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Application of a Simplified Model for the Prediction of Pore Pressure Build-up in Sandy Soils Subjected to Seismic Loading

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ABSTRACT

The most common procedures for predicting pore pressures generated by earthquake loading in saturated soil deposits are usually calibrated on laboratory data obtained in strain- or stress-controlled cyclic tests. A new stress-based approach under one-dimensional conditions is proposed, based on a single variable, called 'damage parameter', allowing the direct generalization of the cyclic test data to irregular stress histories. This method may greatly simplify dynamic response analyses of saturated sand deposits with both total and effective stress approaches, by removing the need for evaluating equivalent uniform stress cycles and avoiding to assess soil parameters for sophisticated plasticity-based models. Due to its simple and explicit formulation, the model can be incorporated in a time domain program to perform coupled effective stress dynamic analyses. Its implementation in a nonlinear 1D code has been examined through the analysis of a case study of a dyke damaged during the 2012 Emilia seismic sequence.

Introduction

The build-up of excess pore water pressure in sandy soils during seismic loading causes reduction in stiffness and strength and can lead to liquefaction. Consequently, the importance of numerical predictions of pore water pressure has been widely recognized for reliable evaluations of strong-motion response and liquefaction of saturated deposits.

Pore water pressure (pwp) changes can be taken into account in seismic soil response in two ways: (1) 'decoupled' seismic response analyses in total stresses, used in combination with simplified semi-empirical models for pwp generation, and (2) 'coupled' analyses, where the pwp change, computed by either a simplified or an advanced soil constitutive model, leads to a reduction of soil strength and stiffness.

Simplified pwp models are all based on the results of cyclic laboratory tests carried out in strain- or stress-controlled conditions. The earliest models were built on the stress-based approach (Seed et al., 1975; Booker et al., 1976); subsequently, Dobry et al. (1985) proposed a strain-based model where the pore pressure build-up occurs only when a threshold strain is exceeded. To apply these empirical models for the prediction of pore pressures induced in saturated soil deposits, it is preliminarily necessary to convert the earthquake into an equivalent number of cycles of uniform shear stress. The latter, in principle, should produce the same pore pressure build-up expected at the site. A wide range of conversion procedures is available for evaluating

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the equivalent number of cycles (Seed et al., 1975; Annaki & Lee, 1977; Biondi, 2002; Green & Terri, 2005). These procedures, however, are rather complex and their results strictly depend on the adopted conversion curve and on the techniques for choosing and counting the stress cycles that significantly affect the pore pressure build-up (Biondi et al., 2012). To bypass such conversion procedures, Park et al. (2014) presented a stress-based model based a on a single variable, called 'damage parameter', which can be computed for both cyclic test data and irregular stress histories. This model can be applied to carry out dynamic response analyses of saturated sand deposits with both total and effective stress approaches, by removing the need for evaluating equivalent uniform stress cycles. The approach is based on well-established empirical relationships, thus allowing for a simple calibration of the parameters.

In this paper, the formulation of the model, its generalization and guidelines for calibrating its parameters are briefly recalled. The model has been incorporated in a 1D time domain program, in order to carry out both decoupled and coupled dynamic analyses. Both approaches were applied and compared to analyse the case history of a dyke damaged during the seismic sequence occurred in Emilia plain (Italy) in May, 2012.

Main features of the simplified pore pressure model based on the damage parameter

Very recently, Park et al. (2014) suggested a pore pressure model based on the measurements in cyclic undrained stress-controlled tests. The proposed approach permits to compare the irregular time-history of shear stress induced by earthquake with the liquefaction resistance, this latter evaluated in the laboratory by applying uniform series of cycles of shear stress to soil specimens. The comparison is expressed in terms of the so-called 'damage parameter', which can be computed for any loading pattern avoiding equivalence criteria.

