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SYNOPSIS: An effective stress based, finite element procedure of modeling earthquake soil and soil-structure interaction problems is described. Elasto-plastic constitutive models are used to describe the stress-strain behavior of soils. A fully-coupled finite element formulation is employed, which allows the pore pressure build-up and dissipation to be modelled simultaneously. Undrained behavior is modelled as a special case of this general formulation. In a previous study, the procedure was used to perform a "before the event" prediction of the liquefaction behavior of a 10-meter thick saturated sand deposit subjected to an earthquake loading. In the present paper, the predictions are compared with experimental centrifuge data. In a second study, an earthquake soil-structure interaction problem modelled in a centrifuge was analysed and the results compared. The results of this study are also summarized in this paper.

INTRODUCTION

Earthquake problems have in the past been analysed using total stress based linear or equivalent linear approaches. These procedures are, however, inadequate to capture the inherent nonlinearity of soils' behavior and the nonlinearity caused by changes in the pore water pressure. With advancements in the effective stress based constitutive modeling, nonlinear finite element modeling and computer technology, it is now possible to analyse earthquake soil and soil-structure interaction problems by a more rational, effective stress based procedure. In this paper, the earthquake behavior predicted using such a procedure is compared with centrifuge observations for a liquefaction problem and for a soil-structure interaction problem.

LIQUEFACTION BEHAVIOR OF A SATURATED SAND DEPOSIT

An effort was recently made to evaluate the capabilities of existing numerical procedures in predicting the behavior of liquefiable prototype structures subjected to earthquake loading. A number of structures were tested in geotechnical centrifuges employing simulated earthquakes, and their liquefaction behavior observed in terms of the time histories of pore pressures, accelerations and displacements at selected locations. Researchers with predictive capabilities were called upon to perform "before the event" predictions of the proposed centrifuge events.

The author made "before the event" predictions of the behavior of three such centrifuge models and the details may be found in Anandarajah (1993a). The recorded data on the behavior of these models were disclosed at the symposium. In the present paper, the predictions of one of the models is compared with experimental data.

A generalized, elasto-plastic stress-strain model was recently developed for describing the behavior of granular materials under monotonic (Anandarajah 1994b) and cyclic (Anandarajah 1994a) loading conditions. A simplified version of this model was used in the predictions reported in the present paper, which involves a total of nine parameters that are to be determined from test data. The details of this simple model may be found in Anandarajah (1993a). The model is developed within the framework of the bounding surface plasticity. While most models proposed in the past employed non-associated flow rules, the model used here employs an associated flow rule. Consequently, the theory yields a symmetric stiffness matrix, allowing finite element analyses to be performed in an efficient manner. Another unique feature of the model is that it employs a special procedure to model the pore pressure response during stress reversals. As a result, the model is able to simulate the pore pressure response more realistically than most of the existing plasticity models. An example of the comparison between experimental and theoretical results is presented in Fig. 1. The results represent the triaxial undrained behavior of Nevada sand (the one used in centrifuge models) of 40% relative density. The values of the model parameters for Nevada sand is listed in Table 1.

Table 1: Model Parameters for Nevada Sands of Dr=40% (N-Sand40)

Soil	λ^*	M_{fc}	G_0	n_e	m_1	h_0	h_1	h_r	α_r
N-Sand40	.014	1.354	90	1.0	0.2	10	8	1.0	.01

The numerical procedure is based on a set of fully-coupled dynamic equations for two-phase porous media. The finite element formulation employed is that of Zienkiewicz and Shiomi (1984). Specifically, the most general formulation, where the final matrix equations are in terms of solid and fluid displacements and pore pressure, is employed. The matrices involved in this form are symmetric. As indicated in the preceding section, the constitutive model also yields a symmetric stiffness matrix. The entire analysis, therefore, involves symmetric global stiffness matrices. An active column solver is used to solve the final equations. Two-dimensional finite element models are used to approximate the centrifuge models. Eight noded finite elements with solid and fluid displacement fields expressed in terms of eight nodal unknowns and pore pressure field expressed in terms of four unknowns at the corner nodes are employed. 3x3 and 2x2 Gauss integrations are employed for the respective matrices. One-dimensional (e.g., simple shear) condition is simulated by slaving appropriate degrees of freedom. A simple trapezoidal rule is employed in integrating the constitutive rate equations. Sub-increments are used to integrate the equations accurately. The time integration is performed using the Hilber-Hughes-Taylor method with a suitable value for numerical damping parameter. A computer program named HOPDYNE (Anandarajah 1990), which, among many other features, has the above numerical aspects coded into it. Capabilities of HOPDYNE are summarized in Appendix 1. In addition to the constitutive model parameters, the fully-

