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Seismic Design of Solid Waste Containment Facilities

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ABSTRACT

Despite recent advances, seismic design of solid waste containment facilities must still be regarded as an emerging field of practice. However, sufficient data is available to provide a framework of knowledge for environmentally protective design. Observations of performance in earthquakes show that solid waste landfills can sustain very strong shaking with limited damage and without a harmful discharge of contaminants. The excellent record of seismic performance of landfills is tempered by limited experience with geosynthetic containment systems. The potential for amplification of ground motions by the waste mass and the low yield acceleration often associated with geosynthetic cover systems indicate that attention is warranted to this issue. The limited data from regions outside of California, limited experience with saturated waste, and evolutionary changes in waste streams and waste disposal practices provide continuing challenges to engineers designing solid waste disposal facilities subject to seismic loading.

INTRODUCTION

In response to societal concerns over the potential adverse environmental impacts of waste disposal activities, great strides have been made in engineering practice for design of containment facilities for municipal, industrial, and hazardous solid wastes in recent years. During this time, landfills have evolved into complex structures with sophisticated engineered containment systems, as illustrated in Figure 1.

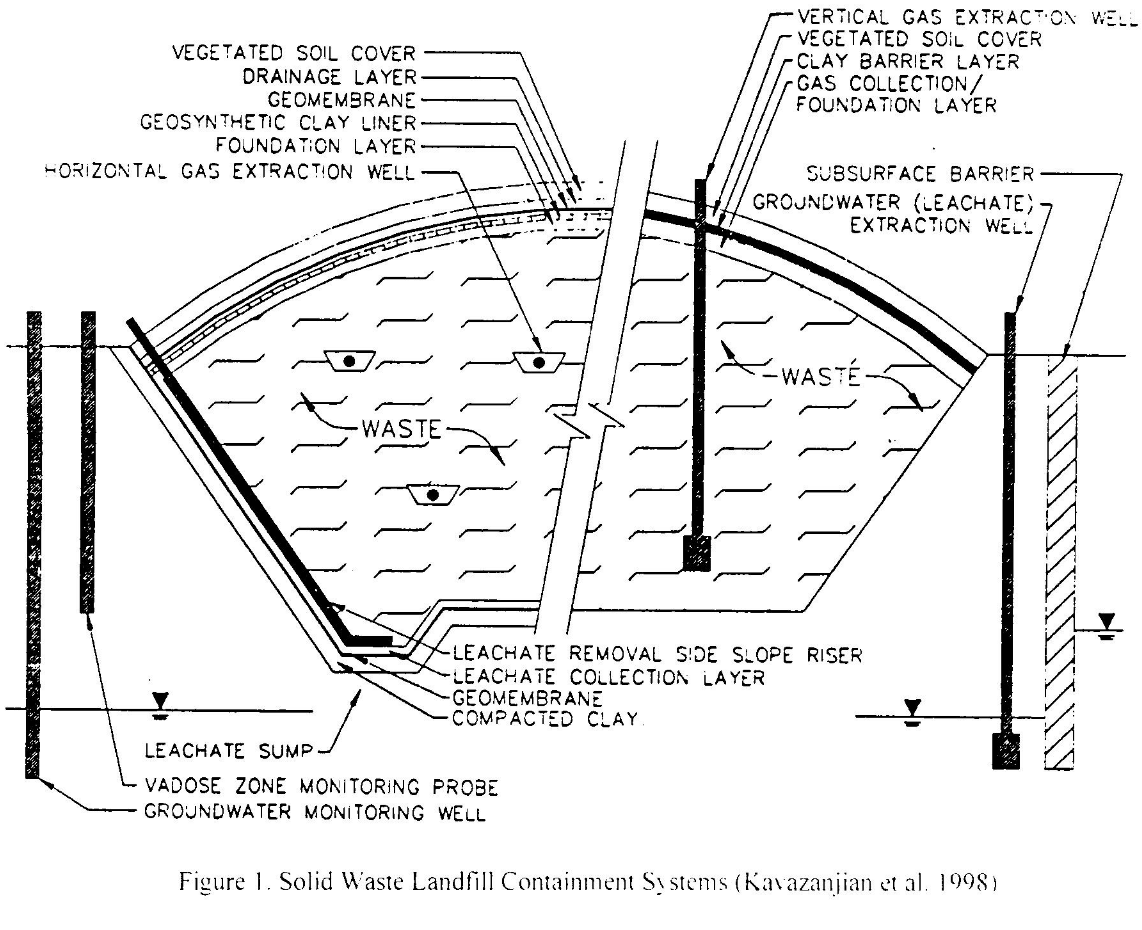

Figure 1. Solid Waste Landfill Containment Systems (Kavazanjian et al. 1998)

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In the past decade, both the technology employed for waste containment and the engineering analyses used to design these facilities for short and long-term static conditions have advanced to the point where they may be considered mature, though still rapidly evolving, disciplines. However, despite significant advances in recent years, seismic design of waste containment facilities must still be considered an emerging discipline due to significant uncertainties with respect to the physical properties and mechanical properties of waste and the lack of documented case histories. The available data does however provide a basis for sound, environmentally protective seismic design of waste containment systems while leaving open the opportunity for more economical and cost-effective solutions in the future as our knowledge base increases and the seismic design sub-discipline matures.

The performance of solid waste containment facilities subject to seismic loading has, to date, been good (Matasovic et al. 1998a). There is no documented incidence of an earthquake-induced release of contaminants harmful to human health or to the environment from an engineered waste containment facility. However, this finding must be qualified by the observation that our experience is limited primarily to relatively dry solid waste landfills in California. The record of satisfactory performance includes several landfills with geomembrane liner systems subjected to free field peak ground accelerations in excess of 0.3 g. One of these lined landfills did suffer significant damage but without a harmful discharge of contaminants. However, no solid waste landfill with a geosynthetic cover system (i.e., a cover system including a geomembrane, geosynthetic clay liner, or drainage geocomposite) has ever been subjected to strong ground shaking. Observations and analyses of the seismic response of solid waste landfills indicate that geosynthetic cover system may be particularly vulnerable to seismic loading. Furthermore, at least two landfills with geosynthetic liners have suffered dramatic failures under static conditions and there are numerous cases of failures of geosynthetic cover systems under static conditions. Therefore, based upon observed static stability problems with geosynthetic liners and covers and the limited observations of the seismic performance of landfills with geosynthetic components, attention is clearly warranted to the seismic design of solid waste containment facilities.

SEISMIC PERFORMANCE

Unlined landfills have performed extremely well when subject to strong shaking in earthquakes. Numerous landfills have been subjected to ground motions in excess of 0.1 g in earthquakes of moment magnitude (M_w) as great as 7.4 without serious damage (Matasovic et al., 1998a). Some of the landfills subject to the most intense shaking, ground motions in excess of 0.5 g in the M_w 6.9 Loma Prieta earthquake, had side slopes in excess of 60 m in height with side slope inclinations on the order of 2 horizontal to 1 vertical. Landfills constructed since 1994 almost always include geomembrane liner systems. While these liner systems provide enhanced protection against liquid and gas migration from the landfill, they also introduce a potential plane of weakness at the base of the landfill. Observations of the performance of geomembrane liner systems subject to strong ground shaking in earthquakes are essentially limited to five solid waste landfills subject to the M_w 6.7 Northridge earthquake (Matasovic et al., 1995). Post-earthquake reconnaissance surveys indicate that the geomembrane liner system one of these facilities, the Chiquita Canyon landfill, suffered significant damage. Four other geomembrane lined facilities were subject to strong ground motions estimated to be in excess of 0.3 g in the Northridge event: the Bradley Landfill, the Simi Valley Landfill, the Lopez Canyon landfill, and the Calabasas Landfill. The integrity of the containment system was not compromised at any of these other facilities. The performance of these facilities provides positive evidence that properly designed and constructed geomembrane liner systems can withstand strong ground shaking in a major earthquake without significant damage.

