Simplified Seismic Design Procedure for Geosynthetic-Lined, Solid-Waste Landfills

ABSTRACT: This paper critically reviews seismic design practices in light of the observed performance of landfills during recent earthquakes. Developments in these areas are summarized as follows: earthquake ground motions, dynamic waste fill properties, dynamic responses of geomembranes and their interfaces, nonlinear dynamic response analysis, and seismic stability evaluation. A newly developed simplified seismic analysis procedure that requires the most critical factors be addressed during a seismic performance evaluation is presented. The underlying seismic analysis procedure has been validated against observed performance of landfills shaken by the 1989 Loma Prieta and 1994 Northridge, California earthquakes. The procedure is comprehensive in that it requires: (i) characterization of the design bedrock motions in terms of intensity, frequency content, and duration; (ii) estimation of the seismic loading at the base and cover of the landfill; (iii) evaluation of performance in terms of seismically induced permanent deformations; and (iv) appropriate engineering judgment.

KEYWORDS: Analysis, Case record, Design, Earthquake, Geomembrane, Municipal solid-waste landfill, Seismic response, Seismic performance, Waste containment system.

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DATES: Original manuscript received 3 July 1997, revised version received 18 December 1997 and accepted 2 January 1998. Discussion open until 1 September 1998.

1 INTRODUCTION

Geosynthetics are a routine component of modern landfill design. The specification of geosynthetic components, such as geomembrane liners, in the waste containment system requires analysis of the overall response of the landfill. An important design consideration in seismic regions is the evaluation of the landfill's seismic performance. The current paper begins with a review of seismic design practice in light of observed landfill performance and, consequently, identifies issues relevant to the design of cover and base liner systems, which typically incorporate several layers of geosynthetics. The estimation of seismically induced deformations within these containment systems forms the basis of evaluating the likely performance of specific geosynthetic layers used in waste containment design. Hence, the current paper focuses on this issue.

The seismic performance of waste fills is typically evaluated using analytical methods developed for seismic stability evaluations of earth embankments, namely pseudostatic slope stability and seismically induced permanent deformation analyses. Although more sophisticated analytical procedures are available, such as coupled stick-slip finite element analysis (Gazetas and Uddin 1994), these procedures are not often utilized in current design due to the relative uncertainties of the inputs to an analysis (i.e., waste and liner properties and earthquake ground motions) compared to the limitations of commonly used analytical tools. Hence, much of the development in seismic design practices for municipal solid-waste landfills (MSWLFs) has been in the application of better understood analytical procedures, with attention to characterizing solid-waste and geosynthetic liner dynamic properties and evaluating the influence of earthquake ground motions on the likely seismic performance of MSWLFs.

Largely in response to U.S. Federal regulations (United States Environmental Protection Agency 1993, Subtitle D), seismic design procedures for MSWLFs in the United States have evolved rapidly over the past few years. In the current paper, recent developments are critically reviewed to summarize the current understanding of seismic design procedures for MSWLFs. Dynamic response and stability procedures are reexamined in light of recent observations of the seismic performance of solid-waste landfills during the 1994 Northridge, California, USA and the 1995 Hyogokken Nanbu (Kobe), Japan earthquakes. Specific improvements to current U.S. design procedures are proposed. A newly developed simplified seismic analysis procedure requiring that the most critical factors be addressed is presented. This procedure has been validated against the observed performance of landfills shaken by recent earthquakes. The use of this procedure is illustrated in the seismic evaluation of a typical MSWLF unit.

Although much progress has been made since the 1988 static failure of the Kettleman Hills Class I landfill in California, and a sound simplified seismic analysis procedure for MSWLFs can be proposed at this time, caution is warranted given the significant level of uncertainty remaining in this field. With additional investigations and possibly new lessons to be learned from future earthquakes, design practices will continue to evolve, and the issues explored in the current paper will require reevaluation.
2 DEVELOPMENTS IN SEISMIC DESIGN PRACTICES

2.1 General

The rapid evolution of seismic design practices for MSWLFs can be appreciated by reviewing previous state-of-the-art papers such as Seed and Bonaparte (1992), Bray et al. (1993), and Anderson and Kavazanjian (1995). All of the advancements resulting from previous investigations cannot be addressed in the current paper, but some of the key developments in seismic design practice, with an emphasis on those developed by the authors of the current paper, are discussed. This focused review also provides summary data required for the simplified design procedure presented in Section 3. Specifically, developments in these areas are summarized as follows: seismic landfill case histories, earthquake ground motions, dynamic waste fill properties, dynamic responses of geomembranes and their interfaces, nonlinear dynamic response analysis, and seismic stability evaluation.

2.2 Seismic Landfill Case Histories

Recent earthquake events have provided excellent opportunities to document the seismic performance of waste fills. The Northridge earthquake (moment magnitude, $M_w = 6.7$) is a particularly important event, as 22 landfills were subjected to ground motions in excess of 0.05g, and eight of these landfills were lined with geosynthetics (Augello et al. 1995). Surficial cracking in the cover soil, primarily near the transitions between the waste fill and natural ground areas and at changes in landfill geometry, was the most commonly observed damage pattern at landfills. This pattern of damage is consistent with damage observed after the 1989 Loma Prieta earthquake in California (Buranek and Prasad 1991). The cracking can be attributed to the contrast in the dynamic response characteristics between the relatively soft waste material and the stiff adjacent native ground. Cracking of the relatively brittle cover soil overlying more ductile waste fill was also observed at many landfills. Cracks were typically 10 to 75 mm wide with 10 to 75 mm of vertical offset. However, larger cracks (up to 300 mm) were observed at some landfills (e.g. Sunshine Canyon landfill in California).

Cracking of the soil cover due to limited downslope movement (typically less than 200 mm) was observed at the Chiquita Canyon landfill in California, where localized tears in the high density polyethylene (HDPE) liner of two cells of the landfill were observed. Two other geosynthetic-lined landfills (Bradley Avenue and Lopez Canyon landfills in California) at similar distances from the zone of energy release suffered no apparent damage to their liner systems. However, these two landfills did suffer moderate damage evidenced by cracking in the cover soil at waste fill/native ground transitions.

A temporary shut down of the landfill gas extraction systems occurred at a number of landfills due to the loss of power as a result of the earthquake, and breaks in the landfill gas system headers and lines were reported at several landfills. The temporary loss of a waste landfill’s gas extraction system is an important consideration because of the potential for a fire or an explosion.

A few solid-waste landfills located in the Kobe/Osaka area of Japan were surveyed following the 1995 Kobe earthquake (Akai et al. 1995). There were a number of waste fills setup to burn and dispose of construction debris being cleared from Kobe.
were located on reclaimed land that underwent significant liquefaction-induced lateral spreading. Two large permanent waste fills located on reclaimed islands at distances of 12 and 20 km from the zone of energy release were also surveyed. Several ground cracks, which were tens of meters long and a few centimeters wide, were identified near the edges of these islands. These cracks appeared to have resulted from lateral movements toward the island edges. A sand liquefaction boil with some garbage ejecta (Figure 1) was observed at one of the landfills. There was no evidence of any compromise of the containment of waste at these sites which are eventually underlain by thick clay deposits. Waste fill performance was dominated by the subsurface conditions at the surveyed landfills. These sites were reclaimed by dumping granular materials through standing water, and it is not surprising that they experienced ground cracking resulting from liquefaction-induced lateral spreading. These case records illustrate the potential problems associated with waste fills sited on ground susceptible to liquefaction.

