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ABSTRACT

Liquefaction has occurred during numerous earthquakes and it has caused damages and catastrophic failures. This phenomenon takes place due to the excess pore pressure development in loose saturated granular soils. Researchers have attempted to predict these phenomena (excess pore water pressure and liquefaction) using constitutive modeling and numerical approaches. In this paper, a numerical modeling procedure is presented to predict the seismic excess pore water pressure using a fully coupled effective stress analysis. A few cyclic and monotonic element tests and a level ground centrifuge test conducted during VELACS project were utilized to calibrate the numerical models. The Mohr-Coulomb elastic-perfectly plastic and the Martin et. al. (1975) excess pore water pressure build up models were concurrently incorporated in the analysis. This study focuses on a reasonable step by step procedure in order to adjust and obtain the calibration parameters of these models. Comparing the excess pore pressure buildup time histories of the numerical and experimental models (both element and centrifuge tests) showed that the Martin et al. (1975) models can be used in the numerical assessment of excess pore water pressure with an acceptable degree of preciseness.

INTRODUCTION

Evaluation of the excess pore water pressure build up in the granular soils is one of the important issues in the geotechnical earthquake engineering. The contraction forces between the grains in water saturated granular soil may gradually decrease, when they are subjected to the earthquake excitation and either drainage is prevented. This is due to the fact that the normal stresses transmit from the soil grains to the pores water. The earthquake induced excess pore water pressure may gradually increase and approach to the in-situ effective stress and so cause liquefaction.

There are two main approaches for dynamic analysis of soil systems; total and effective stress methods. The major deficiency of the total stress method is the fact that it can not take into account the progressive stiffness degradation of soils due to pore pressure increments. Only effective stress methods can model the gradual loss of soil strength due to the buildup of excess pore water pressure.

Dynamic analyses based on the effective stress method can be divided into four main categories: methods based on plasticity theory (Prevost 1985, Pastor and Zienkiewicz 1990, Wang and Dafalis 1990, Ishihara 1993, Muraleetheran et al.1994, Fukutake et al. 1995) stress path methods (Ishihara and Towhata 1982, Kiku et al. 1996), methods based on correlations between pore pressure response and volume change tendency of dry soils (Finn et al.1977), and finally direct use of experimentally observed pore pressure response (Seed et al. 1977, Sheriff et al. 1978, Kagawa and Kraft 1981).

State-of-the-art procedures involve dynamic finite element or finite difference effective stress analyses coupled with fluid flow equations. These analyses can estimate the displacements, the accelerations and the excess pore water pressure induced by a given input motion.

The VELACS model # 1 centrifuge test (Dobry and Taboada 1994) representing a level ground site constituted of Nevada sand at 40% relative density has been numerically simulated in the current study. The Mohr-Coulomb constitutive model with a non-associate flow rule coupled with Martin et. al. (1975) excess pore pressure build up model have been employed in the numerical modeling. The main objective of this study is to evaluate the capability of these models in the prediction of excess pore pressure variations during cyclic loading. Some correlation relationships were proposed to determine the calibration parameters of the Martin et al. (1975) excess pore pressure build up model. The preliminary analyses and the comparisons between measured and numerical results showed that their accuracies are not satisfactory for all conditions. The results of the cyclic and monotonic tests were utilized in order to set up the calibration parameters of these models for the Nevada 40% sand.

THEORITICAL BACKGROUND

The Mechanism of progressive increase of excess pore-water pressure during undrained cyclic loading has been investigated by many researchers. Development of the quantitative relationships between volume reductions during drained and corresponding excess pore-water pressure increases in undrained conditions has been suggested by some researchers as a reasonable way to predict excess pore water pressure build up (Finn et al. 1977; Byrne 1991). These relationships allow the numerical of the excess pore-water pressure increment during undrained cyclic loading using physical parameters of the sand.
An effective stress analysis approach was initially proposed by Martin et al. (1975). Their proposed model is an equation linking the increment of the volumetric strain per cycle of loading to the shear strain occurred during that particular cycle.