One of the more important set of observations of solid waste landfill performance in earthquakes is the strong motion records obtained at the Operating Industries, Inc. (OII) landfill in southern California (Hushmand Associates, 1994). At the OII landfill strong motion instruments were installed in 1987 by USEPA at the base and crest of the landfill. These stations captured landfill response in a number of nearby small earthquakes ($M_w > 5.1$) and two relatively distant large events: the M_w 6.7 Northridge event (site-to-source distance approximately 43 km) and the M_w 7.4 Landers earthquake (site-to-source distance approximately 140 km). These data provided conclusive evidence of the potential for amplification of seismic motions by solid waste landfills contrary to previous suggestions that solid waste landfills may unconditionally attenuate earthquake motions. Fourier analysis of several small magnitude events by Anderson et al. (1992) yielded spectral amplification factors over 10 around the resonant period of the waste mass for small amplitude vibrations. Analysis of the recorded response at OII in both the small and large magnitude events by Matasovic et al. (1998) indicate a potential for amplification of peak acceleration at the top of the landfill similar to that observed in earth dams, as illustrated in Figure 2. Using waste properties back-calculated from the strong motion records supplemented by large strain cyclic simple shear testing on reconstituted samples (described subsequently in the paper), Matasovic and Kavazanjian (1998) predicted peak accelerations at the top of the OII landfill in excess of 1.0 g for free-field bedrock motions with a peak horizontal ground acceleration of 0.6 g. Using their own set of waste properties back calculated from the recorded data at OII, Bray and Rathje (1998) evaluated municipal solid waste (MSW) landfill amplification potential by subjecting a suite of 15 waste profiles to a suite of four time histories. Results of the Bray and Rathje analyses, presented in Figure 3, also indicate that the amplification potential of MSW landfills is similar to the amplification potential of earth dams as established by Harder (1991). These data, which indicate significant ground motion amplification potential for solid waste landfills, have significant implications with respect to the seismic performance of landfill cover systems.

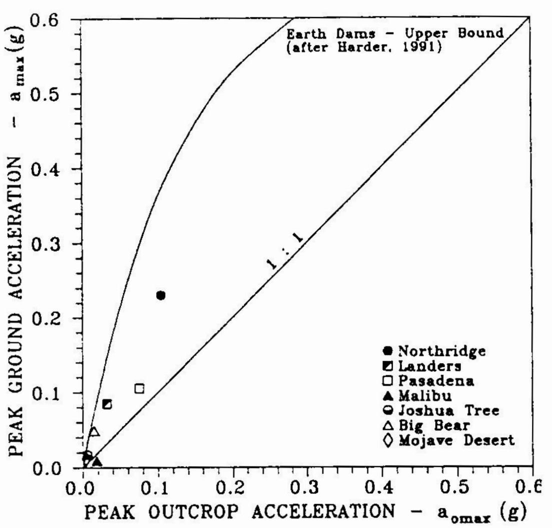

Figure 2. Amplification at OII (Matasovic et al., 1998a)

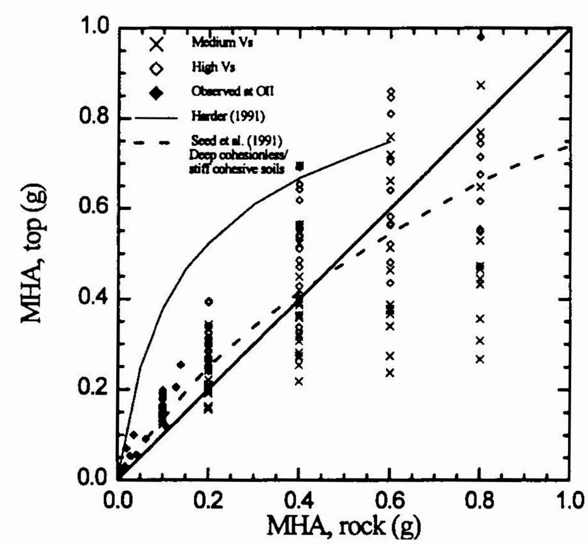

Figure 3. MSW Landfill Amplification (Bray and Rathje, 1998)

Although not due to seismic loading, the recent failure at the Doña Juana Landfill in Colombia is of particular importance with respect to the seismic stability of landfills in wet climates, landfills that receive liquid wastes, and landfills that recirculate liquids. At Doña Juana, excess pore pressures generated by liquid recirculation (< biblio >) triggered a massive flow slide. This phenomenon is particularly important with respect to seismic stability because it raises the possibility of seismically induced waste liquefaction and flow slides in saturated and shear-saturated wastes if cyclic loading induced a tendency for the waste to compress (decrease in volume). While little is known about the potential for waste compression when subject to seismic loading, the associated potential for strength loss of saturated waste subject to seismic loading clearly warrants attention.

WASTE PROPERTIES

General

The physical and mechanical properties of the waste mass are an important consideration in seismic design analyses, as the seismic response of the waste mass governs the seismic loads applied to the containment system. Waste mass properties of interest include unit weight, shear strength, compressibility, small strain shear stiffness (modulus), and modulus reduction and damping curves. Most available information on these properties is on MSW. Only limited information is available on the properties of industrial and hazardous solid waste. Due to its large particle size and heterogeneous nature, characterization of the physical and mechanical properties of solid wastes present unique and significant challenges to the engineer, some of which have not yet been completely resolved.

Unit Weight

Despite its fundamental nature and importance to both static and dynamic analysis, relatively little information is available on the unit weight of solid waste and much of the available information appears to be contradictory, particularly for MSW. The apparent contradictions stem, to a large extent, from failure to properly define quantities and terms used to derive total unit weight estimates. Reports of the unit weight of MSW found in the literature often fail to distinguish between the weight of the refuse and the combined weight of refuse, soil, and liquids in a typical unit volume. Furthermore, reports of the ratios of refuse to soil in a unit volume generally fail to specify whether the cited ratio was established on a weight or volume basis. In practice, landfill operators typically report only the amount of refuse in a unit volume of landfill and report refuse:soil ratios on a volume basis. However, engineers have frequently interpreted these values as representing the total weight of soil and refuse in a unit volume and/or as a weight-based ratio. Furthermore, the weight of liquids which accumulate in the waste after the refuse has been placed in the landfill is often ignored in making unit weight estimates. These misinterpretations can lead to significant errors in unit weight estimates which, in turn, can lead to significant, often unconservative, errors in stability analyses and may also affect seismic response calculations.

For instance, the Kavazanjian et al. (1995) unit weight profile shown in Figure 4 was developed based upon initial and average unit weights reported by landfill operators and upon values of unit weight and waste compressibility reported in the literature. However, in developing the curve, it was assumed that the operator reported-values of unit weight represented the total unit weight of soil and refuse when, in fact, they represented only the weight of the refuse in a unit volume of landfill. Upon recognizing this error, the initial and average unit weight values were adjusted upwards by 20 to 25 percent to account for operator reports of refuse:soil ratios on the order of 3:1 to 4:1, based on the realization that these ratios were weight-based ratios. However, test pits and large diameter (750-mm) bucket auger borings at the Azusa and OII landfills in southern California revealed typical refuse:soil ratios at depth of 1:1, and more, by weight. Upon further investigation, it was determined that the 3:1 to 4:1 ratios reported by the operators represented volume ratios (i.e., three trucks of refuse to one truck of soil) and were thus consistent with a typical 1:1 weight ratio at depth, after the solid waste had been compressed by overburden. (Note that, because the refuse is more compressible than soil, the percentage of soil by weight in a unit volume will increase with depth). Thus, assuming an approximate 1:1 ratio for soil to refuse, by weight, initial refuse unit weight values of 4 to 6.5 kN/m^3 become total unit weights on the order of 8 to 13 kN/m^3 and the 1995 Kavazanjian et al. curve in Figure 3 shifts significantly to the right, particularly at shallow depths.

