CHAPTER 14
SIMPLIFIED SEISMIC SLOPE DISPLACEMENT PROCEDURES

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Abstract. Simplified seismic slope displacement procedures are useful tools in the evaluation of the likely seismic performance of earth dams, natural slopes, and solid-waste landfills. Seismically induced permanent displacements resulting from earthquake-induced deviatoric deformations in earth and waste structures are typically calculated using the Newmark sliding block analogy. Some commonly used procedures are critiqued, and a recently proposed simplified procedure is recommended for use in engineering practice. The primary source of uncertainty in assessing the likely performance of an earth/waste structure during an earthquake is the input ground motion, so the proposed method is based on the response of several realistic nonlinear fully coupled stick-slip sliding block models undergoing hundreds of recorded ground motions. The calculated seismic displacement depends primarily on the ground motion's spectral acceleration at the degraded period of the structure and the structure's yield coefficient and fundamental period. Predictive equations are provided for estimating potential seismic displacements for earth and waste structures.

1. Introduction

The failure of an earth dam, solid-waste landfill, or natural slope during an earthquake can produce significant losses. Additionally, major damage without failure can have severe economic consequences. Hence, the potential seismic performance of earth and waste structures requires sound evaluation during design. Seismic evaluations of slope stability range from using relatively simple pseudostatic procedures to advanced nonlinear finite element analyses. Performance is best evaluated through an assessment of the potential for seismically induced permanent displacements. Following largely from the landmark paper of Newmark (1965) sliding block analyses are utilized as part of the seismic evaluation of the likely performance of earth and waste structures. Simplified Newmark-type procedures such as Makdisi and Seed (1978) are routinely used to provide a rough assessment of a system's seismic stability. Some of these procedures are critiqued in this paper, and a recently proposed simplified method for estimating earthquake-induced deviatoric deformations in earth and waste structures is summarized and recommended for use in practice.

K.D. Pitilakis (ed.), Earthquake Geotechnical Engineering, 327–353.

2. Seismic displacement analysis

2.1. CRITICAL DESIGN ISSUES

Two critical design issues must be addressed when evaluating the seismic performance of an earth structure. First, are there materials in the structure or its foundation that will 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. The soil liquefaction evaluation procedures in Youd et al. (2001) are largely used in practice; however, recent studies have identified deficiencies in some of these procedures. For example, the Chinese criteria should not be used to assess the liquefaction susceptibility of fine-grained soils. Instead, the recommendations of recent studies such as Bray and Sancio (2006) based on soil plasticity ($PI < 12$) and sensitivity ($w_c/LL > 0.85$) should be followed. Flow slides resulting from severe strength loss due to liquefaction of sands and silts or post-peak strength reduction in sensitive clays are not discussed in this paper.

Second, if materials within or below the earth structure will not lose significant strength as a result of cyclic loading, will the structure undergo significant deformations that may jeopardize satisfactory performance? The estimation of seismically induced permanent displacements allows an engineer to address this issue. This is the design issue addressed in this paper.

2.2. DEVIATORIC-INDUCED SEISMIC DISPLACEMENTS

The Newmark sliding block model captures that part of the seismically induced permanent displacement attributed to deviatoric shear deformation (i.e., either rigid body slippage along a distinct failure surface or distributed deviatoric shearing within the deformable sliding mass). Ground movement due to volumetric compression is not explicitly captured by Newmark-type models. The top of a slope can displace downward due to deviatoric deformation or volumetric compression of the slope-forming materials. However, top of slope movements resulting from distributed deviatoric straining within the sliding mass or stick-slip sliding along a failure surface are mechanistically different than top of slope movements that result from seismically induced volumetric compression of the materials forming the slope.

Although a Newmark-type procedure may appear to capture the overall top of slope displacement for cases where seismic compression due to volumetric contraction of soil or waste is the dominant mechanism, this is merely because the seismic forces that produce large volumetric compression strains also often produce large calculated displacements in a Newmark method. This apparent correspondence should not imply that a sliding block model should be used to estimate seismic compression displacements due to volumetric straining. There are cases where the Newmark method does not capture the overall top of slope displacement, such as the seismic compression of large compacted earth fills (e.g., Stewart et al., 2001). Deviatoric-induced deformation and volumetric-induced deformation should be analyzed separately by using procedures based on the sliding block model to estimate deviatoric-induced displacements and using other procedures (e.g., Tokimatsu and Seed, 1987) to estimate volumetric-induced seismic displacements.

The calculated seismic displacement from Newmark-type procedures, whether the procedure is simplified or advanced, is viewed appropriately as an index of seismic performance. Seismic displacement estimates will always be approximate in nature due to the complexities of the dynamic response of the earth/waste materials involved and the variability of the earthquake ground motion. However, when viewed as an index of potential seismic performance, the calculated seismic displacement can and has been used effectively in practice to evaluate earth/waste structure designs.

3. Components of a seismic displacement analysis

3.1. GENERAL

The critical components of a seismic displacement analysis are: (1) earthquake ground motion, (2) dynamic resistance of the structure, and (3) dynamic response of the potential sliding mass. The earthquake ground motion is the most important of these components in terms of its contribution to the calculation of the amount of seismic displacement. The variability in calculated seismic displacement is primarily controlled by the significant variability in the earthquake ground motion, and it is relatively less affected by the variability in the earth slope properties (e.g., Yegian et al., 1991b; Kim and Sitar, 2003). The dynamic resistance of the earth/waste structure is the next key component, and the dynamic response of the potential sliding mass is generally third in importance. Other factors, such as the method of analysis, topographic effects, etc., can be important for some cases. However, these three components are most important for a majority of cases. In critiquing various simplified seismic displacement procedures it is useful to compare how each method characterizes the earthquake ground motion and the earth/waste structure's dynamic resistance and dynamic response.

3.2. EARTHQUAKE GROUND MOTION

An acceleration-time history provides a complete definition of one of the many possible earthquake ground motions at a site. Simplified parameters such as the peak ground acceleration (*PGA*), mean period (T_m), and significant duration ($D_{5\text{–}95}$) may be used in simplified procedures to characterize the intensity, frequency content, and duration, respectively, of an acceleration-time history. Preferably, all three, and at least two, of these simplified ground motion parameters should be used. It is overly simplistic to characterize an earthquake ground motion by just its *PGA*, because ground motions with identical *PGA* values can vary significantly in terms of frequency content and duration, and most importantly in terms of its effects on slope instability. Hence, *PGA* is typically supplemented by additional parameters characterizing the frequency content and duration of the ground motion. For example, Makdisi and Seed (1978) use earthquake magnitude as a proxy for duration in combination with the estimated *PGA* at the crest of the embankment; Yegian et al. (1991b) use predominant period and equivalent number of cycles of loading in combination with *PGA*; and Bray et al. (1998) use the mean period and significant duration of the design rock motion in combination with its *PGA*.

Spectral acceleration has been commonly employed in earthquake engineering to characterize an equivalent seismic loading on a structure from the earthquake ground motion. Similarly, Travasarou and Bray (2003a) found that the 5% damped elastic spectral acceleration at the degraded fundamental period of the potential sliding mass was the optimal ground motion intensity measure in terms of efficiency and sufficiency (i.e., it minimizes the variability in its correlation with seismic displacement, and it renders the relationship independent of other variables, respectively, Cornell and Luco, 2001). The efficiency and sufficiency of estimating seismic displacement given a ground motion intensity measure were investigated for dozens of intensity measures. Other promising ground motion parameters included *PGA*, spectral acceleration (S_a), root mean square acceleration (a_{rms}), peak ground velocity (*PGV*), Arias intensity (I_a), effective peak velocity (*EPV*), Housner's response spectrum intensity (*SI*), and Ang's characteristic intensity (I_c). For period-independent parameters (i.e., no knowledge of the fundamental period of the potential sliding mass is required), Arias intensity was found to be the most efficient intensity measure for a stiff, weak slope, and response spectrum intensity was found to be the most efficient for a flexible slope.

