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SYNOPSIS: This paper analyses the behaviour of an experimental unsupported excavation taken to failure in a soft clay deposit near Rio de Janeiro, Brazil. The excavation was originally analyzed by Pontes Filho and Medeiros (1982) assuming undrained conditions. In this paper, the same excavation is analyzed by Biot's coupled theory of consolidation and deformation using linear elastic, non-linear-elastic and elasto-plastic constitutive models and simulating the excavation process in time, without a-priori hypothesis on the drainage conditions. Details of the excavation construction, geotechnical profile and instrumentation are briefly described. Subsequently, the constitutive model calibrations are discussed in view of laboratory tests available. Finally, for each excavation step, comparisons are made between surface settlement profiles, horizontal displacements and pore-pressure measured in the field and analytically calculated.

INTRODUCTION

As part of a research project developed at PUC/RJ, a two-dimensions (2-D) computer program using the finite element method and capable of simulating the construction of excavations as well as embankments, was written. The program uses a coupled deformation-pore pressure dissipation theory(Biot, 1941; Sandhu and Wilson, 1969). It is based on a program developed by Osaimi (1977), the main differences being the implementation of a new version of the hyperbolic stress-strain constitutive model (Duncan, 1980) and the implementation of the Modified Cam-Clay elasto-plastic work-hardening constitutive model (Zienkiewicz and Naylor, 1971).

Parallel to this development, a few years ago (Ribeiro, 1981), an experimental excavation(Sarapui) was executed as part of another research project undertaken simultaneously by the Institute of Highway Research of Rio de Janeiro (IPR) and the Departments of Civil Engineering of PUC/RJ and of the Post-Graduation School of Federal University of Rio de Janeiro (COPPE/UFRJ). Besides the experimental excavation, the research project also included two trial embankments: one taken to failure to investigate strength properties (Ramalho Ortigão et al., 1983) and another one for settlement studies (Figure 1).

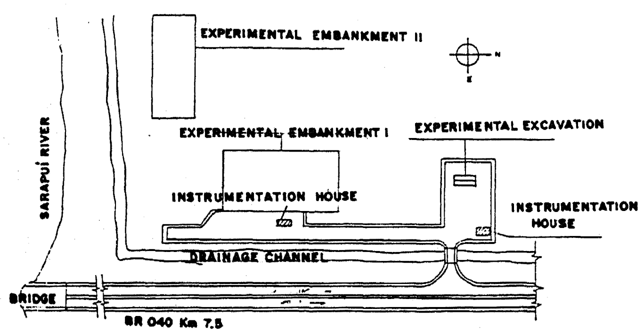


Fig. 1 - Sarapui Experimental Excavation (Pontes Filho and Medeiros, 1982)

The Sarapui experimental was taken to failure in few hours. A first analysis, made with a 2-D uncoupled finite element program with linear-elastic and non-linear-elastic constitutive models, assumed undrained conditions and utilized undrained laboratory tests and soil parameters (Pontes Filho and Medeiros, 1982).

This paper describes the utilization of the coupled finite element program mentioned above in a second analysis of Sarapui experimental excavation. In this second analysis, drained laboratory tests are used to determine soil parameters. The time spent to execute the excavation is taken into consideration, therefore avoiding, a priori, any hypothesis with respect to the drainage conditions of the problem.

Although there are few doubts about these drainage conditions, this second analysis aims, first, to test the coupled model's ability to predict partially drained behaviour and, second, to discuss two different undrained analyses: one made uncoupled with undrained parameters and, another one, made coupled with drained parameters.

Initially, the case history is briefly described. This part was essentially extracted from the work of Pontes Filho and Medeiros (1982) and is included here only to insure completeness. Subsequently, laboratory test results are presented, together with their analytical modeling. Finally, comparisons between field and analytical results are presented.

THE SARAPUI UNSUPPORTED EXPERIMENTAL EXCAVATIONS

The experimental excavation was performed on 4 steps, with failure occuring approximately 5 hours and 45 minutes after the construction beginning (Figure 2).