Substantiation of the above logic was provided by in-situ unit weight tests performed at the Azusa and OII landfills in southern California. At both locations, unit weight was evaluated using a "modified sand cone" technique in which the waste removed from an excavation (a trench or large diameter (750 mm) bucket auger borehole) is carefully weighed and the volume of the excavation is measured by backfilling with calibrated gravel. At OII, where the refuse:soil weight ratio often was less than 1:1 (i.e., more soil than refuse), these measurements yielded a representative total unit weight of 15.7 kN/m^3 to a depth of 30 m, with no apparent trend with depth (Matasovic and Kavazanjian, 1998). At Azusa, where the refuse:soil weight ratio was typically somewhat greater than 1:1, measured total unit weight values varied from 9.4 kN/m^3 near the surface to over 15 kN/m^3 at depth. The in-situ unit weight measurements at the Azusa landfill were correlated with shear wave velocity to develop a relationship between overburden pressure-unit weight and shear wave velocity that was accurate within 10 percent (Kavazanjian et al., 1996). This correlation was then used to infer unit weight profiles at five other MSW landfills in southern California. Figure 4 summarizes the unit weight profiles developed at these six southern California landfills using the Azusa shear wave velocity-overburden correlation along with the Kavazanjian et al. (1995) recommended profile. Limited data from European landfills (Manassero et al., 1996) is consistent with the mean profile.

The unit weight profiles shown in Figure 4 that were developed from the shear wave velocity data were based upon calibration with field data on the total unit weight. However, these profiles should not be indiscriminately considered as representative of the total unit weight of MSW in all climates. As southern California is a semi-arid to arid region, and local regulations generally prohibit liquid disposal in MSW landfills, the unit weight profiles in Figure 4 should be considered representative of the dry unit weight of solid waste. For landfills in more temperate climates, in areas with greater rainfall, where liquids are disposed of or re-circulated, or where waste is below the water table, the weight of the liquid in an elevated volume must be factored into the total unit weight.

The influence of moisture content and soil content on the unit weight of solid waste is illustrated in Figure 5. This figure presents the results of seven large-diameter (457-mm) one-dimensional compression tests performed on reconstituted waste samples from the OII Landfill (GeoSyntec Consultants, 1996). Samples were re-constituted at their field moisture content after removing oversized particles (particles larger than 100 mm). In general, the samples with higher soil content and higher moisture content yielded the higher unit weights. For instance, specimen CONMH was so moist that free liquids squeezed out of the specimen during consolidation. Specimen SWCONDM composed almost entirely soil or soil-like material. The in-situ testing at OII showed a similar trend, with the highest in-situ unit weight measured in zones where the waste was wettest and/or soil and soil-like material content was highest.

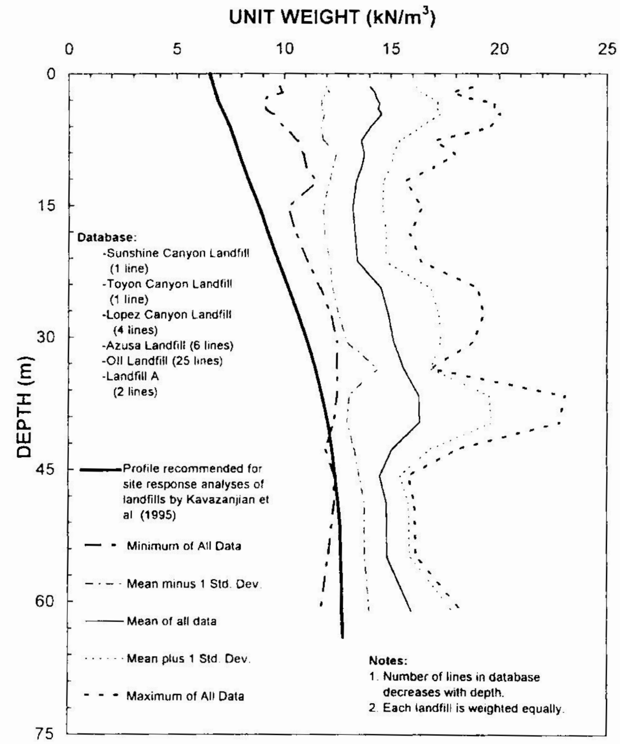

Figure 4. Unit Weight of Solid Waste

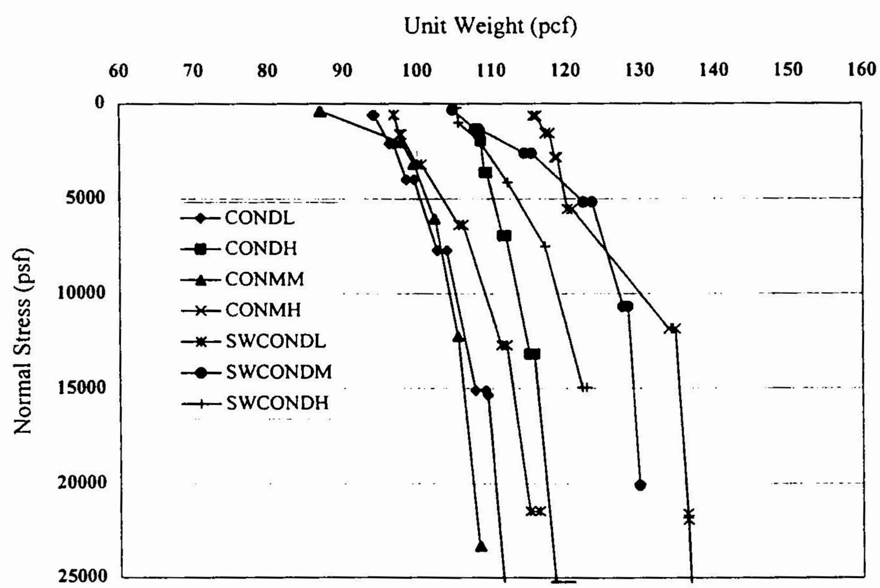

Figure 5. OII Unit Weight Values (GeoSyntec Consultants, 1996)

Shear Strength

In evaluating the seismic performance of solid waste containment facilities, the strength of the waste itself is an important consideration. In 1995, Kavazanjian et al. developed the bi-linear strength envelope presented in Figure 6 from published data on laboratory and in-situ testing supplemented by back analysis of stable waste slopes. The strength envelope shown in Figure 6 (the bi-linear envelope) was presented as a lower bound envelope for the drained strength of municipal solid waste.

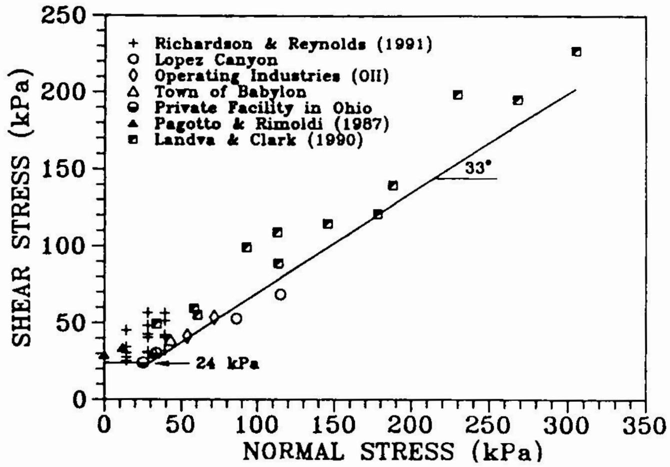

Figure 6. Bi-Linear MSW Strength Envelope (Kavazanjian et al., 1995)

As part of pre-design studies for closure of the OII Landfill, large diameter (457 mm) direct shear and simple shear tests were conducted on reconstituted specimens of solid waste (GeoSyntec Consultants, 1996). The specimens were reconstituted to their estimated field density at their field moisture content. Particles larger than 100 mm in dimension were removed and specimens were compacted in horizontal lifts approximately 100 mm thick. The long axis of the large particles was oriented along the horizontal plane. Shear wave velocities measured in the lab on the reconstituted specimens were within the range of values measured in the field, indicating that the stiffness of the reconstituted specimens was at small strains, similar to that of the waste in-situ. Figure 7 presents the results of the direct shear tests based upon the strength at a strain of 10 percent. Included in this figure is the percentage of refuse material in the specimen corresponding to each data point. Figure 7 indicates that, in general, the direct shear strength increased with increasing refuse content. Furthermore, except for the specimens with less than 15 percent refuse, the direct shear strengths agreed well with the bi-linear strength envelope shown in Figure 6.

