard deviation terms by Bray and Rathje (1998); and the integration is performed over a range of durations (taken as the median $\pm 2.5$ standard deviations of duration for the given $M$ and $r$). Similar calculations were performed for $u>15$ cm.

To illustrate the application of Equation 5, Figure 3 shows for $M=7$, $r=20$ km, $\mathrm{MHA_r}=0.4g$, and a 5-cm limiting displacement the variation of the probability term on the left-hand side of Equation 5 with $f_{eq}$. The distribution in Figure 3 is unity minus a normal cumulative distribution function with median 0.56 and standard deviation 0.117 (arithmetic units). The standard deviation term is related to the dispersion of the duration attenuation model and the Bray and Rathje displacement model, and is independent of $M$, $r$, $\mathrm{MHA_r}$, and $u$. We evaluated median $f_{eq}$ values for a range of $\mathrm{MHA_r}$, $M$, and $r$ (e.g., 0.56 for the example in Figure 3) and for limiting displacements of 5 cm and 15 cm. We chose to use the median because, in our judgment, probabilities departing significantly from the 50th percentile would unnecessarily bias the effective return period for exceedance of the specified level of slope displacement (i.e., $u>5$ cm) from the re-

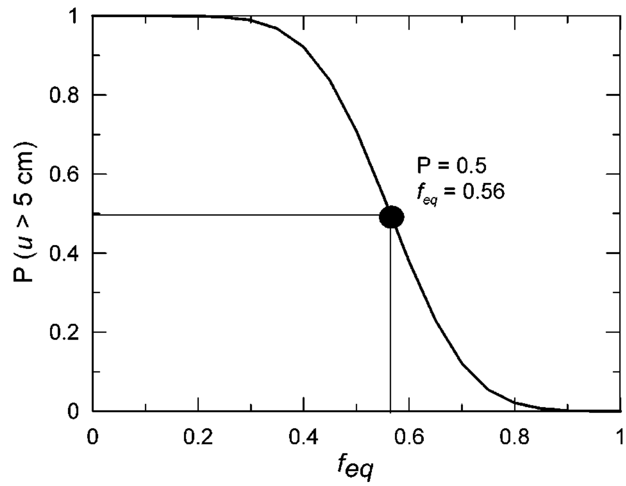


**Figure 3.** Variation of exceedance probability for 5-cm slope displacement with $f_{eq}$ for $M=7$, $r=20$ km, and $MHA_r=0.4g$.

turn period for the ground motion (typically 475 years). However, $f_{eq}$ values for other percentiles can be readily evaluated from the median because the standard deviation is fixed at 0.117.

The variation of median $f_{eq}$ values with $M$, $r$, and $MHA_r$ are shown in Figure 4a for the 5-cm threshold displacement and in Figure 4b for the 15 cm threshold. The values in Figures 4a and b were derived using the Abrahamson and Silva (1996) attenuation model for duration at rock sites, and is applicable for tectonically active regions. Near-fault effects on ground motion parameters were neglected in the development of Figure 4; such effects would tend to increase the amplitude of long-period components of the ground motion but decrease the duration, and hence the net effect on seismic slope displacements would likely be small. Focal mechanism does not affect these calculations because the Abrahamson and Silva attenuation model for duration does not contain a focal mechanism term (i.e., focal mechanism has not been shown to exert a statistically significant effect on duration).

The equation of the curves in Figure 4 is as follows:

$$f_{eq}=\frac{NRF}{3.477}\times\left[1.87-\log_{10}\left(\frac{u}{(MHA_r/g)\times NRF\times D_{5\text{-}95,m}}\right)\right] \tag{6}$$

where $u=5$ or 15 cm, and $D_{5\text{-}95,m}$=median duration from Abrahamson and Silva (1996) relationship, defined by

