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EFFECT OF EARTHQUAKE INDUCED PORE-WATER PRESSURE IN CLAY SLOPES

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ABSTRACT

A procedure to asses the influence of earthquake-induced pore-water pressure on the behavior of natural clay slopes is presented in the paper. An experimental-based pore-water pressure generation model was adopted to detect the occurrence and to evaluate the magnitude of the excess pore-water pressure. Refereeing to the infinite slope scheme the response was evaluated in terms of earthquake-induced permanent displacements taking into account the reduction of critical acceleration due to excess pore-water pressure. A parametric analysis shows that soil plasticity and peak ground acceleration are the parameters which mostly affect the response in terms of displacements.

Keywords: seismic slope stability, excess pore-water pressure, Newmark analysis, weakening effect

INTRODUCTION

Despite its widely acknowledged importance, the effects of the increase in pore-water pressure are frequently disregarded in the design procedure for the evaluation of seismic stability condition and post-seismic serviceability of natural slopes. This is mainly because rigorous computations for 2-D and 3-D applications are mainly involved in terms of solution algorithm and input soil properties. In these cases, if the analysis neglects the effects of excess pore-water on the strength and stiffness reduction, it could lead to an unsafe estimation of the slope response: Biondi & Cascone (2006) described a number of case-histories of clay slopes and earth-structures that suffered several damages due to this weakening effect.

Based on these considerations, the need of simplified procedures devoted to the estimation of the effect of earthquake-induced pore-water pressure on the seismic and post-seismic stability condition of natural clay slopes is clearly evident. Among the available procedures for seismic stability analysis of natural slopes, the sliding block approach (Newmark, 1965) is considered the better conciliation between computational effort the results accuracy. This is mainly due to the following reasons: (i) earthquake-induced permanent displacements represent a suitable measure of the seismic slope stability condition and can be adopted to judge the post-seismic serviceability of both the slope and the structures in the nearness; (ii) the sliding block approach can be applied taking into account some effects of the cyclic behavior of soils; in particular the effect of soil strength reduction can be accounted for in the evaluation of the slope critical acceleration and in the computation of permanent displacements; (iii) due to the uncertainties in the selection of input ground motion, soil dynamic properties and hydraulic condition of natural soil slopes, the displacement approach represent the better compromise between the crude pseudo-static analysis of the earthquake effects and more sophisticated 2-D and 3-D seismic response analyses including non linear soil behavior.

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OUTLINE OF THE METHODOLOGY

The methodology adopted to develop the procedure presented herein is similar to those described by the same Authors in some previous works (Biondi et al., 2002; Biondi & Maugeri 2005) which were recently summarized in the General Report of the ETC-12 Workshop held in Athens in 2006 (Biondi & Maugeri, 2006). In particular, using the infinite slope scheme, the previous published work describes two procedures to perform a seismic stability analyses, including the evaluation of earthquake-induced permanent displacements, with reference to:

a) saturated cohesionless soil slopes, including the effect of pore-water pressure build-up through an effective stress approach (Biondi et al., 2002);
b) cohesive soil slopes including the effect of cyclic degradation of the undrained soil shear strength through a total stress approach (Biondi & Maugeri, 2005).

Both the procedures introduce two threshold values of the earthquake-induced pore-pressure ratio (procedure *a*) or of the degradation coefficient (procedure *b*) which allow to predict several slope responses to a selected design earthquake including flow and deformation failure, inertial, weakening and inertial-weakening instability and finally a seismic stability condition with or without the occurrence of soil shear strength reduction.

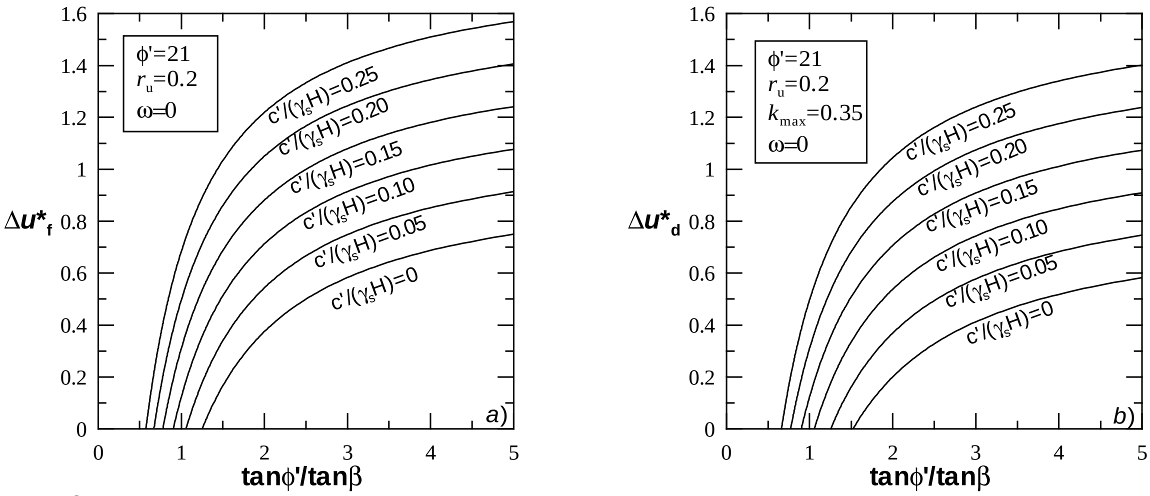
Since the procedure presented herein adopt an effective stress approach, it is similar to those presented for saturated cohesionless soil slopes (procedure *a*). However, two main differences are of interest: (i) the presence of the cohesion which is accounted for in the evaluation of the static and seismic stability condition of the slope, in the evaluation of the initial and current values of the slope critical acceleration and, finally, in the evaluation of the threshold values of the earthquake-induced pore-pressure ratio; (ii) a different pore-water pressure generation model which, herein, was selected among those available in the literature for cohesive soils subjected to cyclic loading.

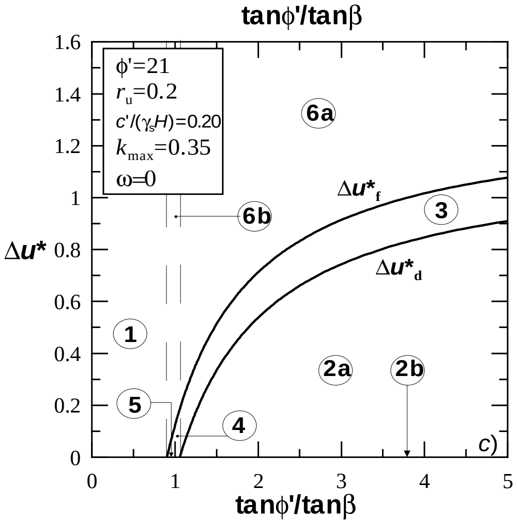
The selected pore-water pressure generation model is described in the following paragraph. Here, the main aspects of the procedure are summarized focusing the attention on the parameters through which the influence of the excess pore-water pressure is accounted for. Due to the lack of space (i) a detailed description of the possible failure mechanisms of cohesive slopes and their relation to the inertial, to the weakening and to the inertial-weakening effect is omitted herein (a detailed description can be founded in Biondi & Maugeri, 2005); (ii) the main equations giving the parameters relevant for the procedure are described in the *Appendix A* together with the reference scheme and the notation adopted in the paper (more details about the parameters can be founded in Biondi et al. 2002).

