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ABSTRACT

During recent earthquakes small dams and embankments suffered large settlements as a result of earthquake-induced liquefaction. One such case is the mole embankment that settled about 1.2m and was displaced horizontally by about 2m at King Harbor Redondo Beach, California as a result of the Northridge earthquake of 1994 (Kerwin and Stone, 1997). The conventional sliding-block model has shortcomings in back-estimating the critical acceleration and corresponding strength of such earthquake-induced slides when seismic displacement is large. The reason is that the change on geometry of the sliding mass, that greatly affects the seismic displacement, is not modeled. Stamatopoulos et al (2000) proposed a two-body sliding system that models this change in geometry. In the present paper, the Stamatopoulos et al (2000) sliding system model is used to back-estimate the residual shear strength of the mole embankment at King Harbor Redondo Beach. Then, the correlation of the residual soil strength and the blow count resistance of the SPT of this case is compared to the relationship that has been proposed by Seed and Harder (1990).

INTRODUCTION

During recent earthquakes, small dams and embankments were badly damaged as a result of earthquakes (e.g. Stamatopoulos, 2003). The excessive deformation of these earth structures was a result of liquefaction within the earth structures, or at the top of the underlain soil. Some of these case studies are well-documented: the initial and deformed geometries have been recorded, and field standard penetration tests were performed. Characteristics of the applied earthquake are also known. One such study is the mole embankment that settled about 1.2m and was displaced horizontally by about 2m at King Harbor Redondo Beach, California as a result of the Northridge earthquake of 1994 (Kerwin and Stone, 1997).

Analysis of such slides provides a unique opportunity to correlate the blow count resistance of the Standard Penetration Test (SPT) to the residual strength of a liquefied soil. Evaluation of the residual strength of a liquefied soil is one of the most difficult problems in contemporary geotechnical engineering practice, mainly because it is difficult to obtain undisturbed samples in sands. An approach has been developed to estimate the shear strength of liquefied soils from the SPT blow count resistance. This approach is based on the shear strength back-calculated from observed slides (e.g. Seed and Harder, 1990). The proposed relationship has much scatter, and comparison of more field data with this correlation are desirable.

The conventional sliding-block model has shortcomings in back-estimating the critical acceleration of earthquake-induced slides when seismic displacement is large. The reason is that the change on geometry of the sliding mass, that greatly affects the seismic displacement, is not modeled. Stamatopoulos et al (2000) proposed a two-body sliding system that models this change in geometry. The model consists of two bodies that slide in different inclinations. The inclination of the internal slip surface that separates the two bodies corresponds to the minimum critical acceleration (or factor of safety) value, and affects the ratio of the displacement of the two bodies. The model has been used by Stamatopoulos et al (2000) to analyze three dam slides: the Lower San Fernando Dam slide triggered by the 1971 earthquake, the LaPalma Dam slide triggered by the Chilean earthquake of 1985, and the La-Marquessa Dam slide triggered by the Chilean earthquake of 1985.

In the paper, the Stamatopoulos et al (2000) sliding system model is used to back-estimate the residual shear strength of the mole embankment at King Harbor Redondo Beach. Then, the correlation of the residual soil strength and the blow count resistance of the SPT of this case is compared to the relationship that has been proposed by Seed and Harder (1990).

THE STAMATOPOULOS ET AL (2000) MODEL

The model assumes that a horizontal earthquake is applied on the sliding 2-dimensional mass shown in Fig.1. The mass is divided into two bodies: body 1 on a sub-plane with inclination α_1 and body 2 on a sub-plane with higher inclination α_2. An internal sub-plane with inclination $(90^o-\delta)$ separates the two bodies.

The model assumes that the angle of the internal sub-plane of the slide, δ, is constant and does not change as a function of the distance moved. In addition, the 2-dimesional total mass of the slope is taken to be constant throughout the sliding period. The above gives that the incremental change in cross-sectional area of body 1 should equal the change of area of body 2, or,

$$\frac{u_1}{u_2} = \frac{du_1}{du_2} = \frac{\cos(-\delta-\alpha_2)}{\cos(-\delta-\alpha_1)} = \lambda_1 \qquad (1)$$

where u_1 and u_2 are the distances moved along the first and second slip sub-plane respectively.