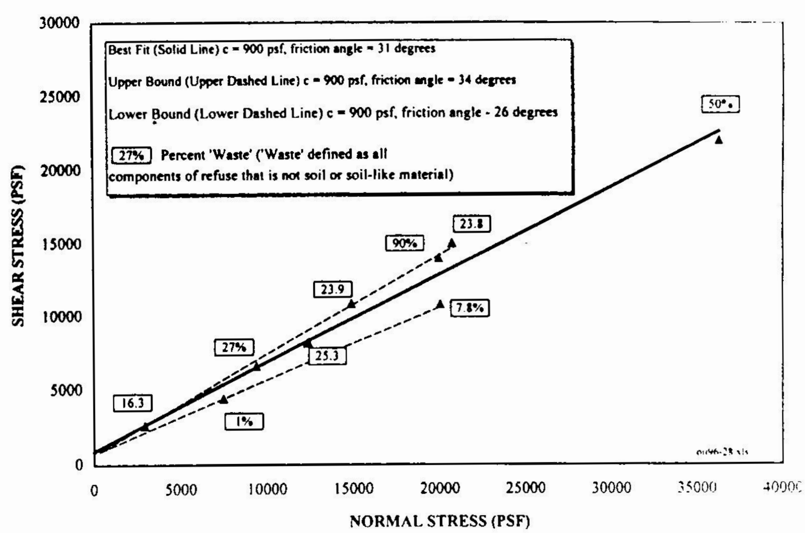

Figure 7. OII Direct Shear Test Results (GeoSyntec Consultants, 1996)

Figure 8 shows the Mohr circles at failure for the simple shear strength tests performed for the OII project. Included in this figure are three different strength envelopes: the lower bound developed by assuming that failure occurs along the horizontal plane and upper and lower bound envelopes developed assuming failure occurs on the plane of maximum obliquity (the plane with the maximum ratio of shear stress to normal stress). The failure envelope for maximum obliquity applies to planes that cut diagonally across the specimen and yields waste strengths significantly greater than the bi-linear strength envelop or the direct shear tests. The interpretation that assumes a horizontal plane of weakness yields a lower bound strength consistent with, but slightly smaller than, the bi-linear envelope or the direct shear tests. However, even if this interpretation of the simple shear tests on the reconstitute specimens is correct, it is unlikely that solid waste in the field is as highly structured (layered) as the reconstituted specimens in which the long axis of all particles were manually aligned in the horizontal plane. Furthermore, the dis-assembled samples showed no indication of shearing along a horizontal plane. Regardless of which interpretation is correct, Figure 8 indicates that solid waste may possess significantly higher strengths than indicated by the bi-linear envelope when sheared across the preferred orientation of the particles.

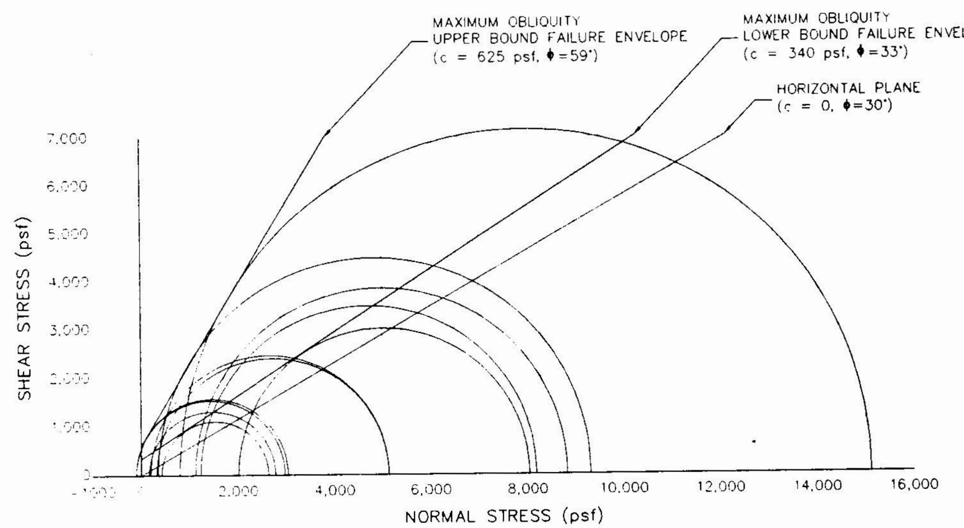

Figure 8. OII Simple Shear Strength Envelopes (GeoSyntec Consultants, 1996)

In March 1996, at the Rumpke Landfill in Cincinnati, Ohio, approximately 1.1×10^{-6} m^3 of solid waste, covering an area of approximately 8 hectares, slid into a 4.5 hectare excavation at the toe of the waste slope. Prior to the slide, the slope rose approximately 110 m above grade at an average inclination of 2.6 horizontal to 1 vertical. After the slide, there was a 36-m high, nearly vertical, head scarp and many large prismatic blocks of intact waste within the slide mass (Figure 9). Mitchell (1996) reports that the Kavazanjian et al. (1995) bi-linear strength envelope is consistent with waste behavior observed at the site and "may be conservative". Mitchell states that this conclusion was based on the strength required to maintain a stable 36-m high head scarp, recently completed laboratory tests on waste, and back analyses of landfill performance in the Northridge earthquake. Thus, the available data indicates that the bi-linear envelope is a reasonable representation of the effective stress static shear strength of MSW, though some adjustment may be necessary to account for anisotropy of the waste mass (e.g., along horizontal planes of weakness created by waste placement practices).

The bi-linear strength envelope in Figure 7 was developed based upon static tests. The data from the Rumpke failure and the testing at OII also pertain to static loading. Data on MSW shear strength during rapid, dynamic loading was developed by Augello et al. (1995) from back analysis of the performance of landfills in the 1994 M_w 6.7 Northridge earthquake. Table 1 presents back calculated friction angles for solid waste. These values are based upon observation of stable slopes and thus represent lower bound values. The back-calculated waste strengths reported in Table 1 are consistent with and, in general, greater than strengths represented by the bi-linear envelope. Thus, the bi-linear envelope would appear to be valid for both static and dynamic shear strengths.

