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Calibration of non-linear effective stress code for seismic analysis of excess pore pressures and liquefaction in the free field

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A B S T R A C T

The paper presents numerical predictions of excess pore pressure, liquefaction and settlement response of four centrifuge model tests of 6 m uniform deposits of saturated clean Ottawa sand, placed by dry pluviation and having a relative density ranging from 38% to 66%. The deposits were subjected to 1D uniform base shaking consisting of 10–15 cycles of peak acceleration ranging from 0.04 to 0.12 g. All predictions were conducted with the nonlinear effective stress numerical code Dmod2000. Significant effort was spent in calibrating Dmod2000 by matching the pore pressure and settlement measurements of the first shaking (S1) of a series of shakings conducted in centrifuge Experiment 3. This resulted in very good predictions of both pore pressures and settlement measured in this shaking S1. The exercise showed the importance for realistic simulations of having the correct soil compressibility and permeability. This calibrated version of Dmod2000 was used for a good pore pressure prediction of the preshaken deposit in the same Experiment 3 (S36), by modifying only one parameter in the undrained pore pressure model; and also well predicted pore pressure responses in Tests FFV3 and PFV1, without *any* change in the parameters of Dmod2000 except for use of the new input motions (Type B predictions). The experimental and numerical results showed that *both* cyclic shear stress/strains *and* upward water flow determine *together* the pore pressure buildup and liquefaction phenomena. The soil response is partially drained rather than undrained, and pore pressure dissipation does take place during shaking both before and after liquefaction occurs.

1. Introduction

Liquefaction of saturated soil due to earthquakes has been observed in many places around the world causing tens of billions of dollars in damage to buildings, ports, highways, buried pipes and other parts of the civil infrastructure [12]. Prediction of the effects of liquefaction within the context of performance-based engineering requires, between other tools, experimentally calibrated numerical simulations such as those being developed for shallow foundations by Dashti and Bray [10].

One classification found useful by researchers dealing with liquefaction and its effects in level or mildly sloping sites has been to divide the phenomenon in two parts [39]: (i) liquefaction triggering in the free field; and (ii) post-triggering phenomena which cover most of the engineering effects of liquefaction. Observations show that triggering of liquefaction of clean sands in the free field is usually a necessary condition for engineering consequences to develop. Therefore, realistic analyses of the triggering phenomenon in the free field turn out to be a necessary and important part of the whole prediction exercise. Furthermore, numerical simulation of pore water pressure buildup and triggering in the free field under 1D or 2D shaking is simpler than simulation of the response of an engineering system in the presence of liquefaction. Such free field analysis focuses on the levels of excess pore pressure caused by the shaking, up to the excess pore pressure ratio, r_u = 1.0, that defines triggering. While still complex, the pore pressure buildup and triggering phenomenon – having a well-defined objective restricted to the prediction of r_u in what is typically approximated as a 1D system of horizontal or slightly inclined soil layers – is much easier to model analytically. In addition, this free field analysis typically also allows computation of post-liquefaction ground settlement – an important parameter on its own right when evaluating engineering consequences to foundations of buildings and other systems.

Hashash et al. [36] reviewed time domain, non-linear, effective stress analysis 1D numerical tools used in the evaluation of earthquake site response which also account for excess pore water pressure generation, redistribution and dissipation. Several researchers have developed sophisticated constitutive soil models that aim at capturing the

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contractive and dilative soil behavior representing pore pressure generation and dissipation during cyclic mobility [22,33,47,53,56,70,9]. These models often require tens of parameters that are not typically available to practicing engineers. Other simplified non-linear analysis codes, more commonly used by practitioners, require less parameters and are easier to use, like DESRA [45], Dmod2000 [52], and DEEPSOIL [35]. Two systematic international efforts have taken place over the years to validate and calibrate numerical codes of varying degrees of complexity simulating liquefaction and its effects, including the free field response 1D models discussed here. These efforts include VELACS [4] and the ongoing project LEAP [50,74]. The authors have participated in both efforts in various capacities. Based on these experiences, it is clear that a good understanding of the basic soil mechanics behavior during free field liquefaction is critical to the identification and quantification of the key parameters needed to run realistic numerical simulations. Specifically, the main constitutive parameters needed to run these 1D liquefaction triggering simulations in the free field can be grouped into four groups of soil properties:

(1) Shear stiffness and damping of the soil medium including changes with shear strain level (stress-strain nonlinearity) and excess pore water pressure (stress-strain softening or degradation);
(2) Pore water pressure buildup behavior due to undrained cyclic loading;
(3) Compressibility characteristics of the soil under cyclic loading and possible change due to shaking and excess pore water pressure; and
(4) Permeability of the soil and possible change due to shaking and excess pore water pressure.

The main purpose of this paper is to calibrate and validate the nonlinear effective stress numerical code Dmod2000 [52]. This is accomplished by using the results of a centrifuge test simulating a uniform deposit of saturated clean Ottawa sand subjected to a number of 1D base shakings, conducted by one of the authors [23]. This test is labeled Experiment 3 throughout the paper. Most of the key parameters used as input to the code were experimentally obtained, either backfiguring them at a system level from the Experiment 3 results themselves (shear stiffness and compressibility), or calibrating them at the element level from laboratory small sample 1 g testing (relationship between excess pore pressure, cyclic shear strain and number of cycles in undrained condition). While the permeability used in the code was also based on a small sample 1 g permeability constant head test, its value had to be adjusted upwards in order to produce realistic results.