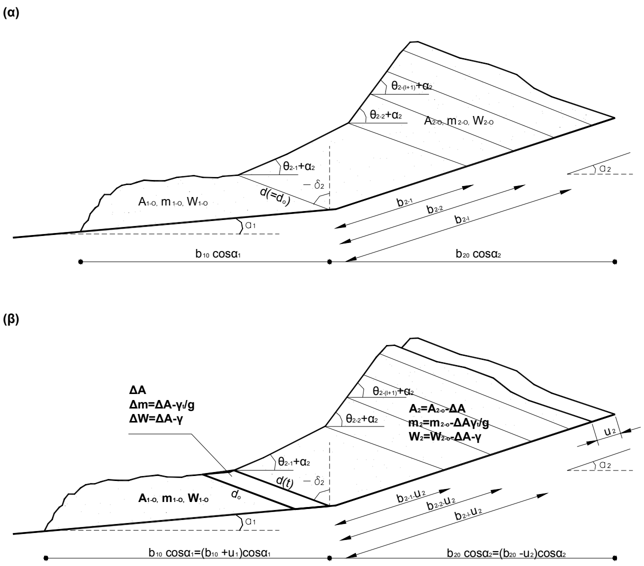

Fig. 1. The Stamatopoulos et al (2000) model: (a) the initial configuration and (b) when the distance moved is u_2.

Shear resistance is taken to act along the external slip surfaces and the sub-plane of internal shearing. Different values are assumed, denoted as c_1 and c_2 along the external slip lines of bodies 1 and 2, and c_3 along the line of internal shearing. For saturated soil, the soil resistances c_i correspond to the undrained soil strength. In reality, as the two-body sliding system moves, the material of the external sub-plane is sheared to failure and stays in this condition during motion. On the other hand, in order the two-body sliding system to move, the internal sub-plane must be sheared to failure. Different material exists in the internal sub-plane as the two-body system slides. It is inferred that for saturated conditions, the strengths at the two external subplaces, c_1 and c_2 correspond to the residual shear strength of the soil, while the strength at the internal sublane, c_3, corresponds to the peak undrained soil strength.

The parameters that define the model can be classified as parameters (a) of initial mass, weight and geometry, (b) of strength and (c) of the geometry and unit mass and weight of the part of the second body that is transferred to the first. Specifically, regarding parameters (a), the initial masses and weights of the two bodies m_{1o} and m_{2o} and W_{1o} and W_{2o} must be defined. In addition, the inclinations of the two bodies, α_1 and α_2 , the initial contact lengths of the two bodies, b_{1o}, b_{2o}, and the initial length and inclination of the internal sub-plane d and δ must be specified. Regarding parameters (b), the cohesional resistances along the first and second slip sub-planes, c_1 and c_2, as well as along the internal sub-plane, c_3 must be specified. Regarding parameters (d), the total and effective unit weights of the transferred mass, γ_t and γ_b, as well as the inclinations of the top surface of the transferred part of the second body relative to α_2 in terms of the length along b_2, (θ_{2-i}, b_{2-i}), must be defined. As in the case that will be studied $u_2 < b_{2-1}$, the parameters, (θ_{2-i}, b_{2-i}), are reduced to only one parameter, θ_2.

As Stamatopoulos et al (2000) illustrates, the governing equation of motion of the sliding system is:

$$\ddot{u}_1 = Z_1 \, g \,(k(t) - k_c) \qquad (2a)$$

where g is the acceleration of gravity, $\{g\ k(t)\}$ is the applied acceleration, the critical acceleration factor k_c equals

$$k_c = \frac{AA}{BB} \quad \text{with} \qquad (2b)$$

$$AA = W_1 \sin(-\alpha_1) + c_1 b_1 + c_3 d \sin(-\delta-\alpha_1) + (W_2 \sin(-\alpha_2) + c_2 b_2 - c_3 d \sin(-\delta-\alpha_2))$$

$$BB = m_1 \sin(\alpha_1) + m_2 \sin(\alpha_2)$$

and the factor Z_1 equals

$$Z_1 = (m_1 \cos\alpha_1 + m_2 \cos\alpha_2) \,/\, (m_1 + m_2 / \lambda_1) \qquad (2c)$$

