SEISMIC ANALYSIS FOR FOUR GEOSYNTHETIC-LINED LANDFILLS

by

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ABSTRACT

A practical approach for seismic impact analysis of geosynthetic-lined landfills is described and then illustrated through the use of four case histories of recently completed projects in the United States. The approach is intended to satisfy new United States federal and state regulations regarding landfills constructed in areas that may be subjected to seismic motions. The elements of the approach are similar to those often used for seismic analysis of earth embankments. The ability of landfill structures, such as liner and cover systems with geosynthetic components, to tolerate seismically induced deformations is discussed.

INTRODUCTION

Prior to the 1990s, seismic analysis of landfills in the United States was only performed within regions known to have significant seismic activity, such as much of California. The rationale for omitting seismic analysis in most regions of the country may have been that massive earth structures were considered to be resistant to seismic motions and that most regions were not considered seismically active, particularly in the eastern and central United States. More recently, however, seismic analysis has become an important consideration in landfill engineering in much of the United States.
Two primary factors account for the increased emphasis on seismic analysis in landfill engineering in the United States. The first factor is that newly promulgated federal and state regulations require seismic analysis for many regions that have not previously been considered seismically active. Furthermore, the regulations require the use of a relatively severe earthquake event for the seismic analysis. A summary of the federal regulations is provided in the next section of this paper.

The second factor is that modern landfills in the United States typically employ multiple-layer liner and cover systems that include geosynthetic elements such as geomembranes, geotextiles, and geonets. These types of liner and cover systems are used because they provide a high degree of waste and leachate isolation and because they are required by current federal and state landfill regulations. Older landfills typically did not include geosynthetic elements. Experience has shown that the interfaces between geosynthetic elements and soils and between adjacent geosynthetic elements within liner and cover systems have relatively low shear strengths when compared to liner and cover systems without geosynthetic elements. Thus, the increased use of multiple-layer systems with geosynthetic elements, although superior for waste isolation, has led to the creation of near-continuous interface zones within critical landfill structures that can potentially reduce landfill stability.

The purpose of this paper is to describe a practical approach for seismic analysis of geosynthetic-lined landfills that is intended to satisfy federal and state regulations. The approach is presented and then illustrated through the use of four abbreviated case histories of recently completed projects in the United States that illustrate noteworthy aspects of the seismic design of landfills.
UNITED STATES SEISMIC REGULATIONS

Federal regulations that address the siting and design of landfills in seismically active areas became effective in late 1993 throughout the United States. The federal regulations provide a minimum standard with which every state must comply. In some states, state regulations for seismic design of landfills may vary significantly in some aspects from the federal regulations.

The federal regulations apply only to seismic impact zones, which are defined as areas in which there is at least a ten percent probability that horizontal seismic accelerations equal to or greater than 0.1 g (where g represents the acceleration of gravity at the earth’s surface) will occur on bedrock during an exposure period of 250 years. Based on a seismic hazard map prepared by the United States Geological Survey [Algernisse et al., 1990], approximately 40 percent of the area of the contiguous United States is in a seismic impact zone, including approximately 45 percent of the area east of the Mississippi river (Figure 1).

The federal regulations state that the horizontal seismic acceleration to be used in the seismic analysis corresponds to a bedrock acceleration which has a ten percent probability of being exceeded during a 250 year exposure period. This value is referred to as the federal design acceleration. The regulations indicate that the landfill containment structures must be designed to resist the design acceleration. The bedrock acceleration level is used as a reference point in recognition of the fact that seismic ground motions at a site can be strongly affected by local soil conditions.

Based on the assumption that earthquake occurrence in a region is a Poisson process, the probability and time interval used to define the design acceleration corresponds to an earthquake with a mean recurrence interval (i.e., expected return period) of approximately 2400 years. This is a relatively severe earthquake for landfill performance evaluation. In fact, design codes for structures such as high-rise buildings and mass transit facilities often specify less severe earthquake return periods, typically on the order of 500 to 1000 years, for design to a life safety standard.
FIGURE 1
SEISMIC IMPACT ZONES IN THE CONTIGUOUS UNITED STATES
The federal regulations provide a framework for conducting a seismic analysis, but do not provide guidance on all issues that must be addressed in order to carry out a complete analysis. Issues that are subject to interpretation include:

- the design acceleration is only one of the earthquake characteristics that defines a design earthquake for use in seismic analysis; guidance on other important earthquake characteristics, such as magnitude, duration, and predominant vibration frequency, is not provided;

- the regulations are based upon a maximum horizontal site acceleration, but the earthquake inducing the maximum horizontal acceleration at the site is not necessarily the most damaging earthquake;

- the federal design acceleration logically applies to post-closure landfill conditions; the appropriate acceleration for seismic analysis of temporary conditions that will exist at the landfill during its active life is not specified;

- a comprehensive definition of the term 'landfill containment structures' is not provided; of particular importance is whether final cover systems are to be included; and

- the performance standard associated with the term 'to resist the design acceleration' is not well defined; in particular, what, if any, amount of permanent seismic deformation constitutes acceptable performance for the various components of the containment system.
ANALYSIS APPROACH

Seismic Impacts

The goal of a seismic impact analysis is to address the ability of landfill containment structures to resist the design acceleration. This is interpreted to mean that the landfill containment structures must maintain their integrity, i.e., not be breached or rendered unfunctional, when subjected to earthquakes which induce the design acceleration at the project site. Seismic analysis of a landfill will typically consider the following seismic impacts which are believed to have the potential to cause landfill containment structures to lose their integrity:

- loss of adequate foundation support for the landfill liner system due to liquefaction of foundation soils; and

- accumulation of excessive permanent deformations along slip surfaces that pass through or along the landfill liner system or cover system, considering both active-life and post-closure landfill conditions.

It is noted that specific state regulations may require that additional hazards be considered. If a particular landfill has additional containment system structures which could harm the environment if damaged during an earthquake, then the integrity of these structures should also be considered.

The level of permanent deformation involving the liner system that is considered acceptable is usually based on engineering judgement. In current practice, a permanent seismic deformation greater than 6 to 12 in. (150 to 300 mm) is generally considered to be excessive [Seed and Bonaparte, 1992]. This calculated level of deformation is considered to be a typical index of continued acceptable liner system performance after the design earthquake, rather than a deformation level at which a specific mode of liner system damage might occur. If design details for a particular site are such that the liner system has a greater-than-typical or less-than-
typical tolerance to deformations, the use of the above deformation limits should be reconsidered.

No general recommendations are available for acceptable permanent seismic deformation of the landfill cover system. However, since cover system deformations are readily observable in post-earthquake inspections and are easily repaired, acceptable deformation may logically be assumed to be greater than that for the liner system. Some regulators have indicated that they consider seismic deformation of the cover system to be primarily a maintenance problem.

Outline of Approach

The seismic impact analysis is conducted to evaluate the potential seismic hazards mentioned above. A variety of analysis methods are used in engineering practice to evaluate soil liquefaction. The most common method, the Simplified Method, uses the results of the Standard Penetration Test to evaluate liquefaction potential [Seed et al., 1983]. This method, or slight variations of this method, was used in the liquefaction analyses described in this paper.

Permanent deformations involving the landfill liner system or cover system were evaluated using a method that is consistent with that used by others for landfills [e.g., Repetto et al., 1993] and with existing engineering analysis procedures for seismic analysis of earth embankments [e.g., Marcuson et al., 1992]. Validation of the applicability of these methods to landfill engineering based open observations of landfill performance is not currently possible due to the lack of suitable case histories.