No one period-independent ground motion parameter, however, was found to be adequately efficient for slopes of all dynamic stiffnesses and strengths. Spectral acceleration at a degraded period equal to 1.5 times the initial fundamental period of the slope (i.e., $S_a(1.5T_s)$) was found to be the most efficient ground motion parameter for all slopes (Travasarou and Bray, 2003a). An estimate of the initial fundamental period of the potential sliding mass (T_s) is required when using spectral acceleration, but an estimate of T_s is useful in characterizing the dynamic response aspects of the sliding mass (e.g., Bray and Rathje, 1998). Spectral acceleration does directly capture the important ground motion characteristics of intensity and frequency content in relation to the degraded natural period of the potential sliding mass, and it indirectly partially captures the influence of duration in that it tends to increase as earthquake magnitude (i.e., duration) increases. An additional benefit of selecting spectral acceleration to represent the ground motion is that spectral acceleration can be computed relatively easily due to the existence of several attenuation relationships and it is available at various return periods in ground motion hazard maps (e.g., http://earthquake.usgs.gov/research/hazmaps/).

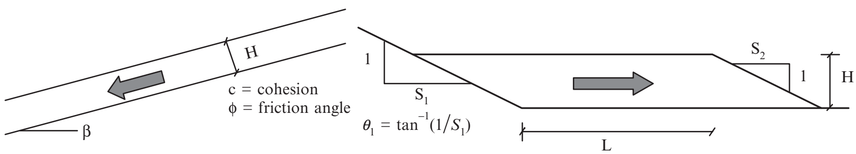
3.3. DYNAMIC RESISTANCE

The earth/waste structure's yield coefficient (k_y) represents its overall dynamic resistance, which depends primarily on the dynamic strength of the material along the critical sliding surface and the structure's geometry and weight. The yield coefficient parameter has always been used in simplified sliding block procedures due to its important effect on seismic displacement.

The primary issue in calculating k_y is estimating the dynamic strength of the critical strata within the slope. Several publications include extensive discussions of the dynamic strength of soil (e.g., Blake et al., 2002; Duncan and Wright, 2005; Chen et al., 2006), and a satisfactory discussion of this important topic is beyond the scope of this paper. Needless to say, the engineer should devote considerable resources and attention to developing realistic estimates of the dynamic strengths of key slope materials. In this paper, it is assumed that k_y is constant, so consequently, the earth materials do not undergo severe strength loss as a result of earthquake shaking (e.g., no liquefaction).

Duncan (1996) found that consistent (and assumed to be reasonable) estimates of a slope's static factor of safety (*FS*) are calculated if a slope stability procedure that satisfies all three conditions of equilibrium is employed. Computer programs that utilize such methods as Spencer, Generalized Janbu, and Morgenstern and Price may be used to develop sound estimates of the static *FS*. Most programs also allow the horizontal seismic coefficient that results in a $FS = 1.0$ in a pseudostatic slope stability analysis to be calculated, and if a method that satisfied full equilibrium is used, the estimates of k_y are fairly consistent. With the wide availability of these computer programs and their ease of use, there is no reason to use a computer program that incorporates a method that does not satisfy full equilibrium. Simplified equations for calculating k_y as a function of slope geometry, weight, and strength are found in Bray et al. (1998) among several other works. The equations provided in Figure 14.1 may be used to estimate k_y for the simplified procedures presented in this paper.

The potential sliding mass that has the lowest static *FS* may not be the most critical for dynamic analysis. A search should be made to find sliding surfaces that produce low k_y values as well. The most important parameter for identifying critical potential sliding masses for dynamic problems is k_y/k_{max}, where k_{max} is the maximum seismic coefficient, which represents the maximum seismic loading considering the dynamic response of the potential sliding mass as described next.



$$k_y = \tan(\phi - \beta) + \frac{c}{\gamma \cdot H \cdot \cos^2\beta \cdot (1+\tan\phi \cdot \tan\beta)}$$

$$FS_{static} = \frac{\tan\phi \cdot (S_1 \cdot H/2 \cdot \cos^2\theta_1 + L + S_2 \cdot H/2)}{\cos\theta_1 \cdot \sin\theta_1 \cdot S_1 \cdot H/2}$$

$$k_y = \frac{(FS_{static} - 1) \cdot \cos\theta_1 \cdot \sin\theta_1 \cdot S_1 \cdot H/2}{H \cdot (S_1 + S_2)/2 + L}$$

Fig. 14.1. Simplified estimates of the yield coefficient: (a) shallow sliding and (b) deep sliding

3.4. DYNAMIC RESPONSE

Research by investigators (e.g., Bray and Rathje, 1998) has found that seismic displacement also depends on the dynamic response characteristics of the potential sliding mass. With all other factors held constant, seismic displacements increase when the sliding mass is near resonance compared to that calculated for very stiff or very flexible slopes (e.g., Kramer and Smith, 1997; Rathje and Bray, 2000; Wartman et al., 2003). Many of the available simplified slope displacement procedures employ the original Newmark rigid sliding block assumption (e.g., Lin and Whitman, 1986; Ambraseys and Menu, 1988; Yegian et al., 1991b), which does not capture the dynamic response of the deformable earth/waste potential sliding mass during earthquake shaking.

As opposed to the original Newmark (1965) rigid sliding block model, which ignores the dynamic response of a deformable sliding mass, Makdisi and Seed (1978) introduced the concept of an equivalent acceleration to represent the seismic loading of a potential sliding mass (Figure 14.2) based on the work of Seed and Martin (1966). The horizontal equivalent acceleration (*HEA*)-time history when applied to a rigid potential sliding mass produces the same dynamic shear stresses along the potential sliding surface that is produced when a dynamic analysis of the deformable earth/waste structure is performed. The decoupled approximation results from the separate dynamic analysis that is performed assuming that no relative displacement occurs along the failure plane and the rigid sliding block calculation that is performed using the equivalent acceleration-time history from the dynamic response analysis to calculate seismic displacement.

Although the decoupled approximation of Makdisi and Seed (1978) inconsistently assumes no relative displacement in the seismic response analysis and then calculates a seismically induced permanent displacement, it has been judged by many engineers to provide a reasonable estimate of seismic displacement for many cases (e.g., Lin and Whitman, 1983; Rathje and Bray, 2000). However, it is not always reasonable, and it can lead to significant overestimation near resonance and some level of underestimation for cases where the structure has a large fundamental period or the ground motion is an intense near-fault motion. A nonlinear coupled stick-slip deformable sliding block model offers a more realistic representation of the dynamic response of an earth/waste structure by accounting for the deformability of the sliding mass and by considering the simultaneous occurrence of its nonlinear dynamic response and periodic sliding episodes

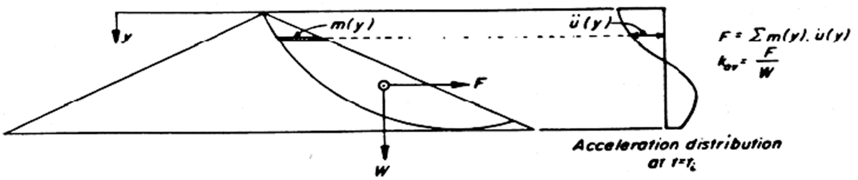


Fig. 14.2. Equivalent acceleration concept for deformable sliding mass (Seed and Martin, 1966)

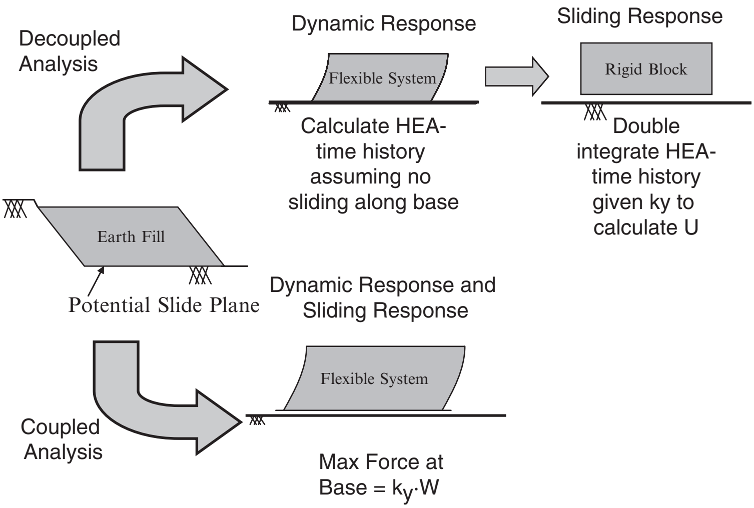


Fig. 14.3. Decoupled dynamic response/rigid sliding block analysis and fully coupled analysis