2.3 Earthquake Ground Motions

The seismic response of a MSWLF depends greatly on the characteristics of the selected suite of input ground motions. Hence, the importance of a site-specific seismicity evaluation cannot be overemphasized. To assist this process in the United States, the U.S. Geological Survey (USGS) has developed improved maps that define the probability of ground motion parameters being exceeded for various levels of uncertainty in the continental United States. The Frankel et al. (1996) maps are available via the Internet (http://geohazards.cr.usgs.gov/eq/) and form the basis for several design codes, such as the 1997 National Earthquake Hazards Reduction Program Recommended Provisions for Seismic Regulations for New Buildings (BSSC, Building Seismic Safety Council 1997). Ground motion parameters, such as the maximum horizontal acceleration, MHA, are available at the 10% probability of exceedance in 250 years level (i.e. 2%
in 50 years) cited in Subtitle D, as well as other levels such as the 10% exceedance in 50 years. The USGS maps provide a useful check to site-specific seismicity evaluations as well as reasonable values for some designs.

Although these map values may be appropriate for some designs, a site-specific seismicity study is often warranted. For example, the use of one ground motion parameter (e.g. \( MHA \)) as a design basis is overly simplistic. Subtitle D is potentially misleading, with its focus on \( MHA \) as describing the seismic hazard at a site, because the frequency content and duration of a ground motion are equally important.

The potential seismic hazard at a site typically results from several earthquake events, each with their particular characteristics and potential impact on the landfill. The hazard often consists of a near-field (< 10 to 20 km), high intensity ground motion with significant short period energy and a short duration of strong shaking and an intermediate-field (20 to 100 km), lower intensity ground motion (i.e. lower \( MHA \) value) with significant long period energy and long duration. The design of MSWLFs, which are typically longer period systems, will often be governed by a less intense motion with significant long period energy and a long duration of strong shaking.

In developing a suite of design ground motions that capture reasonable scenario earthquake events, empirical attenuation relations are helpful for characterizing ground motions with respect to magnitude, distance, site conditions, and fault type, as well as other factors. Many updated relations are available in the *Seismological Research Letters* (1997).

Although the most complete characterization of a ground motion is through its time history, it is often useful to characterize key aspects of a ground motion with simplified parameters. Probabilistic ground motion maps and attenuation relationships are useful for estimating \( MHA \) as a ground motion intensity parameter (Figure 2a). The frequency content may be characterized by using the mean period, \( T_m \), which is a more stable parameter than the predominant period, \( T_p \), with the relationship shown in Figure 2b (Rathje et al. 1998), and the significant duration, \( D_{5-95} \) (i.e. the time between 5 and 95% of the Arias Intensity of the acceleration-time history), may be estimated with the relationship shown in Figure 2c (Abrahamson and Silva 1996). A number of other ground motion issues may be important for a particular project, for example, near-field directivity effects at sites close to a major fault (e.g. Somerville et al. 1997). The reader may refer to the seismological literature for a discussion of these issues.

### 2.4 Dynamic Waste Fill Properties

Recent studies have increased the state of knowledge regarding shear wave velocities, \( V_s \), of solid-waste fill. A large number of spectral analysis of surface waves (SASW) tests and one suspension logging test were performed at the Operating Industries, Inc. (OII) landfill as a part of its closure study (Idriss et al. 1995). Additionally, SASW tests were performed at five other landfills in Southern California (Kavazanjian et al. 1996). SASW measurements were taken at 43 profiles at these six sites. A weighting scheme was employed such that equal weighting was given to each landfill site. Figure 3 presents a median curve along with a recommended range for the shear wave velocity of solid-waste versus depth. This range of values is somewhat lower than the \( V_s \) profile recommended by Kavazanjian et al. (1995), but it is still significantly higher than the range of \( V_s \) values initially proposed (Singh and Murphy 1990).
A unit weight profile based upon direct measurements of initial weight upon placement; in situ measurements from boreholes and test trenches; inferred values from SASW measurements based on a correlation between depth, \( V_s \), and calibrated unit weights from borehole data; and one-dimensional (1-D) compression tests on large (754 mm) reconstituted samples was also recently developed (Augello et al. 1997). The unit weight profile is 11 kN/m\(^3\) at a depth of 0 m, 14 kN/m\(^3\) at 24 m, and 15 kN/m\(^3\) at and beyond 90 m.

The installation of a pair of accelerometers at the OII landfill in 1987 provided a unique opportunity to evaluate the shear modulus reduction and damping characteristics of solid-waste through back-analysis. Several investigators (e.g. Idriss et al. 1995;
GeoSyntec Consultants (1996; Augello et al. 1998) have proposed shear modulus reduction and damping relationships based upon the OII landfill recordings. Figure 4 shows the various shear modulus reduction and damping curves proposed for municipal solid-waste. In general, analytical results are more sensitive to the choice of damping curves compared to the choice of shear modulus reduction curves. The Poisson’s ratio of waste fill was found to be approximately 0.3. The dynamic strength of waste fill was back-calculated to be at least equal to the static strength recommended by Kavazanjian et al. (1995) (i.e. cohesion intercept, \( c = 24 \text{ kPa} \) and internal friction angle, \( \phi = 0^\circ \) for a total normal stress, \( \sigma_n < 30 \text{ kPa} \); and \( c = 10 \text{ kPa} \) and \( \phi = 33^\circ \) for \( \sigma_n > 30 \text{ kPa} \)), and a reasonable back-calculated range of dynamic friction angles for the Southern California landfills is 33 to 38° (Augello et al. 1997).

### 2.5 Dynamic Responses of Geomembranes and Geomembrane Interfaces

Geomembranes made of HDPE, polypropylene (PP), and polyvinyl chloride (PVC) are often used as low-permeability layers within MSWLF units. The primary function of a geomembrane is to minimize the migration of leachate through the base liner or the infiltration of water through the cover system. Geomembranes, unlike geogrids, are rarely designed as structural members that carry loads. Rather, the desired response for a geomembrane is to deform without failure. To maintain system integrity, the ability of a geomembrane to elongate in response to the design loads, including additional seismic forces, without failure is of paramount importance. For design calculations, the stress-strain response of a geomembrane is often based on wide-strip tension tests performed at a specified strain rate of typically, 1%/minute.
These numerical models, when used to extrapolate standard test results to higher strain rates, suggest that the failure strain of HDPE reduces significantly under rapid loadings. This potential increase in brittleness of HDPE at strain rates representative of those expected during seismic events warrants concern, and experimental data are required to verify this predicted response.

Multiaxial tests have been performed on 1.0 and 1.5 mm thick, smooth HDPE geomembrane specimens by inducing a static strain of 5% and then loading rapidly to fail-
ure (on the order of 5000%/minute or 5 Hz loading to 15% strain). These results indicate that the failure strain for commercially available, untextured HDPE geomembranes is at least 10% (Merry 1995). In addition, cyclic multiaxial tension test results indicate that HDPE geomembranes can undergo cyclic loadings at less than their peak strength without losing ductility and strength (Figure 5b).

Cyclic wide-strip uniaxial tests were also performed by the authors of the current paper, and these tests indicated that the failure strain for untextured HDPE geomembranes was at least 15%. Moreover, some HDPE geomembrane specimens were loaded between 5 and 20% strain at a frequency of 1 Hz for 500 cycles without rupturing; however, there was significant modulus degradation in these tests. Geomembranes may possess sufficient ductility to deform rapidly during an earthquake without failing, but additional experimental work is warranted to verify this finding.

