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Modified newmark analysis of geosynthetic cover system

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ABSTRACT: For engineered cover systems involved in waste landfills, seismic slope stability problems are often critical since (i) geosynthetic-geosynthetic and soil-geosynthetic interfaces introduce plane of weakness; (ii) many interfaces are characterized by a strain softening force-displacement curves when subject to shearing. In the paper a modified sliding block procedure including the main aspects of geosynthetic interface dynamic behavior is proposed to assess the earthquake-induced permanent displacement of cover systems. Yield acceleration degradation pattern is assumed as a function of the post peak force-displacement curves of the interface. A parametric study on the proposed model was performed using some real earthquake records. The obtained results points out (i) a significant dependence of displacement response on dynamic interface behavior; (ii) the need of experimental tests on interface behavior, (iii) the inadequacy of traditional displacement approaches when significant strength reduction occurs along the interface.

1 INTRODUCTION

Modern procedure for landfill or earth-reinforced structures design has been receiving increased attention in recent years due to failures occurred in some recent strong earthquakes and due to the need of proper provisions to be included in codes for design in seismic areas. A proper approach for seismic stability analysis of such structures requires the understanding of geosynthetic-geosynthetic and soil-geosynthetic interface behavior; moreover the assessment of earthquake effects and thus the evaluation of post-seismic serviceability of the structures is necessary for design purposes. This latter aspect is a crucial point especially for municipal solid waste landfills or any other waste containment structure where geosynthetic are used as impervious barriers along cover and bottom sealing system. Therefore the assessment of seismic induced displacements in those structures becomes a fundamental step for evaluating the potential leachate leakage due to sliding. 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, therefore suggesting that a displacement analysis procedure including the main aspects of geosynthetic interface dynamic behavior should be used to assess the level of seismic induced risk.

In the design procedure of geosynthetic liners and covers for landfill, Newmark (1965) sliding block approach has been widely used. The aim of applying such procedure is the estimation of seismic induced damage due to the sliding of sealing system through the assessment of the potential permanent displacements that may occur during the earthquake shaking. Permanent deformation analysis is usually performed using a procedure mainly based on the assumption of constant yield acceleration and only one-way sliding. However, for earth structures including geosynthetics such assumptions do not appear realistic and a modified Newmark sliding block procedure is generally required. The only one-way (down-hill) sliding hypothesis is commonly accepted for many practical purposes involving natural slopes or soil structures such as dams and embankments. In this case assuming an inclined or circular failure surface, and considering no strength reduction effects, the value of the yield acceleration for up-hill sliding rarely be exceeded during an earthquake. So the up-hill displacement usually does not occur and the only one-way sliding hypothesis can be accepted. Conversely, in modern landfills, interfaces are usually characterized by relative low friction angles and the up-hill yield acceleration is not great enough to ensure that only down-hill sliding occurs. Consequently a two-ways sliding model is required to perform permanent displacement analysis. 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 is typical of a strain softening material; the measured shear force reaches a peak value and after decreases until residual vales 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 residual condition after the peak are strictly depending on the interface properties and on the acting stress level. Therefore, the constant yield acceleration assumption does not appear fully satisfactory for Newmark displacement analysis of structures including geosynthetics. A strength degradation model is required to evaluate the displacement response and the post-peak behavior of the interface should be taken into account in choosing the pattern of degradation of the yield acceleration.

2 MODIFIED NEWMARK DISPLACEMENT ANALYSIS

In the traditional sliding block procedure no strength reduction is accounted for along the potential sliding surface. Thus a constant value of yield acceleration is considered in the displacement analysis. The use of different type of geosynthetic in the cover and bottom barrier system of landfills requires removing this simplifying assumption. In the original paper by Newmark (1965) this aspect was emphasized with reference to seismic stability of earth structures; however, although Newmark sliding procedure was largely used in geotechnical earthquake engineering, in most cases strength degradation was neglected for both earth structures and waste containment systems.