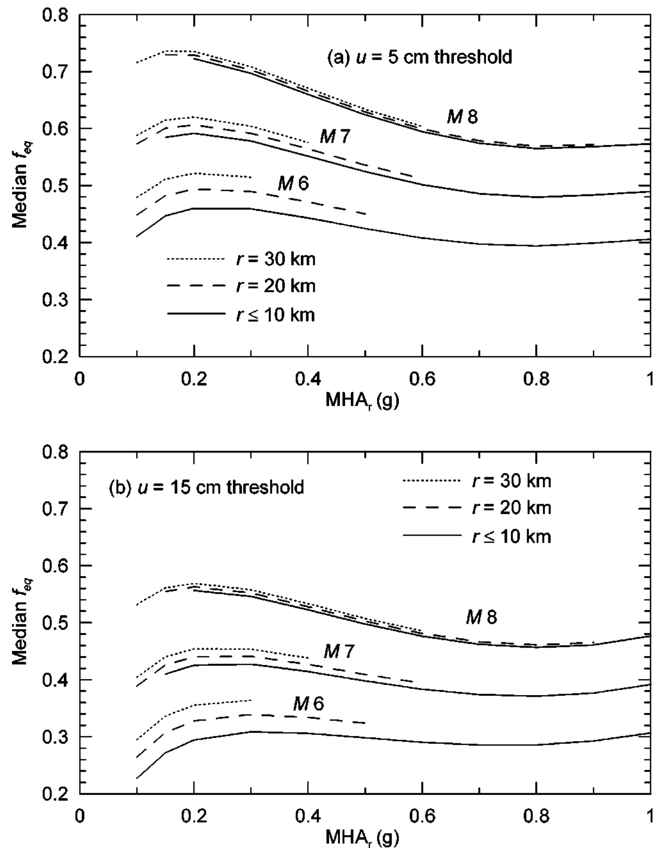


**Figure 4.** Required values of $f_{eq}$ as function of $\mathrm{MHA_r}$ and seismological condition for indicated values of Newmark displacement.

$$r>10\ \text{km:}\quad \ln(D_{5\text{-}95,m}) = \ln\left[\frac{\left(\dfrac{\exp[5.204+0.851\cdot(M-6)]}{10^{1.5M+16.05}}\right)^{-1/3}}{15.7*10^6} + 0.063\cdot(r-10)\right] + 0.8664 \tag{7a}$$

$$r<10\ \text{km:}\quad \ln(D_{5\text{-}95,m}) = \ln\left[\frac{\left(\dfrac{\exp[5.204+0.851\cdot(M-6)]}{10^{1.5M+16.05}}\right)^{-1/3}}{15.7*10^6}\right] + 0.8664 \tag{7b}$$

and *NRF* is defined by the relationship tabulated in Figure 1, which can be approximated by

$$NRF \approx 0.622 + 0.920\exp(-2.25\times MHA_r/g) \tag{8}$$

for $0.1 < \mathrm{MHA_r}/g < 0.8$.

Referring to Figure 4, the strong increase in $f_{eq}$ with magnitude and small increase with distance are driven by the duration attenuation model, which shows similar variations in $D_{5\text{-}95}$ with magnitude and distance. The variation with $MHA_r$ is driven by the statistical displacement model (Equation 4) and the NRF parameter. Without the NRF parameter, the curves in Figure 4 would increase linearly with the logarithm of $MHA_r$. Inclusion of the NRF parameter increases $f_{eq}$ at small $MHA_r$ and decreases $f_{eq}$ at large $MHA_r$ to the extent that $f_{eq}$ is only weakly dependent on $MHA_r$.

As noted previously, $f_{eq}$ values for percentiles other than 50 (i.e., the median) can be evaluated through use of the fixed standard deviation term of 0.117. For example, the 84th-percentile values can be obtained by adding 0.117 to the $f_{eq}$ values estimated from Equation 6. Individual users or regulators who wish to use a more conservatively developed screen procedure (i.e., one for which the return period on exceedance of a slope displacement exceeds that of the ground motion) can simply add an appropriate number of standard deviations to the median values in Figure 4.

## APPLICATION

### DESIGN GROUND MOTIONS

A critical issue associated with the application of the above screen procedure is selection of appropriate design-basis earthquake ground motions. For applications in tectonically active regions, the Committee recommends that the $MHA_r$ having a 475-year return period (10% probability of exceedance in 50 years) be estimated using probabilistic seismic hazard analysis (PSHA). The relative contributions of earthquake events at different magnitudes and distances to this $MHA_r$ hazard should then be evaluated through a de-aggregation analysis, and the mode magnitude ($\overline{M}$) and mode distance ($\overline{r}$) identified for use in the screen. Each of these parameters ($MHA_r$, $\overline{M}$, and $\overline{r}$) are available from California statewide hazard mapping (e.g., Petersen et al., 1996).

