DISPLACEMENT-BASED ANALYSIS OF WASTE LANDFILL COVER SYSTEMS

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Abstract: Current design procedures of waste landfills include the seismic stability analysis and the post-seismic serviceability evaluation of engineered cover systems. In this paper a modified Newmark-type procedure allowing a performance-based design of these geotechnical systems is presented. Since a cover system generally includes geosynthetic-geosynthetic and soil-geosynthetic interfaces, the proposed approach accounts for the possible strain softening force-displacement curves which may characterize these interfaces when subject to earthquake shearing. Using a set of earthquake records a parametric analysis was performed to detect the influence of the dynamic interface behavior on the displacement response.

Keywords: cover system, displacement analysis, interfaces, earthquake effects.

INTRODUCTION

The seismic design procedure of waste landfill cover systems, for both force-balance and displacement-based approaches, requires the understanding of the behavior of the geosynthetic-geosynthetic and of the soil-geosynthetic interfaces generally involved in these modern engineered systems. Current methods for seismic stability analysis of waste landfills cover systems are based on the limit equilibrium or limit analysis methods and are applied using the pseudo-static approach. In these methods the effects of the transient seismic action, characterised by abrupt changes in modulus and sign, are represented by an equivalent static force that is demanded to represent the overall effect of the earthquake on the slope. This force is typically defined as a percentage of the weight of the potential sliding soil mass, through an equivalent seismic coefficient, k_{eq}, of horizontal and vertical components $k_{h,eq}$ and $k_{v,eq}$, respectively. Seismic stability is measured by a factor of safety F_{psd} that is defined as the ratio of the total resisting force to the total driving force acting on a potential failure surface.

Alternatively, in the displacement-based approach, seismic stability is evaluated by comparing the earthquake-induced displacement with threshold values representing the maximum displacement that can be tolerated by the cover system without reaching an ultimate or a serviceability limit state. The evaluation of the occurrence of a limit state represents a crucial point in the design procedures especially for municipal solid waste landfills, or any other waste containment structure, where geosynthetics are used as impervious barriers along cover and bottom sealing systems.

In the traditional displacement-based approach the earthquake-induced displacement of a potential sliding mass is evaluated using the sliding block analysis, originally outlined by Newmark (1965). The computed permanent displacement is assumed as a measure of the earthquake effect and is compared with a tolerable permanent displacement related to the damage associated with the instability.

Concerning cover systems, only a few studies in the literature attempt to relate earthquake-induced displacements with levels of damage, and definition of maximum tolerable displacements is rarely given by seismic codes. Traditionally, permanent displacements are evaluated assuming a constant yield acceleration value and an only one-way displacement analysis is carried put. However, such assumptions do not appear realistic for cover systems including geosynthetics and a modified Newmark sliding block procedure should be applied.

Concerning the 1-way (down-hill) hypothesis it should be stressed that it can be generally accepted for many practical purposes involving natural slopes or soil structures, such as dams and embankments. For these geotechnical systems, assuming an inclined or circular failure surface, and neglecting any soil shear strength reduction, the value of the yield acceleration for up-hill sliding is rarely exceeded during an earthquake. Thus, up-hill permanent displacements usually do not occur and the one-way sliding hypothesis can be considered satisfactorily.

Conversely, in modern landfills, interfaces involved in the engineered cover system are usually characterized by relative low friction angles. Then, the up-hill yield acceleration is not great enough to ensure that only down-hill sliding occurs during the ground motion. Consequently a two-ways sliding model is generally required to perform a reliable estimation of the earthquake-induced permanent displacement.