To account for the effect of soil strength degradation in the evaluation of the slope stability condition and in the assessment of earthquake-induced permanent displacements, an effective stress analysis is applied. The strength capacity is estimated as a function of the pore-water pressure build-up due to the cyclic loading and is evaluated introducing the earthquake-induced pore-water pressure ratio Δu^* (Appendix A). Two threshold values, Δu^*_f and Δu^*_d, of Δu^* are introduced allowing to detect the causes of the instability possibly occurring and the proper approach for the displacements analysis. To define the threshold values, the expressions of the effective stress state acting in both static and seismic conditions, the expressions of the static F_s and pseudo-static F_d safety factor, and finally the expressions giving the initial k_{co} and the current value of the slope critical acceleration are required. All these parameter are listed in the appendix A together with the expression of Δu^*_f and Δu^*_d. Despite the introduction of the cohesion, the meanings of Δu^*_f and Δu^*_d is unchanged with reference to those introduced by Biondi et al. (2002) for cohesionless soil slopes; then, the meanings of Δu^*_f and Δu^*_d is only synthetically described herein.

Δu^*_f represent the earthquake induced pore-water pressure ratio which makes the available shear strength mobilized along the potential sliding surface equal to the minimum static shear stress required to maintain equilibrium; hence, Δu^*_f separates two different seismic behaviors of the slope: flow and deformation failure, that is a failure condition of the slope or the occurrence of a deformation failure characterized by permanent displacements which may affect the serviceability.

Δu^*_d is the value of Δu^* beyond which the reduction of soil shear strength is high enough to bring the initial slope critical acceleration k_{co} below the maximum earthquake acceleration k_{max}; hence, if the condition $\Delta u^{*}=\Delta u^{*}_{d}$ is reached, permanent displacements may occur in the slope even if it was seismically stable (i.e. $k_{co}\geq k_{max}$) before the strength reduction take place. Unlike Δu^{*}_{f}, the ratio Δu^{*}_{d} is not an intrinsic characteristic of the slope, since it is affected by the earthquake induced state of stress. Due to their significances, a comparison between Δu^{*}_{max}, Δu^{*}_{f} and Δu^{*}_{d} allows to detect the possible slope response for a selected ground motion and to detect the effects of excess pore-water pressure on the seismic stability conditions and post-seismic serviceability. Using Δu^{*}_{f} and Δu^{*}_{d} a complete seismic stability analysis of the slope can be performed and useful stability charts can be developed (Biondi et al., 2002). In the present paper, even if the attention is focused on the displacement response instead of the stability analysis, the relationships giving Δu^{*}_{f} and Δu^{*}_{d} including the effect of the cohesion c' were derived in the Appendix A; Figure 1 shows the influence of the cohesion c' on both Δu^{*}_{f} and Δu^{*}_{d} and describe a typical stability chart proposed in the procedure together with the possible slope behavior and the required displacements analysis.





Zone	Clay slope behavior / type of analysis
1	Slope unstable under static condition.
2	Seismic and post-seismic stability. No permanent displacements occur despite the excess pore-water pressure (2a) or in absence of strength reduction (2b).
3	Likely permanent displacements due to combine inertial and weakening effects. Modified Newmark-type analysis.
4	Permanent displacements due to combine inertial and weakening effects. Modified Newmark-type analysis.
5	Permanent displacements occur due to the inertial effect. Traditional Newmark-type analysis.
6	Excess pore-water pressure induces a failure in either initially stable (6a) and initially unstable (6b) slopes.

Figure 1. Influence of the cohesion on Δu^{*}_{f} (a) and Δu^{*}_{d} (b); c, d) Example of stability chart.

PORE-WATER PRESSURE GENERATION MODEL

The increase in pore-water pressure of saturated clay soils subjected to uniform or irregular loading cycles represent a common phenomenon with well understood effects on the strength and deformability. However, due to the complexity of clay microstructure and its change under cyclic loads, the modelling of cyclic behaviour of saturated clays is quite complex. Two approaches are possible (Matasovic & Vucetic, 1995): to model the degradation accurately and calculate the cyclic pore water pressure from the degradation; or to model the cyclic pore-water pressure response

accurately and calculate the degradation from the pore-water pressure. In the present study the first approach was preferred; then an accurate calculation of the excess pore-water pressure is required to perform an effective stress analysis of the slope response. Generally, the pore-water pressure in saturated soils varies during each cycle of loading or shear straining regardless the level of the applied cyclic shear-strain amplitude γ_c. However, if the applied γ_c is smaller than a volumetric threshold value γ_v, no residual cyclic pore-water pressure remains after the cyclic loading or the shear straining has stopped. In the same way when γ_c larger than γ_v is applied, permanent pore-water pressure continuously develop and remain after the cyclic straining has stopped. A number of experimental studies showed that γ_v depends on the plasticity index *PI*, on the type of soil and on the over consolidation ratio *OCR*. Generally γ_v increase as *PI* increase; conversely, the residual pore-water pressure is positive in normally consolidated (*NC*) clays and may be negative in overconsolidate (*OC*) clays.
Due to this complex behaviour, the selection of the pore-water pressure generation model to be adopted in the stability analysis represents a crucial point. A great number of predictive model for granular saturated soil were proposed in the literature due to the devastating consequence of liquefaction and liquefaction landslides. Otherwise, a few numbers of experimental-based empirical relationships are available referring to cohesive soils despite the experienced influence of the excess pore-water pressure on slope instability and consequent damages; these few relationships were developed applying different regression models to a database of experimental results. In the following subparagraphs some of these models were analysed to detect the proper for the purpose of the present study focusing the attention on *NC* clay.

