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A simplified method for pore pressure buildup prediction: from laboratory cyclic tests to the 1D soil response analysis in effective stress conditions

Anna Chiaradonna[a,*], Giuseppe Tropeano[b], Anna d'Onofrio[a], Francesco Silvestri[a]

[a]*University of Napoli Federico II, via Claudio 21, Napoli 80125, Italy*
[b]*University of Cagliari, Piazza d'Armi, Cagliari 09123, Italy*

Abstract

The importance of predictions of earthquake-induced pore water pressure has been widely recognized for reliable evaluations of strong-motion response of saturated soil. The build-up of excess pore water pressure, in fact, causes reduction in soil stiffness and strength, in some cases leading to liquefaction. Simplified predictive models in the literature are empirically based on the results of cyclic laboratory tests carried out in strain- or stress-controlled conditions. Most of such empirical models require, preliminarily, that the irregular earthquake load is reduced to an equivalent number of cycles of uniform shear stress, in order to reproduce the effect of a laboratory cyclic test. To avoid such conversion, rather complex and not always reliable, a stress-based model is here proposed that allows for the direct generalization of the cyclic test data to irregular stress histories. More realistic predictions should take into account also the dissipation and redistribution of pore pressure within a soil deposit, which can be effectively modelled using the one-dimensional theory of consolidation. The build-up and dissipation models have been incorporated in a program for 1D seismic response analyses in the time domain, in order to carry out coupled dynamic analyses in terms of effective stress. In this paper, after a brief recall of the proposed PWP model, the code performance will be verified with reference to a case history of a dyke damaged during the seismic sequence occurred in Emilia plain (Italy) in May, 2012.

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Keywords: pore pressure; liquefaction; seismic response analyses

* Corresponding author. Tel.:+039-0817685916.
E-mail address: anna.chiaradonna@unina.it

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1. Introduction

Earthquakes induce excess pore water pressure build-up in saturated soils, causing reduction in stiffness and strength, in some cases until liquefaction. Thus, the numerical prediction of excess pore water pressure is important for a reliable evaluation of strong-motion response of sandy deposits.

Simplified semi-empirical models for the prediction of pore water pressure build-up require converting the earthquake motion into an equivalent number of cycles of uniform shear stress. The results of this procedure strictly depend on the conversion curve adopted and on the techniques for choosing and counting the stress cycles that significantly affect the pore pressure generation [1]. To bypass such conversion procedures, Park et al. [2] presented a stress-based model working with a single variable, called 'damage parameter', which can be computed on both cyclic test data and irregular stress histories. This model can be applied to carry out dynamic response analyses of saturated sand deposits by removing the need for evaluating equivalent uniform stress cycles. This approach has been implemented in conjunction with the theory of one-dimensional consolidation [3] in the numerical code SCOSSA [4], in order to carry out one-dimensional dynamic analyses in effective stresses taking into account for generation and dissipation/redistribution of excess pore water pressure.

In this paper, the formulation of the 'PWP model' is briefly recalled. The implementation in the SCOSSA code has been then used to analyze the case history of a dyke damaged during the Emilia 2012 earthquake.

2. Simplified model for generation and dissipation of excess pore water pressure

The proposed 'PWP model' permits to compare the irregular time-history of shear stress induced by earthquake with the soil liquefaction resistance, evaluated in stress-controlled cyclic laboratory tests. The comparison is expressed through the so called 'damage parameter', which can be computed for any loading pattern. The damage parameter, κ, is an incremental function of the applied load that takes into account the cyclic strength of the soil [5]. This latter is expressed in terms of cyclic resistance curve, analytically described by the following equation:

$$\frac{(SR - SR_t)}{(SR_r - SR_t)} = \left(\frac{15}{N}\right)^{\frac{1}{\alpha}} \tag{1}$$

where SR is the shear stress amplitude normalized either by the mean effective confining pressure in a cyclic triaxial test or by the effective vertical stress in a simple shear test; N is the number of cycles, SR_r is the ordinate of the curve for $N = 15$ (usually adopted as a reference number of cycles). The parameters α and SR_t respectively describe the steepness and the horizontal asymptote of the curve, as shown in Figure 1a.

The damage function, κ, increases when SR overcomes SR_t, which represents the threshold below which there is no pore pressure build-up. It assumes the following expression for a uniform cyclic stress history:

$$\kappa = 4N \cdot (SR - SR_t)^{\alpha} \tag{2}$$

Substituting equation (1) into equation (2), it is possible to compute the maximum value of the damage parameter at liquefaction, κ_L:

$$\kappa_L = 60 \cdot (SR_r - SR_t)^{\alpha} \tag{3}$$

which can be considered a synthetic expression of the liquefaction potential of the soils, since it depends on the parameters that define the cyclic resistance curve. For example, Figure 1a shows the cyclic resistance curves for a uniform and a silty sand [6]. The clean sand is characterized by a value of κ_L about 700 times lower than that of the silty sand (Table 1).