The instrumentation consisted of 44 surface monuments, 15 piezometers (7 of the Casagrande type and 8 of the hydraulic type), 2 slope indicators and 4 magnetical extensometers (Figure 3).

The soil profile consisted of three layers (Figure 4): a 1 to 1.4 meters thick sandy embankment dumped in the area three years before the excavation, a 3.2 to 3.8 meters thick organic silty clay layer (Sarapui clay) and 10 meters thick stiff sandy clay layer. The sandy embankment consisted of a well-graded sand-gravel mixture excavated from a nearby pit of young gneissic residual soil with average specific unit weight of 2.0 tf/m³ and moisture content between 9% and 11%. Sarapui soft clay has been investigated by many authors: Costa Filho et al. (1977); Werneck et al. (1977); Ramalho Ortigão and Costa Filho (1982). At excavation site, average liquid limit of 98% and plastic limit of 49% were encountered. Natural moisture content, void ratio, specific unit weight and degree of saturation were equal to 141%, 3.9, 1.33 tf/m³ and 97%, respectively. The organic matter content was about 5% and is responsible for the dark gray color of Sarapui clay (Ramalho Ortigão et al., 1983).

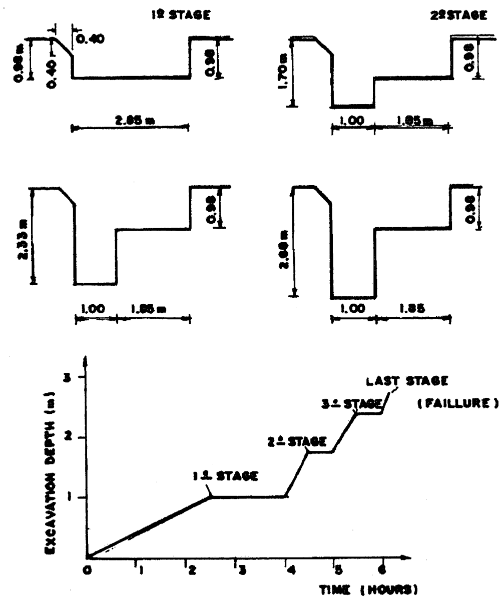
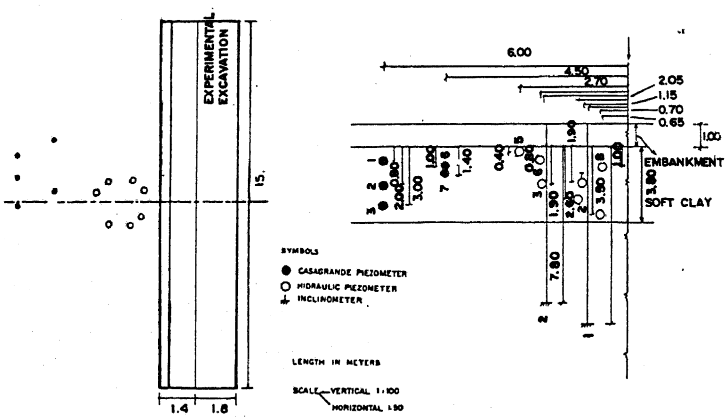


Fig. 2 - Excavation Sequence and Duration (Ribeiro, 1981)



(a) Plan View (b) Transversal View

Fig. 3 - Excavation Instrumentation

An investigation carried out by Ribeiro (1981) indicated a moderate degree of overconsolidation in the soft clay deposit. However, the sandy embankment constructed three years before the excavation initiated a consolidation process which, based on piezometer readings, was completed before the excavation beginning. Therefore, the deposit to be excavated was normally consolidated (Pontes Filho and Medeiros, 1982).

