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ABSTRACT

Seismic slope stability analysis is a topic of great interest in geotechnical and geoenvironmental engineering particularly in seismic a in fact the occurrence of earthquake induced landslides is documented in many recent post-earthquake damage report (Japan 1993-1 Greece 1995, Turkey 1999). Generally saturated slopes of loose sand or silty-sand and earth dams and embankment resting on l cohesionless soil deposit are highly susceptible to liquefaction-induced damage and during strong earthquake several landslides cause soil liquefaction may occur. In this paper a numerical model to evaluate seismic response of submerged cohesionless slopes is descri Slope stability conditions are evaluated taking into account the inertial effect of seismic forces and the earthquake induced pore press which reduce soil effective stress state. Displacement analysis has been performed using an extension of Newmark's sliding block m for rotational failure mechanism and taking into account the reduction of slope critical acceleration due to changes in pore pressu Applying the proposed model a numerical analysis has been performed in order to point out those parameters which mostly affect sei slope response and some useful stability charts are provided.

INTRODUCTION

The stability of continental shelf and slopes is a topic of great interest in the field of near shore and off-shore constructions, particularly in seismic areas; in fact the occurrence of earthquake induced landslides on those geotechnical structures is documented in many post-earthquake damage report.

During recent large earthquake in Japan (1993, 1995) and Turkey (1999) settlements and lateral spreading in the order of several hundreds of centimetres, and consequently several damage on structures has been observed. As a result of the 1995 Hyogo-ken Nambu earthquake, wide spread occurrence of lateral spreading of the ground, resulting from soil liquefaction, has been observed (Ishihara, 1997); in the 1999 Kokaeli earthquake there were several sites of land loss into the sea along south coastal line of the Izmit Bay which appeared to have been caused by submarine landslides (Ishihara et al, 2000). During the Kozani-Grevena earthquake (1995) damages were observed to the Rimnio bridge embankment; lateral spreading was induced by liquefaction of the silty sand layer where the embankment was founded and the observed horizontal displacement was 0.8 to 2.0m (Tika & Pitilakis, 1999). Generally saturated slopes of loose sand or silty-sand and earth dams and embankments resting on loose cohesionless soil deposit are highly susceptible to liquefaction-induced damage and during strong earthquake several landslides caused by soil liquefaction may occur.

Based on this observation a numerical model has been developed to evaluate seismic and post-seismic behaviour of cohesionless saturated slopes with rotational failure mechanism: slope stability conditions are evaluated taking into account the inertial sei effects and the earthquake induced pore pressures which re the soil effective stress state and consequently affect the s stability conditions.

Both for translational and rotational failure mechanism a rigo solution to evaluate seismic behaviour of natural slopes is no available because of the great number of parameters which c play an important role on seismic slope response. Conseque simplified method based on approximate models are c preferred to more rigorous methods which are less practical more time-consuming. Thus seismic slope response is curre performed referring to Newmark's sliding block model; simple model, although approximate, gives much n information than the classical pseudo-static analysis and requ a smaller modelling effort than FEM analysis, allowing to pre the occurrence of permanent displacement under seismic loa For the case of translational failure mechanism the sliding b model was firstly extended by Sarma (1975) to include the e of the cyclic pore water pressure changes; recently Krame Arduino (1999) have shown the importance of pore w pressure changes on permanent seismic displacement for ge slopes using an energy-based model and different soil constit laws. Moreover for gentle slopes with translational fa mechanism, Biondi et al. (1999 a) have shown that earthqu induced pore pressure undoubtedly affect slope stability cond and play an important role in the accumulation of perma displacements. Particularly for cohesionless soils, whicl undrained conditions show liquefiable behaviour, a signifi reduction in shear strength may occur depending on sev

factors: the initial effective stress state, the earthquake induced shear stress time history and the soil relative density. A stability analysis taking into account all these factors should thereby be performed; in fact even for gentle slopes the failure condition may be achieved with large displacements (Biondi et al. 1999 b).
In this paper a displacement analysis has been performed using an extension of Newmark's sliding block model for rotational failure mechanism and taking into account the reduction in shear strength due to the pore pressure build-up. The proposed model is capable to point out the influence of the above mentioned parameters which affect seismic and post-seismic slope response in terms of permanent displacements.

