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STATE OF THE ART (SOA14)
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SYNOPSIS: The record of performance of landfills in earthquakes is excellent. However, the advent of geosynthetic liner and cover systems has increased the susceptibility of modern landfills to seismically-induced instability and deformations. Analyses used to assess the performance of landfills in earthquakes include site response, limit equilibrium stability, and Newmark deformation analyses. Well documented case histories of the behavior of landfills subject to seismic loading are necessary to improve knowledge of the parameters required for these analyses and thereby enhance the reliability of seismic performance evaluations for landfills.

INTRODUCTION

The performance of landfills under seismic loading has been a subject of increasing interest to the geotechnical profession in recent years. Articles discussing some element of the seismic design of landfills or waste containment systems can be found in virtually every major conference involving earthquake engineering during the past five years. The recently completed ASCE Geoenvironment 2000 conference in New Orleans and the upcoming ASCE 1995 National Convention in San Diego both contain sessions dedicated to the seismic response of waste containment systems. As additional evidence of this interest, the National Science Foundation (NSF) recently sponsored a two-day workshop entitled Seismic Design of Solid Waste Landfills (USC, 1993), at least six research proposal on evaluating landfill performance were submitted in response to NSF's 1994 solicitation for research related to the Northridge Earthquake, and the United States Environmental Protection Agency (USEPA) will soon publish a guidance document titled RCRA Subtitle D (258) Seismic Design Guidance for Municipal Solid Waste Landfill Facilities (USEPA, 1994).

The significant increase in professional interest in the performance of landfills under seismic loading over the past five years appears to be the result of several factors. Perhaps the most important force driving this surge in interest results from the recent promulgation of Section 258 of the Federal Resource Conservation and Recovery Act (RCRA) regulating the siting, design, monitoring, and closure of Municipal Solid Waste (MSW) landfill facilities. These regulations, commonly known as Subtitle D, had two impacts on seismic design of MSW landfills. First, by setting relatively stringent standards on the areas in which seismic performance of landfills must be evaluated and on the intensity of the design earthquake, Subtitle D significantly increased the number of landfills for which seismic loading is a factor in design. Second, by explicitly mandating geomembrane liners for all Subtitle D landfills and implicitly requiring geomembranes in the prescriptive cover standard, Subtitle D has increased the susceptibility of modern landfills to instability and deformations induced by seismic loading.

Fig. 1: Seismic Impact Zones (after Algermissen, 1990)

Subtitle D requires that seismic performance be evaluated for new MSW landfills and lateral expansions of existing MSW landfills if they are located in a "seismic impact zone." A seismic impact zone is defined as any location in the United States where the peak horizontal ground acceleration (PGA, also referred to in Subtitle D as the Maximum Horizontal Acceleration) in lithified earth with a 90 percent probability of not being exceeded in 250 years, expressed as a percentage of the acceleration of gravity, is greater than 10 percent. Figure 1 shows that, based upon the most recent version of United States Geological Survey (USGS) Map Sheet MF-2120 (Algermissen, et al., 1990), seismic impact zones cover not only most of the western United States but also large parts of the central and eastern

United States. As a result, many practicing engineers are now faced with the need to consider seismic loading in their landfill design projects in areas where seismic design was previously not a design consideration.

Subtitle D explicitly requires that all new MSW landfills and lateral expansions of existing MSW landfills have composite liner systems containing geomembrane liners on their base and side slopes. By requiring that the liquid flux through the cover of the landfill is less than the liquid flux through the base, Subtitle D also implicitly requires a geosynthetic cover over areas with a geosynthetic liner, unless a properly designed alternative soil cover (e.g., a soil cover with a capillary break) is employed. The potential for reduced stability at soil/geosynthetic interfaces due to relatively low interface shear strengths is well recognized in the profession (e.g., the problem with the waste repository at Kettleman Hills described by Seed et al., 1990). The potential for weak interfaces in the liner and cover systems of Subtitle D landfills due to the inclusion of geomembranes has also increased the importance of seismic considerations in design of Subtitle D landfills.

Seismic design is also an important consideration for other types of landfills and waste containment systems besides MSW landfills. As the hazardous waste containment design standards provided in Subtitle C of RCRA do not explicitly address seismic loading, it is reasonable to assume that the Subtitle D MSW design criteria provide a minimum design standard for Subtitle C landfills. Furthermore, seismic design is an important consideration for many Superfund remediation projects and for design of mixed, low level, and high level waste repositories. One of the writers has recently worked on two Superfund projects where Subtitle D was cited as the Applicable or Relevant and Appropriate Requirement (ARAR) for seismic design.

In addition to the regulatory imperative, seismic design of landfills represents one of the areas within geotechnical earthquake engineering that, until recently, has not been treated extensively. For example, it has been over 20 years since the first research work on soil liquefaction was conducted. Countless studies on a wide variety of aspects of liquefaction have been completed and published in that period. Published studies on the seismic design of landfills only began to appear five years ago.

An additional reason for the interest in the seismic response of landfills is the unique characteristics of the solid waste material (refuse) that is found in landfills. This material is difficult to test using conventional dynamic or cyclic testing methods. This difficulty has led to a number of investigations of the performance of landfills in recent earthquakes in order to be able to understand and predict landfill performance during seismic events. In attempting to satisfy regulatory mandates, these and other limitations associated with current practice for seismic design of landfill slopes have become apparent to the profession. In response to these limitations, a variety of organization and individuals have engaged and are currently engaged in research on and development of design methods for the seismic design of MSW landfills.

The purpose of this paper is to review the state-of-knowledge on the seismic performance of landfills and the state-of-the-practice for evaluating the stability and deformation of landfills subject to seismic loading. The paper will start with a summary of the current USEPA Subtitle D regulatory requirements, which serve as one of the main impetuses for the increased interest in seismic performance of landfills. Following this discussion, observations of performance during the 1994 Northridge event. The paper will then review the state-of-the practice relative to 1) estimating ground motions for landfill design, 2) predicting the seismic response of solid waste landfills, and 3) evaluating the stability and deformation of landfill slopes. To the extent possible, limitations and uncertainties associated with each of these topics will be identified with the hope that the prudent engineer will consider them in their design studies. The paper will conclude with a summary of the authors' conclusions and recommendations regarding the seismic design of landfills.

REVIEW OF SUBTITLE D REQUIREMENTS

On October 9, 1993, the RCRA Subtitle D regulations (40 CFR Part 258) went into effect. These regulations are applicable to landfills receiving non-hazardous municipal solid waste and establish minimum Federal criteria for siting, design, ground-water monitoring, and closure/post closure care of Municipal Solid Waste Landfill Facilities (MSWLF). Every State and Indian Tribal authority is required to adopt regulations that are at least as stringent as the Subtitle D requirements related to the seismic design of landfills. In many cases, this has simply meant that the Subtitle D seismic design criteria were adopted directly. However, in some cases, for instance in California, regulations and guidelines for the seismic design of landfills are more specific than Subtitle D.

Subtitle D Regulations

Seismic design requirements for MSW landfills are covered in Section 258.14 of the Subtitle D regulations. This section states that

> *New MSWLF units and lateral expansions shall not be located in seismic impact zones, unless the owner or operator demonstrates to the Director or operator demonstrates to the Director of an approved State/Tribe that all containment structures, including liners, leachate collection systems, and surface water*

control systems, are designed to resist the maximum horizontal acceleration in lithified earth material for the site. The owner or operator must place the demonstration in the operating record and notify the State Director that it has been placed in the operating record.

Section 258.14 goes on to provide specific definitions for seismic impact zone, the maximum horizontal acceleration in lithified earth materials, and lithified earth materials. These definitions are as follows:

> ***Seismic Impact Zone**: This zone involves areas with a 10 percent or greater probability that the maximum horizontal acceleration in lithified earth material, expressed as a percentage of the earth's gravitation pull (g) will exceed 0.10 g in 250 years.*
>
> ***Maximum Horizontal Acceleration**: This refers to the maximum expected horizontal acceleration depicted on a seismic hazard map, with a 90 percent or greater probability that the acceleration will not be exceeded in 250 years, or the maximum expected horizontal acceleration based on a site-specific seismic risk assessment.*
>
> ***Lithified Earth**: This refers to rock, including all naturally occurring and naturally formed aggregates or masses of minerals or small particles of older rock that formed by crystallization of magma or by inundation of loose sediments. This term does not include man-made materials, such as fill, concrete, and asphalt, or unconsolidated earth materials, soil, or regolith lying at or near the earth surface.*

Impact of Subtitle D

One of the primary impacts of Subtitle D on seismic design of MSW landfills has been the definition of seismic impact zones, the areas in which satisfactory performance of the landfill under seismic loading must be demonstrated. Based upon the latest version of USGS Map Sheet MF-2120, identified in Subtitle D guidance documents produced by USEPA as the prescriptive means of defining seismic impact zones (USEPA, 1992; USEPA, 1994), approximately 40 percent of the continental United States lies within a seismic impact zone and is thus subject to Subtitle D standards of care. The seismic impact zones defined by USGS Map Sheet MF-2120, sometimes known as the Algermissen maps, are shown in Figure 1.

As the prescriptive MHA defined by Subtitle D corresponds to a PGA with a mean recurrence interval of approximately 2,375 years, the design earthquake for a MSW landfill in a seismic impact zone is a relatively rare, extreme event. As a result of these factors, MSW landfills in large portions of the continental United States, areas in which seismic design has not traditionally been of concern, must now be designed to resist relatively high levels of earthquake loading. Paradoxically, in many of these seismic impact zones, transportation and communication lifelines and critical facilities are either designed for lower levels of seismic loading or are simply not designed for seismic loading at all.

Seismic Performance Requirements

Subtitle D does not explicitly treat analyses of the seismic performance of MSW landfills. Rather, it requires owners or operators of new MSW landfill units and lateral expansions of existing MSW landfills located in a seismic impact zone to place in the operating record a demonstration that engineering measures have been incorporated into the design to "ensure" the integrity of the structural components of the MSW landfill unit. However, a seismic stability and deformation analysis of the waste mass is generally a necessary step in demonstrating that the containment system will maintain its integrity in the design earthquake. Other topics related to the seismic performance of MSW landfills, not covered in this paper, are addressed in Sections 258.13 and 258.15 of Subtitle D.

Section 258.13 of Subtitle D provides siting restrictions with respect to fault areas. These restrictions preclude locating new MSW landfill units or lateral expansions within 60 m of a fault that has had displacement in Holocene time, unless the owner or operator demonstrates that an alternative setback distance of less than 60 m will prevent damage to the structural integrity of the MSW landfill unit and will be protective of human health and the environment. Subtitle D defines Holocene time as the most recent epoch of the Quaternary period, extending from the end of the Pleistocene Epoch to the present (approximately the past 10,000 to 11,000 years).

Section 258.15 of Subtitle D provides siting restrictions with respect to geologically unstable areas. This section requires that new MSW landfill units or lateral expansion that are located in an unstable area include engineering measures designed to "ensure" the integrity of the structure components of the MSW landfill unit. Unstable areas are defined in USEPA Subtitle D guidance documents (e.g., USEPA, 1992) to include areas susceptible to liquefaction-induced instability and other modes of seismically-induced displacement of natural slopes and foundation soils.

OBSERVATIONS OF LANDFILL PERFORMANCE DURING EARTHQUAKES

As with most areas in geotechnical earthquake engineering, observations of the performance of landfills during earthquakes provide the most reliable means of both identifying modes of damage for which seismic performance analyses are required and for calibrating the performance analyses of the landfill contaminant system. Ideally, calibration of seismic performance analyses involve case histories where material properties and physical conditions before the seismic event are well-established, where instrumented recordings of performance during the event exist, and where secondary effects have not led to ambiguous interpretations of performance. Realistically, few case histories of any kind in geotechnical practice and no landfill case histories meet these ideal requirements. In the absence of well-documented case histories of modern landfill performance in recent earthquakes, much must be left to observations and back analyses of the performance of older landfills based upon assumptions on landfill geometry, solid waste properties, and base motions. Despite these limitations, observation of the performance of solid waste landfills in past earthquakes represents the most important source of information for design of modern landfills to resist seismic loading.

Landfill Performance During Recent Earthquakes

In general, landfills have performed well when subject to strong ground shaking in earthquakes. This observation is based primarily on field inspections of MSW landfills conducted after three recent earthquakes in California. These earthquakes include the 1987 M 6.1 Whittier Narrows event in the greater Los Angeles area, the 1989 M 7.1 Loma Prieta event in the San Francisco Bay area, and the 1994 M 6.7 Northridge earthquake in the greater Los Angeles area.

The Loma Prieta and Northridge events were significant earthquakes felt over wide areas. Both of these events subjected numerous landfill facilities to strong shaking. Following both events, specific efforts were made to document the effects of seismic loading on landfills. Documentation of landfill performance in the Whittier Narrows event was relatively limited, in part because regulatory guidelines were not particularly specific about the need for seismic analysis of landfills and in part because the moderate magnitude of the event limited the intensity and duration of strong ground motions and the extent of the impacted area. However, this earthquake provides important information on the performance of landfills in earthquakes due to its proximity to the Operating Industries, Inc. (OII) landfill superfund site. Furthermore, this event provided the impetus for instrumentation of the OII landfill site, to date the only landfill for which strong motion records are available. The strong motion records obtained at the OII landfill in the Northridge earthquake and a number of other recent events of smaller magnitude or further epicentral distance from the site provide important observational data on the seismic performance of landfills.

Whittier Narrows Event

The epicenter of the M 6.1 Whittier Narrows earthquake of 1 October 1987 was located at the eastern edge of the Los Angeles basin (Figure 2), near the border of the basin with the San Gabriel Valley to the east. The main shock occurred as a reverse (thrust) motion on a buried fault at an approximate depth of 10 to 14 kilometers with no surface trace of fault displacement. Performance information from six landfills located in the area is available. Two of the landfills, the OII landfill and the Puente Hills landfill, were within the zone of strong ground motion for the event. Three other landfills, Savage Canyon, BKK, and Azusa, were subject to ground motions of moderate to strong intensity. A sixth landfill, Mission Canyon, was at a considerable distance from the epicenter but is noteworthy because data indicating the absence of earthquake-induced displacement is available. None of the landfill facilities subjected to strong shaking in the Whittier Narrows event was equipped with geosynthetic or compacted clay liner systems at the time of the event.

OII Landfill

Siegel, et al. (1990) report on observations at the OII landfill made immediately following the Whittier Narrows event. Slopes at the OII landfill range from 3H:1V (horizontal to vertical) to as steep as 1.3H:1V for heights up to 75 m. The epicenter for the earthquake is reported to be approximately 4 km from the OII landfill. The closest point to the landfill at which ground motions were recorded was the Garvey reservoir, also located approximately 4 km from the OII landfill. Recordings at Garvey reservoir suggest that the PGA at the base of the OII landfill could have been as high as 0.45 g. However, a contour plot of PGA during the Whittier Narrows event prepared by Trifunac (1988) indicates that the free-field PGA in the vicinity of the OII landfill may have been closer to 0.30 g.