The change in the lengths and cross-sectional area of the second body are

$$b_2 = b_{20} - u_1 / \lambda_1 \qquad (3)$$

$$d = d_0 + \frac{\sin\theta_2}{\cos(\theta_2+\alpha_2+\delta)} \cdot u_2 = d_0 + \frac{\sin\theta_2}{\cos(\theta_2+\alpha_2+\delta)} \cdot \frac{u_1}{\lambda_1} \qquad (4)$$

$$\Delta A_2 = d_0 \cdot \cos(\alpha_2 + \delta\ ) \cdot \frac{u_1}{\lambda_1} + \frac{0.5 \cdot \cos(\alpha_2 + \delta\ ) \cdot \sin\theta_2}{\cos(\theta_2 + \alpha_2 + \delta)} \cdot \left(\frac{u_1}{\lambda_1}\right)^2 \tag{5}$$

In the case that there is space in order that the lower slip sub-plane of the slide can increase its length by u_1, its change in length and cross-sectional area is

$$b_1 = b_{10} + u_1 \tag{6}$$

$$\Delta A_1 = \Delta A_2 = d_0 \cdot \cos(\alpha_2 + \delta) \cdot \frac{u_1}{\lambda_1} + \frac{0.5 \cdot \cos(\alpha_2 + \delta) \cdot \sin\theta_2}{\cos(\theta_2 + \alpha_2 + \delta)} \cdot \left(\frac{u_1}{\lambda_1}\right)^2 \tag{7}$$

In addition, it always holds that

$$\Delta W_i = \gamma_b\ \Delta A_i \tag{8}$$

$$\Delta m_i = \gamma_t \Delta A_i \ / \ g \tag{9}$$

where the subsript "i" equals to 1 and 2. The above equations are identical to those presented by Stamatopoulos et al (2000), but expressed in a simpler form.

THE LIQUEFACTION FAILURE AT KING HARBOR

Kerwin and Stone (1997) present in detail this case study of ground displacement induced by liquefaction at Redondo Beach King Harbor during the Northridge Earthquake of January 17, 1994. The earthquake had a moment magnitude of 6.7. The site was located about 50km from the earthquake epicenter.

The most severe damage occurred to the central portion of Mole B, one of four offshore fills. Settlements on the order of about 1.2m were typical in the central portion of the failure area. Figure 2 gives a cross-section through the failure area of the embankment illustrating the initial and deformed geometries. The peak acceleration on site was estimated around 0.15g.

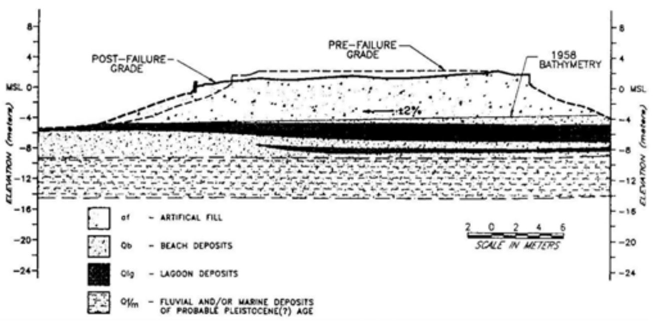

Fig. 2. Cross-section of the initial and deformed embankment, after the liquefaction failure of the mole at King Harbor during the Northridge Earthquake of January 17, 1994 (Kerwin and Stone, 1997).

Approximately three weeks after the failure, six borings were drilled. Sieve analyses indicated that the fill was relatively uniform and consisted of fine to medium grained sands. The blow count during the SPT test of the artificial fill ranged from 2 to 10. The corresponding average measured corrected (e.g. European Prestandard, 1994), blow count ($N_1(60)$) for the liquefied sand layer is about 6.

