Prediction of seismically induced voids and pore fluid volume/pressure redistribution in geotechnical earthquake engineering

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ABSTRACT
The interaction of pore fluid and solid skeleton in porous materials is very complicated and not well understood (or used) by most practicing engineers especially in the area of geotechnical earthquake engineering. In some cases this interaction can have significant impacts on the practice of geotechnical engineering. In this paper we present a numerical approach for prediction of the redistribution of voids and pore fluid volume/pressure in saturated granular materials due to seismic loading. Seismic loading may create conditions for liquefaction depending on the density and permeability of soil layers at the site. An especial case has been analyzed to more closely look at the impacts of this phenomenon in a layered sloping site. In this case, the elasto-plasticity of porous solid, and its full interaction with the pore fluid play crucial roles in realistic prediction of the system response.

RÉSUMÉ
L'interaction des fluides interstitiels et du squelette solide dans les matériaux poreux est très complexe et mal comprise (ou utilisée) par la plupart des ingénieurs en particulier dans le domaine du génie parasismique géotechnique. Dans certains cas, cette interaction peut avoir des répercussions importantes sur la pratique de la géotechnique. Dans cet article, nous présentons une approche numérique pour la prédiction de la redistribution des volumes des vides et de la pression des fluides dans les pores de matériaux granulaires saturés en raison de charges sismiques. Les chargements sismiques peuvent créer des conditions pour la liquéfaction en fonction de la densité et la perméabilité des couches de sol sur le site. Un cas particulier a été analysé de plus près afin d'examiner les impacts de ce phénomène dans un terrain en pentes stratifiées. Dans ce cas, l'élastoplasticité des solides poreux et son interaction complète avec le fluide des pores jouent un rôle crucial dans la prévision réaliste de la réponse du système.

1 INTRODUCTION

The response of a saturated sand deposit to earthquake motions is a very important and difficult problem for which a completely satisfactory generalized solution is not yet available. The dynamic response, at least for loose to medium sands, is dominated by the effects of progressive pore water pressures that develop during an earthquake. Dynamic shear stresses and shear strains generated by the earthquake cause slip at grain to grain contacts. This intergranular slip, in dry sands, would lead to volumetric compaction at the typical shear strain levels that are generated in sand during earthquakes. In saturated sands, the volumetric compaction is retarded because the water cannot drain instantaneously to accommodate the volume change. Consequently the relaxing sand skeleton transfers some of its intergranular or effective stresses to the pore water and the pore water pressure increases. The corresponding reduction in effective stress leads to a structural rebound in the sand skeleton to absorb the difference in volume between the compaction due to grain slips and the reduction in pore water volume due to increased pore water pressure and drainage. In the extreme case, the pore water pressures developed during the earthquake may increase leading to extensive decrease in the intergranular or effective stresses and liquefaction. In this state the sand has no significant shearing resistance and deformations may become considerable. In reality, depending on the rate of loading and the properties of the soil layers there could be simultaneous generation and dissipation of pore water pressure. Thus the rate of increase of pore water pressure will be less than for completely undrained sand. The pore water pressures generated by an earthquake will not be in instantaneous equilibrium in the system and a continuous redistribution takes place under the gradients established by the earthquake loading. The pore water pressure established at any time reflects the net effects of contemporaneous generation and redistribution. At

the same time the density or void ratio of the soil layers change during the redistribution of the pore water pressure. Void redistribution due to earthquake loading may have an important influence on the residual shear strength, or steady-state strength, of saturated sands (NRC 1985).
The very complex response of the saturated granular deposit in seismic events have been better understood in recent years through numerous physical model and element test studies. In this paper we present a numerical approach for prediction of the redistribution of voids and pore fluid volume/pressure in saturated granular materials due to seismic loading. To this end, the elasto-plasticity of porous solid and its full interaction with the pore fluid have been taken into account as they play crucial roles in realistic prediction of the system response. Numerical simulations used in this study are carried out using our implementation of fully coupled u-p-U elements with the SANISAND material model in an advanced finite element framework for application in the area of geotechnical earthquake engineering. A brief introduction to this numerical framework is presented in sections 2 and 3. The embedded verified and validated theoretical framework in our numerical analyses approach allow us to look more closely at some fundamental mechanistic details governing the response of layered soil and in particular the mechanism of the redistribution of voids and pore fluid volume/pressure. Some details and capabilities of our analysis tool have been briefly presented. Then an especial case has been analyzed to more closely look at the impacts of the mentioned redistribution phenomenon in a layered sloping site. Details of the observations have been presented and discussed.