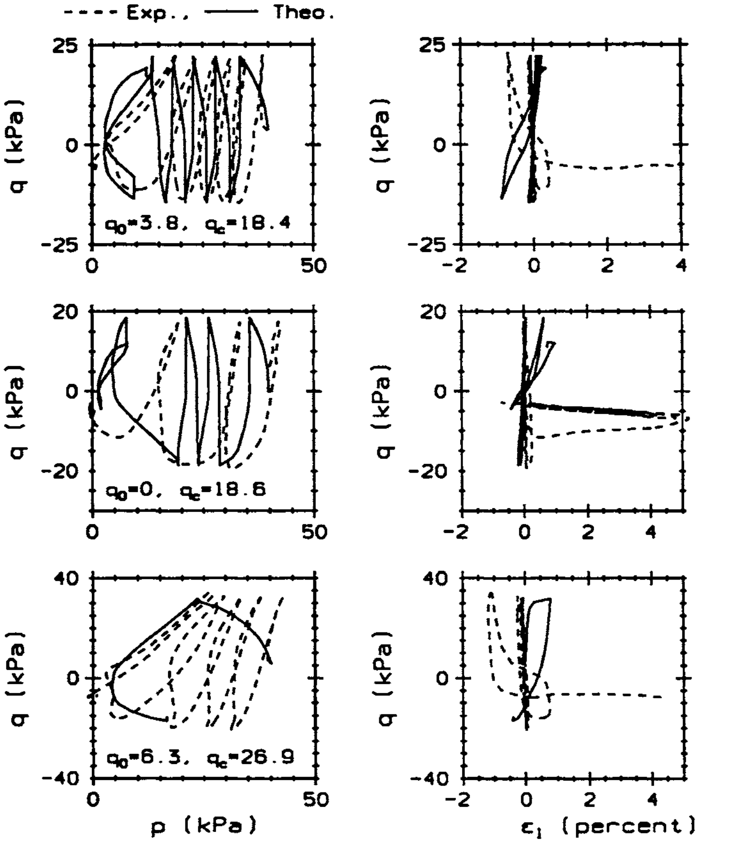

Fig. 1. Comparison of the Model Behavior with Experimental Data of Cyclic Triaxial Behavior of Nevada Sand of Dr=40 percent at p_0=40 kPa.

Table 2: Parameters used in analysing Model 1 by HOPDYNE

k m/sec	n	ρ_s kN-sec^2/m^4	ρ_f kN-sec^2/m^4	K_0	γ' kN/m^3
3.3x10^{-3}	.420	2.7	1.0	.55	9.55

-coupled analysis requires the following properties of the soil: permeability (k), porosity (n), and density of solid (ρ_s) and fluid (ρ_f). The coefficient of earth pressure at rest (K_0) and the effective density (γ') are required in establishing the initial stresses in the soil. These properties are listed in Table 2.

The details of the centrifuge test may be found in Taboada and Dobry (1993). Fig. 2 presents a sketch of the model, along with the locations of pore pressure transducers, accelerometers and LVDT. The test was conducted in a "laminar box" to simulate a horizontal deposit of prototype soil. The model was constructed using uniform Nevada sand at 45% relative density, and fully saturated. The prototype thickness of the soil layer was 10 meter.

Assuming a one-dimensional condition, the finite element mesh shown in Fig. 3 was used to model this problem. The results of the "before the event" predictions are compared with centrifuge observations in Figs. 4-7. The accelerations at the transducer locations AH1 (base), AH5 (mid-height) and AH3 (ground surface) are compared with centrifuge data in Fig. 4. Note that the soil looses its ability to transmit the waves due to liquefaction during the shaking. The numerical predictions compare reasonably well with observed data. The short term pore pressure histories at the transducer locations P6, P7 and P8 (see Fig. 2) are compared in Fig. 5. The apparent good agreement indicates the ability of the constitutive model to capture the pore pressure response reasonably well. The long term pore pressure histories are compared in Fig. 6. The rate of dissipation is predicted to be slower than the observation. The ground settlements are compared in Fig. 7. While the final settlements are reasonably close, the theory under predicts the settlement during shaking. The observed higher rate of pore pressure dissipation and settlements are suspected to be due to increase in the permeability of soil during liquefaction.