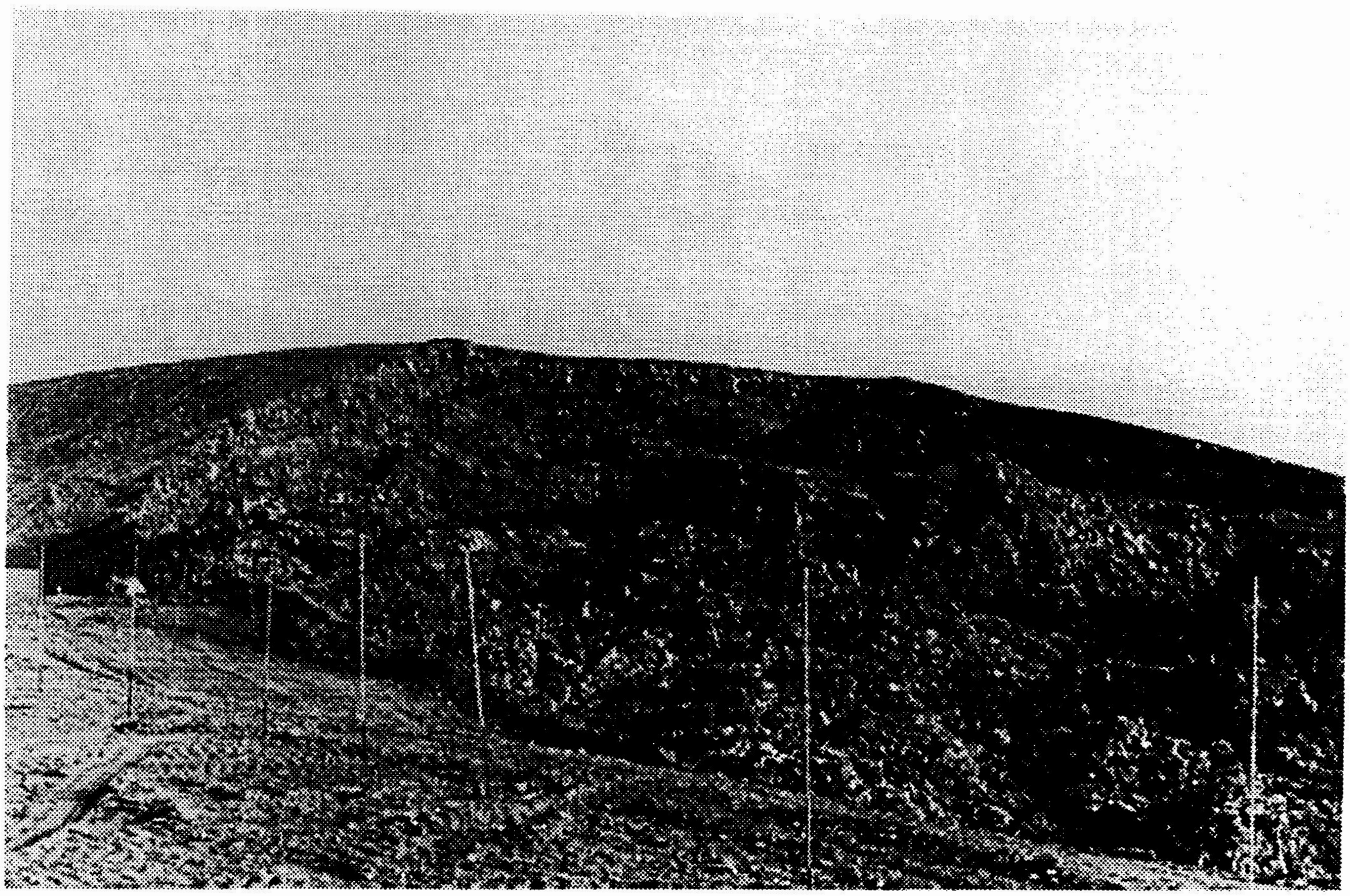

Figure 9. MSW Slope Failure at the Rumpke Landfill

The bi-linear envelope, back analyses of the waste strength at Rumpke, and the OII laboratory data represent drained strength data. Manassero et al. (1996) presented an undrained strength envelope for MSW that is consistent with, but slightly below, the Kavazanjian et al. (1995) bi-linear drained strength envelope. A lower undrained shear is consistent with contractive behavior during shear. Contractive behavior was observed in the cyclic simple shear tests at OII (Matasovic et al., 1998c), where volumetric strains on the order of 3 to 5 percent were typically observed in response to five cycles of loading with a peak-to-peak axial strain of 6 percent. Contractive behavior of MSW during cyclic loading is of concern because, in a saturated material, this tendency for contractive volumetric strain may manifest itself as a pore pressure increase and associated strength decrease if the loading is faster than the rate of pore pressure dissipation. The recent waste landslide at the Doña Juana Landfill in Bogotá, Colombia, where mass movement of 1.5 million metric tonnes of waste was triggered by pore pressure rise due to leachate reinjection, is graphic evidence of the potential strength loss associated with pore pressure increase in MSW. The potential for strength loss in saturated waste subject to cyclic loading is a serious concern that has not been fully explored and must be considered in landfills with waste beneath the phreatic surface, where significant liquids are disposed of, where significant infiltration occurs, or where leachate reinjection is practiced.

Table 1. Shear Strengths Back Calculated from Observations of Landfill Performance in the Northridge Earthquake (Augello et al., 1995)

Landfill	Estimated Seismic Coefficient	Analysis Assumptions	Observed Deformation Factor	Clay Liner Strength, S, (ϕ-0) (kPa)	Geosynthetic Liner Strength ϕ (degrees) (c-0)	Waste Strength ϕ (degrees) (c-0)
OII Cross Section H - H	0.11	1, I, A_{syn}, WS	0.7			39
	0.16	1, I, A_{rec}, WS	0.7			42
	0.18	2, I, II, A_{rec}, WS	0.7			43
	0.23	1, II, A_{rec}, WS	0.7			44
Toyon Canyon	0.13	1, I, A_{syn}, WS	0.7			27
	0.20	1, I, A_{rec}, WS	0.7			30
	0.21	1, II A_{syn}, WS	0.7			31
	0.32	1, II, A_{rec}, WS	0.7			36
Sunshine Canyon	0.18	1, I, A_{rec}, WS	0.4			33
	0.22	1, I, A_{syn}, WS	0.4			35
	0.29	1, II, A_{rec}, WS	0.4			39
	0.56	1, II, A_{syn}, WS	0.4			52
Lopez Area A	0.28	1, I, A_{syn}, WS	0.7			36
	0.43	1, I, A_{rec}, WS	0.7			45
Lopez Area C Cross Section A - A'	0.19	1, I, A_{syn}, A_{rec}, LS	0.7		14	
	0.28	1, II, A_{syn}, LS	0.7		19	
	0.46	1, II, A_{rec}, LS	0.7		28	
			0.7			
Lopez Area C Cross Section B - B'	0.23	1, I, A_{syn}, LS	0.7		18	
	0.25	1, I, A_{rec}, LS	0.7		19	
	0.36	1, II, A_{syn}, LS	0.7		25	
	0.52	1, II, A_{rec}, LS	0.7		32	
Chiquita Canyon C	0.09	1, I, A_{syn}, LS	0.4		10	
	0.18	1, II, A_{syn}, LS	0.4		14	
	0.21	1, II, A_{rec}, LS	0.4		16	
Chiquita Canyon D	0.10	1, I, A_{rec}, LS	0.4	44	8	
	0.10	1, I, A_{rec}, LS	0.4	50	4	
	0.23	1, II, A_{rec}, LS	0.4	75	8	
	0.23	1, II, A_{rec}, LS	0.4	80	4	
	0.24	2, I, A_{syn}, LS	0.4	76	8	
	0.24	2, I, A_{syn}, LS	0.4	81	4	
	0.25	2, I, A_{rec}, LS	0.4	78	8	
	0.25	2, I, A_{rec}, LS	0.4	83	4	
Chiquita Canyon C	0.18	1, II, A_{syn}, WS + LS	0.4		8	34
	0.18	1, II, A_{syn}, WS + LS	0.4		4	42
	0.21	1, II, A_{rec}, WS + LS	0.4		8	38
	0.21	1, II, A_{rec}, WS + LS	0.4		4	44
Lopez Area C Cross Section A - A'	0.23	1, I, A_{syn}, WS + LS	0.7		25	35
	0.23	1, I, A_{syn}, WS + LS	0.7		20	41
	0.25	1, I, A_{rec}, WS + LS	0.7		25	37
	0.25	1, I, A_{rec}, WS + LS	0.7		20	43
	0.36	1, II, A_{syn}, WS + LS	0.7		25	49
	0.36	1, II, A_{syn}, WS + LS	0.7		20	54

ANALYSIS ASSUMPTIONS:

1	=	one dimensional dynamic analysis
2	=	two dimensional dynamic analysis
I	=	average modulus reduction and damping curve (Kavazanjian and Matasovic 1995)
II	=	upper bound modulus and lower bound damping curve (PI = 200 clay, Vucetic and Dobry 1991)
A_{rec}	=	modified closest recorded rock motion
A_{syn}	=	synthetic rock motion
WS	=	slip surface passing entirely through waste fill
LS	=	slip surface along liner interfaces
WS+LS	=	slip surface partially along liner interface and partially through waste fill

Stress-Strain Response

Stress-strain properties of interest with respect to the seismic behavior of MSW containment facilities include the small strain stiffness (or modulus), modulus reduction and damping curves, and Poisson's ratio. The small strain modulus can be calculated directly from the shear wave velocity and total unit weight. Shear wave velocity can be directly measured by a variety of techniques, including cross hole and downhole soundings, ambient vibration measurements, and surface wave measurements. Kavazanjian et al. (1996) developed the shear wave velocity profile for southern California landfills shown in Figure 10 from measurements at six different landfills made using the Spectral Analysis of Surface Wave technique. Data collected by Rix et al. (1998) at a landfill in Georgia and data collected by Manassero et al. (1996) from the Netherlands indicates that, in the absence of site-specific data, the southern California profile may provide a reasonable representation of the range of shear wave velocity found in MSW landfills in temperate as well as arid climates.