Concerning the hypothesis on the constant value of the yield acceleration it should be stressed that experimental tests (Mitchell et al. 1990) revealed that for some interfaces typically used in landfills, the stress strain behavior is related to the acting stress level, to the imposed loading history and to the excitation frequency adopted in the tests. Moreover, cyclic shear strength tests, which are recommended for evaluating interface behavior under seismic excitation (De and Zimmie 1998), show that for different soil-geosynthetic interfaces the stress-strain relationship present the typical shape of a strain softening material. Generally, the measured shear force reaches a peak value and then decreases until residual values of strength are attained for large displacements. The values of displacements for which peak and residual condition are achieved as well as the path followed to reach the residual condition depend on the interface material properties and on the acting stress level. Therefore, the constant yield acceleration assumption does not appear fully satisfactory. A strength degradation model, describing the post-peak behavior of the interfaces, is required to reliably evaluate the permanent displacement response.

In the paper a modified Newmark-type analysis is outlined referring to a previous published paper. The results of a parametric analysis are adopted to describe the influence of the dynamic interface behavior in the computed displacement response of some ideal cover system scheme.

DISPLACEMENT ANALYSIS INCLUDING AM INTERFACE STRENGTH DEFORMATION MODEL

In the traditional sliding block procedure the yield acceleration of the potential failure mass is evaluated as a function of the material and/or interface shear strength. Then, a possible reduction of yield acceleration should be taken into account when sliding occurs. Biondi & Maugeri (2002) proposed a modified Newmark-type analysis which include the possible reduction in the cover system yield acceleration as a consequence of a strain softening force-displacement curves assumed at the interface. In the analysis the infinite slope scheme is assumed. Figure 1a shows the strain softening force-displacement curves assumed in the analysis. The available strength at the considered interface is assumed to be a function of the values of the displacement s occurred along the interface during sliding. The two threshold displacement values s^p and s^r as well as the post-peak degradation path could be easily evaluated using the measured force-displacement curves obtained in direct shear interface tests.

Due to this interface behavior the yield acceleration of the cover system changes during sliding, being a function of the available interface shear strength. The scheme of the yield acceleration degradation path assumed in the analysis, for both up and down-hill sliding, is shown in Figure 1b. The suffix "*p*" and "*r*" respectively denoting the values for peak and residual condition of the yield acceleration for both up and down-hill directions. In the proposed approach the initial values of yield acceleration are evaluated using the traditional pseudo-static approach referring to the peak value of strength parameters. Similarly, the residual values of yield acceleration are obtained using the strength parameters for residual failure envelope.

The expression of the peak value of down-hill $k_c^{p,o}$ and up-hill $k_c^{p,i}$ yield acceleration coefficients as well as the residual values of down-hill $k_c^{r,o}$ and up-hill $k_c^{r,i}$ yield acceleration coefficients are:

$$k_c^{p,o} = \frac{a^p/(\gamma \cdot H \cdot \cos\beta) + \cos\beta \cdot \tan\phi^p - \sin\beta}{\cos\beta^* + \sin\beta^* \cdot \tan\phi^p} \qquad k_c^{p,i} = \frac{a^p/(\gamma \cdot H \cdot \cos\beta) + \cos\beta \cdot \tan\phi^p + \sin\beta}{\cos\beta^* + \sin\beta^* \cdot \tan\phi^p} \qquad [1]$$

$$k_c^{r,o} = \frac{a^r/(\gamma \cdot H \cdot \cos\beta) + \cos\beta \cdot \tan\phi^r - \sin\beta}{\cos\beta^* + \sin\beta^* \cdot \tan\phi^r} \qquad k_c^{r,i} = \frac{a^r/(\gamma \cdot H \cdot \cos\beta) + \cos\beta \cdot \tan\phi^r + \sin\beta}{\cos\beta^* + \sin\beta^* \cdot \tan\phi^r} \qquad [2]$$

where β and H are the angle to the horizontal and the depth of the sliding plane, respectively; a^p, ϕ^p and a^r, ϕ^r are the adhesion and the friction angle of the interface for the peak and residual failure envelope respectively, γ is the unit weight of cover soil over the considered interface and $\beta^*=\beta+\omega$, ω being the inclination assumed for the earthquake induced acceleration.