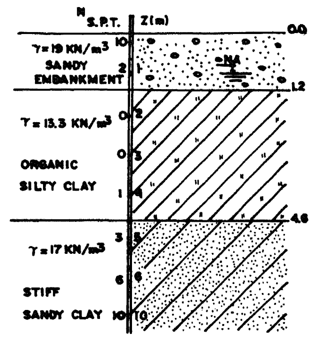


Fig. 4 - Soil Profile and Excavation Site

LABORATORY TESTINGS AND CONSTITUTIVE MODELS

The sandy embankment stress-strain and strength behaviour was investigated by Pontes Filho (1981) who performed two series of drained axisymmetric triaxial tests using a stress-controlled cell (Bishop and Wesley, 1975). The first series consisted of three conventional triaxial compression tests (CTC - increasing axial stress and constant confining pressures of 10, 20 and 30 tf/m²). The second series consisted of three reduced triaxial compression tests (RTC - constant axial stress of 20, 30 and 40 tf/m² and decreasing confining pressure). Only the CTC were used for calibrating the constitutive models.

The Sarapui soft clay stress-strain and strength behaviour was investigated by a series of undrained and drained axisymmetric triaxial tests. The undrained tests were performed by Sayão (1980). A total of 45 tests was executed: 24 rapid tests (UU or CU without pore-pressure measurements) and 21 slow tests (CU with pore-pressure measurements). The drained tests were performed by Bressani (1983). A total of 11 tests was executed, always with volume change measurements: 5 tests were strain-controlled and 6 were stress-controlled.

Only the drained tests were utilized for the hyperbolic calibration of Sarapui soft clay. For the Modified Cam-Clay elasto-plastic model, besides the drained and undrained tests, oedometer tests were also necessary for calibration (Sayão, 1980).

TABLE I

	E (kPa)	ν
SAND	5100.	0.30
CLAY	625.	0.17

TABLE II

	K	n	Rf	c'	ϕ'	Kb	m
SAND	403.7	0.785	0.71	0	34	—	—
CLAY	9.36	0.36	0.606	0	24	3.33	0.678

TABLE III

	λ	k	M	eo	ν
CLAY	0.73	0.11	1.25	3.90	0.17

COMPARISONS BETWEEN FIELD AND ANALYTICAL PERFORMANCE

Three different finite element analyses were made. In the first one, herein called elastic, both sandy embankment and Sarapui clay were modeled with a linear-elastic model using parameters shown in Table I. In the second analysis, herein called hyperbolic, both sandy embankment and Sarapui clay were modeled with the hyperbolic model using parameters presented in Table II. In the third analysis, herein called Cam-Clay, the sandy embankment was modeled with the hyperbolic model using parameters shown in Table II and the Sarapui clay was modeled with the Modified Cam-Clay elasto-plastic model using parameters presented in Table III.

Figure 5 shows comparisons of surface settlement profiles due to excavation stages: (a) 1, (b) 2 and (c) 3, respectively. The analytical results do not show a good quantitative agreement with the field ones. Results obtained with the hyperbolic and the Cam-Clay analyses are reasonably close and only qualitatively in agreement with the field ones. Field settlement are in general much smaller than the analytical ones. A probable reason for this difference may be found in the analytical simulation of the sand embankment. Due to low stress levels from the beginning of excavation some elements in the embankment fail or become too flexible in the analysis, therefore exaggerating the analytical vertical displacements.

Figure 6 and 7 present comparisons of horizontal displacements due to excavation along vertical lines corresponding to inclinometers I1 and I2, respectively. In this case, results obtained with the hyperbolic and Cam-Clay analyses agree reasonable well with the field ones, except the results from the inclinometer I1, stage 3. The elastic analysis was clearly the worst ones, whereas the Cam-Clay analysis was a little better than the hyperbolic one. It is important to remember that the volumetric response of the sand embankment was estimated, therefore features like dilatancy, that may explain certain observed differences in the horizontal movements, were not either evaluated in the laboratory or modeled.

