PORE PRESSURE EFFECT ON SEISMIC RESPONSE OF SLOPES

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SUMMARY

Seismic slope stability is a topic of great interest in geotechnical and geoenvironmental engineering. To evaluate the stability conditions and the serviceability of a slope after a seismic event a displacement analysis should be performed. In this paper an extension of Newmark's sliding block is presented; the slope stability is evaluated taking into account the earthquake induced pore pressure on saturated cohesionless soil. The effect of shear strength reduction on seismic displacement is evaluated and some useful stability charts are provided to predict the occurrence of liquefaction landslides and to evaluate the influence of soil relative density and static hydraulic conditions on seismic slope response. Applying the proposed model a parametric analysis has been performed considering both harmonic and real accelerograms in order to point out those parameters which affect the system response.

INTRODUCTION

The prediction of earthquake induced landslides and the analysis of permanent seismic displacement can be used for many types of seismic hazard analysis and for earthquake design of engineered slopes.

Usually the analysis of seismic response has been based on Newmark's sliding block model [Newmark, 1965]; this simple model, although approximate, gives much more information than the classical pseudo-static analysis and is more practical than the FEM analysis; consequently, it is widely used to predict permanent displacement under seismic loading. Sarma [1975], has performed the first attempt to evaluate the earthquake-induced pore pressure effect on seismic stability of dams and embankment; except for a few number of works relative to cohesive soil slopes [Lemos et al.,1985,1994, Crespellani et al.,1996, Cascone et al.,1998] traditionally, in the displacement analysis the reduction of shear strength and the other aspect of soil cyclic behaviour have not been taken into account. However the experience of last 20 year on earthquake geotechnical engineering has shown that seismic response of slopes is affected by many factors: seismic amplification, pore pressure build-up due to the undrained conditions of seismic loading, liquefaction of cohesionless soil, cyclic degradation of shear strength in cohesive soil. Recently [Biondi, 1998] a model for the assessment of the effect of earthquake-induced pore pressure on seismic stability of sandy slopes has been developed. The model is capable to take into account the static pore pressure distribution under given hydraulic conditions and the pore pressure build-up due to cyclic loading caused by an earthquake. For cohesionless soil the reduction on effective stress due to the earthquake-induced pore pressure may produce a significant reduction on soil shear strength. Depending on the combination of the time history of induced shear stress, soil relative density and static effective stress, failure may occur in the soil mass. Biondi et al. [1999] have shown that for cohesionless slope with translational failure mechanism the conventional approach for liquefaction analysis (zero effective stress) is unconservative. However, even if the failure condition is not reached, seismic displacements of the slope after the earthquake are strongly affected by the reduction of the available shear stress along the sliding surface. In this paper for infinite slope of cohesionless saturated soil, the classical stability equations are rewritten to take into account the induced pore pressure effect on seismic response. Applying the proposed model a parametric analysis has been proposed and some useful stability charts have been developed to predict seismic and post-seismic stability on the basis of slope initial

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stability condition and soil relative density. To evaluate the pore pressure effect on seismic response of slopes an extension of Newmark's sliding block model is proposed. Displacement analysis shows that seismic stability of saturated cohesionless slopes may be greatly underestimated if pore pressure build-up is neglected in the analysis.

SEISMIC STABILITY OF INFINITE SLOPE

In the stability analysis of infinite slopes the potential failure plane is taken parallel to the slope profile. For saturated soil a steady seepage is assumed to take place in a direction parallel to the slope. The slope safety factor is computed as the ratio between the resisting and the driving forces acting on the potential failure surface. In figure 1 the geometry of the slope and the stress condition along the sliding surface are shown. If H is the depth of the failure plane and β is the slope angle, the static stress condition are given by the following expression:

$$\sigma_o = \gamma_t \cdot H \cdot \cos^2\beta \qquad \sigma'_o = \gamma_t \cdot H \cdot \cos^2\beta \cdot (1 - r_u{}^\circ)$$
$$\tau_o = \gamma_t \cdot H \cdot \cos\beta \cdot \sin\beta \qquad r_u{}^\circ = \eta \cdot \frac{\gamma_w}{\gamma_t} \qquad [1]$$
$$u_o = \gamma_w \cdot \eta \cdot H \cdot \cos^2\beta$$

where γ_t is the unit weight of soil and $r_u°$ is the static pore pressure ratio.