The considered strength degradation path at the interface (Figure 1) is describe by three parameters: a degradation parameter m, describing the post peak strength degradation path, a displacement ratio $\xi=s^r/s^o$, representing the ratio between the values of displacement for which residual and peak strength values are attained, and a yield acceleration reduction factor $\eta= k_c^r/k_c^p$, representing the ratio between the residual and the peak value of the yield acceleration factors. Using the notation introduced in Figure 1, the relationship between the current yield acceleration, for both up and down-hill directions, and the cumulated permanent displacement s can be written as:

$$\frac{k_c(s)}{k_c{}^p} = \eta + (1-\eta) \cdot \left[1 - \left(\frac{s/s^o - 1}{\xi - 1} \right)^m \right] \qquad [3]$$

Permanent displacement can be computed through a double integration of the dynamic equilibrium equation:

$$\ddot{s} = \left[k(t) - k_c(t) \right] \cdot g \cdot \frac{\cos(\phi - \delta \cdot \beta^*)}{\cos\phi} \qquad [4]$$

where $k(t)$ is the earthquake induced acceleration expressed as a fraction of gravity acceleration g, $k_c(t)$ is the current value of the yield acceleration coefficient and δ assumes the values δ=-1 and δ=1 for up and down-hill sliding, respectively.

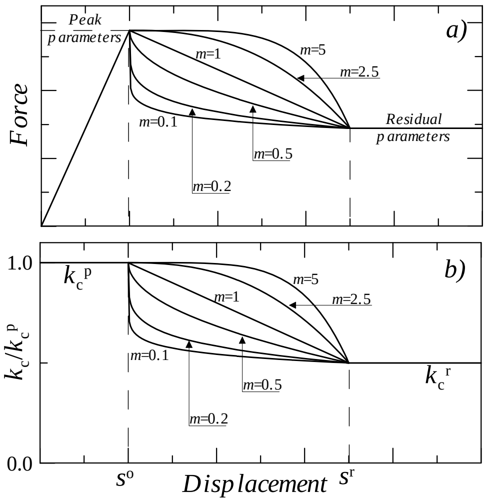

Figure 1. a) Scheme of the strain softening force-displacement curves considered in the model by Biondi & Maugeri (2002); b) corresponding yield acceleration degradation paths.

NUMERICAL ANALYSIS AND DISCUSSION OF RESULTS

Numerical analyses were carried out to using a set of 6 records of earthquakes that occurred in USA, Japan and Turkey with magnitude M ranging from 6.1 to 7.3. Each record consists of two (N-W and E-W) accelerograms whose main characteristics are described in Table 1. In particular, for each accelerograms the peak ground acceleration (*PGA*), velocity (*PGV*) and displacement (*PGD*) are reported together with the values of the total D and the bracketed D_b durations, of the Arias intensity I_a, of the destructiveness potential factor P_D and, finally, of the predominant period T_o. The influence of the interface strength degradation on displacement response will be described with reference to an ideal cover system with 1V:4H inclination (β=14°). In the displacement analysis carried out the following values of the model parameters are also assumed: ϕ^p=20° and a^p=0; a parametric analysis on s^p, m, η and ξ were considered.

Figure 2 shows the displacement response computed for the record #*E* (see Table 1) assuming s^p=0.5cm, η=0.5 and ξ=5 and different values of the degradation parameter m. In the analysis, the values of the yield acceleration for up and down-hill sliding were evaluated for both peak and residual conditions; the obtained results are $k_c^{p,o}$=0.105, $k_c^{p,i}$=0.674, $k_c^{r,o}$=0.052 and $k_c^{r,i}$=0.337. The time histories of the yield acceleration obtained using the proposed approach are superimposed on the considered earthquake records in Figure 2a; the displacement responses, computed for both sides of the accelerogram, are plotted in Figures 2b and 2c. In Figures 2b and 2c the results of a traditional Newmark analysis (*NA*), performed using constant peak (k_c=k_c^p) or residual (k_c=k_c^r) yield acceleration values, are also shown.