**Figure 5.** Stress-strain-time multiaxial response of a smooth HDPE geomembrane: (a) monotonic strain-controlled loading; (b) quasi-cyclic loading (adapted from Merry 1995).
This is an important issue, because two of the few MSWLF units that were lined with a HDPE geomembrane as required by Subtitle D were damaged as a result of strong shaking during the 1994 Northridge earthquake (Augello et al. 1995). Tears in the geomembrane liners at Canyons C and D of the Chiquita Canyon landfill both occurred at the top of the slope near the anchor trench, where the largest static, pre-seismic stresses in the liner would be expected due to side slope downdrag as the waste fill settled over time. Moreover, both 1.5 mm (60 mil) thick, smooth HDPE geomembrane liners were directly overlain with protective soil and waste, without the inclusion of a protective geotextile or slip layer, and the tear at Canyon C initiated at the location of an extrusion welded patch along a longitudinal seam where a sample was removed for destructive testing (EMCON Associates 1994).

From this experience, the incorporation of flexibility and ductility in the design of the containment system is judged to be crucial, even though a quantitative assessment of its merit is difficult with the available analytical tools. “Defects” and “inflexible anchor trenches” which may induce stress concentrations during static and seismic loadings should be avoided, and a slip layer should be placed above the base/side slope liner to accommodate slip in the system without undermining the integrity of the critical base liner.

The 1988 static failure of the Kettleman Hills Waste Landfill Unit B-19 (Mitchell et al. 1990) emphasized the importance of evaluating the interface strengths of the many different types of geosynthetics incorporated in waste containment designs. As with the response of geomembranes, most studies have investigated the static response of common interfaces (e.g. Negussey et al. 1989; O’Rourke et al. 1990; Byrne et al. 1992; Orman 1994; Gilbert et al. 1995). Pseudo-static stability analyses of geosynthetic-lined landfills during the Northridge earthquake resulted in back-calculated interface strengths for smooth and textured HDPE geomembranes that are consistent with those from laboratory test results for similar interfaces reported in the literature (Augello et al. 1995). Shaking table tests have provided important insights regarding the dynamic response of common geosynthetic interfaces. Yegian and Lahla (1992) found that the dynamic friction angles measured during instrumented shaking table tests are close to those measured in static friction tests. Hence, conventional, static interface friction tests are believed to provide reasonable estimates of the dynamic interface strengths.

The clay-smooth HDPE geomembrane interface has been found to be one of the most variable, which emphasizes the need for high-quality, project-specific interface testing. The clay-geomembrane interface strength is dependent on the compaction water content, compacted density, normal stress, soil type, and post-compaction changes in water content (Seed and Boulanger 1991; Stark and Poeppel 1994). Residual clay-geomembrane interface strengths as low as 4° have been measured. Peak strengths are higher, but peak strengths are often mobilized at shear displacements of only 2 to 4 mm, with the shear stress falling to close to residual strength at displacements of only 20 mm (Stark and Poeppel 1994). However, the clay-geomembrane interface is not necessarily the weakest interface. Stark and Poeppel (1994) show that under low normal stresses (up to approximately 150 to 300 kPa), geosynthetic-geosynthetic interfaces, such as geotextile-geomembrane, may be weaker. For higher normal stresses, the clay-geomembrane interface strength is likely lower. This suggests that in the case of a base liner system where normal stresses may be high, the use of a geotextile cushion as a sacrificial slip surface does not necessarily prevent stresses from becoming induced in the geomembrane.
The engineer must still decide if peak or residual (large displacement) interface strengths are appropriate. A sacrificial “slip layer” interface should be designed using residual strength, as some (and maybe significant) slippage is assumed to occur along this interface. Installation procedures, post-installation environmental changes, sequencing of layers within the base, cover and side slope liner systems, and the stress-deformation response of layered systems to placement of the waste all may contribute to relative displacement within specific interfaces. These contributing factors, and the fact that peak strengths are mobilized at small displacements, suggest that the use of large displacement (residual) strength values for seismic stability analyses is appropriate.

2.6 Nonlinear Dynamic Response Analysis

Parametric seismic response studies (Bray et al. 1995; Bray and Rathje 1998) have found that reasonable variations in waste fill dynamic properties, fill heights, foundation conditions, and input bedrock motions produce significant variations in the landfill response. Moreover, developing good characterizations of waste fill properties (dynamic strength, stiffness, and damping) remains a top priority, as the response of a MSWLF unit for a given suite of motions is largely a function of its dynamic response characteristics.

The seismic loading for a potential sliding mass within the waste fill can be represented by the horizontal equivalent acceleration as

$$ HEA = \left( \frac{\tau_h}{\sigma_v} \right) g $$

where $\tau_h$ = horizontal shear stress and $\sigma_v$ = total vertical stress at the depth of the sliding surface which was originally conceived by Seed and Martin (1966) and defined for a 1-D system. The maximum seismic loading is designated $M_{HEA}$. Using the 1-D equivalent-linear wave propagation program SHAKE91 (Idriss and Sun 1992), Bray et al. (1995) found that $M_{HEA}$ for the important base sliding case depends primarily on the dynamic properties and height of the waste fill (i.e. its fundamental period, $T_s$, as described by $T_s = 4H/V_s$, where $H$ = height of waste fill, and $V_s$ = average initial shear wave velocity of the waste fill) and the $M_{HA}$ and $T_p$ of the input earthquake rock motion.

For cases where $T_{\text{fill}} > T_{p-EQ}$, the normalized maximum horizontal equivalent acceleration, $M_{HEA}/M_{HA_{\text{rock}}}$, was shown to be inversely proportional to the normalized fundamental period of the waste fill, $T_{\text{fill}}/T_{p-EQ}$. This finding reemphasizes the importance of characterizing the dynamic stiffness of waste fill, which is in sharp contrast to the design practices of the early 1990s described in Seed and Bonaparte (1992) where a site response analysis at a project site without the waste fill in place was often used to estimate the seismic loading.

Using the fully nonlinear 1-D site response program D-MOD (Matasovic and Vucetic 1995), which is based on the established DESRA-2 program (Lee and Finn 1978), Bray and Rathje (1998) reexamined this normalization at higher levels of acceleration (up to 0.8g) with updated waste fill property characterizations. D-MOD and other DESRA-based codes have been shown to calculate seismic responses similar to the established SHAKE91 program at low acceleration levels, but calculate lower responses at high accelerations (Kavazanjian and Matasovic 1995), which is consistent with prevailing views (e.g. Seed et al. 1991; UBC, International Conference of Building Officials 1997).

To achieve a useful normalization, the nonlinear response factor ($NRF = M_{HA_{\text{site}}} / M_{HA_{\text{rock}}}$) was developed to account for the nonlinear site response of materials, such as solid-waste, that exhibit strain-dependent shear modulus and damping characteristics. The nonlinear response factor for solid-waste is based on the site factor proposed
by Seed et al. (1991) for deep cohesionless and stiff cohesive soils, because the range in dynamic stiffness of this site category is close to that of waste fill. Additionally, the mean period, $T_m$, was used in lieu of the predominant period, because this parameter better represents the overall frequency content of a ground motion and can be estimated more reliably with empirical ground motion data (Rathje et al. 1998).