Figure 8 and 9 present comparisons between pore-pressures analytically calculated and measured in the field by Casagrande and hyperbolic piezometers, respectively. In general, results of the three analyses do not differ much, the largest differences being encountered between the elastic and the other two analyses. The analytical results are in agreement with the field ones specially before failure (time equal to 15:00 hours). After this point some of the piezometer registered an abrupt change in pore water pressure that could not be acounted for in the analysis.

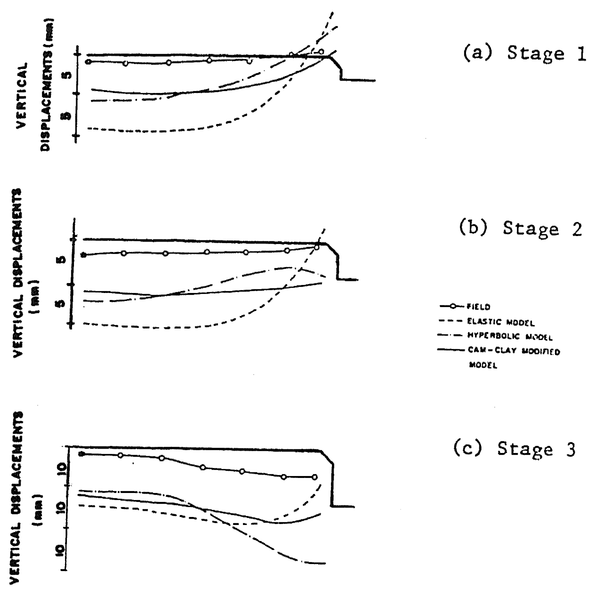


Fig. 5 - Surface Settlement Profile Comparisons

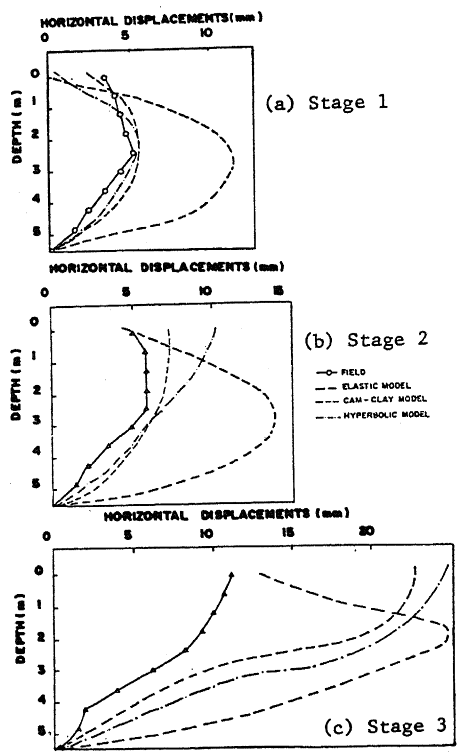


Fig. 6 - Horizontal Comparisons for Inclinometer I1

executed. This is only true if the stress-paths in the laboratory tests and in the field are exactly the same which is seldom the case. On the other hand, the pore-pressures generated by the coupled theory are in accordance with the equations governing the behaviour of a two-phase material (Biot, 1941). The main assumptions in this case are, laminar flow and infinitessimal strains during consolidation, and modeling of the effective stress-strain relation observed in drained tests.