Martin et al. (1975) proposed the following incremental equation for the sands under simple shear loading condition:

$$\Delta\varepsilon_{vd} = C_1 \cdot (\gamma - C_2 \cdot \varepsilon_{vd}) + \frac{C_3 . \varepsilon^2{}_{vd}}{\gamma + C_4 . \varepsilon_{vd}} \qquad (1)$$

Where;

$\Delta\varepsilon_{vd}$ *:* increment of volumetric strain in percent per each cycle of shear strain

ε_{vd} *:* accumulated volumetric strain from previous cycles in percent

γ : amplitude of shear strain in percent for the cycle in question

C_1, C_2, C_3, C_4 : Model constants depended on the relative density of sand.

Byrne (1991) proposed a modified and simpler volume change model with two calibration parameters. The governing equation was expressed as:

$$\frac{\Delta\varepsilon_{vd}}{\gamma} = C_1 \cdot \exp(-C_2 \cdot \frac{\varepsilon_{vd}}{\gamma}) \qquad (2)$$

Where; C_1 and C_2 are model constants.

Byrne (1991) recommended a correlation equation in order to obtain the model constant C_1 in term of sand relative density, D_r as:

$$C_1 = 7600(D_r)^{-2.5} \qquad (3)$$

C_1 and C_2 control the amount of volume changes and the shape of the accumulative volume changes with respect to the number of cycles, respectively. Since the shape of the accumulative volume change with number of cycles is the same for all densities, the parameters C_2 is a constant fraction of C_1 for all relative densities and can be prescribed as:

$$C_2 = \frac{0.4}{C_1} \qquad (4)$$

It is interesting to note that, these relationships for the calibration parameters could not properly work for all soils and loading conditions, since they have been developed based on a limited number of cyclic tests.

Cyclic stresses induced in the level ground during earthquakes are generally assumed to be propagated upward in the soil deposit. Various types of laboratory test procedures have used to investigate and simulate the cyclic stresses of level ground soil deposits induced by earthquake. Since the object of a laboratory cyclic test is to reproduce the stresses acting on an element of soil by an earthquake, cyclic simple shear test provide better representation of the field conditions. Therefore, in this study, the calibration parameters of the models for the Nevada 40% sand have been extracted using cyclic simple shear test results obtained during the VELACS project. In addition, drained monotonic and undrained cyclic tests data have been employed for a reasonable estimation of the dilation angle and initial shear modulus of the soil, respectively. Finally, the estimated calibration parameters have been implemented in the numerical modeling of the centrifuge test.

NUMERICAL MODELING PROCEDURE

The initial shear modulus ($G_{\max}$) is an essential parameter required for dynamic analysis. A typical procedure to obtain the equation that yields the initial shear modulus of the

Nevada sand at any given confining pressure is presented herein. This procedure is presented for the Nevada sand specimens with relative density of 40%.

Different relationships have been proposed to estimate the initial shear modulus of cohesionless soils. These equations yield the maximum shear modulus as a function of mean confining pressure and void ratio. Seed and Idriss (1970) proposed the following equation:

$$G_{\max} = 1000 K_{2\max} (\sigma'_m)^{0.5} \tag{5}$$

Where:

$G_{\max}$: maximum (small strain) shear modulus in psf,

$K_{2\max}$: shear modulus number (Seed and Idriss, 1970), and

σ'_m : mean effective confining stress in psf

In the above equation for Nevada 40% sand, the value of the $K_{2\max}$ is obtained 35 psf.

Such relationships are not precise in any soil and loading condition. In this study, it is tried to find a more precise equation to evaluate the initial shear modulus of the Nevada 40% sand. Using the laboratory tests data of the Nevada 40% sand, the tangents of the shear stress-strain curves were calculated and values of $K_{\max}$ and n in the following $G_{\max}$ equation have been obtained.

$$G_{\max} = K_{\max} \cdot (\sigma'_m)^n \tag{6}$$

This process was done for laboratory cyclic tests conducted at different confining pressures but with the same relative density (D_r =40%). The diagram of a cyclic simple shear test conducted at initial effective cell pressure of 80 kPa is seen in the Fig 1.