Incorporating these two improvements, a revised normalization was developed (Figure 6), with data from over 300 analyses of landfills at rock sites (Bray and Rathje 1998). The results follow a well-defined trend, except near the resonance condition which occurs at $T_s/T_m < 1$, which is due to degrading waste stiffness at higher strain levels. Results for landfills located at soil sites indicate that site conditions play a less significant role in the estimate of $M_{HEA}$, with slightly lower $M_{HEA}$ values for soil sites. In an extreme case, $M_{HEA}$ values at deep, soft clay sites were about two-thirds of the $M_{HEA}$ values calculated at rock sites, but this apparent conservatism for soft sites is balanced by the larger displacements calculated at these sites, and the implications of these compensating errors will be discussed in Section 2.7.1. Figure 6 may be used as a guide in the selection of an appropriate seismic coefficient for simplified pseudo-static and deformation analyses, as this graph has been prepared with normalization parameters that may be estimated for most projects.

The use of a 1-D model to represent the seismic response of an earth/waste fill has been discussed in Vrymoed and Calzascia (1978), Elton et al. (1991), and Bray et al. (1996), and it has been found that dynamic shear stresses near the base of a two-dimensional (2-D) earth/waste fill can be approximated reasonably well with a 1-D analysis. This issue, as well as the reliability of capturing a landfill’s cover response with a 1-D analysis, was investigated using the programs SHAKE91 and QUAD4M (Hudson et al 1994) to analyze five generic landfill configurations and five landfills shaken by the Northridge earthquake (Rathje 1997).

Figure 6. Normalized maximum horizontal equivalent acceleration for base sliding versus normalized fundamental period of waste fill (adapted from Bray and Rathje 1998).
In these comparisons, the \( MHEA \) or \( MHA \) from the 1-D analyses was calculated for several 1-D columns and weighted by their respective percentage of the total sliding mass. Results shown in Figure 7a indicate that the seismic loading predicted by 1-D analyses for deep sliding surfaces within the waste are generally conservative when compared to 2-D results. Therefore, the seismic coefficient estimated by a weighted 1-D analysis can be used in a simplified seismic stability evaluation of base sliding.

While a 1-D analysis also provides a rough estimate of the \( MHA \) acting along the top deck of a landfill, it tends to underestimate accelerations at points along the cover slope, especially at the crest, because of topographic amplification (Figure 7a). However, the \( MHA \) calculated at the crest of the cover slope need not be applied as the seismic loading

\[ \text{Figure 7. Results from SHAKE91 and QUAD4M: (a) comparison of seismic loading predicted by 1-D and 2-D analyses; (b) variation of MHEA along the cover system.} \]
for the stability evaluation of the entire cover slope. As shown in Figure 7b, the equivalent acceleration acting on a potential cover sliding mass decreases as the normalized length of the slide mass increases, due to incoherence of the seismic response of the long cover slope.

One-dimensional nonlinear dynamic analyses were performed to identify factors affecting the seismic loading for cover systems. At low input rock accelerations ($MH_{\text{Rock}} < 0.2g$), the D-MOD results are consistent with accelerations recorded at the OII landfill and the scatter is small; however, at higher accelerations, the scatter increases and cover acceleration estimates are less reliable (Bray and Rathje 1998). This scatter is a result of different landfill heights and stiffnesses, and different input ground motions. The normalization used for $MHE_{\text{Top}}$ was applied to the $MHA_{\text{Top}}$ results (Figure 8), and this normalization allows for a better estimate of the seismic loading at the top of a landfill.

2.7 Seismic Stability Evaluation

2.7.1 Seismically Induced Permanent Deformations

The seismic stability of a MSWLF unit can be best evaluated in terms of seismically induced deformations. Deformations should be expressed in terms of deviatoric and volumetric components. The overall response of a cover system will be a function of seismically induced fill settlement due to contractive volumetric strains and seismically induced fill bulging/lurching toward a slope-face due to deviatoric strains.

Simplified techniques, such as the Tokimatsu and Seed (1987) method for clean sand deposits, however, are not available for evaluating seismically induced settlements of MSWLFs. Hence, this analysis is not typically performed, although many of the surface cracks observed in landfill covers following the 1994 Northridge earthquake have been attributed to this mechanism (Augello et al. 1995). Simplified techniques, such as Mak-
disi and Seed (1978), are available to evaluate displacements that may occur due to sliding along a distinct, rigid-perfectly plastic slip surface. Hence, Newmark (1965)-type deformation analyses constitute the basis for design of most MSWLF units. The assumptions involved in this approach are actually more reasonable for slippage along a geosynthetic interface within a Subtitle D base/cover liner system than for the homogeneous earth embankment for which it was originally developed (Bray et al. 1995). Hence, its use is judged to be reasonable for geosynthetically lined landfills. Although this procedure is also often used to evaluate sliding within the waste fill, sliding is not likely to occur along a distinct shear plane for this case, thus its use requires a calibration for this application. The results from a Newmark-type seismically induced permanent deformation analysis actually provides only an index of likely performance during an earthquake.

Seismically induced permanent displacements can be calculated using a procedure developed by Franklin and Chang (1977) requiring two inputs: (i) horizontal equivalent acceleration-time history based on the horizontal shear stress-time history computed at the depth of sliding from a dynamic response analysis; and (ii) yield acceleration coefficient, $k_y$, calculated as the seismic coefficient required to obtain a factor of safety of one in a pseudo-static slope stability analysis. The yield acceleration coefficient at the cover (infinite slope analysis, Equation 1) and base (adaptation from Shewbridge 1996, Equation 2) can be estimated for typical landfills (Figure 9) using the following formulas:

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

$$k_y = \frac{(FS_{static} - 1) \cos \theta_1 \sin \theta_1 S_i H/2}{H(S_i + S_j)/2 + L}$$  \hspace{1cm} (2)

$$FS_{static} = \frac{\tan \phi(S_i H/2 \cos^2 \theta_1 + L + S_2 H/2)}{\cos \theta_1 \sin \theta_1 S_i H/2}$$  \hspace{1cm} (3)

$$\theta_1 = \tan^{-1} (1/S_i)$$  \hspace{1cm} (4)

where: $\beta$ = slope angle of the cover measured from the horizontal; $\gamma$ = total unit weight of the cover soil; $c$ = soil cohesion; $\phi$ = internal friction angle; $FS_{static}$ = static factor of safety; $\theta_1$ = back-slope geometry parameter; $S_i$ = back-slope run to height ratio; $S_i$ = front-slope run to height ratio; and $L$ = length of the midsection of the landfill.

Use of the equivalent acceleration allows for the seismic response of the deformable potential sliding mass to be represented in the Newmark rigid sliding block procedure (Makdisi and Seed 1978). However, a potentially important limitation of this procedure is that the seismic response of the potential sliding mass is decoupled from the subsequent double integration of the computed equivalent acceleration-time history. A number of investigators have examined the limitations of this assumption (e.g. Lin and Whitman 1983; Chopra and Zhang 1991; Gazetas and Uddin 1994; Kramer and Smith 1997), and for the most part, have arrived at similar conclusions. For example, Lin and Whitman (1983) concluded that “the errors arising from the decoupled assumption are insignificant compared to other uncertainties involved in the use of the sliding block analogy.”
Some investigators have noted that occasionally the decoupling assumption was not conservative. This was reexamined by the authors of the current paper. A decoupled analysis generally provides an estimate of the coupled sliding displacement within a factor of three (Bray and Rathje 1998). Conservative decoupled displacements are calculated for lower yield acceleration coefficient values (i.e. \( k_y / k_{max} < 0.6 \)) when the fundamental period of the sliding mass is less than twice the mean period of the input ground motion (Figure 10). For systems with larger values of \( k_y / k_{max} \) or large fundamental periods, a decoupled analysis may predict smaller displacements than a fully coupled analysis. However, in these cases, the displacements calculated from both analyses are generally small (i.e. less than a few centimeters), because of the relatively low \( k_{max} \) and high \( k_y / k_{max} \) values for these cases. As a seismic performance index, a decoupled analysis is judged to be reasonable for current design practice. Conversely, a rigid sliding block analysis (i.e. direct integration of the input acceleration-time history) can be significantly unconservative, and it should not be used (Figure 10).