GENERAL ASSUMPTIONS

The static and dynamic stability analysis is based on the assumption of plane strain condition; the potential failure surface and the slope safety factor, both for static and seismic conditions, are evaluated considering for the soil a plastic behaviour and the Mohr-Coulomb failure criterion. Figure 1 shows the geometrical scheme of the slope considered in the analysis. The phreatic surface is defined by Dupuit's formula for two fixed points (a,b) represented in figure 1.
As shown by Spencer (1978) and Chang et al. (1984) the failure mechanism described by a circular surface is the most critical for slopes subjected to lateral acceleration; following this result a circular failure surface is assumed in the analysis and the slope is supposed to rotate, as a rigid block, along it.
During seismic excitation the slope safety factor, evaluated as the ratio between the resisting and the overturning moment, changes due to the inertia forces induced by seismic excitation and to cyclic degradation of soil shear strength. A more reliable criterion to evaluate seismic stability and post-seismic slope serviceability is provided by potential permanent displacements which mainly depend on the seismic acceleration time-history and on the slope critical acceleration, i.e. the seismic acceleration that brings the soil mass to a limit equilibrium condition. This parameter is a more realistic index of slope seismic stability and, consequently, the critical failure surface is evaluated searching for minimum slope critical acceleration.

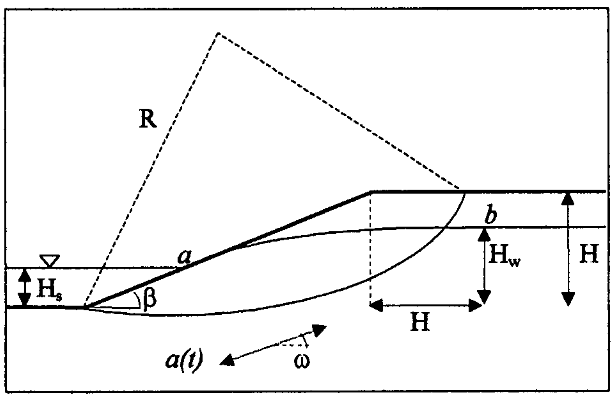

Fig. 1. Schematic of slope.

SEISMIC STABILITY CONDITION

Slope stability is evaluated using the Bishop's modified method extended to take into account pseudostatically the soil inertia effect due to the seismic excitation; however, any other slice method can be used consistently. Usually the earthquake induced acceleration is considered to be horizontal; however several studies show that, both for circular and translational failure mechanism, a stability analysis in which the vertical component of seismic acceleration is neglected may result unconservative. Accordingly an acceleration vector inclined of an angle ω to the horizontal was applied in the centre of gravity of each slope slice. Referring to figure 1 the slope static safety factor F_s and the slope seismic safety factor $F_d°$ may be evaluated using the following expressions:

$$F_s = \frac{\sum_{i=1}^{n}\left[(W_i - u_i \cdot b_i)\tan\phi'\right]\cdot M_i}{\sum_{i=1}^{n}\left[W_i \cdot \sin\alpha_i\right]} \qquad (1)$$

$$F_d° = \frac{\sum_{i=1}^{n}\left[\left(W_i \cdot (1 - K_{Max,v}) - u_i \cdot b_i\right)\cdot \tan\phi'\right]\cdot (M_i)}{\sum_{i=1}^{n}\left[W_i \cdot \left((1 - K_{Max,v})\cdot \sin\alpha_i + K_{Max,h}\cdot \frac{d_i}{R}\right)\right]} \qquad (2)$$

$$M_i = (\sec\alpha_i)\Big/\left(1 + \tan\alpha_i \cdot \frac{\tan\phi'}{F}\right) \qquad (3)$$

