Proceedings of the 16th International Conference on Soil Mechanics and Geotechnical Engineering

doi:10.3233/978-1-61499-656-9-2711

Back analysis of liquefaction failure and relationship between the residual soil strength and the N value of the SPT

Analyse en arriere de une instabilite pendant liquefaction et relation entre resistance de sol residielle et de la valeur N des essais de SPT

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ABSTRACT

The conventional sliding-block model has shortcomings in back-estimating the soil strength when displacement is large. Recently models have been developed that simulate this change in geometry (Olson et al, 2000, Stamatopoulos et al, 2000, Sarma and Chlimintzas, 2001). The Sarma and Chlimintzas (2001) model is used to analyze the kinematics of the liquefaction-induced large deformation that a dike at the Kabutono Bank suffered during the Nansei-Oki earthquake of 1993 and to back-estimate the residual soil strength, c_u. Then, the correlation of c_u and the corrected blow count resistance of the SPT, N_{1-60}, of this case, and all previous cases analyzed by these improved models are collected and analyzed. The data illustrates that the relationship between c_u and N_{1-60} depends a lot on the fines content. A relationship is derived using linear regression.

RÉSUMÉ

Le model conventionel du "bloc glissant" a des defauts qui concernent l'estimation en arriere de la resistance de sol pendant les glissements des terrains sous un regime des grandes deplacements. Des models recents qui simulent le changement a la geometrie sont proposes (Olson et al, 2000, Stamatopoulos et al, 2000, Sarma et Chlimintzas, 2001). Pour l' analyse de la kinematique des grandes deformations a cause de liquefaction, d' une barrage a Kabutono Bank pendant le seisme de Nansei-Oki en 1993 et apres pour l'estimation en arriere de la resistance residuelle du sol, c_u, le model de Sarma et Chlimintzas (2001) est utilise. Ensuite, la correlation de c_u et de la resistance correctee par SPT , N_{1-60}, pour ce cas aussi avec des cas precedents, examines avec des models ameliores sont collectes et anlyses. Les donnees montrent que la relation entre c_u et N_{1-60} , presente une forte dependence au contenu des grannulometries fines. A l'aide de la 'regression lineaire' une relation est derivee.

1. INTRODUCTION

During recent earthquakes, dams and embankments were badly damaged. 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 geometry has been recorded, and field standard penetration tests were taken. Characteristics of the applied seismic motion are also known.

Analysis of such slides provides a unique opportunity to correlate the corrected blow count resistance of the Standard Penetration Test (SPT), N_{1-60}, to the residual strength of a liquefied soil, c_u. 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. Approaches have been developed to relate c_u with N_{1-60}: Seed and Harder (1990) give a range of the shear strength of liquefied soils, c_u.

The conventional sliding-block model has shortcomings in back-estimating the soil strength of liquefaction-induced slides when displacement is large. The reason is that the change on geometry of the sliding mass, that greatly affects the seismic displacement (e.g. Stamatopoulos, 1996), is not modeled. Olson et al (2000) analyzed this effect in the slide of Wachusett Dam by simulating the translation of the center of mass of the slide. Stamatopoulos et al (2000) proposed a two-body sliding system that models this change in geometry (Fig. 1). The model is an extension of the Ambraseys and Srbulov (1995) and Stamatopoulos (1992) models. The Stamatopoulos et al (2000) model has been used to back-analyze a number of liquefaction-induced slides (Stamatopoulos and Aneroussis, 2003, 2004). Sarma and Chlimintzas (2001) generalized the geometry to n bodies.

The Stamatopoulos et al (2000) and Sarma and Chlimintzas (2001) models are extensions of Sarma's (1979) multi-block stability method (Fig. 2). The dynamic equations of motion are solved and mass transfer between adjacent blocks is simulated. The inclination of the internal slip surface that separates consecutive bodies corresponds to the minimum critical acceleration (or factor of safety) value, and affects the ratio of the displacement of the two bodies. In particular, it holds that

$$u_i / u_{i+1} = \cos(\delta_i + \alpha_{i+1}) / \cos(\delta_i + \alpha_i) \qquad (1)$$

where u_i is the displacement of body i along the slip sub-plane, α_i is the angle, positive anti-clockwise from the horizontal to the slip sub-plane of body i, and δ_i is the angle, positive clockwise from the vertical to the internal surface between bodies (i) and (i+1).

