06 Apr 1995, 1:30 pm - 3:00 pm

Seismic Design of Landfills for NE United States

C. Soydemir
Haley & Aldrich. Inc., Cambridge, Massachusetts

Proceedings: Third International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, April 2-7, 1995, Volume I, St. Louis, Missouri

Seismic Design of Landfills for NE United States

Paper No. 6.05

C. Soydemir
Vice President, Haley & Aldrich, Inc., Cambridge, Massachusetts

SYNOPSIS Northeastern U.S. seismicity is briefly discussed and design earthquake is established for landfill projects in accordance with current federal regulations. Methods previously developed for embankment dams are reviewed relative to the seismic stability evaluation of solid waste landfills. Charts were developed to expedite bottom liner and cover system selection while meeting particular seismic design requirements.

INTRODUCTION

Recently promulgated U.S. Federal Environmental Protection Agency Subtitle D (1991) regulations specify that functional integrity of the waste containment components, including liners and covers of the newly constructed solid waste landfills as well as lateral expansion of existing landfills shall be maintained against destabilizing effects of earthquakes, if they are located in a "seismic impact zone". Some states (e.g., Massachusetts) extend this requirement to include the vertical expansions as well. "Seismic impact zones" are defined as those areas where the horizontal acceleration in lithified earth (rock) has a probability of 10% or greater to exceed a threshold level of 0.1 g during a period of 250 years. It is further specified that the 250-year map, or map MF-2120, prepared by Algermissen et al. (1990) shall be used in the designation of an area as a "seismic impact zone", or alternatively a site specific seismic risk assessment shall be conducted for the particular project site. In the former case, the horizontal acceleration in rock as indicated in map MF-2120 would be considered in meeting the seismic design requirements. The portion of the map MF-2120 covering the northeastern United States (NEUS) is reproduced in Figure 1. It is observed that almost all New England States, New Jersey and major portion of New York and Pennsylvania are classified as "seismic impact zones".

Seismic design evaluation of solid waste landfills currently follow the methods and procedures previously developed for embankment dams. However, there are some key differences between the make-up of these two particular structures, which to a large extent govern their respective performances under static and earthquake induced loading conditions. The major difference is that the critical liner and cover components of the landfill structures are relatively less tolerant to seismically induced permanent displacements because of the physical and mechanical limitations of the geosynthetic elements (i.e., geomembranes, geotextiles, geonets, etc.) which are to be incorporated in these components to meet important environmental design requirements.

The paper reviews the currently practiced methodologies for seismic design of solid waste landfills with particular reference to the NEUS. Charts developed as design tools are provided to allow the expeditious evaluation of a range of proposed bottom liner and cover systems which may be considered by the designer.

REGIONAL SEISMICITY AND DESIGN EARTHQUAKE

NEUS is located in an intraplate zone of low to moderate seismicity, distant from any plate boundary and without direct identifiable source mechanisms. Recently Adams et al. (1994) noted that the entire Atlantic Margin, including the continental shelf and slope off the eastern United States, contain relatively young (under 500 million years) rift-margin faults formed due to gravitative settling during the opening of the Atlantic, and proposed that earthquakes in the stable continental regions occur due to the reactivation of these rift-faults which break the integrity of the continental crust in the present compressive state. Adams et al. also indicated that these strike-slip and thrust fault activities are characteristically hidden (or blind), where the 1989 M6.3 Ungava, Quebec earthquake produced the first historical surface rupturing in Eastern North America (ENA).

Atkinson and Boore (1990) based on the recently obtained ENA strong motion records observed that ENA earthquakes contain more energy at high frequencies than the western events. Specifically, they reported that the western North America (WNA) earthquakes exhibit a high frequency cut-off level at 10 to 15 Hz, whereas for ENA this is at about 40 Hz or even greater. Atkinson and Boore further noted that since peak ground accelerations increase with high frequency content, relatively higher accelerations would be encountered in ENA than WNA for records at the same magnitude and distance. On this matter, however, Adams et al. (1994) indicated that the eastern earthquakes lack low frequency energy and have a shorter duration, and thus would have a relatively lower damage potential. Within the context of this study, it is pertinent to note that landfill structures typically are associated with low natural frequencies.