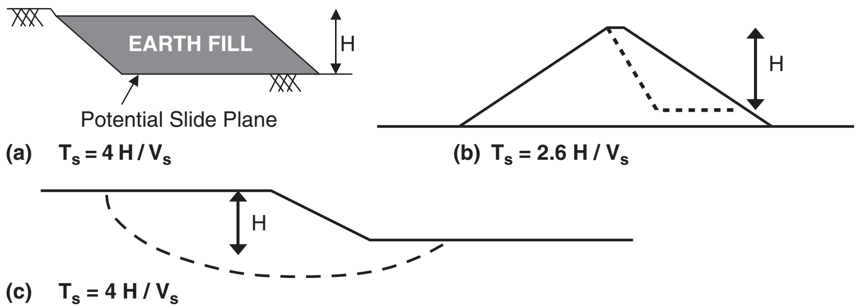


Fig. 14.4. Estimating the initial fundamental period of potential sliding blocks

(Figure 14.3). In addition, its validation against shaking table experiments provides confidence in its use (Wartman et al., 2003).

For seismic displacement methods that incorporate the seismic response of a deformable sliding block, the initial fundamental period of the sliding mass (T_s) can normally be estimated using the expression: $T_s = 4H/V_s$ for the case of a relatively wide potential sliding mass that is either shaped like a trapezoid or segment of a circle where its response is largely 1D (e.g., Rathje and Bray, 2001), where H = the average height of the potential sliding mass, and V_s is the average shear wave velocity of the sliding mass. For the special case of a triangular-shaped sliding mass that largely has a 2D response, the expression: $T_s = 2.6H/V_s$ should be used. Examples of the manner in which T_s should be estimated are shown in Figure 14.4.

4. Critique of some simplified seismic displacement methods

4.1. GENERAL

Comprehensive discussions of seismic displacement procedures for evaluating the seismic performance of earth/waste structures have been presented previously by several investigators (e.g., Makdisi and Seed, 1978; Seed, 1979; Lin and Whitman, 1983; Ambraseys and Menu, 1988; Yegian et al., 1991a, b; Marcuson et al., 1992; Jibson, 1993; Ambraseys and Srbulov, 1994; Bray et al., 1995; Ghahraman and Yegian, 1996; Kramer and Smith, 1997; Bray and Rathje, 1998; Finn, 1998; Jibson et al., 1998; Rathje and Bray, 2000; Stewart et al., 2003; Rathje and Saygili, 2006). There is not sufficient space in this paper to summarize and critique all pertinent studies. In this paper, some of the most commonly used simplified procedures for evaluating seismic displacement of earth and waste fills will be discussed with a focus on methods that do not assume that potential sliding mass is rigid.

4.2. SEED (1979) PSEUDOSTATIC SLOPE STABILITY PROCEDURE

First, several simplified pseudostatic slope stability procedures are commonly used in practice. They include Seed (1979) and the Hynes-Griffin and Franklin (1984). Both methods involve a number of simplifying assumptions and are both calibrated for evaluating earth dams wherein they assumed that <1 m of seismic displacement constituted acceptable performance. They should not be applied to cases where seismically induced permanent displacements of up to 1 m are not acceptable, which is most cases for evaluating base sliding of lined solid-waste landfills or houses built atop compacted earth fill slopes. Additionally, they provide a limited capability to assess seismic performance, because they do not directly address the key performance index of calculated seismic displacement.

The Seed (1979) pseudostatic slope stability method was developed for earth dams with materials that do not undergo severe strength loss that have crest accelerations less than $0.75\,g$. Using a seismic coefficient of 0.15 with appropriate dynamic strengths for the critical earth materials, performance is judged to be acceptable if $FS > 1.15$. The characteristics of the earthquake ground motion and the dynamic response of the potential slide mass to the earthquake shaking are represented by the seismic coefficient of 0.15 for all cases. Use of $FS > 1.15$ ensures that the yield coefficient (i.e., dynamic resistance of the earth dam) will be greater than 0.15 by an unknown amount. Thus, the earthquake ground motion and dynamic resistance and dynamic response of the earth dam are very simply captured in this approach, and the amount of conservatism involved in the estimate and the expected seismic performance is uncertain. An earth structure that satisfies the Seed (1979) recommended combination of seismic coefficient, *FS*, and dynamic strengths may displace up to 1 m, so satisfaction of this criteria does not mean the system is “safe” for all levels of performance.

4.3. MAKDISI AND SEED (1978) SIMPLIFIED SEISMIC DISPLACEMENT METHOD

The first step in the widely used Makdisi and Seed (1978) approach is the evaluation of the material's strength loss potential. They recommend not using their procedure if the loss of material strength could be significant. If only a minor amount of strength loss is likely, a slightly reduced shear strength, which often incorporates a 10% to 20% strength reduction from peak undrained shear strength, is recommended. The strength reduction is applied because of the use of a rigid, perfectly plastic sliding block model, wherein if peak strength was used the accumulation of nonlinear elasto-plastic strains for cyclic loads below peak would be significantly underestimated (i.e., zero vs. some nominal amount). Based on these slightly reduced best estimates of calibrated dynamic strengths and slope geometry and weight, k_y is then calculated in the second step.

In step three, the *PGA* that occurs at the crest of the earth structure is estimated. This is one of the greatest limitations of this method. As shown in Figure 14.5, which presents results of 1D SHAKE analyses of columns of waste placed atop a firm foundation for a number of ground motions, the *PGA* (or maximum horizontal acceleration, *MHA*) at the top of the landfill varies significantly. There is great uncertainty regarding what value of *PGA* to use. This is critical, because in the next step, the maximum seismic coefficient (k_{max}) is estimated as a function of the *PGA* at the crest and the depth of sliding below the crest. Thus, the uncertainty in the estimate of k_{max} is high, because the uncertainty

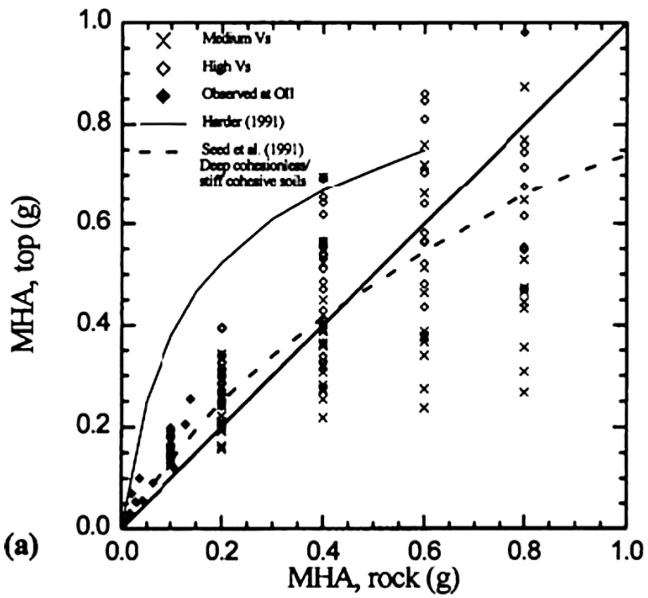


Fig. 14.5. Maximum horizontal acceleration at top of waste fill vs. *MHA* of rock base (Bray and Rathje, 1998)

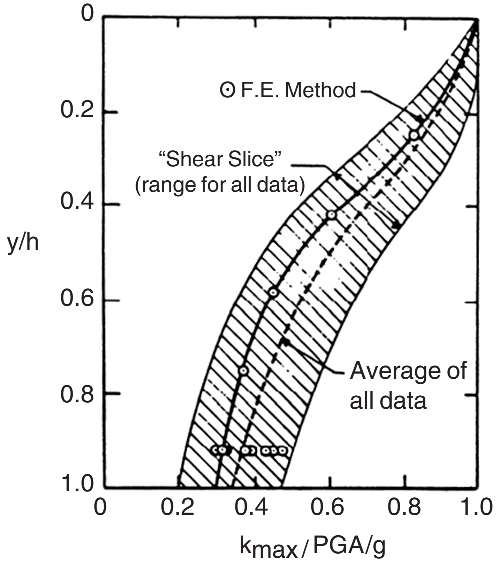


Fig. 14.6. Estimating seismic coefficient as a function of the peak acceleration at the crest and the depth of sliding (Makdisi and Seed, 1978)

in estimating the crest *PGA* is high. Even with advanced analyses, estimating the crest *PGA* is difficult, and the need to perform any level of dynamic analysis to estimate the crest *PGA* conflicts with the intent of a simplified method that should not require more advanced analysis.