As permanent displacement accumulates, (a) the nodes of the ground surface of the slide at body (i) are displaced by u_i, where u_i is the displacement of the block where the node is located and (b) mass transfer is simulated. In this way, as the slide progresses, the lowest block possesses increasingly larger mass and the upper block increasingly smaller mass than originally.

A special case, that is referred to the toe problem (Sarma and Chlimintzas, 2001), occurs when the slope is such that the increased soil mass that corresponds to the lowermost block cannot maintain contact with the rest of the material and is subsequently detached from the system. As illustrated in Fig. 4 this occurs when the angle α_1 representing inclination of the first (downhill-most) slip surface segment, is less than the angle α_0 representing the slope of the free ground surface immediately preceding the first block of the system.

Below, first the Sarma and Chlimintzas (2001) multi-block model is applied at a dike that suffered liquefaction-induced large displacement to back-estimate the residual strength. Then, the correlation of c_u and $\mathbf{N_{1\text{-}60}}$ of this case, and previous cases found in the bibliography and estimated using the improved models described above are analyzed.

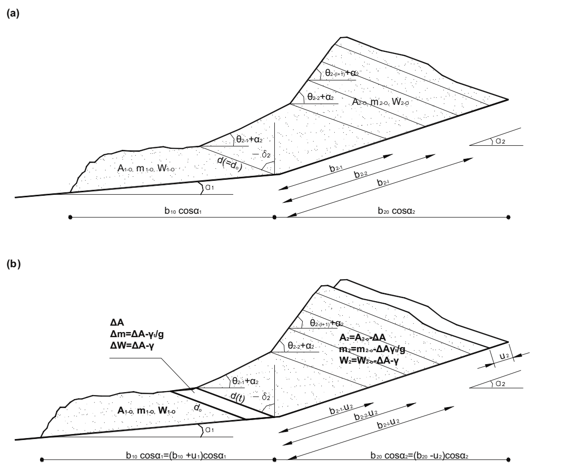

Figure 1. The 2-body sliding system considered by Stamatopoulos et al (2000): (a) Initial position, (b) position when the distance moved by the second body is u_2 .

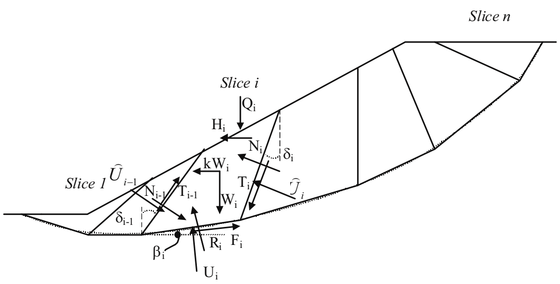

Figure 2. The multi-block stability method proposed by Sarma (1979).

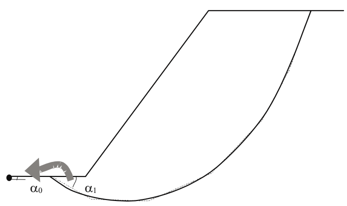

Figure 3. The toe problem (Sarma and Chlimintzas, 2001).

2. THE DIKE AT KABUTONO BANK OF THE SHIRIBESHI-TOSHIBETSU RIVER (KANEKO ET AL, 1995)

The Nansei-Oki earthquake of 1993 had magnitude 7.8. The dike at the Kabutono Bank of the Shiribeshi-toshibetsu river was located about 75kM from the epicenter. Fig. 4 sketches the geometry of the dike before and after the earthquake and the geotechnical profile below the dike. The dike suffered large displacement presumably due to liquefaction of the loose sand layer 2.3m below ground level. The fines content of this layer is not given.

