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The Role of Slope Geometry on Flowslide Occurrence

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Abstract: The paper reports a study aimed to the prediction of susceptibility to flowslide of granular soil slopes as a consequence of the in situ state of stress. In particular, the slope geometry has been investigated as a factor influencing the initial state of stress. For this purpose the results of numerical models, performed by a finite different approach (FLAC), allowed the complete definition, in any point of the slope, of the stress conditions by changing slope height and inclination. By relating this state of stress to parameters used to describe potential for liquefaction of loose granular soils a chart of instability has been set up.

Keywords: Slope, flowslide, numerical modelling

INTRODUCTION

The prediction of the occurrence of flowslides has received great interest over the last decade, because these phenomena have a great potential for destruction.

Flowslides are slope movements caused by (sudden) liquefaction of a loosely packed saturated sand mass that flows out into a more gentle slope[1].

Nevertheless the term liquefaction is here used to indicate all phenomena involving excessive deformation in saturated cohesionless soils and is not limited to the development of 100% excess pore pressure, as defined by Uthayakumar and Vaid[2].

Static liquefaction is associated with sand deforming in a strain softening (or limited strain softening) manner that results in limited or unlimited unidirectional flow deformation.

Controlled laboratory tests supply fundamental understanding of the undrained response of sands.

Ishihara[3] showed that the degree of fabric dependence changes significantly with the level of shear strains to which the sand is deformed. When the sand is deformed largely to reach the Steady State (SS), the effects of fabric disappear and the behavior is determined only by the void ratio irrespective of the initial confining stress. When the shear strain is moderately large, producing the Quasi Steady State (QSS) with a minimum strength, the fabric dependency is significant. In this range of strain the initial confining stress (consolidation state of stress) is an important parameter. The QSS condition corresponds to a temporary drop in shear stress and is governed by the void ratio and confining stress at the time of consolidation and could occur only in loose samples sheared from large initial confining stress.

Soil instability is a phenomenon that resembles liquefaction in that there is a sudden decrease in the soil strength under undrained conditions. This loss of strength is related to the development of large pore pressures reducing effective stresses in the soil. Lade[4] showed that there exists a region of instability inside the failure surface. The loss of strength occurs in undrained condition as a consequence of disturbances small but fast enough to prevent water drainage.

In a recent work Yamamuro and Lade[5] showed that both phenomena can lead to flowsliding, but the mechanism of generation is quite different. On the basis of a series of triaxial ICU tests on loose sand samples they showed that liquefaction (zero effective stress) is a low pressure phenomenon and that the effect of increasing the confining pressure was to increase the resistance to liquefaction. Instability occurs at stresses far greater than those in the liquefaction region. The soil exhibits increasing contractiveness with increasing confining pressure. These different behaviours occur because the stress required for temporary instability is much higher than those in the liquefaction and temporary liquefaction regions. This corresponds to a change in the mechanism of volumetric contractiveness from grain arrangement to particle breakage. At high pressure particle crushing should be the predominant mechanism of volumetric contraction in the sand[5].

Olson[6] and Olson and Stark[7] correlated the liquefied shear strength (minimum strength in undrained condition) to the prefailure vertical effective stress, due to the difficulty of determining in situ principal stresses, to back analyse several flowslide case histories. They showed that an approximately linear relationship exists between the liquefied shear

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strength and weighted average prefailure vertical effective stress for liquefaction flow failures, despite differences in density, mode of deposition, grain size distribution, grain shape, state parameter, mode of shear and steady state friction angle of the liquefied soils.

In a recent work Ishihara *et al.*[8] presented the results of a series of laboratory tests, using triaxial apparatus, on saturated samples of Toyoura sand consolidated anisotropically. They found that with an increasing degree of anisotropy at the time of consolidation the sample becomes more contractive and susceptible to triggering flow failure. They found that the major effective principal stress at the time of anisotropic consolidation is a parameter controlling dilative or contractive behaviour of the sand. As a result the most appropriate way to normalise the residual strength of anisotropically consolidated sand is by the use of major principal stress at consolidation. The quasi steady state strength is then a function of void ratio and the major effective stress at consolidation. Figure 1 reports the final results of this study.