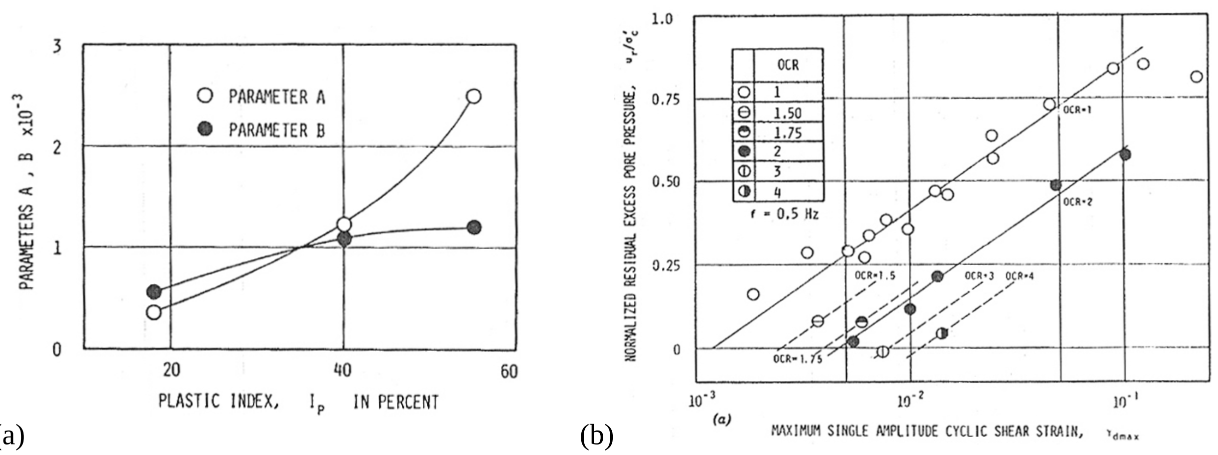
The model by Matsuo et al. (1980)
Starting from the results of undrained cyclic triaxial tests, Matsuo et al. (1980) developed a model for the prediction of the residual pore-water pressure for both *NC* and *OC* clays subjected to uniform cyclic loading. In the study, the Authors investigate the influence of the loading frequency, effective confining pressure σ'_C and cyclic shear stress on the maximum shear strain $\gamma_{c,max}$ and on residual pore water pressure Δu in soil specimens. Using the obtained experimental data, the Authors detect a linear relationship between the threshold value γ_v and *OCR* and express the dependence of γ_v from *PI* was expressed by means of the two functional parameters *A* and *B* described in Figure 2a:

$$\gamma_v = A \cdot (OCR - 1) + B \tag{1}$$

In the proposed model, the residual pore-water pressure is expressed in terms of excess pore-water pressure ratio Δu^* which depend on $\gamma_{c,max}$, *OCR* and *PI*:

$$\Delta u^* = \frac{\Delta u}{\sigma'_c} = \eta \cdot \log \frac{\gamma_{c,max}}{\gamma_v} = \eta \cdot \log \frac{\gamma_{c,max}}{A \cdot (OCR + 1) + B} \tag{2}$$

In the model η represent a material constant which can be easily evaluated through the results of cyclic tests, plotting the excess pore-water pressure ratio versus $\gamma_{c,max}$ in a *log* scale (Figure 2b).



(a) (b)

Figure 2. Pore pressure model by Matsuo et al. (1980): a) parameters *A* and *B*; b) estimation of η

The model by Matasovic & Vucetic (1995)

Matasovic & Vucetic (1995), proposed a generalised cyclic degradation-pore water pressure generation model relating the cyclic strength degradation and pore-water pressure increase in clay soils. The model was developed using a curve-fitting technique, based on the results of NGI-type constant volume equivalent cyclic undrained direct simple shear tests performed on several marine clays. In the model, the excess pore-water pressure ratio Δu^* (i.e. the ratio between the excess pore-water pressure Δu and the initial vertical effective consolidation stress σ'_{VC}) is a cubic function of the degradation index δ and depends on four fitting constant *A, B, C, D*; δ is expressed in terms of number of loading cycles *N*, degradation parameter *t*, γ_c and γ_v and two experimental constants *r* and *s*:

$$\Delta u^* = \frac{\Delta u}{\sigma'_{vc}} = A\cdot\delta^3 + B\cdot\delta^2 + C\cdot\delta + D = A\cdot N^{-3\cdot s\cdot(\gamma_c-\gamma_v)^r} + B\cdot N^{-2\cdot s\cdot(\gamma_c-\gamma_v)^r} + C\cdot N^{-s\cdot(\gamma_c-\gamma_v)^r} + D \qquad (3)$$

As described by Matasovic & Vucetic (1995), the six numerical constants of the model (*A, B, C, D, r* and *s*) can be easily evaluated from cyclic test data.

The model by Egglezos & Bouckovalas (1998)

Using a large number of experimental data related to 12 triaxial and 32 direct simple shear tests on *NC* clays and to 6 triaxial and 6 direct simple shear tests on silts, Egglezos & Bouckovalas (1998) proposed a set of empirical relationships to evaluate the excess pore-water pressure during earthquakes. Apart from the effect of soil type, the proposed relationships distinguish between constant cyclic stress and constant cyclic shear strain loading as well as between triaxial and simple shear test conditions. In the model the excess pore-water pressure ratio Δu^* is defined as the ratio between the excess pore-water pressure Δu and the octahedral effective consolidation stress σ'_{oct}. The relationship relating Δu to the cyclic strain amplitude γ_c or to the cyclic stress amplitude τ_c is:

$$\Delta u^* = \frac{\Delta u}{\sigma'_{oct}} = \frac{2}{\pi}\cdot\arcsin\left[N^{\frac{1}{2\cdot c_0}}\cdot\sin\left(\frac{\pi}{2}\cdot A\cdot P_a^{1-c_2}\cdot\sigma'^{\,c_2-1}_{oct}\cdot R^{c_3}\cdot e^{c_4}\right)\right] \qquad (4)$$

where *R* is equal to γ_c or τ_c and *A*, c_o, c_1, c_2, c_3 and c_4 are material and test dependent constants. As described by Egglezos & Bouckovalas (1998), the choice of *R* depends only whether γ_c or τ_c is known in the first loading cycle and not whether loading is stress or strain controlled.

Selection of the model to be adopted for *NC* cohesive soils

To detect the pore-water pressure generation model to be adopted in the present study, the applicability of each models to *NC* clays were discussed herein focusing the attention on the model accuracy, on the number of the required parameters and on the applicability of the models.

In the model proposed by Matsuo et al. (1980) the evaluation of Δu^* require the knowledge of the maximum shear strain $\gamma_{c,max}$, *OCR, PI* and η. Even if the value of the material constant η should be evaluated starting from the results of cyclic tests, Matsuo et al. (1980) demonstrated that a value η=0.45 well fits all the data obtained for different type of clays (see Figure 2b). Concerning the evaluation of the maximum shear strain due to irregular loading path such as those imposed by earthquake, different procedures are available in the literature ranging from a simplified decoupled analysis to more sophisticated dynamic response analysis. For the purposes of the present study a decoupled evaluation of the maximum shear strain is convenient. Several procedures for seismic stability analysis of slopes were proposed in the literature including this kind of computation (Crespellani et al., 1990, 1996). Finally, the model by Matsuo et al. (1980) clearly states the influence of the threshold value γ_v on the occurrence and on the magnitude of excess-pore water pressure.