The damage, κ, is an incremental function of the shear stress ratio, SR, i.e. the shear stress normalized by either the mean effective confining pressure in a cyclic triaxial test or the effective vertical stress in a simple shear test. The function increases when SR overcomes a value, SR_t, that represents the threshold below which no pore pressure can build up, as follows:

$$SR > SR_t \Rightarrow \Delta\kappa = (SR - SR_t)^{\alpha} \tag{1}$$

$$SR < SR_t \Rightarrow \Delta\kappa = 0 \tag{2}$$

The parameters SR_t and α can be evaluated from the cyclic resistance curve obtained from a stress-controlled cyclic test. In detail, the parameter α describes the steepness of the (N, SR) curve, where N is the number of cycles, while SR_t is the horizontal asymptote of the curve, that is defined adopting the following equation (Park & Ahn, 2013):

$$\frac{(SR - SR_t)}{(SR_r - SR_t)} = \left(\frac{N_r}{N}\right)^{\frac{1}{\alpha}} \tag{3}$$

where (N_r, SR_r) is a reference point on the cyclic resistance curve.

Since for a regular shear stress history the damage parameter, κ, is proportional to the number of cycles, N, it is possible to represent the pore pressure ratio, r_u, as a function of the damage parameter and to define the best fitting relationship on the available experimental results.

Dynamic analysis of a dyke damaged during 2012 Emilia seismic sequence

The above mentioned model was used to estimate the pore pressure build-up, likely induced by the Emilia earthquake in Italy (20.V.2012, M_w=6.1), in the sandy soil deposits constituting a river bank at the site of Scortichino, where significant evidences of soil deformation and building damages were observed after the main seismic event (Gottardi et al., 2014). Ground cracks about 2 to 5 cm wide developed along the dyke crest, while sand boils were not observed at surface, so that it is not yet clear if soil liquefaction triggered the damage. Cracks and local sand flows were conversely observed on the ground surface near the downstream side of the dyke, so as in other sites with similar soil deposits (Fioravanti et al., 2013).

To this aim, updated dynamic analyses were carried out by adopting decoupled and coupled approaches, both implemented in the 1D non-linear code SCOSSA (Tropeano et al., 2011). The code models the soil profile as a system of consistent lumped masses, connected by viscous dampers and springs with hysteretic behaviour. The stress-strain relationship is described by the MKZ model (Matasovic & Vucetic, 1993) and the modified Masing rules (Phillips & Hashash, 2009).

In the decoupled approach, the shear stress-time history, as computed at specific depths by a total stress seismic response analysis, was then introduced as seismic demand into the proposed pwp model, to predict 'off-line' the accumulation of pore pressure build-up.

In the coupled approach, instead, an 1D effective stress analysis was performed by implementing the proposed pwp model in SCOSSA. In this case, excess pore water pressures were computed at each time step and the resulting reduction of effective stress and stiffness were taken into account in the response analysis.

Input motion

Seismic response analyses were carried out with reference to the main event (20.V.2012) of the sequence. Since no acceleration records were available at the site of Scortichino, the input motion was defined through a selection of records within the Italian database ITACA, based on the magnitude and distance bins approach. Five accelerometric time histories matching the requirements were then scaled to the PGA estimated at the site through the attenuation law by Bindi et al. (2011) and further processed following the criteria suggested by Athanasopoulos-Zekkos & Saadi (2012) for selecting reference ground motions for liquefaction analysis of earth levees. The NS component of the mainshock of Irpinia earthquake (11/23/1980), recorded at the Lauria station (Figure 1c) was finally selected and adopted as input motion in the analyses.

Geotechnical model

An extensive in-situ and laboratory geotechnical investigation carried out after the earthquake allowed to define an accurate subsoil model for the dynamic analyses (Gottardi et al., 2014). Figure 1a shows the stratigraphic sequence and the related shear wave velocity profile, as obtained by analyzing the results of boreholes 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.

Figure 1b shows the normalized shear modulus and damping ratio curves, obtained from resonant column and cyclic simple shear tests and adopted to simulate the non-linear soil behaviour. The shear modulus reduction curves were analytically fitted by the MKZ model, modified according to the procedure for strength compatibility proposed by Gingery & Elgamal (2013), in order to better match the soil behaviour at large strains up to failure. As an example, Figure 2 reports the analytical curves obtained for the silty sand deposit (B) at a depth of 10.3 m.