The results of the studies reported above show that liquefaction occurrence is strongly influenced by the consolidation state of stress. This means that liquefaction of granular slopes is influenced by the prefailure state of stress in the slope.

It is well known that the behaviour of soils is non linear and path dependent. The study of slope behaviour should take into account the history of slope formation by considering evolving strains and the path dependency by means of appropriate constitutive models. Other factors influencing slope stability are pore pressures and geometry.

Conventional slope stability analysis methods (limit equilibrium methods) are widely used to investigate landslide problems and to determine the state of stress in slopes. In particular several studies of slope instability are based on the assumption of infinite slope. In the context of flowslides several complex constitutive models have been proposed and used to study the stability conditions of slopes[9,10]. The assumption of infinite slope leads to a resulting state of stress independent by the slope height and by local changes in slope profile. As a consequence it is not possible to localise the extension of the zone of the slope subjected to instability.

In this paper attention is given to the influence of slope geometry to the flowslide occurrence. To take into account this aspects it is necessary to assess initial or prefailure state of stress in finite slopes.

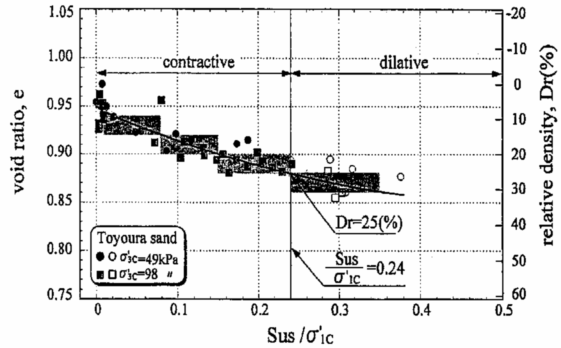


Fig. 1: Void ratio versus normalised residual strength (after Ishihara *et al.*[8])

For this purpose a series of numerical models of finite slopes have been set up, by using a finite difference code (FLAC).

The non linearity of soil behaviour and the history of slope formation have not been considered because these aspects are beyond the scope of this study.

The model results have been used to analyse the potential for flowsliding of sand slopes of different geometry, by changing height and inclination. The prediction of flowsliding occurrence in sand slopes is then performed by using the correlation between the quasi steady state strength and the major effective stress at the end of consolidation proposed by Ishihara *et al.*[8].

Numerical modelling of finite slopes: The dependence of the potential for liquefaction from the in situ state of stress has been investigated by calculating the state of stress in all zones of finite slopes. For these purposes numerical models reproducing different slopes have been set up by using a finite difference code (FLAC Itasca, ver 4[12]). The state of stress in slopes has been evaluated in both elastic and elastic-plastic field.

The analysis does not considers the causes that can lead to failure and subsequently to the loss of strength due to the transformation from drained to undrained conditions (from sliding to flow), but has the main aims to investigate the .geometry dependency of the slope.

Hence the numerical simulations considered drained conditions, as the purpose of this phase of the work was to assess the initial (prefailure) state of stress, representative of the state of stress at the end of consolidation in triaxial tests. For the sake of simplicity the majority of analyses has been set up considering fully submerged slopes .

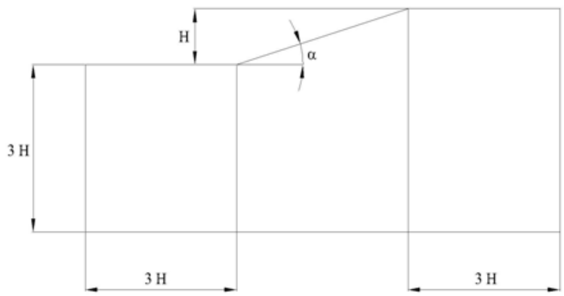


Fig. 2: Size of the slopes used in the numerical analysis

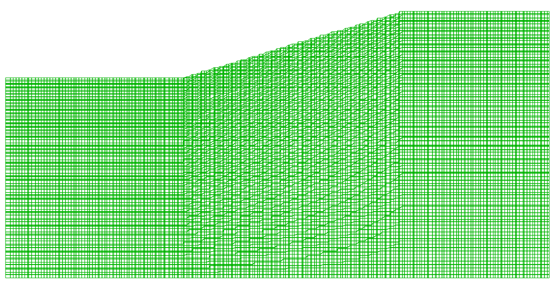


Fig. 3: Example of grid used in the numerical model

The numerical modelling has been performed in the following steps:

* Geometric generation of the model and discretisation of the domain in a finite difference grid;
* Definition of boundary conditions;
* Definition of physical and mechanical properties of the soil;
* Consolidation under gravity in elastic field (by considering a isotropic linear elastic behaviour), in order to obtain the convergence of the initial conditions;
* Implementation of elastic-plastic behaviour;
* Analysis of the state of stress in the grid points of the model.

The slope angle α was varied from 0° to 30°; the slope height from 3 m to 50 m. In order to minimise boundary effects, all the models have been generated according to the scheme reported in Fig. 2.

The physical and mechanical parameters of the sand have been assumed on the basis of the available data of Toyoura sand[3,8,11]. The steady state angle of the

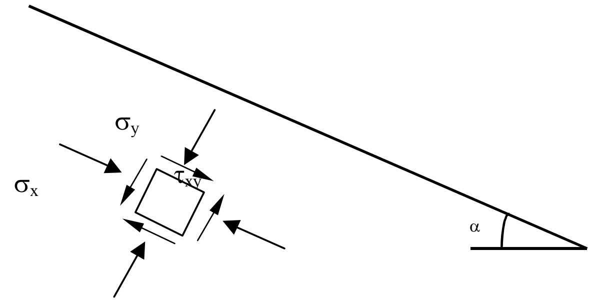


Fig. 4: Stress in an infinite slope

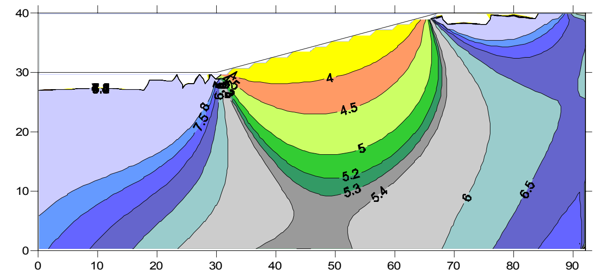


Fig. 5: State of stress in a finite slope (α=15°, H=10 m) in terms of apparent slope angle. The curves represent the equal values of cot α_a.

sand has been assumed ϕ'_{ss}=30°. A non-associated flow rule with Mohr Coulomb failure criterion has been implemented.

The dependence of plasticity localisation on grid shape and size was investigated by a series of preliminary analyses. The better results, by considering also the calculation time, were obtained with a squared grid of size of 50*50 cm. Figure 3 shows the grid configuration used in numerical models.

RESULTS AND DISCUSSION

The numerical model results in terms of stress state have been represented in any point of the slope in terms of an apparent slope angle α_a of an infinite slope at a depth y_a[1].

This can be done by calculating (Fig. 4) σ'_y and τ'_{xy} in the plane parallel to the apparent slope angle α_a.

$$\sigma'_y = \gamma' y \cos\alpha \tag{1}$$

$$\tau'_{xy} = \gamma' y sen\alpha \tag{2}$$

By replacing y by y_a and α by α_a one obtains:

$$y_a = \frac{\tau'_{xy}}{\gamma' \cdot \sin\alpha_a} \tag{3}$$

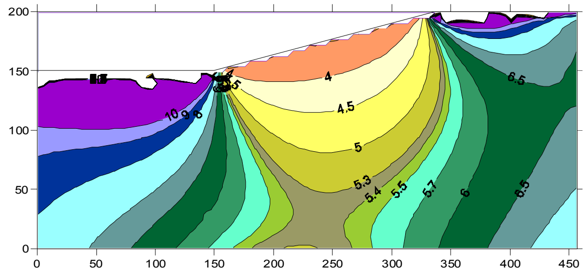


Fig. 6: State of stress in a finite slope (α=15°, H=50 m) in terms of apparent slope angle. The curves represent equal values of cot α_a