Seismically induced permanent displacements, \( U \), for the base sliding case for 19 landfill configurations sited on rock undergoing 33 input rock motions were calculated at \( k_y / k_{max} \) ratios of 0.2, 0.4, 0.6, and 0.8 (i.e. 309 values at each \( k_y / k_{max} \) ratio) with the conventional procedure discussed previously (Bray and Rathje 1998). As expected, there was considerable scatter both with respect to results from different input motions and results from different landfill configurations. For example, for a given input motion, significantly larger displacements are calculated for landfill configurations with stiffer response characteristics that more closely match the high frequency motions contained in most rock records. At a specified \( k_y / k_{max} \) ratio, the calculated displacement is roughly proportional to \( MHEA \), indicating that those factors that have been shown to affect \( MHEA \), also affect \( U \).

Considerably less inter- and intra-earthquake scatter was produced by normalizing the calculated displacement, \( U \), by the maximum seismic coefficient, \( k_{max} (= MHEA / g) \) and the significant duration, \( D_{3-95} \), of the input motion (Figure 11). Hence, an estimate...
of earthquake-induced displacement can be made given an estimate of the intensity, $MHA$, frequency content, $T_m$, and duration, $D_{5-95}$, of the design rock motion, and the dynamic response characteristics, $T_s$, and strength, $k_y$, of the landfill. The maximum
Figure 11. Normalized base liner sliding displacements (from Bray and Rathje 1998).

Seismically induced displacements were found to vary systematically with site conditions, with generally larger calculated displacements at deep, soft sites (Bray and Rathje 1998). However, the larger displacements calculated at soil sites at a specified $k_i/k_{max}$ ratio are offset by the seismic loading results discussed previously which indicate that soil sites produce lower $k_{max}$ values for identical rock motions and landfill configurations. As the seismic loading and seismic displacement calculations vary predictably with site conditions in opposite and nearly equal proportions, simplified analysis based on the rock site results presented in Figures 6, 11, and 12 can be used for all site conditions without introducing significant errors in the final displacement estimate.

Alternatively, a pseudo-static slope stability analysis may be performed with a seismic coefficient calibrated with the preceding seismically induced permanent displace-
ment results. Analytical results indicate that for a wide range of landfill heights, waste fill properties, and input rock motions, the calculated base sliding permanent displacements for the rock site case are less than approximately 150 mm (with about a 10% probability of exceedance) when the yield acceleration coefficient is 60% of $k_{max}$ (Bray and Rathje 1998). Hence, 60% of $k_{max}$ (= MHEA/g) may be used as the seismic coefficient in pseudo-static analyses of base sliding in combination with conservative dynamic strength properties and a factor of safety of one. However, the engineer must decide whether 150 mm of permanent displacement along the landfill/liner interface is tolerable.

Geosynthetic and soil materials comprising the base liner and cover systems are susceptible to damage by seismically induced deformations that can produce tensile strains in the materials. Geosynthetic material performance can be evaluated in terms of its yield strain compared to induced tensile strains (e.g. Koerner 1998). In evaluating the induced tensile strain in a geosynthetic layer, the zone over which strains can accumulate may be small (Zornberg 1994). Hence, small deformations may damage or even rupture a geomembrane. The compacted clay liner’s integrity can be assessed in terms of the potential for shear or tensile zones to develop within the layer (e.g. Singh 1992). Finally, transient stresses and deformations within the waste containment system may affect performance; however, validated analytical procedures to estimate these are not currently available.
2.7.2 Comparison of Analytical Results to Observed Performance

Analytical results presented in the current paper (e.g. Figures 6 and 11) were used to estimate the expected range of earthquake-induced permanent displacements for a number of California landfills that were shaken by the 1989 Loma Prieta and 1994 Northridge earthquakes (Bray and Rathje 1998). The calculations and observations are summarized in Table 1.

Table 1. Back-analysis of seismic landfill performance (from Bray and Rathje 1998).

<table>
<thead>
<tr>
<th>Landfill</th>
<th>EQ</th>
<th>$M_{HA}$ (g)</th>
<th>$T_m$ (s)</th>
<th>$D_{5-95}$ (s)</th>
<th>$T_s$ (s)</th>
<th>$k_y$</th>
<th>$k_{max}$ ($\mu +1\sigma$)</th>
<th>$U$ (mm)</th>
<th>Observed damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Buena Vista</td>
<td>LP</td>
<td>0.40</td>
<td>0.6</td>
<td>13</td>
<td>0.64</td>
<td>0.26$^1$</td>
<td>0.17/0.23</td>
<td>None</td>
<td>No cracking$^1$</td>
</tr>
<tr>
<td>Guadalupe</td>
<td>LP</td>
<td>0.45</td>
<td>0.6</td>
<td>13</td>
<td>0.64</td>
<td>0.20$^1$</td>
<td>0.24/0.32</td>
<td>3 to 40</td>
<td>Minor downslope movement$^1$</td>
</tr>
<tr>
<td>Pacheco Pass</td>
<td>LP</td>
<td>0.30</td>
<td>0.6</td>
<td>14</td>
<td>0.76</td>
<td>0.30$^1$</td>
<td>0.16/0.21</td>
<td>None</td>
<td>No cracking$^1$</td>
</tr>
<tr>
<td>Marina</td>
<td>LP</td>
<td>0.21</td>
<td>0.6</td>
<td>15</td>
<td>0.59</td>
<td>0.26$^1$</td>
<td>0.12/0.16</td>
<td>None</td>
<td>No cracking$^1$</td>
</tr>
<tr>
<td>Zanker Road</td>
<td>LP</td>
<td>0.19</td>
<td>0.6</td>
<td>15</td>
<td>0.56</td>
<td>0.14$^1$</td>
<td>0.07/0.10</td>
<td>None</td>
<td>No cracking$^1$</td>
</tr>
<tr>
<td>Lopez Canyon C-A</td>
<td>NR</td>
<td>0.42</td>
<td>0.5</td>
<td>11</td>
<td>0.64</td>
<td>0.27</td>
<td>0.18/0.25</td>
<td>None</td>
<td>No cracking$^2$</td>
</tr>
<tr>
<td>Lopez Canyon C-B</td>
<td>NR</td>
<td>0.42</td>
<td>0.5</td>
<td>11</td>
<td>0.45</td>
<td>0.35</td>
<td>0.24/0.33</td>
<td>None</td>
<td>No cracking$^2$</td>
</tr>
<tr>
<td>Chiquita Canyon C</td>
<td>NR</td>
<td>0.33</td>
<td>0.6</td>
<td>11</td>
<td>0.64</td>
<td>0.09</td>
<td>0.18/0.24</td>
<td>30 to 200</td>
<td>240 mm tear in HDPE$^2$</td>
</tr>
<tr>
<td>Chiquita Canyon D</td>
<td>NR</td>
<td>0.33</td>
<td>0.6</td>
<td>11</td>
<td>0.64</td>
<td>0.10</td>
<td>0.18/0.24</td>
<td>20 to 150</td>
<td>300 mm tear in HDPE$^2$</td>
</tr>
<tr>
<td>OII Section HH</td>
<td>NR</td>
<td>0.11</td>
<td>0.6</td>
<td>16</td>
<td>0.47</td>
<td>0.14</td>
<td>0.10/0.13</td>
<td>none</td>
<td>50 to 150 mm cover cracks$^2$</td>
</tr>
<tr>
<td>OII Section HH$^3$</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>0.81</td>
<td>0.08</td>
<td>0.13/0.16</td>
<td>20 to 250</td>
</tr>
<tr>
<td>Sunshine Canyon</td>
<td>NR</td>
<td>0.46</td>
<td>0.5</td>
<td>11</td>
<td>0.77</td>
<td>0.31</td>
<td>0.17/0.23</td>
<td>none</td>
<td>20 to 300 mm cover cracks$^2$</td>
</tr>
<tr>
<td>Sunshine Canyon$^2$</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>1.3</td>
<td>0.25</td>
<td>0.30/0.37</td>
<td>2 to 50</td>
<td>20 to 300 mm cover cracks$^2$</td>
</tr>
</tbody>
</table>