Since the yield acceleration of the potential failure mass is established as a function of the material and/or interface shear strength, a possible reduction of its value should be taken into account when sliding occurs. Assuming strain softening force-displacement curves, as schematically shown in Figure 1a, the available strength will be assumed as a function 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. Consequently the yield acceleration changes during sliding, being a function of the available interface shear strength. Figure

1b shows the schematic of the yield acceleration degradation pattern assumed in this paper for both up and down-hill sliding. 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. The initial values of yield acceleration are evaluated using the traditional pseudo-static approach and referring to the peak value of strength parameters. Using the same procedure the residual values of yield acceleration can be obtained using the strength parameters for residual failure envelope. With reference to the infinite slope scheme and assuming the Mohr-Coulomb failure criterion the following 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 obtained:

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

$$k_c^{p,i} = \frac{a^p/(\gamma \cdot H \cdot \cos\beta) + \cos\beta \cdot \tan\phi^p + \sin\beta}{\cos\beta^* + \sin\beta^* \cdot \tan\phi^p} \tag{2}$$

$$k_c^{r,o} = \frac{a^r/(\gamma \cdot H \cdot \cos\beta) + \cos\beta \cdot \tan\phi^r - \sin\beta}{\cos\beta^* + \sin\beta^* \cdot \tan\phi^r} \tag{3}$$

$$k_c^{r,i} = \frac{a^r/(\gamma \cdot H \cdot \cos\beta) + \cos\beta \cdot \tan\phi^r + \sin\beta}{\cos\beta^* + \sin\beta^* \cdot \tan\phi^r} \tag{4}$$

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. Introducing the degradation parameter m which describes the post peak strength degradation path (see Figure 1), the displacement ratio $\xi=s^r/s^o$ and the yield acceleration reduction factor $\eta= k_c^r/k_c^p$ the yield acceleration degradation laws can be written as follows, for both up and down-hill directions:

$$\frac{k_c(s)}{k_c^{\,p}} = \eta + (1-\eta)\cdot\left[1-\left(\frac{s/s^o-1}{\xi-1}\right)^m\right] \tag{5}$$

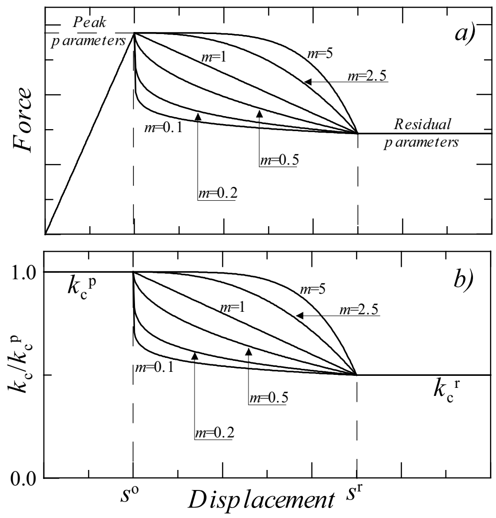

Figure1. Schematic of yield acceleration degradation.

Assuming a linear relationship (m=1) for the post-peak force displacement curves the trilinear model by Matasovic et al. (1997) will be obtained. Since s is a time dependent value, the displacement analysis must be performed assuming a time dependent value of the yield acceleration for both up and down-hill movements. Assuming a time history of the earthquake induced acceleration, permanent displacements can be evaluated following Newmark procedure, by double integration of the following equation of motion:

$$\ddot{s} = \left[k(t) - k_c(t)\right]\cdot g \cdot \frac{\cos(\phi - \delta\cdot\beta^*)}{\cos\phi} \tag{6}$$

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

3 PARAMETRIC ANALYSIS

Using the proposed approach a parametric analysis was carried out to show the influence of the interface strength degradation path on displacement response of a typical cover liner system. A 1V:4H (β=14°) infinite slope scheme was considered and the following values of peak strength parameters were assumed: ϕ^p=20° and a^p=0. A parametric variation of s^p, m, η and ξ was analyzed. Six recent earthquakes were selected, among those available in the PEER strong motion database; for each earthquake two different horizontal acceleration records were used in the displacements analysis, that is ω=0 is assumed. Table 1 lists the earthquake records selected for the analysis. In the same table the earthquake magnitude M is reported. For each record the following seismic parameters have been evaluated (Table 2): peak values of ground motion (*PGA, PGV, PGD*), duration D of the record, Arias intensity I_a, destructiveness potential factor P_d, bracketed duration D_b, predominant period T^o.