EARTHQUAKE SOIL-STRUCTURE INTERACTION BEHAVIOR OF A PILE-SUPPORTED TWO STOREY STRUCTURE

A two-dimensional model of the Cypress Freeway that collapsed during the 1989 Loma Prieta earthquake was made and tested in a one-meter radius centrifuge. The complete details of the tests and analyses may be found in Anandarajah et al. (1994). The model was constructed in a rigid centrifuge bucket. The foundation soil was made from a silty clay named Yolo loam. The superstructure consisted of a two storey structure, supported by two columns resting on pile caps. Each pile cap was supported by 20 cylindrical piles. The soil was fully saturated. The results presented here are in prototype scale.

The foundation soil was represented by a bounding surface elasto-

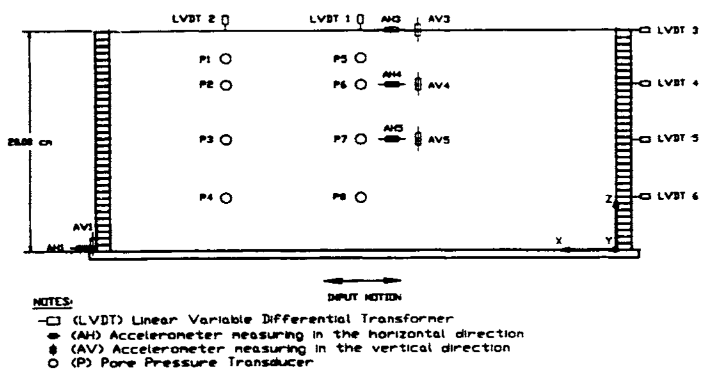

Fig. 2. Centrifuge Model.

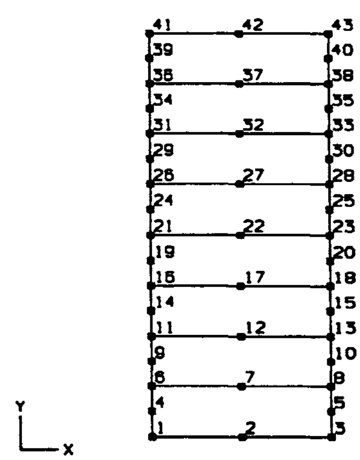

Fig. 3. Finite Element Mesh

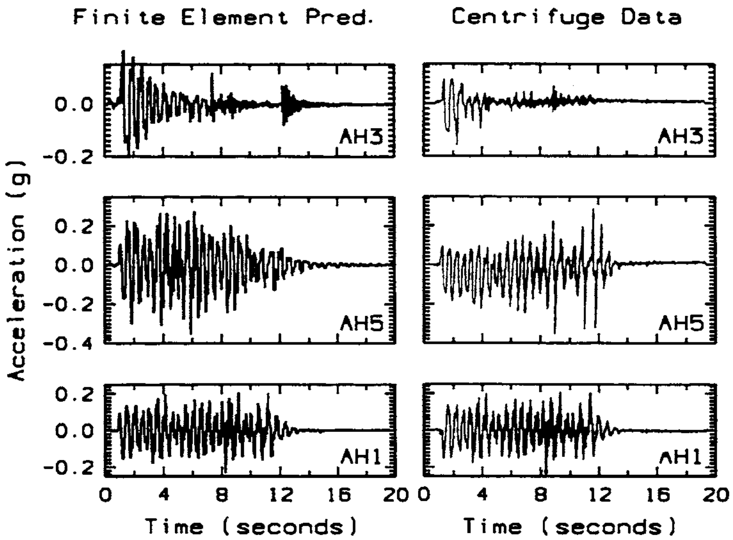

Fig. 4. Horizontal Acceleration Histories.

