NUMERICAL SIMULATION OF FIELD TESTS ON MODEL PILE SEGMENTS

SIMULATION NUMERIQUE DES EXPERIMENTES EN CHAMP DE MODELES DES SEGMENTS DE PILES

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SYNOPSIS: A constitutive model based on the hierarchical single surface approach is developed for cyclic behavior of cohesive soils. It is verified with respect to laboratory triaxial tests, and implemented in a nonlinear finite element procedure based on the Generalized Biot's theory. The numerical procedure is used to simulate insitu, driving, consolidation and cyclic loading for tests on two instrumented model pile segments (7.6cm diam. and 4.4cm diam.). The back predictions, in terms of shear stresses, strains and fluid pressures, provide very good correlations with the field observations.

INTRODUCTION

The traditional approach to solving most geotechnical engineering problems tends toward empirical design techniques. These methods are based on years of experience with particular materials and types of loadings, and work well within those limited conditions. However, these empirical methods may not perform well under general materials and loading conditions encountered in problems such as offshore structures. In contrast, a general approach based on the proper characterization of material properties and the governing equations would be applicable to a wide range of situations.

Here, saturated soil is considered to be a mixture of soil particles and water. Material behavior of the soil skeleton is modelled using the δ_0^* model (Wathugala, 1990) of the Hierarchical Single Surface (HiSS) modelling approach (Desai et al, 1986). The theory of dynamics of porous media, pioneered by Biot (1941), is used to characterize the soil water mixture. The finite element method is used to develop the numerical solution procedure from the governing differential equations. The proposed procedure is verified by back predicting laboratory and field tests.

FIELD TESTING

One of the unique aspects of this study is the integration of field testing and verification with theoretical constitutive modelling and laboratory testing. Field testing was performed by the Earth Technology Corporation at a site near Sabine Pass, Texas (The Earth Technology Corporation, 1986). The field testing program included: (a) collection of 'undisturbed' samples, (b) installation of instrumented pile segments with different diameters and cutting shoes, (c) monitoring consolidation, (d) performing axial tension tests at different levels of consolidation and (e) performing cyclic axial load tests at the end of consolidation. The pile segments were instrumented so as to measure total lateral earth pressure and pore pressures at the pile wall and shear transfer versus pile displacement.

'Undisturbed' Sampling and Laboratory Testing

Over the years, a considerable number of laboratory and field pile load tests, both axial and lateral have been performed on the Sabine Clay. However, no cylindrical triaxial or multiaxial (cubical) tests with pore pressure measurements for the Sabine Clay are reported in the literature. Therefore, a comprehensive series of laboratory tests, involving various, hydrostatic, conventional triaxial compression and triaxial extension stress paths, for the Sabine clay was performed (Katti, 1991), so as to define the required constitutive parameters for the δ_0^* model. 'Undisturbed' samples of 7.6cm diameter cylindrical, and 12.7cm X 12.7cm square were obtained. Samples were carefully transported to the University of Arizona, and the cylindrical and multiaxial tests were performed under different stress paths.

Installation and Load Tests of Pile Segments

Two instrumented model pile segments of diameter 7.6cm and 4.4cm (Bogard et al, 1985) with full displacement cutting shoes were used in this study. Bore holes of 0.152m diameter were driven to the top of the uniform clay layer which is of about 3.0-3.7m thickness and occurs at a depth of about 19.5m. The bore holes were cased using PVC pipes. Each model pile was connected to the end of a N-rod string and lowered into the bore hole until it reached the bottom of the bore hole. Then it was driven into the clay layer. A loading frame was setup on top of the bore hole which could apply slow cyclic or one way loading while pile displacements, shear transfer, pore pressure and total horizontal earth pressure at the pile segment are recorded by the data acquisition system.

Tension tests at different consolidation levels were also carried out to investigate the increase in the pile capacity during consolidation. For both model piles, two-way cyclic load tests were carried out near the end of consolidation.

CONSTITUTIVE MODEL

This model allows for virgin (monotonic) loading (VL), unloading (UL) and reloading (RL); the latter two are also referred to as non-virgin loadings. Brief details of the model are given below.

Virgin Loading

From the theory of plasticity, the incremental stress-strain relationship for virgin loading can be determined from the yield surface (F), the potential surface (Q) and the hardening function. In this section, the proposed yield function and the hardening function are presented. The δ_0^* model uses associative plasticity, and therefore F = Q (Desai et al, 1986).

The yield surface F is defined in terms of stress invariants J_1, the first invariant of the stress tensor, σ_{ij}, J_{2D}, the second invariant of the deviatoric stress tensor, and J_{3D}, the third invariant of the deviatoric stress tensor as

$$F = \left(\frac{J_{2D}}{p_a^2}\right) - \left[-\alpha\left(\frac{J_1}{p_a}\right)^n + \gamma\left(\frac{J_1}{p_a}\right)^2\right]\left[1 - \beta\left(\frac{\sqrt{27}}{2}\frac{J_{3D}}{J_{2D}^{1.5}}\right)\right] = 0 \tag{1}$$

where α is the hardening or growth function. p_a is the atmospheric pressure. β, γ n and m (= -0.5 used) are material parameters.