The Committee considered the use of supplemental deterministic seismic hazard analyses for sites located near large-magnitude, high slip-rate faults (such as the San Andreas Fault system). However, it was found for many checked locations that $k$ values computed deterministically were less than $k$ values evaluated from PSHA. The PSHA results used in these checks are from published statewide maps for California (Petersen et al. 1996). In our checks, the deterministic $k$ values were evaluated using the characteristic earthquake event (as compiled by Petersen et al. 1996) on the largest fault segment nearest the site, and the 84th percentile $MHA_r$ value associated with that characteristic event. The Committee recognizes that more severe deterministic scenario events could be conceived, but these would likely be sufficiently rare as to have a return period that significantly exceeds the 475-year target.

### LIMITATIONS

The screen analysis procedure presented herein represents a simplification of true Newmark-type displacement analyses, and obviously provides less insight into the physics of the problem than more sophisticated analyses. As with any simplified analysis, the

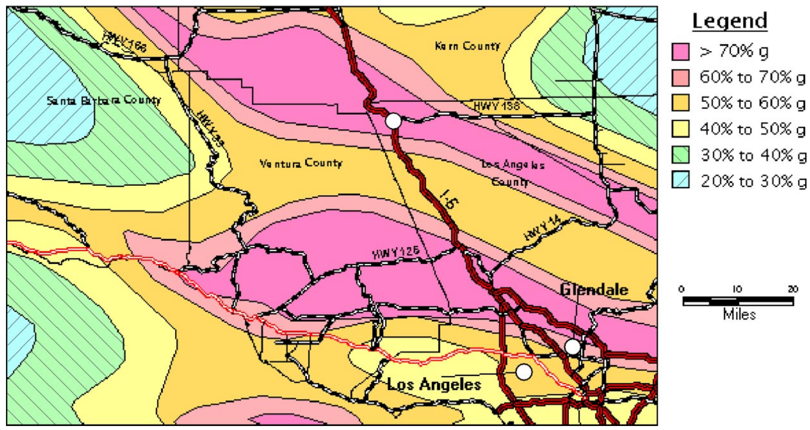


**Figure 5.** Probabilistic seismic hazard map by Petersen et al. (1996) for MHA on rock at the 475-year hazard level.

present procedure could potentially be abused by users unfamiliar with the problem and the process by which the simplified analysis procedure was formulated. Accordingly, to ensure proper application of the procedure, users are strongly encouraged to familiarize themselves with this paper, the guidelines document (Blake et al. 2002), and other relevant references cited herein.

As with other screen analysis procedures, the present procedure should not be used for slopes comprised of geologic materials that could be subject to significant strain softening, such as liquefiable soils. The procedure should only be used when shear strengths are evaluated in accordance with the recommendations contained in Blake et al. (2002). The procedure is not applicable to slopes constructed over soft clay soils (e.g., NEHRP Category E sites), because as noted previously the Bray et al. (1998) relationship for MHEA (Figure 1) does not apply for this site condition. The $f_{eq}$ values given in Figure 4 are only applicable to tectonically active regions, such as California. The procedure should not be applied to situations for which 5 cm (or 15 cm) displacement is an inappropriate threshold. The procedure should not be applied to surficial slope stability problems such as raveling and rockfalls.

Finally, it should be noted that this screen analysis procedure, and any analysis of seismic slope stability based on Newmark sliding block models, only provides a general indication or index of slope performance that is related to the accumulation of permanent shear deformations within the ground. Moreover, volumetric ground deformations associated with postliquefaction pore-pressure dissipation or seismic compression of unsaturated soil are not considered in Newmark-type models and need to be evaluated separately.

