Seismic Response of a Composite Landfill Cover

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Abstract: The Olympic View Sanitary Landfill (OVSL) near Port Orchard, Washington, is a modern solid waste sanitary landfill covered, in part, by a composite cover system. The site was subjected to a free-field peak horizontal ground acceleration on the order of 0.16 g during the 28 February 2001 Moment Magnitude 6.8 Nisqually earthquake. To the knowledge of the writers, this is the first documented case history of a composite landfill cover shaken by strong ground motions. Postearthquake reconnaissance did not find any signs of earthquake-induced permanent displacement of the composite cover system. Accelerograms recorded within 1 km of the facility, the results of site-specific shear wave velocity measurements and laboratory interface shear testing, and these postearthquake observations provide a unique opportunity for a posteriori numerical analysis of this important case history. The seismic performance of the OVSL composite cover system was evaluated using four commonly used methods for seismic design of landfill cover systems. These methods include two simple screening procedures, a more rigorous screening procedure, and a decoupled equivalent-linear site response/Newmark-type permanent deformation analysis using the accelerograms recorded at the nearby strong motion station. The yield accelerations of the composite cover system, required for all four methods, were calculated using the results of construction quality assurance interface shear strength conformance testing. All four methods produce results consistent with the observed performance. However, the two simple screening procedures were significantly more conservative than the other two more rigorous methods.

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Introduction

This paper documents a case history of the observed behavior of several different composite (geomembrane/low hydraulic conductivity soil) final cover systems at the Olympic View Sanitary Landfill (OVSL) near Port Orchard, Washington, subjected to strong ground shaking. Observations of the behavior of municipal solid waste landfills during earthquakes provide the most reliable means of validating and calibrating seismic performance analyses for landfill design. Ideally, validation and calibration of seismic performance analyses employs case histories where material properties and physical conditions are well established, where instrumented strong motion recordings and detailed observations of performance during a seismic event exist, and where secondary or combined effects do not lead to ambiguous interpretations of performance. Realistically, in geotechnical practice, few case histories of any kind meet these criteria. Furthermore, to the knowledge of the writers, there are no case histories of the response of a composite landfill cover system subjected to strong ground shaking [i.e., a bedrock peak horizontal ground acceleration (PHGA) in excess of 0.1 g].

The availability of a pair of strong ground motion recordings obtained during the 28 February 2001 Nisqually earthquake at a site with similar geologic conditions approximately 1 km from the OVSL, coupled with the availability of the results of site-specific shear wave velocity measurements and laboratory interface shear testing data for the composite cover system components, provided a unique opportunity for calibration of analyses of the seismic performance of landfill cover systems. Therefore, the site-specific information was employed in conjunction with four commonly used analytical methods for evaluating the seismic performance of landfill covers to evaluate the performance of the OVSL cover systems in the Nisqually event. Comparison of cover performance calculated using these four methods to the observed performance of the OVSL cover systems provides an index of the accuracy of each method with respect to seismic design of composite landfill cover systems.

Landfill

The OVSL is located approximately 15 km southwest of Port Orchard, Washington. The site, now closed, started receiving municipal solid waste and construction debris in the 1960s as an unlined waste disposal site and was subsequently converted to a modern lined sanitary landfill. The Phase I area of the landfill, indicated in Fig. 1, includes the original unlined portion, founded in the glacial moraine that underlies the entire site, and three areas lined with composite liners. At the time of the earthquake, three of the lined areas in Phase I were closed and capped by composite cover systems. The unlined portion of the Phase I area (the "old landfill") was also closed and capped by a composite cover.

Each of the closed areas was capped by a different type of composite landfill cover system, as indicated in Fig. 1. The three composite landfill cover configurations, referred to as cover Types A, B, and C, are schematically illustrated in Fig. 2. All three

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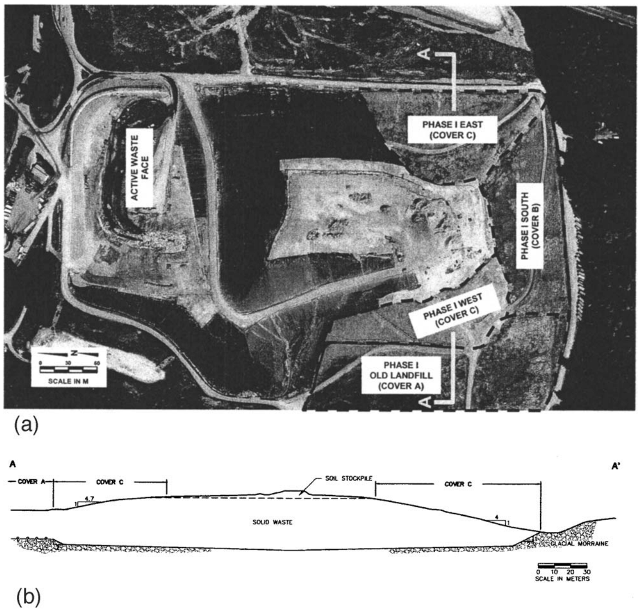