Two tests from the performed tests having the same relative densities but at different confining pressures, 80kPa and 160kPa, were selected. The following equation has been obtained to hand in the initial shear modulus of the Nevada 40% sand:

$$G = 16828.17(\sigma'_m)^{0.341} \tag{7}$$

Table1 shows the values of the $K_{\max}$ and n factors in the Equation 6 based on the Seed and Idriss (1970) recommendations and the equation 7.

Table 1. Comparison between coefficients of the initial shear modulus equation

Equation	$K_{\max}$ (shear modulus number)	n
Seed and Idriss equation	1674	0.5
Cyclic undrained test	16828.17	0.341

The value of the soil Poisson's ratio should be known in the analyses. Typically, ν is little sensitive to soil type, confining pressure and void ratio, but it depends very much on the degree of saturation and drainage condition (Gazetas, 1991). Consequently, it is not difficult to make a reasonable prediction of ν , if saturation and drainage condition are known. As it was recommended by Gazetas (1991), the assumption of ν =0.5 for saturated sand and ν =0.25 for nearly dry sands are reasonable values. The values of the shear and the bulk modulus are related by the following equation:

$$B = \frac{2G(1+\nu)}{3(1-2\nu)} \tag{8}$$

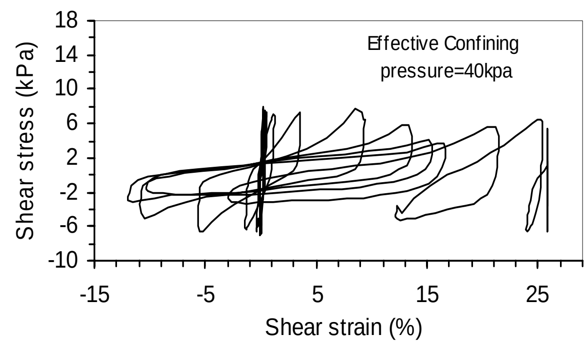

Fig1. Shear stress-strain loop of the Nevada 40% sand at $\sigma'_0 = 80kPa$ *under cyclic simple shear test condition, VELACS Test No. CSS40-09*

Use of the Drained Monotonic Test Data

Seismic excess pore water pressure causes large reduction in the shear stiffness and large strains may occur due to such large stiffness reduction. This condition is known as flow failure and it commonly occurs in loose sands. In contrast, when soil grains are very close to each other and constitute a dense cluster, they have a tendency to dilate. This dilation causes the excess pore water pressure to drop and the stiffness to increase and so it limits the strains induced by loading. The drained monotonic test number CIDC 40-107, conducted during the VELACS project, was modeled by FLAC software to calibrate the value of dilation angle which is needed in the non-associated Mohr-Coulomb flow rule. For this purpose, the values of dilation angle have been changed to find the target value which matches the results of the numerical model and laboratory test. Figure 2 shows the deviatory stress variations versus the axial strain obtained from the numerical model and the experimental test data.

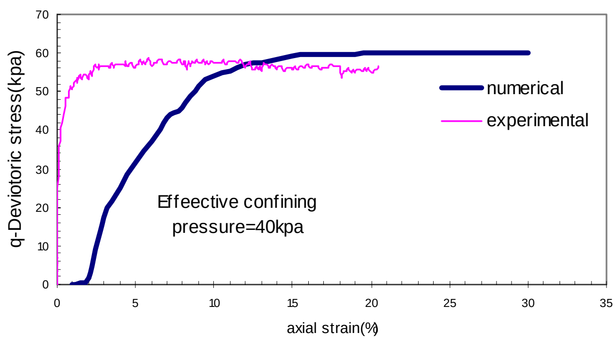

Fig 2 .comparison between the numerical and measured deviatory stress versus axial strain in a monotonic drained triaxial test on the Nevada Sand at Dr = 40%, VELACS Test No. CIDC 40-107

The results demonstrate that at the relative density of 40% and the confining pressure of 40kPa, a dilation angle equal to 0.2 could reasonably match the experimental and numerical results. This low dilation angle can be reasonable since it shows low dry density of the Nevada 40% sand and verifies the contractive and the hardening data behavior of such loose sand.