Moreover, the bounds shown on the Makdisi and Seed (1978) plot of k_{max}/PGA vs. y/h (Figure 14.6) are not true upper or lower bounds. Stiff earth structures undergoing ground motions with mean periods near the degraded period of the earth structure can have k_{max} values exceeding 50% of the crest *PGA* for the base sliding case (i.e., $y/h = 1.0$), and flexible earth structures undergoing ground motions with low mean periods can have k_{max} values less than 20% of the crest *PGA* for base sliding.

When typically used in practice, the final step is to estimate seismic displacement as a function of the ratio of k_y/k_{max} and earthquake magnitude. Again the range shown in Figure 14.7 does not constitute the true upper and lower bounds of the possible seismic displacement, as only a limited number of earth structures were analyzed with a very limited number of input ground motions. As recommended by Makdisi and Seed (1978): "*It must be noted that the design curves presented are based on averages of a range of results that exhibit some degree of scatter and are derived from a limited number of cases. These curves should be updated and refined as analytical results for more embankments are obtained.*" Similar to how the Seed and Idriss (1971) simplified liquefaction triggering procedure was updated through Seed et al. (1985) and then Youd et al. (2001), it is time to update and move beyond the Makdisi and Seed (1978) design curves.

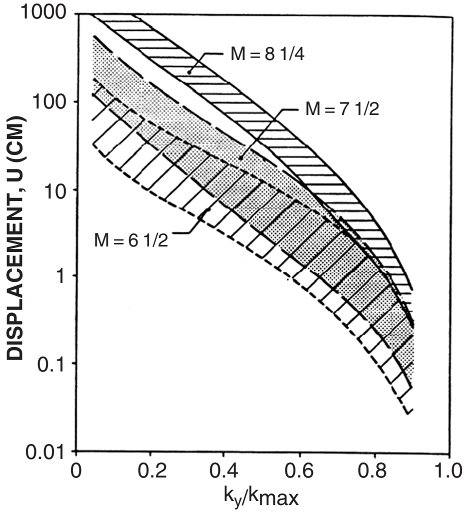


Fig. 14.7. Seismic displacement vs. *ky/kmax* and magnitude (Makdisi and Seed, 1978)

The Makdisi and Seed (1978) simplified seismic displacement method is one of the most significant contributions to geotechnical earthquake engineering over the past few decades. But as they recommended, their design curves should be updated as the profession advances. Since 1989, the number of recorded ground motions has increased dramatically. Thousands of well recorded ground motions are now available. The Makdisi and Seed (1978) work is based on a limited number of recorded and modified ground motions. Moreover, the important earthquake ground motion at a site is characterized by the *PGA* at the crest of the slope and earthquake magnitude. The *PGA* at the crest of the slope is highly variable and important frequency content aspects of the ground motion are not captured. The analytical method employed was relatively simple (e.g., primarily the shear slice method and a few equivalent-linear 2D finite element analyses). The decoupled approximation was employed, there is no estimate of uncertainty, and the bounds shown in the design curves are not true upper and lower bounds.

4.4. BRAY ET AL. (1998) SIMPLIFIED SEISMIC DISPLACEMENT APPROACH

The Bray et al. (1998) method is largely based on the work of Bray and Rathje (1998) which in turn follows on the works of Seed and Martin (1966), Makdisi and Seed (1978), and Bray et al. (1995). The methodology is based on the results of fully nonlinear decoupled one-dimensional D-MOD (Matasovic and Vucetic, 1995) dynamic analyses combined with the Newmark rigid sliding block procedure. To address the importance of the dynamic response characteristics of the sliding mass, six fill heights with three shear wave velocity profiles each with multiple unit weight profiles and two sets of strain-dependent shear modulus reduction and material damping relationships were used.

More importantly, taking advantage of the greater number of recorded earthquake ground motions available at the time, dozens of dissimilar scaled and unmodified recorded earthquake rock input motions were used with *PGA*s ranging from 0.2 g to 0.8 g. Their method was calibrated against several case histories of waste fill performance during the 1989 Loma Prieta and 1994 Northridge earthquakes, and later validated against observed earth fill performance.

The Bray et al. (1998) procedure provides a more comprehensive assessment of the earthquake ground motions, seismic loading, and seismic displacement calculations, but it requires more effort than the Makdisi and Seed (1978) procedure. In the first step, the ground motion is characterized by estimating the *MHA*, T_m, and $D_{5\text{–}95}$ for outcropping rock at the site given the assigned design moment magnitude and distances for the identified key potential seismic sources. The intensity, frequency content, and duration for the median earthquake ground motion level for deterministic events are estimated using several available ground motion parameter empirical relationships (e.g., Figure 14.8). The rock site condition is used, which is also consistent with the site condition used in the development of probabilistic ground motion hazard maps. Additionally, a seismic site response analysis is not required to estimate the *PGA* at the top of slope.

For the deep sliding case, the initial fundamental period of the potential sliding mass (T_s) is estimated as discussed previously (i.e., $T_s \approx 4H/V_s$). With the ratio of T_s/T_m, the normalized maximum seismic loading (i.e., $(MHEA)/((MHA_{rock})(NRF))$, where *MHEA* is the maximum horizontal equivalent acceleration and *NRF* is the nonlinear response factor) can be estimated with the graph shown in Figure 14.9, or the equation provided below, when $T_s/T_m > 0.5$

$$\ln(MHEA/(MHA_{rock}NRF)) = -0.624 - 0.7831\ \ln(T_s/T_m) \pm \varepsilon \qquad (14.1)$$

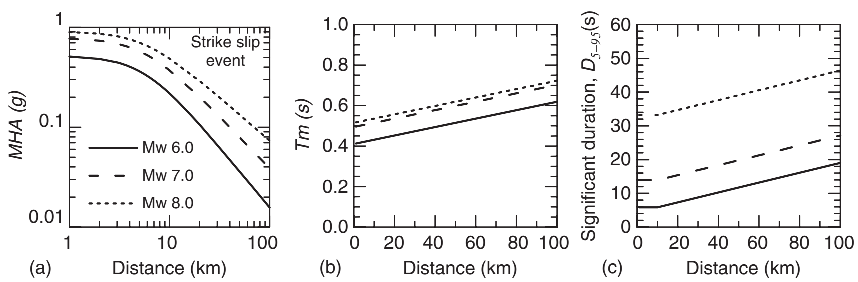


Fig. 14.8. Simplified characterization of earthquake rock motions: (a) intensity—*MHA* for strike-slip faults (for reverse faults, use 1.3*MHA for $M_w \geq 6.4$ and 1.64*MHA for $M_w = 6.0$, with linear interpolation for $6.0 < M_w < 6.4$) (Abrahamson and Silva, 1997), (b) frequency content—T_m (Rathje et al., 2004), and (c) duration—$D_{5\text{–}95}$ (Abrahamson and Silva, 1996)