Visual observations at the OII landfill immediately following the Whittier Narrows Earthquake identified significant ground cracking in cover soils, but no clear evidence of waste slope instability (Siegel, et al., 1990). The OII landfill was subsequently instrumented with accelerometer stations at the base and on the top deck of the landfill. Based on three small earthquakes recorded at the landfill in 1988 and 1989, Siegel, et al. (1990) back analyzed performance of the OII landfill slopes during the Whittier Narrows earthquake to estimate the strength of refuse. Figure 3 presents the results of pseudo-static stability analyses performed by these investigators for the

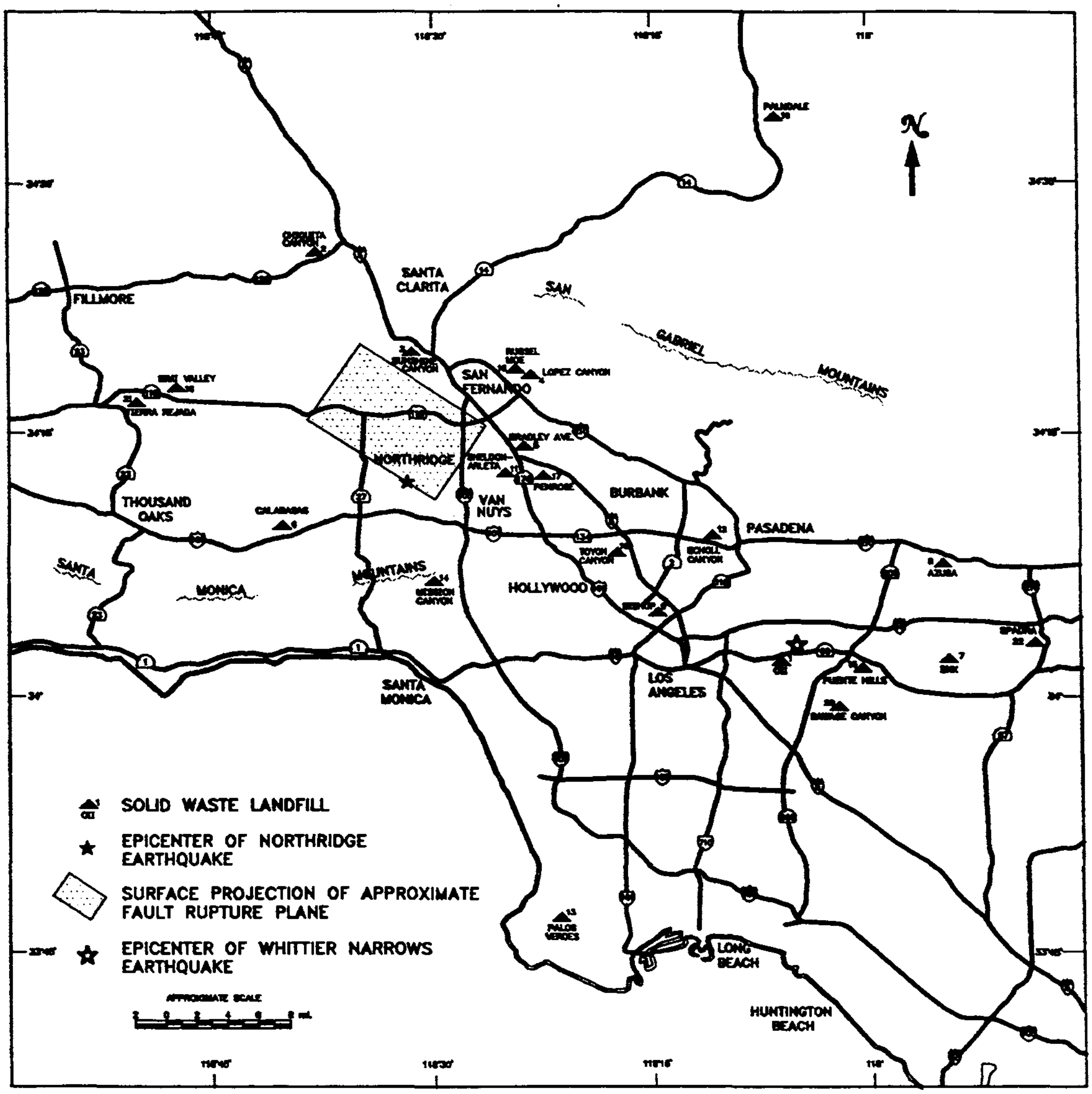

Fig. 2: Landfills Impacted by the Whittier Narrows and Northridge Earthquakes (after Matasović et al., 1995)

"critical sections" of the landfill. Figure 3 shows combinations of friction angle, cohesion, and yield acceleration resulting in a pseudo-static factor of safety of 1.0. Based upon a "conservatively assumed" peak average acceleration of 0.10 g for the waste mass in the Whittier Narrows event, Siegel, et al. (1990) concluded that the combinations of Mohr-Coulomb shear strength parameters presented in Table 1 represent a family of values that can be used to conservatively describe the dynamic shear strength of solid waste mobilized at the OII landfill in the Whittier Narrows event.

Puente Hills Landfill

The Puente Hills landfill is located approximately 8 km from the epicenter of the Whittier Narrows event. The free-field PGA at the Puente Hills landfill during the Whittier Narrows earthquake is estimated to have been on the order of 0.25 g (Trifunac, 1988). No damage was reported at the landfill from the earthquake (Earth Technology, 1988).

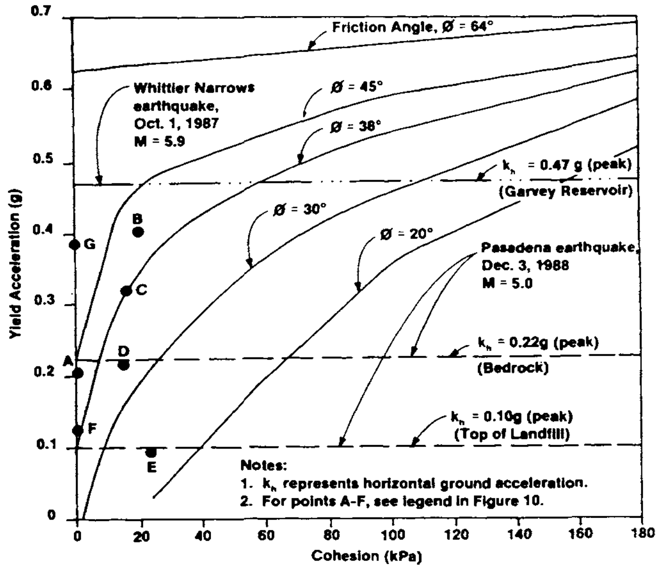

Fig. 3: Results of Pseudo-Static Stability Analyses for OII (Siegel, et al., 1990)

Table 1. Shear Strength Parameters for a Pseudo-Static Factor of Safety of 1.0 at OII (Siegel, et al., 1990)

FRICTION ANGLE degrees	COHESION INTERCEPT kPa
38	0
30	10
20	40

Other Landfills

Free-field PGA's at Savage Canyon, BKK, and Azusa are estimated to have been between 0.15 g and 0.25 g (Trifunac, 1988). There were no reports of damage at any of the facilities by the facility operators. However, formal post-earthquake damage surveys by independent agencies are not available. Coduto and Huitric (1990) report that no noticeable deformation was recorded in inclinometers and settlement monuments at the Mission Canyon landfill approximately 22 km west of the earthquake epicenter following the Whittier Narrows event. However, the free-field PGA at Mission Canyon was only on the order of 0.10 g from the Whittier Narrows event (Trifunac, 1988).

Loma Prieta Earthquake

The Loma Prieta earthquake of 17 October 1989 was a M 7.1 strike slip event located in the Santa Cruz Mountains at the southern end of the San Francisco Bay area. Orr and Finch (1990) report on inspections of ten landfills after the 1989 Loma Prieta event. These inspections were performed by the California Integrated Waste Management Board (CIWMB). The PGA at the base of these landfills during the Loma Prieta event were estimated to have ranged from 0.1 g to 0.45 g. The four sites with the lowest estimated PGA's (PGA from 0.10 g to 0.15 g) were Bay Mud sites, where the potential for amplification of damaging long period motions from the earthquake was the greatest. None of the ten landfills inspected by CIWMB was reported to have engineered liner systems. Only minor damage was reported at any at the ten landfills. The most common types of observed damage included minor cracking of the landfill slopes. However, the authors note that it was often difficult to distinguish between "normal" cracks induced by waste settlement and decomposition and earthquake- induced cracking. The authors further note that many landfill gas recovery systems were temporarily affected by power loss and above-ground pipe breakage. However, all landfill gas recovery systems were repaired and back in operation within 24 hours of the earthquake and no post-earthquake changes in quantities of leachate and landfill gas recovery were reported. The authors report that a total of 13 solid waste landfills, including the ten inspected by CIWMB, experienced minor damage in the Loma Prieta event.

The performance of landfills during the Loma Prieta earthquake was also investigated by Johnson, et al. (1991). These investigators report on the behavior of the ten landfills, including seven of those reported on by Orr and Finch (1990). PGA's at the ten landfills investigated by Johnson, et al. varied from 0.04 g to 0.50 g. The authors report that, in general, slopes of landfills performed very well. This included 2H:1V (horizontal:vertical) slopes up to 45 m high at the Santa Cruz landfill, where the estimated free-field PGA was 0.45 g, 3H:1V slopes up to 45 m high at the Ben Lomond landfill, where the estimated free-field PGA was 0.50 g, and 2H:1V slopes up to 75 m high at the Kirby Road landfill where the estimated free-field PGA was 0.50 g. The authors note that cracking of slopes at these landfills was generally limited to contact zones between areas of dissimilar materials and areas of changes in geometry. These are the same areas where cracks tend to form under normal operating conditions.

Buranek and Prasad (1991) report on the performance of six landfills in the Loma Prieta earthquake, including two landfills reported on by Johnson, et al. (1991) and Orr and Finch (1990). PGA's at the base of the six landfills reported on by Buranek and Prasad were estimated to range from 0.15 g to 0.45 g. Minor cracking was observed at four of these sites. Transition zones between different materials (e.g., waste fill and natural ground) and between areas of different waste face geometry were cited for most crack locations. Typical crack displacements were on the order of 25 to 75 mm. At one site, minor downslope cover soil movement was observed. At another site, apparent horizontal displacement was observed in rigid landfill gas control piping. Limit equilibrium analyses and the Makdisi and Seed (1978) seismic deformation charts were employed at the six landfills to back-calculate anticipated displacements. The shear strength of the solid waste was represented by a cohesion of 20 kPa and a friction angle of 20 degrees. The estimated PGA at the base of the landfill appears to have been used as the maximum acceleration of the potential failure mass in the deformation analyses. Results of the analyses yielded deformation magnitudes consistent with the magnitude of observed crack deformations at the six sites surveyed.

Sharma and Goyal (1991) report on analysis of the performance of the West Contra Costa Sanitary Landfill (WCCSL) in Richmond, California during the Loma Prieta earthquake. The WCCSL is located on a 12 to 18 m thick deposit of relatively soft recent San Francisco bay mud overlying older, stiffer bay mud. The WCCSL is located over 100 km from the earthquake epicenter. Due to the large distance from the epicenter, the estimated free-field bedrock PGA at the WCCSL site was only 0.06 g. However, one-dimensional site response analyses conducted using SHAKE (Schnabel, et al., 1972) indicated an amplification factor of three for the free-field motion at the top of the bay mud, resulting in a free-field PGA of 0.18 g at the top of the bay mud.

SHAKE response analyses were also conducted by Sharma and Goyal to evaluate the influence of the waste fill on earthquake motion. The solid waste was assigned a shear wave velocity of approximated 170 m/sec based upon down hole measurements at the site. Modulus reduction and damping curves for MSW were based upon recommendations from Singh (1989). Results of the analyses indicated slight amplification of peak accelerations for waste thicknesses of less than 15 m, with a maximum PGA of 0.21 g reported for a 6 m thickness of waste. Inclinometer measurements indicated that ground shaking during the Loma Prieta earthquake did not result in any significant deformation at the WCCSL.

Northridge Earthquake

The Northridge earthquake of 17 January 1994 was a M 6.7 event. The main shock occurred as a reverse (thrust) motion on a southward-dipping plane at a depth of approximately 15 km at the northern end of the San Fernando Valley in the Los Angeles metropolitan area. Numerous active, inactive, and closed solid waste landfills are located within 100 kilometers of the earthquake epicenter. Stewart, et al. (1994) provide preliminary data on the performance of several major landfills in the epicentral region. Matasović, et al. (1995) summarize information on the performance of 22 landfills that experienced shaking estimated to be in excess of 0.06 g. The locations of these landfills are shown in Figure 2. At 16 of these sites the free-field PGA at the base of the landfill was estimated to be in excess of 0.24 g. At six sites the free-field PGA at the base of the fill was estimated to be in excess of 0.38 g.

Figure 4 shows the generic configurations of the 22 landfills encompassed by the Matasović, et al. study. Slope heights at these landfills were up to 90 m at inclinations as steep as 1.3H:1V. Damage at the 22 landfills was classified according to the five damage categories presented in Table 2, varying from "Little or No Damage" (Damage Category I) to "Major Damage" (Damage Category V). Of the 22 landfills, none suffered Major Damage, only one suffered Significant Damage, four suffered Moderate Damage, and the remaining 17 suffered Minor Damage or No Damage, according to the authors.

Three of the landfills subject to the strongest shaking in the Northridge event had geosynthetic composite liner systems that met Subtitle D requirements. The Chiquita Canyon landfill, subject to an estimated free-field PGA of 0.39 g, suffered perhaps the most notable damage. Damage at this landfill, classified as Significant Damage, consisted of two tears in the geomembrane liner, one approximately 3 m in length and the other approximately 23 m in length. Both tears occurred parallel to an anchor trench on a bench above the waste. The second tear was not discovered until several weeks after the earthquake as the geomembrane on the bench was covered with a protective layer of soil. No disruption of the underlying low permeability soil liner was reported in either case. Cracking of cover soils was also observed at the Chiquita Canyon landfill following the earthquake.

At both the Bradley landfill, subject to an estimated free-field PGA of 0.45 g, and the Lopez Canyon landfill, subject to an estimated free-field PGA of 0.44 g, a local tear in the geotextile overlying the side slope liner was observed by the CIWMB in post-earthquake inspections (CIWMB, 1994). However, at both landfills subsequent investigations indicated that the tear was caused by operating equipment (GeoSyntec Consultants, 1994; personal communication, 1995). The damage at the Bradley landfill was classified as Moderate Damage and included cover soil cracking. Damage at the Lopez Canyon landfill was also classified as Moderate Damage and included cracking in the cover soils and damage to the gas recovery system.