The results of the analyses point out the influence of the post-peak degradation path on the computed values of the cumulated permanent displacements. As an example, for m varying in the range 0.1-2, permanent displacements increase by about 75%. Conversely, the permanent displacements computed through the traditional displacement approach (*NA*) can be greatly underestimated using peak strength parameters and may be excessively conservative using the residual ones.

Table 1: Characteristics of the accelerograms selected for the displacement analyses.

Earthquake	M	Record name - Station / Component	*PGA* (g)	*PGV* (m/s)	*PGD* (cm)	*D* (sec)	I_a (m/s)	P_D 10^{-4} ($g \cdot s^3$)	D_b (sec)	T_o (sec)
Morgan Hill (USA 1984)	6.1	#*A* - Gilroy#4/ Gil04270	0.224	19.3	4.33	60	0.72	38.0	13.8	1.15
		#*B* - Gilroy#4/ Gil04360A	0.348	17.4	3.11	60	0.77	37.8	12.7	0.50
Imperial Valley (USA 1979)	6.5	#*C* - El Centro#6/ A-E06-140	0.366	20.8	2.83	20	0.32	1.91	1.62	0.26
		#*D* - El Centro#6/ A-E06-230	0.189	12.1	1.15	20	0.11	0.58	1.34	0.37
Northridge (USA 1994)	6.7	#*E* - Hollywood Storage/ HOL360	0.358	27.5	3.04	40	2.00	26.2	15.3	0.85
		#*F* - Hollywood Storage/ HOL090	0.231	18.3	4.81	40	0.94	16.8	14.8	0.70
Loma Prieta (USA 1989)	7.1	#*G* - Gilroy#7/GMR000	0.226	16.4	2.52	40	0.78	9.3	13.5	0.46
		#*H* - Gilroy#7/GMR090	0.323	16.6	3.26	40	0.84	12.7	10.7	0.47
Kobe (Japan 1985)	7.2	#*I* - KJM/KJM000	0.821	81.3	17.7	50	8.39	230.9	21.4	0.68
		#*J* - KJM/KJM090	0.599	74.3	19.9	50	5.43	178.8	17.2	0.70
Ducze (Turkey 1999)	7.3	#*K* - Ducze /DCZ180	0.348	60.0	42.1	26	2.69	138.9	16.1	0.43
		#*L* - Ducze /DCZ270	0.535	83.5	51.6	26	2.93	148.4	14.1	1.29

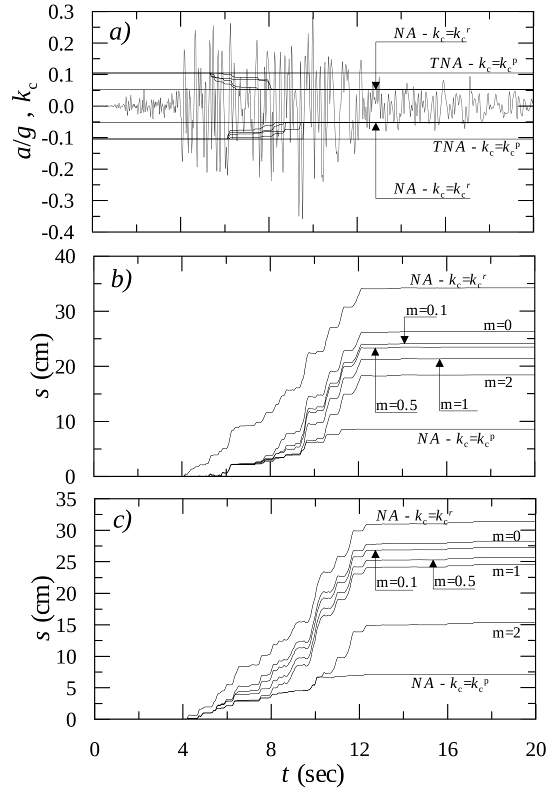

Figure 2. Influence of the yield acceleration degradation path on the earthquake-induced permanent displacements. a) yield acceleration degradation path; b,c) permanent displacements computed for the positive and negative side of the accelerogram #*E*.