Fig. 1. (a) Aerial photo of the landfill six months after the February 28, 2001 M_w 6.8 Nisqually, Washington earthquake, and (b) cross section A-A'

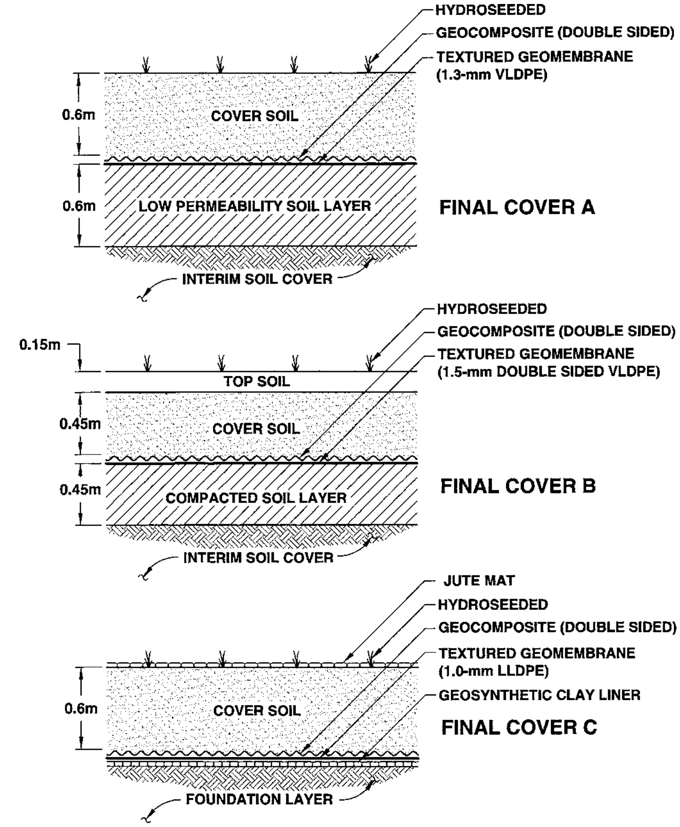

Fig. 2. Composite cover configurations of the OVSL site [data adapted from GeoSyntec Consultants (2003)]

composite cover configurations include a form of textured low-density polyethylene geomembrane overlain by a double-sided geocomposite drainage net with nonwoven filter fabric heat bonded to both sides. Cover Types A and B employ native soil mixed with bentonite as the low hydraulic conductivity soil layer beneath the geomembrane. Cover Type C employs a needle-punched reinforced geosynthetic clay liner (GCL) as the low hydraulic conductivity soil layer instead of the bentonite-amended native soil. All three composite landfill cover configurations are locally inclined at up to 3H:1V (horizontal:vertical) on the side slopes, with a typical side slope inclination (between horizontal benches) of 3.6H:1V in the longitudinal direction and 4.7H:1V in the transverse direction. All three cover configurations include an approximately 0.6 m thick vegetative cover soil layer at the top of final cover. In cover Type B, the top 0.15 m of this cover soil layer is select top soil. In all three cover configurations, the infiltration barrier layer is underlain by a minimum 0.45 m thick foundation layer of compacted soil and an interim soil cover of unknown thickness.

Earthquake

The 28 February 2001 moment magnitude, M_w, 6.8 Nisqually, Washington earthquake was a relatively deep event. The main shock occurred at the interface of the Juan De Fuca and North American tectonic plates, approximately 52 km below the ground surface. Based upon information presented in Pacific Northwest Seismograph Network (2001), the OVSL site was approximately 39 km from the earthquake epicenter and 65 km from the zone of energy release.

The Kitsap County Moderate Risk Waste (KIMR) strong motion station is a free-field station founded on “soft rock/dense soil” approximately 1 km from the OVSL site. Soft rock/dense soil is defined as a site with a shear wave velocity over the upper 30 m of between 360 and 720 m/s or an average standard penetration test blow count in excess of 50. This is Site Class C as defined in the NEHRP (BSSC 1998) Site Classification system. This classification is based upon geologic maps for Kitsap County (where the strong motion station and landfill are located) and is consistent with boring data from the landfill site. During the Nisqually event, the KIMR station digitally recorded acceleration time histories in three orthogonal directions. The recorded PHGA values in the north-south (KIMR-NS) and east-west (KIMR-EW) directions were 0.15 and 0.16 g, respectively. The significant duration of strong shaking, as defined by Trifunac and Brady (1975), was 22.1 and 13.4 s in the NS and EW directions, respectively. The 5% damped acceleration response spectra of both horizontal records are shown in Fig. 3. The peak vertical ground acceleration recorded at the site was 0.07 g. Consistent with the current state of practice for seismic stability analysis of earth structures and seismic design of landfills and with available information on the effect of the vertical component of ground motions on the seismic deformation of geosynthetic cover systems (Matasovic et al. 1998a,b) vertical accelerations during the earthquake were not considered in the analysis described herein.