Use of the Undrained Cyclic Test

The test number CSS40-09 is a direct simple shear laboratory test conducted during the VELACS project on the Nevada 40% sand under the initial effective confining pressure of 80kPa. This laboratory test was numerically simulated to obtain the parameter that is required to calibrate the Martin et al. (1975) excess pore pressure model constants for this soil. In the numerical model, the initial shear modulus was implemented according to the estimated values mentioned before.

The results showed that, the Martin et al. (1975) model with four constants obtains more reasonable results than the Byrne (1991) model with two constants. Therefore, the Martin et al. (1975) model was used in the numerical modeling. Table 2 shows the value of the Martin et al. (1975) model constants that could provide the best match between the numerical and experimental excess pore pressure build up values in this undrained element test.

Table 2.values of the model constant obtained from the back analyses of the undrained cyclic direct simple shear test

Test	C_1	C_2	C_3	C_4
Undrained cyclic test	0.8	0.75	0.438	0.73

Soil engineering parameters used in the numerical modeling of the undrained tests are presented in the Table 3.

Table 3. Evaluation of the values of the soil parameters to be used in the numerical modeling of the laboratory tests

tests	Dry Density(γ) (N/m^3)	Friction angle (ϕ)	Dilation angle (ψ)	G(MPa)	B(MPa)
Undrained cyclic	1527	33	0.2	75	912..35

Figure 3 shows the excess pore pressure build up resulted from the numerical model and also laboratory test. The results illustrate that the mentioned numerical procedure can produce reasonable results, as seen from the comparison between numerical and actual excess pore pressure build up values.

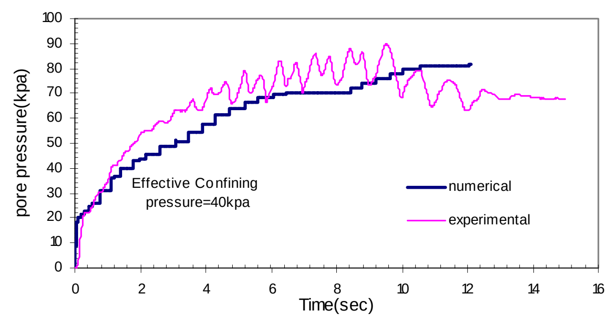

Fig 3. Comparison between the excess pore pressures obtained fromthe numerical analysis and cyclic simple shear test on the Nevada Sand at Dr=40%, VELACS Test No. CSS40-09

Numerical Modeling of the Centrifuge Test

In the previous sections, calibration parameters of the Martin et al. (1975) model of the Nevada 40% sand were appropriately obtained via back calculating of the drained and undrained cyclic and monotonic tests. In this part of the paper, the performance of the Martin et al. (1975) excess pore water pressure model is evaluated by the direct use of the obtained calibration parameters in the numerical modeling of the VELACS centrifuge model No.1. Figure 4 illustrates this centrifuge test model conducted during the VELACS project at the RPI. The geometrical positions of the transducers are shown in Table 4.

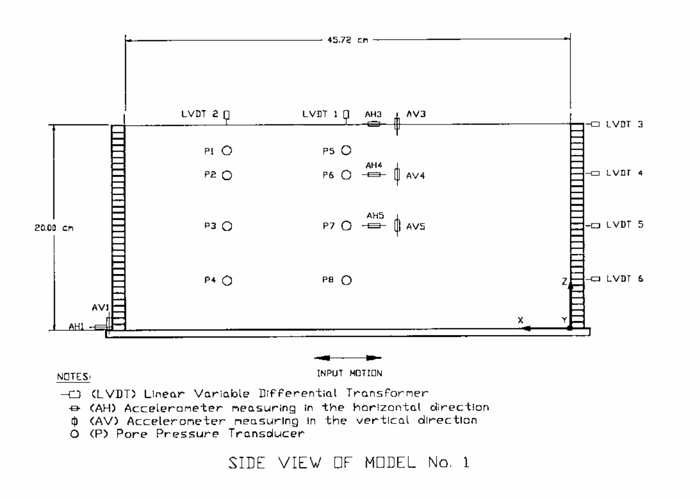

Fig 4. Centrifuge model # 1 conducted at the RPI during the VELACS project

This test was performed at the relative density of 40%. The parameters obtained from the previous sections were used in the numerical modeling.