ANALYSIS OF THE LIQUEFACTION FAILURE

Figure 3 gives the initial slide geometry of the model slide considered. The external boundaries of the slope are according to the actual slide of Figure 2. The lower slip sub-plane was taken, according to the failure mechanism, on the base of the embankment. The location and inclination of the upper slip sub-plane was estimated based on the theory of limit equilibrium, as the sub-plane that corresponds to the minimum value of the critical acceleration. No data exists on the unit weight of the soil. A reasonable assumption, used in the analysis, is that the total unit weight of the soil, γ_t, equals to 2 t/m^3.

The inclinations and initial lengths of the external sub-planes of the model slide are given in Table 1. In order to finalize the model geometry, the inclination of the internal slip surface must be determined. This inclination was determined based on the ratio of the measured displacement of the upper and lower sub-plane. Using equation (1), as u_1=2m and u_2=1.5m, the inclination of the internal sub-plane, is estimated as $\delta=-25^o$.

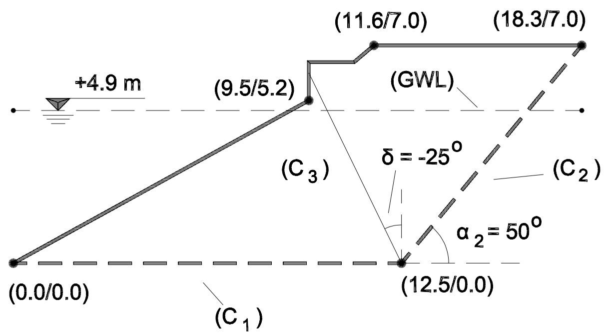

Fig. 3. The initial model geometry used to simulate the slide of Fig. 2.

Table 1. The inclinations, lengths, masses and weights defining the model slide.

α_1, α_2 (deg.)	b_1, b_2, d (m)	δ, θ_2 (deg.)	W_1, W_2* (kPa/m)	m_1g, m_2g^+ (kPa/m)	γ_t, γ_b (kPa/m^2)
0,	12.5,	-25,	287.3,	574.6,	20,
50	9.1,	-8	521.7	729.0	10
	6.2				

* W is weight per unit length
+ m is mass per unit length, g is the acceleration of gravity

According to the theory of limit equilibrium, the internal sub-plane must correspond to the minimum value of the critical acceleration (Sarma, 1979). Figure 4 gives the critical acceleration value versus δ for different values of soil strength and for the initial slide configuration. The value of soil strength applicable in the analysis, as illustrated later, corresponds to about 10kPa. It can be observed that the range of the angles δ, that produce a minimum in the critical acceleration is about -25°, in agreement with the value of δ from the ratio u_1/u_2 . The above illustrates the validity of the proposed approach.

Based on all of the above, all the parameters (except those of the soil strength) defining the model are given in Table 1. Partial submergence of the second body is considered when estimating its initial weight.

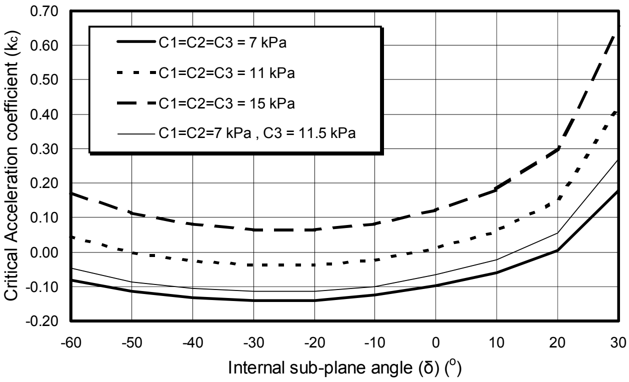

Fig. 4. Critical acceleration in terms of inclination of internal sub-plane.