Field Observations, Geophyscial Survey, and Site-Specific Testing

The Nisqually earthquake occurred while landfilling operations were in progress at the site. Operators at the site working on native ground reported that they were immediately alarmed by the earthquake, while operators working on the solid waste fill reported that they barely noticed that the ground was shaking.

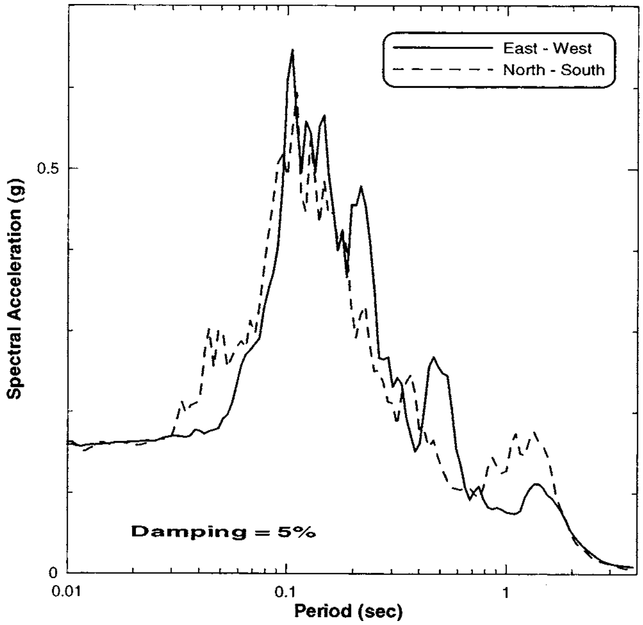

Fig. 3. Strong-motion records recorded in soft rock approximately 1 km from the OVSL site

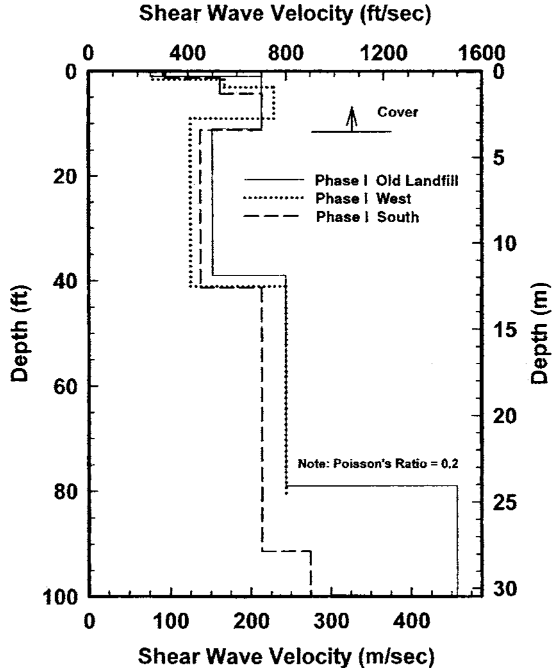

Fig. 5. Results of SASW measurements at OVSL site

The composite landfill cover system was inspected immediately after the earthquake by a landfill crew. A formal reconnaissance team, including the first writer, arrived at the site 5 days after the earthquake to conduct additional reconnaissance. Neither the landfill crew nor the formal reconnaissance team found any evidence of seismically induced permanent lateral displacements of the composite landfill cover. Both teams, however, observed that many of the landfill gas risers had moved laterally relative to the cover by up to approximately 30 mm, as shown by a gap/and or disturbed soil at the riser/final cover contact, as shown in Fig. 4.

A spectral analysis of surface waves (SASW) geophysical survey was conducted at the landfill approximately 2.5 years after the 28 February 2001 event. SASW is a nonintrusive geophysical measurement technique ideal for measuring shear wave velocity profiles at solid waste landfills as it eliminates many of the technical and health and safety issues associated with the borings required for conventional shear wave velocity measurements, e.g., cross hole and down hole shear wave velocity measurements.