The seismic impact analysis approach described in this paper consists of the following seven steps:

- characterize the site subsurface conditions and the landfill design geometry and liner system details;
• characterize the design earthquake in terms of the value of maximum horizontal ground acceleration at the landfill site and a representative magnitude (more detailed characterization of the design earthquake may be necessary for subsequent steps);

• evaluate the resistance of the landfill and landfill foundation to seismic motions in terms of the potential for foundation liquefaction and the threshold acceleration (yield acceleration) for accumulation of permanent deformation along slip surfaces passing through or along the landfill liner and cover systems;

• perform an initial evaluation of the response of the landfill and foundation to the design earthquake using the maximum ground acceleration and earthquake magnitude only; the seismic impact analysis is complete if this initial evaluation indicates that the potential for damage from the various seismic hazards is unlikely; otherwise, the analysis continues with the next step;

• fully characterize the design earthquake by selecting several appropriate accelerograms (time histories of horizontal acceleration) for use in seismic response analysis;

• predict the response of the landfill and foundation to the design earthquake using one-dimensional site response analysis; and

• evaluate the seismic impacts by comparing the resistance of the landfill and foundation to their seismic response; if necessary, perform seismic deformation analyses using the predicted seismic response and compare the results to acceptable deformation levels.

Each step in the seismic analysis approach is discussed in the following sections of this paper.
Subsurface Conditions and Landfill Design

Subsurface conditions at the site are characterized using the results of standard geotechnical subsurface investigations and engineering judgement. Review of available geological and ground-water resource literature is often necessary for sites with deep soil profiles. Required subsurface information includes depth to bedrock, soil stratigraphy, soil index properties, soil penetration resistance, and the location of the ground-water table.

Landfill design information is generally obtained from design drawings and documents. Required information includes active-life and post-closure grades, liner system base grades, and liner system and cover system details. Design details should be examined to determine if the liner system has a greater- or less-than-typical tolerance to deformations.

Design Earthquake - Magnitude and Maximum Acceleration

The representative magnitude and maximum horizontal ground acceleration for the design earthquake can be characterized by using published seismic hazard studies for regions that include the site, by performing a site-specific seismic hazard analysis using the method outlined by Cornell [1968], or by a combination of both methods. For sites with high seismicity or with numerous nearby seismic source zones, a site-specific seismic hazard analysis is often conducted. For sites with low to intermediate seismicity, published seismic hazard studies are typically used for initial analysis. Site-specific seismic hazard analysis for these sites may be considered if the results of the initial seismic impact analysis show a significant sensitivity to the seismic hazard results.

The most commonly used seismic hazard study for landfill design was published by the United States Geological Survey (USGS) [Algermissen et al., 1990] and covers the fifty United States. The bedrock design acceleration specified in state and federal landfill regulations can be obtained directly from the USGS map. The magnitude of the design earthquake can be
conservatively obtained from the background information for the Alghermisen seismic hazard study [Alghermisen et al., 1982] as the maximum magnitude assigned to the seismic source zone that governs seismicity at the site. The maximum horizontal ground acceleration at the project site can be estimated based on the USGS bedrock design acceleration and typical relationships between peak accelerations at bedrock and at ground level as a function of the local soil conditions. Alternatively, a formal site response analysis can be performed to evaluate the influence of the local soil conditions on the bedrock ground motions.

If a site-specific seismic hazard study is conducted using acceleration attenuation curves for rock sites, the bedrock design acceleration at the site may be obtained directly from the hazard analysis. The maximum horizontal ground acceleration at the site can then be estimated as described in the preceding paragraph. Alternatively, if the hazard analysis is conducted using attenuation curves for soil site conditions representative of the project site the peak ground surface acceleration at the site is obtained directly from the hazard analysis results. The magnitude of the design earthquake may also be conservatively obtained as described in the preceding paragraph, or by interpretation of the magnitude distribution from the seismic hazard analysis. Most available seismic hazard analysis computer programs must be modified in order to output the magnitude distribution required for this interpretation.

For some sites, it may be necessary to define two design earthquakes: a relatively small magnitude event in a nearby source zone that generates the design acceleration, and a larger magnitude event from a more distant source zone that generates a lower acceleration but is potentially more damaging because of a longer duration of strong shaking. Determination of this lower intensity, larger magnitude event is based upon engineering judgement.
Seismic Resistance of Landfill and Foundation Soils

Analysis Methods

The seismic resistance of the landfill and foundation is determined using conventional geotechnical analyses. The liquefaction resistance of the foundation material is assessed on the basis of Standard Penetration Test (SPT) blow counts in terms of a cyclic shear strength ratio using the simplified method of Seed et al. [1983]. This method requires soil grain size information, SPT blow counts or cone penetrometer data, vertical effective stresses, and the design earthquake magnitude to estimate the cyclic shear strength ratio.

Resistance of the landfill to deformation along slip surfaces that pass through or along the landfill liner or cover system is evaluated in terms of yield acceleration. A yield acceleration is the average horizontal acceleration that must be imposed on a potential sliding mass to produce a marginally stable condition. The seismic analysis requires evaluation of the critical yield accelerations, i.e., the smallest yield accelerations obtained for potential slip surfaces through or along the landfill liner system and cover system. The critical yield acceleration is obtained using traditional pseudo-static limit equilibrium slope stability analysis methods. The critical yield acceleration for the liner system may be evaluated for both active-life and post-closure landfill conditions.

Analysis Parameters

The results of the pseudo-static slope stability analyses conducted to determine the critical yield acceleration are often sensitive to the shear strength parameters used for the landfill and foundation materials. Selection of these parameters is, therefore, a significant step in the analysis, as described below.

- Shear strength of foundation soils is usually evaluated using penetration test data, from the results of laboratory tests, or from soil index properties. Although pseudo-static analysis should be conducted using cyclic shear strength parameters, it is generally
assumed that static shear strength parameters are appropriate for pseudo-static analysis. This assumption is believed to be reasonable for most soils, with the possible exception of granular soils that may liquefy, saturated soft and sensitive clays, and stiff clays which show significant post-peak strength reduction.

- Waste shear strength is typically based upon experience and recommended values from previous studies, as site-specific testing of waste shear strength is difficult and expensive. No attempt is usually made to distinguish between cyclic and static shear strength parameters because insufficient information is currently available.

- Shear strength along the landfill liner and cover systems is taken as that of the weakest interface or material within the system. The shear strength is typically based either on project experience and recommended values from previous studies or on site-specific, static direct shear testing of liner system interfaces. Cyclic and static shear strengths are generally assumed to be equal for interfaces between adjacent geosynthetic materials based on judgement and limited experimental results (e.g., Kavazanjian et al., 1991; Yegian and Lahlaf, 1992). Similarly, cyclic and static shear strength are generally assumed to be equal for the interfaces between geosynthetic materials and soils based on the fact that the soils typically used in liner system construction (i.e., compacted clays and granular drainage soils) are not likely to be sensitive, brittle, or subject to cyclic degradation.

Initial Evaluation of Seismic Response

An initial evaluation of seismic response is performed by making simplifying assumptions for the response of the landfill and foundation to the design earthquake and comparing the assumed response to the seismic resistance of the landfill and foundation. The assessment of the seismic resistance is described in the preceding section. This initial, simplified evaluation of seismic response is performed because it often shows that sites in areas with low to intermediate
seismicity have a such low likelihood of unsatisfactory seismic performance that no additional analysis is necessary. If this is the case, the seismic impact analysis is complete.

The initial liquefaction evaluation is performed by comparing the cyclic shear strength ratio of the foundation soil to the cyclic shear stress ratio induced in the foundation soil by the design earthquake. The cyclic shear strength ratio is obtained using the Simplified Method of Seed et al. [1983] as described in the preceding section. The cyclic shear stress ratio is also obtained using the Simplified Method. Evaluation of the cyclic stress ratio requires only the maximum horizontal ground acceleration and the vertical total and effective stress profiles at the site. If the cyclic shear strength ratio exceeds the cyclic shear stress ratio by a sufficient amount, then it is concluded that liquefaction of the foundation soils is unlikely.

The initial evaluation of the potential for permanent deformations is performed by comparing the critical yield accelerations for surfaces that pass through or along the landfill liner and cover systems to an estimate of the maximum horizontal acceleration induced within the critical sliding mass by the design earthquake. The critical yield accelerations are obtained as described in the preceding section. The maximum horizontal acceleration within the waste mass for analysis of the liner system is assumed to be equal to the value of the maximum horizontal ground acceleration at the project site for the design earthquake. This assumption has generally been found to be conservative for typical landfill and foundation soil conditions, with the possible exception of landfills with thin waste thicknesses or very stiff wastes. For the landfill cover system, the maximum acceleration at the top of the landfill may be estimated using amplification curves for soft soil sites, e.g., Idriss [1990] and compared to the yield acceleration.