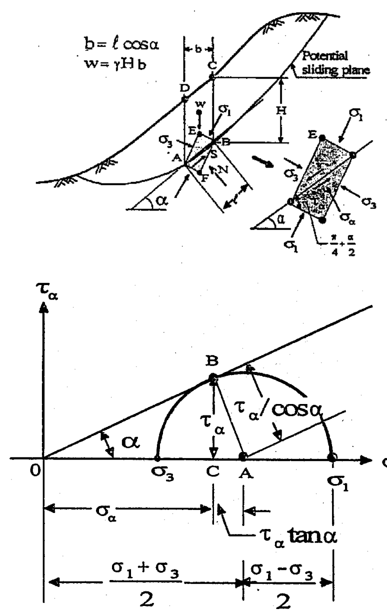
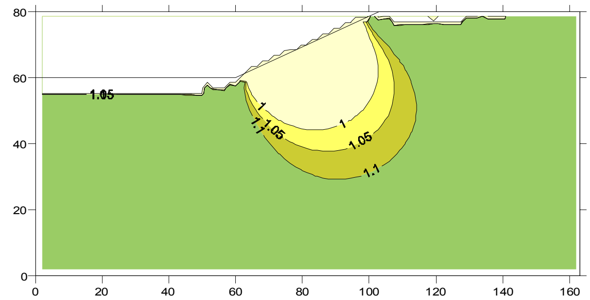


Fig. 7: Soil element of the slope and correspondent Mohr circle (after Ishihara *et al.*[8])

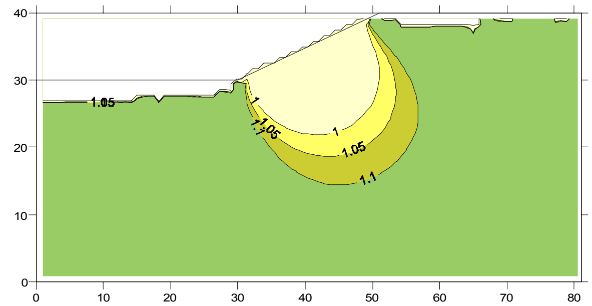
$$\cot\alpha_a = \frac{\sigma'_y}{\tau'_{xy}} \tag{4}$$

The diagrams reported in Fig. 5 and 6 are obtained by imposing this condition.

The results reported in these figures supply useful information. As expected the apparent slope angle decreases with increasing depth. For example in the case of a finite slope 10 m high with an angle α=15° (Fig. 5) at a depth of about 25 m, measured perpendicularly from the slope face, the inclination and then the stress condition is equivalent to an infinite slope with an inclination of about 11°. This condition is achieved at a depth of about 125 m when the finite slope is 50 m high. Therefore the same condition in terms of apparent slope angle is encountered at a distance of two times and half the height of the slope.



(a)



(b)

Fig. 8: Safety factor against flowsliding in a finite slope. The curves represent the equal values of Fs. a) α=25°, H=20 m; b) α=25°, H=10 m

The slope height can be considered a scaling factor for the pre-failure state of stress. If the slope height is doubled the pre-failure state of stress is twice, at twice the depth. Furthermore mean stress and deviator stress are twice as high.

These results have been used to analyse the susceptibility to flowslide in any point of the slopes, according to the findings of Ishihara *et al.*[8]. They evaluated the safety factor for different slope inclinations and void ratios by comparing the residual strength Su_s with the gravity-induced shear stress τ_d, (Fig. 7):

$$F_s = \frac{Su_s}{\tau_d} \tag{5}$$

The residual strength is represented by the quasi steady state strength (QSS) of the sand, which is dependent on the major effective principal stress at the time of anisotropic consolidation (Fig. 1), with χ depending on the initial void ratio of the sand:

$$S_{us} = \chi\sigma'_{1c} \tag{6}$$

The principal stress ratio K_c is then calculated as:

$$K_c = \frac{\sigma'_3}{\sigma'_1} = \frac{1-\sin\alpha}{1+\sin\alpha} \tag{7}$$

where α is the angle of stress mobilisation or angle of obliquity of stress application.
The maximum deviator stress corresponds to:

$$\tau_d = \gamma H sin\alpha \quad (8)$$

Finally the safety factor against liquefaction is calculated as:

$$F_s = \chi \frac{1 + sin\alpha}{sin\alpha} \quad (9)$$

The safety factor reported in equation (9) depends on the slope angle and in limit equilibrium analysis is not influenced by the depth of the potential sliding plane.