Using the model by Matasovic & Vucetic (1995), the evaluation of Δu^* requires the knowledge of the six numerical constants which can be evaluated through cyclic test data and the knowledge of the number *N* of imposed loading cycles and their amplitude γ_c; as described by the Authors, the threshold values γ_v can be evaluated using the relationship by Vucetic (1994) as a function of *PI*. Concerning the computation of the number of loading cycles equivalent to the earthquake-induced loading path, a number of standard weightings procedures (Biondi et al., 2004; Green et al., 2005) are available for granular saturated soils; otherwise, similar procedure for cohesive soils are not available in the

literature and a few empirical procedures to estimate the loading cycles involved in the cyclic strength degradation of cohesive soils are presented even not fully investigated (Crespellani et al, 1992; Cascone et al. 1998).
To adopt one of the relationships by Egglezos & Bouckovalas (1998), the values of the numerical constants provided by the Authors for different soil type can be adopted and the corresponding accuracy can be evaluated. Moreover, an evaluation of γ_c or τ_c is required together with the knowledge of the number of loading cycles N. Even the accuracy of the proposed relationships is well documented by the Authors, the use of the relationships to predict the pore-water pressure induced by irregular loading paths such as those imposed by earthquakes is not demonstrated in the original paper. Moreover, the previous discussed difficulties in the computation of N must be considered in the adoption of this relationship. Finally, even if the presence of a threshold value of the shear strain is implicitly accounted for in the model, for the purpose of the present study a direct correlation between the magnitude of excess pore-water pressure and the threshold values γ_v of shear strain is convenient.
Based on these considerations the model by Matsuo et al. (1980) will be adopted in the present study.

EVALUATION OF EARTHQUAKE-INDUCED PORE-WATER PRESSURE RATIO

To apply the selected pore-water pressure generation model in the computation of the excess pore-water pressure induced on a given soil slope by a selected design earthquake, the maximum shear strain $\gamma_{c,max}$, introduced for the case of uniform loading, was assumed equal an average shear strain level γ_{ave} compatible to an average value τ_{ave} of the shear stress time-history imposed by the earthquake in the slope. The procedure considered to this purposes consists of several steps:
- estimation of an average amplitude τ_{ave} of the shear stress time-history induced by the earthquake;
- evaluation of the compatible shear strain level γ_{ave} introducing a constitutive model of the soil;
- evaluation of the threshold value γ_v (eq. 1) of the shear strain level and comparison with γ_{ave};
- if $\gamma_{ave} > \gamma_v$ computation of the excess pore-water pressure ratio through eq. (2).

All these steps are described herein.
The maximum value of the earthquake-induced shear stress can be assumed equal to (appendix A):

$$\Delta\tau_d = \gamma_s \cdot H \cdot k_{max} \cdot \cos\beta^* \cdot \cos\beta \qquad (5)$$

Introducing the effect of the soil compliance, through the stress reduction factor r_d, and considering the transient nature of the earthquake-induced shear stress time-history by means of an equivalent stress level α, the average value τ_{ave} of the shear stress amplitude can be estimated; in the proposed procedure the expression r_d=1-0.015·H by Iwasaky et al. (1978) and α=0.65 were considered:

$$\tau_{ave} = 0{,}65 \cdot \gamma_s \cdot H \cdot k_{max} \cdot \cos\beta^* \cdot \cos\beta \cdot (1 - 0{,}015 \cdot H) \qquad (6)$$

Denoting with $G(\gamma)$ and G_o respectively the shear modulus for a given shear strain level γ and its initial value, the average amplitude γ_{ave} of the shear strain induced in the slope can be computed as:

$$\gamma_{ave} = \frac{\tau_{ave}}{G(\gamma)} = \frac{0{,}65 \cdot \gamma_s \cdot H \cdot k_{max} \cdot \cos\beta^* \cdot \cos\beta \cdot (1 - 0{,}015 \cdot H)/G_o}{G(\gamma)/G_o} \qquad (7)$$

In the previous equation, $G(\gamma)/G_o$ denote the shape of modulus reduction curves; for cohesive soil, this shape is influenced more by the PI than the void ratio e and the effective confining pressure σ'_o. For this reason the modulus reduction curves proposed by Vucetic & Dobry (1991) and described in Figure 3a were adopted herein. It is important to note that different values of α and r_d should be adopted in the procedure and other modulus reduction curves should be selected to introduce in the analysis the effects of e and σ'_o which can be influent for soils of low plasticity.
Using eq. (7), γ_{ave} can be computed using an iterative procedure required by the non linearity of the equation itself. To this purpose (i) it is convenient to use analytical relationships describing the shape of the modulus reduction curves and (ii) it is necessary to introduce an expression giving G_o.
Concerning the first matter the curves proposed by Vucetic & Dobry (1991) were fitted using the relationships proposed by Yokota et al. (1981) to describe the shear modulus decay with strain level γ:

$$G(\gamma)/G_o = 1/\left(1 + a \cdot \gamma^b\right) \qquad (8)$$

The values of the fitting constants *a* and *b* showed in Figure 3b) were computed using a numerical regression procedure; the accuracy of the regression is showed in Figure 3b) where the original (Vucetic & Dobry) and the computed (Eq.8) values of $G(\gamma)/G_o$ were compared for several *PI*.

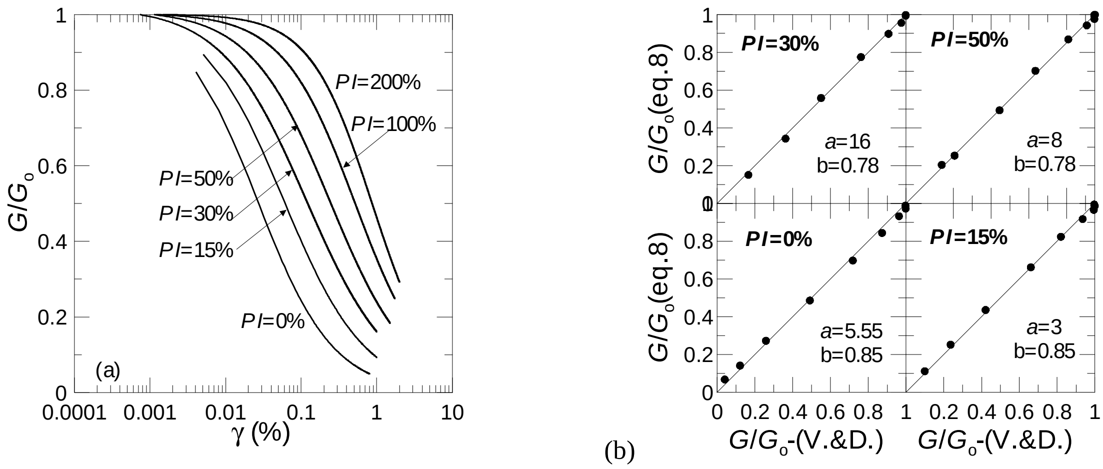


Figure 3. a) Shear modulus reduction curve by Vucetic & Dobry (1991); b) accuracy of the fitting procedure with the equations by Yokota et al. (1981) for some values of *PI*.