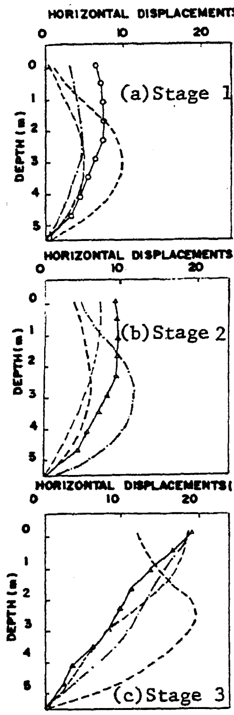


Fig. 7 - Horizontal Displacement Comparisons for Inclinometer I2

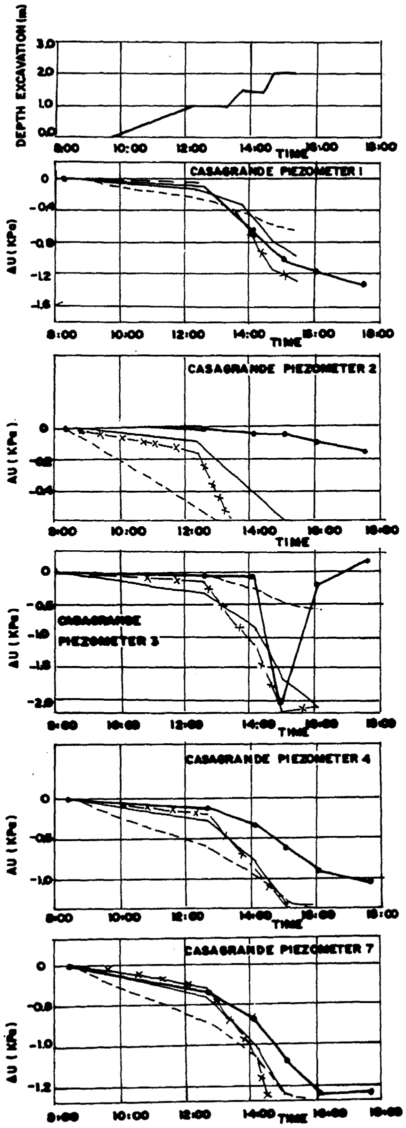


Fig. 8 - Pore-Pressure Comparisons for Casagrande Piezometers

CONCLUSIONS

With the exception of the surface settlement profiles, comparisons were, in general, both qualitative and quantitatively good particularly for the hyperbolic and Cam-Clay analyses.

With respect to the pore-pressure comparisons, the three analyses gave rise to close results, therefore not justifying the utilization of more sophisticated models as the hyperbolic and Cam--Clay. However, the displacement comparisons were clearly more favorable for the hyperbolic and Cam-Clay analyses than for the linear elastic one, therefore entirely recommending the utilization of these analyses instead of the linear-elastic one.

The Cam-Clay analysis gave rise to results that were sligthly better than the hyperbolic ones. Nevertheless, the differences were small and, for the case analyzed, there was no major gain in accuracy that justyfied the use of this more sophisticated elasto-plastic model instead of the simpler and widely known hyperbolic model.

Comparisons between results obtained by Pontes and Medeiros (1982) with their uncoupled-undrained analysis with results presented in this paper show small differences justifiable by different values of effective stress in the two analyses. In fact, both analyses induce same total stresses, since they depend on the geometry and equilibrium conditions of the problem. The effective stresses, however, may be different, depending on the pore-pressures. In the analysis made by Pontes and Medeiros (1982) it is assumed, first, that the excavation was executed in undrained conditions and, second, that the pore-pressures generated during the laboratory undrained tests are equal to the field ones while the excavation is being

Since results of the two analyses are reasonably close, it is logical to say that they gave rise to values of pore-pressure approximately the same. However, the coupled approach is more attractive because it does not involve an assumption about the drainage conditions in which the excavation is executed which, for pratical rather than experimental excavations, is frequently difficult to determine (Osaimi and Clough, 1979).