The applied maximum acceleration value was of the order of 0.15 to 0.20g. Minimum acceleration to cause liquefaction was 0.126g (Kaneko et al, 1995). It is inferred, according to Eurocode (European Prestandard, 1994), that $N_{1\text{-}60}$ is about 8.

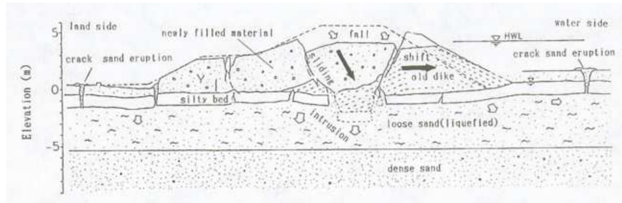

Figure 4. The dike that suffered large displacement during the Nansei-Oki earthquake of 1993 (Kaneko et al, 1995).

3. ANALYSIS OF THE KINEMATICS OF THE FAILURE OF THE DIKE

As Fig. 4 indicates, the kinematics of the dike are well-defined: A slip surface with inclination about 60^o was formed at the dike. The central part of the dike moved downwards along this slip surface and pushed horizontally the outer part of the dike. The depth of the slide is also known: It is located at the top of the loose layer that liquified, or at depth of 2.3m.

As the bottom of the slide is below the ground surface, a three-body model is needed to simulate the slide movement: Body 3 corresponds to a slip sub-plane with inclination of 60^o in the dike, body 2 to the horizontal slip sub-plane at the top of the liquified layer, 2.3m below surface and body 1 is needed in order the slip surface to reach the ground surface.

The extent of body 2 towards the ground free surface, the inclination of body 3 and the inclination of the two internal slip surfaces are not known. They were obtained according (a) to the theory of limit equilibrium, which states that they must correspond to the minimum value of the critical acceleration (Sarma, 1979) and (b) the kinematical restriction that the internal slip surface between bodies 1 and 2 should be at least 2m away in the horizontal direction from the base of the embankment (in order that the movement of body 2 horizontally by 2m is not restricted). Parametric analyses illustrated that under these conditions, the inclination δ_1, the extent of body 2 and the inclination of body 3 are as shown in Fig. 5. In addition, for these parameters, as illustrated in Fig. 6, analysis gave that the minimum critical acceleration in terms of the inclination to the vertical between the second and third bodies, δ_2, occurs at δ_2 =-30^o. The values of soil strength used in the analyses are similar to those back-estimated below. The resulting complete initial geometry is given in Fig. 5.

The multi-block model described above was then used to predict the deformed geometry of the slide when, the displacement of the upper slip surface is 2m. According to the previous discussion, it is assumed that when the sliding mass reaches level ground, it does not translate upwards, but is dispersed on level ground. The obtained deformed configuration is given in Fig. 5. It can be observed that the computed deformation of the dike agrees with the measured deformation. In particular, equation (1) for δ_2 =-30^o predicts u_2/u_1=1, that is in complete agreement with the measured ratio of u_2/u_1.

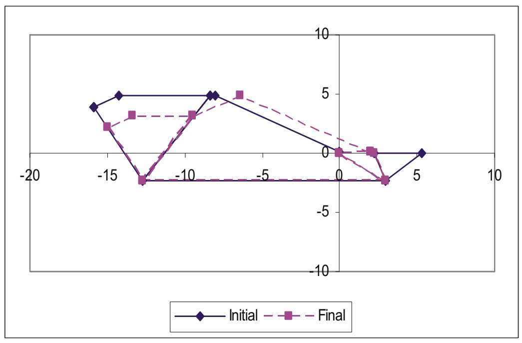

Figure 5. Simulation of the geometry of the slide of Fig. 4 with the 3-body model of Sarma and Chlimintzas (2001): The initial configuration and the configuration when the upper part of the slope moves 2m.