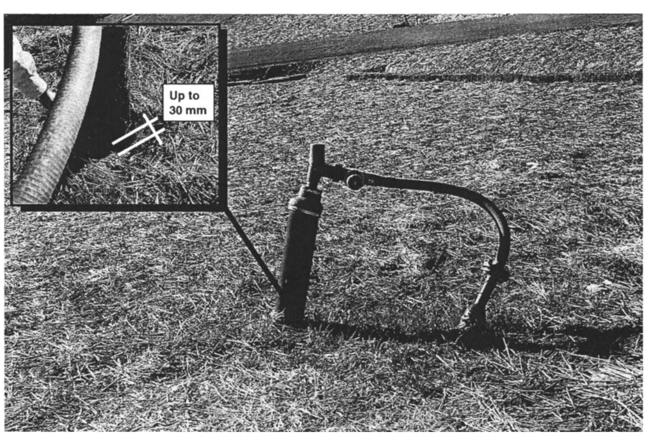

Fig. 4. Landfill gas riser displaced by cover soil movement

Furthermore, unlike surface refraction techniques, SASW can (with certain limits that depend upon depth, layer thickness, and uniformity of layering) detect lower shear wave velocity layers underlying higher shear wave velocity layers. Neither the earthquake shaking nor the 2.5 year lag between the earthquake and the SASW measurements were considered to have substantially influenced the shear wave velocity of the waste mass due to the minimal settlement (i.e., waste densification) associated with these events. The SASW survey was conducted by University of Texas personnel as part of National Science Foundation-sponsored research of the static and dynamic properties on municipal solid waste.

The results of the site-specific SASW measurements conducted on top of each of the landfill areas covered by a composite cover are presented in Fig. 5. Fig. 5 shows a relatively uniform shear wave velocity profile in all three areas below a depth of approximately 3 m. While the thickness of the cover systems illustrated in Fig. 2 is between 0.6 m and 1.2 m, these cover systems are underlain by a compacted foundation layer at least 0.45 m thick and an indeterminate thickness of interim cover soil. The SASW profiles indicate the combined thickness of the interim soil cover/foundation layer/composite final cover cap is approximately 3 m at the locations where SASW testing was conducted. Consistent with the landfill construction records (e.g., GeoSyntec Consultants 2003), the results of the SASW measurements indicate that the waste mass in the vicinity of the SASW surveys was on the order of 30 m thick. Despite the rather coarse discretization of the waste profile into just two layers, the results of the SASW measurements further indicate that the shear wave velocity of the OVSL waste mass is stress dependent, as it is greater at depth (within the second waste layer) than near the surface (within the first waste layer) in all three profiles.

Construction quality assurance (CQA) interface shear strength conformance testing was conducted during the construction of all three composite landfill covers (GeoSyntec Consultants 1997). The interface direct shear testing was conducted in accordance with ASTM D 5321 (ASTM 1992) for each cover system.

Individual composite cover components were tested for cover Types A and C, while "sandwich" testing was performed for cover Type B. In all three sets of CQA tests, the composite cover interfaces were sprayed with water, subjected to three different normal stresses (up to a maximum of 35 kPa) and sheared at a constant rate of 1 mm/min. Results of the interface shear strength were interpreted in terms of apparent friction angle, the calculated friction angle for a normal stress of 10 kPa (considered representative of the normal stress at the geomembrane interface) assuming that the cohesion was zero. As discussed subsequently, interface behavior was assumed to be governed by the large displacement interface friction angle, defined in accordance with ASTM D 5321 (ASTM 1992) as the interface friction evaluated using the shearing resistance at a lateral deformation of 75 mm. The results of the interface direct shear testing indicated that the geocomposite/geomembrane interface was the weakest interface in all three of the composite landfill covers present at the site at the time of the earthquake. The lowest large-displacement apparent friction angle for the three cover systems, 28°, was found for cover Type C and applied to the interface between the bottom of the textured geomembrane liner and the nonwoven geotextile of the GCL (GeoSyntec Consultants 2003). The textured geomembrane/double-sided drainage geocomposite interface yielded the lowest large-displacement apparent friction angle for cover Types A and B, with a value of 33° for both of these covers, based upon CQA direct shear tests conducted using the actual materials procured for construction (GeoSyntec Consultants 1997).