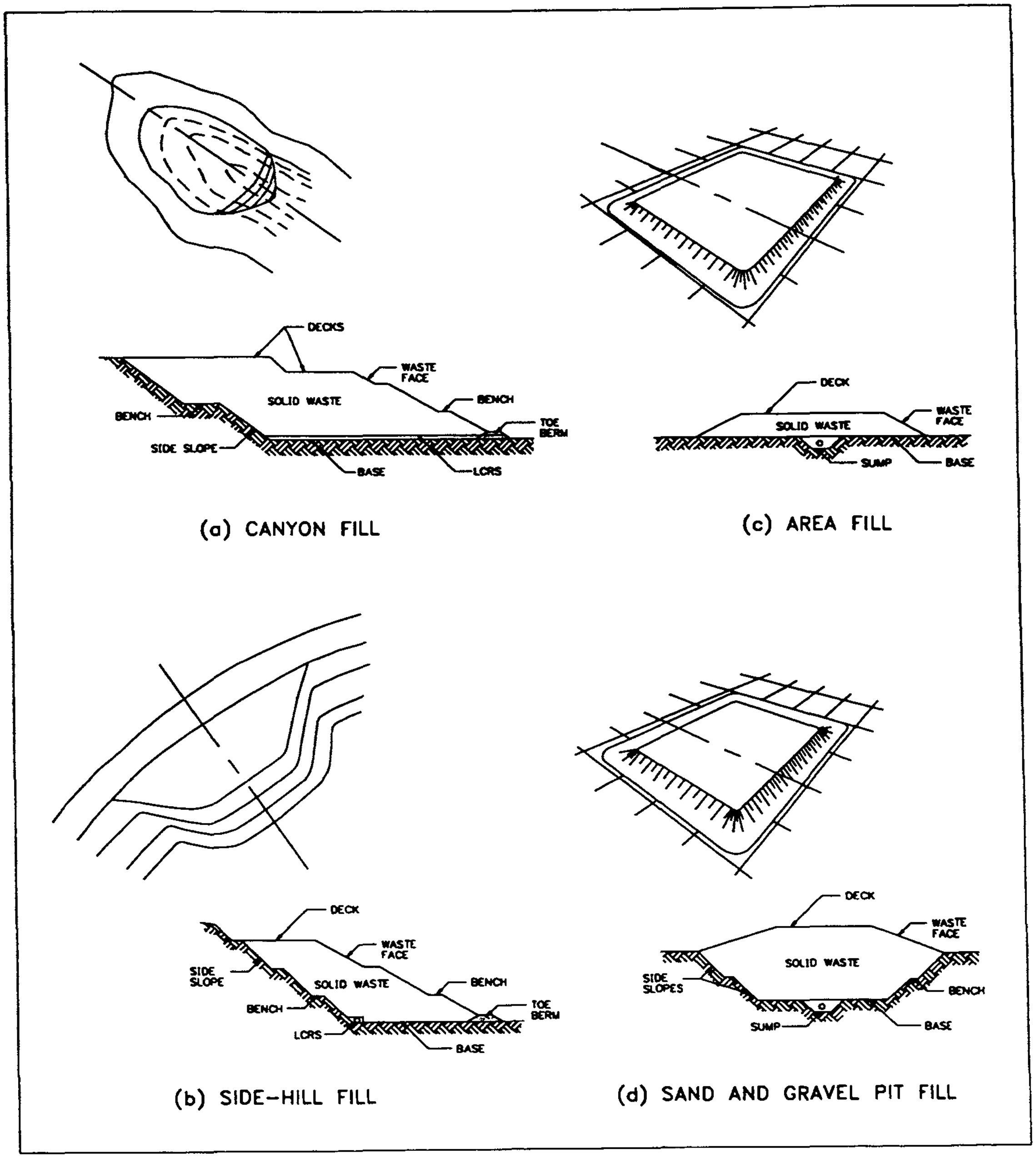

Fig 4: Generic Landfill Types (Matasović, et al., 1995)

Matasović, et al. report that the most prevalent damage to landfills in the Northridge event was superficial brittle cracking in cover soil at transitions between waste fill and natural ground areas. Cracks were typically 10 to 70 mm wide and of similar vertical relief. Perhaps the most pronounced cracking of this type was at the Sunshine Canyon Landfill, the closest landfill to the zone of energy release from the earthquake. At Sunshine Canyon, the observed cracks were over 300 mm in height and width along the contact between the refuse fill and the canyon wall at the back of the landfill. The authors suspect that some of this cracking may have been due to earthquake-induced settlement of the refuse, as well as to the differential dynamic response of the waste fill and the natural ground.

Table 2. Damage Categories for Solid Waste Landfills (Matasović, et al., 1995)

DAMAGE CATEGORY	DESCRIPTION
V. Major Damage	General instability with significant deformations. Integrity of the waste containment system jeopardized.
IV. Significant Damage	Waste containment system impaired, but no release of contaminants. Damage cannot be repaired within 48 hours. Specialty contractor needed to repair the damage.
III. Moderate Damage	Damage repaired by landfill staff within 48 hours. No compromise of the waste containment system integrity.
II. Minor Damage	Damage repaired without interruption to regular landfill operations.
I. Little or No Damage	No damage or slight damage but no immediate repair needed.

As in previous earthquakes, disruption to landfill gas recovery systems was common during the Northridge earthquake. Loss of power was perhaps the most common source of disruption, followed by breakage of gas and condensate lines and well heads. In all cases, gas recovery systems were back in operation within 24 hours. Shut down of the gas recovery system at a MSW landfill for up to 48 hours following a major earthquake is not considered to present a significant environmental hazard.

OII Landfill Ground Motion Data

Perhaps the most significant data captured in the Northridge earthquake are the strong motion records obtained at the OII landfill (Hushmand Associates, 1994). Strong motions stations located at the base and top deck of the eastern end of the landfill (Figure 5) recorded for the first time the response of a solid waste landfill to ground motion in excess of 0.10 g. While the OII landfill is not a typical MSW landfill in that it received industrial waste and liquids during its operating life, the eastern end of the landfill is believed to be primarily MSW. Supplemented by strong motion data obtained at the OII landfill site from earlier, smaller and/or more distant events, the data from the Northridge event provide for the first time a means for calibrating back analysis of the dynamic properties of MSW. Stewart, et al. (1994) and Kavazanjian and Matasović (1995) have already reported on dynamic properties for MSW back calculated from the Northridge earthquake strong motion records obtained at the OII landfill. The results of these back analyses are discussed in a later section of this paper.

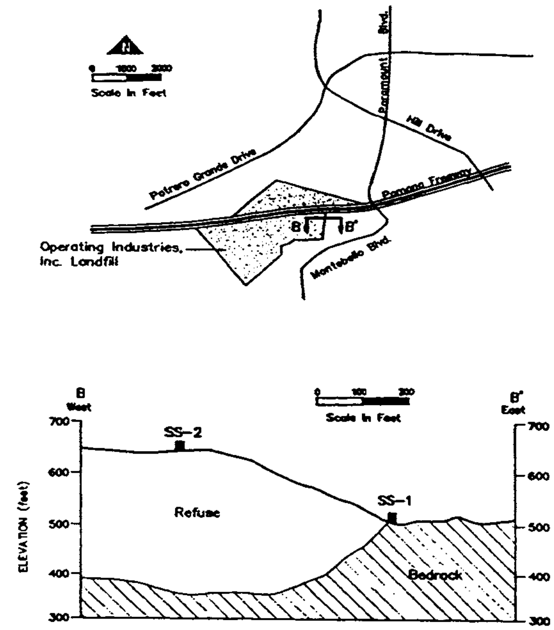

Fig. 5. OII Landfill Instrumentation (Anderson, et al., 1992)

One notable aspect of the strong motion records obtained at the OII landfill is that they dispel the notion that solid waste landfills unconditionally attenuate earthquake ground motions, contrary to the suggestions of some previous investigators. Initial data from the OII strong motion instruments obtained in three small earthquakes in 1988 and 1989, reported by Anderson, et al. (1992) and presented in Figure 6, show spectral acceleration amplification factors as great as 12 between the base and top deck for a spectral period in the vicinity of 1 second. Figure 7 shows strong motion records obtained at the base and top deck of the OII landfill in the M 7.4 Landers earthquake of 1992 and the M 6.7 Northridge event (Hushmand Associates, 1994). In the Landers earthquake, where the predominant period of the base motion was in the 0.5 to 1 second period range, the peak acceleration on the top deck was amplified by a factor of three from that of the base motion. In the Northridge event, where the predominant period of the base motion was in the 0.25 to 0.5 second range, the peak acceleration on the top deck equaled the peak acceleration at the base and a spectral acceleration amplification factor of over six was observed at a spectral period of about one second.

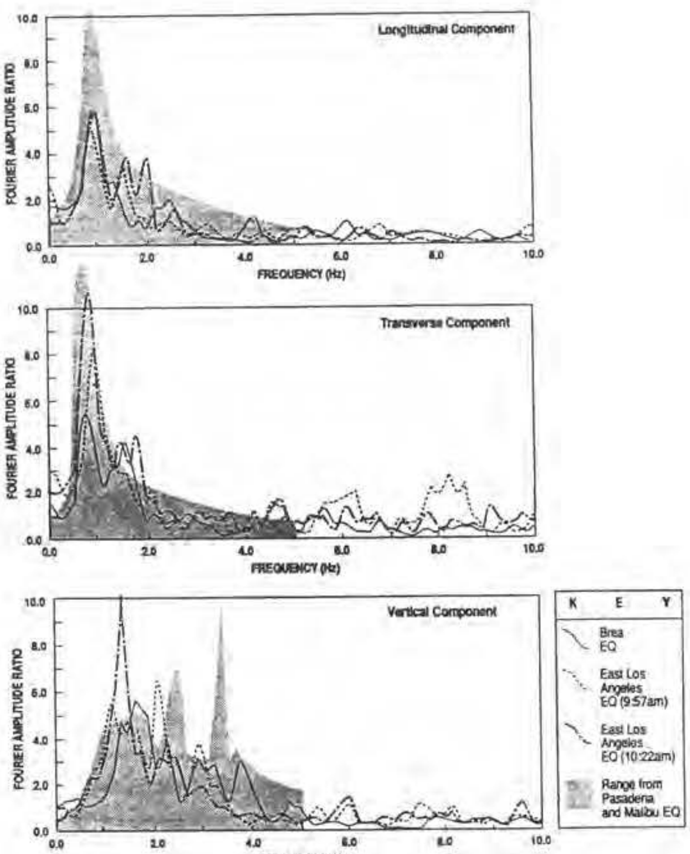

Fig. 6. Spectral Response of the OII Landfill
(Anderson, et al., 1992)

STATE-OF-THE-PRACTICE FOR SEISMIC DESIGN OF SOLID WASTE LANDFILLS

Seismic design of solid waste landfills can be broken down into four essential steps:

- characterization of the earthquake ground motions to be used for design;
- evaluation of the response of the landfill to the design earthquake motions;
- calculation of the stability and deformation of the waste mass as it responds to the design earthquake motion; and
- determination of the ability of the structural elements of the waste containment system to maintain their integrity when subject to the calculated deformations.

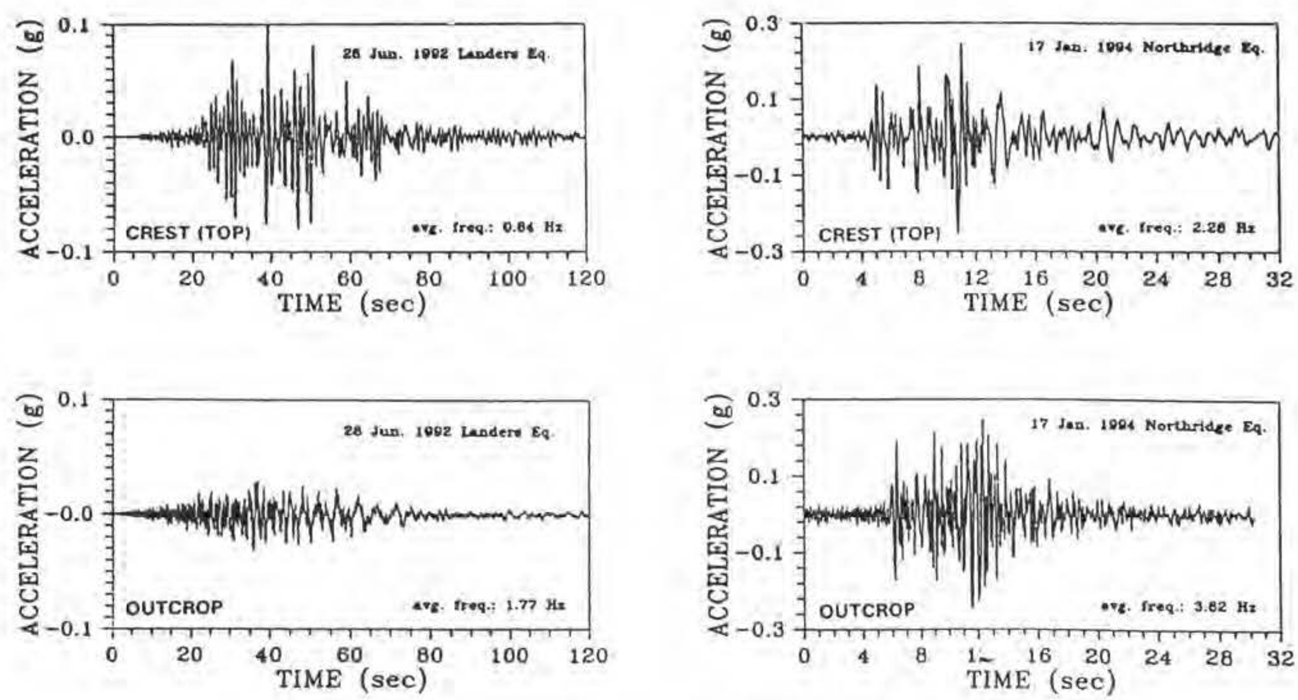

Fig. 7. Acceleration Time Histories from the OII Landfill
(Kavazanjian et al., 1995)

The first step in the process, characterization of the design ground motion, is frequently driven by regulations. But even within the framework of Subtitle D and state regulations, judgment and interpretation are frequently required to complete this task. The second and third steps of the process, evaluating the seismic response of the landfill and the resulting seismic stability and deformation of the waste mass, while inter-related, are typically performed in an independent, de-coupled fashion. Seismic response is generally evaluated in the form of an acceleration time history for the waste mass and cover. Seismic deformation is then calculated in a "Newmark" sliding block on a plane deformation analysis using the yield acceleration from a pseudo-static limit equilibrium analysis and the computed acceleration response. Determining the dynamic properties of the solid waste and the liner and cover system elements is the primary challenge facing the engineer performing these analyses.