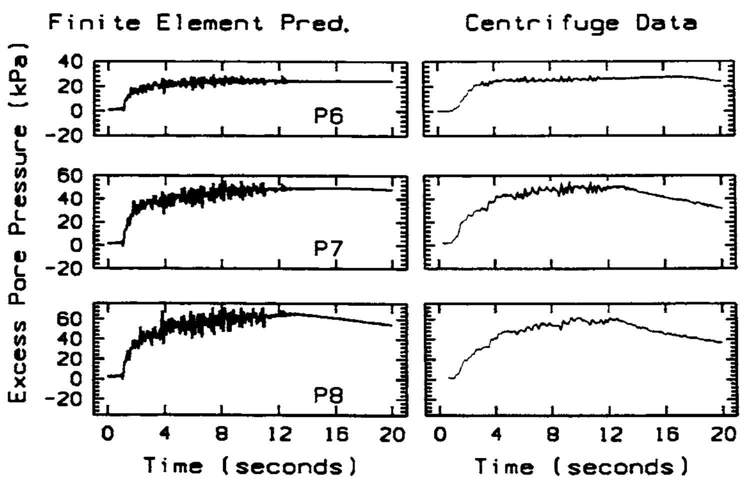

Fig. 5. Short Term Pore Pressure Histories.

plastic model (Anandarajah and Dafalias 1986; Dafalias and Herrmann 1986) with the model parameters listed in Table 3. The problem was analysed under undrained condition using the computer program HOPDYNE (Anandarajah 1990). Fig. 8 shows an example of the type of deformation predicted by the finite element analysis. A series of tests were performed, varying each time the amplitude of base acceleration by scaling up or down the motion shown in Fig. 9. Some of the results for the case with a base acceleration of 0.19g are compared in Fig. 10. Compared in this figure are the time histories of the following quantities: (1) Bending moment (M) on the top beam near the column, (2) horizontal displacement (δ_h) of the superstructure at the level of the top beam, and (3) horizontal acceleration (a_h) of the superstructure at the level of the bottom beam. It may be seen that both the time histories as well as the peak values compare reasonably well.

An important aspect of the earthquake behavior of the superstructure, which cannot be predicted by using a total stress, linear or equivalent linear approaches, concerns the residual displacement after the earthquake. Table 4 compares the predicted and observed permanent horizontal displacements of the superstructure at the level of the top beam for six different tests. The apparent reasonable agreement illustrates the power of

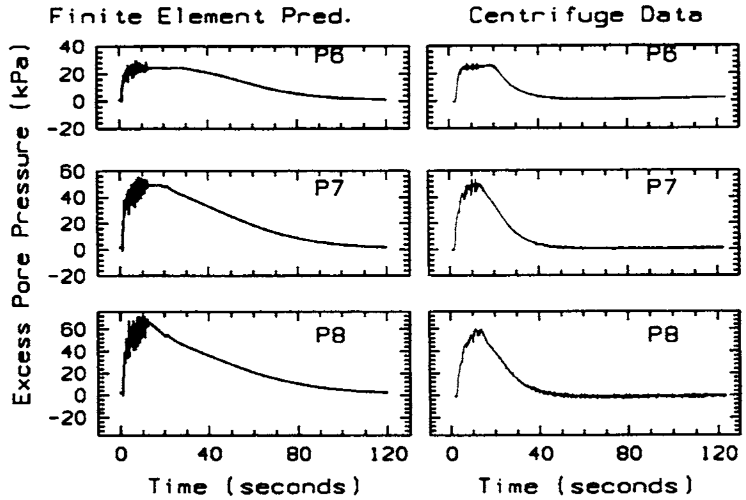

Fig. 6. Long Term Pore Pressure Histories.

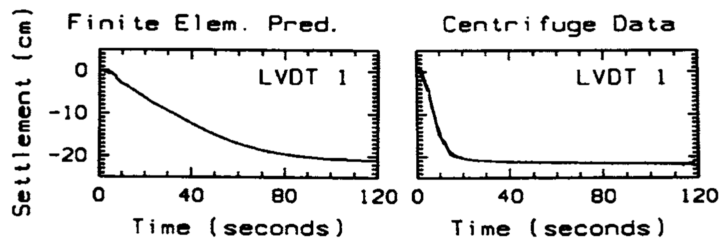

Fig. 7. Ground Settlement History.

Table 3: Model Parameters for Yolo Loam

λ	κ	ν	M_c	R	S_0	α	h
0.167	0.012	0.3	1.4	2.0	10.0	10.0	10.0

truly nonlinear procedures such as the one used here in providing the most important quantities needed for a reliable design of structures founded on soil foundations.