Following the guidelines for *Geotechnical design in seismic areas* proposed by the Italian Geotechnical Society (AGI, 2005), the expression by D'Onofrio & Silvestri (2001) for *NC* clays was adopted to compute the dependence of G_o (kPa) from the effective confining pressure σ'_c and *PI*:

$$G_o / p_a = S \cdot (\sigma'_c / p_a)^n \tag{9}$$

In eq.(9) p_a=100 kPa, the effective confining pressure σ'_C is estimated as the mean effective confining pressure σ'_o ($\sigma'_o=\sigma'_{vo}\cdot[1+2\cdot k_o]/3$ being k_o=1-sin ϕ') and *S* and *n* are the stiffness coefficient and the stiffness index respectively which depends on *PI* (D'Onofrio & Silvestri, 2001):

$$S = 217 + 805{,}84 \cdot \exp(-PI/18{,}94) \qquad n = 0{,}68 - 0{,}162 \cdot \exp(-PI/23) \tag{10}$$

Using the above described procedure, for a given slope scheme (i.e for given values of γ_s, *H*, r_u, β, ϕ', *PI*) and design earthquake (i.e for given values of k_{max} and Ω) the value of γ_{ave} can be computed. As an example, for the case γ_s=20 kN/m^3, *H*=22m, r_u=0.5, ϕ'=25° and Ω=0 (i.e. $k_{max}=k_h$), Figure 4 describes the strain level γ for β ranging from 10° to 25°, $k_h=k_{max}$ ranging from 0 to 0.5 and several values of *PI*.

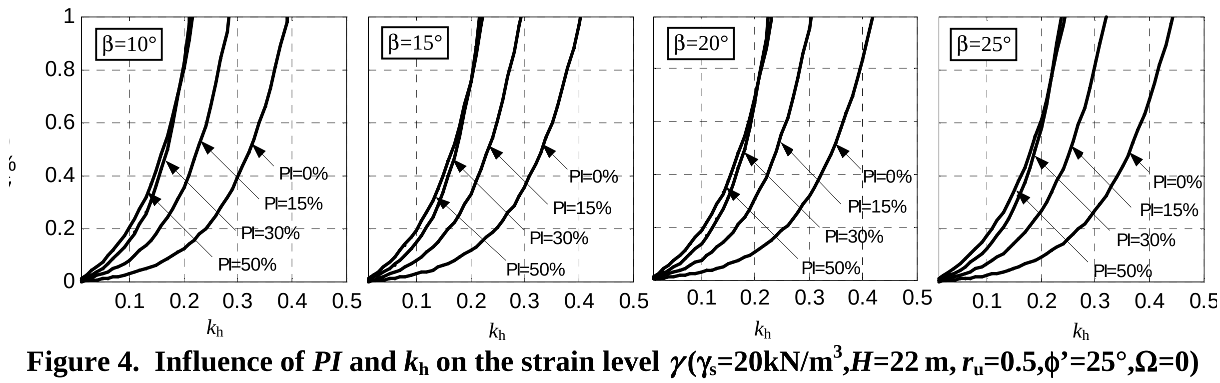


Figure 4. Influence of *PI* and k_h on the strain level γ(γ_s=20kN/m^3,*H*=22 m, r_u=0.5,ϕ'=25°,Ω=0)

From the plotted data it is evident a non linear relationship between k_h and the shear strain level; since eqs.(5) and (6) states a linear relationship between k_{max} and the earthquake-induced shear stress, the obtained k_h-γ non linear relationship must be ascribed to the adopted constitutive model. For all the combination described in the figures, a significant influence of both *PI* and k_h on γ_{ave} was evident.

For the same slope scheme considered in Figure 4 and for *PI*=15%, Figure 5 shows the strain level γ computed through Eq.(7) for several values of ω or Ω: ω=0, ω=-β, $\omega=\phi'-\beta$, Ω=-1/2 and Ω=-1/3. The first three values describe the following conditions: vertical component of seismic acceleration negligible, seismic acceleration directed parallel to the failure plane, direction of the seismic acceleration which minimize the slope critical acceleration k_{co}; the latter two vales corresponds to one couple of the conditions imposed by Eurocode 8 (2003) for the pseudo-static stability analysis of slopes ($\Omega=\pm1/2$ and $\Omega=\pm1/3$). From the plots it is evident that the influence of ω (Ω) on γ_{ave} increases as β increases; generally the condition Ω=-1/2 result in the more conservative values of γ_{ave}.

Starting from the computed values of γ_{ave}, the comparison with γ_v can be performed and, if the condition $\gamma_{ave}>\gamma_v$ occurs, Δu^* can be computed using eq.(2). Following this procedure the values of Δu^* showed in Figure 6 were computed for the same combination of the slope and earthquake parameters considered for Figure 4. Differently from γ_{ave} (Figure 4), a less evident influence of *PI* on Δu^* is apparent from Figure 6; conversely, a significant influence of the horizontal acceleration level k_h is again evident. Concerning the first results, it is important to stress that for *NC* clays, the threshold value γ_v is equal to the parameter *B* (eq.1) for which (Figure 2b) the influence of *PI* is lower than for the parameter *A*; then *PI* influences significantly the computation of γ_{ave}. Concerning the influence of the slope angle, from Figure 5 appears that for β varying in the range 10°-25° a variation of Δu^* at least of 25-30% was estimated for all the considered k_h values.
The most important result obtained with the procedure proposed for computing Δu^* is related to the influence of k_h. As shown in Figure 6, with the exception of the case *PI*=0%, which is related to sandy soils, for all the considered schemes it seems that for k_h lower than about 0.07g no increase in pore pressure occurs; this is due to the values of γ_{ave} that for k_h<0.07g are lower than γ_v in all the cases. Then, for the considered slope scheme the value $k_{max}=k_h$=0.07g can be considered as a threshold values below which the earthquake-induced shear strain level do not produce excess pore-water pressure in the soil.
The opportunity and the effectiveness to introduce this threshold value is obviously: through a combined use of eq.(1), (7) and (8) the expression giving $k_{max,v}$ can be obtained:

$$k_{max,v}=\frac{G_o}{0{,}65\cdot\gamma_s\cdot H\cdot\cos\beta^*\cdot\cos\beta\cdot(1-0{,}015\cdot H)}\cdot\frac{[A\cdot(OCR-1)+B]}{1+a\cdot([A\cdot(OCR-1)+B])^b} \quad (11)$$

Due to the significant influence of Ω (see Figure 5), the values of Δu^* were evaluated for the same case of Figure 6 and assuming Ω=-1/2; the obtained results are showed in Figure 7.