The fourth step, determining the ability of the structural elements of the waste containment system to resist seismic deformation, may be the most difficult task for the practicing engineer. Little guidance exists on the ability of the liner and cover system elements to resist displacement along the interface and differential movements across the interface.

In the following sections, the state-of-the-practice for conducting these tasks is summarized.

Estimating Ground Motions

The point of reference for criteria for ground motions for seismic design of landfills is the USEPA Subtitle D regulations. Subtitle D provides two alternatives for evaluating earthquake ground motions for MSW landfill design. The PGA in lithified earth evaluated from a map depicting the peak acceleration with a 90 percent probability of not being exceeded in 250 years is the prescriptive means of evaluating the Subtitle D design acceleration. Subtitle D guidance documents (USEPA, 1992; USEPA, 1994) identify the latest version of USGS Map Sheet MF-2150, the "Algermissen" map, as the default basis for evaluation of this acceleration. Presumably, similar maps developed on a regional or local basis can be used. Alternatively, site-specific studies can be used to determine the peak ground motions in lithified earth at the landfill site for design. Little guidance is given by USEPA for the site-specific analysis alternative. Instead, authority for deciding what constitutes an acceptable site-specific analysis is delegated to the director of an approved State or Tribal regulatory agency. The site-specific analysis may be either probabilistic or deterministic in nature. Some regulatory authorities allow site-specific probabilistic analyses for the peak acceleration with a 90 percent probability of not being exceeded in 250 years, while other states require deterministic analyses based upon entirely different criteria.

USGS Acceleration Maps

The most recent set of USGS acceleration maps was published by Algermissen, et al. (1990). There are two sets of maps associated with this publication. The first set consists of maps showing contours of the PGA with a 10 percent probability of exceedance (90 percent probability of non-exceedance) in 50-year and 250-year exposure periods. These two exposure periods define mean return periods of approximately 475 years and 2,375 years, respectively, at the 10 percent probability level. In other words, on the average, one event with an acceleration equal to that shown on the map is expected to occur every 475 or 2,375 years, depending on which map is consulted. For reference, the current standard of practice in seismic design in many areas of the United States for determining the survivability of buildings, bridges, and other important facilities where life safety is an issue is the 475-year return period.

The second set of ground motion maps developed by Algermissen, et al. present velocity-derived acceleration values, (peak velocity is commonly used in structural engineering to assess the damage potential of earthquake ground motions to long period structures). Inspection of this second set of maps reveals that the velocity-based acceleration values exceed the directly-derived acceleration values at some locations. Typically, these locations are more distant from the earthquake source. The differences reflect the fact that higher frequency components of earthquake ground motions attenuate more quickly than lower frequency components, resulting in lower frequency motions at a distant site. Velocity-derived acceleration values provide a more accurate indication of this phenomenon, and, as a result, may provide a more representative PGA for design when evaluating sites located at larger distances from seismic sources.

There are a number of limitations and problems that the design engineer must be aware of when using the Algermissen maps. Perhaps the biggest problem associated with the use of Algermissen maps is that there is no design magnitude associated with the map acceleration. Geotechnical analyses frequently require a magnitude as well as a PGA (e.g., liquefaction potential and seismic deformation assessment). The acceleration on the Algermissen map is typically composed of contributions from earthquake of many different magnitudes. Even if this distribution of magnitudes is known, it is not clear how to select the magnitude for use in a determinative design analysis.

The approach suggested in the USEPA guidance document for determining the magnitude associated with the Algermissen map acceleration is to use the maximum magnitude from the "host" seismic zone (the zone the site in question is in) and from all zones contiguous to the host zone. While this typically results in a conservative assessment of the magnitude associated with the Algermissen map, there can occasionally be cases where the source zone containing the governing event is more than one zone removed from the host zone. Therefore, considerable judgment is required in evaluating the magnitude associated with the Algermissen map.

Selection of the design magnitude is further complicated by the fact that the event generating the maximum PGA may not be the most damaging earthquake anticipated for the specified exposure period. A larger magnitude but more distant earthquake may produce ground motions at the site of lower intensity but of longer period, longer duration, and greater damage potential than the event associated with the Algermissen map acceleration.

There are a number of other factors which must be considered when using the Algermissen map acceleration for design. The significant assumptions that have to be made when developing these and other similar probabilistic seismic risk maps are well-documented by the USGS and others. For example, assumptions must be made regarding the geologic structures causing the earthquakes, the maximum size of the earthquake for any geologic structure, the recurrence rate for earthquakes within the structure, and the attenuation of earthquake motions as they propagate from the source to the site of interest. These assumptions are based on the best data available at the time the evaluation is made. However, as researchers collect and analyze data from recent earthquakes and geological and seismological studies, some of the assumptions made in developing the Algermissen acceleration maps will no longer be valid. In areas where new information has been developed regarding the cause and frequency of earthquakes, the Algermissen maps may either underestimate or overestimate motions of the ground. This means that these maps must be used with caution.

Several of the factors associated with the development of the Algermissen seismic risk maps are worth noting:

- Unrecognized Faults: The discovery of previously unrecognized faults, such as the blind thrust fault associated with the 1994 Northridge earthquake, could result in higher ground motions than predicted by the USGS maps. Similarly, it is now generally believed that in many parts of the western United States, random shallow crustal earthquakes could occur virtually anywhere. Such earthquakes may have a magnitude of at least 5 and typically be located 15 to 20 km below the ground surface. The consequence of these random earthquakes will likely result in increases in acceleration in areas previously thought to have low seismicity, such as eastern Washington.

- Inadequate Recurrence Relationships: Recurrence relationships indicate the frequency of occurrence for earthquakes having different sizes. In some parts of the United States, such as California, these relationships are relatively well-established. However, in other areas the frequency of earthquake occurrence is so low that it is necessary to make significant assumptions regarding the relationship between size and frequency. Even in California where earthquake recurrence relationships are thought to be well-behaved, seismic activity varies, from relative quiet periods to periods of high activity. While current thinking among geologists and seismologists tends to favor characteristic event earthquake models that consider the occurrence of earthquakes within a narrow-magnitude band at relatively regular recurrence intervals, most probabilistic models still employ the Gutenberg-Richter log normal distribution for earthquake magnitude and assume random arrivals of these events. Some researchers (Krinitsky, 1993) argue that conducting probabilistic hazards analyses using these relationships has limited, if any, value, particularly when considering the design of important facilities.

- Modified Attenuation Relationships: Additional data are being collected continually on the attenuation of ground motions from seismic sources. This is particularly the case in the Pacific Northwest and Alaska, where the dominating source mechanism is often a subduction zone earthquake. Crouse (1990) has shown that ground motion attenuation can differ appreciably for subduction zone earthquakes compared to crustal earthquakes. The USGS maps do not necessarily account for these differences. Similarly, studies have shown that thrust faults will have a different attenuation characteristics than strike-slip faults (Campbell, 1990). Even the definition of bedrock in the attenuation relationships used is often open to question. For example, bedrock in the eastern United States is significantly different in terms of stiffness than bedrock in the western United States (EPRI, 1994). This difference in stiffness affects the attenuation of ground motions.

In light of these continuing changes in the profession's understanding of earthquake loading mechanisms, it should be clear that use of the published USGS maps must be done with considerable care. While these maps often provide the statutory basis for identifying the intensity of ground shaking at a site, key issues related to seismic response may have changed since the map was developed or may not have been included in developing the maps. Therefore, for those sites where the maps indicate potentially critical conditions related to design, it is strongly recommended that individuals with expertise in seismic hazard analyses and who are familiar with the current state-of-the-practice and have a sound understanding of local geologic conditions should be consulted regarding the applicability of the maps to the particular area in question.

Site-Specific Acceleration Determination

An alternative method of determining the bedrock acceleration for landfill designs involves conducting a site-specific seismic hazard analyses (SHA). Either probabilistic or deterministic methods may be used to conduct these site-specific analyses. Procedures used to conduct probabilistic SHA normally follow those used by the USGS in developing the seismic hazards maps for the United States. The deterministic approach is based on the location of contributing seismic sources, the magnitude of the maximum event for each source, and an appropriately selected attenuation relationship. The maximum magnitude may depend on the exposure period or may be selected without regard for the likelihood of occurrence. A primary difference between a site-specific SHA and the use of the Algermissen maps is that, in a site-specific analysis, more detailed consideration can be given to the local geologic structure and recent developments in earthquake recurrence intervals and attenuation relationships. As this allows a more precise, less uncertain assessment of the seismic hazard at a site, site-specific analyses sometimes use a shorter exposure period than the 250 years specified for the PGA from the Algermissen map. In California, a 100-year exposure period is used for site-specific analysis of MSW landfills.

Probabilistic Seismic Hazards Assessment: A site-specific probabilistic SHA requires use of individuals or consultants with specific expertise in the regional seismogenic structure, the appropriate attenuation relationships, and the probabilistic method. In some cases, results of the probabilistic SHA may be very similar to results given by the USGS maps. For example, the authors performed seismic hazards studies for the Port of Los Angeles (CH2M HILL, 1993) and found little difference in predicted and published ground accelerations. On the other hand, a recent probabilistic SHA conducted for the Anchorage Regional Landfill (Earth Mechanics, 1994) resulted in PGA values for design that were nearly 0.2 g lower than those shown on the USGS map. This difference was due in large part to the use of a different attenuation relationship for the subduction zone earthquakes in the area than used by USGS.

One benefit of a site-specific probabilistic SHA can be development of a plot showing the probability of nonexceedance or return period versus ground acceleration for different exposure periods. Figure 8 shows such a plot. These plots can be useful when judging the appropriate level of ground acceleration for interim phases of design. The resistance of the landfill to seismic loading during interim stages of landfill development is often less than the seismic resistance once filling is complete. It may not be cost effective to use the full 250-year exposure period for evaluating seismic resistance of the interim phases of development. Rather, a shorter exposure period may be selected based upon an evaluation of the risks and costs. Selection of a shorter exposure period is a judgment decision that should be made in conjunction with the owner and/or operator of the landfill. If a shorter exposure period is selected, it may well result in a significantly lower design acceleration for interim conditions. As long as the owner is made aware of the risks associated with this lower acceleration level, this can result in significant economies.

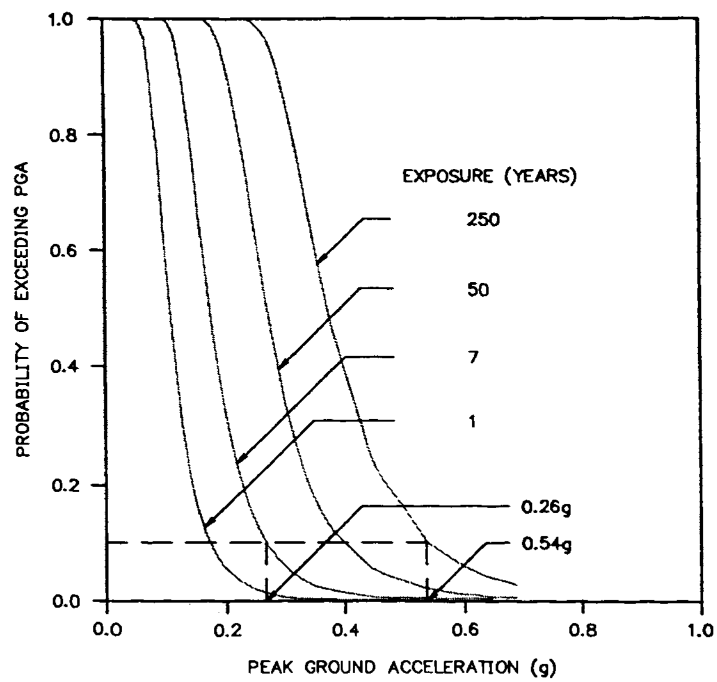

Fig. 8. Probabilistic Hazard Analysis Results

Another potential benefit of a site-specific probabilistic SHA is evaluation of the magnitude distribution associated with the probabilistic acceleration. Figure 9 presents the magnitude distance distribution associated with the design acceleration for a hypothetical site in the Los Angeles basin. This information can be used either to select a single representative magnitude for a deterministic design analyses or to calculate a corresponding distribution of the results of the seismic performance assessment.

While there can be several significant benefits to the performance of site-specific probabilistic SHA, these analyses need to be conducted by knowledgeable individuals. Unfortunately, the simplicity of the method has led to the publication of computer software that allows virtually any person to conduct such analyses. In the hands of a knowledgeable person, such software can provide a reasonable estimate of the seismic hazard at a site. Unfortunately, the analyses can also be conducted by individuals who have little understanding of the assumptions and limitations of the software and use cook book or black box recommendations for input parameters. The latter type of probabilistic SHA can result in questionable acceleration values for use in design.

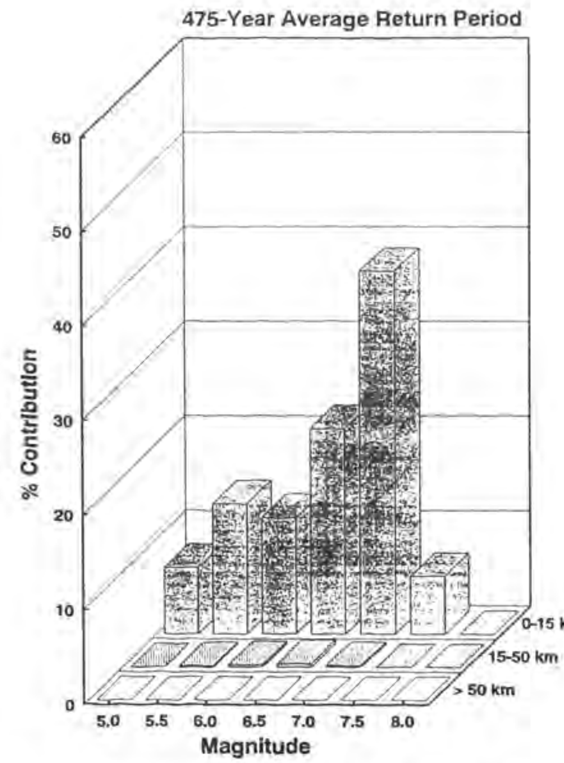

Fig. 9. Magnitude-Distance Distribution for the Design Acceleration (Moriwaki, et al., 1994)

As a final comment on site-specific probabilistic SHA's, it is recommended that sensitivity studies to evaluate the effects of different assumptions on the predicted acceleration level be performed whenever possible. Where small changes in uncertain parameters result in a large range in estimated acceleration, it may be prudent to conduct a deterministic analyses instead of or in addition to a probabilistic analyses.