7. CONCLUSIONS

The effective stress based, elasto-plastic numerical procedure described in this paper appears to provide a reasonable means of predicting the liquefaction behavior of saturated sand deposits, and soil-structure interaction behavior during earthquakes. Theoretical predictions are verified by comparing them with centrifuge data. The two problems considered in this paper involve high degree of nonlinearity during earthquake loading, and an effective stress based, truly nonlinear procedure such as the one used in this study only can provide realistic predictions in these cases.

References

[1] Anandarajah, A. (1994a), "Procedures for Elasto-Plastic Liquefaction Modeling of Sands," *Journal of Engineering Mechanics Division*, ASCE, 120, 1563-1593.

[2] Anandarajah, A. (1994b), "A Constitutive Model for Granular Materials Based on Associated Flow Rule," *Soils and Foundations*, Japanese Society of Soil Mechanics and Foundation Engineering, 34, 81-98.

[3] Anandarajah, A. (1993a), "VELACS Project: Elasto-Plastic Finite Element Prediction of the Liquefaction Behavior of Centrifuge Models Nos. 1, 3 and 4a," *Proc. Intl. Conf. on Verification of Numerical Procedures for the Analysis of Soil Liquefaction Problems*, Davis, California, Oct. 17-20, eds. K. Arulanandan and R. F. Scott, 1, 1075-1104.

Table 4: Measured (Test 3-6) and Theoretical Residual Horizontal Displacements of the Superstructure (Note: 1 in = 2.54 cm)

Quantity	Test 3 (.192g) Exp.	Test 3 (.192g) FE	Test 4 (.18g) Exp.	Test 4 (.18g) FE	Test 5 (.227g) Exp.	Test 5 (.227g) FE	Test 6 (.3g) Exp.	Test 6 (.3g) FE
δ_h-Top Beam (in)	0	0.18	0.50	0.12	0.50	0.30	2.00	6.30
δ_h-Bot Beam (in)	0	0.16	0	0.10	NA	0.28	1.50	6.20
δ_h-Pile Cap (in)	0	0.14	1.00	0.08	1.00	0.26	NA	6.10

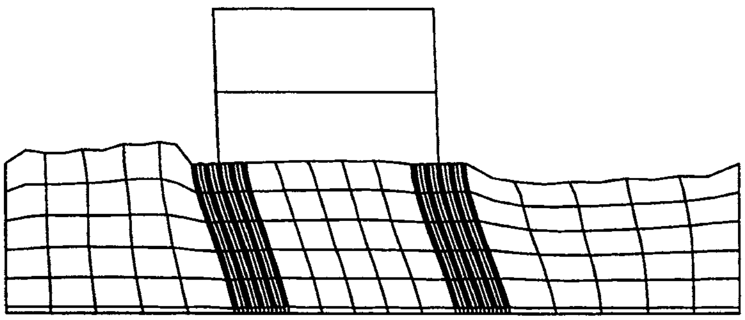

Fig. 8. A Typical Response of the Soil-Structure System at t=6 seconds, a_{base}=0.192g.

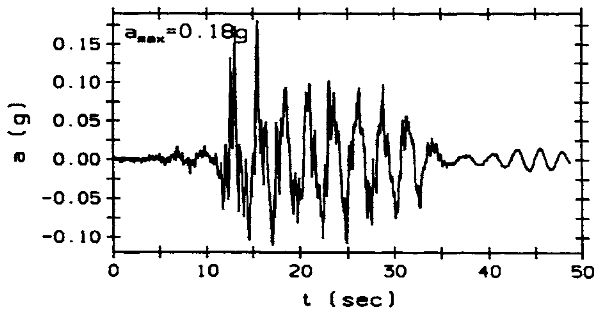

Fig. 9. A Sample Base Acceleration History Used in the Soil-Structure Interaction Centrifuge Tests.