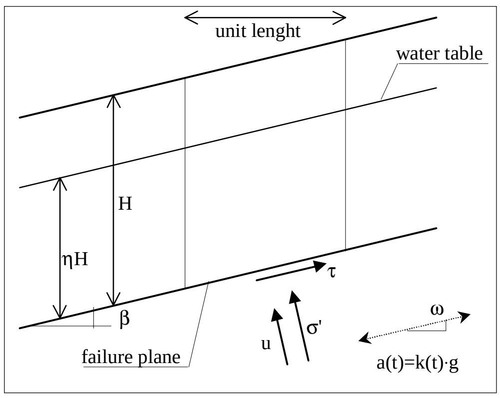

Fig. 1 - Infinite slope.

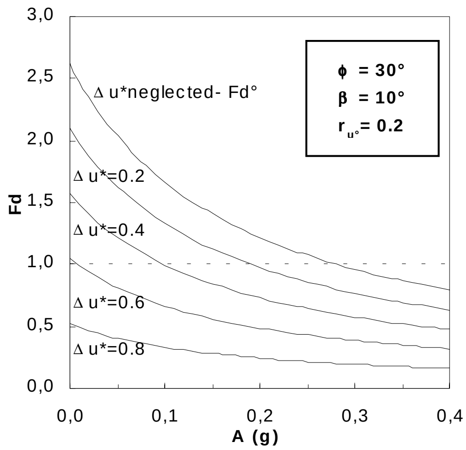

Fig. 2 - Seismic slope safety factor.

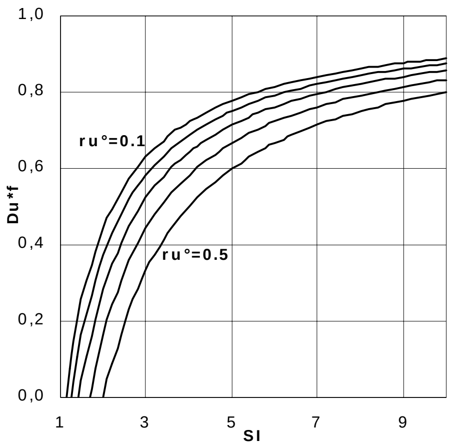

Fig. 3 - Induced pore pressure required for slope failure.

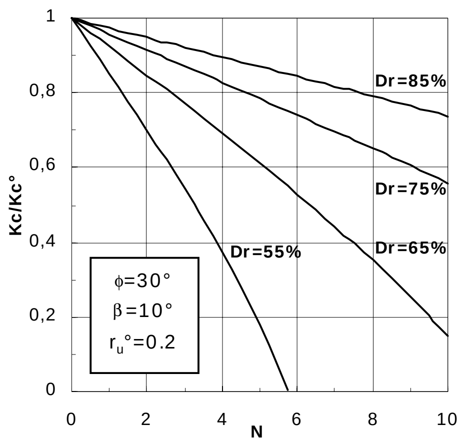

Fig. 4 - Cyclic degradation of slope critical acceleration.

Using the Mohr-Coulomb failure criterion the available shear stress on the failure plane is: $\tau_f° = c' + \sigma' \cdot \tan\phi$, where c' and ϕ' are the effective cohesion and the effective angle of shear strength.
For cohesionless soil the static slope safety factor can be expressed as follows:

$$F_s = \frac{\tau_f{}^\circ}{\tau_o} = \frac{\tan\phi'}{\tan\beta} \cdot (1 - r_u{}^\circ) = SI \cdot (1 - r_u{}^\circ) \qquad [2]$$