During the analyses of the slide, initially, regarding soil strength, it was assumed that the strength of the liquefied fill is uniform, or that $c_1=c_2=c_3$. An earthquake record measured in the region was not available to the authors. As the region is susceptible to liquefaction, an earthquake record measured in such an area was first applied. More specifically, the accelerogram that was recorded in the earthquake of Hyogoken-Nambu of Kobe at Port Island and at a depth of 16m was applied. The earthquake had magnitude of 7.2. The site was about 20km from the earthquake epicenter. The accelerogram has a fundamental period of 0.7s and maximum acceleration of 0.58g. It was normalized at the estimated peak acceleration value at the region reported by Kerwin and Stone (1997), of a_m=0.15g. The results of the back-analysis are given in Fig. 5, and Table 2. For u_{1-m}=2m, c_u=11 kPa. The corresponding value of the critical acceleration factor of the initial configuration, k_{c-o}, was -0.03g. The negative value of k_{c-o} indicates static instability. It can be inferred that the large earthquake-induced displacements of the mole were primarily a result of static instability (or $k_{c-o}<0$), presumably caused by an earthquake-induced reduction of strength.

Then, other earthquake records were also applied. All accelerograms were normalized at a_m=0.15g. In particular, the following accelerograms are applied:
- El-Centro (California, USA), 18/5/1940, component NS, M(=earthquake magnitude in the Richter scale) = 6.5, R(=distance from the epicenter) = 5 Km, a_m(=maximum acceleration) = 0.35g, T_f (=fundamental period) =0.6 s.
- San Fernando - Avenue of Stars (California, USA), 1971, component EW, M=6.5, R=40 Km, a_m=0.15g, T_f=0.15 s.
- Kalamata (Greece), 13/9/1986, M=5.75, R=9 Km, Municipality Building, component longitudinal: a_m=0.24g, T_f=0.35s.
- Gazli (USSR), 17/5/1976, M=7.3, a_m=0.70g, T_f=0.1 s.

It can be observed that in these accelerograms the magnitude lies betwen 5.75 and 7.3 and the fundamental period between 0.1 and 0.6 seconds, thus, covering a wide range of values for possible earthquakes.

Table 2. Back-estimated residual soil strength in terms of the applied accelerogram and the ratio c_3/c_1.

Accelerogram	c_3/c_1	c_u (kPa) for u_1 = 2 m
El-Centro	1	11
San Fernando	1	11
Kalamata	1	11
Gazli	1	11
Kobe	1	11
	1.5	10
	2	8

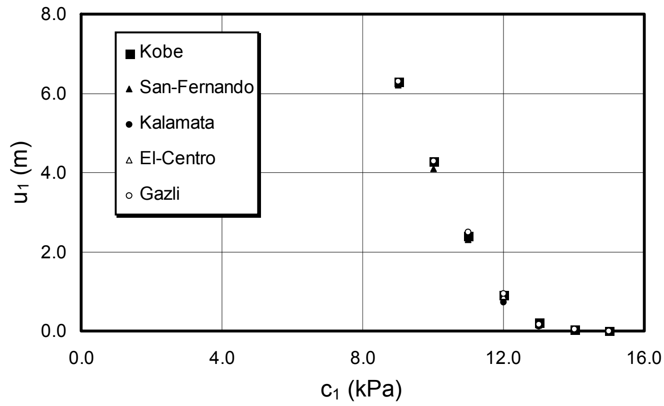

Fig. 5. Permanent displacement in terms of the soil resistance and applied acceleration. Case of $c_1=c_2=c_3$.

Figure 5 gives the back-estimated residual soil strength in terms of the distance moved. It can be observed that for the measured values of u_{1-m} , the back-estimated value of soil strength is not affected considerably by differences in the applied earthquake, and for u_{1-m}=2m equals to 11kPa for all cases. This is presumably because for large displacements, the total distance moved corresponds to movement of the slope from static instability to static stability, that is relatively insensitive on the applied excitation.