Table 1. Selected earthquake records

Earthquake	M	Record name - Station / Component
Morgan Hill (USA 1984)	6.1	*#A* - Gilroy#4/ Gil04270 *#B* - Gilroy#4/ Gil04360A
Imperial Valley (USA 1979)	6.5	*#C* - El Centro#6/ A-E06-140 *#D* - El Centro#6/ A-E06-230
Northridge (USA 1994)	6.7	*#E* - Hollywood Storage/ HOL360 *#F* - Hollywood Storage/ HOL090
Loma Prieta (USA 1989)	7.1	*#G* - Gilroy#7/GMR000 *#H* - Gilroy#7/GMR090
Kobe (Japan 1985)	7.2	*#I* - KJM/KJM000 *#J* - KJM/KJM090
Ducze (Turkey 1999)	7.3	*#K* - Ducze /DCZ180 *#L* - Ducze /DCZ270

Table 2. Characteristics of the selected earthquake records.

Record	*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)
#A	0.224	19.3	4.33	60	0.72	38.0	13.8	1.15
#B	0.348	17.4	3.11	60	0.77	37.8	12.7	0.50
#C	0.366	20.8	2.83	20	0.32	1.91	1.62	0.26
#D	0.189	12.1	1.15	20	0.11	0.58	1.34	0.37
#E	0.358	27.5	3.04	40	2.00	26.2	15.3	0.85
#F	0.231	18.3	4.81	40	0.94	16.8	14.8	0.70
# G	0.226	16.4	2.52	40	0.78	9.3	13.5	0.46
#H	0.323	16.6	3.26	40	0.84	12.7	10.7	0.47
#I	0.821	81.3	17.7	50	8.39	230.9	21.4	0.68
#J	0.599	74.3	19.9	50	5.43	178.8	17.2	0.70
#K	0.348	60.0	42.1	26	2.69	138.9	16.1	0.43
#L	0.535	83.5	51.6	26	2.93	148.4	14.1	1.29

Figure 2 shows the displacement response to the record *#E* evaluated for different values of the degradation parameter m and assuming s^p=0.5cm, η=0.5 and ξ=5. For the selected case the values of yield acceleration for up and down-hill sliding were evaluated for both peak and residual conditions using eqns. 1 to 4. Peak and residual yield acceleration coefficient resulted: $k_c^{p,o}$=0.105, $k_c^{p,i}$=0.674, $k_c^{r,o}$=0.052, $k_c^{r,i}$=0.337. The seismic and yield acceleration time histories (Figure 2a) and the displacement time histories for both sides of the accelerogram (Figure 2b and 2c) are plotted. In the same figures the results obtained using the traditional Newmark analysis (*NA*) performed with peak (k_c=k_c^p) or residual (k_c=k_c^r) yield acceleration values are shown. The results clearly point out the influence of the post-peak degradation path on the magnitude of final permanent displacements: for *m* varying in the range 0.1-2, permanent displacements increase of about 75%. The results obtained with the traditional approach show that the magnitude of permanent displacements can be greatly underestimated using peak strength parameters and may be excessively conservative using the residual ones. Therefore the knowledge of post peak interface behavior is necessary to better estimate displacement response. Although the strength reduction no significant up-hill sliding occurs due to the high values of both peak and residual yield acceleration values.