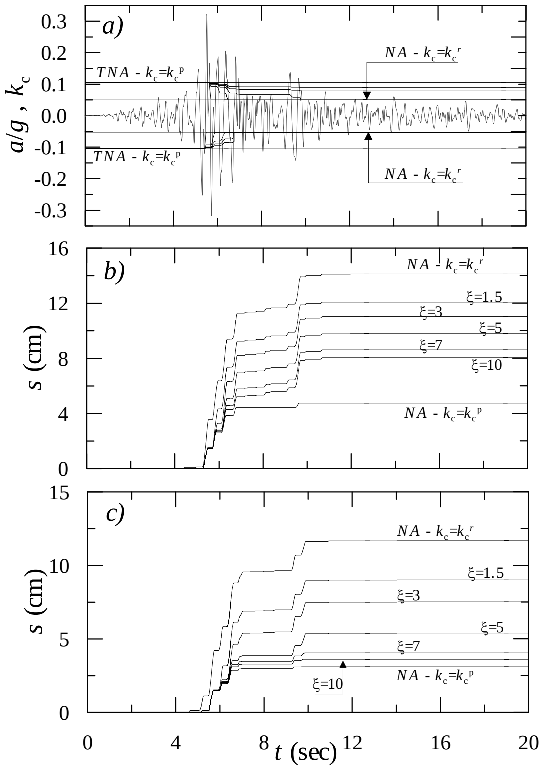

Figure 3. Influence of the displacement ratio ξ on the earthquake-induced permanent displacements. a) yield acceleration degradation path; b,c) permanent displacements computed for the positive and negative side of the accelerogram #*H*.

Using the record #*H* (see Table 1) the influence of the displacement ratio ξ on the displacements response of the considered cover system has been evaluated. In the analyses, values of ξ ranging from 1.5 to 10 were considered. The obtained displacement responses are plotted in Figure 3. Again, the seismic and yield acceleration time history are superimposed in Figure 3a; the displacements response computed for both sides of the accelerogram are shown in Figure 3b and 3c. From the figure showing the computed displacement responses it is apparent that the magnitude of the permanent displacements increase as the ratio ξ decreases. As an example the cumulated permanent displacement computed for $\xi = 1.5$ is three times larger than those computed for $\xi = 10$.

Figure 4 shows the influence of the yield acceleration reduction factor η on displacement response computed for m=1 using the record #E (see Table 1). In the analyses values of η ranging from 0.2 to 1 were considered. In detail, the degradation path followed by the yield acceleration is plotted in Figure 4a together with the acceleration time history considered in the analysis. The displacement responses, computed for both positive and negative sides of the accelerogram, are plotted in Figure 4b and 4c respectively. From the Figure the significant dependence of the computed permanent displacement on the post-peak strength drop is apparent. As an example, for η varying in the range 0.2-0.8, the computed permanent displacement s increases from a few to tens of centimeters. The displacement responses obtained through the traditional approach (*NA*) using the peak strength parameters (η=1) underestimates the displacement response of the considered cover system. Generally, the underestimation of the permanent displacement depends on the magnitude of the post-peak strength reduction and thus, on the assumed values of the yield acceleration reduction factor η.