2 GLOBAL FINITE ELEMENT FORMULATION

The mechanical model of the interaction between solid skeleton and pore fluid, when combined with a suitable constitutive description of the solid phase and with efficient, discrete, computation procedures, allows one to solve most transient and static problems involving deformations. The modeling framework described here, based on the concepts originally outlined by Biot (1956), is appropriate for saturated porous media. Three general continuum formulations (Zienkiewicz and Shiomi, 1984) are possible for modeling of the fully coupled problem (soil skeleton and pore fluid) in geomechanics, namely the (a) u-p, (b) u-U, and (c) u-p-U formulations. Here, the unknowns are the soil skeleton displacements u; the pore fluid (water) pressure p; and the pore fluid (water) displacements U.
The u-p formulation captures the movements of the soil skeleton and the change of the pore pressure, and is the most simplistic one of the three mentioned above. This formulation neglects the accelerations of the pore fluid (except for combined (same) acceleration of pore fluid and solid), and in one version neglects the compressibility of the fluid (assuming complete incompressibility of the pore fluid). This formulation must rely on Rayleigh damping to model velocity proportional energy dissipation (damping). The majority of the currently available implementations are based on this formulation (e.g., Gajo et al. 1994, Elgamal et al. 2002, Taiebat et al. 2007).
The u−U formulation tracks the movements of both the soil skeleton and the pore fluid. This formulation is complete in the sense of basic variable, but might still experience numerical problems (volumetric locking) if the difference in volumetric compressibility between the pore fluid and the solid skeleton is large.
The u-p-U formulation resolves the issues of volumetric locking by including the displacements of both the solid skeleton and the pore fluid, and the pore fluid pressure as well. This formulation uses (dependent) unknown field of pore fluid pressures to stabilize the solution of the coupled system. The pore fluid pressures are connected to (dependent on) displacements of pore fluid, as, with known volumetric compressibility of the pore fluid, pressure can be calculated. The u−p−U formulation involves a somewhat complex implementation, with 7 DOFs per node (in 3D), that is 3 solid displacements u_i, one pore fluid pressure p and three fluid displacements U_i. A very important advantage of u−p−U formulation over commonly used u−p formulations is that velocity proportional damping is introduced directly through the damping tensor which is a functions of porosity and permeability of the soil skeleton. This damping provides for physically based energy dissipation due to interaction of pore fluid and the solid (soil) skeleton. It should be emphasized that u-p-U approach does not use Rayleigh damping. Another important advantage is that large difference of compressibility of pore fluid and solid skeleton does not influence the computational process as two dependent fields for pore fluid (pore fluid displacements and pressures) are among unknowns, which stabilizes the solution procedure. In addition to those advantages, the inclusion of both solid skeleton and pore fluid displacements in the field of unknowns allows for independent treatment of accelerations of both constituents (skeleton and fluid) which improves accuracy of simulations. Despite it's power, this formulation has rarely been implemented into finite element codes. The formulation takes into account velocity proportional damping (usually called viscous damping) by proper modeling of coupling of pore fluid and solid skeleton, while the displacement proportional damping can be appropriately modeled using elasto-plasticity with a powerful material model. No additional (and artificial) Rayleigh damping has been used in the FEM model. Detailed description of the u-p-U formulation, finite element discretization, and time integration are presented in Jeremic et al. (2008).

3 SOIL CONSTITUTIVE MODEL

The SANISAND constitutive model is used here for modeling of soil response. SANISAND is the name for a family of Simple ANIsotropic SAND constitutive models within the frameworks of critical state soil mechanics and bounding surface plasticity. Manzari and Dafalias (1997) constructed a simple stress-ratio-controlled constitutive model for sand in a logical sequence of simple steps. The model is fully compatible with critical state soil mechanics principles; it renders the slope of the dilatancy stress ratio (also known as the phase transformation line), a function of the state parameter ψ, such that at the critical state where ψ=0 the dilatancy stress ratio coincides with the critical state failure stress ratio. In addition, softening of dense samples is modeled within a collapsing peak stress ratio bounding surface formulation. The peak stress ratio is again made a function of ψ such that at the critical state where ψ=0 it becomes the critical state stress ratio, following an original suggestion by Wood et al. (1994). The SANISAND model has been later extended by Dafalias and Manzari (2004), Dafalias et al. (2004), and Taiebat and Dafalias (2008). In the present paper the focus is on wave propagation in granular media. To involve fewer model parameters and for simplicity, the version of the SANISAND model with fabric change effects (Dafalias and Manzari 2004) has been considered as the constitutive model for the soil. Table 1 shows the parameters of the SANISAND model for Toyoura sand. These parameter are based on the model calibration for the experimental data of Verdugo and Ishihara (1996).