Deterministic Seismic Hazards Assessment: A deterministic site-specific SHA typically consists of the following steps:

- identify the seismic sources that can generate significant ground motions at the site;
- assign a maximum or characteristic magnitude to each source;
- assign an appropriate acceleration attenuation relationship to each source;
- calculate the PGA at the site resulting from the characteristic magnitude event for each source; and
- select from the family of magnitude/PGA pairs the most damaging event or events for use in the seismic performance evaluation.

The seismic sources and characteristic magnitudes should be identified using the best available information on the regional tectonic environment. Selection of the characteristic magnitude may be for a specified exposure period or may be independent of the exposure period or relative likelihood of occurrence. If more than one type of source is identified, then more than one attenuation relationship may be required for the analysis. Determination of the magnitude/PGA pair with the greatest damage potential may not be obvious. It may be necessary to carry multiple events through the performance analysis to determine the most damaging event.

California regulations for seismic design of landfills provide an excellent example of the deterministic approach to evaluating the design earthquake. California regulations specify two different deterministic design level earthquakes: the Maximum Probable Earthquake (MPE) for design of MSW (Subtitle D) landfills and the Maximum Credible Earthquake (MCE) for design of hazardous waste landfills. The MPE is defined by the California Department of Natural Resources Division of Mines and Geology (CDMG) as the maximum event likely to occur in a 100-year period. CDMG defines the MCE as the maximum event anticipated considering the currently known tectonic environment. Current practice is to use mean value attenuation relationships to calculate the PGA for landfill design for both the MPE and MCE. This is consistent with current California Department of Transportation practice of using mean value attenuation relationships to calculate the PGA from the MCE for highway bridge design. However, it should be noted that for some critical facilities, notably nuclear power plants and earth dams, mean plus one standard deviation attenuation relationship are sometimes used.

Performance of a deterministic site-specific SHA is subject to many of the same limitations as performance of probabilistic SHA. Seismic source characterization should be performed by a qualified professional familiar with current studies on regional geology and tectonics and attenuation relationship development. Use of outdated data and cook book solutions or black box computer programs can result in erroneous evaluation of the PGA for landfill design.

Predicting Landfill Response

Acceleration values selected from the USGS maps or estimated from a site-specific SHA represent free-field ground motions at a rock outcrop but not necessarily in the free-field at the landfill. If the landfill is not founded upon rock, the design engineer must consider the modification of the ground acceleration selected from the USGS maps or from a site-specific SHA to account for soil conditions existing at the site. Once the free-field acceleration at the site has been established, the design engineer must consider the effect of wave propagation through the refuse on the design motion. Both simplified and detailed procedures are available for accomplishing these modifications. The use of both procedures is reviewed below.

Simplified Response Analyses

The simplified approach for evaluating the influence of local soil conditions was first introduced in the early 1980's by Seed and Idriss (1982) and subsequently modified by Idriss (1990) following the 1989 Loma Prieta earthquake. Figures 10a and 10b show the two plots developed by these researchers. These plots show that either amplification or attenuation of the bedrock PGA may occur at a soil site, depending on the level of input motion and the stiffness of soil at the site. Soft ground conditions may be particularly critical, as significant amplification can occur where peak bedrock accelerations are less than 0.4 g. Figures 10a and 10b may be used as a simple method for estimating the modification of bedrock ground motions for local site soil conditions. Based upon Figure 10a, the PGA at stiff and medium stiff soil sites may be assumed equal to the bedrock PGA. Amplification potential of the PGA at soft soil sites for bedrock PGA values less than 0.4 g may be evaluated using Figure 10b. It may not be prudent for design purposes to rely upon the attenuation shown in Figure 10b for bedrock PGA's in excess of 0.4 g.

Stiff, medium stiff, and soft soil sites may be defined on the basis of the average shear wave velocity in the upper 30 m of site using the classification scheme suggested by Borcherdt (1994) and presented in Table 3. In accordance with Borcherdt's recommendations, this simplified approach is not suitable for special study sites with a shear wave velocity of less than 100 m/s.

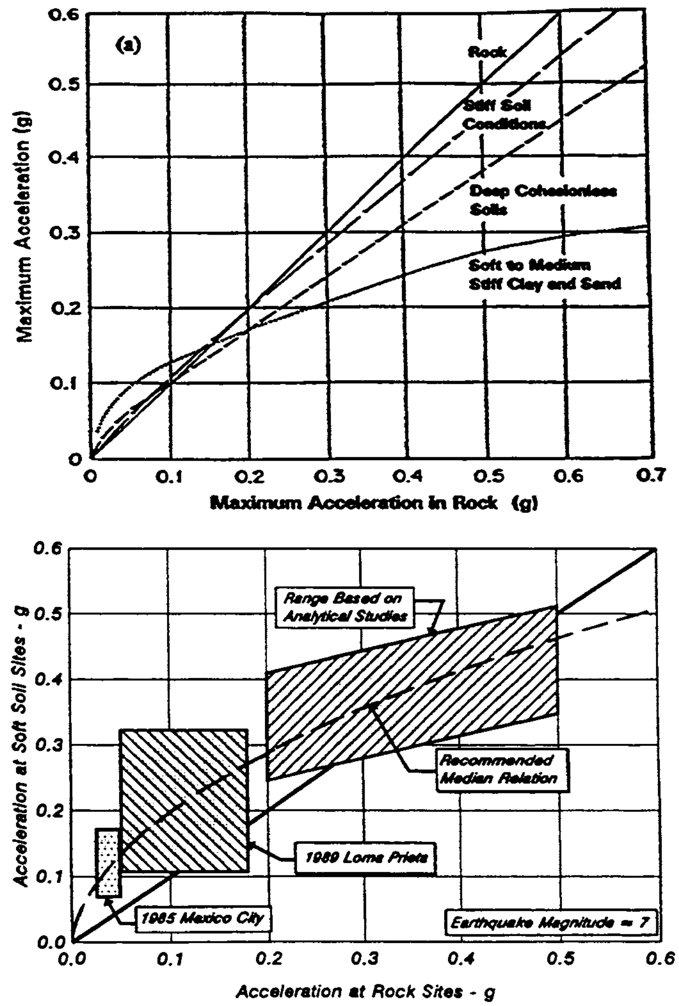

Fig. 10. Influence of local Soil Conditions on Site Response
a) Seed and Idriss (1982)
b) Idriss (1990)

Table 3. Classification of Soil Sites Based Upon Shear Wave Velocity (Borcherdt, 1994)

Classification	Shear Wave Velocity (m/s)
Special Study	Less than 100
Soft	100 to 200*
Medium Stiff	200 to 375*
Stiff	375 to 700*
Rock	Greater than 700*

* Average shear wave velocity over upper 30 m

Noting that the shear velocity of MSW appears to be between that of soft and medium stiff soil, Kavazanjian and Matasović (1995) suggest that the soft soil curve in Figure 10b can also be used to evaluate the peak acceleration at the top of the landfill. Figure 11 shows the soft soil curve from Figure 10b plotted along with the observed response of the OII landfill and the results of non-linear site response analyses of "typical" landfills performed by Kavazanjian and Matasović using soil properties back calculated from the observed response of the OII landfill. Results of these analyses indicate that the Idriss soft soil site amplification curve may also be used as an average or representative curve for the peak acceleration at the top of a MSW landfill. It should be noted that Singh and Sun (1995) suggest that the amplification curve developed by Harder (1991) for earth dams, presented in Figure 12, may serve as an upper bound on amplification of the free-field PGA at the top of landfills.

Figures 11 and 12 apply to amplification of PGA at the top of the landfill. This PGA at the top of the landfill is applicable to seismic performance evaluation of the landfill cover. For landfill liner performance analyses, it is not the PGA at the top of the landfill but the peak average acceleration of the entire waste mass above the liner that is needed for seismic design. Bray, et al. (1995), term the average acceleration of the waste mass the Horizontal Equivalent Acceleration (HEA) and the peak average acceleration the Maximum Horizontal Equivalent Acceleration (MHEA).

Work by Makdisi and Seed (1978) indicates that, for earth dams, the MHEA is typically 40 to 50 percent of the PGA at the dam crest. Based upon similar results from response analyses of landfills and a maximum amplification factor from Figure 11 of 2.5 for base accelerations greater than 0.10 g, Kavazanjian and Matasović (1995) concluded that the free-field PGA at the base of the landfill could be used as the MHEA for all but thin deposits of waste. Results presented by Bray, et al. (1995), shown in Figure 13, allow quantification of the limits of Kavazanjian and Matasović's statement. Assuming an average shear wave velocity of 150 to 200 m/s for up to 30 m of solid waste and a predominant frequency of 0.25 to 0.5 seconds for the earthquake input motion, the mean curve in Figure 13 implies that the MHEA will be less than or equal to the PGA at the base of the landfill for waste thickness in excess of 15 to 25 m. If the characteristics of the waste mass and design earthquake are known, Figure 13 can be used directly to estimate the MHEA from the PGA at the base of the landfill. Note that the predominant period of the waste must be estimated to use this method. The predominant period of the waste can be estimated as a function of the average shear wave velocity of the waste using the equation

$$T_w = 4H/V_s \qquad (1)$$

where H_w is the waste thickness and V_s is the average shear wave velocity of the waste. Guidance on determining the average shear wave velocity of the waste is provided subsequently in this paper.

These simplified approaches to estimating ground accelerations within a landfill involve a number of assumptions that can affect the reliability of the estimate. These include the decoupling of the wave propagation process as the seismic waves pass through different geologic and solid waste materials, the use of average or mean value curves for amplification factors and waste properties, and the estimation of the predominant period for the design earthquake. It is also apparent based on the range of data collected at the OII site and obtained from one-dimensional response analyses conducted by Kavazanjian and Matasović (1994) and Bray, et al. (1995) that significant variation in accelerations can occur from the average curves. These variations likely reflect the unique frequency characteristics of different earthquake records in combination with the stiffness and thickness of the landfill material. As a result of these factors, the simplified approaches should not be used in marginal design cases or for important or critical facilities. Furthermore, it should not be used where very soft special study soils occur (i.e., shear wave velocities less than 100 m/s).

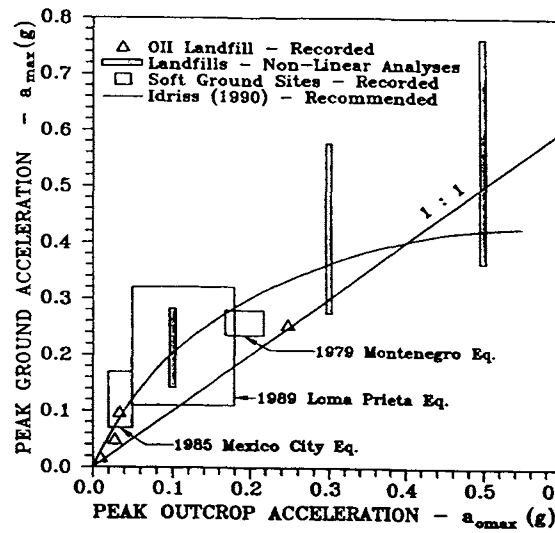

Fig. 11. Amplification of Peak Acceleration by Landfills (Kavazanjian and Matasović, 1995)

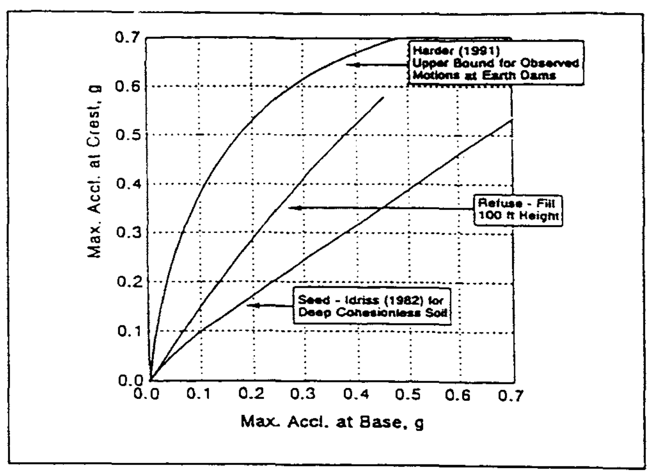

Fig. 12. Amplification of Acceleration by Earth Dams and Waste Fills (Singh and Sun, 1995)

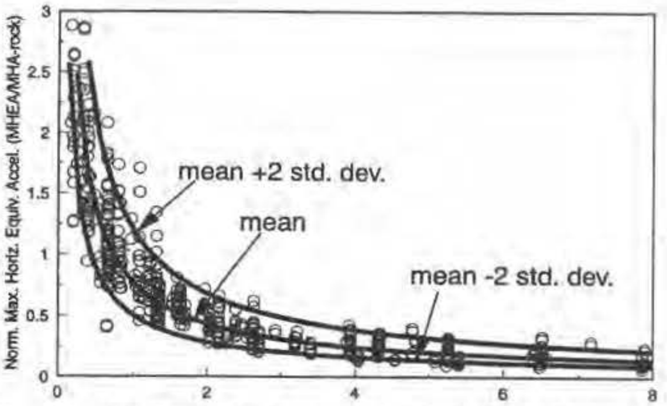

Fig. 13. Maximum Horizontal Equivalent Acceleration from Landfill Response Analyses (Bray, et al., 1995)

While these simplified approaches are easy and may eventually gain wide acceptance in the geotechnical profession, it is clear that these approaches should not be blindly used. Appropriate consideration should be given to the variation in peak ground motion shown in the amplification plots. The prudent engineer may want to perform upper and lower bound analyses to establish the possible range in response. In conducting the slope stability and deformation analyses discussed subsequently in this paper, the effects of these upper and lower bound response estimates on the seismic deformation of the landfill can be quantified. The degree of conservatism to apply in these calculations is likely related to the consequences of inadequate performance. If excessive deformations can be relatively easily detected and repaired, such as in the case of a landfill cover failure, and no human lives will be jeopardized, then the average or lower bound analysis might be suitable.

Numerical Response Analyses

The alternative method of determining the effects of local soil conditions and the landfill on the earthquake ground motions involves conducting a formal seismic site response analyses. A variety of these analyses are available to the geotechnical professional. The available analyses include one-dimensional equivalent linear procedures, such as the computer program SHAKE (Schnabel, et al., 1972; Idriss and Sun, 1992) and one-dimensional non-linear procedures (Matasović, 1993; Lee and Finn, 1978). Two-dimensional equivalent-linear and non-linear models are also available (Hudson, et, al., 1994; Bardet, 1992). One-dimensional equivalent linear analyses are by far the most common analyses used to evaluate seismic site response in practice today. The key factors in performing these one-dimensional equivalent linear analyses are described in the following sections.