where, for the i-th slice, M_i represents the iterative factor (evaluated for $F=F_s$ in equation (2) and for $F=F_d°$ in equation (3)), W_i represents the total weight, u_i represents the pore pressure value acting in static conditions, $c'=0$ and ϕ' are the Mohr-Coulomb shear strength parameters, b_i is the length of the slice base inclined of an angle α_i to the horizontal, $K_{max,h}$ and $K_{max,v}$ are the two components of the maximum value of earthquake acceleration *a(t)* (expressed as a fraction of gravity acceleration g), d_i is the vertical distance between centre of gravity of the slice and the centre of the circular failure surface having radius R. $F_d°$ represents the slope seismic safety factor obtained referring to the maximum value of the earthquake acceleration and neglecting the degradation of the soil shear strength.
During a seismic event the safety factor will vary with $K(t)=a(t)/g$ dropping below unity in the time intervals when the seismic acceleration exceeds the slope critical acceleration (Newmark, 1965). Equating to unity equation (2) and solving for K the following expression for the initial value of slope critical acceleration may be obtained:

$$k_{cr}° = \frac{\sum_{i=1}^{n} W_i \cdot (M_{i1} \cdot \tan\phi' - \sin\alpha_i) - u_i \cdot b_i \cdot \tan\phi' M_{i1}}{\sum_{i=1}^{n} W_i \cdot \left[\cos\omega \frac{d_i}{R} + \sin\omega \cdot (M_{i1}\cdot \tan\phi' - \sin\alpha_i)\right]} \qquad (4)$$

M_{i1} represents the iterative factor evaluated for $F_d°=1$.

PORE PRESSURE BUILD-UP

Saturated cohesionless soils subjected to a cyclic stress history may develop an increase in pore pressure; the pore pressure build-up depends on the rate of loading, on soil relative density and on effective stress state acting before cyclic loading. Field observations and many post-earthquake reports show that many aspects of seismic behaviour of natural slopes depends on soil behaviour during cyclic loading. As shown by many studies, the increase in pore pressures causes a significant reduction of shear strength in cohesionless soil. Depending on soil relative density, on initial effective stress state and on the amplitude of earthquake induced shear stresses, pore pressure build-up is likely to lead soil to liquefaction: many earthquake induced landslides have been triggered by this phenomenon. In this paper the pore pressure build-up effect on seismic slope response is evaluated using an analytical relationship based on experimental data. Recently, Coumoulos and Bouckovalas [1996] proposed a modification of the Seed and Booker's [1977] relationship for use in connection with some experimental data obtained by De Alba et al. [1976] in cyclic DS; the proposed relationship is:

$$\Delta u^*(N) = \frac{2}{\pi}\cdot sin^{-1}\left[N^{1/2a}\cdot sin\left(\frac{\pi}{2}\cdot\Delta u^*{}_1\right)\right] \quad (5)$$

$$\Delta u^*_1 = C_1\cdot\left(\tau_d^*\right)^{C2}\cdot D_r^{C3} \quad (6)$$

where $\Delta u^*(N)$ represents the induced pore pressure after N uniform cycles normalised with respect to the initial effective normal stress, Δu^*_1 is the induced pore pressure after the first loading cycle normalised with respect to the initial effective normal stress, D_r is the soil relative density, a is an empirical coefficient assumed 0.7, τ_d^* is the ratio between the amplitude of shear stress applied in the test (τ_d) and the initial effective normal stress, C_1, C_2, C_3 are numerical constant function of soil nature. Using the experimental data obtained by De Alba et al. [1976] for Dr varying in the range 54%÷90%, the numerical value of these constants are: C1=2.7, C2=2.78, C3=-4; these values will be adopted in the analysis. To apply this relationship to slope stability analysis the following assumptions are made:

– for the i-th slice the induced pore pressure Δu^*_1 and $\Delta u^*(N)$ are normalised with respect to the effective stress acting in static condition normally to the slope failure surface;

– for the i-th slice the earthquake induced shear stress τ_d is evaluated using the following expression (Seed & Idriss, 1971): $\tau_d = 0{,}65\cdot K_{Max}\cdot\sigma_V\cdot r_d$; where σ_V is the total stress acting, on each slice, normally to the slope failure surface and r_d is the depth reduction factor;

– τ_d^* is evaluated normalising τ_d with respect to the effective stress acting, in static condition, normally to the failure surface.