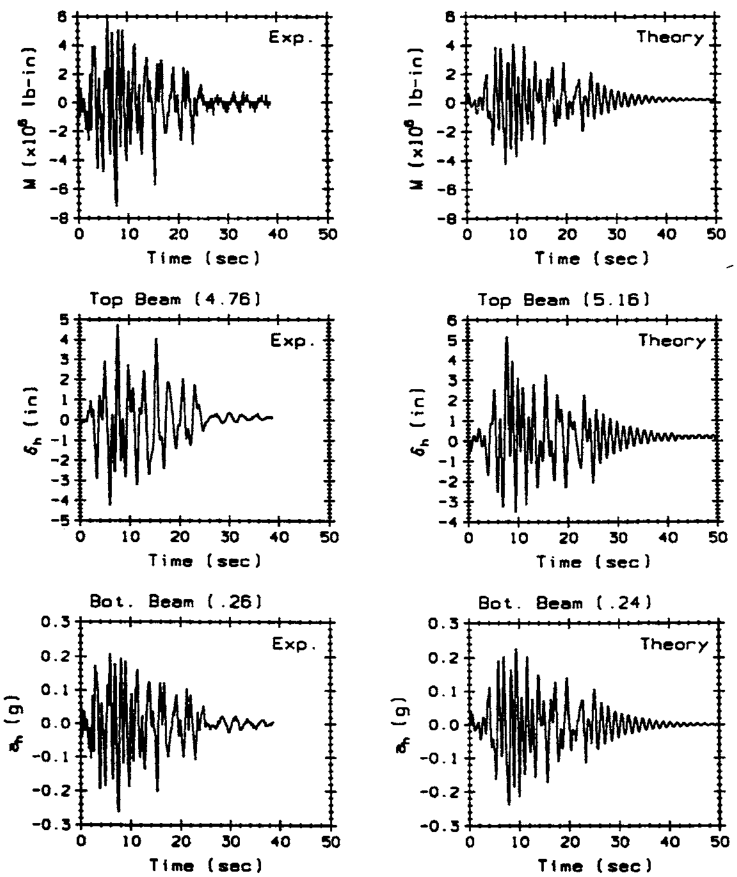

Fig. 10. Theoretical and Experimental Responses of Soil-Structure Systems (a_{base}=0.192g). Note: 1 in = 2.54 cm, 1 lb-in = 0.000109 kN-m.

[4] Anandarajah, A. (1990), "HOPDYNE – A Finite Element Computer Program for the Analysis of Static, Dynamic and Earthquake Soil and Soil-Structure Systems," The Johns Hopkins University, Baltimore, Maryland.

[5] Anandarajah, A. and Dafalias, Y.F. (1986), "Bounding Surface Plasticity, Part 3: Application to Anisotropic Soils," *Journal of Engineering Mechanics Division*, ASCE, 112, 1292-1318.

[6] Anandarajah, A., and Agarwal, D. (1991), "Computer-Aided Calibration of a Soil Plasticity Model," *International Journal for Numerical and Analytical Methods in Geomechanics*, 15, 835-856.

[7] Anandarajah, A., Meegoda, N.J., and K. Arulanandan, K. (1986), "In Situ Stress-Strain Prediction of Fine Grained Soils," *Proceedings of ASCE specialty conference on "Use of In Situ Testing in Geotechnical Engineering,"* held in June 22-25 at Virginia Tech, Blacksburg, Virginia.

[8] Anandarajah, A., Rashidi, H. and Arulanandan, K. (1994), "Elasto-Plastic Finite Element Analyses of Earthquake Pile-Soil-Structure Interaction Problems Tested in a Centrifuge," *Computers and Geotechnics*, in press.

[9] Anandarajah, A. (1990), "Time-Domain Radiation Boundary for Finite Element Analysis of Semi-Infinite Solids", *International Journal of Numerical Methods in Engineering*, 29, 1049-1063.

[10] Anandarajah, A. (1993b), "Dynamic analysis of Axially-Loaded Footings in Time Domain," Journal of *Soils and Foundations*, Japanese Society of Soil Mechanics and Foundation Engineering, 33, 40-54.

[11] Dafalias, Y.F. and Herrmann, L. R. (1986), "Bounding Surface Plasticity, Part 2: Application to Isotropic Soils," *Journal of Engineering Mechanics Division*, ASCE, 112, 1263-1291.

[12] Taboada, V. M. and Dobry, R. (1993), "Experimental Results of Model 1 at RPI," *Proc. Intl. Conf. on Verification of Numerical Procedures for the Analysis of Soil Liquefaction Problems*, Davis, California, Oct. 17-20, eds. K. Arulanandan and R. F. Scott, 1, 3-18.

[13] Zienkiewicz, O. C. and Shiomi, T. (1984), "Dynamic Behavior of Saturated Porous Media: The Generalized Biot Formulation and Its Numerical Solution," *International Journal for Numerical and Analytical Methods in Geomechanics*, 8, 71-96.