Dynamic Material Properties: Determination of the dynamic properties of MSW for use in the analytical modelling remains one of the most challenging tasks for the design engineer. The key material properties for use in modelling landfill site response are the unit weight, shear modulus, and internal (material) damping of the refuse. The variation of these properties with shearing strain amplitude and effective confining pressure is also important. Variation in properties with the age of the waste, method of waste placement, and waste composition only further complicates this task. Determination of these properties and their variation with depth and confining pressure with any degree of confidence can be problematic because of the general inability to obtain and test samples in the laboratory and because of the difficulties in conducting in situ tests.

Unit weight is generally one of the simplest parameters to evaluate for a geotechnical analysis. Yet, little information exists on the unit weight of solid waste within the landfill, particularly with respect to its variation with depth. Based upon as-placed estimates of the initial unit weight of solid waste developed from landfill gate receipts, on average values of the in-place unit weight based upon landfill volumes estimated by operators over the life of the landfill, and upon typical compressibility values cited by Repetto, et al. (1993), Kavazanjian, et al. constructed the "typical" solid waste landfill unit weight profile shown in Figure 14.

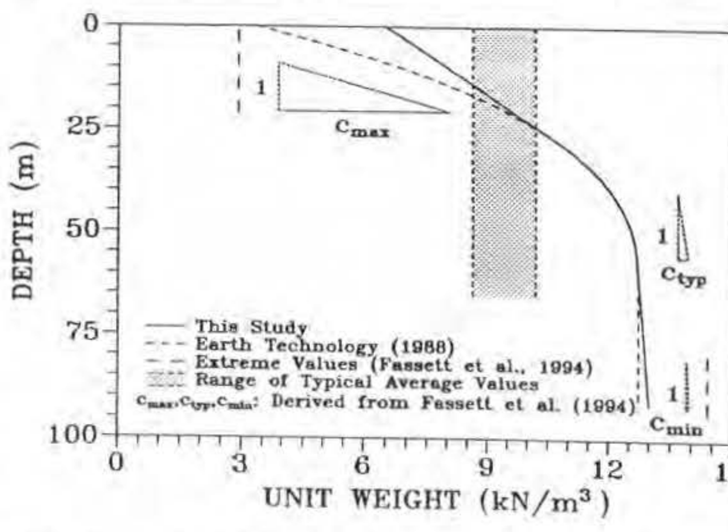

Fig. 14. Unit Weight of MSW (Kavazanjian, et al., 1995)

Probably the most important quantity governing the dynamic response of the waste mass is the impedance of the waste. The variation of impedance with depth and its contrast with the foundation impedance are key factors governing wave propagation through the landfill. Impedance is the product of the unit weight and shear wave velocity divided by the acceleration of gravity. To determine the impedance profile, the shear wave velocity profile is required. The low-strain shear wave velocity (V_s)

profile also provides the basis for determining the initial stiffness characteristics of the solid waste material. In soils, the V_s profile can be evaluated from empirical correlations between V_s and Standard Penetration Test (SPT) blow counts or Cone Penetrometer Test (CPT) friction ratio and end resistance. However, these empirical correlations do not exist for solid waste landfill materials. The only alternative approach is to conduct in situ velocity measurements. While crosshole and downhole methods are routinely used at soil sites to collect these data, the difficulties of drilling boreholes in landfills have limited the number of cases where these tests have been conducted at landfill sites. Shear wave refraction surveys can be used to determine the low-amplitude V_s profiles in an non-intrusive manner. However, this method provides only a gross indication of the shear wave velocity profile and will generally not detect low-velocity layers within the landfill.

Spectral analysis of surface waves (SASW) provides a non-intrusive method for determining V_s profiles in situ (Stokoe and Nazarian, 1985). The benefit of the SASW procedure is that it can provide relatively accurate V_s profiles without the need for drilling and sampling the landfill material. Recently, Kavazanjian, et al. (1994) used a version of SASW to obtain V_s profiles to depths up to 20 m at eight MSW landfills in the greater Los Angeles area. Stokoe (personal communications, 1994) reports that he and his colleagues are conducting SASW tests at a landfill in the greater Los Angeles area to depths of nearly 100 m.

Figure 15 shows available in situ V_s data for MSW compiled by Kavazanjian, et al. (1995). Figure 15 shows a relatively wide range of reported V_s values for MSW. Results of these in situ V_s measurements indicate that V_s in landfill material can range from less than 100 m/s near the surface to more than 500 m/s at depths of 300 m. It is not certain whether the variation V_s values is related to a variation in MSW density or composition. Due to the uncertainties associated with Figure 15, judgment and prudence are required if design values are assessed on the basis of these data.

The other parameters required for equivalent linear dynamic response analysis of MSW are the reduction in shear modulus and variation in internal (material) damping with shearing strain level. In the absence of laboratory measurements, early investigators suggested that solid waste material would behave similar to peat, clay, or a combination of peat and clay (e.g., Earth Technology, 1988; Singh and Murphy, 1990; Sharma and Goyal, 1991). Back analyses of earthquake records obtained at the OII landfill are now providing new insight into shearing strain amplitude effects.

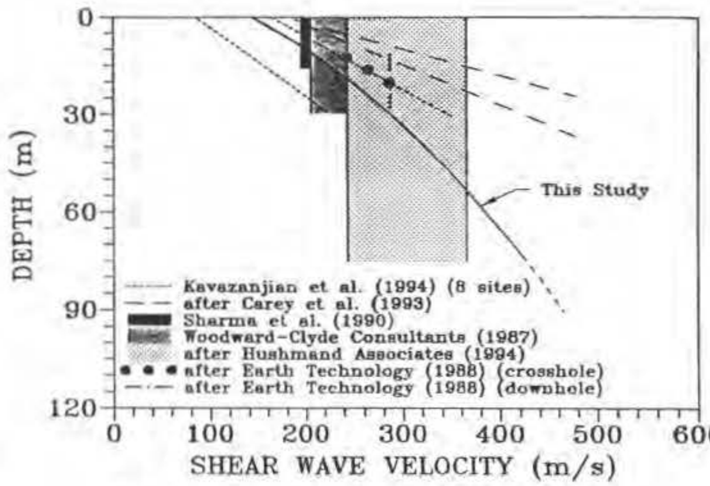

Fig. 15. Shear Wave Velocity for MSW (Kavazanjian, et al., 1995)

Stewart, et al. (1994) report that the MSW modulus reduction and damping curves recommended by Singh and Murphy (1990), representing a response somewhere between the response of peat and the response of clay, give reasonably good agreement between observed and predicted response at the top deck of the OII landfill. Kavazanjian and Matasović (1995) present modulus reduction and damping curves developed from best-fit parameters for a non-linear time-domain site response model (Matasović, 1993). The Kavazanjian and Matasović curves are presented in Figure 16. The modulus reduction and damping curves are compared to the modulus and damping curves for peat and clay in this figure. The modulus degradation of MSW is slower than either peat or clay and the damping characteristics of MSW are similar to clay soils. These dynamic properties were developed using the typical profiles of MSW unit weight and initial shear wave velocity presented in Figures 14 and 15.

Both the Stewart, et al. and Kavazanjian and Matasović back analyses of the OII landfill response assumed unit weight and shear wave velocity profiles based upon "typical" values for MSW. Site-specific density and shear wave velocity profile data for the OII landfill, now being developed under USEPA oversight as part of site remediation activities, should facilitate additional back analyses to further refine these estimates of MSW properties. However, it must be noted that the maximum shearing strain induced in MSW at the OII landfill in the Northridge event was on the order of 2×10^{-2} percent. Until additional data from more intense shaking are obtained, the shape of the modulus reduction and damping curves at strains greater than 2×10^{-2} percent is based solely upon engineering judgment.

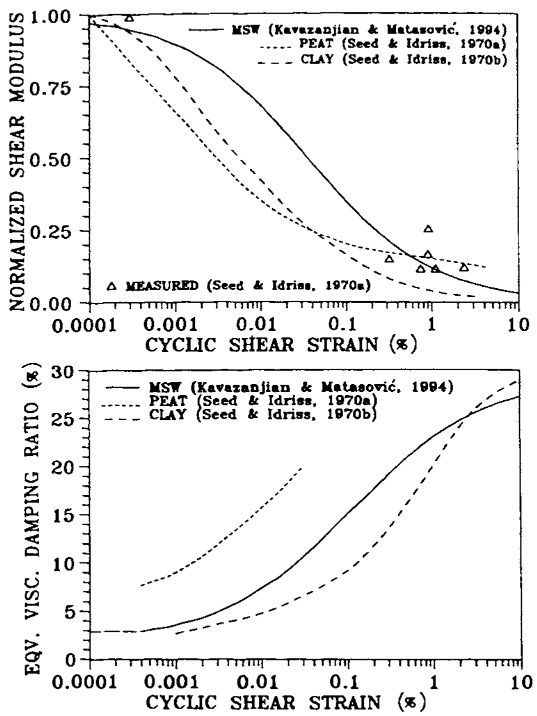

Fig. 16. Modulus and Damping of MSW (Kavazanjian and Matasović, 1995)

As additional data are collected at more landfills, the variation of MSW dynamic landfill properties with confinement, shearing strain amplitude, and waste composition will be better known. Once these data are available, empirical correlations and normalized curves can be calibrated with measured behavior, thereby enabling the uncertainty in the ground response modelling effort to be better quantified. Until then, for important projects and critical analyses involving existing landfills, it seems prudent that in situ shear wave velocity measurements be made at one or more locations within the landfill footprint to quantify the low-strain response of the landfill material. For design of new landfills, this may not be possible. Therefore, the prudent engineer should consider the possible range in both V_s values and in the normalized modulus and damping ratio curves to develop an estimate of landfill response.

GeoSynthetic Interfaces: In a modern geosynthetic-lined landfill, the dynamic properties of the geosynthetic material interfaces may play an important role in the dynamic response of the landfill and the performance of the waste containment system. The geosynthetic interfaces are usually ignored in a dynamic response analysis. This is tantamount to assuming that there is perfect adhesion between the materials on both sides of the interface. However, it is very possible that relative displacement can occur at weak interfaces within the waste mass during seismic loading. The impact of this displacement on the dynamic response of the waste mass and cover and on the stresses imposed on the material at the interface by the relative displacement may have important consequences for landfill design.

Kavazanjian and Matasović (1995) modelled relative displacement at liner and cover interfaces in non-linear one-dimensional response analyses of a solid waste landfill. The results of the analysis indicated that relative displacement along a weak interface had potentially beneficial effects on the response of the waste and cover. The weak interface appeared to function as a frictional base isolation system, limiting the average acceleration of the mass above the interface to a value corresponding to the interface strength. Bray, et al. (1995) cite a study by Whitman and Lin in 1983 that indicated that ignoring relative displacement when computing seismic response for use in Newmark deformation analyses lead to conservative estimates of the permanent seismic deformation. Relative displacement at the interface may also limit the shear stress applied to the geosynthetic material at the interface to the value of the interface shear strength.

Very little information exists on the dynamic behavior of geosynthetic interfaces. Limited test data presented by Yegian, et al. (1995a, 1995b) indicates that geosynthetic interfaces exhibit complex non-linear hysteretic stress-strain behavior. Figure 17 presents a set of hysteresis loops for a geosynthetic interface, developed by Yegian, et al. from shaking table tests, illustrating this complex behavior. The test results of Yegian, et al. also demonstrate the potential for horizontal geosynthetic interfaces to beneficially modify the seismic response of the overlying material. These results support similar conclusions drawn from earlier work by Kavazanjian, et al. (1991) and Yegian and Lahlaf (1992).

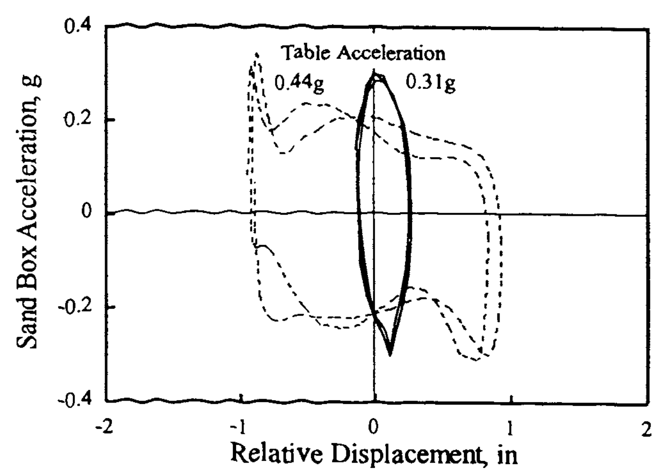

Fig. 17. Hysteretic Behavior of Geosynthetic Interfaces (Yegian, et al., 1995b)

Selecting Acceleration Records: As with any ground response analysis, selection of the earthquake records is a key step in the analysis procedure. Records must be selected that provide reasonable representations of the duration and frequency content of earthquakes that could affect the landfill. This can be clearly demonstrated by the initial response measurements made at the OII landfill. For small, high frequency earthquakes, attenuation of the ground motion occurred as the seismic wave travelled from the base to the top of the landfill (Anderson, et al., 1992). This was incorrectly interpreted by many that landfill materials always attenuate ground accelerations.

Results of Fourier analyses of records obtained at OII, presented in Figure 6, show that some of the low-frequency energy (e.g., 0.5 to 1 Hz) was amplified by a factor of more than 10 (Anderson, et al., 1992). However, little energy arrived at the landfill with frequencies less than 2 or 3 Hz, due to the small size of the earthquakes that were recorded. Hushmand, et al. (1990) and Anderson, et al. (1992) warned that more distant large earthquakes, with a larger contribution of low-frequency energy, could actually result in amplification of the ground motions at the OII landfill. Results of ground motion records obtained at the OII landfill during the Northridge earthquake, presented in Figure 7, indicate that the acceleration at the top of the landfill was nearly the same as at the base but that the characteristic frequency at the top was lower, again suggesting that higher frequency energy is attenuated and lower frequency energy is amplified by the landfill. This behavior suggests the prudent engineer should consider a suite of earthquake strong motion records for most ground response modelling efforts. These records may represent a range of potential earthquakes for the area, potentially including both distant large magnitude events and nearby smaller magnitude events.

Two- and Three-Dimensional Response Effects: The importance of two- and three-dimensional effects on landfill performance has received considerable attention recently, particularly in light of the landfill instability at Kettlemen Hills Landfill. Two- and three-dimensional effects are an important consideration in the evaluation of landfill stability and will be addressed again in the next section of this paper. With respect to dynamic response analysis, the issue related to two- and three-dimensional effects is whether or not ground acceleration values will be modified because of the shape of the landfill, at least relative to predictions made with a one-dimensional ground response analysis.