where SI is the stability index (SI=tan ϕ'/tanβ) and represent the safety factor in static condition for a dry slope. During a seismic event slope stability is affected by two kind of factor: the inertial effect of seismic forces and the reduction of shear stress due to the cyclic degradation. The slope response depends on which of these factors prevails. If the shear strength of the soil remains relatively constant during the seismic excitation, permanent displacement may occur for temporary excedence of the strength by the earthquake-induced shear stresses. On the contrary weakening instabilities may occur if the soil shows unstable behaviour under dynamic induced shear stress: in sandy saturated slopes the most common causes of weakening instabilities are flow liquefaction and cyclic mobility. Referring to figure 1 and assuming the direction of ground acceleration parallel to the slope ($\omega=-\beta$), the stress condition during the seismic event is given by the following expressions:

$$\begin{aligned} &\sigma(t) = \gamma_t \cdot H \cdot \cos^2\beta && u(t) = u_o + \Delta u(t) \\ &\tau(t) = \tau_o + \tau_d = \gamma_t \cdot H \cdot \cos\beta \cdot (\sin\beta + K(t)) && \sigma'(t) = \sigma(t) + u(t) \end{aligned} \quad [3]$$

where K(t) is the seismic acceleration expressed as a percentage of gravity accelerations and $\Delta u(t)$ is the time history of pore pressure build-up due to the undrained condition imposed by the seismic excitation; $\Delta u(t)$ depends on the shear stress time history at any point of the slope, on soil relative density and on effective stress condition before the earthquake.

The slope safety factor changes during the seismic event as follows:

$$F_d(t) = \frac{\tau_f(t)}{\tau(t)} = \frac{\cos\beta \cdot (1 - r_u°) \cdot (1 - \Delta u^*(t))}{\sin\beta + k(t)} \cdot \tan\phi' \quad [4]$$

From equation [4] it is apparent that seismic slope stability is mainly affected by the inertial effect of seismic excitation and the shear strength reduction caused by the pore pressure build-up. $\Delta u^*(t)$ is the earthquake induced pore pressure ratio: $\Delta u^*(t) = \Delta u(t)/\sigma'_o$.

In the classical pseudo-static stability analysis the seismic safety factor is evaluated referring to the maximum ground acceleration and neglecting the shear strength reduction:

$$F_d° = \frac{\cos\beta \cdot (1 - r_u°)}{\sin\beta + k_{max}} \cdot \tan\phi' \quad [5]$$

As shown in figure 2 when significant induced pore pressure develop, stability analysis must be performed taking into account the reduction in shear strength; only if the shear strength reduction is negligible the classical seismic safety factor $F_d°$ applies. However [Biondi, 1998], especially when seismic response of saturated soils is characterised by cyclic degradation, slope stability must be evaluated taking into account the earthquake induced pore pressure and its time history. To take into account the shear strength reduction on seismic response a Newmark's sliding block analysis can be performed evaluating the slope's critical acceleration from equation [4] by imposing $F_d(t)=1$ and solving for $K_c(t)$:

$Kc(t)=\cos\beta\cdot\tan\phi\cdot (1-r_u°)\cdot (1-\Delta u^*_{(t)})-\sin\beta$ [6]

Because of the pore pressure build-up and, consequently, the reduction of effective stresses condition, the critical acceleration of a slope, as shown by equation [6], is not a constant but it decreases during the earthquake excitation. This reduction is a function of the initial stability condition of the slope (SI, $r_u°$) and of the time history of pore pressure build-up. Only if the induced pore pressure are negligible and the shear strength soil remains relatively constant, the displacement analysis may be carried out referring to the initial value of critical acceleration:

$Kc°=\cos\beta\cdot\tan\phi\cdot (1-r_u°)-\sin\beta$ [7]

INDUCED PORE PRESSURE EFFECT ON SEISMIC STABILITY

For cohesionless soil the increase in pore pressure causes a significant reduction of shear strength; in relation to the soil properties, particularly to relative density, and to the effective stress condition before the earthquake, the induced pore pressure may lead soil to liquefaction. Biondi et al. [1999] had shown that for lateral spreading failures the conventional criterion to evaluate liquefaction induced damage (zero effective stress state: $\Delta u^*(t)=1$) is unconservative. In fact, because of the shear stress state already acting in static condition, the slope critical acceleration reaches the zero value prior to achieving zero effective stress. In particular this happens when Δu^* is:

$\Delta u^*_f=1-1/F_s$ [8]

In this condition, the post-seismic factor is unity:

$$F_d^{PS} = \frac{\cos\beta \cdot (1 - r_u°) \cdot (1 - \Delta u^*_f)}{\sin\beta} \cdot \tan\phi' = 1 \quad [9]$$

that is, the slope reaches a limit equilibrium condition, regardless the presence of the seismic excitation.