In compliance with the Subtitle D (1991) regulations, Figure 1 has been adopted in defining a design acceleration in base rock as discussed in the Introduction. Values of 0.17 g and 0.34 g were selected from Figure 1, approximately representing the "low" and "high" values of base rock design accelerations for the NEUS region. Also, for the design earthquake a predominant frequency range of 5 Hz to 50 Hz has

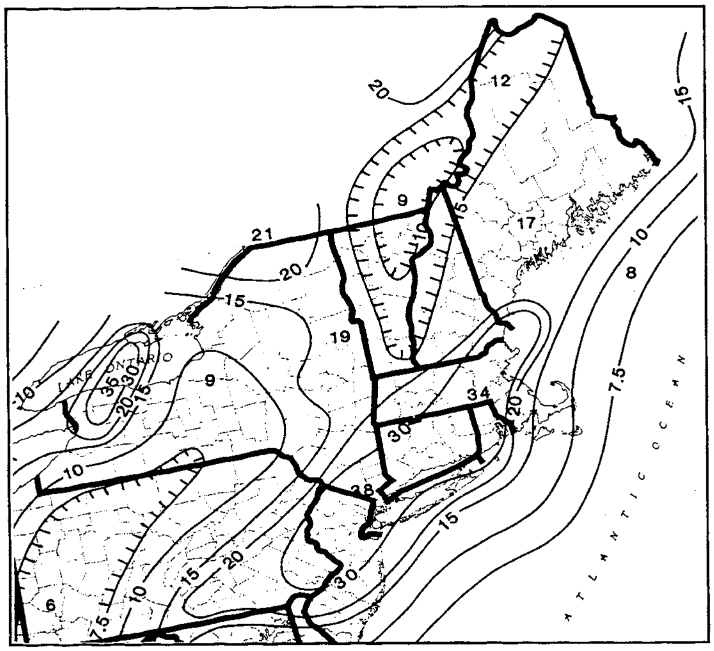

Fig. 1. Horizontal Acceleration in Rock (in g) with 90% Probability of Not Being Exceeded in 250 years (After Algermissen et al., 1990)

been considered. Finally, a M6.5 earthquake was adopted based on the historical regional seismicity. Significance of the earth-quake magnitude is that it establishes the duration of the ground shaking.

SEISMIC STABILITY EVALUATION OF LANDFILLS

The procedures currently used for seismic stability evaluation of solid waste landfills follow the approach developed earlier for embankment dams, namely: a) force (moment) equilibrium or pseudo-static seismic coefficient methods (Seed, 1979), and b) permanent displacement evaluation based on Newmark's (1965) sliding block analogy (Makdisi and Seed, 1978; Hynes and Franklin, 1984). As indicated in the Introduction, since stability consider-ations in landfill structures are intimately connected with the bottom liner and cover components, the discussion presented herein would follow that order.

Bottom Liner Stability

Subsequent to the Kettleman Hills, California, landfill stability failure (Mitchell et al., 1990), it has been well established that instability by sliding along a liner (Figure 2) is a major issue to be considered in landfill design. This mode of instability involves the sliding of a wedge or block of the waste fill along the bottom and side liners due to the mobilization of relatively low frictional resistances (i.e., interface friction angles) between the geosynthetic to geosynthetic and/or geosynthetic to soil interfaces.

In conducting a pseudo-static, limit equilibrium seismic stability analysis, the crucial step is the determination of the appropriate seismic coefficient which correctly represents the destabilizing effect of the ground shaking as an equivalent statically applied horizontal inertia force acting on the potential sliding mass.

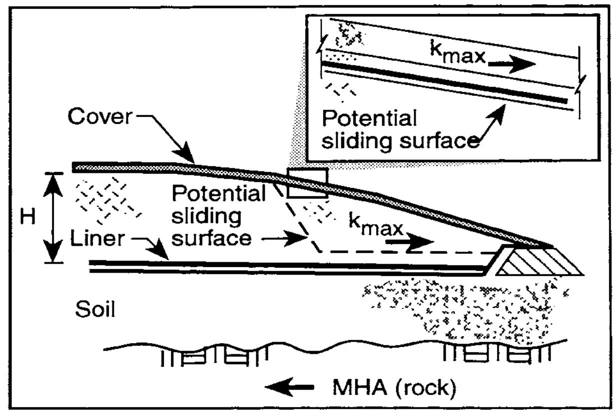

Fig. 2. Common Modes of Instability for Solid Waste Landfills

Accordingly, seismic coefficient times the gravitational acceleration may be considered as a destabilizing horizontal equivalent acceler-ation, HEA (Bray et al., 1993). Similarly, maximum horizontal equivalent acceleration, MHEA, determines the maximum horizontal destabilizing (inertia) force. MHEA usually occurs only once during the duration of an earthquake and acts only for a brief instant. Thus, MHEA divided by the gravitational acceler-ation would conservatively correspond to the seismic coefficient, k_{max}, for seismic stability analysis.