The hardening function α is defined as a function of the trajectory of deviatoric plastic strains, ξ_D and the trajectory of volumetric plastic strains, ξ_V as

$$\alpha = h_1 / \left(\xi_V + h_3\, \xi_D^{h_4}\right)^{h_2} \tag{2}$$

where h_1, h_2, h_3 and h_4 are material parameters; with h_3=0, the function yields volumetric hardening response suitable for cohesive soils. The increments of ξ_V and ξ_D are defined as

$$d\xi_D = \left(d\epsilon_{ij}^p d\epsilon_{ij}^p\right)^{\frac{1}{2}} \text{ and } d\xi_v = \begin{cases} (1/\sqrt{3})d\epsilon_v^p & \text{for } d\epsilon_v^p > 0 \\ 0 & \text{for } d\epsilon_v^p \le 0 \end{cases} \tag{3}$$

where $d\epsilon_{ij}^p$ $(= d\epsilon_{ij}^p - \delta_{ij} d\epsilon_{ii}^p/3)$ is the incremental deviatoric plastic strain tensor and $d\epsilon_v^p$ is the incremental volumetric plastic strain during to virgin loadings. Now the incremental stress-strain relationship for virgin loading can be derived using the theory of plasticity,

Non-Virgin Loading

During cyclic loading, material experiences non-virgin loadings (unloading and reloading). Over consolidated clays experience non-virgin loading during the initial part of any loading path. Unless inelastic strains developed during these stress paths are modeled, it is not possible to predict proper pore pressure generations during undrained loadings.

The convex reference surface (R), which passes through the current stress point in the stress space as shown in Fig. 1 is used to distinguish unloading from reloading. The shape of R is similar to that of the F surface and is given by

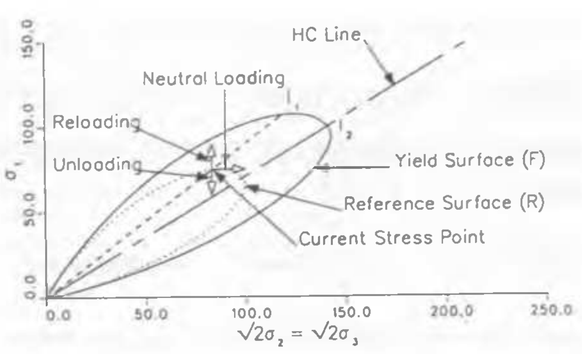

Figure 1 Yield Surface (F) and Reference Surface (R) in Triaxial Plane

$$R = \left(\frac{J_{2D}}{p_a^2}\right) - \left[-\alpha_r\left(\frac{J_1}{p_a}\right)^n + \gamma\left(\frac{J_1}{p_a}\right)^2\right]\left[1 - \beta\left(\frac{\sqrt{27}}{2}\frac{J_{3D}}{J_{2D}^{1.5}}\right)\right] = 0 \tag{4}$$

The value of α_r is obtained by equating R = 0 and substituting current stresses into the Eq. (4). All other symbols have the same meaning as that for the yield surface.

The incremental stress-strain relationship for non-virgin loading is assumed to be similar to that for virgin loadings. However, R surface is used instead of the F surface in calculating the elasto-plastic matrix. Plastic modulus H^{RL}, for reloading is found using an interpolation function. Unloading is assumed to be elastic and this is equivalent to using a very high value for H^{UL}, i.e. $H^{UL} \to \infty$ (Wathugala, 1990).

The following simple interpolation function is used to evaluate reloading plastic modulus, H^{RL}, in this study:

$$H^{RL} = H_{I_1}^{VL} + H_{I_2}^{VL}\, r_1 \left(1 - \alpha / \alpha_r\right)^{r_2} \tag{5}$$

where r_1 and r_2 are material parameters, and $H_{I_1}^{VL}$ and $H_{I_2}^{VL}$ are virgin plastic moduli at points I_1 and I_2 (Fig. 1) on the yield surface. The image point I_1 is located at the intersection between the radial line passing through the current stress and the yield surface. The point I_2 is located at the intersection of the hydrostatic compression line and the yield surface.

LABORATORY VERIFICATION FOR THE MODEL

Material parameters for a marine clay found near Sabine Pass, Texas have been determined from laboratory triaxial tests performed by Katti (1991) on 'undisturbed' samples obtained from the field test site for the instrumented pile segments (Earth Technology Corporation, 1986). They are Young's Modulus, E = 4147 kPa; Poisson's Ratio, ν = 0.42; γ = 0.047; β = 0; m = 0.5; n = 2.4; h_1 = 0.0034; h_2 = 0.78; h_3 = 0; h_4 = na; r_1 = 500; r_2 = 2.4 and permeability = 2.39 x 10^{-10}m/s.