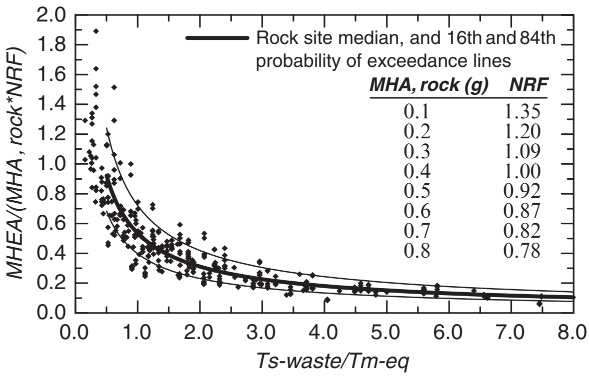


Fig. 14.9. Normalized maximum horizontal equivalent acceleration vs. normalized fundamental Period of sliding mass (Bray and Rathje, 1998)

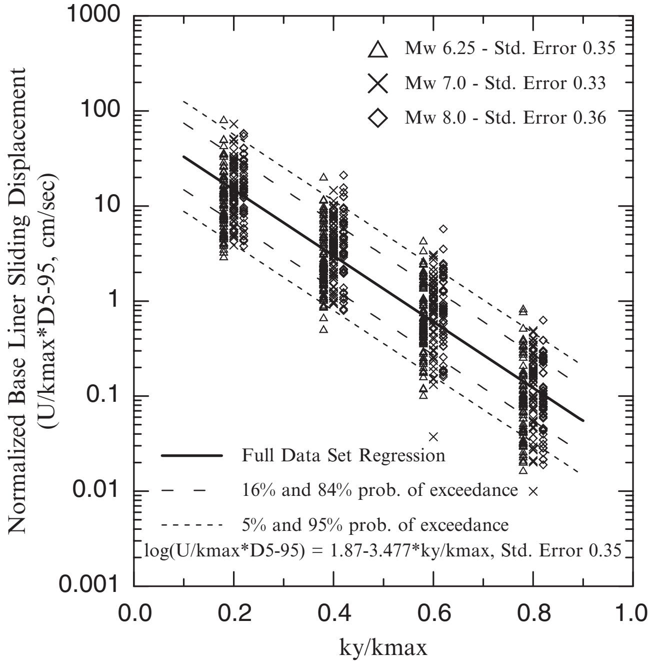


Fig. 14.10. Normalized base liner sliding displacements (from Bray and Rathje, 1998)

where $\sigma = 0.298$. The seismic coefficient $k_{max} = MHEA/g$. With an estimate of k_y, the normalized seismic displacement can be estimated as a function of k_y/k_{max} using Figure 14.10, or this equation

$$log_{10}(U/(k_{max}D_{5\text{–}95})) = 1.87 - 3.477(k_y/k_{max}) \pm \varepsilon \tag{14.2}$$

where $\sigma = 0.35$. The seismic displacement (U in cm) can then be estimated by multiplying the normalized seismic displacement value by the median estimates of k_{max} and $D_{5\text{–}95}$. The normalized seismic loading and displacement values are estimated at the median and 16% exceedance levels to develop a range of estimated seismic displacements.

The Bray et al. (1998) seismic slope displacement procedure provides median and standard deviation estimates of the seismic demand and normalized seismic displacement, but does so only in an approximate manner to develop a sense of the variability of the estimated displacement. It is limited in that it was not developed in a rigorous probabilistic manner. However, Stewart et al. (2003) were able to use this procedure to develop a probabilistic screening analysis for deciding if detailed project-specific seismic slope stability investigations are required by the 1990 California Seismic Hazards Mapping Act. Additionally, the Bray and Rathje (1998) simplified seismic displacement procedure was adopted in the guidance document by Blake et al. (2002) for evaluating seismic slope stability in conformance with the "Guidelines for Evaluating and Mitigating Seismic Hazards in California" (CDMG, 1997).

As noted previously, the Bray et al. (1998) method is also limited by the decoupled approximation employed in the seismic response and Newark sliding block calculations. Although many more ground motions were used than were used by Makdisi and Seed (1978), with the large number of well-recorded events since 1998, significantly more ground motions are now available. These shortcomings motivated a more recent study, which is summarized in the next section of this paper.

5. Bray and Travasarou (2007) simplified seismic displacement procedure

5.1. EARTHQUAKE GROUND MOTIONS

Currently available simplified slope displacement estimation procedures were largely developed based on a relatively modest number of earthquake recordings or simulations. This study took advantage of the recently augmented database of earthquake recordings, which provides the opportunity to characterize better the important influence of ground motions on the seismic performance of an earth/waste slope. As discussed previously, the uncertainty in the ground motion characterization is the greatest source of uncertainty in calculating seismic displacements.

The ground motion database used by Bray and Travasarou (2007) to generate the seismic displacement data comprises available records from shallow crustal earthquakes that occurred in active plate margins (PEER strong motion database ⟨http://peer.berkeley.edu/smcat/index.html⟩). These records conform to the following criteria: (1) $5.5 \leq M_w \leq 7.6$, (2) $R \leq 100$ km, (3) Simplified Geotechnical Sites B, C, or D (i.e., rock, soft rock/shallow stiff soil, or deep stiff soil, respectively, Rodriguez-Marek et al., 2001), and (4) frequencies in the range of 0.25 to 10 Hz have not been filtered out. Earthquake records totaling 688 from 41 earthquakes comprise the ground motion database for this study (see Travasarou, 2003 for a list of records used). The two horizontal components of each record were used to calculate an average seismic displacement for each side of the records, and the maximum of these values was assigned to that record.

5.2. DYNAMIC RESISTANCE OF THE EARTH/WASTE STRUCTURE

The seismic coefficient is calculated as described before using a computer program that has a slope stability method that satisfies all three conditions of equilibrium, or for preliminary analyses, a simplified estimate of k_y can be calculated using the equations provided previously in Figure 14.1.

5.3. DYNAMIC RESPONSE OF THE POTENTIAL SLIDING MASS

The nonlinear coupled stick-slip deformable sliding model proposed by Rathje and Bray (2000) for one-directional sliding was used by Bray and Travasarou (2007). The seismic response of the sliding mass is captured by an equivalent-linear viscoelastic modal analysis that uses strain-dependent material properties to capture the nonlinear response of earth and waste materials. It considers a single mode shape, but the effects of including three modes were shown to be small. The results from this model have been shown to compare favorably with those from a fully nonlinear D-MOD-type stick-slip analysis (Rathje and Bray, 2000), but this model can be utilized in a more straightforward and transparent manner. The model used is one-dimensional (i.e., a relatively wide vertical column of deformable soil) to allow for the use of a large number of ground motions with wide range of properties of the potential sliding mass in this study. One-dimensional (1D) analysis has been found to provide a reasonably conservative estimate of the dynamic stresses at the base of two-dimensional (2D) sliding systems (e.g., Vrymoed and Calzascia, 1978; Elton et al., 1991) and the calculated seismic displacements (Rathje and Bray, 2001). However, 1D analysis can underestimate the seismic demand for shallow sliding at the top of 2D systems where topographic amplification is significant. For this case, the seismic loading (which can be approximated by *PGA* for the shallow sliding case) can be amplified as recommended by Rathje and Bray (2001) for moderately steep slopes (i.e., $\sim$1.25 *PGA*) and as recommended by Ashford and Sitar (2002) for steep ($>60°$) slopes (i.e., $\sim$1.5 *PGA*).

The nonlinear coupled stick-slip deformable sliding model of Rathje and Bray (2000) can be characterized by: (1) its strength as represented by its yield coefficient, k_y, (2) its dynamic stiffness as represented by its initial fundamental period, T_s, (3) its unit weight, and (4) its strain-dependent shear modulus and damping curves. Seismic displacement values were generated by computing the response of the idealized sliding mass model with 10 values of its yield coefficient from 0.02 to 0.4 and with 8 values of its initial fundamental period from 0 to 2 s to the entire set of recorded earthquake motions described previously. Unit weight was set to 18 kN/m^3, and the Vucetic and Dobry (1991) shear modulus reduction and damping curves for a PI $=$ 30 material were used. For the baseline case, the overburden-stress corrected shear wave velocity (V_{s1}) was set to 250 m/s, and the shear wave velocity profile of the sliding block was developed using the relationship that shear wave velocity (V_s) is proportional to the fourth-root of the vertical effective stress. The sliding block height (H) was increased until the specified value of T_s was obtained. For common T_s values from 0.2 to 0.7 s, another reasonable combination of H and average V_s were used to confirm that the results were not significantly sensitive to these parameters individually. For nonzero T_s values, H varied between 12 and 100 m, and the average V_s was between 200 and 425 m/s. Hence, realistic values of the initial fundamental period and yield coefficient for a wide range of earth/waste fills were used.