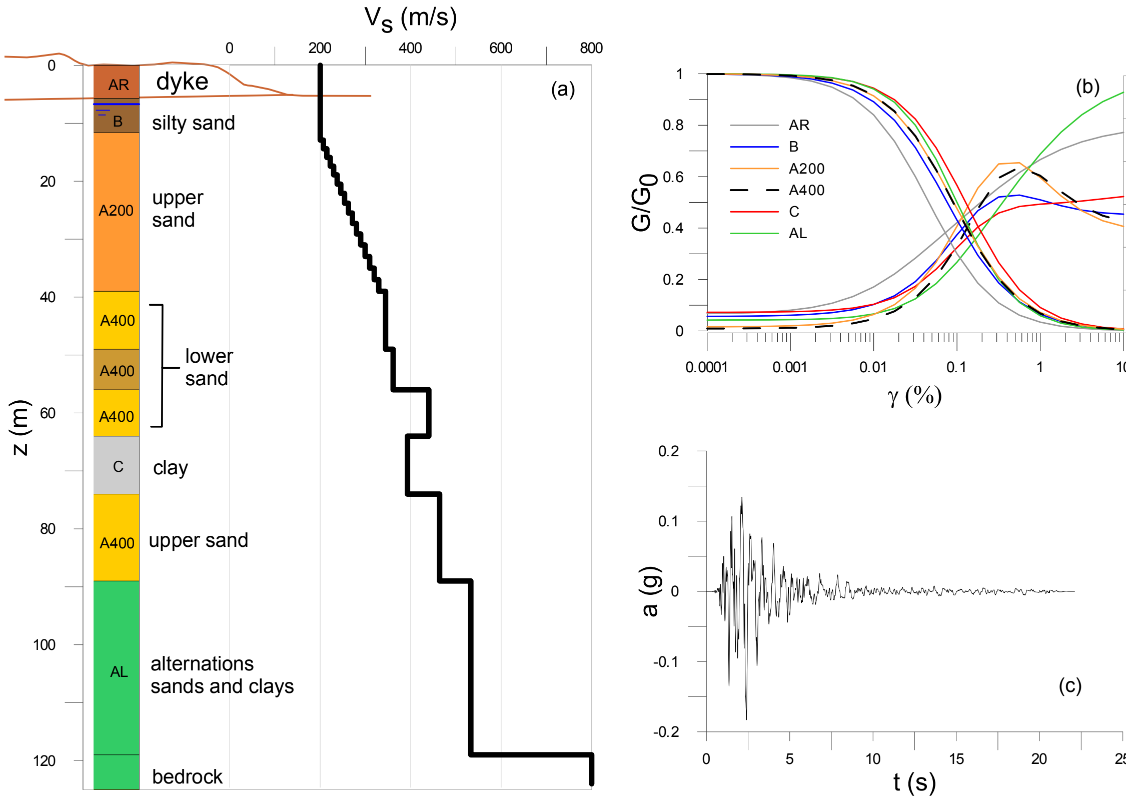

Figure 1. (a) Soil profile, (b) stiffness and damping vs strain, and (c) reference input motion