In order to evaluate the depth reduction factor the following expressions will be used (Crespellani et al. 1999):

$r_d = 1 - 0.00765\cdot Z$	for	$Z \le 9.15$ m
$r_d = 1.174 - 0.0267\cdot Z$	for	$9.15 < Z \le 23$ m
$r_d = 0.074\cdot Z$	for	$23 < Z \le 30$ m
$r_d = 0.5$	for	$Z > 30$ m

Following these assumptions and considering a sinusoidal excitation of N cycles, the slope seismic safety factor time history can be evaluated taking into account the earthquake induced pore pressure; for the N-th cycle $F_d(N)$ is calculated as:

$$F_d(N) = \frac{\sum_{i=1}^{n}\left[\left(W_i\cdot\left(1-K(t)_{,v}\right)-\left(u_i+\Delta u_i(N)\right)\cdot b_i\right)\tan\phi'\right]\cdot\left(M_i\right)}{\sum_{i=1}^{n}\left[W_i\cdot\left(\left(1-K(t)_{,v}\right)\cdot\sin\alpha_i + K(t)_{,h}\cdot\frac{d_i}{R}\right)\right]} \quad (7)$$

Equation (7) clearly shows the two factors which mainly affect seismic slope stability: the inertial effect due to seismic excitation and the shear strength reduction caused by the pore pressure build-up. Usually stability analysis is performed neglecting the latter aspect and evaluating slope stability condition referring only to the maximum ground acceleration (that is evaluating $F_d°$). Likewise, the critical acceleration coefficient at the N-th cycle can be expressed as follows:

$$k_{cr\,(N)} = \frac{\sum_{i=1}^{n} W_i\left(M_{i1}\cdot\tan\phi' - \sin\alpha_i\right) - \left(u_i + \Delta u_{i(N)}\right)b_i\cdot\tan\phi' M_{i1}}{\sum_{i=1}^{n} W_i\cdot\left[\cos\omega\frac{d_i}{R} + \sin\omega\cdot\left(M_{i1}\cdot\tan\phi' - \sin\alpha_i\right)\right]} \quad (8)$$

It is apparent from equation [8] that pore pressure build-up causes a reduction on slope critical acceleration; this reduction depends on the time history of earthquake induced shear stresses, on soil relative density and on slope initial hydraulic condition. Figure 2 and figure 3 show (for a slope with β=20°, H=20m, $H_w/H=0.8$, $H_s/H_W=0.5$) the cyclic degradation of slope critical acceleration and the corresponding pore pressure build-up obtained applying five cycles of a sinusoidal excitation with amplitude 0.25g and frequency 1Hz.

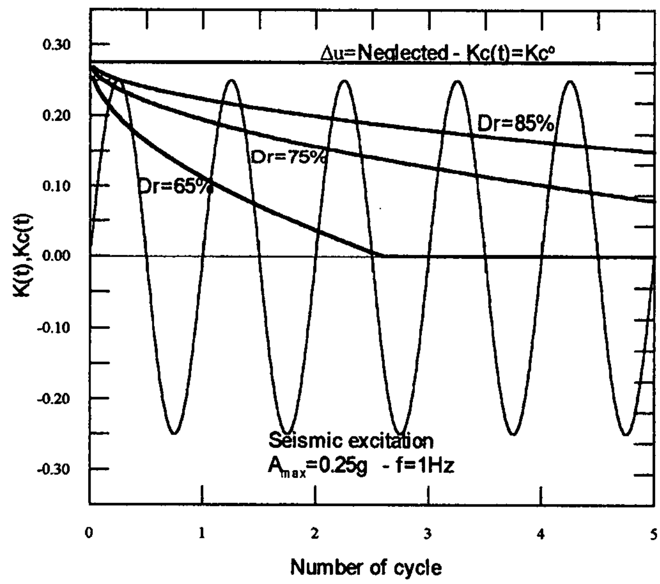

Fig. 2. Cyclic degradation of slope critical acceleration.