Table 1. SANISAND parameters for Toyoura sand.

Model parameter	Symbol	Value
Elasticity	G_0	125
	v	0.05
Critical state	M	1.25
	c	0.712
	e_0	0.934
	λ	0.019
	ζ	0.7
Dilatancy	n^d	2.1
	A_0	0.704
Kinematic hardening	n^b	1.25
	c_h	0.968
Fabric dilatancy	z_{max}	2.0
	c_z	600

The numerical implementation of the u-p-U formulation and the SANISAND model have been available within the OpenSees framework for some time and with termination of PEER related work on OpenSees is now maintained at the Computational Geomechanics group of UCDavis (http://geomechanics.ucdavis.edu).

4 VERIFICATION AND VALIDATION

Verification is the process of determining that a model implementation accurately represents the developer's conceptual description and specification. Validation is the process of determining the degree to which a model is an accurate representation of the real world from the perspective of the intended uses of the model. It is a Physics issue. Verification and Validation process will give us a better insight of the capabilities and limitations of the available numerical tools. The implementation of the u-p-U formulation and the capabilities of the SANISNAD model have been extensively examined in Jeremic et al. (2008) and Taiebat et al. (2010).

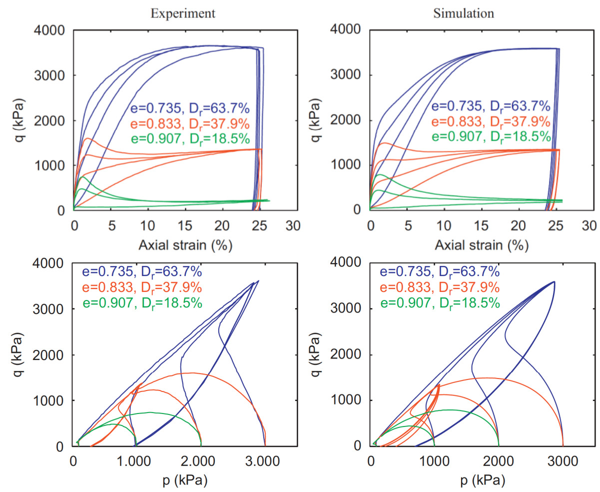
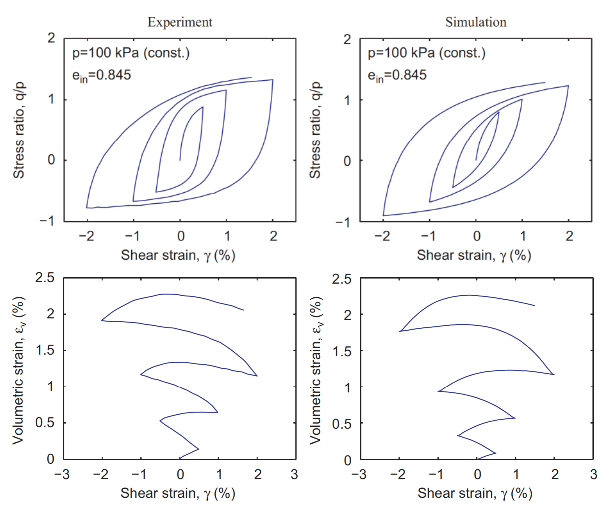


Figure 1. Simulations vs. experiments in undrained triaxial compression tests on isotropically consolidated samples of Toyoura sand (Taiebat et al., 2010)

Figures 1 and 2 show the comparison between the experimental element tests on Toyoura sand and the corresponding numerical simulations using the SANISNAD model. In particular, Fig. 1 shows 11 different undrained triaxial compression tests on isotropically consolidated samples of Toyoura sand (Data after Verdugo and Ishihara 1996).