The ratio of the critical yield acceleration to the maximum earthquake-induced acceleration is then computed for both the liner and cover systems. If the ratio exceeds one-half (0.5), then it is concluded that permanent deformations are negligible. If the ratio is less than 0.5 it is concluded that significant permanent deformations may occur and more sophisticated response and deformation analyses are required. The use of a ratio of 0.5 is based upon work published by Hynes and Franklin [1984] which suggests that excessive permanent seismic deformation is
unlikely if the ratio of the yield acceleration to the peak acceleration of the potential sliding mass exceeds 0.5.

**Design Earthquake Accelerograms**

If a full seismic response analysis is to be performed, earthquake accelerograms are required as input to the analysis. Earthquake accelerations are known to vary in duration, frequency content, maximum acceleration, and other characteristics of engineering significance. Seismological factors which affect these characteristics include earthquake magnitude, earthquake source mechanism, epicentral distance (i.e., distance from epicenter to site), focal depth, rock characteristics along travel path from source to site, site soil conditions, and other poorly understood factors. With such a large number of factors affecting earthquake accelerations, it is not surprising that no two accelerograms are precisely alike, even accelerograms from the same site from different events of the same magnitude on the same fault.

Due to this variable nature of earthquake motions, the characteristics of engineering significance of a design earthquake cannot be predicted precisely for a given site. Accepted practice is to use a minimum of three accelerograms recorded on rock for seismological conditions similar to those anticipated at the site for the design earthquake in order to establish a range of design earthquake motions. Selection of an appropriate suite of design earthquake accelerograms should be made by attempting to match as many of the relevant earthquake characteristics as possible. This often requires examining available accelerogram catalogs maintained by universities and other research institutions in order to select time histories appropriate for the seismic impact analysis. In particular, the use of accelerograms from the western United States for sites in the eastern and central United States is not always appropriate. For areas where representative accelerograms from previous earthquakes are not available, synthetic accelerograms may be used along with the best available recorded accelerograms.

After the suite of representative accelerograms is selected, their accelerations values are scaled so that the maximum acceleration value of each record is equal to the design acceleration. The accelerograms may be selected to represent either anticipated motions at the top of bedrock
or at the ground surface at the site. For sites influenced by multiple seismic source zones, the
design earthquake may be defined by both a relatively small magnitude, high intensity event in
a nearby source zone and a larger magnitude, lower intensity, event in a more distant source
zone. It may not be possible to evaluate which event is potentially more severe for a particular
landfill design without actually conducting the seismic impact analysis. In this case, the selected
suite of accelerograms should reflect both possible events and more than three accelerograms
may be required. This is necessary because, along with difference in peak acceleration, the
duration, frequency content, and other earthquake motion characteristics of the two events would
differ.

Site Response Analysis

If the preliminary seismic response analysis indicates that foundation liquefaction or
significant permanent seismic deformation of the landfill liner or cover system is possible, a site-
specific seismic response analysis of the landfill and foundation to the design earthquake may
be performed. The site response analysis models the occurrence of the design earthquake at the
site and predicts the motions induced in each layer of the foundation soil and landfill. The
design earthquake motions are typically input as horizontal bedrock accelerograms and are
assumed to propagate upward as shear waves. The input motion is modified by its passage
through soil and landfill waste and may be amplified or attenuated (i.e., diminished) to varying
degrees at different elevations in the soil and landfill waste mass.

The site response analysis requires dynamic material properties for landfill liner and cover
elements, waste, and foundation soils. In an equivalent linear site response analysis, the most
common type of site response analysis performed in practice, the primary dynamic properties
of concern are shear modulus, damping ratio, and the pattern of their variation with changing
shear strain level. Selection of dynamic property parameters is described below.

- For the foundation soils, the dynamic parameters for shear modulus and damping ratio
  may either be directly measured in field or laboratory tests or selected based on
available information for dynamic soil properties [e.g., Seed and Idriss, 1970; Hardin, 1978; Whitman and Dobry, 1988; Vucetic and Dobry, 1991]. Past experience has shown that dynamic material parameters depend on soil type, soil density, effective stress, and, for clays, the plasticity index and overconsolidation ratio.

- For the landfill waste, unit weight and dynamic parameters for shear modulus and damping ratio are selected based on available technical information and experience [e.g., Earth Technology Corporation, 1988; Singh and Murphy, 1990; Repetto et al., 1993; Kavazanjian et al., 1994]. It is noted that the available information indicates that the age of landfill waste may have a significant effect on its dynamic properties, particularly density and shear modulus.

For the analyses described in this paper, the site response analysis is performed using the latest version of the SHAKE computer program [Schnabel et al., 1972; Idriss and Sun, 1992]. The SHAKE program uses the equivalent linear response method and models the landfill and foundation as a sequence of horizontal layers of infinite areal extent. Such one-dimensional analysis is typically performed for profiles at three or more locations with varying thicknesses of landfill waste, plus one profile location with no landfill waste, in order to approximately encompass the range of likely site response. Two- and three-dimensional analyses are performed only rarely because of their complexity and cost.

The response characteristics of greatest interest from the site response analysis are: (i) the peak shear stresses induced in the foundation soils (for use in liquefaction analyses); and (ii) time histories of the average acceleration of the landfill mass and of the final cover system corresponding to the critical potential slip surfaces that were identified in the pseudo-static slope stability analyses. The peak shear stresses in the foundation soils are directly output by the SHAKE program. The average acceleration time histories are obtained by either processing output shear stress time histories, as described by Repetto et al. [1993], or by a weighted averaging of the acceleration time histories for all waste layers above the critical potential slip surface. The site response analysis is generally performed for the entire suite of design
earthquake accelerograms and for all analysis profile locations. The use of the predicted response characteristics is discussed in the following section.

Final Evaluation of Seismic Impacts

The final liquefaction evaluation is performed by comparing the cyclic shear strength ratio of the foundation soil to the cyclic shear stress ratio that is induced in the foundation soil by the design earthquake. The cyclic shear strength ratio is obtained as described previously. The cyclic shear stress ratio is obtained from the peak shear stresses in the foundation soils that are predicted by the site response analysis. The maximum values of shear stress from the range of predicted responses are used. If the cyclic shear stress ratio induced by the earthquake exceeds the predicted cyclic shear strength ratio, then it is concluded that liquefaction of the foundation soils is possible.

If liquefaction is found to be possible, a liquefaction impact assessment is performed. The liquefaction impact assessment includes evaluation of the seismic settlement potential of the liquefied soil and the potential for a liquefaction flow failure. The assessment of seismic settlement potential is based upon soil density or penetration resistance as described by Ishihara [1990]. The potential for a liquefaction flow failure is assessed by assigning residual (i.e., post-liquefaction) shear strengths to the liquefiable zone and performing a static limit equilibrium stability analysis, as described by Marcuson et al. [1990]. Residual shear strengths are based upon the penetration resistance of the liquefiable soil.

The evaluation of permanent deformations is performed by comparing the critical yield accelerations for potential slip surfaces that pass through or along the landfill liner and/or cover system to the predicted time histories of the average acceleration of the potential failure mass during the design earthquake. The critical yield accelerations are obtained from limit equilibrium analysis as described in an earlier section. The predicted acceleration time histories are obtained from the site response analysis, as described in the preceding section.
Permanent seismic deformation is calculated using the yield acceleration and the acceleration time history of the potential failure mass. The deformation may be calculated either with the aid of simplified design charts [Makdisi and Seed, 1978; Hynes and Franklin, 1984], or with a detailed dynamic deformation analysis. The Newmark [1965] method is the basis for both the simplified and detailed deformation analyses. This method considers deformations along a given slip surface and requires the yield acceleration for the surface and the time history of average acceleration of the sliding mass above the slip surface. The full range of acceleration time histories predicted by the site response analysis are be considered, and analyses are performed for deformation that could occur in either of two directions, one direction corresponding to the positive acceleration peaks in the time history and the other to the negative peaks. If the largest predicted permanent seismic deformations from the Newmark analyses are less than the maximum acceptable level, then it is concluded that excessive permanent seismic deformations are unlikely to occur.
PROJECT A - MUNICIPAL SOLID WASTE LANDFILL IN THE SOUTHEASTERN UNITED STATES

Introduction

This project illustrates the application of the seismic analysis approach to a municipal solid waste landfill site in the southeastern United States coastal plain. This project provides an example of analyses performed for a site with a potential liquefaction hazard.