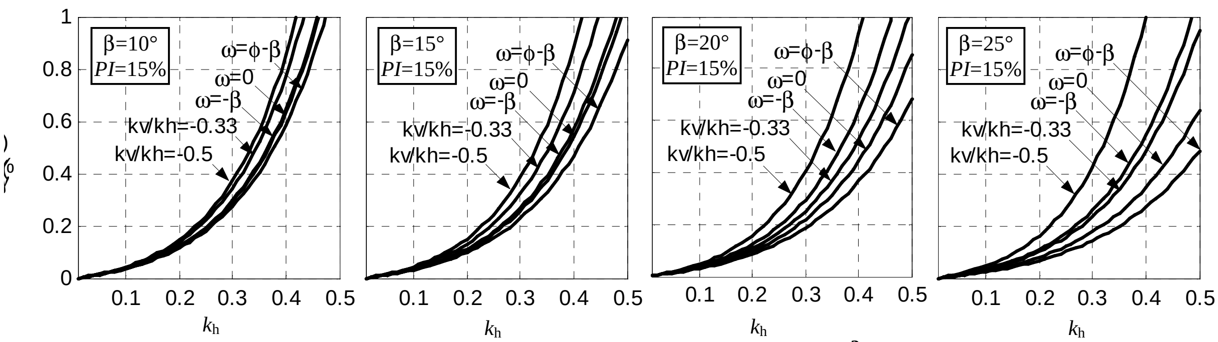


Figure 5. Influence of ω and k_H on the strain level γ (γ_s=20kN/m^3, *H*=22 m, r_u=0.5, ϕ'=25°)

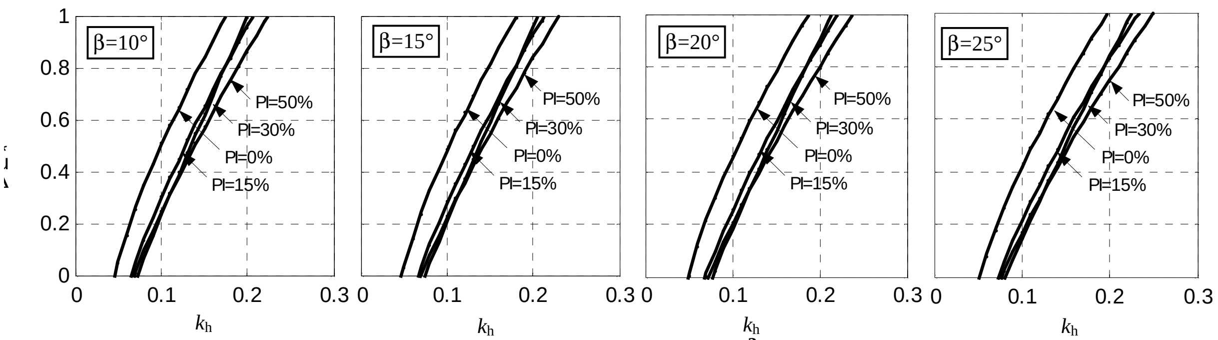


Figure 6. Influence of *PI* and k_h on Δu^* (γ_s=20 kN/m^3, *H*=22 m, r_u=0.5, ϕ'=25°,Ω=0)

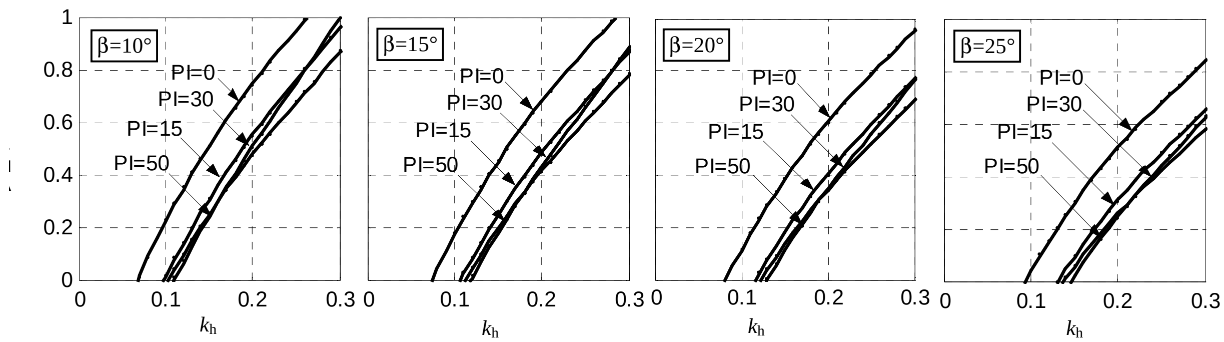


Figure 7. Influence of *PI* and k_h on Δu^* (γ_s=20 kN/m^3, *H*=22 m, r_u=0.5, ϕ'=25°,Ω=-1/2)

In this case the same consideration about the influence of *PI* and β described for Figure 6 can be performed and (with exception of the case *PI*=0%) values of $k_{max,v}$ ranging from 0.09-0.16 (i.e. values of $k_{h,v}$=0.1-0.15) can be estimated. Concerning this last result it is important to underline that it fits very well with the provisions of Eurocode 8 (2003) that, for the application of the pseudo-static approach states: "*For saturated soils in areas where* k_h*>0.15, consideration shall be given to possible strength degradation and increases in pore pressure due to cyclic loading…*".