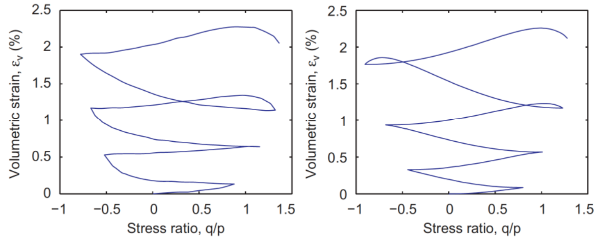


Figure 2. Simulations vs. experiments in constant-p cyclic triaxial test on a relatively loose sample of Toyoura sand (Taiebat et al., 2010)

The experiments and simulations cover a wide range of initial void ratio (density) and mean effective stress. Figure 2 shows details of the experimental results and the model simulation for a constant-p cyclic triaxial test on a relatively loose sample of Toyoura sand (Data after Pradhan et al. 1989). All these simulations are conducted using the single set of model parameters presented in Table 1. The good comparison between the experimental results and model simulations shows the capabilities of the constitutive model. In general, confidence in predictions relies heavily on proper verification and validation processes.

5 NUMERICAL PREDICTIONS

A gently sloped (1.5°) profile of saturated sand is considered consisting of dense Toyoura sand (Dr=75%) with a 1.0m thick interlayer of loose Toyoura sand (Dr=25%) in an average depth of 3.0m. The bedrock is considered at the depth of 10m and an input motion with frequency of 2 Hz and maximum acceleration of 0.4g has been applied at the bedrock in a direction parallel to the ground surface (see Fig. 3).

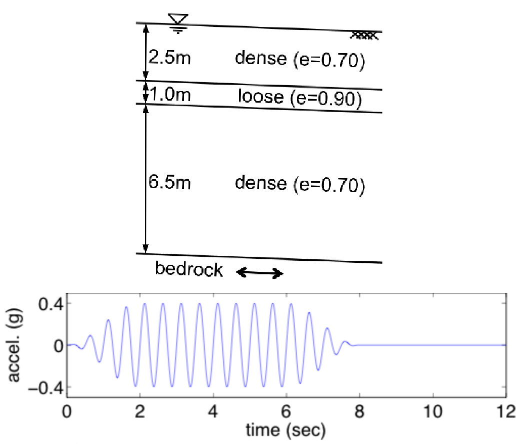


Figure 3. Schematic of site conditions and the applied acceleration at bedrock.

The response of the system has been analyzed using the u-p-U element and the SANISAND model with the calibration for Toyoura sand (Table 2). Other parameters used in the simulation are listed in Table 3. The predicted response of the soil profile in terms of variations of excess pore pressure ratio, changes of void ratio, shear strain and lateral displacements are presented in Figs. 4 and 5.

Table 3. Other parameters used in the simulation.

Parameter	Symbol	Value
Solid bulk modulus	K_s	3.6×10^7 kN/m^2
Solid density	ρ_s	2700 kg/m^3
Fluid bulk modulus	K_f	2.2×10^6 kN/m^2
Fluid density	ρ_f	1000 kg/m^3
Permeability	k	10^{-3} m/s
HHT parameter	α	-0.2

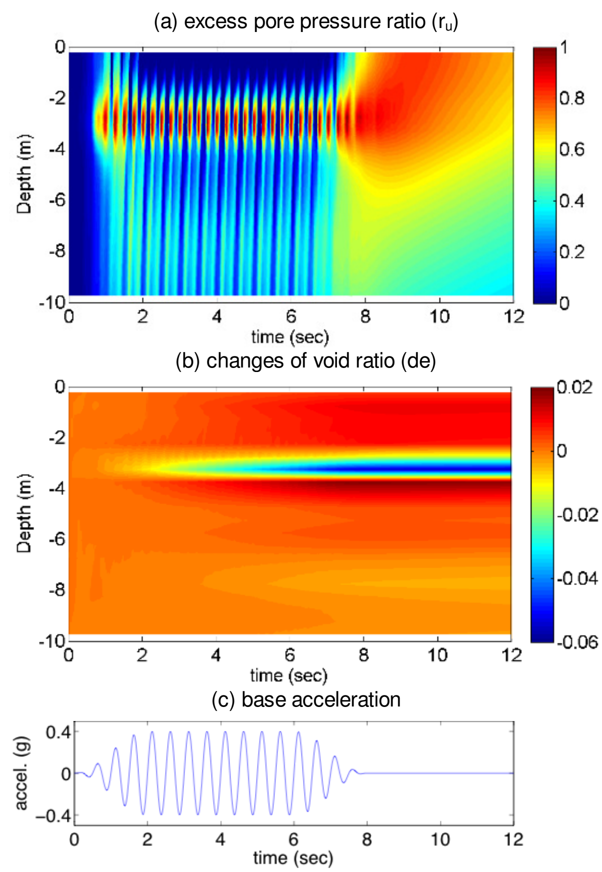


Figure 4. Variations of excess pore pressure ratio (r_u) and changes of void ratio (de) vs. time and depth in the 10m layered sloping soil column.