Notes: $^1$ Observations of damage and $k_y$ values are from Buranek and Prasad (1991). $^2$ Observations of damage are from Augello et al. (1995). $^3$ Analyses performed for shallow sliding along the cover system using Figure 12. EQ = earthquake event; NR = Northridge; LP = Loma Prieta. $\mu$ = median, $\sigma$ = standard error.
The yield acceleration coefficient recommended by Buranek and Prasad (1991) was used for the Loma Prieta earthquake case records, and $k_y$ was calculated with the 2-D limit equilibrium slope stability analysis program UTEXAS3 (Wright 1990) for the Northridge earthquake cases using the strength values recommended by Augello et al. (1995). The median and 16% probability of exceedance $k_{max}$ values were estimated using Figure 6 based on the median estimates of $T_s$, $MHA_{Rock}$, and $T_m$ shown. The calculated displacements, $U$, were then estimated using the median and 16% probability of exceedance lines shown in Figure 11.

The calculated earthquake-induced displacements are consistent with the magnitude of permanent displacements observed at these landfills. Generally, no cracking was observed when the calculated displacement was zero, and some distress was observed for cases when the calculated displacement was nonzero. As the calculated earthquake-induced displacement should be viewed as merely an index of earthquake performance, the proposed procedure is judged to provide an index of expected seismic performance that correlates reasonably well with the actual seismic performance of these landfills.

3 SIMPLIFIED SEISMIC ANALYSIS PROCEDURE

3.1 General

A two stage, simplified seismic analysis procedure is outlined in this section. This procedure is based on the analytical results discussed in Section 2 which have been validated against the observed performance of lined and unlined solid-waste landfill units shaken by the 1989 Loma Prieta and 1994 Northridge earthquakes. Although “simplified”, the procedure is comprehensive in that it requires that the most important factors be addressed during a seismic performance evaluation, namely: (a) characterization of the design bedrock motions in terms of intensity, frequency content, and duration; (b) estimation of the seismic loading at the base and cover of the landfill; (c) evaluation of performance in terms of seismically induced permanent deformations; and (d) application of appropriate engineering judgment. A straightforward, conservative screening analysis is incorporated as the first stage of the procedure, and situations that warrant more sophisticated analysis are identified. The screening analysis employs a reasonable level of conservatism by including the following: amplification factors in the range of 1.2 to 1.5 (see Figures 6 and 8 at $T_s/T_m < 0.75$); a value of $NRF = 1$ which is appropriate for the larger values in Figures 6 and 8; and a seismic coefficient calibrated using Figures 11 and 12 (i.e. $k = 0.55k_{max}$) for calculated seismically induced displacements generally less than 150 and 300 mm for base and cover sliding, respectively (i.e. 10% probability of exceedance).

3.2 Screening Analysis

Note that very deep, soft clay sites and sites susceptible to liquefaction (UBC Site Category $S_f$) are not applicable to this screening analysis. Also, unstable geologic site conditions, such as native slope instability and surface faulting, should be assessed separately. The following is an outline of the screening analysis:
1. Estimate $MHA_{rock}$ from the applicable ground motion maps (e.g., Frankel et al. 1996 USGS maps for the continental USA) based on an appropriate probability level (generally 2% exceedance in 50 years for Subtitle D).

2. Use $k = 0.75$ ($MHA_{rock}/g$) as the seismic coefficient for base and cover sliding.

3. Calculate $k_s$ for the base and cover using conservative strengths (e.g., residual) with Equations 2 and 1, respectively.

4. If $k_s \geq k$, the calculated seismically induced displacements will generally be less than 150 and 300 mm for base and cover sliding, respectively. The engineer must decide if these commonly accepted displacement levels are tolerable.

3.3 Simplified Procedure

Unstable geologic conditions, such as liquefaction, soft clay instability, native slope instability, and surface faulting should be assessed first. This simplified procedure is not appropriate for landfills with potentially unstable foundations.

3.3.1 Characterize Ground Motions

Estimate $MHA$, $T_m$, and $D_{50}$ for each design earthquake event.

(a) Review the regional and site geology and seismicity, and identify potential earthquake sources.

(b) Assign the earthquake magnitude and distance to each earthquake source. Select representative earthquakes: likely, a high-intensity, short-duration, near-field event (higher $MHA$, lower $T_m$, and lower $D_{50}$), and a lower-intensity, long-duration, intermediate-field event (lower $MHA$, higher $T_m$, and higher $D_{50}$).

(c) Use rock relationships (Figure 2) to estimate $MHA_{rock}$, $T_m$, and $D_{50}$ at the site. Check the design $MHA$ values against the USGS maps for the 475 year and 2,375 year earthquake events (i.e., 10 and 2% exceedance in 50 years, respectively.)

3.3.2 Develop Seismic Loading

Estimate $MHEA$ of waste column for base sliding and $MHA$ at top of landfill for cover stability assessment.

(a) Estimate $MHEA_{base}$ of the waste column for base liner sliding using the plot of $MHEA_{base}/[(MHA_{rock})(NRF)]$ versus $T/T_m$ (Figure 6). Record the median and 16% probability of exceedance values. From Section 3.3.1, $MHA_{rock}$ and $T_m$ have been estimated. Estimate $NRF$ from Figure 6. Estimate $T_s = 4H/V_s$, where $H$ = waste height, and $V_s$ = weighted average initial shear wave velocity of waste. Estimates of $V_s$ for solid-waste are available in Figure 3. Figure 6 may also be used to estimate $MHEA$ at intermediate depths within the waste fill if $T_s$ is calculated using the height and the average initial shear wave velocity of the potential sliding surface.

(b) Estimate $MHA_{top}$ using the plot of $MHA_{top}/[(MHA_{rock})(NRF)]$ versus $T/T_m$ (Figure 8). Use the median and 16% probability of exceedance values. For localized cover sliding near the crest and along the slope of the landfill, use $MHEA_{cover} = 1.25$
MHATop to account for 2-D effects. For seismic loading of the entire cover slope, use $MHEA_{cover} = 0.65 \ MHA_{top}$ to account for incoherence.

3.3.3 Calculate Seismic Stability

Calculate Newmark seismic displacements (or calibrated pseudo-static $FS$).