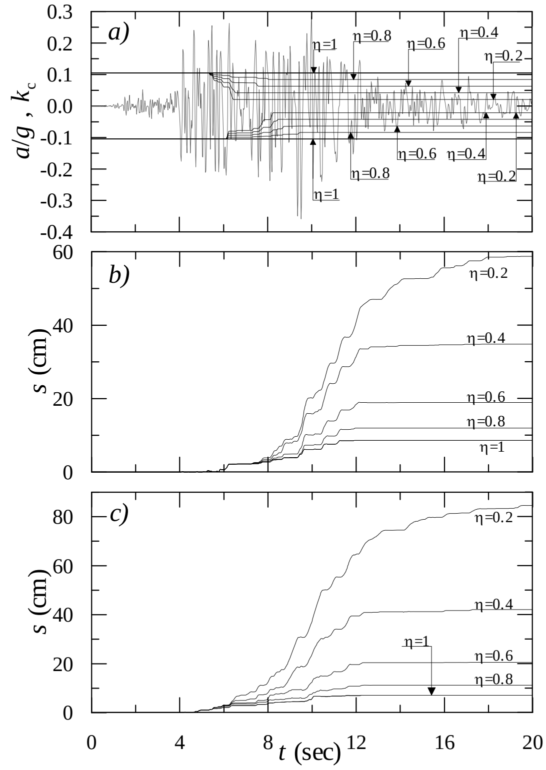

Figure 4. Influence of the yield acceleration reduction factor η on the earthquake-induced permanent displacements. a) yield acceleration degradation path; b, c) permanent displacements computed for the positive and negative side of the accelerograms #*E*.

The results of a Newmark type analysis are frequently adopted to propose empirical regression models that allow prediction of permanent displacements for a given level of confidence. The regression models are usually obtained through a best-fit regression analyses and, usually, link the maximum value of the computed permanent displacement *s* (evaluated for both positive and negative sides of the accelerogram) to some seismic parameters computed for the accelerograms adopted in the analysis.

Commonly, relationships between *s* and the maximum value of the seismic-induced acceleration coefficient k_{max}, the ratio between the down-hill yield acceleration k_c and k_{max}, the Arias intensity I_a and the destructiveness potential factor P_d are proposed. From a practical point of view, polynomial relationships between *s* (or *log s*) and k_{max}, k_c/k_{max}, I_a and P_d are proposed (Ambraseys & Menu 1988, Yegian et al. 1991). All the regression models available in the literature were obtained starting from the results of displacement analyses performed neglecting the possibly occurring strength degradation along the potential failure plane or interface. Thus, an unsafe estimation of permanent displacements can be provided, especially for geosynthetic interfaces which usually shows a strain softening behavior during the shaking.

To stress the importance of the strength reduction, displacement analyses were performed using all the selected accelerograms (Table 1) and a given set of values of the parameters involved in the proposed model. The considered values of the parameters are: β = 14°, ϕ^p=20°, a^p=0, s^p = 0.5 cm, η = 0.5, ξ=5, m = 0.1 and m = 1. Thus, three groups of displacement analyses (m=0.1, m=1 and *NA*) were performed. For each record the final values of the permanent displacement were computed for both sides of the accelerogram. The computed permanent displacements are plotted versus k_{max}, k_c/k_{max}, I_a and P_d in the Figures 5a, b, c and d respectively.

The results of a traditional Newmark analysis (*NA*), performed using peak yield acceleration values ($k_c=k^p$), are also plotted for comparison. In detail, in Figure 5a the obtained permanent displacements are plotted as a function of k_{max} and, as an example, polynomial (grade 2) best fitting relationships are also plotted. In Figures 5b-5d the permanent displacements are plotted in a semi-log scale as a function of k_c/k_{max}, I_a and P_d respectively. In the same figures polynomial (grade 1) regression functions are plotted for the performed displacements analysis.

All the data clearly shown that, when strength reduction takes place along the interface, a larger dispersion of the computed displacement occurs. The influence of the post-peak degradation path on the magnitude of final permanent displacement is pointed out in all the shown best-fit relationships. Moreover, the results of a traditional Newmark analysis (*NA*), performed using peak yield acceleration values (*NA*-$k_c=k^p$), emphasize the inadequacy of traditional approach in predicting seismic induced displacements of the cover system when significant strength reduction occurs along the interface.