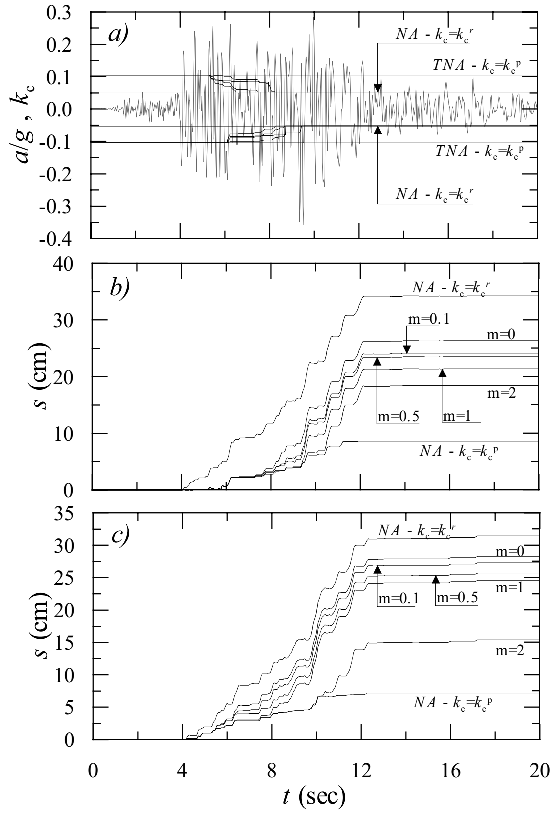

Figure 2. Effect of degradation path on displacement response.

The effect of yield acceleration reduction factor η on displacement response is shown in Figure 3 with reference to the same scheme of Figure 2 and assuming *m*=1. The degradation time history of yield acceleration (figure 3a) as well as displacement response for both positive (Figure 3b) and negative (Figure 3c) sides of the accelerogram, were evaluated using record *#E*. The displacement time history shows a significant dependence on the values of η adopted in the analysis. For η varying in the range 0.2-0.8, *s* changes from a few to tens of centimeters. Therefore a significant dependence of post-seismic serviceability conditions on the post-peak strength drop is apparent. Cyclic direct shear tests performed on different type of interface has generally shown that for geotextile–geonet interface the dynamic friction angles slightly changes during the cycles, while for a geotextile-geomembrane interface a significant reduction may occurs during shearing (De and Zimmie, 1998). Therefore an experimental evaluation of post-peak behavior is necessary to perform a more accurate displacements analysis. For practical purposes the values of η should be computed from the post-peak strength reduction that many interfaces revealed during direct shear tests. The obtained results show that the *NA* performed using the peak strength parameters (η=1) underestimates the displacement response, to an extent depending on the magnitude of the post-peak strength reduction. Also in this case no significant in-slope sliding occurs.

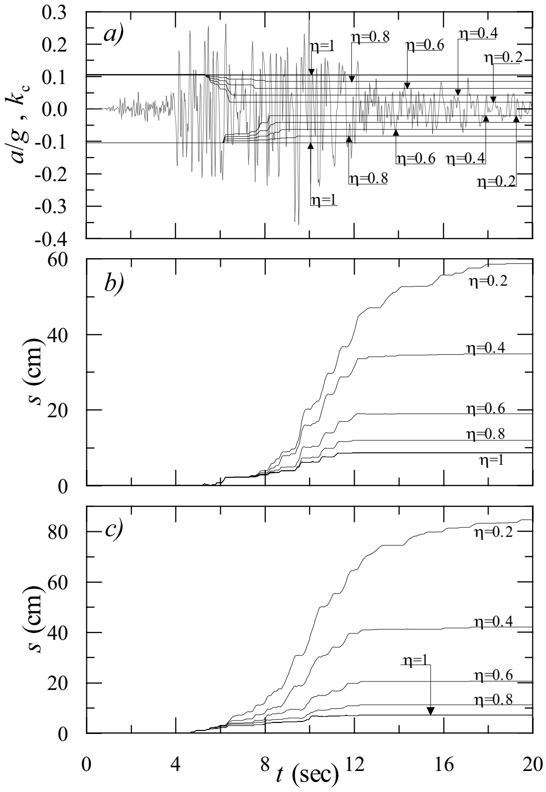

Figure 3. Effect of yield acceleration reduction on displacements.