(a) Seismically induced permanent deformation analysis:
1. Calculate the yield acceleration coefficient, $k_y$, for potential sliding masses (i.e. seismic coefficient that results in a pseudo-static $FS = 1.0$). Estimate $k_y$ at the cover and base for generalized landfill systems using Equation 1 and 2, respectively. Otherwise, use a limit equilibrium slope stability program which has a method that satisfies full equilibrium, such as Spencer, Morgenstern and Price, or Generalized Janbu. The unit weight and strength of the municipal solid-waste and geosynthetic interface strengths can be estimated using the summary data provided in Sections 2.4 and 2.5.
2. Base sliding: Estimate the displacements, $U$, at the base given the estimates of $k_y/k_{max}$, $k_{max}$, and $D_{5-95}$ (Figure 11). Note that $k_{max} = MHEA_{base}/g$ for base sliding. Use the median and 16% probability of exceedance lines.
3. Cover sliding: Estimate the displacements, $U$, at the cover given the estimate of $k_y/k_{max}$ and $M_w$ (Figure 12). Use $k_{max} = MHEA_{cover}/g$, with the appropriate $MHEA_{cover}$ value for either localized cover sliding or sliding along the entire cover.
4. Other cases: Estimate range of displacements, $U$, from Figure 11, given the $MHEA$ estimate from Figure 6 where $H$ is equal to the depth of sliding.

(b) Alternative calibrated base and cover pseudo-static stability analyses:
Select the limiting displacement level with the corresponding reduction factor, $R$, for base and cover sliding, with $R = k_y/k_{max}$ at the selected displacement. Use $R_b = 0.6, 0.8$, or 1.0 for the limiting cover displacements of < 300 mm, < 75 mm, or < 20 mm, respectively. Use $R_b = 0.6, 0.8$, or 1.0 for limiting base displacements of < 150 mm, < 50 mm, or < 10 mm, respectively. Apply $(R_b)(MHEA_{cover})/g$ as the seismic coefficient, $k$, to a shallow cover failure mass and calculate the pseudo-static $FS$. For base sliding, use $k = (R_b)(MHEA_{base})/g$. If $FS > 1$, displacement is likely to be less than the corresponding limiting displacement value (i.e. there is a 10% probability of exceedance).

3.3.4 Evaluate Seismic Stability

Given the seismic displacement estimates (i.e. small (< 25 to 50 mm), moderate (< 150 to 300 mm), or large (> 0.3 to 1 m) displacements), evaluate the ability of sensitive landfill waste containment components to accommodate this level of deformation. Penetrations (e.g. vents and leachate sump risers) and changes in geometry (e.g. side liner to base liner transition) are potentially vulnerable. Cover systems are more easily repaired than base systems, and hence, they have a less stringent design level. The consequences of failure and conservatism of the hazard assessment and stability analysis are important design considerations. Defensive measures that “shield” critical components
from damage, such as a sacrificial low-friction slip surface to concentrate deformations away from critical components, are useful. Anderson and Kavazanjian (1995) and Bray et al. (1993) give guidance with regard to evaluating seismic stability, but considerable engineering judgment is still required.

This simplified approach is most appropriate when the waste fill dynamic response characteristics indicate that it will not be close to a resonance condition with the input motions (i.e. $T_s > T_m$). When close to resonance, one would need to incorporate significant conservatism with the simplified approach, and this may not be economical. A site-specific dynamic analysis could be used to calculate the base and cover equivalent acceleration-time histories, and these time histories could be double integrated to calculate displacements, $U$, at selected values of $k_y$ to construct a plot of $U$ versus $k_y$. Such a plot is useful in evaluating a landfill’s potential seismic performance.

### 3.3.5 Example

**Base sliding.** Analyze the base sliding stability for a 35 m high landfill founded on rock for a $M_w = 7$ strike-slip earthquake at a distance of 12 km. Slope stability analyses indicate that the yield acceleration coefficient for sliding along the base liner is 0.15.

1. Estimate the median $MHA$, $T_m$, and $D_{5-95}$ values of the rock ground motion:

   $MHA_{Rock} = 0.33g$ (Figure 2a)
   $T_m = 0.52$ s (Figure 2b)
   $D_{5-95} = 14$ s (Figure 2c)

2. Calculate the seismic loading, $MHEA_{base}$. An average $V_s$ profile might be approximately 120 m/s at the waste surface, approximately 275 m/s at a depth of 30 m, and approximately 375 m/s at a depth of 60 m. Hence, a reasonable weighted average of $V_s$ for a 35 m high waste fill would be approximately 200 m/s, with $T_s = 4H/V_s = 0.7$ s, and $T_s/T_m = 1.3$:

   $MHEA_{Base}/[(MHA_{Rock})(NRF)] = 0.43$ to $0.58$ (50% / 16% exceedance) (Figure 6)
   
   $NRF = 1.05$ for $MHA_{Rock} = 0.33g$ (Figure 6)
   
   $MHEA_{Base} = (0.33g)(1.05)(0.43$ to $0.58) = 0.15g$ to $0.20g$

3. Estimate the seismically induced permanent displacements:

   $k_{max} = MHEA/g = 0.15$ to $0.20$, and $k_y = 0.15$ (given), so $k_{s}/k_{max} = 0.75$ to $1.0$

   $U/k_{max}D_{5-95} = 0$ to $4$ mm/s (from Figure 11, using 50 and 16% exceedance lines)

   $U = (0$ to $4$ mm/s)(0.15)(14$ s) = 0$ to $8$ mm (i.e. small displacements)
Cover sliding. Analyze the landfill for localized cover sliding near the crest for the design earthquake. Slope stability analyses indicate that the yield acceleration coefficient for localized cover sliding is 0.16.

1. Calculate the seismic loading, \( M_{HEA}^{Cover} \):

\[
M_{HA}^{Rock} = 0.33g, \quad NRF = 1.05, \quad \text{and} \quad T_s/T_m = 1.3 \quad \text{from previous analysis}
\]

\[
M_{HA}^{Top}/[(M_{HA}^{Rock})(NRF)] = 0.9 \text{ to } 1.1 \quad (50\% \text{ / } 60\%) \quad \text{(Figure 8)}
\]

\[
M_{HA}^{Top} = (0.33g)(1.05)(0.9 \text{ to } 1.1) = 0.31g \text{ to } 0.38g
\]

\[
M_{HEA}^{Cover} = (1.25)(0.31 \text{ to } 0.38g) = 0.39g \text{ to } 0.48g
\]

2. Estimate the seismically induced permanent displacements:

\[
k_{max} = M_{HEA}^{Cover}/g = 0.39 \text{ to } 0.48, \quad \text{and} \quad k_y = 0.16 \quad \text{(given)}, \quad \text{so} \quad k_y/k_{max} = 0.33 \text{ to } 0.41
\]

\[
U = 100 \text{ to } 500 \text{ mm} \quad \text{(from Figure 12, using } 50 \text{ and } 16\% \text{ exceedance for } M_w = 7)
\]

4 CONCLUSIONS

Landfill performance during recent earthquakes has been generally satisfactory. However, one geosynthetic-lined landfill experienced significant damage, with two tears observed in the HDPE geomembrane liner, and two unlined landfills were damaged due to liquefaction-induced lateral spreading. Several landfills have experienced moderate damage, evidenced by cracking in the interim soil cover at waste/natural ground interfaces, cracking and limited downslope movement in cover soils, breaking of gas extraction header lines, and a loss of power to the gas collection system. Back-analyses of landfill performance during the Northridge earthquake in California have provided exceptional opportunities to estimate the dynamic properties of solid-waste fill and geosynthetic interfaces. However, due to the importance of evaluating these properties, additional work is warranted.