Bray et al. (1993) and Richardson et al. (1994) suggested that if the dynamic shear resistance of the potential sliding mass is at least equal to the maximum destabilizing force induced by MHEA (i.e., the pseudo-static factor of safety, FOS, is equal to unity or greater) the mass under consideration would not undergo significant permanent displacement during the ground shaking. Newmark (1965) proposed that the dynamic shear resistance just sustaining the maximum destabilizing force induced by MHEA (i.e., FOS is equal unity) can be represented by a yield acceleration, $k_y g$, where k_y is defined as the yield acceleration coefficient.

Bray et al. (1993) conducted a systematic study considering a great number of input rock motions, a wide range of waste fill configu-rations and properties, and foundation (subsoil) profiles and calculated the MHEA (or $k_{max}g$) at the base of the waste fill where the bottom liner is located. The results of this study are reproduced in Figure 3, in which MHEA is normalized with respect to the maximum horizontal acceleration in base rock (MHA, rock), and the natural period of the waste fill (T_s-waste) is normalized with respect to the dominant period of the base rock acceleration (T_p-eq) record. (T_s-waste) may be approximated by $(4H/v_s)$, where H is the height, and v_s is the average shear wave velocity of the waste fill.

Figure 3 would be used to determine the seismic coefficient, k_{max}, or (MHEA/g) to be used in a pseudo-static analysis for the mode of potential sliding along the bottom liner. For this mode of instability, Richardson et al. (1994) suggested to take k_{max} as 0.5(MHA, rock) in preliminary design analysis, which is equivalent to assume that (T_s-waste/T_p-eq) ratio in Figure 3 would be typically about two (2.0) or greater.

Makdisi and Seed (1978) considered a range of embankment dams between 75 and 150 ft. in height with varying slopes and compacted fill (earth) materials. These structures were subjected to

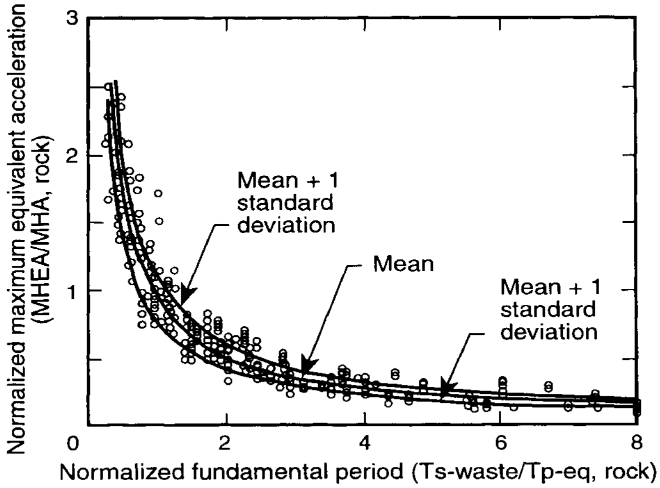

Fig. 3. Normalized Max. Horizontal Equivalent Acceleration vs. Normalized Fundamental Period of Waste Fill for Instability through Bottom Liner (After Bray et al., 1993)

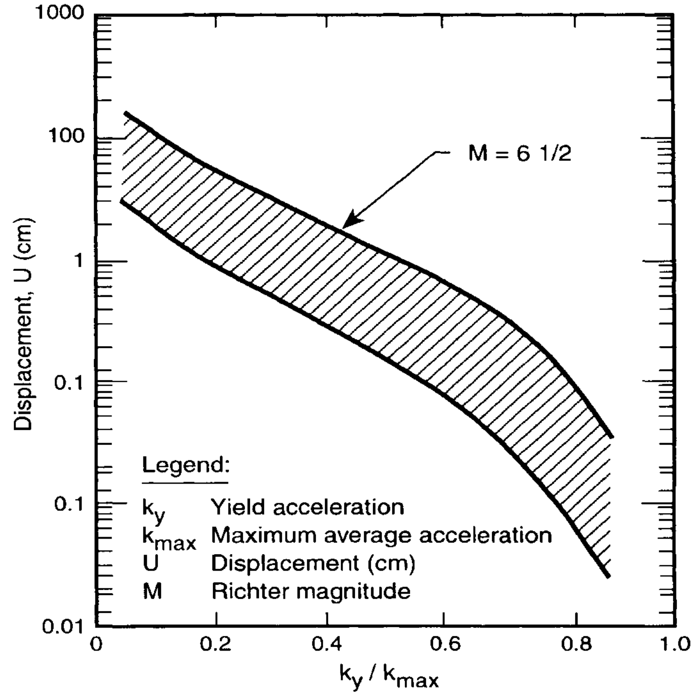

Fig. 4. Variation of Permanent Displacement with Yield Acceleration (After Makdisi and Seed, 1978)