The centrifuge model was divided into some parallel layers. The input base motions are two acceleration time histories in vertical and horizontal directions, shown in the Figs 5 and 6. These input motions were applied at the base of the centrifuge model.

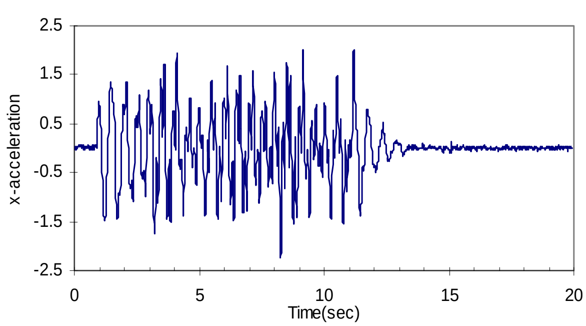

Fig 5. Input horizontal acceleration

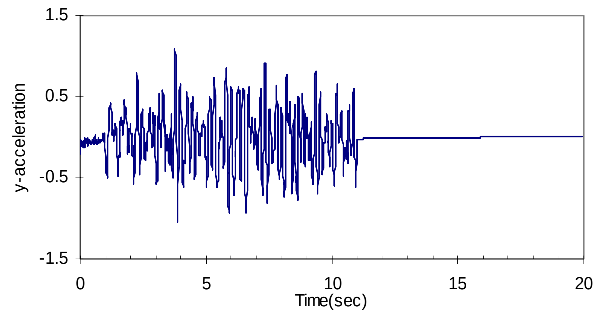

Fig 6 .Input vertical acceleration

Table 4. The positions of the centrifuge model instruments

Transducer	Instrument ID	Coordinates (m)		
		X	Y	Z
Accelerometer measuring in vertical direction	AV1	0	6.25	0
	AV3	13.5	6.25	10
	AV5	13.5	6.25	5
Accelerometer measuring in vertical direction	AH1	0	6.25	0
	AH3	12.5	6.25	10
	AH4	12.5	6.25	7.5
Pore pressure transducer	P5	10.5	6.25	8.5
	P6	10.5	6.25	7.5
	P7	10.5	6.25	5
	P8	10.5	6.25	2.5

The centrifuge model contains a laminar box with slipping "rings" that allows differential horizontal displacements. This was simulated in the FLAC model by free-field boundary conditions which prevent reflection of the waves in the side walls.

Static analysis was carried out before dynamic analysis in order to find initial stress and strain state. The boundary condition and the contour of the initial effective vertical stress at the numerical model are demonstrated in the Figs 7, 8, and 9. In the next stage, the dynamic loads were applied at the base of the model and dynamic analysis was performed. The final comparisons between the estimated excess pore pressure in the numerical model and the measured values are shown in the Figs 10, 11, 12, and 13.

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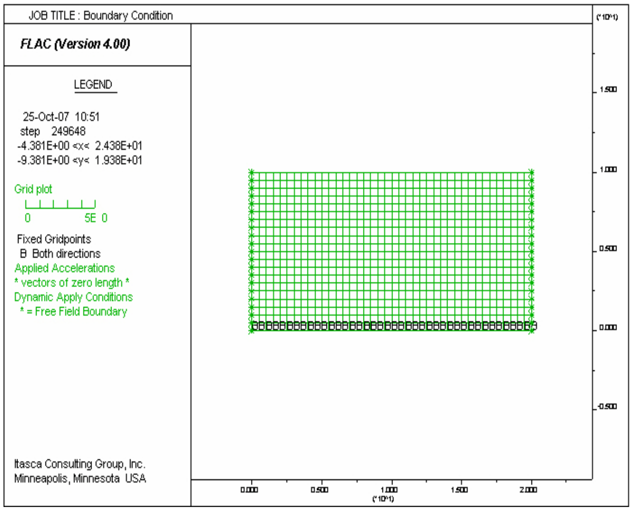