5.4. FUNCTIONAL FORMS OF MODEL EQUATIONS

Situations commonly arise where a combination of earthquake loading and slope properties will result in no significant deformation of an earth/waste system. Consequently, the finite probability of obtaining negligible ("zero") displacement should be modeled as a function of the independent random variables. Thus, during an earthquake, an earth slope may experience "zero" or finite permanent displacements depending on the characteristics of the strong ground motion and the slope's dynamic properties and geometry. As discussed in Travasarou and Bray (2003b), seismically induced permanent displacements can be modeled as a mixed random variable, which has a certain probability mass at zero displacement and a probability density for finite displacement values. Displacements smaller than 1 cm are not of engineering significance and can for practical purposes be considered as negligible or "zero." Additionally, the regression of displacement as a function of a ground motion intensity measure should not be dictated by data at negligible levels of seismic displacement.

Contrary to a continuous random variable, the mixed random variable can take on discrete outcomes with finite probabilities at certain points on the line as well as outcomes over one or more continuous intervals with specified probability densities. The values of seismic displacement that are smaller than 1 cm are lumped to $d_0 = 1$ cm. The probability density function of seismic displacement is then

$$f_D(d) = \tilde{p}\delta(d - d_0) + (1 - \tilde{p})\tilde{f}_D(d) \tag{14.3}$$

where $f_D(d)$ is the displacement probability density function; $\tilde{p}$ is the probability mass at $D = d_0$; $\delta(d - d_0)$ is the Dirac delta function; and $\tilde{f}_D(d)$ is the displacement probability density function for $D > d_0$.

The predictive model for seismic displacement consists of two discrete steps. First, the probability of occurrence of "zero" displacement (i.e., $D \leq 1$ cm) is computed as a function of the primary independent variables k_y, T_s, and $S_a(1.5T_s)$. The dependence of the probability of "zero" displacement on the three independent variables is illustrated in Figure 14.11. The probability of "zero" displacement increases significantly as the yield coefficient increases, and decreases significantly as the ground motion's spectral acceleration at the degraded period of the slope increases. The probability of "zero" displacement decreases initially as the fundamental period increases from zero, because the slope is

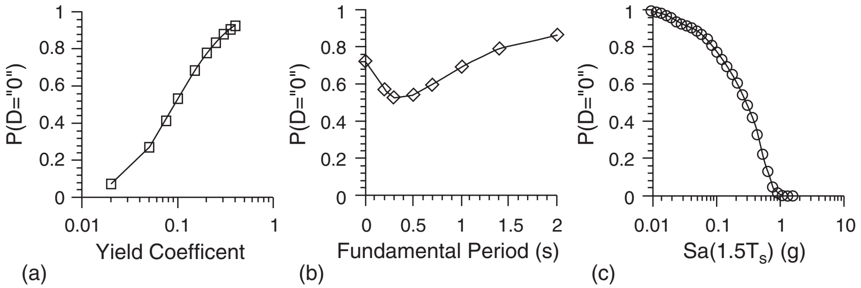


Fig. 14.11. Dependence of the probability of negligible displacement ($D < 1$ cm) on the (a) yield coefficient, (b) initial fundamental period, and (c) spectral acceleration at 1.5 times the initial fundamental period (Bray and Travasarou, 2007)

being brought near to the mean period of most ground motions. However, this probability increases sharply as the slope's period continues to increase as it is now moving away from the resonance condition. A probit regression model was used for this analysis (Green, 2003), and the selection of the functional form for modeling the probability of occurrence of "zero" displacement was guided by the trends shown in Figure 14.11.

In the case where a non-negligible probability of "nonzero" displacement is calculated, the amount of "nonzero" displacement needs to be estimated. A truncated regression model was used as described in Green (2003) to capture the distribution of seismic displacement, given that "nonzero" displacement has occurred. The estimation of the values of the model coefficients was performed using the principle of maximum likelihood.

5.5. EQUATIONS FOR ESTIMATING SEISMIC DEVIATORIC DISPLACEMENTS

As mentioned, the model for estimating seismic displacement consists of two discrete computations of: (1) the probability of negligible ("zero") displacement and (2) the likely amount of "nonzero" displacement. The model for computing the probability of "zero" displacement is

$$P(D = \text{“0”}) = 1 - \Phi\big(-1.76 - 3.22\ln(k_y) \\ -0.484(T_s)\ln(k_y) + 3.52\ln(S_a(1.5T_s))\big) \tag{14.4}$$

where $P(D = \text{“0”})$ is the probability (as a decimal number) of occurrence of "zero" displacements, D is the seismic displacement in the units of cm, Φ is the standard normal cumulative distribution function (i.e., NORMSDIST in Excel), k_y is the yield coefficient, T_s is the initial fundamental period of the sliding mass in seconds, and $S_a(1.5T_s)$ is the spectral acceleration of the input ground motion at a period of $1.5T_s$ in the units of g.

This first step can be thought of as a screening analysis. If there is a high probability of "zero" displacements, the system performance can be assessed to be satisfactory for the ground motion hazard level and slope conditions specified. If not, the engineer must calculate the amount of "nonzero" displacement (D) in centimeters using

$$\begin{aligned}\ln(D) = &-1.10 - 2.83\ln(k_y) - 0.333\left(\ln(k_y)\right)^2 + 0.566\ln(k_y)\ln(S_a(1.5T_s)) \\ &+ 3.04\ln(S_a(1.5T_s)) - 0.244\,(\ln(S_a(1.5T_s)))^2 \\ &+ 1.5T_s + 0.278(M-7) \pm \varepsilon \qquad (14.5)\end{aligned}$$

where: k_y, T_s, and $S_a(1.5T_s)$ are as defined previously for Eq. (14.4), and ε is a normally-distributed random variable with zero mean and standard deviation $\sigma = 0.66$. To eliminate the bias in the model when $T_s \approx 0$ s, the first term of Eq. (14.5) should be replaced with -0.22 when $T_s < 0.05$ s. Because the standard deviation of Eq. (14.5) is 0.66 and $\exp(0.66) \approx 2$, the median minus one standard deviation to median plus one standard deviation range of seismic displacement can be approximately estimated as half the median estimate to twice the median estimate of seismic displacement. Hence, the median seismic displacement calculated using Eq. (14.5) with $\varepsilon = 0$ can be halved and doubled to develop approximately the 16% to 84% exceedance seismic displacement range estimate.

The residuals of Eq. (14.5) are plotted in Figure 14.12 vs. some key independent variables. The residuals of displacement vs. magnitude, distance, and yield coefficient show no significant bias. There is only a moderate bias in the estimate at $T_s = 0$ and 2 s. The overestimation at 2 s is not critical, because it is rare to have earth/waste sliding masses with periods greater than 1.5 s, and Eq. (14.5) is conservative. However, the rigid body case (i.e., $T_s = 0$) can be important for very shallow slides, and Eq. (14.5) is unconservative for this case. The estimation at $T_s = 0$ s can be corrected by replacing the first term (i.e., -1.10) in Eq. (14.5) with -0.22. Hence, it is reasonable to use Eq. (14.5) for cases where T_s ranges from 0.05 to 2 s, and the first term of these equations should be replaced with -0.22 if $T_s < 0.05$ s.

It is often useful to establish a threshold displacement for acceptable seismic performance and then estimate the probability of this threshold displacement being exceeded.