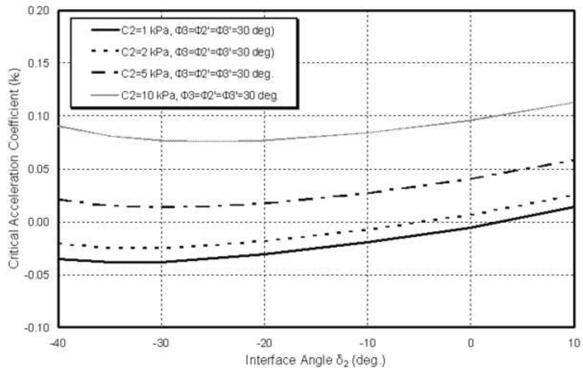

Figure 6. The critical acceleration in terms of the interface angle δ_2 for the initial configuration of Fig. 5.

4. BACK-ESTIMATION OF THE RESIDUAL SHEAR STRENGTH

Based on the analyses of the kinematics of the slide, above, the three-body model of Fig 5 was used to back-estimate the residual strength of the slide.

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 2 t/m^3. Above the water table, a residual friction angle of 30^o and zero cohesional resistance is assumed. Using back analysis, the sand strength c_u of the soil below the water table that corresponds to the measured seismic displacement was estimated.

Different accelerograms, normalized at the maximum accelerations of 0.15 to 0.20g were applied to investigate the effect of the applied accelerogram on the back-figured undrained soil strength. The following accelerograms covering a wide range of fundamental period and earthquake duration were considered:

- Port Island (Kobe, Japan), 17/1/1995, component East-West at depth 16m., t (duration)=40s, T_f (=fundamental period)=0.7 s.
- El-Centro (California, USA), 18/5/1940, component North-South, t=50s, T_f =0.6 s.
- San Fernando - Avenue of Stars (California, USA), 1971, component East-West, t=40s, T_f=0.15 s.
- Kalamata (Greece), 13/9/1986, Municipality Building, longitudinal component: t=25s, T_f=0.35s
- Gazli (former USSR), 17/5/1976, t=10s, T_f=0.1 s.

For all cases of applied accelerogram, the back-estimated c_u was obtained as 3 to 4kPa. The corresponding critical acceleration value was about zero.

5. RELATIONSHIP BETWEEN c_u AND $N_{1\text{-}60}$

The Olson et al (2000) and Stamatopoulos et al (2000) methods have been used to back-estimate the residual soil strength, c_u, of several liquefaction-induced slides. Table 1 gives the slides studied, their references, their characteristics and their ($N_{1\text{-}60}$, c_u) back-estimated pairs. For the Marquessa Dam, the same values of $N_{1\text{-}60}$ and fines content are given for the upstream and downstream slopes. They are the average values all over the dam.

Fig. 7 plots the ($N_{1\text{-}60}$, c_u) pairs of both all cases of table 1 and the current case study. It can be observed that (a) much scatter exists, (b) generally, c_u increases with $N_{1\text{-}60}$ and (c) this increase is about linear.

It can be assumed that when the $N_{1\text{-}60}$ value is very small, or at the limit, zero, the c_u value is very small, or, at the limit zero. It is inferred that a reasonable form of the relationship between cu and $N_{1\text{-}60}$ is

$$c_u \text{ (kPa)} = A\, N_{1\text{-}60} \qquad (2)$$

where A is a model parameter. The parameter A was estimated from linear regression, and found equal to 1.35. The coefficient of correlation R^2 is 0.02, or very small. Closer inspection of the data reveals that generally c_u for given $N_{1\text{-}60}$ value decreases as the content of fines increases. This agrees with what (a) the theory of the critical state, (b) the commonly-used relationship between the N value of the SPT and the relative density of sand and (c) the observation that the critical state line depends on fines content predict.