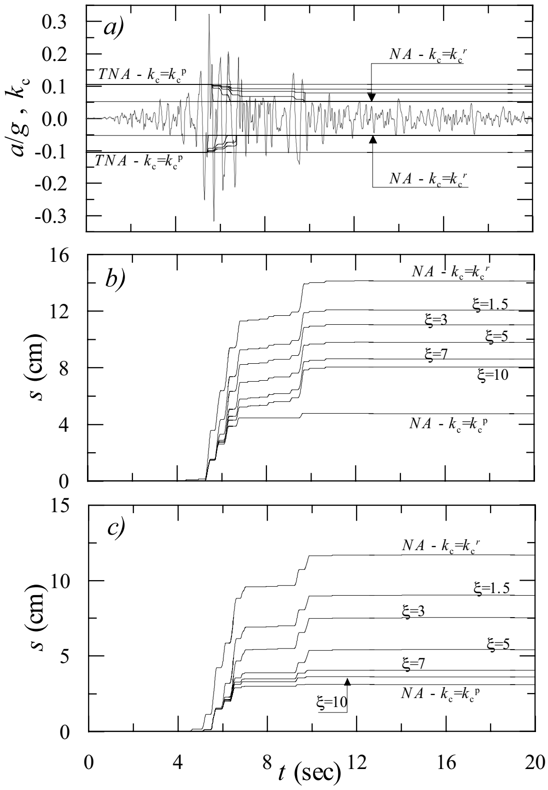

Figure 4. Effect of displacement ratio on system response

The influence of the parameter ξ on the displacements response has been evaluated for the record *#H* using the same scheme of Figures 2 and 3; the obtained results are plotted in Figure 4. The seismic and yield acceleration time history (figure 4a) as well as the displacements response for both sides of the accelerogram, (Figure 4b and 4c) are shown. It is apparent that displacements magnitude increases with decreasing ξ, becoming three times larger when ξ varies from 1.5 to 10. Since the displacement range s^r-s^p, evaluated form shear tests, strictly depends on the normal stress acting during the test, it may be recommended that experiments should be carried out using a normal stress level similar to that expected in the field, so that displacements can be assessed more reliably.

Since Newmark sliding block procedure has been widely used in geotechnical earthquake engineering, many empirical relationships were developed to predict the magnitude of seismic induced displacement in slopes, dams, embankments and landfills. All the proposed relationships are obtained from best fitting of displacement analyses performed using different earthquake databases. Empirical regression relationships available in literature usually link the maximum value of permanent displacement *s* (evaluated for both positive and negative sides of the accelerogram) to some seismic parameters evaluated for the acceleration records adopted in the displacements analysis. Commonly the proposed relationships correlates permanent displacement *s* to the maximum value of seismic induced acceleration k_{max} , to the ratio between the down-hill yield acceleration k_c and k_{max}, to the Arias intensity I_a and to the destructiveness potential factor P_d. Usually polynomial relationships between *s* and k_{max} or between *log s* and k_c/k_{max}, I_a and P_d are proposed (Ambraseys & Menu 1988, Yegian et al. 1991).

Since these relationships are obtained without taking into account any shear strength reduction along the potential failure plane or interface, an unsafe evaluation of seismic induced displacements can be provided, especially for geosynthetic interface which usually shows a strain softening behavior during the shaking. To point out the inadequacy of such relationships to evaluate the system response when significant strength reduction occurs, displacement analyses were performed using all the selected records showed in Table 1. However, the results and the selected records are not sufficient to provide a close form solution performing a regression analysis. The geometrical (β) and mechanical (*a*, ϕ) properties adopted in the displacement analyses are those used for Figures 2, 3 and 4. Two different values of the degradation parameters *m* was adopted; the following values of the other parameters was adopted: η=0.5, ξ=5, s^p=0.5cm. The results of a traditional Newmark analysis performed using peak yield acceleration values (k_c=k^p) are also presented to show the effect of neglecting strength degradation effects. A total of three groups of displacement analysis were performed. For each record the final values of permanent displacement are evaluated for both sides of the accelerogram using the proposed yield degradation model. The results of the analysis are shown in Figures 5. In figure 5a the obtained permanent displacements are plotted as a function of k_{max}; 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. As clearly shown, when strength reduction takes place a larger dispersion of the data occurs. The influence of the post-peak degradation path on the magnitude of final permanent displacement is pointed out in all the showed best fitting relationships. Therefore a significant dependence of post seismic serviceability conditions on the post-peak interface behavior is apparent. Finally the results of a traditional Newmark analysis, performed using peak yield acceleration values (*NA*-k_c=k^p), emphasize the inadequacy of traditional approach in predicting seismic induced displacements when significant strength reduction occurs along the interface.