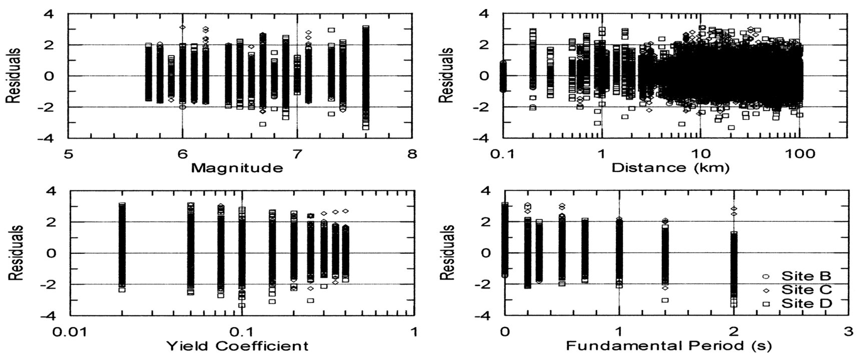


Fig. 14.12. Residuals ($\ln D_{\text{data}} - \ln D_{\text{predicted}}$) of Eq. (14.5) plotted vs. magnitude, rupture distance, the yield coefficient, and the initial fundamental period (Bray and Travasarou, 2007)

Additionally, often a range of expected seismic displacements is desired. The proposed methodology can be used to calculate the probability of the seismic displacement exceeding a selected threshold of displacement (d) for a specified earthquake scenario and slope properties. For example, consider a potential sliding mass with an initial fundamental period T_s, yield coefficient k_y, and an earthquake scenario that produces a spectral acceleration of $S_a(1.5T_s)$. The probability of the seismic displacement (D) exceeding a specified displacement threshold (d) is

$$P(D > d) = [1 - P(D = \text{“0”})] \cdot P(D > d|D > \text{“0”}) \tag{14.6}$$

The term $P(D = \text{“0”})$ is computed using Eq. (14.4). The term $P(D > d|D > \text{“0”})$ may be computed assuming that the estimated displacements are lognormally distributed as

$$P(D > d|D > \text{“0”}) = 1 - P(D \leq d|D > \text{“0”}) = 1 - \Phi\left(\frac{\ln d - \ln \hat{d}}{\sigma}\right) \tag{14.7}$$

where $\ln(\hat{d})$ is computed using Eq. (14.5) and $\sigma = 0.66$.

The trends in the Bray and Travasarou (2007) seismic displacement model are shown in Figures 14.13 and 14.14. For the $M_w = 7$ earthquake at a distance of 10 km scenario (i.e., Figures 14.13a,b), the importance of yield coefficient is clear. As yield coefficient increases, the probability of “zero” seismic displacement increases and the median estimate of nonzero displacement decreases sharply. The fundamental period of the potential sliding mass is also important, with values of T_s from 0.2 to 0.4 s leading to a higher likelihood of seismic displacement. For a $M_w = 7.5$ earthquake at different levels of

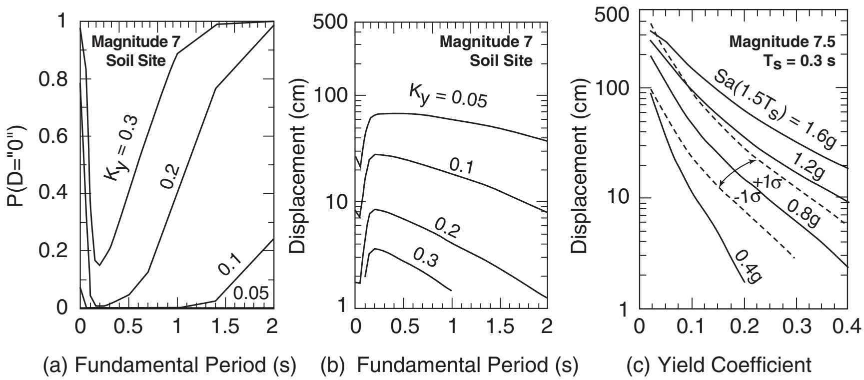


Fig. 14.13. Trends from the Bray and Travasarou (2007) model: (a) probability of negligible displacements and (b) median displacement estimate for a $M_w = 7$ strike-slip earthquake at a distance of 10 km, and (c) seismic displacement as a function of yield coefficient for several intensities of ground motion ($M_w = 7.5$) for a sliding block with $T_s = 0.3$ s

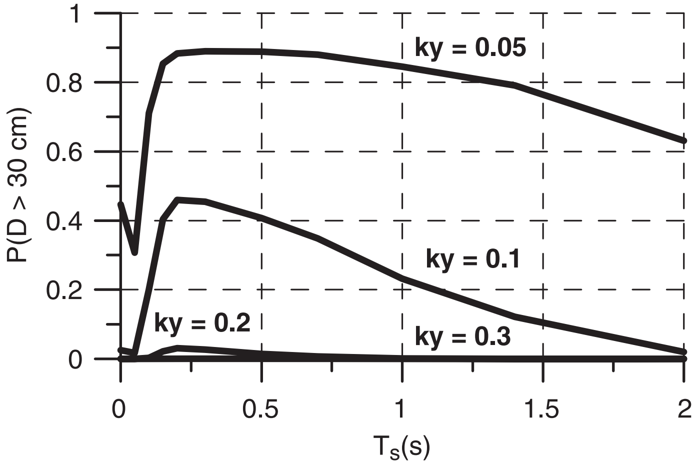


Fig. 14.14. Probability of exceeding 30 cm of seismic displacement for a $M_w = 7$ strike-slip earthquake at a distance of 10 km using Eq. (14.6) for selected k_y and T_s values

ground motion intensity at the degraded period of the sliding mass (Figure 14.13c), yield coefficient is again shown to be a critical factor, with large displacements occurring only for lower k_y values. Of course, the level of ground motion at a selected k_y value is also a dominant factor. The uncertainty involved in the estimation of seismic displacement for $S_a(0.45s) = 0.8\,g$ is shown to be approximately half to double the median estimate. Lastly, Eq. (14.6) was used with the results for the case presented in Figures 14.13a, b to calculate the probability of exceeding a selected threshold seismic displacement of 30 cm as shown in Figure 14.14.

5.6. MODEL VALIDATION AND COMPARISON

The Bray and Travasarou (2007) model was shown to predict reliably the seismic performance observed at 16 earth dams and solid-waste landfills that underwent strong earthquake shaking. Some of the case histories used in the model validation are presented in Table 14.1. In all cases, the maximum observed displacement (D_{max}) is that portion of the permanent displacement attributed to stick-slip type movement and distributed deviatoric shear within the deformable mass, and crest movement due to volumetric compression was subtracted from the total observed permanent displacement when appropriate to be consistent with the mechanism implied by the Newmark method. The observed seismic performance and best estimates of yield coefficient and initial fundamental period are based on the information provided in Bray and Rathje (1998), Harder et al. (1998), and Elgamal et al. (1990). Complete details regarding these parameters and pertinent seismological characteristics of the corresponding earthquakes can be found in Travasarou (2003).

Table 14.1. Comparison of the maximum observed displacement with three simplified methods

Earth Dam/Waste Fill	EQ	Obs. D_{max} (cm)	k_y	T_s (s)	$Sa(1.5T_s)$ (g)	Bray and Travasarou, 2007		Makdisi and Seed, 1978	Bray et al. 1998
						$P(D =$ "0")	D (cm)	D (cm)	D (cm)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
Pacheco Pass LF	LP	None	0.30	0.76	0.12	1.0	"0"	0	0
Marina LF	LP	None	0.26	0.59	0.30	0.9	"0"	0	0
Austrian Dam	LP	50	0.14	0.33	0.94	0.0	20–70	1–30	20–100
Lexington Dam	LP	15	0.11	0.31	0.78	0.0	15–65	0–10	30–110
Lopez Canyon C-B LF	NR	None	0.35	0.45	0.43	0.85	"0"	0	0
Chiquita Canyon C LF	NR	24	0.09	0.64	0.35	0.0	10–30	1–40	3–20
Sunshine Canyon LF	NR	30	0.31	0.77	1.40	0.0	20–70	0	0
OII Section HH LF	NR	15	0.08	0.00	0.24	0.1	4–15	3–30	2–25
La Villita Dam	S3	1	0.20	0.60	0.20	0.95	"0"	0	0
La Villita Dam	S5	4	0.20	0.60	0.41	0.25	"0"–10	0–1	0

LP: 1989 Loma Prieta; NR: 1994 Northridge; S3 and S5 from Elgamal et al. (1990)

The comparison of the simplified methods' estimates of seismic displacement (columns 8–10) with the maximum observed seismic permanent displacement (column 3) is shown in Table 14.1. For this comparison, only the best estimate of the slope's yield coefficient, its initial fundamental period, and the spectral acceleration at 1.5 times the initial fundamental period of the slope are considered. Hence, the computed displacement range is due to the variability in the seismic displacement given the values of the slope properties and the seismic load.