For a regular shear stress history, κ is proportional to the number of cycles, N; it is therefore possible to express the pore pressure ratio, r_u (excess pore pressure normalized by the initial effective confining pressure), as a function of the damage parameter, through the relationship proposed by the authors [7]:

$$r_u = a\left(\frac{\kappa}{\kappa_L}\right)^b + c\left(\frac{\kappa}{\kappa_L}\right)^4 \tag{4}$$

where a, b and c are parameters that control the shape of the curve, which can be easily obtained by best-fitting the data measured in cyclic laboratory tests. Eq. (4) is used to compute the generated excess pore pressure in perfectly undrained conditions. To account for the dissipation of the excess pore pressure, the one-dimensional consolidation theory [3] has been adopted, thus reducing the excess pore pressure produced at each time step in function of the consolidation coefficient of the soil [8]. The normalized damage parameter is also reduced according to eq. (4). Consequently, the damage parameter is computed as a balance between the damage generated by the load and the reduction induced by the consolidation process.

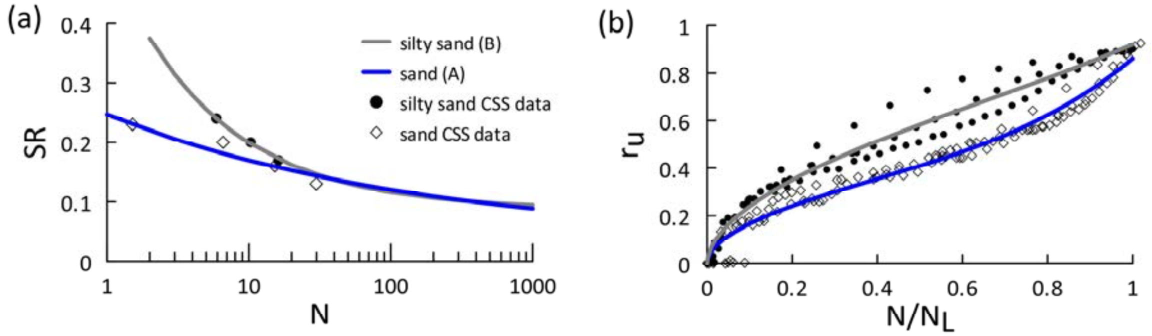

Fig. 1. (a) Cyclic resistance and (b) excess pore pressure ratio curves.

Table 1. Parameters of the 'PWP model'.

Soil deposit	κ_L	α	SR_t	SR_r	a	b	c	k [m/s]
B – silty sand	0.95	1.71	0.087	0.176	0.902	0.534	0.098	1×10^{-6}
A – sand	0.0014	5.20	0.032	0.159	0.702	0.613	0.298	3×10^{-5}

3. Dynamic analysis of a dyke damaged during 2012 Emilia earthquake

The 'PWP model' was implemented in the non-linear code SCOSSA [7,8] which models the soil profile as a system of consistent lumped masses, connected by viscous dampers and springs with hysteretic behaviour [4,9]. The non-linear shear stress-strain relationship is described by the MKZ model [10] and the modified Masing rules [11]. The code permits to select both options of total and effective stress analyses.

The code was used to simulate the behaviour of a sandy soil deposit constituting a river bank at the site of Scortichino (Emilia, Italy), where significant evidences of soil deformation and building damages were observed after the M_w 6.1 earthquake of 20.V.2012 [6].

The input motion was defined through a selection of records within the Italian database ITACA [12], based on the magnitude and distance bins approach. Five accelerograms matching the requirements were scaled to the peak ground acceleration equal to 0.262 g, estimated at the site of Scortichino from the attenuation law [13]. The records were further processed following the criteria suggested by [14] for selecting reference ground motions for liquefaction analysis of earth levees. The seismic ground motion recorded at the Lauria station during the M_w 5.6 Pollino earthquake (9.IX.1998), was finally selected and adopted as reference input motion (Fig. 2b).

The input was applied as an outcrop motion at the bedrock, which was modelled as deformable medium with shear wave velocity V_S = 800 m/s (Fig. 2a). With reference to the boundary condition for consolidation, the interface soil-bedrock was considered as pervious.

In-situ and laboratory geotechnical investigation, carried out after the earthquake, allowed to define an accurate subsoil model for the dynamic analyses [6]. Figure 2a shows the soil layering and the related shear wave velocity profile, as obtained by analyzing the borehole logs and geophysical tests. The core of the dike (AR) and its foundation soil (B) consist of a silty sand, while a thick formation of alluvial sands (A), interbedded by clay (C), overlies an alternation of both materials (AL) and the bedrock.