In figure 3, L represents the horizontal projection of the failure surface while x is the abscissa of the slice for which Δu/σ' is computed. The slices represented in figure 3 are placed respectively at 0.1L, 0.5L, 0.9L from the slope toe. As shown in figure 2 because of the shear strength reduction, slope critical acceleration is not a constant but it varies decreasing during seismic excitation. This result must be taken into account in the evaluation of slope stability when seismic response of saturated soils is characterised by a remarkable increase in pore pressures; only if the shear strength reduction is negligible slope stability analysis and permanent displacement assessment may be consistently carried out referring to the initial value of slope critical acceleration $K_c°$. In figure 4, for the same slope considered in figure 3 and for different values of soil relative density, the portion of potential failure surface in which the vanishing of effective stresses occurs is represented.

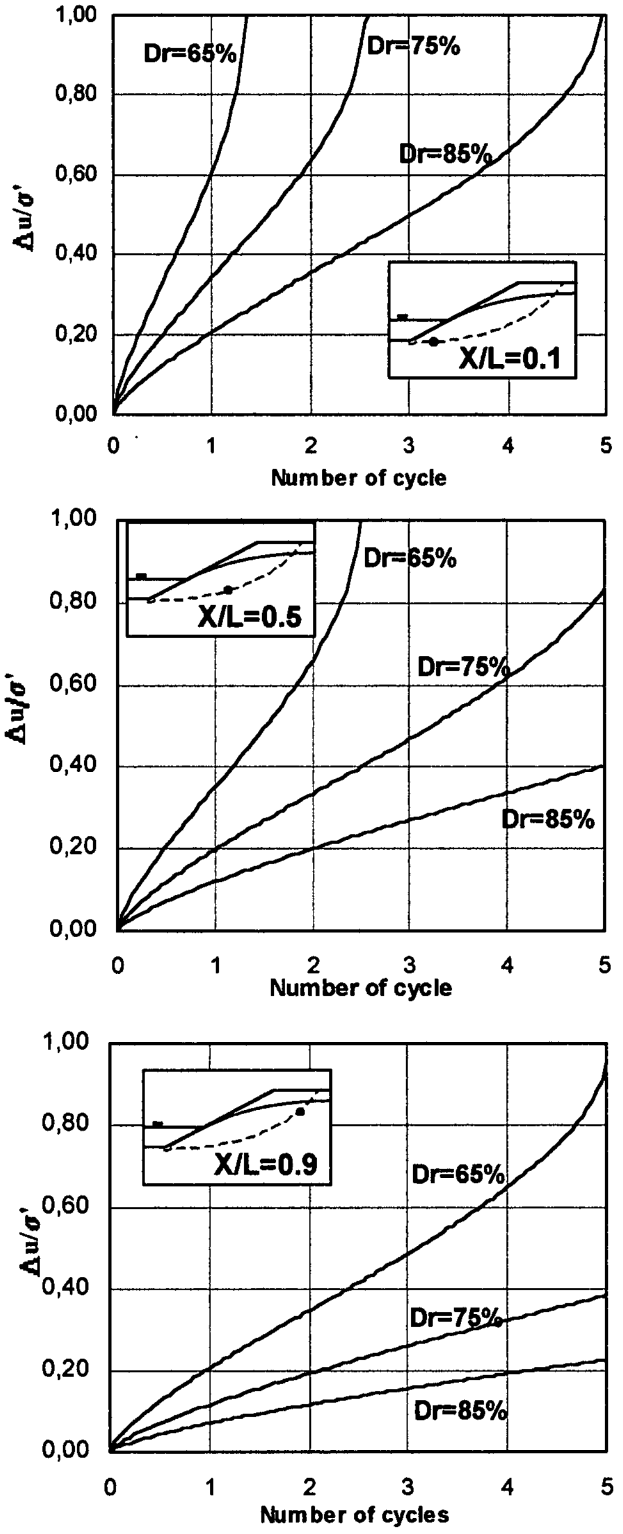

Fig. 3. Pore pressure build-up

It is evident that slight variations in soil relative density have a crucial effect on the results.