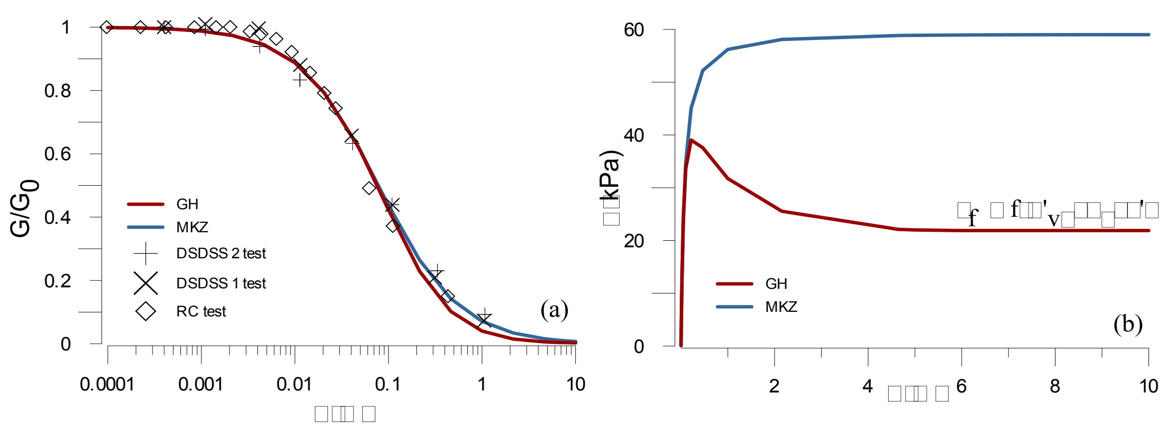

Figure 2. (a) Shear modulus reduction curve and (b) backbone curve reproduced by MKZ and Gingery & Elgamal (2013) models at a depth of 10.3 m in the silty sand deposit B

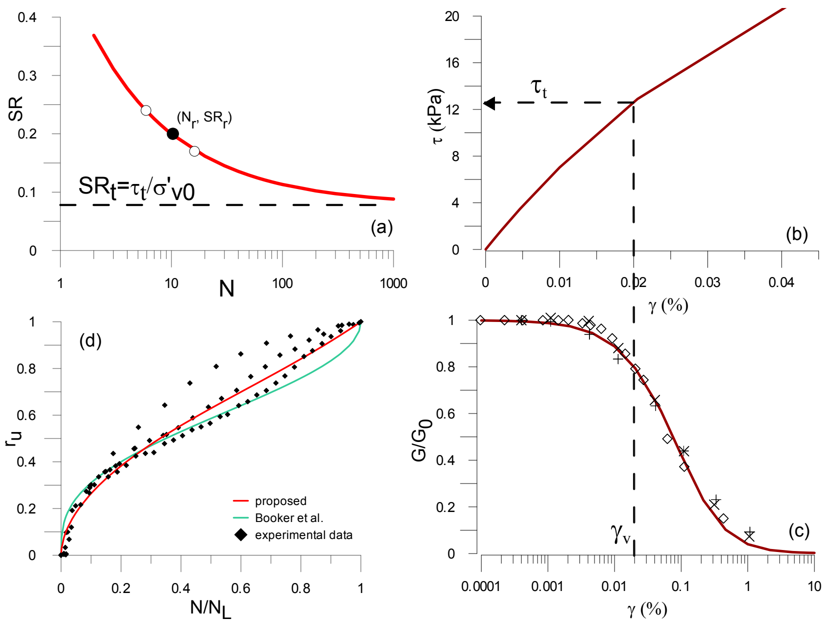

Figure 3. (a) Cyclic resistance curve, (b) backbone curve τ-γ, (c) shear modulus reduction curve, (d) r_u-N/N_L relationship for the silty sand deposit B

Cyclic resistance curves for silty sand (B) and sand (A) deposits were obtained performing Cyclic Simple Shear tests (Working group AGI-RER, 2013). These data were then used to calibrate the parameters of the pwp model by Park & Ahn (2013). In Figure 3a and 3d the experimental results taken on a soil sample at 10.30 m depth are plotted. The number of cycles at liquefaction, N_L, was established assuming that liquefaction occurs at a pore pressure ratio r_u=0.90. Since the threshold shear stress ratio was not clearly defined by the experimental data, SR_t was estimated from the backbone curve as that corresponding to the volumetric threshold strain measured in RC tests (Figure 3b-c). The parameter α was finally determined from the slope of the cyclic resistance curve in the logarithmic plot (Figure 3a).
The relationship between the pore pressure ratio and the damage parameter was defined as the best fitting function through the available laboratory results (Figure 3d), as follows:

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

where a, b and c are curve-fitting parameters. The proposed relationship allows for better describing the experimental r_u dependency on N, compared to other literature approaches.
The parameters calibrated for the two sandy soil deposits are reported in Table 1.