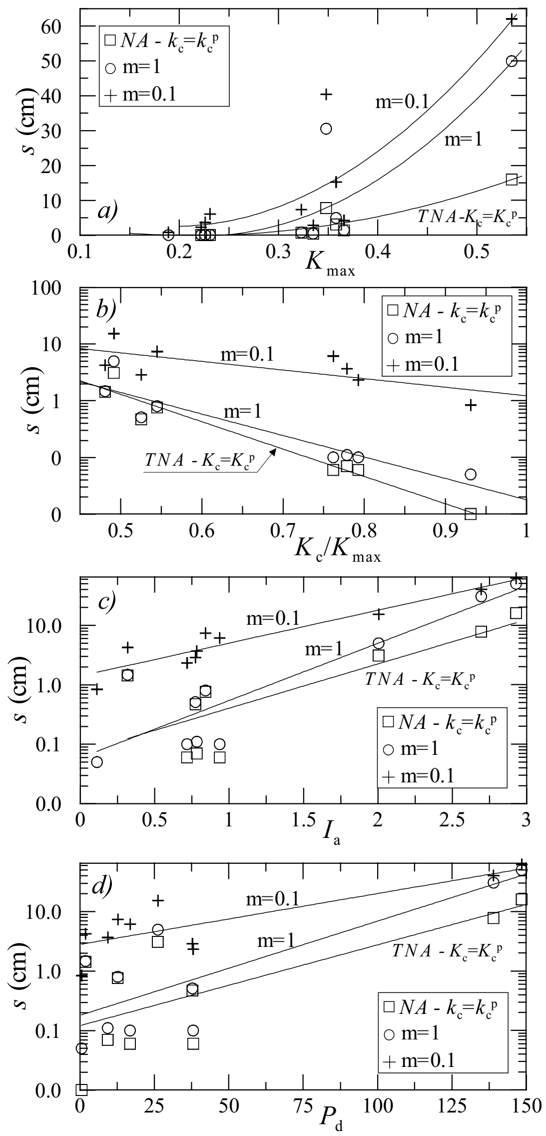

Figure 5. Results of parametric analysis

4 CONCLUDING REMARKS

In this paper a modified Newmark displacement analysis for a typical cover liner system is proposed to takes into account the strain softening behavior of many geosynthetic-geosynthetic and soil-geosynthetic interfaces. A parametric analysis on the proposed model was performed using some real earthquake records to point out the influence of the interface behavior on displacement response. 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 of accurate experimental evaluation of interface behavior is pointed out and the inadequacy of traditional approach in predicting displacement response when significant strength reduction occurs is emphasizes.

REFERENCES

Mitchell, J.K., Seed, R.B., Seed, H.B. 1990. Kettleman Hills waste landfill slope failure. I: liner–system properties. *Journal of Geotechnical Engineering*,116- 4: 647-668.

PEER, Strong Motion Database. http://peer.berkeley.edu/svbin/Quake

Matasovic, N., Kavazanjian, E.Jr., Yan, L. 1997. Newmark deformation analysis with degrading yield acceleration. *Proc. of Geosynthetics '97*: 989-1000. Long Beach, California, USA.

Ambraseys, N.N. & Menu, J.M. 1988. Earthquake-induced ground displacements. *Earthquake Engineering and Structural Dynamics*, 16: 985-1006.

Yegian, M.K, Marciano, E., Ghahraman, V.G. 1991. Earthquake-induced permanent deformations: probabilistic approach. *Journal of Geotechnical Engineering*, 117-1:35-50.