SEISMIC DISPLACEMENT ANALYSIS

During an earthquake slope stability conditions depend on the equilibrium between driving forces (or moments) due to the

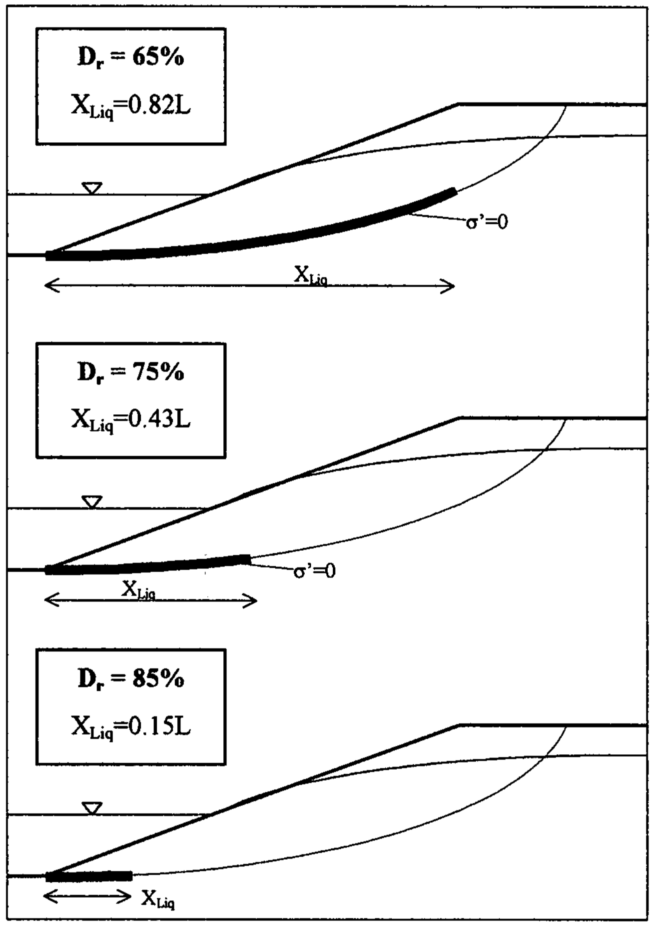

Fig. 4. Soil liquefaction along the potential sliding surface

forces acting in static conditions and the inertia forces induced by the seismic excitation and resisting forces (or moments) due to soil shear strength acting along the potential failure surface. As shown by Newmark (1965) the slope critical acceleration is a useful parameter to foresee seismic slope behaviour: when the seismic acceleration is greater than the slope critical acceleration permanent displacements occur as an effect of unbalanced forces (or moments). Following Newmark's approach and considering a slope with rotational failure mechanism, the dynamic equilibrium condition may be expressed as follows:

$$M_D(t) - M_R(t) = I \cdot \ddot{\vartheta}(t) \quad (9)$$

where M_D is the driving moment, M_R is the resisting moment, I is the mass moment of inertia of the potential sliding mass respect to the centre of the sliding surface and θ is the angular rotation of the sliding mass. Both the driving and the resisting moments vary with time; the former as a consequence of the seismic induced acceleration, the latter as a consequence of the possible reduction of soil shear strength due to pore pressure build-up. By double integration of equation (9) slope rotations are obtained; since the slope moves as a rigid block, permanent displacements of point lying along the failure surface are computed straightforwardly by multiplying rotations for the radius R of the failure surface.
Figure 5, for the same slope used for figure 2, shows the effect of soil relative density on slope permanent seismic displacement.