EFFECT OF EXCESS PORE-WATER PRESSURE ON THE DISPLACEMENT RESPONSE

To describe the influence of the excess pore-water pressure on the displacement response of the slope, the reduction in the critical acceleration k_c due to the strength reduction must be considered. To this purpose two values k_{co} and $k_{c.min}$ of the slope critical acceleration can be computed (Appendix A): k_{co} refers to the initial slope condition, before the excess pore-water pressure takes place, and represent the value of the slope critical acceleration to be adopted in a displacement analysis which neglects the effect of the strength reduction due to the excess-pore-water pressure. Otherwise $k_{c.min}$ represents the value to which the slope critical acceleration reduces when the excess pore-water pressure occurred. If no excess-pore-water pressure develops in the soil ($\gamma_{ave}<\gamma_v$ and Δu^*=0), k_c equals k_{co}; otherwise ($\gamma_{ave}>\gamma_v$ and Δu^*>0), the reduction of k_c from k_{co} to $k_{c.min}$ must be considered in the displacements analysis. Since the aim of the paper is to point out the influence of excess pore-water pressure on the magnitude d_{max} of displacements, an empirical regression model was adopted to compute d_{max} instead of the double integration of the equation of motion of the slope for a given earthquake record. In particular, the relationship by Simonelli & Viggiani (1995) was adopted:

$$\log d_{max}(cm) = C_1 - C_2 \cdot k_c / k_{max} \qquad (C_1 = 2.652\ ,\ \ C_2 = 3.333) \qquad (12)$$

This relationship represents an upper-bound of the permanent displacements d_{max} computed for values of the ratio k_c/k_{max} in the range 0.1-0.9 using a set of records of the *M*=6.9 Irpinia (Italy) 1980 earthquake with peak acceleration k_{max} ranging from 0.107 to 0.316, Arias Intensity ranging from 32.1 cm/s to 143.4 cm/s and total duration ranging from 70.7 sec to 86.1 sec. The selection of the regression model is not a crucial point since (i) the purpose of the paper is just to point out the influence of the excess-pore water pressure on the increase of permanent displacements with respect to the case in which no strength reduction take place and (ii) consideration on the absolute values of the permanent displacements are not the goal of the paper.

Assuming Ω=0, Figure 8a describes the plot of d_{max} versus k_h for the scheme γ_s=20 kN/m^3, *H*=22m, β=20°, r_u=0.1, ϕ'=20° and c'/($\gamma_s \cdot D$)=0.20; in particular the relation corresponding to a traditional analysis which neglect the possible excess pore-water pressure (eq.12 with k_c= k_{co}), together with the relations in which the occurrence of excess pore-water is considered for *PI* equal to 15%, 30% and 50% (eq.12 with k_c=$k_{c.min}$), are plotted. For the considered scheme the static factor of safety and the initial value of the critical acceleration are F_s=1.52 and k_{co}=0.169. From the plots it is evident that the same slope subjected to the same earthquake, encounter different permanent displacements depending on the valued of *PI* since this parameter affect both the occurrence and the magnitude of the excess pore-water pressure Δu. When the reduction in soil shear strength is accounted for in the analysis, the slope of the curve increase with respect to the case in which Δu^* is neglected; for each of the considered case, the threshold value $k_{h,v}$ can be detected from the intersection of the curves computed including Δu with the curve related to the case in which Δu is neglected.

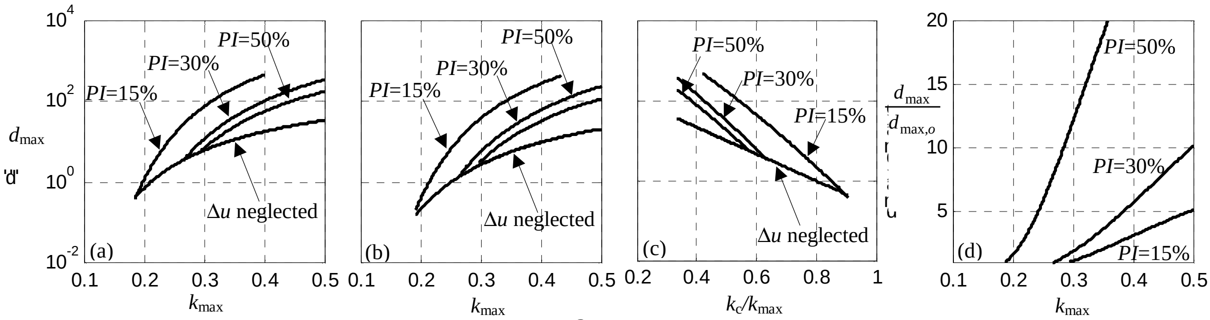


Figure 8. Influence of Δu on d_{max} (γ_s=20 kN/m^3, H=22 m, r_u=0.5, β=20°,ϕ'=20°, $c'/(\gamma\cdot D)$=0.2, Ω=0)

In a traditional displacement analysis, in which the effect of Δu is neglected, if two different slope (i.e. different value of γ_s, H, β, r_u, ϕ' or c') with the same k_{co} are considered, the same value of the displacements will be computed. Otherwise, if the possible increase in pore pressure is considered, different displacement response may occur in the slopes since they are characterized by different stress state and different hydraulic condition. Figure 8b describes the plot of d_{max} versus k_h for the scheme γ_s=20 kN/m^3, H=22m, β=18°, r_u=0.25, ϕ'=20° and c'/($\gamma_s\cdot D$)=0.20 which is characterized by the same value of F_s=1.52 and k_{co}=0.169 of the slope considered in Figure 8a. Despite the same value of k_{co} different value of d_{max} are obtained due to (i) the different acceleration level required to induced excess pore-water pressure (i.e. different value of $k_{h,v}$) and (ii) different magnitude of the excess pore-water pressure induced in the soil. This means that, for a selected slope scheme, a non unique relationship between d_{max} and k_{max} can be considered due to the different response of the slope which may be envisaged as a consequence of the different effective confining pressure and the different shear stress level induced by the earthquake. Just to stress this important results, for the same slope considered in Figure 8a, in Figure 8c the relationship between d_{max} and the ratio k_{co}/k_{max} was plotted referring to the case in which Δu is neglected (eq.12 with k_c= k_{co}) and for the casein which Δu is considered as a function of PI (eq.12 with k_c=$k_{c,min}$); from the plots it is evident that if the excess pore-water pressure induced by the earthquake was neglected in the analysis, the displacements may be underestimated by a factor ranging from 10 to 100 times about.

In order to quantify the influence of the soil shear strength reduction on the magnitude of the permanent displacements, a relationship between the excess pore-water pressure ratio and the reduction of slope critical acceleration is required. Using the equations giving k_{co} and k_c (see Appendix A), the following relationship can be obtained:

$$\frac{k_c}{k_{co}} = \left[\frac{c'/(\gamma_s \cdot D)}{\text{sen}\beta} + \frac{\tan\phi'}{\tan\beta}\cdot(1-r_u)\cdot(1-\Delta u^*)-1\right] \Big/ \left[\frac{c'/(\gamma_s \cdot D)}{\text{sen}\beta} + \frac{\tan\phi'}{\tan\beta}\cdot(1-r_u)-1\right] \tag{13}$$