After a general discussion on experimentally observed pore pressure buildup in uniform deposits subjected to base shaking, the paper presents the results of centrifuge Experiment 3. The compressibility curves obtained both from this Experiment 3 as well as from other tests on different deposits of the same sand by El-Sekelly [23], are singled out for special consideration given the importance of compressibility in the numerical simulations. A parametric study is conducted next with Dmod2000 to evaluate the effect of sand permeability, another key parameter. Numerical simulations of two of the shakings of Experiment 3 are performed using Dmod2000 with best estimates of both compressibility and permeability, and the predictions are compared with the test results. Use of the same numerical model is finally extended to simulate other similar centrifuge experiments conducted on the same sand.

2. Considerations on pore pressure buildup in a uniform sand deposit

Many researchers have conducted experiments where a uniform deposit of saturated sand on an impervious base is excited by a base horizontal acceleration and the pore pressure response is monitored with embedded piezometers. The tests have been either shaking table models at 1 g or centrifuge experiments, conducted using horizontal or slightly

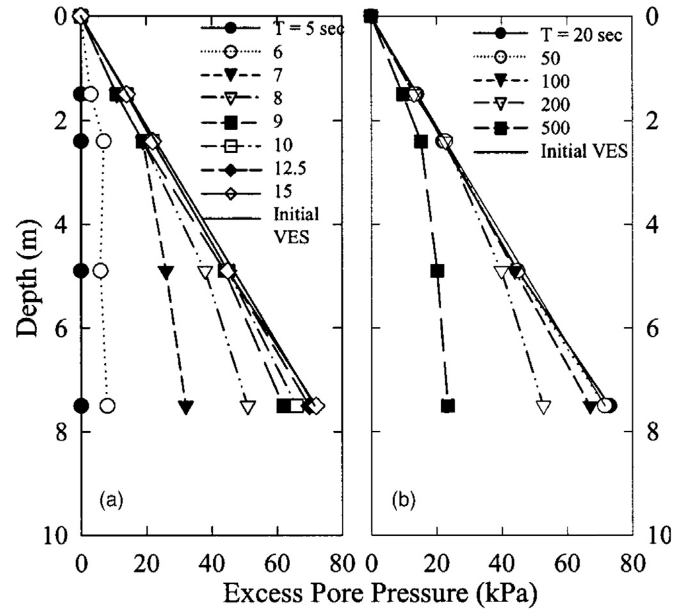

Fig. 1. Excess pore pressure profiles (isochrones) in centrifuge Test L45V-2–10 on saturated loose Nevada sand deposit: (a) during shaking; and (b) after shaking [63].

inclined rigid and laminar box containers, and including both loose and dense as well as clean and silty sands [1,19,23,32,38,5,63,65,69,72]. The results show that: (i) liquefaction starts at the ground surface and propagates downwards during the shaking, sometimes reaching the bottom of the deposit depending on the intensity and duration of the shaking, total thickness, relative density and permeability of the soil; (ii) the excess pore water migrates upwards during and after the shaking, toward the ground surface that acts as drainage boundary; and (iii) this excess pore water drainage, which translates into settlement of the ground surface, typically takes place both during and after the shaking, with the sand at deeper elevations settling (consolidating) first and that at shallow depths settling later.

Fig. 1 includes representative excess pore pressure results from a centrifuge test reported by Sharp et al. [63], simulating a loose (D_r = 45%), 10 m prototype homogeneous deposit of saturated clean Nevada sand deposited by dry pluviation, subjected to 10 s of shaking (between t = 5 s and t = 15 s), with a peak input base acceleration of about 0.2 g. The figure includes isochrones (instantaneous excess pore pressure profiles), between t = 5 s and t = 500 s, when the consolidation process is almost over. The straight line (labeled Initial VES = initial vertical effective stress), corresponds to the case where the excess pore pressure, $u = \sigma'_{v0}$, with σ'_{v0} being the initial vertical effective stress. That is, when an isochrone coincides with this straight line, the effective pressure, $\sigma'_{v0} - u = 0$, the pore pressure ratio, $r_u = u/\sigma'_{v0} = 1.0$ and liquefaction has occurred. The results in Fig. 1 are fairly typical of many tests of this kind.

It is important to recall here the concept of hydraulic gradient in the context of Fig. 1, in order to follow better the discussion below. The upward hydraulic gradient is defined as $i = (h_2 - h_1)/(z_2 - z_1)$, where $z_2 > z_1$, and h_2, h_1 are the hydraulic heads (in m) corresponding to the depths z_2, z_1, respectively. As in all cases of Fig. 1 the groundwater flow is upward, $h_2 > h_1$ and $i > 0$. Furthermore, if the datum for the hydraulic head is located at the ground surface, $h = u/\gamma_w$, where u (kPa) is the excess pore pressure plotted in Fig. 1 and γ_w = 9.81 kN/m^3 is the unit weight of water. Therefore, for Fig. 1, $i = (u_2 - u_1)/[(9.81)(z_2 - z_1)]$. Two numerical examples at different times in Fig. 1 can be used to illustrate this. For t = 15 s, the gradient between z_2 = 7.8 m and z_1 = 0 m can be calculated from $h_2 \approx 72$ kPa and $h_1 = 0$. Therefore, $i \approx (72 - 0)/[(9.81)(7.8 - 0)] = 0.94 \approx 1.0$, approximately equal to the critical hydraulic gradient, creating a quicksand condition consistent

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