Subsurface Conditions and Landfill Design

The landfill site is located in the Atlantic coastal plain. The site is underlain by an alternating sequence of granular and cohesive sedimentary soils with a total thickness of approximately 1700 ft (520 m). The ground-water table at the site is somewhat variable and is as close as 10 ft (3 m) to the ground surface in places. Standard Penetration Test (SPT) blow counts obtained in the near-surface soils indicate the existence of layers of loose, saturated granular soils in some areas of the site.

The landfill design has the following geometric features:

- waste grades as steep as 3H:1V (horizontal:vertical) for conditions during the active life of the landfill;
- waste grades as steep as 4H:1V for landfill post-closure conditions;
- waste thicknesses of 0 to 120 ft (37 m); and
- liner system base grades of approximately two percent.
The landfill design has a single composite liner system with a granular leachate collection layer. The liner system in the base area of the landfill includes two interfaces that potentially have the lowest shear strengths within the system. These interfaces are between the compacted clay liner and the overlying smooth high density polyethylene (HDPE) geomembrane and between the smooth HDPE geomembrane and the overlying cushioning geotextile.

Design Earthquake

The design acceleration for the site from the Algermissen et al. [1990] map is 0.17 g. Seismological literature indicates that the governing seismic source zone is capable of generating a maximum earthquake magnitude of 6.0 to 6.4. This maximum magnitude was used as the representative magnitude for the design acceleration. A simplified site-specific seismic hazard analysis conducted for the site indicated the appropriate epicentral distance for the design earthquake of magnitude 6.0 to 6.4 was approximately 35 mi (56 km).

A suite of three recorded accelerograms was selected to represent the design earthquake. Two of the accelerograms were from eastern North America events. The third accelerogram was from the western United States due to the lack of a third suitable record from the eastern United States. Information on the earthquake magnitude, peak acceleration and epicentral distance for the recording site for each of the design accelerograms, is provided below:

- earthquake of magnitude 6.0 occurring in 1988 in Quebec, Canada; recording site at epicentral distance of 56 mi (90 km); maximum recorded horizontal acceleration of 0.23 g;

- earthquake of magnitude 5.0 occurring in 1981 in New Brunswick, Canada; recording site at epicentral distance of 2.5 mi (4 km); maximum recorded horizontal acceleration of 0.40 g; and
- earthquake of magnitude 6.1 occurring in 1987 in Whittier, California; recording site at epicentral distance of 1.9 mi (3 km); maximum recorded horizontal acceleration of 0.48 g.

The record from the Whittier earthquake in California was chosen as a representative record because of the similar magnitude and the fact that the Whittier earthquake was a relative deep earthquake with no surface faulting. Lack of surface faulting is an important characteristic of eastern United States earthquakes. All three of the accelerograms are from an earthquake event that did not cause ground surface rupture and each was recorded on rock. The maximum recorded acceleration for each accelerogram was scaled to 0.17 g, the design acceleration, prior to use in the site response analysis.

**Seismic Resistance of Landfill and Foundation Soils**

The liquefaction resistance of the loose, saturated granular soil layers was evaluated using the correlation between SPT blow count and liquefaction potential from Seed et. al (1983). The normalized SPT blow counts for the potentially liquefiable soils had an average value of 8. Based upon the mean grain size of the soil and the earthquake magnitude, the cyclic shear strength ratio for these layers was estimated to be approximately 0.115.

For the pseudo-static slope stability analyses, shear strength parameters of a friction angle of 8 degrees and an adhesion of zero were used for sliding along the landfill liner system. These parameters were selected to represent the interface between the smooth HDPE geomembrane and the overlying cushion geotextile. These parameters were based on project experience and published interface shear strength studies.

Pseudo-static slope stability analyses were performed using the computer program XSTABL (Sharma, 1991). The slope stability analyses predicted critical yield accelerations of 0.025 g for active-life conditions and 0.058 g for post-closure conditions. The critical potential slip surfaces for both conditions passed along the base landfill liner system over most of their length.
Site Response Analysis

Site response analyses were conducted for the three design earthquake accelerograms and for five profile locations representing landfill waste thicknesses from 0 to 120 ft (37 m). The dynamic properties used for the landfill waste included: (i) shear wave velocities from 700 ft/sec (215 m/sec) near the surface to 930 ft/sec (285 m/sec) at a depth of 120 ft (36 m); (ii) unit weights from 32 lb/ft² (5 kN/m²) near the surface to 75 lb/ft² (11.8 kN/m²) at a depth of 120 ft (36 m); (iii) modulus reduction curve similar to that given for peat by Seed and Idriss [1970]; and (iv) damping curve similar to that given for clay by Seed and Idriss [1970]. The site response analyses predicted a maximum cyclic shear stress ratio of 0.130 for the loose, saturated granular foundation soil layers. The time histories of average landfill mass acceleration predicted by the analyses had a maximum acceleration of 0.20 g, representing a slight amplification of the design acceleration of 0.17 g.

Evaluation of Seismic Impact

The calculated cyclic shear strength ratio for the loose, saturated granular foundation soil layers was 0.115. The site response analysis predicted a maximum cyclic shear stress ratio of 0.130 in locations where the thickness of landfill waste was 40 ft (12 m) or less. Therefore, in these locations liquefaction was assumed to occur in the loose, saturated granular foundation soil layers during the design earthquake. In order to evaluate whether liquefaction in the identified areas would cause distress in the landfill liner system, seismic settlement and post-earthquake static slope stability analyses were conducted using shear strengths representing liquified conditions for the loose layers within the identified areas.

Seismic settlement was evaluated using the method of Ishihara [1990]. Because the potentially liquefiable layers were a maximum of 5-ft (1.5-m) thick, the seismic settlement analyses indicated the maximum seismic settlement was less than 1 inch (25 mm). Residual shear strength for the post-earthquake stability analyses were estimated to be on the order of 250 lb/ft² (12 kPa) based upon the correlation with SPT blow count from Seed et al. (1988). The
post-earthquake stability analyses indicated a minimum factor of safety of 1.2 for the most critical section (Figure 2).

The site response analysis predicted that the peak average acceleration value for the landfill mass was 0.20 g. This value was more than twice the critical yield accelerations of 0.025 g and 0.058 g that were calculated for active-life and post-closure landfill conditions, respectively. Therefore, seismic deformation analyses were conducted to evaluate the magnitude of permanent deformations that might be induced by the design earthquake. Seismic deformation analyses were performed using the Newmark method of analysis as implemented in the computer program YSLIP (Yan, 1991). These deformation analyses predicted maximum permanent seismic deformations of 4 in (100 mm) for active-life conditions and 1 in (25 mm) for post-closure conditions. These values are below the acceptable limits.

Summary

The seismic analysis indicated that liquefaction might occur during the design earthquake in loose foundation soil layers in some areas of the site. Additional analysis was then conducted to evaluate the stability of the landfill mass in the case that liquefaction did occur in the identified foundation soil layers. The additional analysis indicated that the landfill mass would remain stable and not undergo lateral spreading as a result of potential liquefaction. It was therefore concluded that the integrity of the landfill liner system would be maintained in the design earthquake.