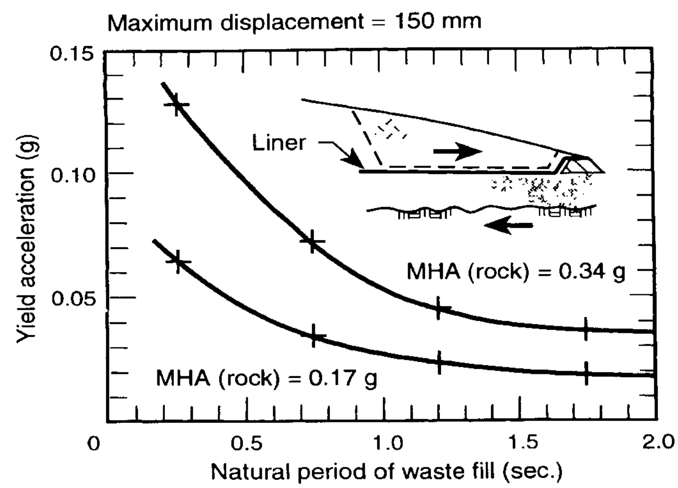

Fig. 5. Variation of Yield Acceleration vs. Natural Period of Waste Fill for Instability through Bottom Liner

five M6.5, three M7.5 and four M8.25 earthquake (base rock) records. For these conditions Makdisi and Seed calculated the dynamic response of the embankments by time-step finite element analysis using the equivalent linear method and obtained for each case time histories of acceleration; a) at the crest, and b) for a potential sliding mass extending through almost the full height of the embankment (i.e., average acceleration). Subsequently, Makdisi and Seed calculated for each time history of acceleration permanent displacements for a range of yield accelerations using the Newmark's (1965) double integration approach. The results (i.e., permanent displacement vs. k_y/k_{max}) were presented with upper and lower bound envelopes for the M6.5, M7.5 and M8.25 earthquakes. Makdisi and Seed's results for the M6.5 earthquakes alone are reproduced in Figure 4 since they pertain to the design earthquake considered in this study.

For the determination of permanent displacement from Figure 4, Makdisi and Seed (1978) introduced a procedure to obtain (k_{max}) for the particular potential sliding mass within the embankment, which requires a knowledge of the acceleration response spectrum (spectral acceleration) for the particular design earthquake. An alternative simplified approach would be to obtain (k_{max}) or (MHEA/g) from Figure 3, and enter into Figure 4 with the obtained (k_{max}) value to estimate the permanent displacement. In using Figure 4, the respective yield acceleration ($k_y g$) value for the particular potential sliding mass being considered would be determined by conventional slope stability methods (e.g., XSTABL computer code). This approach was followed in developing Figure 5 which allows the design engineer to establish the required minimum yield acceleration in order to keep the permanent displacement at or below a level of 150 mm, a magnitude currently considered acceptable (Seed and Bonaparte, 1992) for the bottom liners. In Figure 5 yield accelerations corresponding to (MHA, rock) values between 0.17 g and 0.34 g may be obtained by linear interpolation.

Similar to the Makdisi and Seed (1978) approach, Hynes and Franklin (1984) at the U.S. Army Waterways Experiment Station conducted a more comprehensive study by using the horizontal components of 348 strong motion records obtained primarily from California earthquakes with particular bias to the 1971 San Fernando Earthquake and six synthetic accelerograms. The records were to a large extent from soil sites (i.e., alluvial and deposits of intermediate stiffness) as opposed to base rock motions. The earthquakes ranged between M5.3 and M7.7 events, however, Hynes and Franklin did not separate the data in magnitude groups.