It was indicated previously that c_1 and c_2 correspond to the residual soil strength, while c_3 corresponds to the peak undrained soil strength. Thus, the assumption made previously that $c_1=c_2=c_3$ may not hold. To investigate this effect, analyses were performed where it was assumed that ($c_3/1.5=c_1=c_2$) or that ($c_3/2=c_1=c_2$). The Kobe earthquake, described previously, normalized to a_m=0.15g, was applied. Figure 6 gives the back-estimated residual soil strength in terms of the distance moved. It can be observed that for the measured values of u_{1-m}, the back-estimated value of soil strength is affected by the ratio of c_3/c_1. A reasonable ratio of peak to residual soil strength for loose soils in small stress levels is 1.5. When the ratio c_3/c_1 equals to 1.5, for u_{1-m}=2m the back-estimated value of residual soil strength is 10kPa.

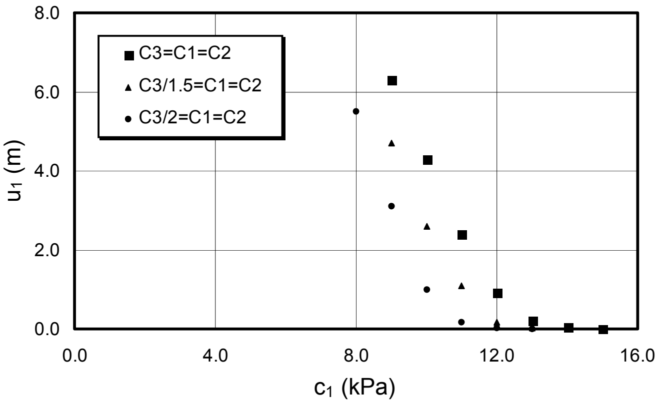

Fig. 6. Permanent displacement in terms of the soil resistance and ratio c_3/c_2. The Kobe accelerogram is applied and it is assumed that $c_1=c_2$.

RESIDUAL SOIL STRENGTH IN TERMS OF THE BLOW COUNT OF THE SPT

Figure 7 compares the back-estimated value of the undrained soil strength in terms of the corrected (e.g. European Prestandard, 1994) blow count value of the SPT of the case analysed, with the range of values proposed by Seed and Harder (1990). It can be observed that the back-estimated strength of 10kPa is within the proposed range of values. More specifically, it is in the upper part of the proposed range.

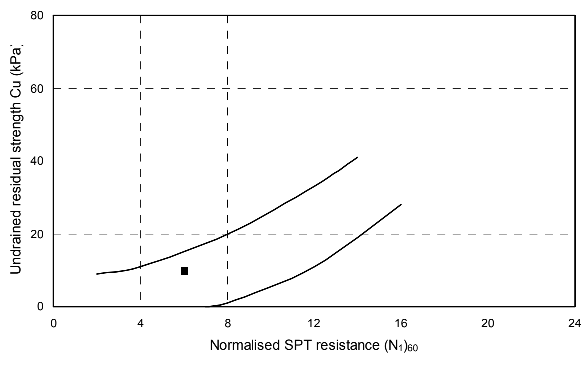

Fig. 7. Back-estimated residual soil strength in terms of SPT, compared with the Seed and Harder (1990) relationship.

CONCLUSIONS

The Stamatopoulos et al (2000) model was applied to back-estimate the residual soil strength that was mobilized during the large displacement of the mole at King Harbor induced during the Northridge Earthquake of January 17, 1994.

The initial configuration of the model slide used is given in Fig. 3. The inclination of the internal slip surface, determined based on the ratio of the measured displacement of the upper and lower sub-plane using equation (1), agrees with that estimated according to the theory of limit equilibrium. This illustrates the validity of the proposed approach.

Parametric back analyses indicated that the undrained soil strength corresponding to the measured distance moved is not affected by the applied accelerogram, presumably because for large displacements the total distance moved corresponds to movement of the slope from static instability to static stability that is relatively insensitive on the applied excitation.

Parametric analyses indicated that the undrained soil strength corresponding to the measured distance moved depends on the ratio of the resistances at the internal and external shearing sub-planes. This ratio corresponds to the ratio of peak to residual soil strength. For the typical ratio of peak to residual soil strength of 1.5, the back-estimated residual soil strength equals to 10kPa. This back-estimated residual soil strength falls in the range of values proposed by Seed and Harder (1990) in terms of the corrected blow count during the SPT test.

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