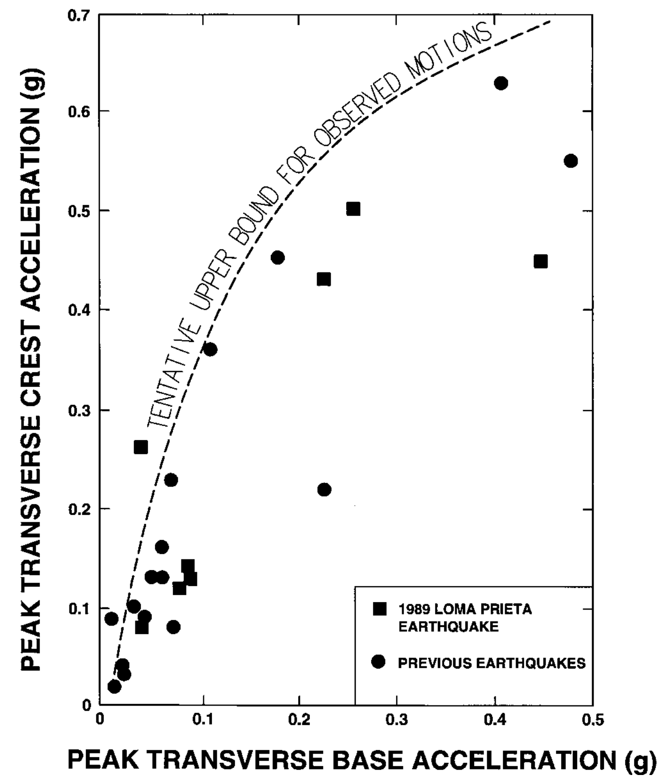

Fig. 6. Approximate relationship between peak base acceleration and peak crest acceleration at the crests of earthen dams [transverse direction] by Harder (1991), with permission

Analysis

The performance of the OVSL composite landfill cover system subject to the Nisqually earthquake was initially evaluated using two simple chart solutions outlined in the United States Environmental Protection Agency (EPA) guidance document on seismic design of solid waste landfills (Richardson et al. 1995). These methods are hereafter referred to as Methods 1 and 2. The seismic performance of the cover was also evaluated using a more rigorous screening procedure proposed by Bray et al. (1998) and a conventional decoupled equivalent-linear site response/Newmark-type (Newmark 1965) permanent seismic deformation analysis. These methods are referred to herein as Methods 3 and 4, respectively. All four methods followed the general three-step procedure outlined below.

1. Evaluate the yield acceleration (k_y) for the cover veneer for each of the three composite landfill cover configurations;
2. Evaluate the peak horizontal acceleration (a_{max}) at the top of the landfill and the peak average acceleration (k_{max}) or average acceleration time history $k(t)$ of the sliding mass representing the cover veneer; and
3. Using k_y and either k_{max} (for Methods 1, 2, and 3) or $k(t)$ (for Method 4), calculate the seismically induced permanent displacement of the cover veneer (u_{max}).

In the first step, it was assumed that the critical sliding mass followed the interface with the lowest peak shear strength for each of the cover configurations shown in Fig. 2. In accordance with current design practice, the evaluation of k_y was based upon the large displacement interface shear strength for the interface with the lowest peak shear strength to provide an upper bound on seismically induced displacement [see Matasovic et al. (1998b) for a discussion on the influence of this assumption on calculated seismic displacements]. Large displacement interface shear strengths were established based upon the CQA conformance testing results, as discussed previously. The yield accelerations were calculated using the infinite slope closed-form solution developed by Matasovic (1991) (see also Richardson et al. 1995). The results of the calculations indicate that k_y ranged from 0.17 g (cover Type C) to 0.22 g (cover Types A and B).

In the second step for Methods 1 and 2, based upon the work of Bray and Rathje (1998), a_{max} was estimated using the Harder (1991) chart, presented in Fig. 6, relating PHGA at the base of an embankment to the upper bound of the crest acceleration (transverse direction) as measured during earthquakes at several California earthen dam sites. Using one-dimensional response analyses, Bray and Rathje (1998) showed that the Harder (1991) relationship also provided a reasonable upper bound for amplification (i.e., for the relationship between the free-field PHGA and a_{max}) at solid waste landfills for PHGA values up to at least 0.4 g. The resulting value of a_{max} was then assumed to be equal to k_{max} and used along with k_y to calculate permanent seismic displacements using the charts developed by Hynes and Franklin (1984) for Method 1 and Makdisi and Seed (1978) for Method 2. For Method 1, as per the recommendation by Richardson et al. (1995) for landfill covers, the mean plus one standard deviation curve was employed. For Method 2, the median and upper bound curves for M_w 6.5 were both used in the analysis.