Fig 7. Boundary condition used in the FLAC model

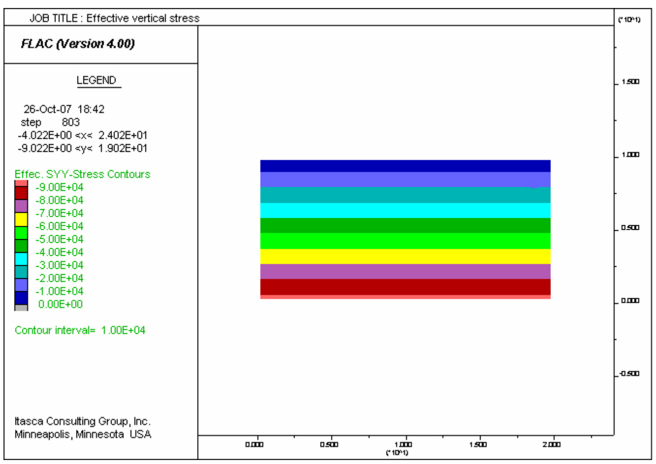

Fig 8. Contours of the static vertical effective stress obtained from the numerical analysis

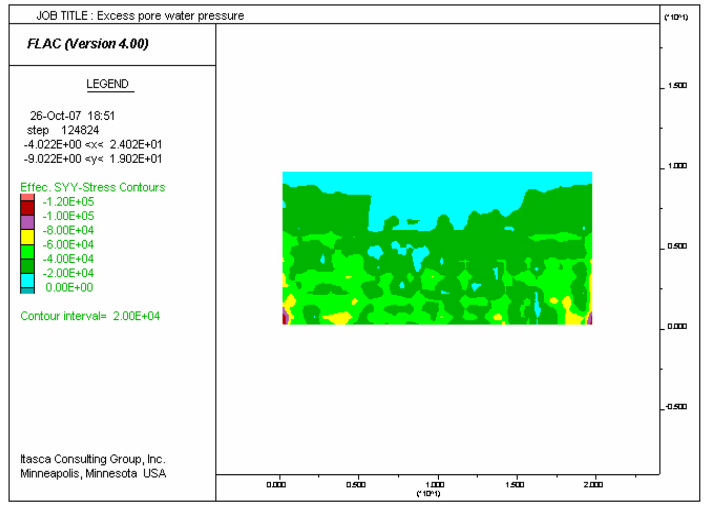

Fig 9. Contours of the pore pressure obtained from the numerical analysis at t=10sec

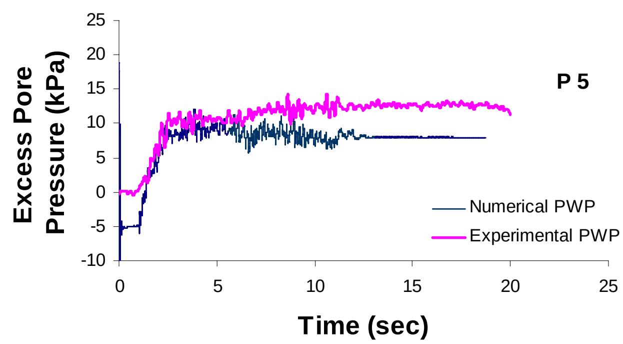

Fig 10. Comparison between the actual and numerical excess pore pressure time histories at the depth 1.45 m.

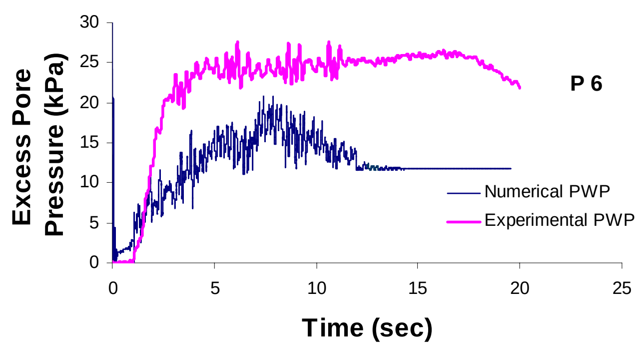

Fig 11. Comparison between the actual and numerical excess pore pressure time histories at the depth 2.6 m.