This reduction in the slope critical acceleration produces an increase in the permanent displacements; denoting with $d_{max,o}$ and d_{max} the values of the earthquake-induced permanent displacement computed respectively neglecting (k_c=k_{co}) and taking into account (k_c=$k_{c,min}$) the increase in the pore-water pressure, the following relationship between $d_{max,o}$ and d_{max} can be derived through eqs.(12) and (13):

$$\log\frac{d_{max}}{d_{max,o}} = C_2 \cdot \left(\frac{k_{co}-k_c}{k_{max}}\right) = C_2 \cdot \frac{\cos\beta\cdot\tan\phi'\cdot(1-r_u)}{\cos\beta^* + \tan\phi'\cdot\text{sen}\beta^*}\cdot\frac{\Delta u^*}{k_{max}} \tag{14}$$

The ratio $d_{max}/d_{max,o}$ can be assumed as a displacement aggravation factor which summarises the effect of the increase in pore-water pressure on the displacement response of the slope; eq. 14 shows that the displacement aggravation factor derived herein is a function of the static stability condition of the slope (through the parameters r_u, ϕ' and β), of the parameters k_{max} and ω which describe the design earthquake and finally of the plasticity index PI which is involved in the computation of the threshold value γ_v and, then, in the pore-water pressure ratio Δu^*. Figure 8d shows the values of the displacement aggravation factor for the same slope of Figure 8a; when $d_{max}/d_{max,o}$=1 no excess pore-water pressure develops in the soil and no correction to the traditional Newmark.type displacement analysis is required. An important aspect is evident from Figure 8d: *PI* significantly affects the magnitude of the correction to be applied to the traditional displacement analysis results and restricts the field of application of the traditional displacement analysis to value of k_{max} lower than $k_{max,v}$ ($k_{h,v}$).

CONCLUDING REMARKS

Despite the effects of the increase in pore-water pressure in saturated soils subjected to cyclic loading are well known, the seismic stability analysis and the evaluation of the post-seismic serviceability of natural slopes are generally performed neglecting these effects. The present paper introduces a procedure to perform an effective stress analysis of the seismic stability condition of natural clay slopes taking into account the possible occurrence of excess pore-water pressure. An experimental-based pore-water pressure generation model was adopted to predict the occurrence and to asses the magnitude of excess pore-water pressure starting from the knowledge of the earthquake-induced shear strain level. Focusing the attention on normally consolidated soil slopes, the earthquake-induced shear strain level was estimated using a decoupled approach which adopts literary available shear modulus reduction curves. The displacement response was estimated using a simplified Newmark-type analysis modified in order to accomplish the effect of the strength reduction. A parametric analysis was performed showing that the acceleration level induced in the slope together with the plasticity of the soil represents the parameters mostly involved in the occurrence of excess-pore-water pressure and mostly affecting the magnitude of earthquake-induced permanent displacements.

APPENDIX A

This appendix describes all the equations giving the relevant parameters of the procedure proposed in the paper. The slope scheme adopted in the analysis is described in Figure 9.

Notation

β: slope angle
H,D: depth of the failure surface/thick of unstable soil
c’: cohesion
ϕ’: angle of soil shear strength
γ_s: soil unit weight
γ_w: water unit weight
r_u: static pore pressure ratio = $(\gamma_w \cdot H_w)/(\gamma_s \cdot H)$
k_{max}: maximum earthquake-induced acceleration in g
k_h,k_v: horizontal and vertical component of k_{max}
$\omega = tg^{-1}(k_v/k_h)$ $\Omega = k_v/k_h$
$\beta^* = \beta + \omega$
t: time

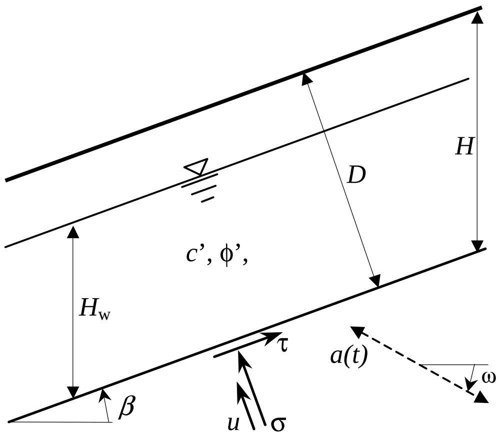


Figure 9. Reference slope scheme

Stress state (static values are obtained for $k(t)$=0 and are denoted with the suffix ‘o’ instead of ‘d’):

$$\sigma_d(t) = \gamma_s \cdot H \cdot \left[\cos^2\beta - k(t)\cdot \mathrm{sen}\beta^* \cdot \cos\beta\right]$$

$$\tau_d(t) = \gamma_s \cdot H \cdot \left[\mathrm{sen}\beta \cdot \cos\beta + k(t)\cdot \cos\beta^* \cdot \cos\beta)\right]$$

$$\sigma'_d(t) = \gamma_s \cdot H \cdot \left[\cos^2\beta \cdot (1-r_u)\cdot\left(1-\Delta u^*(t)\right) - k(t)\cdot \mathrm{sen}\beta^* \cdot \cos\beta\right]$$

$$u_d(t) = \gamma_w \cdot H_w \cdot \cos^2\beta + \Delta u(t)$$

$$r_u = u_o/\sigma_o = (\gamma_w \cdot H_w)/(\gamma_s \cdot H)$$

$$\Delta u^*(t) = \Delta u(t)/\sigma'_o$$

Static (F_s) and pseudo-static (F_d) safety factor:

$$F_s = \frac{c'/(\gamma_s \cdot D)}{\sin\beta} + \frac{\tan\phi'}{\tan\beta}\cdot(1-r_u)$$

$$F_d = \frac{c'/(\gamma_s \cdot D) + \cos\beta \cdot (1-r_u)\cdot\left(1-\Delta u^*\right) - k\cdot \mathrm{sen}\beta^*}{\mathrm{sen}\beta + k\cdot\cos\beta^*}\cdot\tan\phi'$$

Initial and minimum values of the slope critical acceleration:

$$k_{co} = \frac{c'/(\gamma_s \cdot D) + \cos\beta \cdot \tan\phi' \cdot (1 - r_u) - \text{sen}\beta}{\cos\beta^* + \tan\phi' \cdot \text{sen}\beta^*} \qquad k_{c,\min} = \frac{c'/(\gamma_s \cdot D) + \cos\beta \cdot \tan\phi' \cdot (1 - r_u) \cdot (1 - \Delta u^*) - \text{sen}\beta}{\cos\beta^* + \tan\phi' \cdot \text{sen}\beta^*}$$

Earthquake-induced pore-water pressure ratios required for slope failure (Δu^*_f) and to trigger permanent displacements (Δu^*_d):

$$\Delta u^*_f = 1 - 1 \Big/ \frac{\tan\phi'}{\tan\beta} \cdot (1 - r_u) \qquad \Delta u^*_d = \Delta u^*_f \cdot (1 - k_{max}/k_{co})$$

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