If the induced pore pressure reaches the value Δu^*_f , large permanent displacement may occur, because of the driving effect of gravity acting on the soil mass with reduced shear strength: the slope shows a failure mechanism typical of liquefaction landslides. Figure 3 shows the induced pore pressure required for slope failure for several hydraulic conditions and for different values of stability index. The Δu^*_f value for a given slope stability condition (SI, $r_u°$) is a characteristic of the slope and it is not affected by the earthquake-induced stress state; for this reason it may be assumed as a useful parameter for seismic and post-seismic stability analysis.

Known the earthquake-induced pore pressure Δu^*, each of the lines plotted in figure 3 represents, for a given slope stability condition, a limit value: if $\Delta u^* \geq \Delta u^*_f$ failure occurs; if $\Delta u^* < \Delta u^*_f$, however, seismic slope response depends on shear strength reduction and a potential displacement analysis should be performed to evaluate the slope's serviceability after the seismic event. In order to evaluate the increase in pore pressure due to a cyclic loading on saturated sand, usually analytical relationship, based on experimental result, are used.

The Seed and Booker's [1977] relationship is one of the most accurate; recently Coumoulos and Bouckovalas [1996] have proposed a modification of this relationship for use in connection with the experimental data by De Alba et al. [1976]; the proposed relationship is:

$$\Delta u^*(N) = \frac{2}{\pi} \cdot \sin^{-1}\left[N^{1/2a} \cdot \sin\left(\frac{\pi}{2} \cdot \Delta u^*_1 \right) \right] \qquad \Delta u^*_1 = C_! \cdot \left(\tau_d{}^* \right)^{C2} \cdot D_r{}^{C3} \tag{10}$$

where Δu^*_1 is the induced pore pressure after the first cycle, D_r is the soil relative density, a is an empirical coefficient, $\tau_d{}^*$ is the ratio between the amplitude of shear stress applied in the test and the initial effective normal stress, C_1, C_2, C_3 are numerical constant. The empirical coefficient "a" is assumed 0.7 [Seed et al. 1976; using the experimental data by De Alba et al, [1976] obtained for D_r varying in the range 54%÷90%, the numerical constant are: C_1=2.7, C_2=2.78, C_3=-4; these value will be adopted in the analysis.

Using equations [1],[2], the cyclic stress ratio $\tau_d{}^*$ may be evaluated as follows:

$$\tau_d{}^* = \frac{\tau_d}{\sigma_o} = \frac{K}{\cos\beta \cdot (1 - r_u°)} \tag{11}$$

As shown by Seed et al. [1975] by means an appropriate weighting procedure the effect of irregular time history of earthquakes-induced shear stress on pore pressure build-up can be represented, with a reasonable degree of accuracy, by an equivalent number of uniform stress N_{eq}; following this approach and using the depth reduction factor r_d by Iwasaki et al. [1978], the induced pore pressure will be expressed by:

$$\begin{aligned} &\Delta u^*(N) = \frac{2}{\pi} \cdot \sin^{-1}\left[N^{1/2a} \cdot \sin\left(\frac{\pi}{2} \cdot \Delta u^*_1 \right) \right] && \tau_d{}^* = \frac{\tau_d}{\sigma_o} = \frac{K}{\cos\beta \cdot (1 - r_u°)} \\ &\Delta u^*_1 = 2{,}7 \cdot \left(\tau_d{}^* \right)^{2{,}78} \cdot D_r{}^{-4} && \Delta u^*_{max} = \Delta u^*(N_{eq}) \end{aligned} \tag{12}$$

As shown by equation [6], the increase on pore pressure causes a reduction on slope critical acceleration; figure 4 shows the cyclic degradation of critical acceleration due to the pore pressure build-up caused by a sinusoidal excitation of amplitude K=0.1 and for several values of soil relative density.