This approach has been applied to the numerical modelling results by calculating the safety factor with equation (9) in all the gridpoints substituting the stress mobilisation angle α with the apparent slope angle α_a previously determined.

Figure 8 shows curves of equal value of the safety factor for the case of a sand soil void ratio e=0,88, corresponding to χ=0,24, for slopes with inclination of 25° and 20 m and 10 m high. As in the case of the initial state of stress the effect of slope height is significant. In particular slope height strongly affects the volume of soil potentially involved in flowsliding.

The results obtained from all the performed numerical simulations has been used to predict flowsliding initiation on the basis of critical combinations of void ratio, slope angle and slope height. Figure 9 reports curves that establish the limit condition of no flowsliding for a large range of situations.

Previous considerations have shown the importance of taking into account the actual geometry (both inclination and height) of the slope, as a factor that not only can explain the susceptibility of the slope to flowslide but also as a factor that allows the assessment of the volumes of soil potentially involved in the instability phenomenon. This last aspect plays an important role in the hazard analysis of slopes.

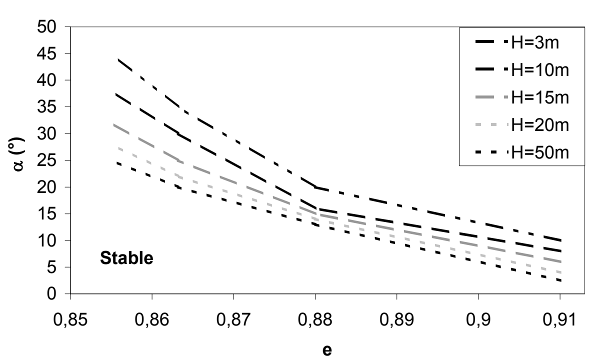


Fig. 9. Critical combination of slope height, slope angle and void ratio on flowslide occurrence

REFERENCES

1. Stoutjesdijk T.P., M.B. de Groot and J. Lindenberg, 1998. Flowslide prediction method: influence of slope geometry. Canadian Geotechnical J., 35: 43-54.
2. Uthayakumar, M. and Y.P. Vaid, 1998. Static liquefaction of sands, under multiaxial loading, Canadian Geotechnical J., 35: 273-283.
3. Ishihara, K. 1993 Liquefaction and flow failure during earthquakes. Rankine Lecture Geotechnique, 43: 351-415.
4. Lade, P., 1992 Static instability and liquefaction of loose fine sandy slopes. J. Geotechnical Eng. Div. ASCE, 118: 51-71.
5. Yamamuro, J. and P. Lade, 1997 Static liquefaction of very loose sands. Canadian Geotechnical J., 34: 905-917.
6. Olson, S.M., 2001, Liquefaction analysis of level and sloping ground using field case histories and penetration resistance. PhD. Thesis, University of Illinois, Urbana.
7. Olson, S.M. and T.D. Stark, 2002. Liquefied strength ratio from liquefaction flow failure case histories. Canadian Geotechnical J., 39: 629-647.
8. Ishihara, K., Y. Tsukamoto and T. Shibayama, 2003. Evaluation of slope stability against flow in saturated sand. Reports on Geotechnical engineering, Soil mechanics and Rock engineering, Jubilee volume of Terzaghi Brandl 2000. Wien, 2000-2001, Vol. 5, Institut fur Grundbau und Bodenmechanik-Technische Universitat Wien Ed., pp: 41-54.
9. Di Prisco, C., R. Matiotti and R. Nova, 1995. Theoretical investigation of the undrained stability of shallow submerged slopes. Geotechnique, 45: 479-496.
10. Laouafa, F. and F. Darve, 2002. Modelling of slope failure by a material instability mechanism. Computers and Geotechnics, 29: 301-325.
11. Cubrinovski, M. and K. Ishihara, 2000. Flow potential of sandy soils with different grain compositions. Soils and Foundations, JGS, 40: 103-119.
12. FLAC Manuals, 2001. Version 4, ITASCA Consulting group, Minneapolis, USA.