Table 1. Parameters of the pore water pressure model

Soil deposit	α	SR_t	N_r	SR_r	a	b	c
Silty sand-B	1.85	0.078	5.9	0.24	0.91	0.53	0.098
Sand-A	3.62	0.08	1.5	0.23	0.70	0.61	0.297

Decoupled analysis

The results of the decoupled analysis are plotted in Figure 4 in terms of vertical profiles of maximum acceleration, shear strain and shear stress (blue lines). The distribution of the maximum strain shows the highest value at a depth of 10.9 m, while the peak of maximum shear stress ratio is attained at 15.1 m depth. The shear stress history computed by the response analysis at these two depths were considered in order to evaluate the excess pore pressure ratio by applying the proposed model. According to this model, liquefaction triggers in sand deposit (A) after 3.2 s from the beginning of the seismic event, while in the silty sand deposit (B) a maximum pore pressure ratio of 0.47 is reached (Figure 5 a-c). Due to the values of SR_t evaluated for the two soil layers, only the first seconds of the records are meaningful to generate excess pore water pressure (Figure 5).

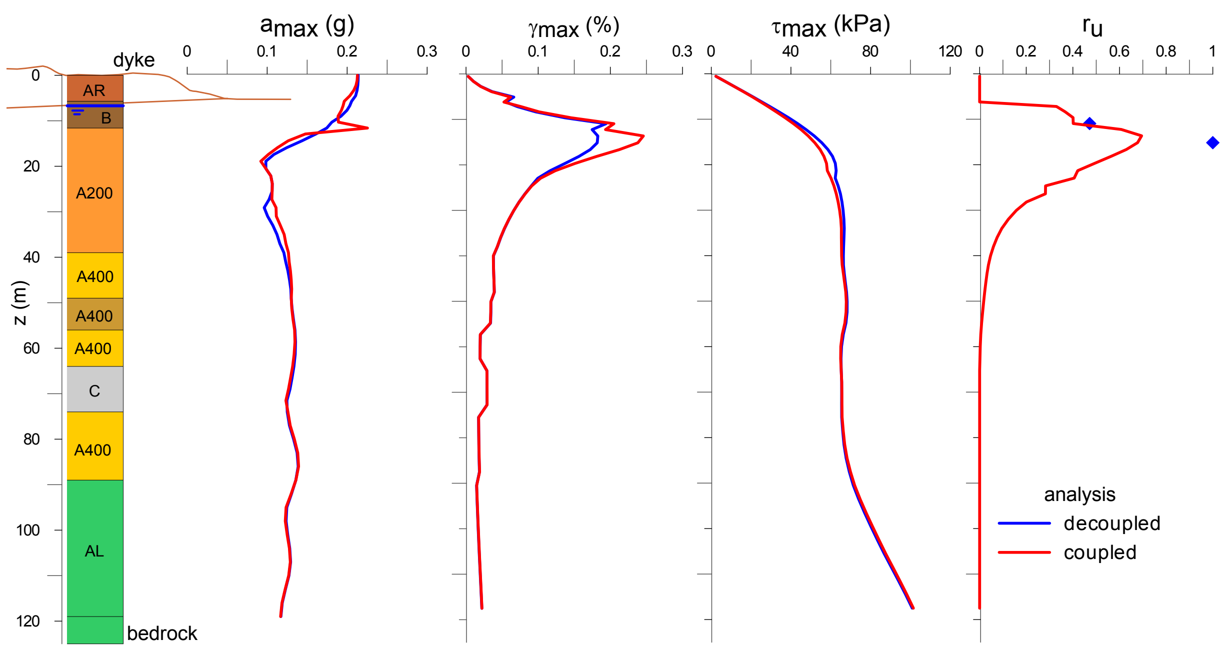

Figure 4. Vertical profiles resulting from coupled and decoupled dynamic analyses

Coupled analysis

For the coupled analysis, the proposed pwp model was implemented into SCOSSA code so that the generation of excess pwp in soils yields a reduction of soil stiffness, which was taken into account adopting the modulus degradation model for cohesionless soils proposed by Matasovic & Vucetic (1993).