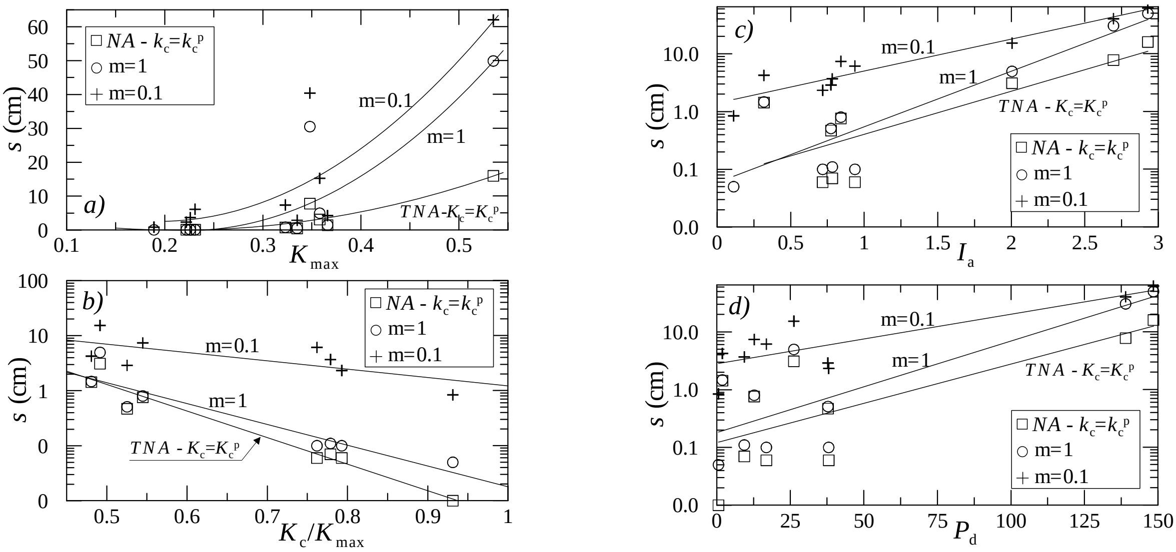

Figure 5. Examples of best-fit regression analysis carried out including the strength degradation at the interface. a) relationships between s and K_{max}; b) relationships between s and the ratio K_c/K_{max}; c) relationships between s and I_a; d) relationships between s and P_d.

CONCLUDING REMARKS

The design of waste landfill cover systems under earthquake loading requires the understanding of the behavior of the geosynthetic-geosynthetic and soil-geosynthetic interfaces generally involved in these engineered systems. In this paper a modified Nemark-type analysis is presented allowing displacement analysis to be performed including the possibly occurring strain softening behavior along the geosynthetic-geosynthetic and soil-geosynthetic interfaces.

In the model, the strength degradation at the interface is described by three parameters: a degradation parameter m, describing the post peak strength degradation path, a displacement ratio $\xi=s^r/s^o$, representing the ratio between the values of displacement for which residual and peak strength values are attained, and a yield acceleration reduction factor $\eta= k_c^r/k_c^p$, representing the ratio between the residual and the peak value of the yield acceleration factors.

Using a set of accelerograms related to earthquakes that occurred in USA, Japan and Turkey with magnitude M ranging from 6.1 to 7.3 a parametric analysis was performed. The analysis was carried out with the aim to point out the influence of the strength degradation on the computed displacement responses of ideal cover systems.

The obtained result clearly shows the influence of the post-peak degradation path and of post-peak strength drop on the magnitude of final permanent displacements. The needed for accurate experimental evaluation of interface behavior is pointed out and the inadequacy of the traditional approach in predicting displacement response when significant strength reduction occurs is emphasized.

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