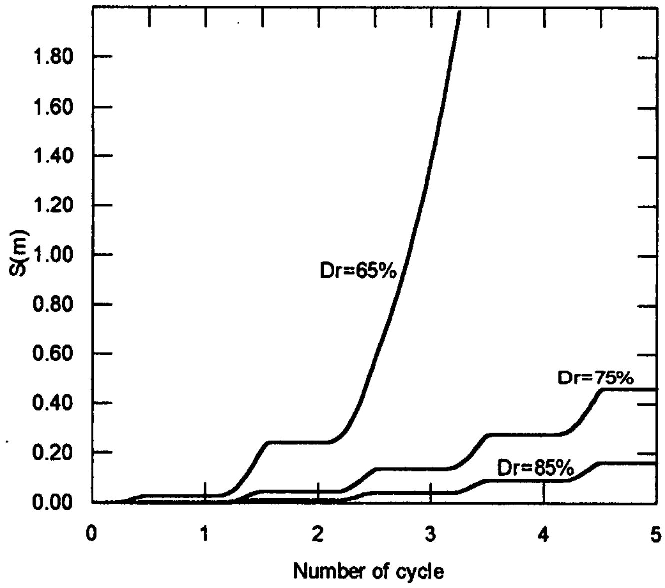

Fig. 5. Effect of soil relative density on seismic displacements

It is apparent that loose sand slopes reach the failure condition after a small number of cycles and correspondingly the slope critical acceleration drops to zero. Seismic slope response is clearly influenced by the occurrence of soil liquefaction and the slope exhibits a behaviour typical of liquefaction landslides. In fact as a consequence of soil liquefaction, displacements increase parabolically because of the effect of gravitational forces. For dense and medium dense sandy slopes the increase in pore pressure is smaller. Nonetheless seismic displacement time histories are strongly affected by pore pressure build-up. If displacement analysis is performed neglecting the reduction of the soil shear strength seismic slope response may be greatly underestimated; in fact Newmark's traditional displacement analysis would give no seismic displacements for the example slope considered in the analysis.

Using the proposed model a parametric analysis was performed to understand which of those parameters mainly affect seismic slope response. The results of such analysis are shown in figure 6. Sinusoidal excitations with amplitude, varying in the range 0.1g to 0.3g, and frequency 1Hz were applied for five cycles; several value of soil relative density and different slope hydraulic condition (expressed by means of the static pore pressure ratio r_u°) were accounted in the analysis. In fact, each of the curves in figure 6, provides, for given slope geometry, hydraulic conditions and soil properties, the relationship between the normalised (with respect to the slope height) maximum displacement *S/H* after a fixed number of cycles and the maximum acceleration amplitude. As exemplified in figure 7, these sets of curves can be regarded as seismic stability charts and can be used as a design tools. Indeed it is possible to define an acceptable limit displacement for a given slope possibly depending on the tolerable level of damage and on the expected maximum acceleration amplitude. The straight line in figure 7, obtained imposing $S_{LIM}=0$ for $a_{Max}=0$ and $S_{LIM}=5\%H$ for $a_{Max}=K_c^{\circ}g$, might, for instance, represent an acceptable displacement function.