Comparisons

Back predictions using the proposed models are compared with typical observed behavior (Wathugala, 1990). Figures 2(a) and (b) show comparisons for undrained conventional triaxial compression (CTC) test with σ_3 = 110 psi (756 kPa) and OCR = 1 for stress vs strain and pore water pressure vs strain respectively. It can be seen that the model provides good predictions for loading, unloading and reloading behavior of the clay.

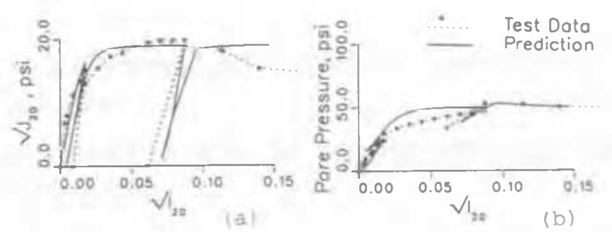

Figure 2 Laboratory Verification for an Undrained Triaxial Test

NUMERICAL SIMULATION OF FIELD TESTS

The constitutive model (δ_0^*) for cohesive soils and the incremental form of the constitutive equations are presented above are introduced in the finite element procedure which is based on the generalized nonlinear form of Biot's equations (Biot, 1941 and Desai and Galagoda, 1989) and details can be found in Wathugala (1990). Robust and reliable algorithms are developed to implement the models into the finite element procedure (Desai et al, 1990).

The field test series was simulated in stages starting from insitu conditions to cyclic loadings. Here, each stage uses the results in terms of stresses, strains, pore water pressures and hardening parameters of the previous stage as the initial conditions.

Initial stresses before pile driving is estimated using the coefficient of earth pressure at rest, K_0 calculated from the Jaky's formula (Jaky, 1944). A self-boring pressuremeter test at the site confirmed the validity of this assumption (Wathugala, 1990); however, as the pressuremeter test was not fully successful due to the occurrence of shell fragments in the top layer, its results are considered to be not complete. Initial hardening parameters are determined by assuming that this stress state is reached through a proportional stress path from zero stress state.

Pile driving is simulated using the strain path method (Baligh, 1985; Wathugala, 1990). Then the subsequent consolidation is simulated by inputing the conditions at the end of pile driving to the finite element program.

Simulation of Final Two-way Cyclic Load Tests

Two-way cyclic load tests were performed with both pile segments. Five loading unloading cycles were performed in both tests. The pile segments were failed before reversing the direction in all the cycles. They are simulated using the finite element procedure and are compared below with field measurements.

Predicted results are compared with field measurements for the X-probe in Fig. 3. The predicted shear transfer vs. pile displacements, Fig. 3a, show good agreement with field measurements. The predicted shear transfer vs. time, Fig. 3b, shows very good agreement with field measurements. Figure 3c compares predicted variation of pore pressures with time with field measurements. Even though all peak magnitudes are not predicted well, their locations indicated by small sized peaks are predicted satisfactorily. Also the accumulation of pore pressures which is indicated by a small increase with time, is predicted well. In practical applications, the accumulation of pore pressures is often more important than the exact shape of the pore pressure variation during a cycle. Total horizontal stresses did not change significantly during the cyclic load test, the predicted values also show similar trend in Fig. 3d. Results for 3-inch probe are similar and are available elsewhere (Wathugala, 1990).