The primary concern related to two- and three-dimensional effects is that ground motions will amplify because of the focusing effects of the landfill geometry. This phenomenon is observed in earth dams, where the typical triangular shaped cross-section results in higher accelerations at the crest of the dam than at the base (Makdisi and Seed, 1976). However, in all but the most unusual configurations for landfills, this phenomenon is not expected to be critical. Analyses of two-dimensional effects in earth dams indicates that one-dimensional analyses conducted at different points in the geometry will generally provide a reasonable estimate (within 15 percent) of ground response (Vrymoed and Calzascia, 1978). Generally, slopes of landfills are flatter than slopes of earth dams and landfill decks are broader than dam crests. Therefore, two-dimensional response effects in landfills should be even less significant than in earth dams. Results of two-dimensional site response analyses at the OII landfill appear to confirm this expectation (Earth Technology, 1989). The inaccuracies associated with using one-dimensional analyses to evaluate landfill seismic response is expected to be significantly less than the uncertainty associated with the material characterization of the solid waste materials. On this basis, two-dimensional response analyses do not appear to be warranted at the present time for most problems.

Landfill Slope Stability

Pseudo-Static Analyses

Seismic design of landfill slopes is typically based either upon a pseudo-static analysis with an appropriately selected seismic coefficient or a quantitative analysis of permanent deformations induced by seismic loading. Even when a deformation analysis is used, pseudo-static stability analyses are generally required as part of the deformation analysis. Furthermore, the results of deformation analyses reported in the literature provide a rational basis for determining the appropriate value of the seismic coefficient for pseudo-static analysis.

Pseudo-static stability analyses are typically performed for both the waste mass/liner system and the landfill cover system. In a pseudo-static analysis, a limit stability equilibrium analysis is performed in which the earthquake loading is represented by an equivalent horizontal force. The horizontal force is the product of a seismic coefficient and the weight of the failure mass. The pseudo-static analysis is typically performed in one of two manners. In a conventional "stand alone" pseudo-static analysis, the factor of safety is calculated for a seismic coefficient that is some pre-determined fraction of the peak acceleration of the failure mass (expressed as a fraction of gravity). If the factor of safety is greater than 1.0 to 1.15, the failure mass is considered seismically stable. This approach has historically been used to evaluate the stability of cut and fill slopes for roadway embankments, earth dams, and other earth structures (Seed, 1979). While this approach is simple, since most slope stability computer programs incorporate this capability, the method has significant limitations relative to determination of the appropriate acceleration coefficient to use in computing the horizontal earthquake force and the acceptable factor of safety.

The second manner in which pseudo-static analyses are employed are in conjunction with Newmark (1965) deformation analysis. In the second approach, the seismic coefficient is varied to determine the value at which the factor of safety is equal to 1.0. The seismic coefficient for a factor of safety of 1.0, termed the yield acceleration when multiplied by the acceleration of gravity, is then used in the Newmark deformation analysis to calculate permanent seismic deformation.

Choosing the Seismic Coefficient: The peak acceleration estimated from the ground response analyses represents an instantaneous peak, in most cases occurring only once during the earthquake. Therefore, common practice is to reduce the peak acceleration to some lower value for use as the seismic coefficient in pseudo-static analyses. However, the amount of reduction varies according to conditions of the analysis and the knowledge and judgment of the person conducting the analysis.

Results of Newmark analyses conducted by the Waterways Experiment Station (Hynes and Franklin, 1984) on 354 accelerograms, presented in Figure 18, suggest that if the coefficient used in the pseudo-static analysis is one-half of the peak acceleration, permanent seismic deformations of the slope will be less than 100 to 300 mm. In many cases, limiting deformations to 100 to 300 mm will be tolerable. In these cases, if a pseudo-static factor of safety of 1.0 or greater is calculated using a seismic coefficient equal to one half the peak acceleration of the failure mass, satisfactory performance may be assumed. For a cover system the peak acceleration of the top of the landfill should be used in the analysis. For a liner system, the MHEA should be used in the analysis.

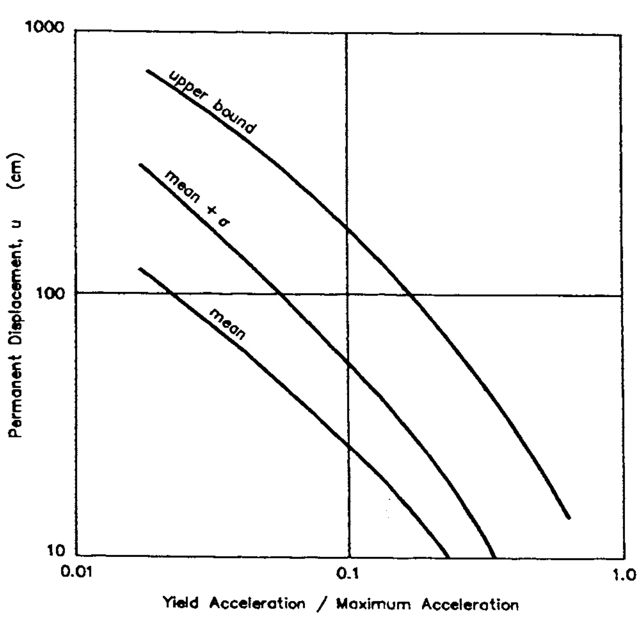

Fig. 18. Results of Deformation Analyses on 354 Accelerograms (Hynes and Franklin, 1984)

Limitations of Pseudo-Static Analyses: The application of pseudo-static analyses to the stability of liner and cover systems is relatively straight forward. These systems have well-defined failure planes corresponding to geosynthetic interfaces. The pseudo-static approach is also often used to evaluate stability along failure surfaces passing through the waste mass. However, it is questionable as to whether such surfaces have any real validity. Well-defined shear surfaces passing entirely through waste have never been reported. Laboratory testing on solid waste indicates that it continues to strain harden almost indefinitely (Jessberger and Kockel, 1993). Therefore, it appears that deformation, and not stability, may control the seismic performance of the waste mass and that pseudo-static analyses of failure surfaces passing entirely through the waste mass are of limited value from a design perspective. However, pseudo-static analyses of surfaces passing through the waste are commonly required to satisfy regulatory agencies.

Evaluating Refuse Shear Strength: If analysis of failure surfaces passing through the waste mass are required for regulatory compliance, the shear strength of the waste must be evaluated. Even if pseudo-static analyses of failure surfaces passing entirely through the waste are not conducted, waste shear strength parameters may still be required for failure surfaces that pass along liner interfaces and exit through the waste. While the strength of earth materials during seismic loading can be deduced with some degree of confidence, the strength of refuse, even for gravity loading, is subject to considerable uncertainly. Generally, the consistency of refuse does not lend itself to normal laboratory testing methods; i.e., the size of the testing equipment is too small relative to the normal size of the refuse material. Given this limitation, refuse properties during static and seismic loading have been deduced from observations of refuse slopes in the field. From these observations, various researchers (e.g., Singh and Murphy, 1990; Kavazanjian, et al., 1994) have concluded that refuse strength envelopes exhibit both apparent cohesion and frictional strength characteristics.

Figure 19 presents the refuse strength envelope deduced by Kavazanjian et al. (1994) in a critical analysis of laboratory and field data. The refuse strength envelope in Figure 19 is based primarily on re-analysis of the stable refuse slopes that have shown acceptable deformations under static conditions. It may be that even greater strengths can be mobilized at large shearing strains or rapid rates of loading than represented by Figure 19. Large diameter triaxial tests performed by Jessberger and Kockel (1993) on solid waste, presented in Figure 20, indicated that solid waste continues to significantly strain harden at strains in excess of 20 to 30 percent. In practice, the static strength properties in Figure 19 are used for pseudo-static analysis. This approach is probably conservative from the standpoint that under short-term loading conditions less creep and relative displacement of the waste constituents will occur in the refuse, resulting in larger short-term stiffness and resistance to loading.

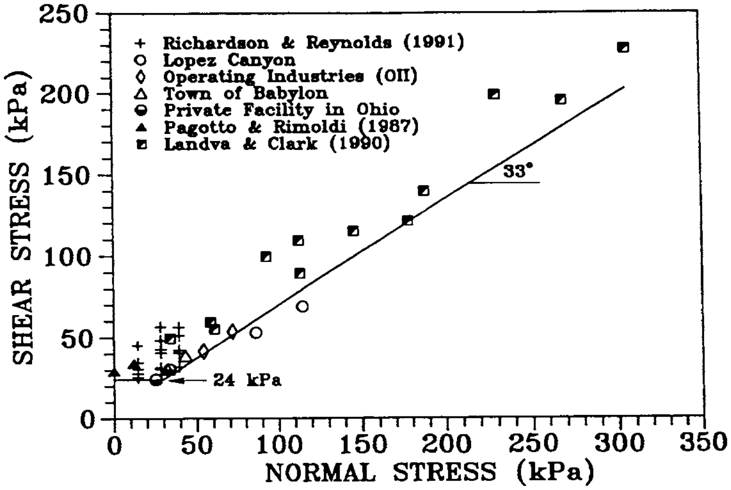

Fig. 19. Shear Strength of MSW (Kavazanjian, et al., 1995)

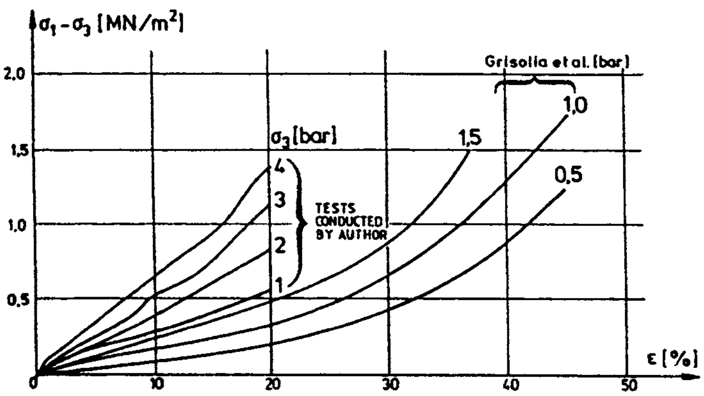

Fig. 20. Triaxial Compression Tests on Reconstructed and Milled Waste (Jessberger and Kockel, 1993)

Accounting for Three-Dimensional Effects: In most limit equilibrium stability analyses, three-dimensional effects are ignored. However, as a result of the Kettlemen Hills landfill instability, the potential for adverse three-dimensional effects on landfill stability have gained considerable attention. Various methods have been suggested for handling these effects for stability under gravity loading. Logically, whenever three- dimensional effects are of concern for gravity loading, they must be considered for seismic loading.

Reductions in seismic stability can be anticipated for those cases where the geometry involves a wedge shape, canyon-type geometry and the resistance at the downslope toe is inadequate to resist the inertial forces from the earthquake. Pseudo-static methods have been used to approximate this case by estimating the weighted average factor of safety for slices taken through the landfill. This approach does not necessarily provide a particularly realistic representation of side walls for such landfills. The weighted average factor of safety approach is, however, useful for those cases where the landfill is shaped like a nob, with a fairly small top deck. For this type of geometry, a two-dimensional analysis through the center the highest point of the landfill will generally be very conservative, as the average driving force on the waste mass is lower in other parts of the landfill. A limited number of three-dimensional solutions are available for regularly shaped failure masses (e.g., blocks, cones, semi-circles, see Chang, 1992). These solutions can often be used to make qualitative evaluations of three-dimensional effects. Three-dimensional limit equilibrium analysis computer programs such as CLARA (Hungr, 1992) and TSLOPE3 (Pyke, 1993) appear to be the only quantitative methods for evaluating the stability of irregularly shaped waste masses.

Newmark Deformation Analyses

Newmark (1965) deformation analyses for permanent seismic deformations represent an extension of the pseudo-static analysis method. In this approach, the acceleration coefficient at which the factor of safety equals 1.0, termed the yield acceleration, is computed using pseudo-static slope stability methods. Seismic deformations are then evaluated as a function of the yield acceleration and the peak acceleration of the ground using some form of Newmark's sliding block on a plane deformation method (Newmark, 1965). In its simplest form, Newmark deformations can be selected from design charts (Franklin and Chang, 1977; Makdisi and Seed, 1978; Hynes and Franklin, 1984). Alternatively, computer programs that calculate deformations as a function of yield acceleration for selected acceleration time histories can be used (Yan, 1991; Repetto, et al., 1993).

Determining the Yield Acceleration: The yield acceleration for the Newmark analysis is computed using the procedures discussed previously for pseudo-static analyses. The seismic coefficient in the pseudo-static analysis is varied until the factor of safety is equal to 1.0. The computed yield acceleration is normally assumed to be constant and applicable at the base of the sliding mass. In some cases, the yield acceleration may decease over the course of an earthquake (e.g., for a geosynthetic interface with a peak and residual strength). In these cases, analyses using a yield acceleration derived from residual strengths will give a conservative answer while analyses using the initial peak strength for the yield acceleration are likely to give unconservative deformation results. In the absence of a computer program in which a deformation-dependent yield acceleration can be specified, a conservative but reasonable approach is to iterate to obtain a strain compatible yield acceleration. The deformation from an initial calculation using the residual strength is used to evaluate a deformation consistent strength from laboratory test results for the next iteration.

Understanding the Accuracy of Deformation Prediction: The Newmark deformation method is relatively easy to apply. In fact, for earth slopes it is conventional practice to use this methods. Unfortunately, it is often applied without regard for an understanding of the accuracy of the prediction. For instance, engineering reports are sometimes submitted giving deformation estimates made using the Newmark method to tenths of a millimeter, when the accuracy of the method is at best one or two orders of magnitude greater.

Results of Newmark deformation predictions indicate whether movements are large or small. Accuracies less than 15 to 30 mm are meaningless in many cases, given the simplifying assumptions and uncertainties associated with the prediction. As an example of these assumptions, the yield acceleration as well as the deformation prediction in a Newmark analysis assume that the failure mass moves as a rigid block. Shearing stresses and resistances at the base of the block are assumed to be uniformly distributed and the base is assumed to be horizontal and smooth. Such assumptions may be numerically gratifying, but they have little resemblance to conditions likely existing at a landfill base. Typical construction practice is to have cross slopes to collect leachate above the liner, resulting in a series of small, relatively flat ridges and valleys. Given the differences in soil confinement from the head to the toe of the landfill, it is also unreasonable to expect soil resistance to develop uniformly during seismic loading. Furthermore, the acceleration time history used in the deformation analysis is typically calculated assuming no relative displacement and is then used to calculate a relative displacement. While the net effect of all of the inaccuracies may well be conservative, caution is warranted in interpreting the results of Newmark seismic deformation analyses, whether from design charts or computer analysis.

Determining Acceptable Levels of Deformation: Seed and Bonaparte (1992) suggest that deformations ranging from 150 to 300 mm are typically accepted in practice for design of geosynthetic liner systems. For cover systems, even larger deformations are usually tolerated, realizing that in most cases a cover can deform significantly without causing failure and that most cover failures can be detected and repaired at reasonable costs. However, calculated deformations of 300 mm may begin to approach the limits of accuracy of the Newmark deformation method and therefore often serve as the maximum allowable calculated liner displacement for this reason.