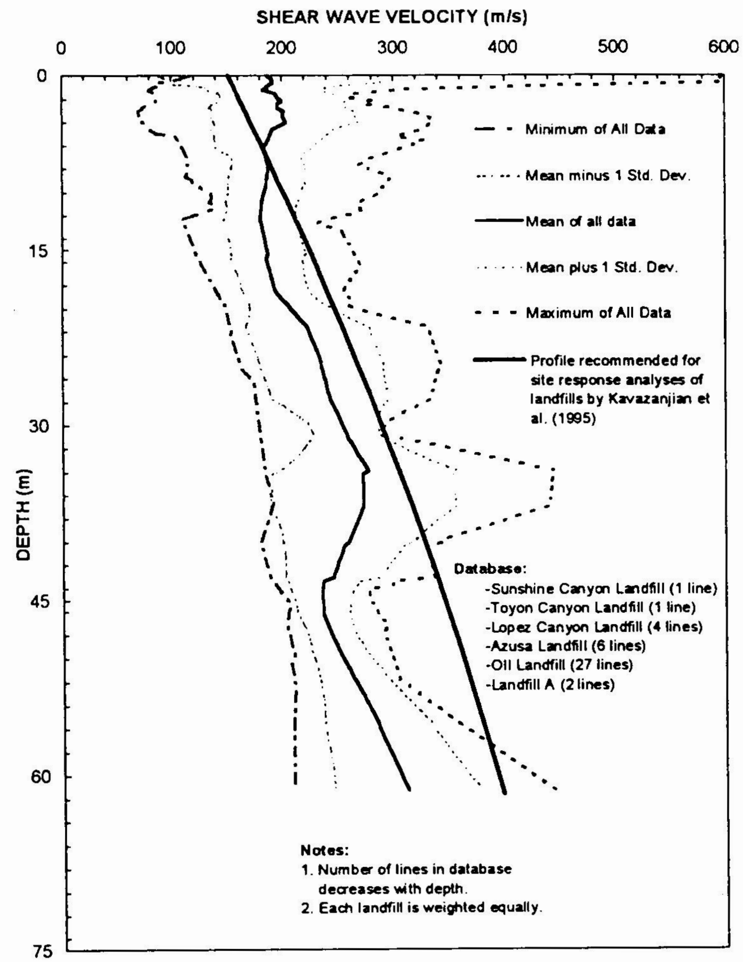

Figure 10. Shear Wave Velocity of Southern California Landfills (Kavazanjian et al., 1996)

Information on modulus reduction and damping curves for MSW is available from only one landfill, the OII Landfill. Strong motion records obtained at the base and top deck of the landfill in several small, nearby and larger, more distant earthquakes (Hushmand Associates, 1994), including the M_w 6.7 Northridge event at a distance of 43 km and the M_w 7.4 Landers event at a distance of 140 km, allow for back calculation of modulus reduction and damping for cyclic strains of up to 0.1 percent. Large diameter (457 mm) cyclic simple shear tests on reconstituted specimens (Matasovic and Kavazanjian, 1998) provide information on behavior at larger strains. Figure 11 shows the modulus and damping curves back calculated from the OII strong motion data by four different groups of investigators. Of the four groups, only Matasovic and Kavazanjian had access to the laboratory cyclic simple shear data when developing their recommendations.

Data from OII also provides the only available information on Poisson's ratio for solid waste. Figure 12 shows Poisson's ratio values for OII developed by Matasovic and Kavazanjian (1998) from measurements of shear wave velocity and compressional wave velocity in the same borehole.

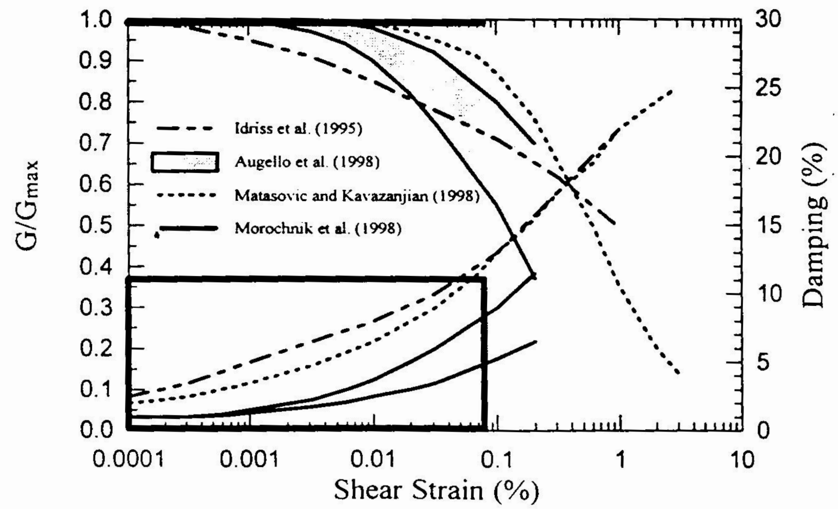

Figure 11. Modulus Reduction and Damping Curves Back Calculated From OII Data By Various Investigators

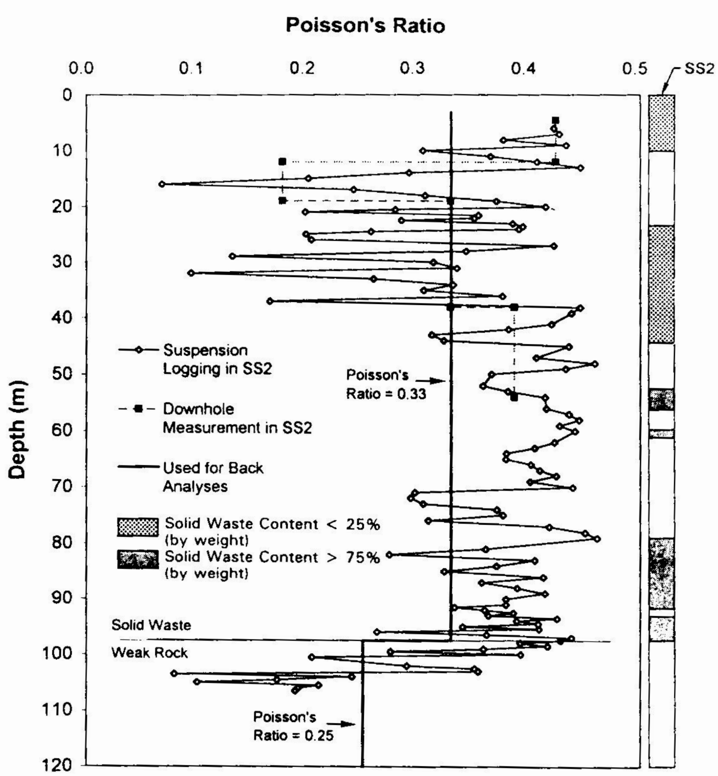

Figure 12. Poisson's Ratio of Solid Waste From OII (Matasovic and Kavazanjian, 1998)

LINER AND COVER PROPERTIES

General

Liner and cover system materials include compacted soil (both cohesive and cohesionless) and geosynthetic materials. Geosynthetic materials include geomembranes, geosynthetic clay liners (gcls), geotextiles (for separation, protection, and filtration), and geocomposite drainage elements. For the soil materials, properties may be evaluated using conventional geotechnical methods. For geosynthetic materials, unit weight is of little consequence due to their minimal thickness and the shear resistance along interfaces with soil and other geosynthetic materials governs their shear strength. For both soil and geosynthetic materials, potential evolutionary changes over the life of the facility (e.g., saturation of compacted clays and gcls) should be considered.

Interface Strengths

There is an abundance of data in the literature on static interface shear strengths. Mitchell and Mitchell (1992) present typical values for common containment system interfaces. Matasovic at al (1998b) summarize the relatively sparse published data on interface strengths measured at shear rates representative of seismic loading that is available, including data from Kavazanjian et al. (1991) and Yegian and Lahlaf (1992). Kavazanjian et al. (1991) report a slight decrease in interface strength with increasing frequency for sinusoidal loading between 1 and 5 Hz for several geosynthetic/geosynthetic interfaces. Conversely, Yegian and Lahlaf (1992) report a slight increase in interface strength with shearing rate. In both cases, the influence of loading rate on interface strength was relatively small.