The seismic analysis also indicated that permanent deformations might develop during the design earthquake along surfaces that passed through or along the landfill liner system. The maximum magnitude of these potential deformations was predicted to be 4 in. (100 mm) if the design earthquake were to occur at the most critical time during active life of the landfill, and 1 in. (25 mm) if the design earthquake were to occur after landfill closure. These predicted permanent deformations are below the maximum allowable value. It was therefore concluded that the integrity of the landfill liner system would be maintained in the design earthquake.
FIGURE 2
CROSS-SECTION FOR POST-EARTHQUAKE STABILITY ANALYSIS
PROJECT A
PROJECT B - MUNICIPAL SOLID WASTE LANDFILL IN THE CENTRAL UNITED STATES

Introduction

This project illustrates the application of the seismic analysis approach to a municipal solid waste landfill in the central United States. In this case, the liquefaction potential of the foundation soil was of primary concern. The yield accelerations of the liner and cover systems were high enough that a seismic deformation analysis was not required. This project provides an example of analyses performed for a large-magnitude earthquake event in the New Madrid seismic zone.

Subsurface Conditions and Landfill Design

The site is located on deep sedimentary soils in the Mississippi River basin. The site soils consist of silts, sands, clays, gravels and loess with a total thickness of approximately 3000 ft (1000 m). The regional ground-water table in the vicinity of the site is at a depth of approximately 90 ft (27 m).

The site was located in an area indicated to have a low liquefaction potential on regional liquefaction potential maps. However, the subsurface investigation identified shallow saturated zones of perched water at depths of 7 to 15 ft (2 to 4.5 m) beneath the ground surface in the area underlying an intermittent stream that runs between the two waste units proposed for the site. Standard Penetration Test (SPT) blow counts indicated loose to medium dense granular soils in some of the saturated zones. Therefore, because of the coincidence of the perched water zones and the loose granular soil, liquefaction was a primary design consideration.

The landfill design has the following geometric features:
• waste grades of 4H:1V (average) for the landfill final cover to a height of approximately 50 ft (15 m) above grade;

• excavation to approximately 70 ft (21 m) below grade surface, using 3H:1V excavation slopes, for liner system placement;

• total waste thickness of 0 to 120 ft (49 m); and

• liner system base grades of approximately two percent.

A cross section through the waste unit is shown in Figure 3.

The landfill design incorporates a single composite liner system. This liner system includes a smooth HDPE geomembrane, a cushion geotextile, a compacted clay liner, and a granular leachate collection layer. It is believed the interface between the smooth HDPE geomembrane and the overlying cushion geotextile will control the minimum shear strength.

Design Earthquake

The design acceleration for the site from the Algermissen et al. [1990] map is 0.25 g. Review of published regional seismic hazard studies confirmed that this design acceleration is appropriate. Seismological literature indicates that the governing seismic source zone is the New Madrid seismic zone, which includes the project site and is estimated to be capable of generating a maximum earthquake magnitude (M) of 7.3. The representative design magnitude was therefore selected as 7.3, with a corresponding epicentral distance of approximately 30 mi (50 km) based on the site location with respect to the New Madrid seismicity zone and typical acceleration attenuation curves.

Selection of representative accelerograms is a difficult task for earthquakes in the New Madrid seismic zone due to the lack of recorded time histories for major earthquakes east of the Rocky Mountains. Selection of representative time histories was made somewhat easier by the
a priori knowledge that the yield accelerations of the liner and cover systems were relatively high and that liquefaction was the seismic impact of primary concern. The primary purpose of site response analysis was therefore to establish the earthquake-induced shear stresses in the liquefiable zone, and not to evaluate acceleration time histories for permanent deformation analyses. Experience has shown that the induced cyclic stress ratio is most sensitive to the peak acceleration and frequency content of the record, and less sensitive to magnitude and frequency content than permanent deformation.

A suite of two recorded accelerograms and two synthetic accelerograms were selected to represent the design earthquake. The following accelerograms were used:

- two high-frequency synthetic accelerograms, D1-SYN and B2-SYN, generated at Caltech [Jennings et al., 1968] for a M 6.5 earthquake 45 miles (72 km) from the causative fault and a M 8.0 earthquake 70 miles (112 km) from the causative fault, respectively;

- the horizontal component N340 of the Old Appleton accelerogram recorded during the Cape Girardeau (M = 4.8) earthquake of 26 September 1990 at an epicentral distance of 29 miles (46 km) with a peak acceleration of 0.17 g; and

- the N21E component of the Taft-Lincoln School accelerogram recorded during the 1952 Kern County, California earthquake (M = 7.7) with a peak acceleration of 0.16 g.

The two synthetic accelerograms and the Kern County record were selected as the best large magnitude records available for use in the analysis. The Cape Girardeau record was selected because it was the best record available from the New Madrid seismic zone. It should be noted that the Cape Girardeau record was an unusually long record for a magnitude 4.8 event, with significant motions for over 50 seconds. The representative acceleration time histories selected for the design earthquake are shown in Figure 4.
FIGURE 4
TIME HISTORIES USED FOR ANALYSIS
CENTRAL U.S. LANDFILL

SYNTHETIC ACCELEROGRAM D1-SYN

ACCELERATION (g)

0.6

0.4

0.2

0.0

-0.2

-0.4

-0.6

0 1 2 3 4 5 6 7 8 9 10 11
TIME (sec)

AVG. FREQ.: 3.92 Hz

TAFT N21E ACCELERO

ACCELERATION (g)

0.2

0.1

0.0

-0.1

-0.2

-0.3

0 10 20 30 40
TIME (sec)

AVG. f

SYNTHETIC ACCELEROGRAM B2-SYN

ACCELERATION (g)

0.4

0.2

0.1

0.0

-0.1

-0.2

-0.4

0 5 10 15 20 25 30 35 40 45 50
TIME (sec)

AVG. FREQ.: 3.28 Hz

OLD APPLETON ACCELER

ACCELERATION (g)

0.2

0.1

0.0

-0.1

-0.2

-0.3

0 10 20 30 40 50 60
TIME (sec)

AVG. f
Seismic Resistance of Landfill and Foundation Soils

Potentially liquefiable soils were identified to be isolated, near-surface pockets of loose granular soil scattered randomly across the site. The liquefaction resistance of these soil pockets was evaluated based upon SPT blow counts and the correlation developed by Seed et al. (1985). The minimum cyclic shear strength ratio for the liquefiable soil pockets was estimated to be 0.124.

The pseudo-static slope stability analyses used a friction angle of 8 degrees and an adhesion of zero for sliding along the liner system. These parameters represent the interface between the smooth HDPE geomembrane and overlying cushion geotextile. These parameters were selected based on project experience and published interface shear strength studies. The analyses conducted for potential slip surfaces through or along the landfill liner system predicted a critical yield acceleration of greater than 0.4 g. The critical potential slip surface passed along the liner system over most of its length.

Design of the final cover system was not completed at the time of the seismic analysis. For purposes of analysis, the cover system was assumed to include a textured geomembrane with an overlying cushion geotextile. The interface between these two geosynthetics was assumed to be the weakest interface in the cover system and was assigned an interface friction angle of 24 degrees and no adhesion. Based upon the equations for the stability of an infinite slope and the cover slope angle of 4H:1V, the yield acceleration of the final cover system was estimated to be 0.18 g.

Site Response Analysis

Site response analyses were conducted using SHAKE for two typical profiles: one profile for the free field condition (no waste) to estimate the earthquake-induced cyclic shear stress ratio for liquefaction analyses, and one with 135 ft (41 m) of waste to evaluate the potential for amplification of motions at the top of the landfill. The dynamic properties used for the existing municipal waste included: (i) MSW unit weights varying from 35 lb/ft³ (5.5 kN/m³) at the top
of the waste to 75 lb/ft$^3$ (11.8 kN/m$^3$) at the bottom; (ii) modulus reduction curve similar to that given for peat by Seed and Idriss [1970]; and (iii) damping curve similar to that given for clay by Seed and Idriss [1970]. Two different shear wave velocity profiles were used; one corresponding to young waste (immediately after closure) and one corresponding to old waste (the end of the post-closure monitoring period). The shear wave velocities ranged from approximately 315 ft/sec (95 m/sec) at the top of the waste for the young waste analysis to 1320 ft/sec (400 m/sec) at the base of the waste for the old waste analysis. For both young and old waste profiles, the MSW shear wave velocity increased systematically with depth.