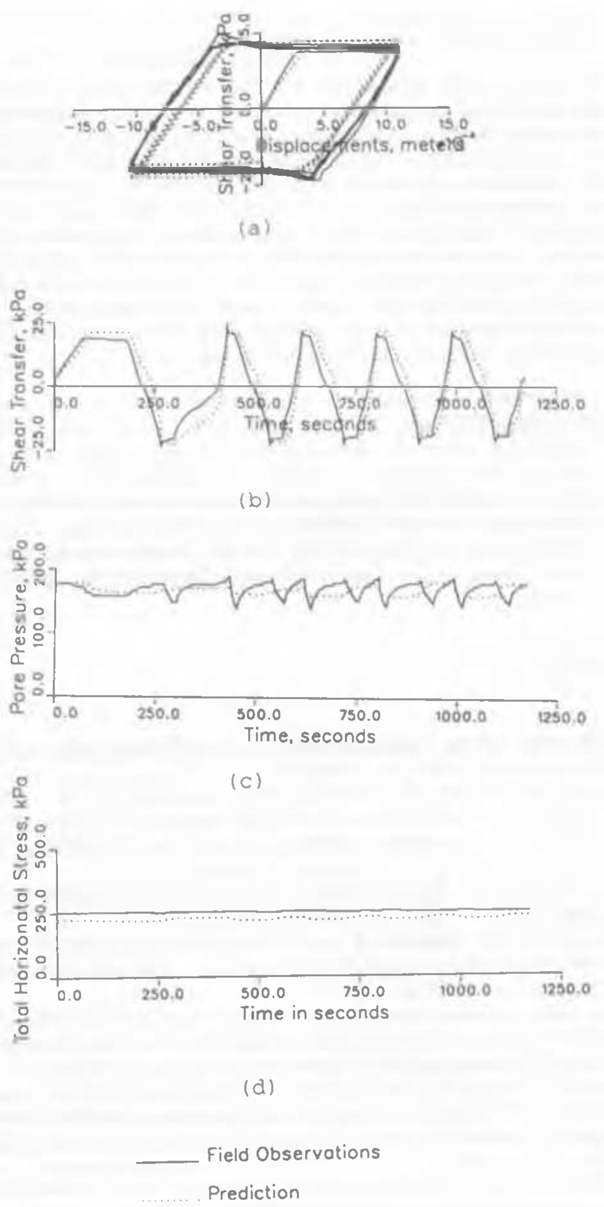

Figure 3 Field Verification for Cyclic Load Test on 4.4cm Pile Segment

SUMMARY AND CONCLUSIONS

Hierarchical Single Surface (HiSS) model for clay was developed and calibrated using laboratory triaxial tests on undisturbed samples of Sabine clay. Then the model was used to simulate the complete field behavior of two instrumented model pile segments starting from insitu stresses to the final cyclic loading. The pile driving was simulated using strain path method and the consolidation, tension tests and cyclic loading at the end of consolidation were simulated using a general dynamic nonlinear finite element procedure based on the theory of dynamics of porous media.

Shear transfer vs pile displacements and shear transfer vs time for the pile segments were predicted well by the procedure. Even though predicted pore pressures were found to be lower than the observed ones during individual cycles, the accumuliation of pore pressure for the entire test was predicted well. Accumulation of pore pressure during cyclic loading is important since it can contribute to the reduction of strength of piles during cyclic loading. Pile capacity can be evaluated from the shear transfer values calculated here. Therefore, it can be concluded that the proposed procedure shows good promise of improved and detailed predictions for cyclic behavior of axially loaded piles in saturated clays. Since a general finite element procedure is used here, it is applicable not only to pile problems but also to a wide range of geotechnical engineering problems with saturated soils.

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REFERENCES

Baligh, M.M. (1985), "Strain Path Method," J. of Geotech. Engg., ASCE, Vol. 111, No. 7, Sept., pp. 1108-1196.

Bogard, J.D., Matlock, H., Audibert, J.M.E., and Bamford, S.R. (1985), "Three Years Experience with Model Pile Segment Tool Tests," Proc. 17th Offshore Technology Conference, Houston, Texas, OTC4848, pp 65-73.

Biot, M.A., (1941), "General Theory of Three-Dimensional Consolidation," J. App. Physics, Vol. 12, pp. 155-164, February, 1941.

Desai, C.S., and Galagoda, H.M., (1989), "Earthquake Analysis with Generalized Plasticity Model for Saturated Soils," J. of Earthquake Engg. Struc. Dyn., Vol. 18, pp. 903-919.

Desai, C.S., Somasundaram, S., and Frantziskonis, G. N., (1986), "A Hierarchical Approach for Constitutive Modelling of Geologic Materials," Int. J. Num. Analyt. Meth. Geomechanics, Vol. 10, pp. 225-257.

Desai, C.S., Wathugala, G.W., Sharma, K.G. and Woo, L. (1990), "Factors Affecting Reliability of Computer Solutions with Hierarchical Single Surface Constitutive Models," Comp. Meths. in App. Mech. and Engg., Vol. 82, pp. 115-137.

Earth Technology Corporation (1986), "Pile Segment Tests - Sabine Pass; Some Aspects of the Fundamental Behavior of Axially Loaded Piles in Clay Soils," ETC Report No. 85-007, December.

Jaky, J. (1944), "The Coefficient of Earth Pressure at Rest," J. of Hungarian Architects and Engineers, Budapest, Hungary, Oct., pp. 355-358.

Katti, D.R. (1991), "Constitutive Modelling and Testing of Saturated Marine Clay," Ph.D. Dissertation, Dept. of Civil Engg. and Engg. Mechs., U. of Arizona, Tucson.

Wathugala, G.W. (1990), "Finite Element Dynamic Analysis of Nonlinear Porous Media with Applications to Piles in Saturated Clays," Ph.D. Dissertation, Dept. of Civil and Eng. Mech., U. of Arizona, Tucson, 475 pages.