For Method 3, a_{max} was evaluated using the chart developed by Bray et al. (1998)—relating the ratio of the maximum horizontal acceleration at the top of the landfill divided by the PHGA times a nonlinear response factor to the ratio of the fundamental period of the waste to the mean period of the earthquake. The mean period of the "design" earthquake was evaluated based on the 65 km epicentral distance and 6.8 moment magnitude of

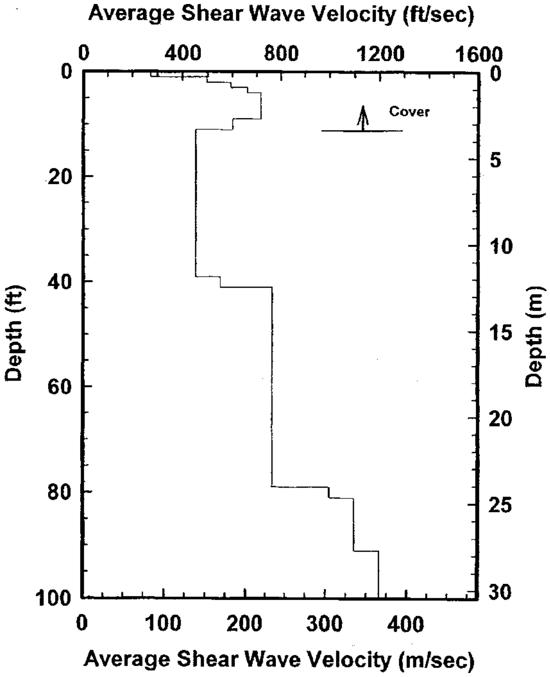

Fig. 7. Average shear wave velocity profile of OVSL site

the design event using the relationship provided by Bray et al. (1998). The fundamental period of the waste mass, $T_{s\text{-waste}}$, was calculated using the thickness, H (=30 m), and average shear wave velocity, V_s (=220 m s), of the "representative" landfill column using the simple one-dimensional relationship $T_{s\text{-waste}}=4H/V_s$. Cover displacement is then related to the ratio of k_y/k_{max} as a function of earthquake magnitude. Because the PHGA recorded at the KIMR station was closer to the median plus one standard deviation ("16% probability of exceedance") value than the mean value predicted from Abrahamson and Silva attenuation relationship employed by Bray et al. (1998), both median and 16% probability of exceedance curves presented by Bray et al. (1998) were used for the calculations employing Method 3.

For Method 4, a formal seismic site response analysis was conducted using the *SHAKE91* computer program (Schnabel et al. 1972; Idriss and Sun 1992) to evaluate a_{max} and $k(t)$. The *SHAKE91* site response analysis employed the average shear wave velocity profile developed from the SASW results, shown in Fig. 7. The unit weight profile was developed from an average shear wave velocity profile using the following equation relating unit weight to shear wave velocity of municipal solid waste based upon in situ tests at the Azusa landfill in southern California (Kavazanjian et al. 1996):

$$\gamma_t = 6.32 + 0.0413\ V_s - 0.0518\ z \qquad (1)$$

where γ_t (in kN m^3) and V_s (in m s)=total unit weight and shear wave velocity of waste mass, respectively, and z (in m)=depth below the ground surface.

The total municipal solid waste unit weight profile established using Eq. (1) varies from 12 kN m^3 at a depth of 4 m to 15 kN m^3 at a depth of 20 m. This profile is very similar to the "typical" solid waste unit weight profile developed by Kavazanjian et al. (1996). This is not surprising, as the shear wave velocity profile of the OVSL waste mass was similar to the waste mass shear wave velocity profile at the Azusa landfill.

The solid waste modulus reduction and damping curves used in the *SHAKE91* analysis were the "recommended" modulus reduction and damping curves developed by Matasovic and Kavazanjian (1998) based upon laboratory testing and back-analysis of recorded accelerograms for the OII landfill Superfund site. The OII data provide the only available basis for evaluating modulus reduction and damping of solid waste, and several investigators have backcalculated modulus reduction and damping curves from the OII accelerograms. However, because only Matasovic and Kavazanjian (1998) had access to large strain cyclic laboratory testing, that could be used to extend the back-analysis data beyond a cyclic shear strain of 0.1 percent, these curves were considered to be most-reliable and were used in the analysis described herein. As the available relationships are relatively similar in the small shear strain range associated with the backanalysis, and as the relatively moderate PHGA of the earthquake did not mobilize very large shear strains in the waste mass, the choice of modulus reduction and damping curves probably does not have a significant effect on the results of the analysis. The Vucetic and Dobry (1991) modulus reduction and damping curves, for a plasticity index of 30, were used for dynamic characterization of compacted cover soil materials. The Shibuya et al. (1990) modulus reduction and damping curves for gravel were assigned to the dense glacial till foundation materials beneath the landfill.

In the *SHAKE91* analyses (Method 4), both the KIMR-NS and KIMR-EW accelerograms were applied as outcrop motions. A representative 30 m high column of landfill material (waste and cover) was employed in the *SHAKE91* analysis to model landfill response. The 5% damped acceleration response spectra at the landfill surface and the average acceleration time history, $k(t)$, of the 0.6 m thick vegetative cover soil veneer from the *SHAKE91* analyses, were employed as the "indicators" of the response of the landfill cover system. The average acceleration time histories $[k(t)]$ were calculated from the earthquake-induced shear stress at the base of the vegetative cover soil veneer.