Sixteen site response analyses were conducted considering the four design earthquake accelerograms, the two waste shear wave velocity profiles, and the two profile locations. The following results were obtained:

- cyclic shear stress ratios for the potentially liquefiable soil pockets ranged up to 0.214; and

- the maximum acceleration at the top of the landfill was predicted to be approximately 0.38 g; the maximum average acceleration of the landfill mass was much lower than this value.

**Evaluation of Seismic Impacts**

The calculated minimum cyclic shear strength ratio for the liquefiable soil pockets was 0.124. The site response analysis gave cyclic shear stress ratios as high as 0.214. Therefore, it was concluded that liquefaction might occur in some of the more critical soil pockets during the design earthquake. However, the identified liquefiable soil pockets were all located beyond the toes of the two waste units at the site, in the vicinity of the intermittent stream. Post-earthquake static stability analyses using residual shear strengths in the liquefiable zones yielded factors of safety greater than 1.0, indicating that lateral spreading would not occur beneath the waste units. Liquefaction settlement analyses yielded settlements of between 1 and 2.5 in. (25
and 60 mm) in the liquefiable zone. This amount of settlement was not considered to be a threat to the integrity of the containment system.

The site response analysis predicted that the peak acceleration value on the waste surface would be 0.38 g. This value is less than the critical yield acceleration of 0.4 g that was calculated for failure surfaces passing along the liner system. Therefore, permanent seismic deformations induced by the design earthquake along the liner system were taken to be negligible. For the cover system, the yield acceleration was estimated to be on the order of 0.18 g. Simplified Newmark deformation analyses using the worst case acceleration time history for the cover system from the response analyses, the case with 0.38 g peak acceleration yielded a permanent seismic deformation of approximately 1 in. (25 mm), well within the range of deformation considered acceptable.

Summary

The seismic analysis indicated that liquefaction might occur during the design earthquake in isolated pockets of loose, granular soil at the site. Additional analysis showed that liquefaction of these soil pockets would not affect the integrity of the landfill liner system. The seismic analysis also indicated that any permanent deformations that might develop during the design earthquake along surfaces that passed through or along the landfill liner system would be negligible and that seismically-induced permanent deformation along surfaces passing through the final cover would be well within the range of deformations considered acceptable. Based on these results it was concluded that the integrity of the landfill liner system would be maintained during the design earthquake.
PROJECT C - MUNICIPAL WASTE COMBUSTION ASFILL IN THE NORTHEASTERN UNITED STATES

Introduction

This project illustrates the application of the seismic analysis approach to a municipal waste combustion (MWC) ashfill site in a glacial outwash plain in the northeastern United States. This project provides an example of use of the results of field geophysical testing to establish dynamic properties for a site response analysis.

Subsurface Conditions and Ashfill Design

The site is located on a glacial outwash plain. The site has a sequence of granular and cohesive sedimentary and glacial moraine soils with total thickness of approximately 1500 ft (460 m). The upper 90 ft (27 m) of soil consists of a granular glacial outwash material, primarily sand. The ground-water table at the site is generally at an elevation 10 ft (3 m) below the ground surface. Standard Penetration Test (SPT) blow counts obtained in the glacial outwash soil averaged from 20 to 80, indicating a medium dense to dense material. Some areas of the ashfill will be founded on an existing municipal waste fill.

The ashfill design has the following geometric features:

- MWC ash grades as steep as 3H:1V for conditions during the active life of the ashfill;
- MWC ash grades of 3.5:1V (average) for ashfill post-closure conditions;
- MWC ash placement in a sidehill fill configuration over an existing municipal waste slope such that the maximum ash slope height is approximately 150 ft (46 m), ash thickness varies from 0 to 75 ft (23 m), and existing municipal waste thickness under the ashfill varies from approximately 0 to 160 ft (49 m) (Figure 5); and
FIGURE 5
TYPICAL CROSS SECTION SHOWING PROFILE LOCATIONS
AND CRITICAL POTENTIAL SLIP SURFACE
PROJECT C

CRITICAL SURFACE
\( a_{\text{Yield}} = 0.097 \text{g} \)

PROFILE 1
PROFILE 2
PROFILE 3
PROFILE 4

MWC ASH
EXISTING WASTE
TOP OF SLIDE SLI LINER SY
TOP OF BASE LINER SYSTEM

SOIL

0 SCALE IN
• base liner system grades of approximately one percent and side slope liner system grades of up to 2.5H:1V.

The ashfill design incorporates a complex double composite liner system in the base areas (Figure 6). This liner system includes two smooth HDPE geomembranes, filter geotextiles, geonets, a geosynthetic clay liner (GCL), a compacted clay liner, and a granular leachate collection layer. Based on the project material and installation requirements, two interfaces within the system may potentially control the minimum shear strength. These interfaces are between smooth HDPE geomembrane and overlying geonet and between the compacted clay liner and the overlying smooth HDPE geomembrane.

The design also incorporates a double liner system in the side slope areas (Figure 6). This liner system consists of a single primary liner and a composite secondary liner and includes two textured HDPE geomembranes, two-sided geocomposites (i.e. geonet with filter geotextiles bonded to both sides), a compacted clay liner, and a granular leachate collection layer. It is believed that the interface between the compacted clay liner and the overlying textured HDPE geomembrane will control the minimum shear strength.

Design of the final cover system was not completed at the time of the seismic analysis. An assumed final cover design incorporating, from bottom to top, a levelling soil course, a textured HDPE geomembrane, a two-sided geocomposite, and a protective cover soil layer was therefore used (Figure 6). It is believed that any of the three interfaces within this system may potentially control the minimum shear strength of the cover.

**Design Earthquake**

The design acceleration for the site from the Algermissen et al. [1990] map is 0.30 g. Seismological literature indicates that the governing seismic source zone is capable of generating a maximum earthquake magnitude of 7.0. However, a site-specific seismic hazard analysis conducted for the site indicated the seismicity of the site was governed by a nearby smaller magnitude earthquake. The analysis indicated that the appropriate design earthquake magnitude
PROJECT C

Cover Soil

Geocomposite
Textured HDPE Geomembrane

Levelling Course

FINAL COVER SYSTEM

LCRS Sand

Geocomposite
Textured HDPE Geomembrane
Geocomposite
Textured HDPE Geomembrane

Low-Permeability Soil

SIDE SLOPE LINER SYSTEM

LCRS Sand

Geotextile Filter
Geonet
Smooth HDPE Geomembrane
GCL

Structural Fill

Geotextile Filter
Geonet
Smooth HDPE Geomembrane

Low-Permeability Soil

BASE LINER SYSTEM
for the 0.30 g design acceleration was 5.0 to 6.0 and the appropriate epicentral distance was approximately 6 to 16 mi (10 to 26 km).

A suite of three recorded accelerograms were selected to represent the design earthquake. Two of the accelerograms were from eastern North America events. These were the Quebec and New Brunswick accelerograms described in the section of this paper on Project A. A third event was chosen from the catalog of available records from the western United States. Information on the third accelerogram is provided below:

- earthquake of magnitude 5.2 occurring in 1975 in Humboldt County, California; recording site at epicentral distance of 21 mi (33 km); maximum recorded horizontal acceleration of 0.20 g;

The Humboldt County record was considered appropriate because of the similarities in magnitude, acceleration, and epicentral distance to the design event and because it was a deep earthquake that did not produce surface rupture. Each of the three accelerograms is from an earthquake event that did not produce ground surface rupture and each was recorded on rock. The maximum recorded acceleration for each accelerogram was scaled to 0.30 g, the design acceleration, prior to use in the site response analysis.

Seismic Resistance of Ashfill and Foundation Soils

The liquefaction resistance of the granular glacial outwash soil was evaluated based upon the SPT blow counts. The minimum cyclic shear strength ratio for the soil was estimated to be 0.125, although the average cyclic shear strength ratio was considerably higher. Available information suggested that the MWC ash would not become saturated and was therefore not potentially liquefiable.