It may be useful to evaluate the induced pore pressure value Δu^*_d for which the critical acceleration of an initially stable slope ($K_{max}<K_c°$) is reduced to the maximum value of the earthquake acceleration:

$$\Delta u^*_d = 1 - \frac{K_{max} - \sin\beta}{\cos\beta \cdot \tan\phi' \cdot (1 - r_u°)} = 1 - \frac{1}{F_d°} \tag{13}$$

in a Newmark's sliding block analysis Δu^*_d represents the value for which permanent displacements will occur. Figure 5 shows the variation of Δu^*_d with initial condition (SI, $r_u°$) for ϕ'=30° and for several values of the seismic coefficient. For a given slope (SI) with assigned hydraulic condition ($r_u°$) subject to a seismic excitation (K), if the induced pore pressure is less than the corresponding Δu^*_d, the slope critical acceleration will remain larger than the seismic acceleration and, thus, seismic displacement will not occur. Conversely, if the induced pore pressure is large than Δu^*_d the slope will experience permanent displacements.

In figure 6, for $\phi'=30°$ and for several value of static pore pressure ratio, some seismic stability charts are shown: the dashed line represents Δu^*_f obtained from equation [8], while the continuous line represents Δu^*_d obtained from equation [13]. For given values of SI and $r_u°$ and for a chosen value of the design seismic coefficient, using the stability charts the two limit value of Δu^* can be evaluated representing one of the following conditions:

$\Delta u^*_{max} \leq \Delta u^*_d$: no seismic displacements occur;

$\Delta u^*_{max} \geq \Delta u^*_f$: the slope reaches a failure condition:

$\Delta u^*_d \leq \Delta u^*_{max} \leq \Delta u^*_f$: displacement analysis should be performed.

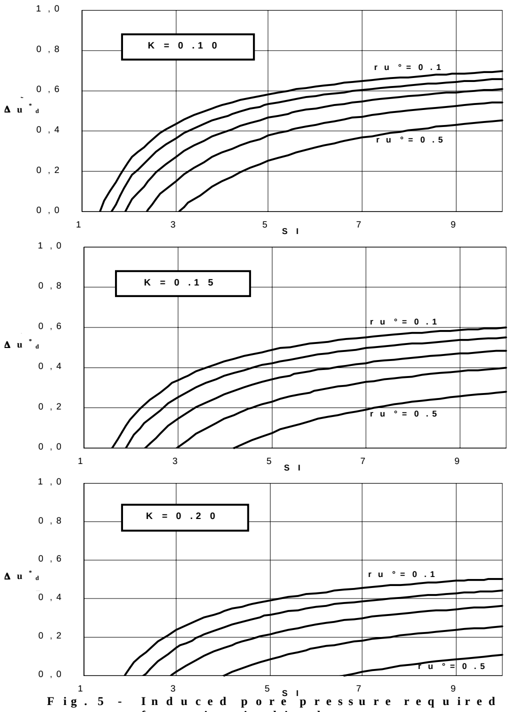

Fig. 5 - Induced pore pressure required for seismic displacement.

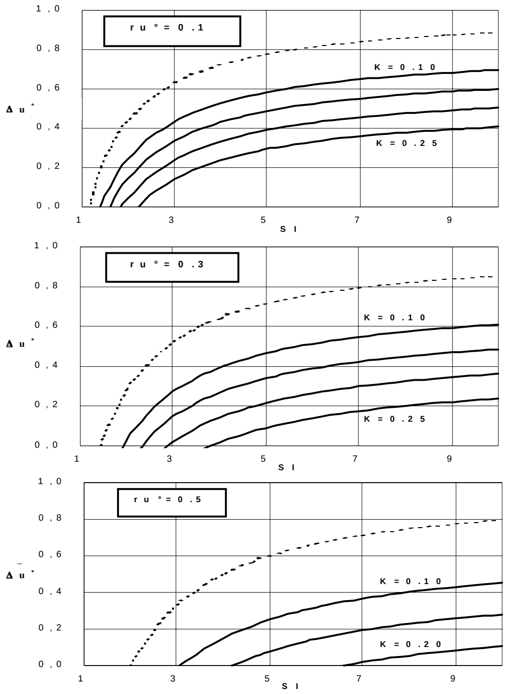

Fig. 6 - Seismic stability charts.