The acceleration response spectra at the top of the landfill were compared to the response spectra for the free-field input motions to evaluate the amplification of earthquake motions at the top of the landfill. Excursions of the average acceleration time histories above the yield acceleration (k_y=0.17 g only) were double integrated in a Newmark-type deformation analysis to evaluate the calculated permanent seismic displacements of the vegetative cover soil veneer. The complete average acceleration time histories at the base of the cover were also double integrated to calculate maximum transient displacement of the cover veneer (d_{max}).

Fig. 8 compares the acceleration response spectrum of average acceleration time history of the 0.6 m cover veneer to the acceleration response spectrum of the input free-field time history (east-west direction). This figure clearly shows the amplification of spectral acceleration around the fundamental period of the waste mass of 0.54 s calculated from the SASW results. The spectral amplification factor at the fundamental period is on the order of 3. Fig. 8 also shows slight amplification of the PHGA (the spectral acceleration at zero period) from 0.16 to 0.19 g, and suppression (attenuation) of the spectral acceleration around the predominant period of the input motion (in the vicinity of 0.1 s).

Results of the four methods of analysis with respect to a_{max}, k_{max}, and u_{max} are summarized in Table 1. The values of d_{max} calculated in the Method 4 analyses were 35 mm (KIMR-EW, the record with a significant duration of 13.4 s) and 55 mm (KIMR-NS, the record with a significant duration of 22.1 s). Note that the a_{max} and k_{max} values listed in Table 1 are identical because the vegetative cover soil veneer (the soil above the geomembrane) is relatively thin (0.6 m).

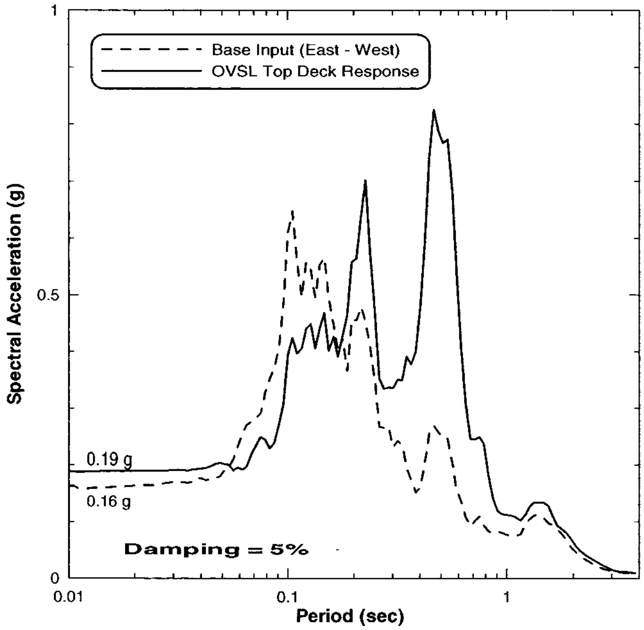

Fig. 8. Base input and OVSL landfill top deck response to Nisqually earthquake in east-west direction

Interpretation of Results

The a_{max} value for Methods 1 and 2 (0.47 g), calculated using the Harder (1991) chart, significantly overestimates the response of the OVSL landfill for this case. This is not surprising, as the predominant period of the input motion (approximately 0.1 s, as shown by the input response spectra in Fig. 3) was significantly different from the predominant period of the landfill mass (approximately 0.54 s, as calculated based upon both $4H/V_s$ and the *SHAKE91* response analysis). As the Harder (1991) chart presents an upper bound on the acceleration at the top of an earth dam or landfill, it implicitly assumes a resonant or near-resonant condition. Values of a_{max} from Method 3 (0.28–0.34 g), the Bray et al. (1998) method which explicitly includes consideration of the relationship between the predominant period of the earthquake motion and the fundamental (resonant) period of the waste mass, were closer to the values computed in Method 4, the *SHAKE91* site-specific response analysis. However, the Method 3 values were still significantly greater than the Method 4 values. The conservatism in Methods 1, 2, and 3 is neither surprising nor inappropriate given the screening nature of these methods.