Hynes and Franklin (1984) calculated permanent displacement values by double integration (Newmark, 1965) of the strong motion records for three levels of (k_y/k_{max}), 0.02, 0.1 and 0.5, and presented the results in the form of three curves; mean, mean plus one standard deviation and upper bound as reproduced in Figure 6. In Figure 6, two other inclusions were made by the author for direct comparison; these are; a) the upper bound curve of the Makdisi and Seed (1978) M6.5 relationship from Figure 4, and b) mean and upper bound curves established by the author by considering only those data points of Hynes and Franklin, for the range of M6.0 to M6.5 events. It is observed in Figure 6 that if the Hynes and Franklin (1984) curves are used for M6.0 to M6.5 events they would produce "conservative" permanent displacement magnitudes. This may be quite appropriate for the case of embankment dams for which an order of magnitude in estimated permanent displacements would suffice in design (Makdisi and Seed, 1978); however, the same "conservative" approach may lead to serious difficulties in the design of landfills for which relatively smaller displacements are tolerable.

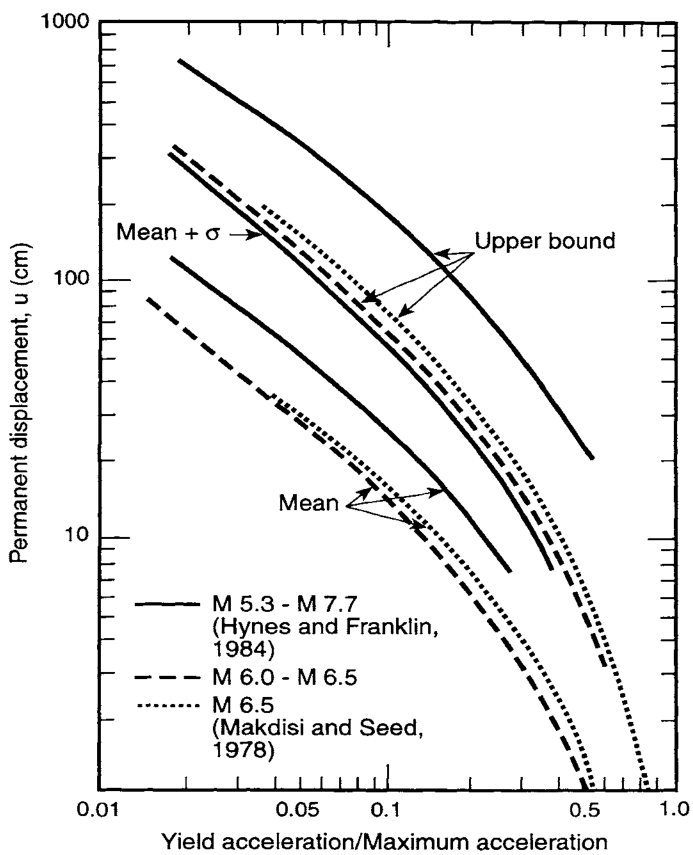

Fig. 6. Variation of Permanent Displacement with Yield Acceleration (Partially after Hynes and Franklin, 1984)

In Figure 6, "Maximum Acceleration" is the maximum horizontal equivalent acceleration, MHEA (as defined earlier) which destabilizes a particular wedge (block) under consideration. In order to estimate "Maximum Acceleration" Hynes and Franklin (1984) developed Figure 7, providing "Amplification Factors" which are to be multiplied with (MHA, rock) to obtain "Maximum Acceleration". In developing Figure 7, Hynes and Franklin (1984) used 27 strong motion records (base rock), and following the shear beam analogy obtained "Amplification Factors" for embankments ranging widely in height and supported directly on rock as well as on various subsoil strata with different embankment to subsoil stiffness ratios. A damping ratio of 15 to 20 percent was used in the analyses.

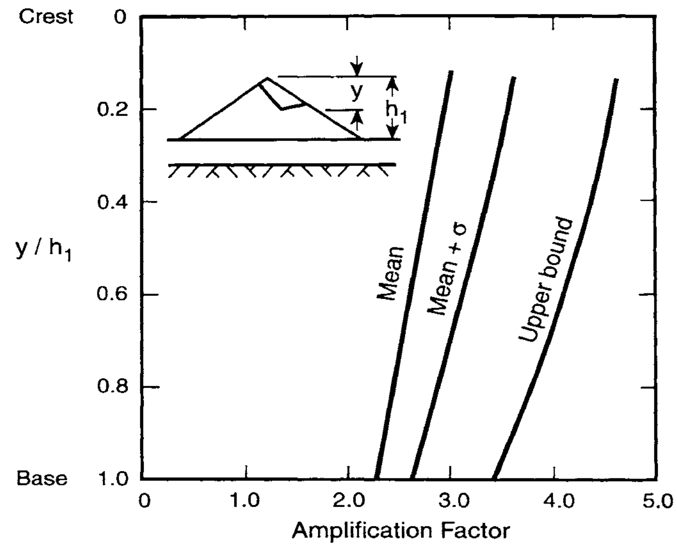

Fig. 7. Amplification Factors for Linearly Viscoelastic Embankments at Resonance (After Hynes and Franklin, 1984)

Finally, Figure 7 was generated by taking the computed resonant response values to represent the amplification effects.