EFFECT OF INDUCED PORE PRESSURE ON SEISMIC DISPLACEMENT

In Order to evaluate the effect of shear strength reduction on permanent displacements of a slope is necessary to known the time history of the pore pressure build-up. Following Seed et al. [1975] the number of equivalent cycles at $\tau=0.65\tau_{max}$ may be evaluated on the basis of earthquake shear stress time history, neglecting the cycles

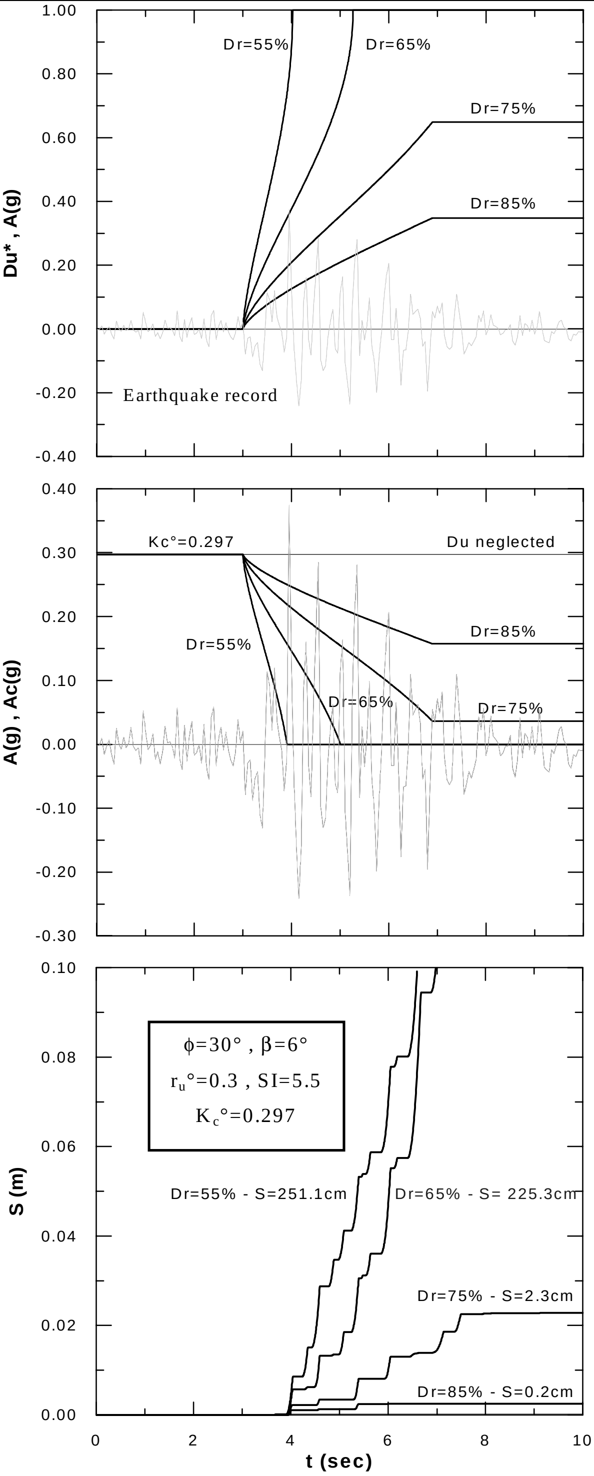

Fig. 7 - Effect of soil relative density on seismic slope response.