Table 1 presents back calculated dynamic shear strengths for clay liners and geosynthetic interfaces based upon observed behavior in the Northridge earthquake. The data reported for the clay liners are consistent with the undrained shear strength of clay soils, with undrained strength values ranging from 40 to 80 kPa. The data reported for the geosynthetic interfaces are consistent with the results of laboratory interface shear tests. For the smooth geomembrane interface and geosynthetic clay liner at Chiquita Canyon, where large displacements occurred in one of the lined areas, back calculated interface strengths are on the order of 4 to 8 degrees. At Lopez Canyon, where there was no evidence of permanent seismic deformation of the waste within the lined area was observed, lower bound interface strengths for the clay/textured geomembrane interface varied from 14 to 28 degrees. These data indicate that observed dynamic behavior of interfaces is generally consistent with interface shear strengths evaluated based upon laboratory test data.

Stress-Strain Response

Response analyses performed by Kavazanjian and Matasovic (1995) and by Yegian and Kadakal (1998) indicate that relative displacement at an interface during an earthquake can have a "base isolation" effect. This effect reduces the peak intensity of motions above the interface and shifts the predominant period of the response of the overlying mass. Yegian and Kadakal (1998) recently developed recommendations for estimating equivalent linear values for modulus and damping at geosynthetic interfaces. These values were developed by representing the interface as a 1-m thick layer in an equivalent-linear one-dimensional wave propagation analysis using the computer program SHAKE to model shaking table tests. Figure 13 presents the modulus reduction curve from this analysis. When normalized by the overburden pressure, values for G_e, the "small strain" modulus in Figure 13, varied from 35 to 63. Damping was found to be relatively constant, with the equivalent linear damping ratio equal to 0.45 (45 percent). This relatively large equivalent linear damping value may be a manifestation of the attenuation of the motions above the interface. Analyses by Kramer and Smith (1997) indicate that in some circumstances the period shift associated with relative displacement at the interface may actually move the response of the mass towards resonance and increase seismic displacements above the interface. However, this effect was only pronounced for long-period motions (i.e., for liners with significant waste thickness on top of them, but not for covers). Furthermore, Bray and Rathje (1998) indicate that cases where an increase in the response above the interface occur only in cases where the displacements are too small to be of engineering significance and thus are not of practical importance.

SEISMIC DESIGN CRITERIA

Performance Criteria

Seismic performance criteria are an important consideration in the design of waste containment systems as it provides the framework for subsequent analyses and detailed design. Mirroring developments in other areas of earthquake engineering and in civil and environmental engineering as whole, seismic performance criteria for MSW containment facilities are evolving towards risk-based design from unrealistic and prohibitive "no-damage" performance standards. This evolution is facilitated by the recognition that MSW landfills can sustain significant damage in an earthquake without a discharge of contaminants harmful to human health or the environment (as discussed in the section on Seismic Performance). Recognizing that landfills can sustain damage without a harmful discharge, prevention of a harmful discharge of contaminants to the environment during and after a seismic event provides a rational basis for economically sound and environmentally protective landfill design. Kavazanjian et al. (1998) discuss the development of design criteria for solid waste containment facilities based upon such a "withstand without discharge" standard.

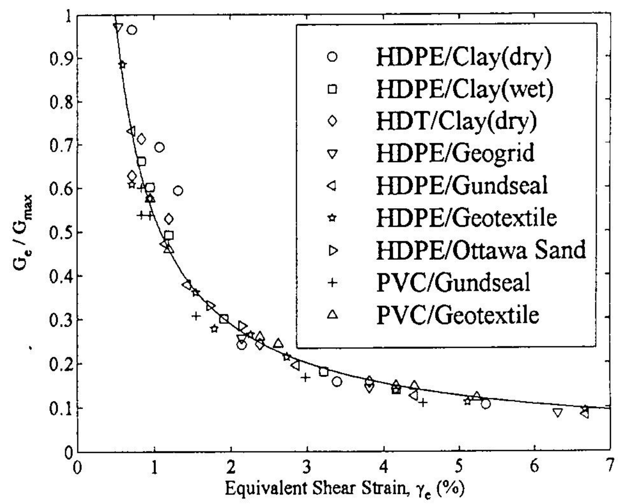

Figure 13. Equivalent Linear Modulus and Damping for an Interface (Yegian and Kadakal, 1998)

Allowable Displacement

In practice, the calculated permanent seismic displacement is the most commonly used index of seismic performance of landfills. The "withstand without discharge" standard is applied in design by specification of maximum allowable values of calculated permanent seismic displacement (the allowable displacement). Allowable displacement depends on the ability to predict seismic deformation, the ability to sustain deformation without loss of function, the impact of a release due to loss of function, the ability to detect loss of function, and the feasibility (and associated cost) of repair (or replacement).

It is difficult to develop generic criteria for the allowable deformation of landfill systems and components, as the allowable deformation depends upon many site- and landfill-specific factors. However, several generalizations can be made. Design details can be an important factor in establishing seismic performance requirements on a system- and component-specific basis. For instance, the allowable displacement at the waste/liner interface can be significantly enhanced by elimination of the liner penetrations. "Best design practices" such as the use of ductile materials, redundancy for critical systems and components, and details that can accommodate anticipated seismic forces and deformations should be used to the greatest practical extent to mitigate the potential for seismic upset.

At the present time, our ability to accurately predict seismic displacements of landfill systems and components is limited to the general order of magnitude (i.e., negligible versus centimeters versus meters). Clearly, many components and systems can sustain substantial deformation without loss of function. Furthermore, with the exception of landfill gas, contaminants discharged from a solid waste landfill may not be very mobile, allowing for detection and repair of breaches in the containment system, even after loss of function, without any contaminant discharge. Damaged landfill covers, above ground pipes and tanks, surface water control systems, and ancillary facilities are generally easy to detect and repair. While damage to the liner may be hard to detect and difficult or impossible to repair, the engineered containment system of a modern landfill has substantial built-in redundancy. The final cover system, LCRS, and liner provide multiple levels of protection against leachate migration to groundwater. One of these components can lose its function without impairing groundwater protection (e.g., the final cover will still provide infiltration control even if the liner is damaged).

Consideration of the above factors on a project specific basis should lead to development of rational, economical seismic design criteria for a solid waste landfill facility. Table 2 provides a list of containment system components with typical values for the allowable calculated seismic displacement used in practice today. These values are based upon the assumption that seismic displacements are calculated in the typical fashion used in conventional practice: using one-dimensional equivalent-linear response analyses, yield accelerations calculated with residual shear strengths, and Newmark-type seismic displacement analyses. If more sophisticated analyses such as two-dimensional equivalent-linear or one-dimensional non-linear response analyses (Matasovic and Kavazanjian; 1995, 1998), advanced Newmark analyses with a degrading yield acceleration (Matasovic et al., 1998b), or analyses which consider slip at the liner interface (Yegian and Kadakal, 1998), are used, lower values of allowable displacement may be warranted as some of the conservatism inherent to conventional practice may be eliminated.

Table 2. Generic Allowable Calculated Seismic Displacements for MSW Landfills

Component	Allowable Calculated Displacement	Comment
Liner System	150 to 300 mm	Actual expected deformation is very small.
Cover System	300 mm to 1 m	Damage is repairable.
Waste Mass	1 m	For displacement not impacting cover or liner.
Roadways, Embankments	1 m	Conventional geotechnical criteria.
Surface Water Controls	1 m	Conventional geotechnical criteria.
Gas Collection System	No Limit	Breakage common under normal operating conditions.

Table 3 presents site-specific criteria for allowable displacements developed closure design of the OII Landfill (Kavazanjian et al., 1998). These criteria were developed in consideration of the two-dimensional equivalent linear site response analyses used in the displacement calculations and the unique urban hazards posed by the facility. As shown in Figure 14, this landfill is constructed immediately adjacent to residential development and looms over a major freeway with slopes up to 65 m high at inclinations of up to 1.3 horizontal to 1 vertical.