Figure 8 was developed by the author relative to the bottom liner stability analysis, utilizing Figure 7 and the mean plus one standard deviation curve of Hynes and Franklin (1984) given in Figure 6 since it reasonably represents the upper bound curve for the M6.0 and M6.5 events. Again, a limiting value of 150 mm was considered for the permanent displacement. A comparison of Figure 8 with Figure 5 shows general agreement except in the very low natural periods for the waste fill structure. Therefore, both Figures 5 and 8 may be used in bottom liner stability analysis for the particular NEUS design earthquake.

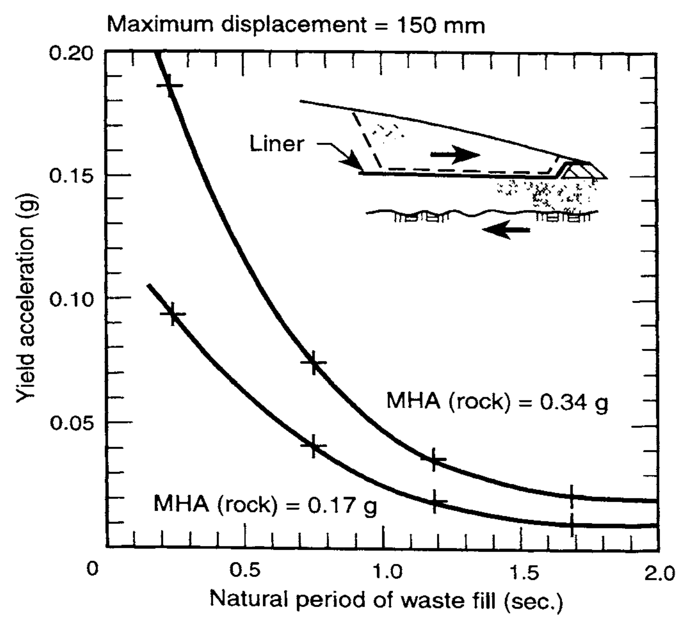

Fig. 8. Variation of Yield Acceleration vs. Natural Period of Waste fill for Instability through Bottom Liner

Cover Liner Stability

In accordance with the Subtitle D (1991) regulations, seismic stability requirements of the cover liners are to be considered as well as the bottom liners. The typical mode of instability in cover liners would be by sliding along the least resistant interface, typically involving a goesynthetic unit as schematically depicted in Figure 2. This failure mechanism may be closely represented by an "infinite slope" model. Matasovic (1991) developed the following formulation for factor of safety against sliding, FOS, for an "infinite slope" under seismic loading:

$$FOS=(\tan\phi - k_{max}\tan\beta .\ \tan\phi)/(k_{max}+\tan\beta) \quad (1)$$

where the shear resistance along the sliding plane has no cohesion component, and there is insignificant seepage flow through the cover element; ϕ = friction angle along the least resistant interface, β = the slope angle, and k_{max} = the seismic coefficient.

In determining k_{max} which is equivalent to the maximum destabilizing horizontal acceleration at the "crest" level, amplification of the base ground acceleration through the waste fill is to be established. Richardson et al. (1994) and Kavazanjian and Matasovic (1994) suggested that amplification correlation for soft soil sites proposed by Idriss (1990), which is reproduced in Figure 9, may be applicable for waste fills as well.

Regarding seismically induced permanent displacements for cover liners, Matasovic (1991) provided the following formulation for yield acceleration (k_y g) for the "infinite slope" model under the same assumptions considered in Equation 1:

$$k_y=(\tan\phi - \tan\beta)/(1+\tan\beta\tan\phi) \quad (2)$$

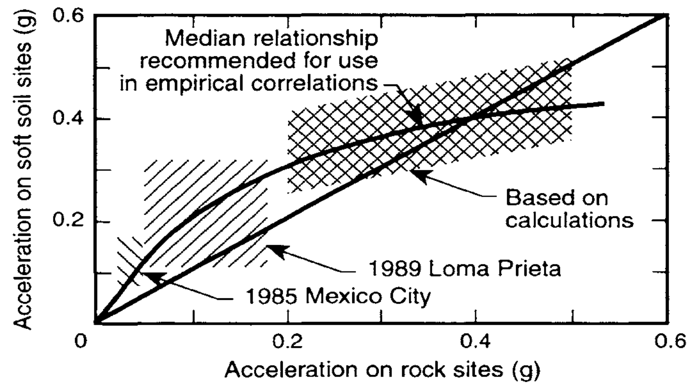

Fig. 9. Relationship Between Maximum Acceleration on Rock and Acceleration on Soft Soil Sites (After Idriss, 1990)