Indeed, according to critical state theory, c_u is related to the sand void ratio, the parameters defining the critical state line of the sand and the steady-state friction angle of the sand, φ_{cs}. The factor φ_{cs} does not vary considerably from sand to sand (e.g. Bouckovalas et al, 2003). The parameters defining the critical state depend on the fines content, f (Bouckovalas et al, 2003). The void ratio depends on the relative density of the sand and its maximum and minimum void ratios, e_{max} and e_{min}. The relative density can be related to N_1, e_{max} and e_{min} (Cubrinovski and Ishihara, 2002). In addition, e_{max} and e_{min} can be related to the fines content (Cubrinovski and Ishihara, 2002).

From all the above it is inferred that (a) the relationship between $N_{1\text{-}60}$ and c_u depends on the percentage of fines and (b) a reasonable form of this relationship is

$$c_u \text{ (kPa)} = N_{1\text{-}60}\,(a + b\, f) \qquad (3)$$

where a and b are model parameters and f is the percentage of fines.

Fig. 8 plots the data of table 1, in order to estimate the parameters a and b. Only slides where data on the content of fines is available are presented. The parameters a and b are estimated as 3.0 and 0.05 respectively. The coefficient of correlation R^2 is 0.50. This value is not too bad considering the uncertainty and variation of $N_{1\text{-}60}$ measurements in soil mechanics.

6. CONCLUSIONS

The conventional sliding-block model has shortcomings in back-estimating the soil strength of liquefaction-induced slides when displacement is large. Recently 2-block and multi-block models have been developed that simulate the change in geometry. The multi-block model was used to analyze the kinematics of the liquefaction-induced large deformation that a dike at the Kabutono Bank suffered during the Nansei-Oki earthquake of 1993 and to back-estimate the residual soil strength. The deformed slide configuration that the model predicted agreed with the measured.

Then, the correlation of c_u and $N_{1\text{-}60}$, of this case, and all previous cases analyzed by these improved models are collected and analyzed. The data illustrates that the relationship depends a lot on the fines content. A relationship is derived using linear regression.

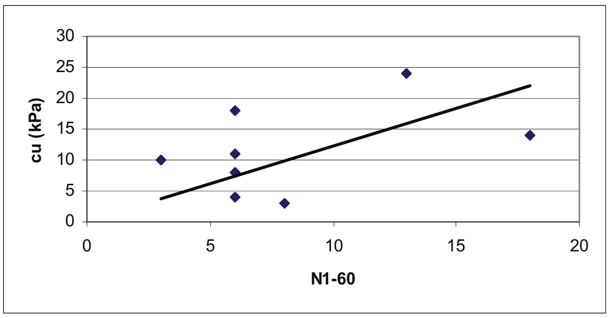

Figure 7. The back-estimated c_u versus $N_{1\text{-}60}$ for the cases of table 1.

Table 1. Case studies that have been analyzed and results.

	Characteristics of the case study			Results of back-analysis	
Case	Reference	N_{1-60}	% of fines	Reference	c_u (kPa)
Marquesa Dam Downstream	4	6	25	18	8
La Palma Dam	4	3	15	18	10
Rimnio embankment	19	18	40	18	14
King Harbor Mole	7	6	≈5	16	11
Marquesa Dam, upstream	4	6	25	16	8
Lower San Fernando Dam	13	13	≈25	16	24
Wachesett Dam	9	6	≈ 8	9	18
Kushiro embankment	6	6	?	17	4

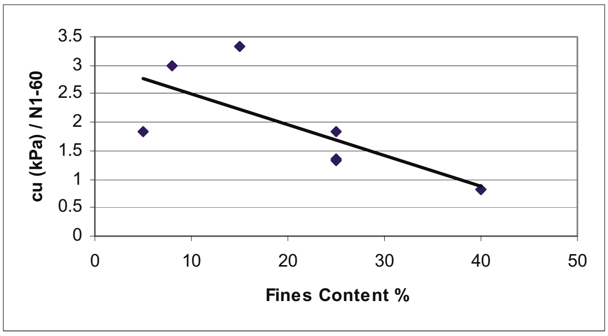

Figure 8. The c_u / N_{1-60} ratio versus fines content for the cases of table 1

ACKNOWLEDGMENT

This work was supported by the General Secretariat of Research and Technology of Greece (Project no DP23).

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