## EXAMPLES

Seismic coefficients ($k$) for three example sites in southern California are evaluated to illustrate application of the screen procedure defined by Equations 1 and 6. Locations

**Table 1.** Evaluation of seismic coefficient for example sites

| | | | | 5 cm threshold | | 15 cm threshold | | |
|---|---|---|---|---|---|---|---|---|
| | $MHA_r$ (g) | $\overline{M}$ | $\overline{r}$ | $f_{eq}$ | k | $f_{eq}$ | k | Reference |
| Los Angeles | 0.54 | 6.4 | 2.0 | 0.46 | 0.25 | 0.33 | 0.18 | CDMG, 1998a |
| Glendale | 0.65 | 7.0 | 7.0 | 0.49 | 0.34 | 0.38 | 0.25 | CDMG, 1998b |
| Hwy 138 and I-5* | 0.70 | 7.5–8.0 | <10 km | 0.55 | 0.39 | 0.44 | 0.31 | Petersen et al., 1996 |

* values approximate since no detailed map of this area

of the sites are shown in Figure 5. The site denoted Los Angeles in Figure 5 is on the north flank of the Santa Monica Mountains, and is not immediately adjacent to any major active fault systems. The site denoted Glendale is near the base of the San Gabriel Mountains, and is close to the Sierra Madre Fault system. The site at the intersection of Highway 138 and Interstate Highway 5 is adjacent to the San Andreas Fault.

Calculations of seismic coefficient $k$ for the three sites are illustrated in Table 1. The values of $MHA_r$, $\overline{M}$, and $\overline{r}$ in Table 1 are obtained from the cited references. The $f_{eq}$ values for the threshold displacement of 5 cm vary from 0.46 to 0.55, which are similar to the value used by Hynes-Griffin and Franklin (1984) for dams ($f_{eq}$=0.5). The similarity of these $f_{eq}$ values results from the compensating effects of the present procedure having smaller threshold displacements (which increases $f_{eq}$) and being formulated less conservatively (which decreases $f_{eq}$). Our values for the 5-cm threshold are smaller than the value used by Bray et al. (1998) for landfills ($f_{eq}$=0.75) because our less conservative formulation overcompensates for our smaller threshold displacements. The $f_{eq}$ values for the 15-cm threshold are considerably smaller than those recommended by either Hynes-Griffin and Franklin or Bray et al. As discussed in Blake et al. (2002), the majority of the Committee recommends the use of a 5-cm threshold, while a minority recommends a 15-cm threshold.

It should also be noted that the $\overline{M}$ values indicated in Table 1 are consistent with the characteristic earthquake magnitudes for faults near the respective sites (as tabulated in Petersen et al. 1996). The similarity of these magnitudes is the principal reason that the Committee does not consider it necessary to perform supplemental deterministic analyses of scenario events (which would also have a magnitude similar to the characteristic earthquake magnitude).

## POST-SCREEN ANALYSIS

For sites that fail the screen analysis, more detailed slope displacement calculations should be performed. Several alternative analysis procedures are recommended by the Committee. These include simplified analysis of Newmark displacement using the procedures formulated by Makdisi and Seed (1978) or Bray and Rathje (1998), or formal Newmark analysis of sliding block displacements using appropriate integration techniques with applicable earthquake time histories. These procedures are well documented in the literature, and are summarized in Blake et al. (2002).

## SUMMARY AND CONCLUSIONS

In this paper, we have presented a screen analysis procedure for seismic slope stability that takes into account local variations in seismicity, as represented by the magnitude ($\overline{M}$) and distance ($\overline{r}$) that most significantly contribute to the ground motion hazard at a site. The screen procedure is based on a statistical relationship previously developed by Bray and Rathje (1998) between seismic slope displacement ($u$), peak amplitude of shaking in the slide mass ($k_{max}$), significant duration of shaking ($D_{5\text{-}95}$), and the ratio of slope resistance to peak demand ($k_y/k_{max}$). The screen is formulated to separate sites expected to undergo small to negligible slope deformation from sites where larger and more damaging slope movements are likely. Application of the screen is straightforward. Pseudo-static seismic coefficient $k$ is calculated using Equation 1, with the parameter $f_{eq}$ in Equation 1 evaluated using Figure 4 (or Equations 6–8) based on the site seismicity and the tolerable slope displacement. This seismic coefficient can then be used in a conventional slope stability analysis, with appropriate estimates of material strengths under dynamic loading conditions, to evaluate the factor of safety (FS). The site fails the screen for $FS<1$ and passes for $FS\geq 1$.

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