Figure 14. The OII Landfill, Monterey Park, California (View from the Northwest)

Table 3. OII Landfill Seismic Performance Criteria (Kavazanjian et al., 1998)

Cover System Component	Design Criteria and Performance Standard	Interim Remediation to Restore Compliance	Repair to Pre-Earthquake Condition
Final Cover			
Soil Monocover on Side Slopes	150 mm of soil deformation. Partial failure contained on site.	3 months to strip vegetation, re-grade and re-compact areas of cracking.	12 months to restore vegetation.
Landfill Gas Control			
Collection Wells	Up to 25 percent of wellheads broken.	1 month to route headers around broken wellheads.	12 months to repair/replace broken wells heads.
Headers	Up to 25 percent of header pipes cracked or broken.	1 month to bypass broken header pipes.	3 months to repair/replace broken headers.
Vacuum Pumps	Power loss. No structural damage.	None required.	1 month to restore off-site power.
Leachate Transmission Pipes	Acceptable breakage of pipes with double containment.	1 month to bypass broken pipes.	3 months to repair broken pipes.
Surface Water Management			
Conveyance Systems (Bench Channels, Down Drains, Culverts)	Cracking and up to 300 mm of displacement.	2 months to completely restore surface pathways.	9 months to replace/rebuild surface pathways.
Sedimentation Basin	Minor cracking of concrete.	2 weeks to 1 month to patch the cracks.	9 months to rebuild the basin (if needed).
Access Roads	300 mm displacement (cracking).	2 months to patch the cracks.	12 months for full repair.

Pseudostatic Analysis

An alternative approach to formal calculation of permanent seismic displacement is pseudo-static limit equilibrium analysis. In this approach, the allowable deformation is implicitly governed by choice of the seismic coefficient. Kavazanjian (1998) discusses general considerations for choice of the seismic coefficient as a function of the allowable displacement and the peak average acceleration of the failure mass. The peak *average* acceleration of the failure mass is used as the normalizing parameter rather than the peak ground acceleration because the earthquake-induced inertial force is proportional to the average acceleration of the entire mass, not the acceleration at a point within or on top of the mass. Table 4 presents mean and mean plus one standard deviation values of calculated deformation versus the ratio of the yield acceleration to the peak acceleration from Newmark displacement analyses performed by Hynes and Franklin (1984). These values were developed by statistical analysis of results of Newmark analyses of 354 accelerograms from earthquakes of magnitude 5 to 8^{+}. A deformation-dependent seismic coefficient can be developed from Table 4 by multiplying the appropriate ratio by the peak average acceleration of the failure mass. Mean values from Table 4 may be used for small magnitude earthquakes (M_w less than 6) and nearby events of intermediate magnitude (distance less than 10 km, M_w less than 7). Mean plus one standard deviation values from Table 4 may be used for more distant intermediate magnitude events and for all large magnitude events (M_w greater than 7). The values in Table 4 indicate that a seismic coefficient greater than or equal to one-third of the peak average acceleration results in small (less than 100 mm) deformations for almost all situations and that for many situations even smaller seismic coefficients may be acceptable. Peak strength may be used with a seismic coefficient established on the basis of Table 4 only if the relevant materials are not susceptible to post-peak strength loss. With materials susceptible to post-peak strength loss, degraded, deformation compatible, or residual strength parameters should be used.

Table 4. Seismic Displacements from Hynes and Franklin (1984) as a Function of the Ratio of Yield Acceleration to Peak Average Acceleration (Kavazanjian, 1998)

Calculated Displacement	Mean Curve	Mean + 1 Std. Dev. Curve
100 mm	0.23	0.35
150 mm	0.17	0.27
300 mm	0.08	0.17
500 mm	0.05	0.11
1 m	0.03	0.06

Response and Recovery

An important part of designing for damage without a harmful discharge is consideration of post-earthquake response and recovery planning. Development of an emergency response and recovery plan includes consideration of preparedness measures, a post-earthquake inspection protocol, interim and long-term repair procedures, and financial assurance for anticipated repairs. The criteria for determining when a post-earthquake inspection is necessary is often based upon combinations of earthquake magnitude and site-to-source distance which induce the threshold value of free-field PHGA considered capable of causing damage, considering amplification due to both foundation soils and the waste itself. For many landfills, provisions in the post-closure Operation, Maintenance, and Monitoring Plan for damage that occurs under "normal" operating conditions may implicitly provide for response to and restoration of the types of damage induced by an earthquake. However, explicit consideration should be given to response and recovery after a seismic event due to the potential for earthquake-induced damage to larger areas of the facility (compared to damage which occurs under "normal" operating conditions) and due to the potential impact of the earthquake on the availability of the required resources (e.g., materials, equipment, and/or skilled labor) for repair and restoration. Consideration should also be given to maintaining an emergency generator, stockpiling special-order materials required for repair, and maintaining lists of approved suppliers and contractors (e.g., geomembrane installers) to facilitate post-earthquake recovery. Table 3 provides an example of the short term and long term recovery goals established for the OII landfill in the post-closure period.

FUTURE CHALLENGES

Observations of the performance of landfills, field measurements, and laboratory testing have created a framework of knowledge for seismic design of solid waste containment facilities. However, the data set is far from complete and is focused primarily on dry municipal solid waste. Observations of the generally good performance of landfills subject to earthquakes are tempered by the fact that relatively few lined landfills have been subject to strong shaking and that no landfill with a geosynthetic cover has experienced strong ground motions. The performance of geosynthetic covers is of particular concern due to the potential for amplification of seismic motions by waste fills and the potential for a relatively low yield acceleration for a soil veneer on top of a geosynthetic cover (Kavazanjian, 1998). Relatively little information is available on the behavior of saturated waste, sludges and other industrial and hazardous wastes, low and high level radioactive wastes, mixed wastes, and other special waste materials. Due to difficulties in sampling and testing these materials, the best source of information on their seismic behavior is field observations, yet little in the way of seismic instrumentation exists at such facilities. Furthermore, changes in the waste stream due to recycling and waste minimization efforts, in the composition of manufactured products and materials, and in landfilling processes requires that consideration be given to potential evolutionary changes in the behavior of municipal solid waste. For instance, the advent of "bio-reactor" landfills, in which liquids are re-circulated by injection into the waste mass, not only raises questions about changes in mechanical properties but heightens concerns about the seismic stability of saturated waste. Until additional field and/or laboratory data is available on these issues, the engineer must use his judgment to assess the importance of these factors and incorporate them in design.

Design to withstand the design earthquake without a discharge of contaminants harmful to the environment provides a rational basis for economical and environmentally protective design of solid waste containment facilities. The concept of allowable permanent seismic displacement provides a practical means for applying this standard. However, uncertainties with respect to waste properties and the accuracy of methods for predicting seismic response of the facility mandate a rather conservative approach to the calculation of permanent seismic displacement and the establishment of allowable values for design. Due to the conservatism inherent of current methods of analysis, current guidelines on what constitutes an allowable calculated displacement are rather crude and are based primarily on whether the allowable displacement should be large, small, or negligible. As our knowledge of waste properties increases and the accuracy of our analytical methods improve, establishment of the allowable deformation will become much more significant. Development of component-specific allowable displacement values that depend upon the ability of the component to sustain displacement and the impact of component failure (excessive displacement) on the containment system will become an important facet of landfill engineering. Furthermore, development of design details that can sustain displacement without "failure" (contaminant discharge) will provide significantly enhanced reliability for landfill systems subject to seismic loading.

CONCLUSION

While the state of knowledge and state of practice for seismic design of solid waste landfills has advanced significantly in the past decade, seismic design of landfill containment facilities is still an emerging field of practice. Data on seismic performance of landfills is primarily from two recent California earthquakes and data on solid waste properties is still restricted primarily to a handful of southern California facilities. The limited data from other geographic areas indicate that the California landfills are not unique and that experience gained there may be generalized to other facilities. However, questions still exist about regional differences in properties due to regional differences in waste composition, the cyclic behavior of solid waste at large strains, the potential for strength loss in saturated waste during seismic loading, and other important issues. Until these questions are investigated more thoroughly, caution is warranted in seismic design of these facilities. For most routine projects, current design methods that employ one-dimensional equivalent linear site response analyses which do not include any relative displacement at geosynthetic interfaces coupled with Newmark displacement analyses based upon residual shear strengths do appear to provide the requisite degree of conservatism. When more sophisticated analyses and/or more aggressive performance criteria are used, site specific waste property measurements and allowable displacement values should be considered.

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