One current argument related to the seismically induced displacements in cover zones is that any surficial distress (e.g., cracking and ruptures) may be treated as a maintenance problem. In general, it would be reasonable to adopt a more tolerant design criteria for the covers than for the bottom liners. A 300 mm limiting displacement has been considered for cover liners in this study.

Yield accelerations for the cover liner design were calculated for the (MHA, rock) range specified for NEUS, and the results are presented in Figure 10. Two approaches were followed in developing Figure 10: a) Makdisi and Seed (1978) correlation from Figure 4 was used where destabilizing accelerations at the "crest" were established by double amplification of (MHA, rock) for the soft soil sites and single amplification for the firm soil sites

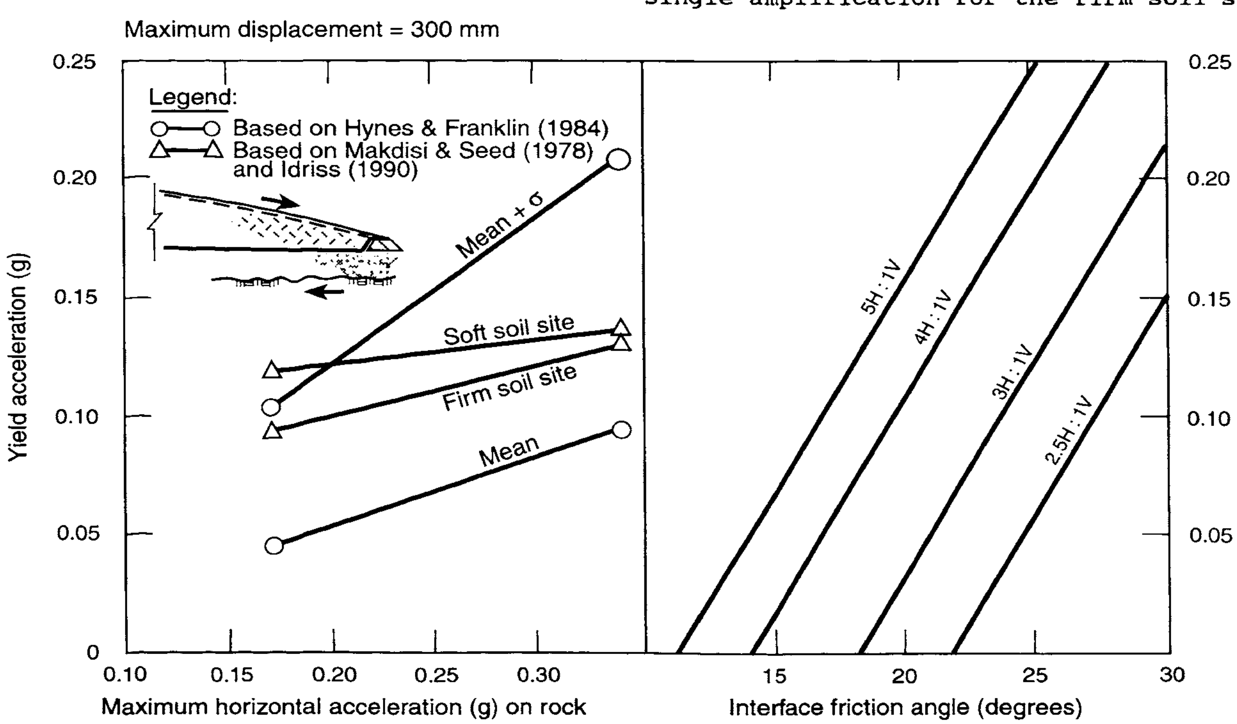

Fig. 10. Relation Between Max. Acceleration on Rock vs. Yield Acceleration and Interface Friction Angle for Instability through Cover Liner

through Figure 9 (Idriss, 1990), b) Hynes and Franklin (1984) mean and mean plus one standard deviation curves (Figure 6) were used. The destabilizing accelerations at the "crest" were established from Figure 7 as recommended by Hynes and Franklin (1984). Figure 10 also provides limiting interface friction angles (Equation 2) corresponding to the calculated yield accelerations for the range of slope angles pertinent to cover liner design.