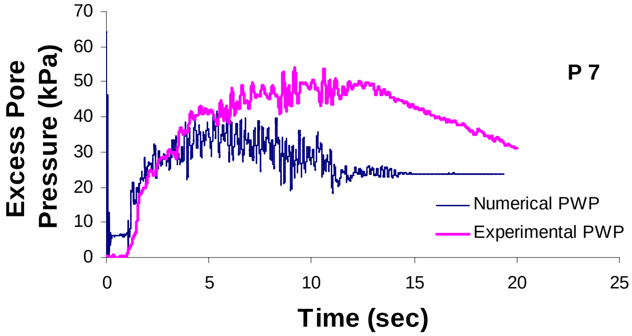

Fig 12. Comparison between the actual and numerical excess pore pressure time histories at the depth 5 m.

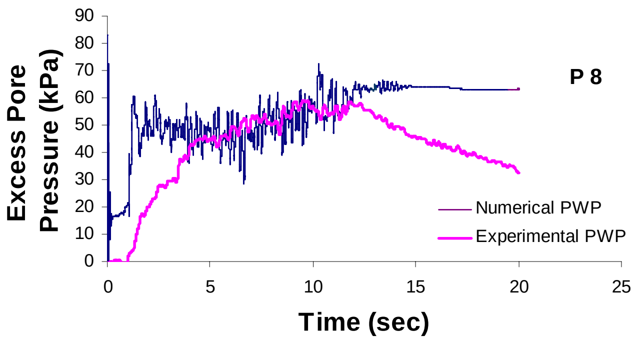

Fig 13. Comparison between the actual and numerical excess pore pressure time histories at the depth 7.5 m.

It can be seen from the Figs 10, 11, 12 and 13 that the numerical and experimental results have not been matched in a perfect manner, but with these results one can have a general sense on the performance of the PWP model considered here. This occurred in spite of the preliminary step by step analyses performed to evaluate the calibration parameters of the excess pore water pressure model.

It is seen that the numerical model has predicted lower EPWP values than the actual values (except for the case of 7.5m). This may be originated from the fundamental assumption of the Martin et al. (1975) EPWP theory, in which excess pore water pressure is directly related to the relevant volume changes. The Mohr-Coulomb constitutive model shows continuous dilative tendency when strain rate vector touches its yielding surface. This tendency decreases the compressive volume changes during cyclic loading. Therefore, the lower EPWP than the actual values may be estimated by the numerical model.
On the other hand, the Martin et al. (1975) model was adopted for one-dimensional measures of shear strain, while, in a 2D analysis under both horizontal and vertical shakings, there are three strain rate measures. FLAC uses some assumptions to solve this problem and it can affect the results. Besides, the Finn model can not consider the increase of shear modulus due to the densification of soil during cyclic loading.

This study illustrates the complexity of the seismic excess pore pressure build up phenomenon.

CONCLUSIONS

A simple numerical framework is presented to assess the excess pore pressure generation during a cyclic loading in a given centrifuge test. A finite difference numerical simulation model, using FLAC program, was prepared to simulate the excess pore pressure time history during seismic loading. Martin et al. (1975) excess pore pressure buildup model was incorporated in the coupled effective stress analyses. Data of the element tests performed on fine, clean, Nevada 40% sand during the VELACS project were used in order to calibrate the excess pore pressure buildup model constants. Then, the VELACS centrifuge model # 1 test was numerically simulated by directly use of the excess pore pressure model constants obtained due to the preliminary calibration analyses of the element tests.
According to the results, predicted excess pore water pressures did not closely match to the measured excess pore pressure values in the centrifuge test. From the comparison between numerical and recorded excess pore water pressure values, it is seen that the Martin et al. (1975) model has underestimated the excess pore water pressure value. These results and findings are only relevant to the soil and loading conditions concerned in the present study.

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