The maximum permanent seismic displacements calculated by Methods 1 and 2 using the Hynes and Franklin (1984) (mean plus one standard deviation) curve and the Makdisi and Seed (1978) M_w 6.5 median and upper bound curves are the highest among the four methods, because these two methods employ the highest value of k_{max}. However, while the values calculated using Methods 1 and 2 are not zero, they are still in the range of calculated permanent seismic displacement values generally considered indicative of no damage (i.e., u_{max} no greater than 150 to 300 mm; see, e.g., Anderson and Kavazanjian 1995; Kavazanjian et al. 1998). Method 3 (Bray et al. 1998) cover deformation estimates, obtained using the M_w 7 median and 16% probability of exceedance curves, and the Method 4 (decoupled equivalent-linear site response/Newmark-type deformation analysis) values, based upon analyses which account for the both the predominant period of the earthquake and the fundamental period of the landfill, are lower than those from Methods 1 and 2 and are in good agreement with observed behavior (i.e., no observable permanent seismic deformation). Furthermore, the range of maximum *transient* displacements of 35 to 55 mm calculated using Method 4 is consistent with the inferred relative movement between the final cover and landfill gas risers of up to 30 mm.

Conclusions

The results of the analyses described in this paper indicate that all of the seismic design methods applied herein would have predicted the satisfactory performance of the OVSL composite landfill cover systems in the Nisqually earthquake. While Methods 1 and 2 (the simple screening methods described in the EPA design guidance document by Richardson et al. 1995) are the most conservative, they still predicted satisfactory performance. As these methods are intended primarily to be used as screening tools, they appear to provide an appropriate and generally conservative means for a rapid preliminary assessment of the seismic performance of composite landfill covers. As appropriate screening tools, Methods 1 and 2 appear to provide conservative results such that, while a calculated deformation of less than 150–300 mm is indicative of satisfactory performance, a calculated deformation greater than 300 mm is not necessarily indicative of unacceptable performance. The more rigorous Method 3 (Bray et al. 1998), that includes consideration of the predominant period of the design earthquake motion and the response characteristics (fundamental period) of the landfill, provides a more refined, though in this case still conservative, estimate of the calculated

Table 1. Summary of Seismic Site Response and Deformation Analysis

Method	Bedrock PHGA (g)	a_{max} (g)	k_{max} (g)	u_{max} (mm) (k_y=0.17 g)	u_{max} (mm) (k_y=0.22 g)
1 (EPA w / H & F Charts)	0.16	0.47	0.47	100[a]	<100[a]
2 (EPA w / M & S Charts)	0.16	0.47	0.47	100–230[b]	50–130[b]
3 (Bray et al. 1998)	0.16	0.28–0.34	0.28–0.34	30–130[c]	6–40[c]
4 (*SHAKE91*)	0.15, 0.16	0.18, 0.19	0.18, 0.19	<1[d]	0

Note: H & F=Hynes and Franklin (1984);
M & S=Makdisi and Seed (1978).

[a]Mean plus one standard deviation curve.

[b]Mean and upper bound for the M_w 6.5 chart.

[c]Median and 16% probability of exceedance; M_w 7.0.

[d]Largest calculated *permanent* seismic displacement of cover veneer (both accelerograms).

seismic deformation. The "cost" of this Method 3 "refinement" is additional computational effort.

For the moderate level of ground motion for the case history presented herein, the formal decoupled seismic site response/deformation analysis using a combination of measured material properties (shear wave velocity and interface shear strength), material properties calculated by correlation (unit weight), and assumed material properties (modulus reduction and damping) resulted in good agreement between the observed and calculated seismic performance of the OVSL composite landfill cover (i.e., good agreement between the calculated maximum permanent displacement and the observed satisfactory performance of the composite final cover system and between the calculated maximum transient deformation and the observed "gapping" at the contact between the landfill cover and the landfill gas risers).

It should be noted that the predominant period of the input earthquake motions (approximately 0.1 s, based on Fig. 3) was significantly different from the fundamental period of the waste mass (0.54 s), such that the potential for resonant phenomenon (e.g., significant amplification of the PHGA at the top of the landfill) is not captured by this case history. In situations where the predominant period of the earthquake approaches the fundamental period of the waste column, a resonant condition may develop. Under such conditions, the accelerations and deformations calculated using Method 3 (Bray et al. 1998) and Method 4 (*SHAKE91*) may be much closer to the values calculated using Methods 1 and 2 and the results of Methods 1 and 2 may not be as conservative as they were for the case history presented herein.

Acknowledgments

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Notation

The following symbols are used in this paper:

$a_{\max}$ = peak horizontal acceleration at the top of the landfill (g);
$d_{\max}$ = seismically induced *transient* displacement of the cover veneer (mm);
$k_{\max}$ = peak average acceleration of the sliding mass (g);
$k(t)$ = average acceleration time history of the sliding mass (g);
k_y = yield acceleration (g);
M_w = moment magnitude (-)
$u_{\max}$ = seismically induced *permanent* displacement of the cover veneer (mm);
V_s = average shear wave velocity of waste mass (m/s);
z = depth below ground surface (m); and
γ_t = total unit weight of waste mass (kN/m^3).

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