Figures 4(a,b) clearly show the phenomenon of redistribution of voids and pore fluid volume/pressure. The loose interlayer initially has an intense contractive tendency. In the first 2-3 cycles of loading this tendency does not show up in the volumetric strain and changes of void ratio (Fig 4b) as the pore water cannot dissipate instantaneously to accommodate the volume change. Therefore, this tendency for contraction of soil volume transfers a part of the soil effective stress to the pore water pressure. The resulting excess pore water pressure can be observed in a normalized form as the excess pore pressure ratio (r_u) in Fig 4(a). In the subsequent cycles of loading and

unloading the loose interlayer soil experiences repetitive cycles with contractive and dilative tendencies (cyclic mobility), which show up in repetitive increase and decrease of r_u. During this process the generated excess pore water pressure in the loose interlayer starts to dissipate slowly. The dissipation of the generated excess pore water pressure allows the soil to show its overall tendency for contraction and therefore by the end of shaking (8sec) the loose interlayer shows an overall decrease in void ratio (contraction) while the rest of the soil profile show an overall increase in void ratio (dilation). Note that in this analysis, the dissipation of the excess pore pressure continues after the end of shaking, however in that part it does not significantly affect the changes of void ratio as before.
Figures 5(a,b) show the progressive accumulation of shear strains and the resulting lateral displacement in the soil profile during and after the shaking event. It can be observed that the accumulated shear strains are mainly concentrated in the loose interlayer and reach up to 20% in this case. The upper 2.5m of soil slides almost as a rigid block during the shaking. The maximum lateral displacement in this case is about 0.35m.

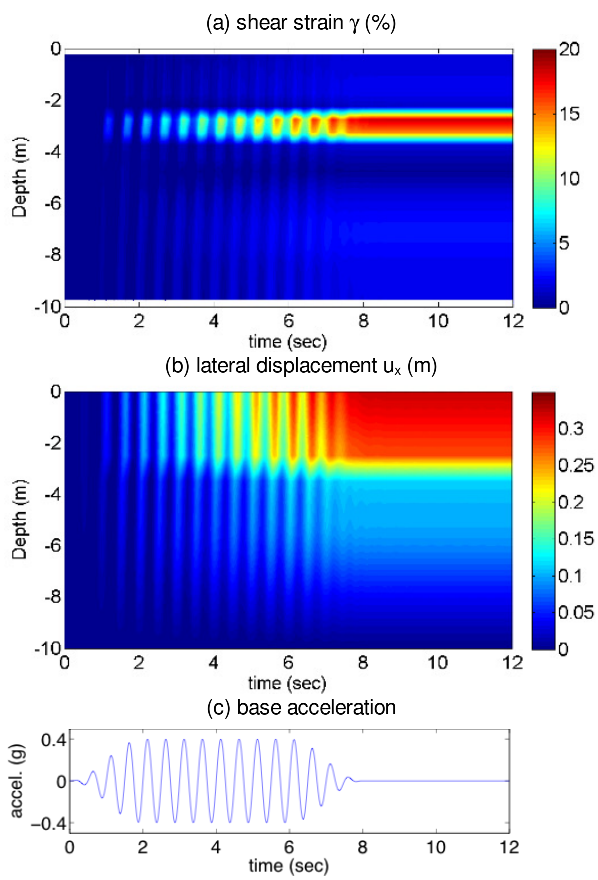


Figure 5. Variations of shear strain and lateral displacement vs. time and depth in the 10m layered sloping soil column.

6 SUMMARY AND CONCLUSION

A rigorous numerical platform has been employed in order to study the mechanism of seismically induced voids and pore fluid volume/pressure redistribution. This numerical tool is capable of simulating seismic excitation, soil softening with the accumulation of excess pore pressures, rapid loss of shear strength as the soil liquefies, redistribution of pore water, possible progressive failure, deformations continuing after dynamic loading ends, and reconsolidation as excess pore pressures drain. The verified and validated formulation in this tool allows us to look closely at the details of the mechanism of the voids and pore fluid volume/pressure redistribution during and after the shaking event. In the presented case in this study, the overall magnitude of the changes in soil's void ratio after the redistribution phenomenon is up to 7% in the loose layer. This shows that the subsequent shaking events can improve the site condition at least in the loose interlayer by densifying it during and after each event.

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