Clearly, a limiting deformation of 150 to 300 mm represents a significant design criterion. The logical question is whether larger deformations could be tolerated. From a fundamental standpoint, larger allowable displacements could be allowed for smooth interfaces with no penetrations and regular geometry. The distance over which the predicted displacements will occur is also an issue. The danger is that if the displacement occurs over a short distance the liner could tear and the tears could be extremely difficult to detect and expensive to repair. Therefore, due to the significant limitations of the Newmark deformation approach, engineering judgment rather than basic engineering principles must be used to rationalize the allowable deformation.

Three-Dimensional Effects: The Newmark deformation method can be extended to account for three-dimensional effects by using a three-dimensional pseudo-static stability analyses to calculate the yield acceleration. However, as three-dimensional pseudo-static analyses introduce additional complexities to the analysis, the accuracy of Newmark method deformation predictions made based upon three-dimensional pseudo-static stability analyses is clearly even less than the accuracy of predictions made using two-dimensional pseudo-static analyses.

Effect of Vertical Vibrations

The state of practice in geotechnical engineering is to explicitly consider only the horizontal acceleration in evaluating the stability and deformation of earth structures and/or landfills. There are two primary reasons that vertical accelerations are not explicitly considered. First, the horizontal acceleration is the principal de-stabilizing force that acts on earth structures as well as the principal source of damage observed in earthquakes. The writers are not aware of reported damage to a geotechnical structure that has been attributed to vertical accelerations. Second, vertical accelerations are implicitly accounted for in geotechnical analyses due to the fact that the analyses are calibrated based upon field observations of the performance of geotechnical structures. To the extent that vertical accelerations acted upon the slopes in the field used for verification and calibration of conventional geotechnical seismic stability analyses, the effect of vertical accelerations has therefore been implicitly accounted for during model calibration and verification.

The second writer is currently directing analyses of the impact of simultaneous horizontal vertical ground motions on the performance of lined landfills using limit equilibrium analyses and Newmark deformation analyses. The results of the limit equilibrium analyses indicate that, in the cases looked at to date, superposition of a pseudo-static vertical acceleration equal to one-half to two-thirds of the horizontal pseudo-static acceleration on the failure mass typically reduces the yield acceleration by less than 10 percent. As the peak vertical acceleration obtained from strong motion records is typically one-half to two-thirds of the recorded peak horizontal acceleration, imposition of a vertical pseudo-static acceleration equal to one-half to two-thirds of the pseudo-static horizontal acceleration simultaneously with the pseudo-static horizontal acceleration is considered to be an extremely conservative assumption for calculating the yield acceleration. Hence, it appears that vertical accelerations may have only a small influence on the yield acceleration for typical landfill liner systems.

In Newmark deformation analyses performed as part of the same evaluation, horizontal and vertical acceleration time-histories from the Taft record obtained in the 1952 M 7.4 Kern County earthquake were imposed simultaneously on the failure mass. The records were maintained in precisely the same phase as they were recorded in the earthquake. Results of the analyses show that, for yield accelerations between 0.05 g and 0.30 g, the imposition of vertical accelerations on the block increased the deformations by approximately 10 percent compared to analyses in which only the horizontal accelerations were imposed on the block. While more analyses using additional pairs of vertical and horizontal acceleration time histories are required before a general conclusion can be drawn from these analyses, these initial results indicate that vertical accelerations may have only a small effect on the magnitude of permanent seismic deformations.

The combined results of the limit equilibrium and Newmark deformation analyses performed by the writer appear to indicate that, in accordance with the state-of-the-practice for cut and fill slopes and earth dams, vertical deformations need not be explicitly considered in analysis of the seismic performance of landfill slopes.

Rigorous Numerical Modelling

Rigorous numerical modelling of landfill deformations under seismic loading can be performed using the same two-dimensional finite element computer codes discussed with respect to seismic site response analysis. The appeal of these codes is that they do not impose arbitrary discrete failure surfaces and that they treat the dynamics and kinematics of the two- or three-dimensional geometry of the landfill more realistically. Some of these codes even offer the potential for modelling relative displacement at weak interfaces using interface elements. However, these codes are rarely, if ever, used in practice. As discussed with respect to site response analysis, a primary limitation of these codes is the inability to accurately determine the material properties of solid waste. As noted in previous discussions, it is very difficult to estimate the dynamic strength and stress-strain properties of refuse. Inaccuracies due to the considerable uncertainty in material properties of solid waste is compounded by the extreme sensitivity of the deformation predicted by the methods to the values of the material properties. Experience in performing two-dimensional finite element response analyses indicates that strains and deformations are much more sensitive than stresses to the values of the material properties.

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

Performance Observations

While landfills have generally performed very well during earthquakes, there are at least five significant caveats regarding the performance of landfills during earthquakes:

- strong motion records have been obtained at only one unlined, atypical solid waste landfill subject to moderate ground motions (PGA equal to 0.25 g);
- observations of the response of geosynthetic-lined landfills subject to strong ground motions (PGA greater than 0.25 g) are limited to three landfills in Northridge earthquake, one of which had a liner tear;
- no landfill with a geosynthetic cover has ever been subjected to strong ground motions;
- cracking has typically been observed in soil cover systems; and
- landfill gas recovery systems are subject to disruption.

Observations of the performance of solid waste landfill slopes in earthquakes indicate that, in general, these slopes perform well. The primary problem reported with respect to the seismic performance of landfill slopes is cracking of cover soils at transitions between the waste fill and natural ground and at locations where the geometry of the cover changes. Observations of the response of the OII landfill in recent earthquakes demonstrate that landfills can, and do, amplify earthquake ground motions. The strong motion records obtained at the OII landfill provide for the first time a means of calibrating dynamic response analyses at solid waste landfills.

The tear in the geomembrane liner at the Chiquita Canyon landfill is certainly cause for caution. One logical hypothesis is that the cause of the tear is related to the use of a conventional "L" shaped anchor trench detail on the anchor bench. The stress concentrations that develop around such an anchor trench may have facilitated the tear. This hypothesis suggests that conventional anchor trenches may not be advisable from a seismic performance standpoint. Flat or "V"-shaped anchors can generally provide adequate anchoring capability while minimizing stress concentrations in the liner. Alternatively, "L"-shaped anchor trenches can be cut and abandoned when the liner system is extended for subsequent phases of landfill development.

The potential for low shear strength interfaces in geosynthetic cover systems and the potential for amplification of ground motions at the top of landfills must be considered in the design of landfill cover systems. Concern over the stability of landfill covers is mitigated by the fact that, in contrast to liner systems, damage to landfill covers can generally be readily detected in post-earthquake inspections and repaired at a relatively low cost. On this basis, many engineers and regulators consider seismic performance of cover system a post-earthquake maintenance concern as much as a design concern. This consideration applies to both the cracking of cover soils and the stability of geosynthetic covers.

Disruption to landfill gas recovery systems is one of the most common impacts of earthquakes on landfills. Sources of disruption typically include loss of power, breakage of well heads, and breakage of gas and condensate lines. Power loss, while common, is easily mitigated through installation of a back-up generator. Well heads and condensate lines require regular maintenance during normal operating conditions. Degradation in structural integrity due to exposure to ultra-violet radiation, landfill gas and gas condensate, and diurnal temperature fluctuations and stress on the piping system due to landfill settlement make repair and replacement of gas recovery system components a common maintenance operation at MSW landfills. As long as spare parts are available, repairs can be quickly effected. Therefore, as shut down of a landfill gas recovery system for less than 48 hours is not considered an environmental hazard, the observed disruption to landfill gas systems in earthquakes is not considered a major problem and is easily mitigated through prudent preparedness and response measures.

Performance Analyses

The Algermissen maps provide the prescriptive statutory basis for evaluating the design earthquake for seismic performance analyses of landfills under Subtitle D. Alternative site-specific hazard analyses may also be allowed at the discretion of State and Tribal regulatory agencies. Use of the Algermissen maps require considerable judgment and interpretation. The Algermissen maps typically do not reflect the best available knowledge on regional tectonics and local geologic conditions and do not provide essential information on the magnitude associated with the design earthquake.

Alternative site-specific approaches to evaluating the design earthquake can include both probabilistic and deterministic site-specific seismic hazard analysis. Site-specific analyses facilitate inclusion of the best available information on regional seismic sources, local geologic conditions, and attenuation of earthquake motions in evaluating the design earthquake. Considerable expertise is required to perform a proper site-specific analysis. The use of cook book or black box computer programs for site-specific seismic hazard analysis should be avoided.

The influence of local soil conditions and of the landfill on the design ground motions must be evaluated in a site response analysis. The peak acceleration at the top of the landfill and the peak average acceleration of the waste mass above the liner are the key parameters from the response analysis required for use in cover and liner system design analyses, respectively. Both analyses and observations from recent earthquakes indicate that earthquake ground motions can be amplified by the waste mass. This amplification potential must be considered in design. Simplified approaches are available for estimating the amplification of the peak acceleration at the top of the landfill and the peak average acceleration of the waste mass. These simplified approaches should be used with caution and should not be used on critical or important projects or at sites underlain by soft special study soils (shear wave velocity less than 100 m/s).

One-dimensional site response analyses using the equivalent linear method are the most common means used in practice to analytically evaluate site response. The major challenge in performing an equivalent linear response analysis is in evaluating the properties of the solid waste. The spectral analysis of surface waves (SASW) method provides a non-intrusive means that can be used to determine the initial shear wave velocity profile. The initial unit weight profile and modulus reduction and damping curves are also required input to the equivalent linear analyses. Limitations on laboratory and in situ testing of solid waste make field observations the most reliable means of assessing these parameters. However, little field data are available for these purposes. Back analyses of strong motion records recovered at the OII landfill in the Northridge earthquake have provided for the first time a means of calculating modulus reduction and damping in solid waste from field measurements.

The pseudo-static approach offers a simple method of evaluating possible effects of seismic loading on landfills. However, extreme care must be used with this approach. Effects of waste property variation should be considered in the analysis. A factor of safety of 1.0 for a seismic coefficient equal to 50 percent of the peak acceleration predicted within the landfill or cover system generally results in acceptable levels of deformation. If the factor of safety is less than 1.0 with a seismic coefficient equal to half the peak acceleration, a Newmark deformation analysis can be performed to calculate permanent seismic deformation of the failure mass.

The Newmark deformation analysis represents standard practice for deformation analysis of landfills subject to seismic loading. While there are clearly uncertainties in this approach, it provides a better understanding of expected landfill behavior than pseudo-static methods. Application of the Newmark deformation approach should not, however, be done blindly. Uncertainties in the method need to be understood and parametric studies to bracket possible behavior are prudent. Evaluation of the allowable deformation from a Newmark deformation analysis for both liner and cover systems is a matter of engineering judgment and not engineering mechanics.

Two-dimensional seismic response and deformation analysis of landfills are rarely, if ever, employed in practice. There appear to be few benefits to performing these types of analyses at the present time, as the difference between the accelerations predicted in one-dimensional and two-dimensional response analyses is typically less than the uncertainty associated with evaluation of material properties. Furthermore, the deformations predicted by these analyses is usually extremely sensitive to the material properties. For these reasons, the added computational accuracy of a two-dimensional response analysis is usually outweighed by the limited benefit derived from this computational accuracy, the added complexity and cost of the analysis, and the uncertainties associated with the material properties.

Conclusions

In general, the performance record of landfills in earthquakes is good. Typical impacts of earthquakes on landfills include cracking and limited downslope movement of cover soils and disruption to landfill gas collection systems. This performance record includes landfills in the epicentral region of magnitude 7 earthquakes with slopes as steep as 2H:1V to heights as great as 90 m. Optimism about the seismic stability of landfills based on this performance record is tempered by the fact that only three modern landfills with geosynthetic liner systems have been subjected to strong ground motions in excess of 0.30 g and one of these landfills suffered a tear in the geomembrane. Furthermore, there are no observations of the performance of a landfill with a geosynthetic cover system on record. Considering the stability problems associated with geosynthetic cover and liner systems under static conditions, the seismic performance of landfills remains a significant design consideration.

Until more observational data are available on the performance of geosynthetic liner and cover systems subjected to strong ground motions, prudence is called for in the design of these systems for landfills subject to seismic loading. Analytical modelling using relatively simple one-dimensional site response analyses, pseudo-static limit equilibrium analyses, and Newmark deformation analyses can provide quantitative estimates of the performance of modern landfills subject to seismic loading, but significant engineering judgment is still required when interpreting the results of these analyses.

At the present time, the ability to numerically calculate the dynamic response of landfills appears to have out-stripped knowledge of the material properties to use in the analysis. Uncertainties about the dynamic behavior of solid waste and geosynthetic materials render complex two- and three-dimensional seismic performance analyses of limited value for engineering design. However, such analyses may still be of benefit in identifying patterns of behavior and failure mechanisms and in focusing attention on the areas where additional research and development are needed. As knowledge about material properties improves, these more rigorous analytical methods will likely provide valuable information on the distribution of stress and deformations during seismic loading from which conclusions on the performance of geosynthetic liner and cover systems can be drawn.

Recommendations

To significantly improve our ability to evaluate the seismic performance of landfills, improved information on the static and dynamic material properties of solid waste and on the dynamic properties of geosynthetic interfaces is required. While evaluation of the dynamic properties of geosynthetic interfaces may be amenable to laboratory testing, problems associated with sampling and testing limit the usefulness of laboratory testing of solid waste. Only through the back analysis of well-documented case histories of the performance of solid waste landfills will our knowledge of the dynamic behavior of solid waste materials improve significantly.

Undoubtedly, the greatest need with respect to improving our ability to evaluate the seismic performance of landfills is strong motion records of landfill response in earthquakes for back-calculating dynamic properties. The records should be obtained on and within the landfill or waste containment system. However, the necessary material properties for seismic analysis are not limited to the dynamic properties of the waste and hence the necessary documentation is not limited to strong motion records. The unit weight of the waste is an important factor in seismic response analyses about which uncertainty still exists. Installation and monitoring of settlement platforms as the waste fill rises combined with tracking of waste receipts and daily cover quantities is a simple, inexpensive monitoring measure that would yield invaluable data for both static and dynamic analysis. Lateral deformation measurements using inclinometers can also yield important data on waste properties. Due to the non-homogeneity of solid waste materials, the changes in the properties of solid waste materials with time, and the variability in solid waste between different landfills, this type of instrumentation and data collection is needed at a substantial number of landfills in different geographic locations and climatological regimes. Only through this type of concerted data collection effort can analysis and design of landfills subject to seismic loading rise to the same level of proficiency as other aspects of geotechnical earthquake engineering.

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