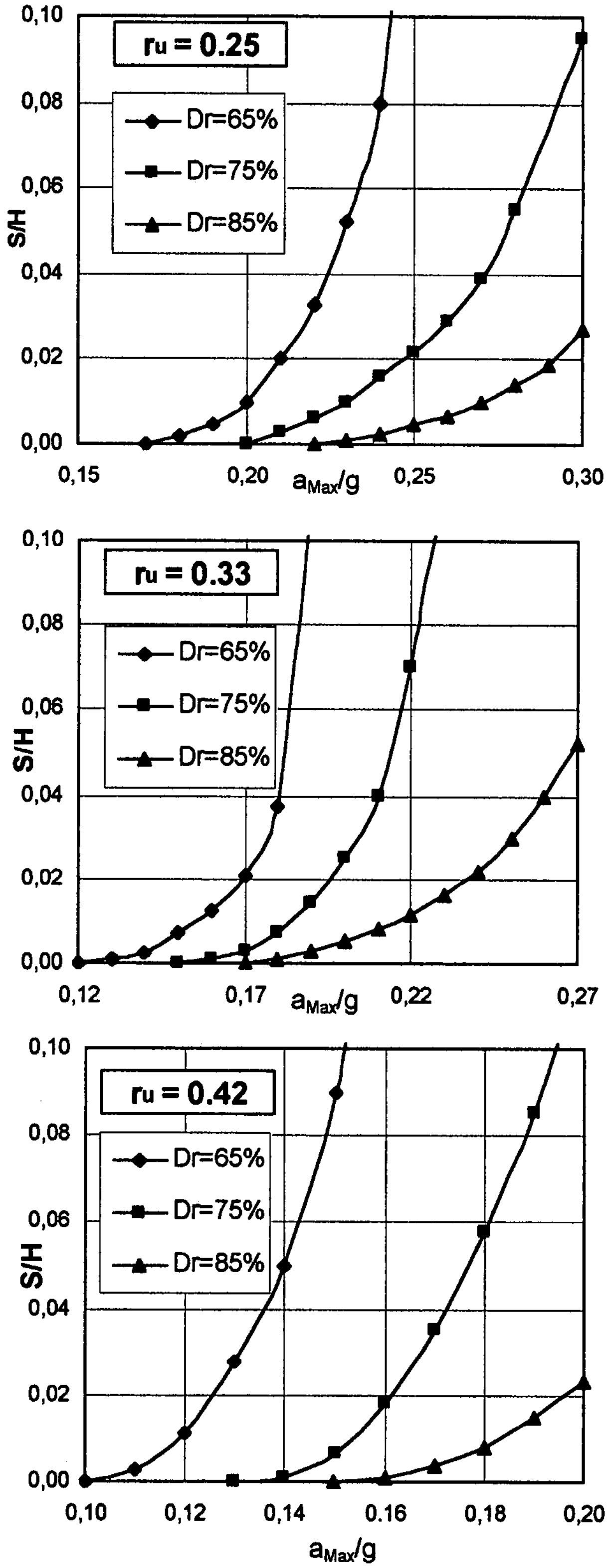

Fig. 6. Slope stability charts

The other curve in figure 7 has the same meaning of the curves represented in figure 6. Three regions can be distinguished: 1) a region in which the seismic acceleration is smaller than critical acceleration and, thus, no permanent displacement occur; 2) a region in which the maximum expected displacement is smaller than the acceptable displacement; 3) a region in which the maximum expected displacement is larger than the acceptable, meaning the loss of post- seismic serviceability for the slope or even failure.

Finally, in figure 7 it is clear that a displacement analysis performed disregarding the cyclic degradation of the soil shear strength would be misleading and provide unsafe result.

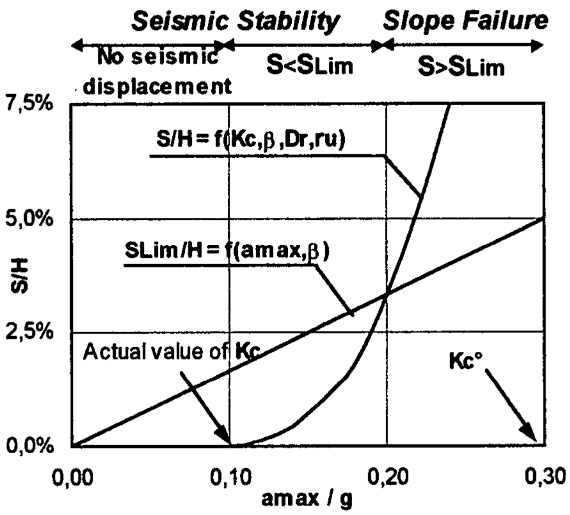

Fig. 7. Scheme of stability chart

CONCLUDING REMARKS

A numerical model to evaluate the effect of earthquake induced pore pressure on seismic slope response is proposed; cyclic behaviour of cohesionless soils are analysed and the effect of soil shear strength reduction on slope response is evaluate using an analytical relationship based on experimental results. The study show that, when significant increase in pore pressure occurs, seismic and post seismic stability analysis should be performed taking into account this phenomenon which greatly affect slope response in terms of permanent displacement. The study allows to evaluate the influence of those parameters, such as soil relative density, slope initial hydraulic conditions and soil effective stress state before the earthquake, which greatly influence the soil cyclic behaviour and consequently seismic slope response. Some useful seismic stability chart are obtained to evaluate the maximum value of permanent seismic displacement taking into account the reduction of slope critical acceleration due to the increase in pore pressures. Finally is clearly shown that a displacement analysis which neglect the soil shear strength reduction may provide unsafe result.

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