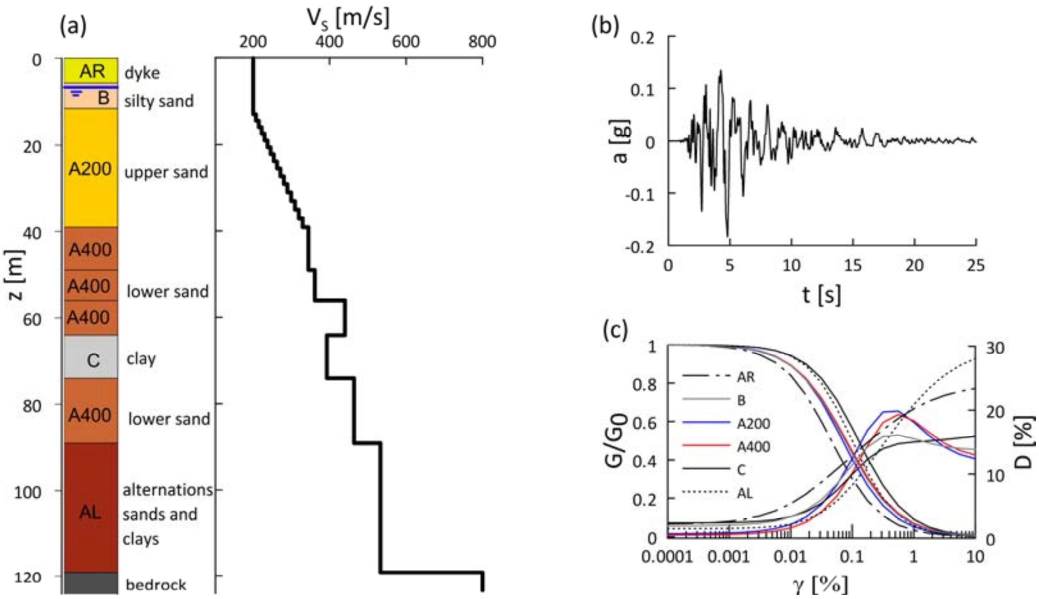

Fig. 2. (a) Layering and V_S profile; (b) reference input motion; (c) stiffness and damping vs strain.

Resonant column and cyclic simple shear tests were carried out [6] to obtain the variation of the normalized shear modulus, G/G_0, and damping ratio, D, with the shear strain, γ, required to simulate the non-linear and dissipative soil behaviour (Fig. 2c). In the present study, the shear modulus reduction curves were analytically fitted by the MKZ model, modified according to the procedure for strength compatibility proposed by [15], in order to better match the soil behaviour at large strains up to failure.

The cyclic resistance curves for silty sand (B) and sand (A) deposits were obtained from cyclic simple shear tests [6]. As anticipated, Figure 1a shows the experimental results and the curves reproduced by the 'PWP model' for two soil samples representative of the silty sand and clean sand deposits, respectively. The number of cycles at liquefaction, N_L, was counted assuming that liquefaction occurs at a pore pressure ratio r_u = 0.90. Since the threshold shear stress ratio, SR_t, was not clearly defined by the experimental data, its value was estimated from the τ:γ backbone curve as that corresponding to the volumetric threshold strain measured in resonant column tests.

The relationship between the pore pressure ratio and the damage parameter was defined as the best-fitting function through the available laboratory results (Fig. 1b); the numerical values of the parameters a, b and c are listed in Table 1. The hydraulic conductivity of the soils was either directly obtained from field measurements (Lefranc tests) or indirectly estimated using empirical correlations with CPT tests [6,8]. The one-dimensional compression modulus was inferred from the shear stiffness, by assuming a Poisson's ratio equal to 0.3.

The results of the total and effective stress analyses are compared in Fig. 3 in terms of vertical profiles of the peak values of acceleration, shear strain, shear stress, and excess pore pressure ratio (for the effective stress analysis only).