Figure 10 would be used in seismic design evaluation of cover liners, first by establishing the yield acceleration for the (MHA, rock) obtained from Figure 1 for the particular project site, and subsequently determining the limiting interface friction angle for the proposed slope such that maximum tolerable permanent displacement of 300 mm would be maintained.

CONCLUSIONS

Seismic design evaluation of solid waste landfills as required by the recent U.S.A. Federal EPA, Subtitle D (1991) regulations can be conducted by appropriate application of the methods previously developed for the embankment dams. Potential sliding in solid waste landfills under seismic loading is expected to occur through the relatively less resistant bottom and cover liners which would incorporate various arrangements of geosynthetics. The interface with the least shear resistance, or minimum friction angle, within the bottom and cover liners would usually control the stability.

Charts as design tools were developed for the northeastern United States (NEUS) in conformance with the Subtitle D (1991) regulations for the expeditious evaluation of various bottom and cover liner arrangements considered by the designer to meet the necessary environmental design requirements. These charts indicate that for the NEUS seismicity, the required levels of shear resistance levels to be mobilized by the various types of synthetic elements would likely be provided by the products currently manufactured by the industry.

ACKNOWLEDGEMENT

This study was supported by the Haley & Aldrich, Inc. Professional Development Program. Ms. A. Welch drafted the figures and Ms. C. Catwell prepared the manuscript. Their contributions are greatly appreciated.

REFERENCES

Adams, J., P.W. Basham and S. Halchuk (1994), "Northeastern North American Earthquake Potential - New Challenges for Seismic Hazard Mapping," Proc. Fifth Nat. Conf. on Earthquake Engr., EERI, Chicago.

Algermissen, S.T. and et al. (1990), "Probabilistic Earthquake Acceleration and Velocity Maps for the United States and Puerto Rico," Misc. Field Studies Map MF-2120, U.S. Geol. Survey.

Atkinson, G.M. and D.M. Moore (1990), "Recent Trends in Ground Motion and Spectral Response Relations for North America," Earthquake Spectra, EERI, 2:1:15-35.

Bray, J.D. et al. (1993), "An Overview of Seismic Design Issues for Solid Waste Landfills," Proc. Geosynthetic Res. Inst. Conference, Philadelphia.

Hynes, M.E. and A.G. Franklin (1984), "Rationalizing the Seismic Coefficient Method," Misc. Paper GL-84-13, U.S. Army Eng. WES, 34.

Idriss, I.M. (1990), "Response of Soft Soil Sites During Earthquakes," Proc. H.B. Seed Symposium, Berkeley, California.

Kavazanjian, E. and N. Matasovic (1994), "Seismic Analysis of Solid Waste Landfills" (Draft), GeoSyntec Consultants, Huntington Beach, California.

Makdisi, F.I. and H.B. Seed (1978), "Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations," JGED, ASCE, 104:7, 849-867.

Matasovic, N. (1991), "Selection of Method for Seismic Slope Analysis," Proc. 2nd Int. Conf. Recent Advances in Geotech. Earthquake Eng. Soil Dynamics, St. Louis, 2:1057-1062.

Mitchell, J.K., R.B. Seed and H.B. Seed (1990), "Kettleman Hills Waste Landfill Slope Failure. I: Liner System Properties," JGED, ASCE, 116:4:647-668.

Newmark, N.M. (1965), "Effects of Earthquakes on Dams and Embankments," Geotechnique, 15:2:139-160.

Richardson, G., E. Kavazanjian and N. Matasovic (1994), "RCRA Subtitle-D (258) Seismic Design Guidance for Municipal Solid Waste Landfill Facilities" (Draft), NYS-ASWM and U.S. EPA Seminar, Saratoga Springs, New York.

Seed, R.B. and R. Bonaparte (1992), "Seismic Analysis and Design of Lined Waste Fills: Current Practice," Proc., Stability and Performance of Slopes and Embankments-II, Berkeley, CA, ASCE, 2:1521-1545.

Seed, H.B. (1979)," Considerations in the Earthquake - Resistant Design of Earth and Rockfill Dams," Geotechnique, 29:3:215-263.

U.S. Code of Federal Regulations, Title 40, Subtitle - D (1991), Part 258, "Solid Waste Disposal Criteria".