The pseudo-static slope stability analyses used the following shear strength parameters to represent the weakest interfaces in the liner and final cover systems. These parameters were selected based on project experience and published interface shear strength studies.
• A friction angle of 8 degrees and an adhesion of zero were used for sliding along the base liner system. These parameters represent the strength of either of the two potential weak links in the base liner system; the interfaces between smooth HDPE geomembrane and overlying geonet and between compacted clay liner and overlying smooth HDPE geomembrane.

• A friction angle of 21 degrees and an adhesion of zero were used for sliding along the side slope liner system. These parameters represent the interface between compacted clay liner and overlying textured HDPE geomembrane.

• A friction angle of 25 degrees and an adhesion of zero were used for sliding along the final cover system. These parameters represent the minimum anticipated strength of the three interfaces within the system.

The slope stability analyses for potential slip surfaces through or along the ashfill liner system predicted critical yield accelerations of 0.085 g for active-life conditions and 0.097 g for post-closure conditions. The critical potential slip surfaces for both conditions passed along the base and side slope liner systems over virtually all of their length. The slope stability analyses also predicted a critical yield acceleration of 0.144 g for the final cover system.

Site Response Analysis

Site response analyses were conducted for the three design earthquake accelerograms and for four profile locations with varying MWC ash and existing municipal waste thicknesses (Figure 5). In the four profiles, MWC ash thickness varied from 0 to 65 ft (20 m) and existing municipal waste thickness varied from 0 to 135 ft (41 m). The dynamic properties used for the existing municipal waste included: (i) shear wave velocities from 700 ft/sec (215 m/sec) near the surface to 1050 ft/sec (320 m/sec) at a depth of 135 ft (41 m); (ii) unit weights from 60 lb/ft³ (9.4 kN/) near the top of the waste to 90 lb/ft³ (14.1 kN/m³) at a depth of 135 ft (41 m); (iii) modulus reduction curve similar to that given for peat by Seed and Idriss [1970]; and (iv) damping curve similar to that given for clay by Seed and Idriss [1970].
A literature search did not identify any information on dynamic properties of MWC ash. Published information on other physical and chemical properties of MWC ash suggested that the properties varied widely depending on incinerator operations. The fact that some MWC ash material is known to harden after placement, like a weak concrete, was particularly significant because hardening would strongly affect dynamic response. Given this uncertainty, geophysical testing was conducted at the site to provide direct measurement of dynamic properties for the site response analysis.

The field geophysical testing involved determination of shear wave velocities in existing MWC ash fills at the project site. Tests were conducted at 12 locations at the site having MWC ash fills of varying thickness and age. The shear wave velocity determinations were made using the controlled-source spectral analysis of surface waves method [Kavazanjian et al., 1994]. The shear wave velocities obtained from the tests for ash depths of 0 to 60 ft (18 m) ranged from 320 to 1140 ft/sec (100 to 350 m/sec), with an average of 625 ft/sec (190 m/sec). These results indicated that the MWC ash behaved like a dense granular soil with significant variability and that there was not widespread hardening of the MWC ash.

Based on the field testing, four models of the MWC ash, representing measured ranges of variation in shear wave velocity versus depth, were developed for use in the site response analysis. The four ash models ranged from a soft model with shear wave velocities from 325 to 700 ft/sec (100 to 215 m/sec) to a very stiff model with a constant shear wave velocity of 1100 ft/sec (335 m/sec). The dynamic properties used for the MWC ash also included a unit weight of 100 lb/ft³ (15.7 kN/m³) and modulus reduction and damping curves after those given by Seed and Idriss [1970] for sand.

The site response analyses were conducted considering a wide range of conditions corresponding to the three design earthquake accelerograms, the four profile locations, and the four ash models. The following results were obtained:

- the maximum cyclic shear stress ratio for the granular glacial outwash soil was 0.115;
- time histories of average ashfill mass acceleration predicted by the analyses had a maximum peak value of 0.10 g, representing a large attenuation of the design acceleration of 0.30 g; and

- time histories of average final cover mass acceleration predicted by the analyses had peak values of 0.17 g, representing significant attenuation of the design acceleration of 0.30 g.

Evaluation of Seismic Impacts

The calculated minimum cyclic shear strength ratio for the granular glacial outwash soil was 0.125. The site response analysis predicted a maximum earthquake-induced cyclic shear stress ratio of 0.115. Therefore, liquefaction of foundation soils is not expected during the design earthquake.

The site response analysis predicted that the peak average acceleration for the ashfill mass was 0.10 g. This value slightly exceeds the critical yield accelerations of 0.085 g and 0.097 g that were calculated for active-life and post-closure ashfill conditions, respectively. Since the peak average acceleration of the landfill was less than twice the yield acceleration, significant permanent seismic deformation is not expected. However, for purposes of completeness, formal seismic deformation analyses were conducted to evaluate the magnitude of permanent deformations that might be induced by the design earthquake. These analyses predicted permanent deformations of less than 0.1 in (2.5 mm), a value considered negligible, for both conditions.

Similarly, the site response analysis predicted that the peak acceleration value for the average final cover system mass was 0.17 g. This value slightly exceeds the critical yield acceleration of 0.144 g that was calculated for the final cover system. Seismic deformation analyses were also conducted for this case to evaluate the magnitude of permanent deformations that might be induced by the design earthquake. These analyses also predicted permanent deformations of less than 0.1 in (2.5 mm), a value considered negligible.
Summary

The seismic analysis indicated that liquefaction was not expected during the design earthquake. The seismic analysis also indicated that, although the yield acceleration might be exceeded for surfaces passing through both the liner and cover, negligible permanent deformations would develop during the design earthquake along these surfaces. Based on these results it was concluded that the integrity of the ashfill liner system and final cover system would be maintained in the design earthquake.
PROJECT D - CANYON LANDFILL IN THE WESTERN UNITED STATES

Introduction

This project illustrates the application of the seismic analysis approach to a canyon landfill in the western United States. This project provides an example of the use of a project-specific seismic hazard analysis and of the need to consider damage potential from distant, large magnitude earthquakes that may produce horizontal accelerations at the site that are smaller than the design acceleration.

Subsurface Conditions and Landfill Design

The site is located in desert terrain. Subsurface conditions consist of thin layers of colluvium on the canyon side slopes and alluvium in the canyon bottom underlain by well-indurated sedimentary and crystalline rock. The colluvium and alluvium will be excavated, processed, and stockpiled for use in landfill construction as foundation materials and as daily and intermediate cover.

The landfill design has the following geometric features:

- waste thicknesses of up to 300 ft (90 m);
- canyon side slopes as steep as 2H:1V;
- waste grades as steep as 3H:1V for the final cover; and
- base liner grades a minimum of 5 percent and as steep as 3H:1V along the axis of the canyon.

The canyon landfill design employs a composite liner system and a single geomembrane in the final cover. Figure 7 provides a detail of the liner and cover systems. The liner system
includes a GCL as the low permeability component of the composite liner system on side slopes steeper than 3H:1V. Cushion geotextiles are used on top of the geomembranes in the liner and cover systems. The cover geomembrane is textured on both sides while the bottom geomembrane is smooth on top, textured on the bottom. The critical interfaces were identified as the geotextile-cover soil interface in the final cover and as the geotextile-smooth geomembrane interface in the liner system.

Based upon project-specific testing using on-site soils and the materials proposed for construction, the shear strength of the geotextile-cover soil interface in the cover system was described by zero adhesion and a friction angle of 16 to 24 degrees, depending on what materials were used. The shear strength of the geotextile-smooth geomembrane interface in the liner was described by zero adhesion and a friction angle of between 6 and 10 degrees, depending on the material.

Design Earthquake

The design acceleration for the site from the Algermissen et al. (1990) map was slightly greater than 0.6 g. A site-specific seismic hazard analysis was performed. Figure 8 shows the site-specific hazard analysis results. Site seismicity was governed by non-fault-specific (random) regional seismicity. The random seismicity was assigned a maximum magnitude of 6.5. The one active fault in the vicinity of the site (Fault 1 in Figure 8) was deemed capable of generating a magnitude 7 event once every 2,000 to 3,000 years. All other sources of seismicity were in neighboring seismic source zones. One of these sources was a major fault (Fault 2 in Figure 7) deemed capable of generating a great earthquake of between magnitude 7.5 and 8 once every 200 to 300 years.