There are four cases shown in Table 14.1 in which the observed seismic displacement was noted as being "None," or $\leq$ 1 cm. For these cases, all of the simplified methods indicate that negligible displacements are expected (i.e., $D \leq 1$ cm), which is consistent with the good seismic performance observed of these earth/waste structures. Only the Bray and Travasarou (2007) method provides sufficient resolution to indicate the correct amount of observed displacement for La Villita Dam for Event S5 and that moderately more displacement should be expected for Event S5 instead of S3.

There are three cases of observed moderate seismic displacement for solid-waste landfills during the 1994 Northridge earthquake (i.e., $D_{max} = 15$–30 cm). For these cases, the Bray and Travasarou (2007) method indicates a very low chance of "zero" displacement occurring. Moreover, the observed seismic displacements are all within the ranges of the seismic displacement estimated by this method. The Makdisi and Seed (1978) and Bray et al. (1998) simplified methods provide reasonable, albeit less precise, estimates of the observed displacements for two of the cases, and both significantly underestimate the level of seismic displacement observed at the Sunshine Canyon landfill (i.e., both estimate 0 cm when 30 cm was observed).

Lastly, there are two cases of moderate seismic displacement of earth dams shaken by the 1989 Loma Prieta Earthquake. The Bray and Travasarou (2007) method provides refined and more accurate estimates of the observed seismic displacement due to deviatoric straining at these two dams than the other two simplified methods. The Bray and Travasarou (2007) screening equation clearly indicates that the likelihood of negligible (i.e., "zero") displacements is very low, and the 16% to 84% exceedance range for the nonzero displacement captures the observed seismic performance.

In judging these simplified methods, it is important to note that they provide predominantly consistent assessments of the expected seismic performance. However, the Bray and Travasarou (2007) method captures the observed performance better than existing procedures. Moreover, it is superior to the prevalent simplified seismic displacement methods, because it characterizes the uncertainty involved in the seismic displacement estimate and can be used in a probabilistic seismic hazard assessment.

5.7. ILLUSTRATIVE SEISMIC EVALUATION EXAMPLE

The anticipated performance of a representative earth embankment in terms of seismically induced permanent displacements is evaluated to illustrate the use of the Bray and

Travasarou (2007) simplified seismic displacement method. The earth fill is 30 m high and has a side slope of 2H:1V with a shape similar to that shown in Figure 14.4a. The embankment is located on a rock site at a rupture-distance of 12 km from a $M_w = 7.2$ strike-slip fault. A simplified deterministic analysis is performed to evaluate the potential movement of a deep slide through the base of the earth embankment.

The average shear wave velocity of the earth fill was estimated to be 300 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 $T_s = 4H/V_s = (4)(30\,\text{m})/(300\,\text{m/s}) \approx 0.4\,\text{s}$. The degraded period of the sliding mass is estimated to be 0.6 s (i.e., $1.5T_s = 1.5(0.4\,\text{s}) = 0.6\,\text{s}$). The yield coefficient for a deep failure surface was estimated to be 0.14 from a pseudostatic slope stability analyses performed with total stress undrained shear strength properties of $c = 10\,\text{kPa}$ and $\phi = 20°$ for the compacted earth fill.

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 attenuation relationships. Using Abrahamson and Silva (1997) and Sadigh et al. (1997) for the rock site condition for a strike-slip fault with $M_w = 7.2$ and $R = 12\,\text{km}$, $S_a(0.6s) = 0.44\,g$ and $0.52\,g$, respectively. Thus, the design value of S_a at the degraded period of sliding mass is $0.48\,g$, its initial fundamental period is 0.4 s, and k_y is 0.14.

The probability of "zero" displacement occurring is computed using Eq. (14.4) as

$$\begin{aligned} P(D = \text{“0”}) = 1 - \Phi\,(&-1.76 - 3.22\ln(0.14) \\ &-0.484(0.4)\ln(0.14) + 3.52\ln(0.48)) = 0.01 \end{aligned} \tag{14.8}$$

There is only a 1% probability of negligible displacements (i.e., $< 1\,\text{cm}$) occurring for this event. Hence, it is likely that non-negligible displacements will occur. The 16% and 84% exceedance values of seismic displacement can be estimated using Eq. (14.5) assuming that these values are approximately half and double the median estimate, respectively. The median seismic displacement is calculated using

$$\begin{aligned} \ln(D) = &-1.10 - 2.83\ln(0.14) - 0.333\,(\ln(0.14))^2 \\ &+ 0.566\ln(0.14)\ln(0.48) + 3.04\ln(0.48) - 0.244\,(\ln(0.48))^2 \\ &+ 1.50(0.4) + 0.278(7.2 - 7) = 2.29. \end{aligned} \tag{14.9}$$

The median estimated displacement is $D = \exp(\ln(D)) = \exp(2.29) \approx 10\,\text{cm}$, and the 16% to 84% exceedance displacement range is 5 to 20 cm. Thus, the seismic displacement due to deviatoric deformation is estimated to be between 5 and 20 cm for the design earthquake scenario. The direction of this displacement should be oriented parallel to the direction of slope movement, which will be largely horizontal for this case. For the total crest displacement of the embankment, a procedure such as Tokimatsu and Seed (1987) would be required to estimate the vertical settlement due to cyclic volumetric compression of the compacted earth fill.

6. Conclusions

A new simplified semi-empirical predictive model for estimating seismic deviatoric-induced slope displacements has been presented after critiquing a few other simplified seismic displacement methods for earth and waste structures. The Bray and Travasarou (2007) method is based on the results of nonlinear fully coupled stick-slip sliding block analyses using a comprehensive database of hundreds of recorded ground motions. The primary source of uncertainty in assessing the likely performance of an earth/waste system during an earthquake is the input ground motion, so this model takes advantage of the wealth of strong motion records that have recently become available. The spectral acceleration at a degraded period of the potential sliding mass ($S_a(1.5T_s)$) was shown to be the optimal ground motion intensity measure. The system's seismic resistance is best captured by its yield coefficient (k_y), but the dynamic response characteristics of the potential sliding mass is also an important influence, which can be captured by its initial fundamental period (T_s). This model captures the mechanisms that are consistent with the Newmark method, i.e., deviatoric-induced displacement due to sliding on a distinct plane and distributed deviatoric shearing within the slide mass.

The Bray and Travasarou (2007) method separates the probability of "zero" displacement (i.e., $\leq$ 1 cm) occurring from the distribution of "nonzero" displacement, so that very low values of calculated displacement that are not of engineering interest do not bias the results. The calculation of the probability of negligible displacement occurring using Eq. (14.4) provides a screening assessment of the likely seismic performance. If the likelihood of negligible displacements occurring is not high, then the amount of "nonzero" displacement is estimated using Eq. (14.5). The 16% to 84% exceedance seismic displacement range can be estimated approximately as half to twice the median seismic displacement estimate or this range can be calculated accurately using Eqs. (14.6) and (14.7). The first term of Eq. (14.5) is different for the special case of a nearly rigid Newmark sliding block.

The Bray and Travasarou (2007) seismic displacement model provides estimates of seismic displacements that are generally consistent with documented cases of earth dam and solid-waste landfill performance. It also provides assessments that are not inconsistent with other simplified methods, but does so with an improved characterization of the uncertainty involved in the estimate of seismic displacement. The proposed model can be implemented rigorously within a fully probabilistic framework for the evaluation of the seismic displacement hazard, or it may be used in a deterministic analysis. In all cases, however, the estimated range of seismic displacement should be considered merely an index of the expected seismic performance of the earth/waste structure.

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