The results are shown in Figure 4 and 5b-d. Excess pore water pressure is significant between 7 and 30 m depth, but no liquefaction conditions are reached. Compared to total stress analysis, the profiles of a_{max} and γ_{max} show two singular spikes at 10.9 m and 15.1 m, respectively in the silty sand (B) and sand (A) layers.

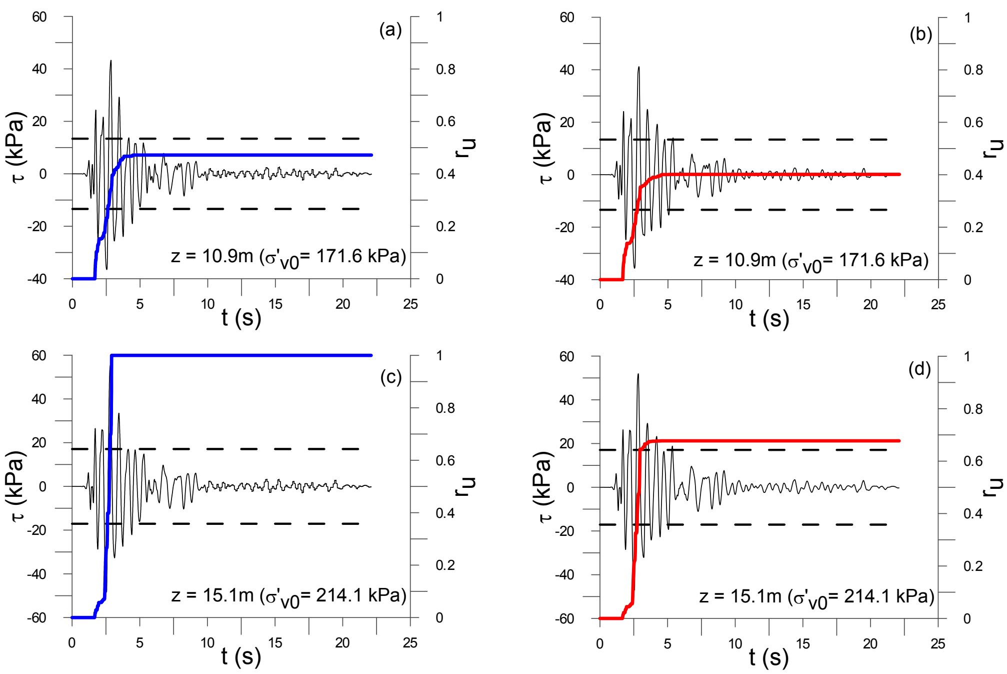

Figure 5. Shear stress and excess pore pressure ratio histories at 10.9 m (a-b) and at 15.1 m (c-d) from decoupled (blue lines, a-c) and coupled (red lines, b-d) analyses

Conclusions

A simplified model for generation of excess pore water pressure has been presented, based on the results of stress-controlled laboratory tests. The main advantage is the ability of the model to directly simulate the development of excess pore water pressures under irregular cyclic loading. From a modelling viewpoint, 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 requires that this value has to be assessed if a threshold volumetric strain is known. The other parameters of the model can be calibrated straightforward from a non-linear regression analysis, avoiding trial and error procedures.

The pwp model was successfully implemented in the numerical code SCOSSA together with the Matasovic & Vucetic (1993) degradation model and Gingery & Elgamal (2013) strength compatibility criterion.

Decoupled and coupled effective stress analysis were performed in order to predict excess pore water pressure induced by 2012 Emilia earthquake in the sandy subsoil of a river bank.
Comparing the results (Figure 4 and 5) it is shown that the coupled analysis is less conservative for predicting pwp with respect to the decoupled approach, because the reduction of soil stiffness at each time step is taken into account by the degradation model. On the other hand, an increase of maximum accelerations and strains is observed within the layers where the pore pressure ratio is above 0.4 in the coupled analysis.

The proposed coupled model is currently not able to simulate the dissipation and redistribution of excess pore pressure within a soil deposit, which should be considered for a more realistic simulation, taking into account for the recovery of stiffness and strength during the post-cyclic consolidation process. Also a relationship between the cyclic resistance curve and the effective confining pressure could be taken into account for a better modelling of the soil.

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