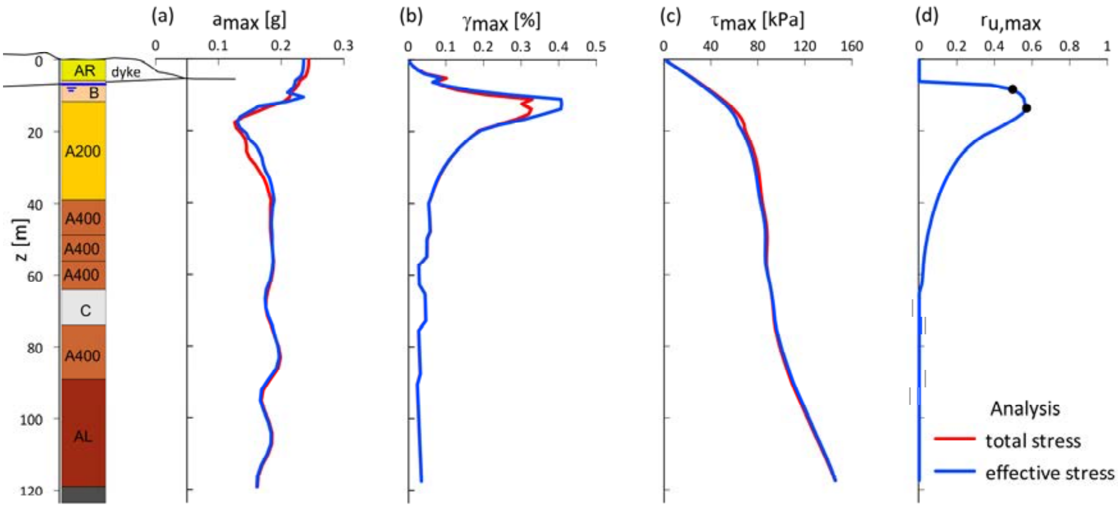

Fig. 3. Vertical profiles of (a) acceleration, (b) shear strain, (c) shear stress and (d) pore pressure ratio resulting from total and effective stress analyses.

The acceleration profile obtained by the effective stress analysis differs in the shallowest 40 m of soil deposits from that obtained by total stress analysis (Fig. 3a), showing a slight increase in the clean sand between 20 m and 40 m and a reduction of the maximum acceleration in the silty sand embankment.

Even if the shear stress profile remains substantially the same in both analyses (Fig. 3c), higher strains are attained in the effective stress analysis (Fig. 3b), as a consequence of the stiffness degradation due to the excess pore pressure build-up. The maximum strain profiles show the highest values at the interface between silty and clean sand, with a peak value of 0.4 % and 0.3 % in effective and total stress analysis, respectively.

In the coupled analysis, excess pore water pressure is significant between 7 and 40 m depth (Fig. 3d), but no liquefaction conditions are reached. Maximum values of excess pore pressure, as high as about 60 % of the initial effective confining pressure, are highlighted in the upper layers of clean (A) and silty sand (B).

Figure 4 reports the time histories of shear stress and excess pore pressure ratio at 8.4 and 14 m depth, i.e. in the silty and clean sand, respectively. It can be noted that, in both soils, the consolidation process starts after the most critical stage of the time history (4 s) and that it still tends to continue after the end of the seismic shaking.

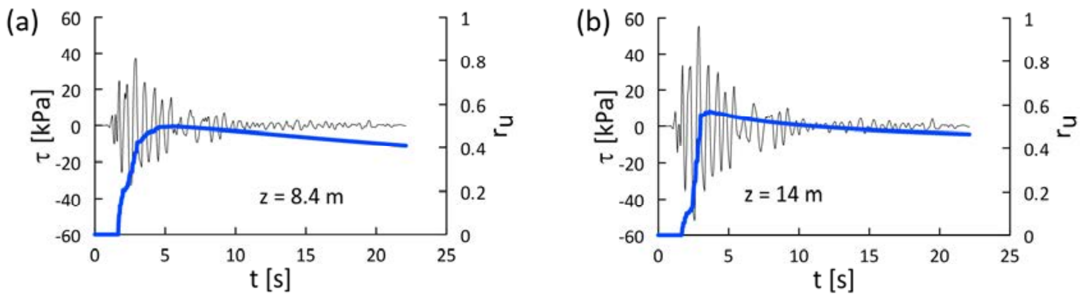

Fig. 4. Time histories of shear stress and excess pore pressure ratio at (a) 8.4 m (b) 14 m depth from ground level.

4. Conclusions

A simplified model for generation and dissipation of excess pore water pressure has been presented, based on the results of cyclic stress-controlled laboratory tests. The main advantage of this approach is the possibility to simulate the development of excess pore water pressures under irregular cyclic loading. The key objective of the 'PWP model' is to accurately simulate the experimental liquefaction resistance curve. The high sensibility of the results to the threshold shear stress ratio [8] requires that this value has to be assessed carefully. An indirect estimation can be obtained if the stiffness decay and the threshold volumetric strain are known from pre-failure cyclic/dynamic laboratory tests. The other model parameters can be calibrated straightforward through a non-linear regression analysis of the data from cyclic liquefaction tests, avoiding trial and error procedures.

With reference to the dissipation model, boundary conditions are determined on groundwater table depth, while the definition of the most appropriate consolidation coefficients for all soil layers still remains a demanding issue.

Effective stress analyses have been performed in order to simulate the seismic response of a river bank damaged during the 2012 Emilia earthquake. The results showed that only partial liquefaction occurred within the shallow sandy layers of the dyke, even though a significant excess pore pressure was induced in the silty and clean sand deposits.

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