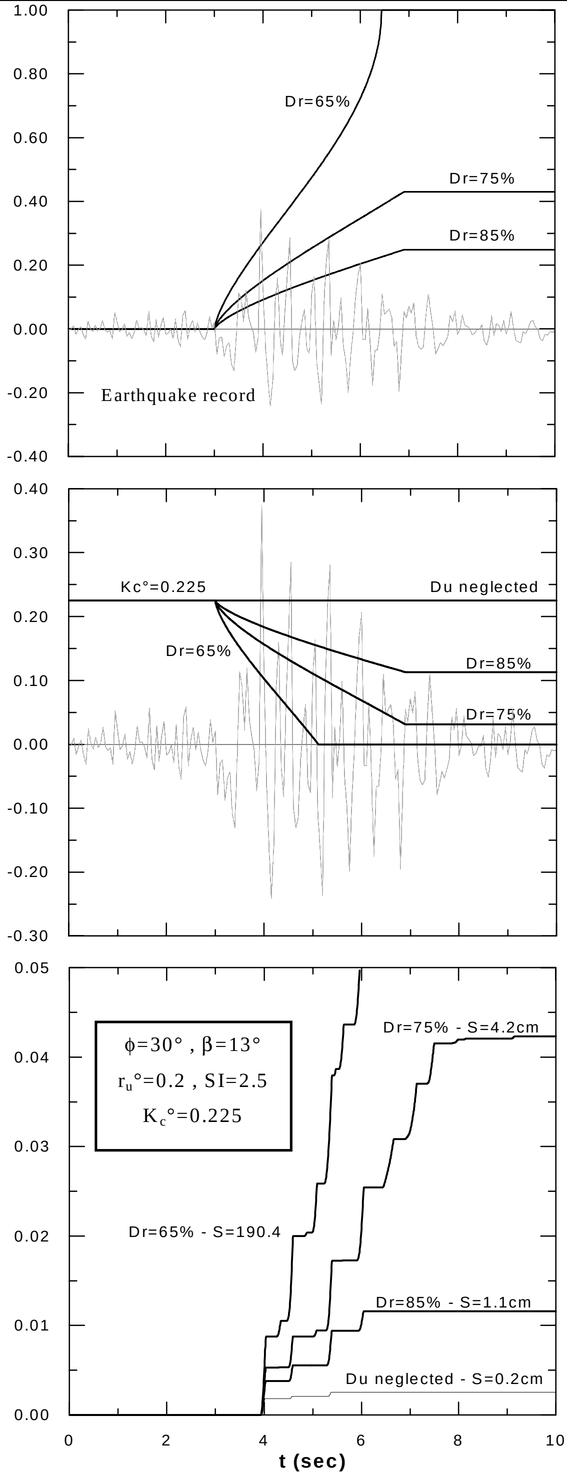

Fig. 8 - Effect of soil relative density on seismic slope response.

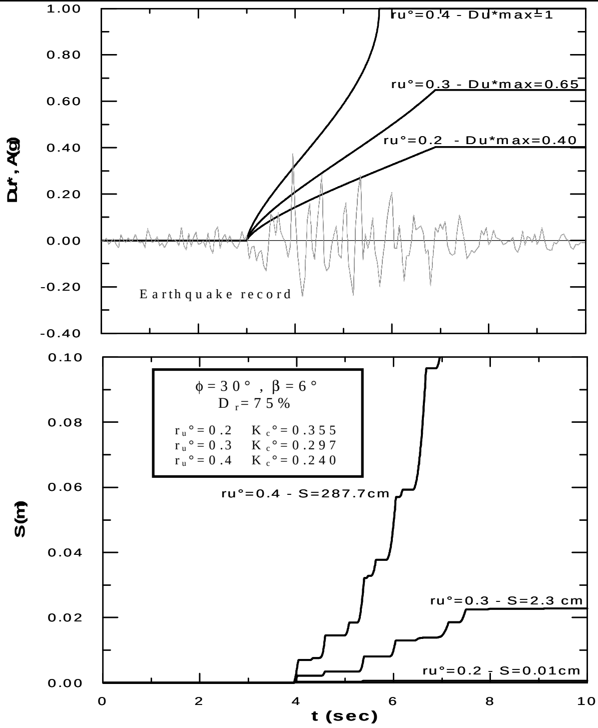

Fig. 9 - Effect of slope hydraulic condition on seismic response.