Results from equivalent-linear and fully nonlinear seismic site response analyses indicate that the dynamic response of a MSWLF can vary significantly due to reasonable variations of dynamic waste properties, fill heights, site conditions, and input rock motions. However, for the base sliding case, the maximum seismic loading represented by \( M_{HEA} \) follows a well-defined trend. \( M_{HEA} \) depends largely on the initial fundamental period of the waste fill, \( T_s = 4H/V_s \), and the mean period, \( T_m \), and \( M_{HA} \) of the input rock motion adjusted by the nonlinear response factor, \( NRF \). Conversely, analytical results indicate that the \( M_{HA} \) calculated at the top of a landfill can vary significantly for relatively modest variations in landfill configurations and input motions. Procedures that first require an estimate of the \( M_{HA} \) at the top of a landfill to estimate the seismic loading at the base of the landfill are limited by the relatively large uncertainty in estimating \( M_{HA} \) at the top of a landfill.

A rigid sliding block analysis can be significantly unconservative and should not be used. However, as a seismic performance index, a reasonable (and generally conserva-
tive) estimate of the seismically induced permanent displacement can be obtained using the decoupled approximation delineated in Makdisi and Seed (1978). The magnitude of earthquake-induced permanent displacements is intimately linked to the ratio of $k_y/k_{max}$, but it is also influenced significantly by the landfill’s dynamic response characteristics, $k_{max}$, and duration of the ground motion, $D_{5-95}$.

The analytical results are consistent with observations from available case records. A simplified seismic analysis procedure that is based on these validated analytical results is presented. Although “simplified”, the procedure is comprehensive in that it requires: (a) characterization of the design bedrock motions in terms of intensity, frequency content, and duration; (b) estimation of the seismic loading at the base and cover of the landfill; (c) evaluation of performance in terms of seismically induced displacements; and (d) the application of appropriate engineering judgment. Its use is illustrated in the seismic evaluation of a typical landfill. Seismically induced permanent displacement estimates for the cover and base liner form the basis of evaluating the likely seismic performance of the geosynthetics used in the waste containment systems.

ACKNOWLEDGEMENTS

Financial support was provided by the David and Lucile Packard Foundation and National Science Foundation under Grants BCS-9157083 and CMS-9416261. Dr. Seed of U.C. Berkeley, Dr. Abrahamson of PG&E, Drs. Kavazjian and Matasovic of GeoSyn-tec Consultants, and the late Dr. Leonards of Purdue University assisted the authors at various stages of their research. The authors would like to thank R. Anderson, J. Clinkenbeard, and D. Petker of the California Integrated Waste Management Board, R. Nelson of the California Regional Water Quality Control Board, P. Mundy of CDM Federal Programs, R. Herzig of the U.S. EPA, and C. Dowdell and J. Hower of the Los Angeles County Sanitation District, all of whom have provided field reconnaissance data.

REFERENCES


BRAY, RATHJE, AUGELLO AND MERRY • Seismic Design for Lined Solid-Waste Landfills


NOTATIONS

Basic SI units are given in parentheses.

\[ c = \text{soil cohesion (Pa)} \]
\[ D_{95} = \text{significant duration of acceleration-time history (s)} \]
\[ FS = \text{factor of safety (dimensionless)} \]
\[ FS_{static} = \text{static factor of safety (dimensionless)} \]
\[ G = \text{shear modulus (Pa)} \]
\[ G_{max} = \text{maximum shear modulus (Pa)} \]
\[ g = \text{acceleration due to gravity (m/s}^2\text{)} \]
\[ H = \text{height of landfill waste or cover thickness (m)} \]
\[ HEA = \text{horizontal equivalent acceleration (m/s}^2\text{)} \]
\[ k = \text{seismic acceleration coefficient (dimensionless)} \]
\[ k_{\text{max}} = \text{maximum seismic acceleration coefficient} = \frac{MHA}{g} \text{ (dimensionless)} \]
\[ k_y = \text{yield acceleration coefficient (dimensionless)} \]
\[ L = \text{length of midsection of landfill (m)} \]
\[ L_s = \text{length of cover slope mass (m)} \]
\[ MHA = \text{maximum horizontal ground acceleration (m/s}^2) \]
\[ MHA_{\text{Crest}} = \text{maximum horizontal ground acceleration at crest of landfill (m/s}^2) \]
\[ MHA_{\text{Rock}} = \text{maximum horizontal ground acceleration of rock (m/s}^2) \]
\[ MHA_{\text{Site}} = \text{maximum horizontal ground acceleration of site (m/s}^2) \]
\[ MHA_{\text{Top}} = \text{maximum horizontal ground acceleration at top of landfill (m/s}^2) \]
\[ MHEA = \text{maximum horizontal equivalent acceleration (m/s}^2) \]
\[ MHEA_{\text{Base}} = \text{maximum horizontal equivalent acceleration at base of landfill (m/s}^2) \]
\[ MHEA_{\text{Cover}} = \text{maximum horizontal equivalent acceleration of landfill cover sliding mass (m/s}^2) \]
\[ M_w = \text{moment magnitude of earthquake event (dimensionless)} \]
\[ NRF = \text{nonlinear response factor (dimensionless)} \]
\[ R = \text{seismic displacement reduction factor} = \frac{k_y}{k_{\text{max}}} \text{ at selected displacement (dimensionless)} \]
\[ R_b = \text{seismic displacement reduction factor} = \frac{k_y}{k_{\text{max}}} \text{ at selected base displacements (dimensionless)} \]
\[ R_c = \text{seismic displacement reduction factor} = \frac{k_y}{k_{\text{max}}} \text{ at selected cover displacements (dimensionless)} \]
\[ S_1 = \text{back-slope run to height ratio (dimensionless)} \]
\[ S_2 = \text{front-slope run to height ratio (dimensionless)} \]
\[ T_m = \text{mean period of acceleration-time history (s)} \]
\[ T_m_{-\text{EQ}} = \text{mean period of earthquake (s)} \]
\[ T_p = \text{predominant period of ground motion (s)} \]
\[ T_p_{-\text{EQ}} = \text{predominant period of earthquake (s)} \]
\[ T_c = \text{fundamental period of column of waste fill (s)} \]
\[ T_c_{\text{FILL}} = \text{fundamental period of fill material (s)} \]
\[ T_c_{\text{WASTE}} = \text{fundamental period of waste (s)} \]
\[ t = \text{time (s)} \]
\[ U = \text{seismically induced permanent displacement (mm)} \]
\[ V_s = \text{average shear wave velocity (m/s)} \]
\[ \beta = \text{slope angle of cover from horizontal (°)} \]
\[ \varepsilon = \text{strain (dimensionless)} \]
\[ \phi = \text{internal friction angle (°)} \]
\[ \gamma = \text{total unit weight (N/m}^3\) \]
\( \theta_i \) = back-slope geometry parameter (Figure 9) (°)
\( \tau_h \) = horizontal shear stress (Pa)
\( \sigma_n \) = total normal stress (Pa)
\( \sigma_v \) = total vertical stress (Pa)

**ABBREVIATIONS**

BSSC: Building Seismic Safety Council
HDPE: high density polyethylene
MSWLF: municipal solid-waste landfill
NEHRP: National Earthquake Hazards Reduction Program
OII: Operating Industries, Inc.
PP: polypropylene
PVC: polyvinyl chloride
SASW: spectral analysis of surface waves
UBC: Uniform Building Code
USGS: United States Geological Survey
1-D: one-dimensional
2-D: two-dimensional