The site specific hazard analysis yielded 0.56 g as the peak horizontal acceleration with a 10 percent probability of exceedance in 250 years. The magnitude distribution corresponding to the acceleration, also shown in Figure 8, indicated that over 98 percent of the earthquakes contributing to this acceleration were from the random regional source and were of magnitude
6.5 or smaller. Based upon this result, a magnitude of between 6.0 and 6.5 was assigned to the 0.56 g acceleration to describe the design earthquake.

A suite of eleven different accelerograms recorded in California from earthquakes of magnitude 5.9 to 7.1 were used to characterize the design earthquake with a 0.56 g peak intensity. These time histories were as follows:

- the Gilroy record from the M5.9 Coyote Lake earthquake; peak acceleration of this record was 0.32 g;

- both components of the M5.9 Whittier recording at the Garvey Reservoir Control Building; peak accelerations of these records were 0.37 g and 0.48 g;

- both components of the Cholame 5 record from the 1966 M6.0 Parkfield event; the peak accelerations of these records were 0.35 g and 0.43 g;

- both components of the Cholame 8 record from the 1966 M6.0 Parkfield event; the peak accelerations from these records were 0.24 g and 0.28 g;

- both components of the Corralitos record from the 1989 M7.1 Loma Prieta earthquake; the peak accelerations of these records were 0.48 g and 0.63 g; and

- both components of the University of California, Santa Cruz record from the 1989 M7.1 Loma Prieta earthquake; the peak acceleration of these records were 0.41 g and 0.44 g.

It was considered prudent to also consider the impact of a great earthquake on Fault 2 on the landfill. Based upon the mean plus one standard deviation value for the acceleration induced at the site from a magnitude 7.7 event on Fault 2, a design acceleration of 0.16 g was assigned to the intensity of this event at the project site.
Selection of representative recorded accelerograms for the magnitude 7.7 event was complicated by the lack of available rock site records from large magnitude earthquakes. The following four accelerograms were selected to represent the magnitude 7.7 design event with a peak acceleration of 0.16 at the site:

- both components of the Taft record from the M7.7 Kern County earthquake of 1952; the peak accelerations of these records were 0.16 g and 0.18 g.

- the Silent Valley record from the M7.4 Landers earthquake of 1992; the peak acceleration of this record was 0.05 g; and

- the Twenty-Nine Palms record from the M7.4 Landers earthquake of 1992; the peak acceleration of this record was 0.06 g.

Seismic Resistance of the Landfill

As the landfill was founded upon rock, a liquefaction analysis was not required for this project. Landfill stability analyses were conducted to find the yield acceleration for potential failure surfaces passing through the liner, waste, and cover. Figure 9 shows the variety of failure surfaces considered in the stability analyses.

Results of the stability analyses indicated that the yield acceleration for failure surfaces passing through the liner system varied from 0.05 g to 0.15 g, depending on the choice of liner system materials. Yield accelerations for the final cover varied from 0.15 g to 0.25 g, depending on the choice of materials. The sensitivity of the yield acceleration to permanent seismic deformation was evaluated to determine what yield acceleration values, and hence what materials, were acceptable for construction of the landfill.
CATEGORY 1: NON-CIRCULAR SURFACE
A-G-H-F
[PASSING ENTIRELY ALONG LINER SYSTEM]

CATEGORY 2: NON-CIRCULAR SURFACES
[PASSING ALONG A PORTION OF LINER SYSTEM]

CATEGORY 3: CIRCULAR SURFACE
A-D
[PASSING ALMOST ENTIRELY THROUGH WASTE]
Seismic Response Analysis

Since the landfill was founded upon rock, response analyses were not required to evaluate the effect of local soil conditions on the earthquake motions. However, response analyses were required to evaluate the response of the landfill to the seismic motions. Landfill response analyses were performed for waste thicknesses of 50, 150, and 300 feet (15, 45 and 90 meters).

Response analyses were performed using SHAKE. Material properties of the waste were based upon the results of field investigations and laboratory tests conducted for the Puente Hills landfill in Los Angeles (Earth Technology, 1988), essentially the same properties as used in Project A. In the analysis for the M6.0 to 6.5 design earthquake of intensity 0.56 g, the average acceleration of the waste mass always showed attenuation, while the acceleration of the cover occasionally indicated amplification. Amplification occurred in cases where the predominant frequency of the waste column closely matched the predominant frequency of the input acceleration. The worst case occurred with the Gilroy record for a 50 ft (15 m) waste thickness, where an acceleration of 0.88 g at the top of the cover was calculated. Even in this case, the average acceleration of the waste mass showed significant attenuation.

Response analyses for the 0.16 g, M7.5 to 8.0 great earthquake on Fault 2 always showed amplification at the top of the landfill. Amplification factors as high as 3.5 were calculated at the top of the cover. However, even when the amplification factor for the cover was 3.5, the average acceleration of the waste mass did not exceed the free field peak ground acceleration.

Evaluation of Seismic Impact

Average acceleration time histories for the waste and cover from the site response analyses were used in Newmark deformation analyses to estimate permanent seismic deformations for the liner and cover. Permanent deformations were evaluated as a function of yield acceleration to aid in the selection of materials and preparation of construction specifications. Results of the analyses are presented in Figures 10 and 11 for the liner and cover systems, respectively.
FIGURE 10
PERMANENT DEFORMATION
OF WASTE MASS
PROJECT D

- ▲ 50 FT. OF WASTE, TAFT RECORD N21E COMPONENT (M7.7)
- □ 50 FT. OF WASTE, TAFT RECORD S89E COMPONENT (M7.7)
- ○ 50 FT. OF WASTE, SILENT VALLEY RECORD (M7.4)
- ▽ ALL OTHER ANALYSES FOR M7+ EVENTS, NO DEFORMATION

- UPPER BOUND, M6.5 EARTHQUAKES, 0.56g
FIGURE 11
PERMANENT DEFORMATION
OF LANDFILL COVER
PROJECT D

- □ 150 FT. OF WASTE, TAFT RECORD N21E COMPONENT (M7.7)
- △ 300 FT. OF WASTE, TAFT RECORD N21E COMPONENT (M7.7)
- ▼ MAXIMUM, ALL OTHER RECORDS, M7+ EVENTS
- ─── UPPER BOUND, M6.5 EARTHQUAKES, 0.56g

SEISMIC DEFORMATION (Inches)

M7+ RECORDS SCALED TO 0.16g

YIELD ACCELERATION (g's)
Summary

Results of the analyses showed that while the larger intensity/smaller magnitude design earthquake clearly governed the seismic performance of the waste mass, the large magnitude/smaller intensity earthquake from Fault 2 was equally important with respect to performance of the cover. If the site had been slightly closer to Fault 2, the larger magnitude-smaller intensity earthquake from Fault 2 may actually have governed cover performance. The analyses also helped determine acceptable materials for construction of the landfill and minimum acceptable interface shear strength values for inclusion in construction specifications.
CONCLUSION

Seismic impact analyses for MSW landfills used in practice today are based upon seismic analyses developed by geotechnical engineers for analysis and design of earth structures. Geotechnical seismic impact analyses applied in landfill engineering include:

- liquefaction potential analyses the stability and settlement of foundation soils;

- limit equilibrium analyses to estimate the yield accelerations of the liner and cover systems and for residual shear strength based post-earthquake foundation stability analyses;

- seismic response analyses to evaluate the influence of local soil conditions on earthquake ground motions and the response of the waste mass to seismic excitation; and

- Newmark seismic deformation analyses to estimate permanent seismic deformations of the liner and cover systems.

Case histories of seismic impact analyses for four landfills distributed across the United States illustrate the application of these geotechnical analyses in landfill engineering practice.
REFERENCES


Jennings, P.C., Housner, G.W., and Tsai, N.C., Simulated Earthquake Motions, Earthquake Engineering Laboratory Report, California Institute of Technology, Pasadena, California, 1968