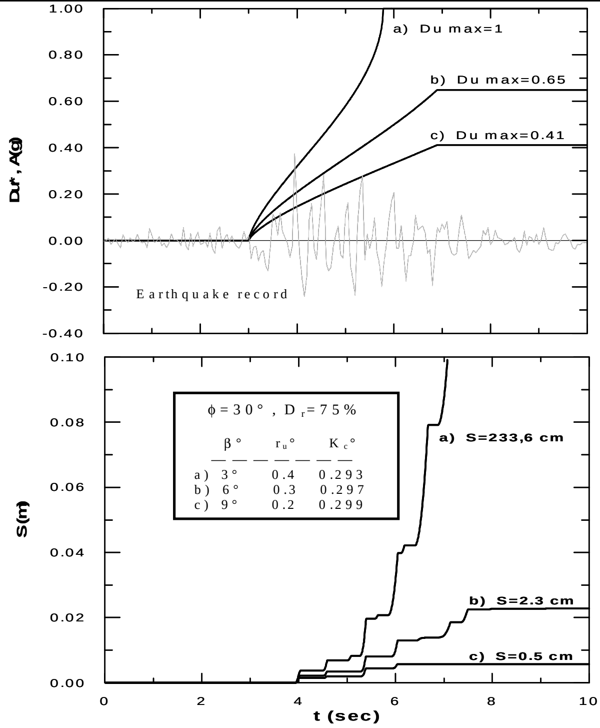

Fig. 10 - Effect of shear strength reduction on seismic response.

at stress level lower than $0.30\tau_{max}$; more simply is possible to compute N_{eq} on the basis of the acceleration time-history. Following this approach it will be assumed that the pore pressure build-up develops only in the portion of the earthquake time-history in which the acceleration results larger than 30% of the maximum acceleration A_{max}; if liquefaction occurs a constant value a constant value ($\Delta u^*=1$) of induced pore pressure will be assumed.

With this assumption a parametric analysis has been performed using the accelerogram of the 1976 Friuli earthquake (M=6.4) recorded at Tolmezzo (A_{max}=0.37g) evaluating the depth reduction factor r_d referring to a failure sliding plane with H=20m. Figures 7 and 8 show the effect of soil relative density on seismic response: loose sand slopes reaches the failure condition with a small number of cycles and the corresponding displacement time-history is clearly influenced by the occurrence of liquefaction. For dense sandy slopes the increase in pore pressure is small, however seismic displacement are strongly affected by pore pressure build-up. As it is apparent shown in figure 8, if pore pressure build-up is neglected in the displacement analysis, seismic slope response may be greatly underestimated.

Figure 9 shows the seismic response of the same slope evaluated for several value of static pore pressure ratio: $r_u°$ influences the initial value of slope critical acceleration ant the pore pressure build-up (equation [12]), therefore the same slope may exhibit different seismic behaviour in relation to its static hydraulic condition. Figure 10 shows seismic response of different slopes with the same initial value of critical acceleration but with different values of $r_u°$. If the increase in pore pressure in neglected in the analysis, the slopes will accumulate the same permanent displacements. If the reduction on shear strength is taken into account, as shown in figure 10, the slopes shows different seismic response and, in relation to the initial effective stress state, the failure condition may be reached.

CONCLUDING REMARKS

This works describes a method for the assessment of seismic stability of sandy saturated slopes. The classical stability equation are rewritten taking into account the earthquake induced pore pressure effect on seismic stability of slopes. The pore pressure build-up is evaluated using an accurate empirical relationship based on experimental results. The main peculiarity of this work consists in a first attempt to consider the effect of hydraulic slope condition and soil relative density on seismic response, evaluating the shear strength reduction due to pore pressure build-up. Some useful stability charts are derived in order to evaluate the seismic and post-seismic stability of sandy saturated slopes. Displacement analysis shows that, when there is significant increase in pore pressure, stability analysis can not be performed referring to the classical pseudo-static safety factor or to the initial value of slope critical acceleration. In fact, this latter value decreases during the earthquake excitation because of the shear strength reduction, so displacements may occur also for slope initially stable for a given seismic acceleration. A good prediction of seismic slope response should be performed referring to the limit values of induced pore pressure Δu^*_f , Δu^*_d.

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