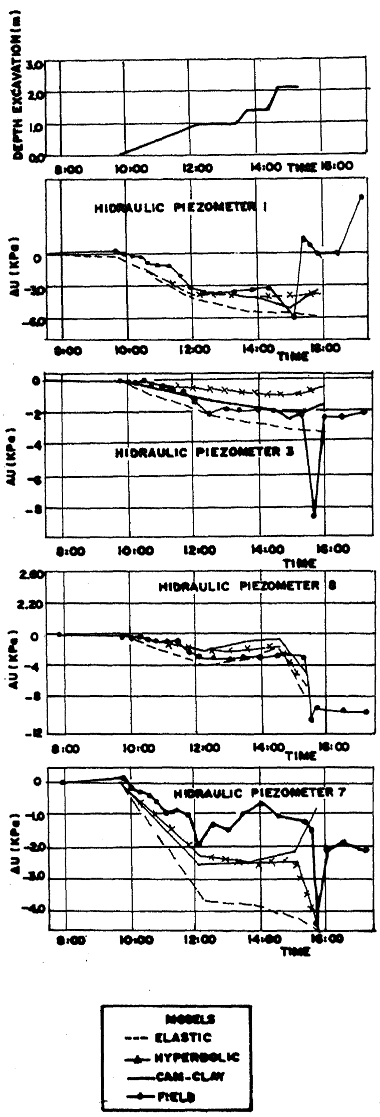


Fig. 9 - Pore-Pressure Comparisons for Hydraulic Piezometers

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REFERENCES

Biot, M.A. (1941), General Theory of Three--Dimensional Consolidation, Journal of Applied Physics, 12, p. 155-164.

Costa Filho, L.M.; Werneck, M.L.G. and Collet, H. B. (1977), The Undrained Strength of a Very Soft Clay, Proceedings, 9th International Conference On Soil Mechanics And Foundation Engineering, Tokyo, 1, p. 69-72.

Duncan, J.M. (1980), Hyperbolic Stress - Strain Relationships, Proceedings, Workshop on Limit Equilibrium, Plasticity and Generalized Stress -Strain in Geotechnical Engineering, Edited by R.K. Yong and H.Y. Ko, McGill University, Canada, p. 443-460.

Osaimi, A.E. (1977), Finite Element Analysis of Time Dependent Deformations and Pore Pressures in Excavations and Embankments, Ph.D. Thesis, Stanford University, California, USA, p. 230 .

Osaimi, A.E. and Clough, G.W. (1979), Pore-Pressure Dissipation During Excavation, Journal of the Geotechnical Division, ASCE, 105, GT.4, p. 481-498.

Pontes Filho, I. (1981), Analysis of Sarapui Experimental Excavation (in Portuguese), M.Sc. Thesis, Pontifícia Universidade Católica do Rio de Janeiro, Rio de Janeiro, Brazil, p. 246.

Pontes Filho, I. and Medeiros, L.V., Finite Element Analysis of an Experimental Excavation, Proceedings, Fourth International Conference on Numerical Methods in Geomechanics, Edmonton, Canada, p. 699-705.

Ramalho Ortigão, J.A. and Costa Filho, L.M. (1982), Discussion of Cam-Clay Predictions of Undrained Strength, by Paul W. Mayne, Journal of the Geotechnical Engineering Division, ASCE, 108, GT. 1, p. 181-183.

Ramalho Ortigão, J.A.; Werneck, M.L.G. and Lacerda, W.A. (1983), Embankment Failure On Clay Near Rio de Janeiro, Journal of the Geotechnical Engineering Division, ASCE, 109, GT. 11, p. 1460-1479.

Ribeiro, J.V. (1981), Planning, Execution, Observation and Preliminar Analysis of an Unretaining Instrumented Excavation (in Portuguese), M.Sc. Thesis, Pontifícia Universidade Católica do Rio de Janeiro, Rio de Janeiro, Brazil, p. 228.

Sandhu, R.S. and Wilson, E.L. (1969), Finite Element Analysis of Flow in Saturated Porous Elastic Media, Journal of Engineering Mechanics, ASCE, 95, p. 641-652.

Sayão, A.S. (1980), Laboratory Testings on the Soft Clay of Sarapui Experimental Excavation (in Portuguese), M.Sc. Thesis, Pontifícia Universidade Católica do Rio de Janeiro, Brazil, p. 201.

Werneck, M.L.G.; Costa Filho, L.M. and França, H. (1977), In Situ Permeability and Hydraulic Tests in Guanabara Bay Clay, Proceedings, Soft Clay Conference, Bangkok, p. 399-418.

Zienkiewicz, O.C. and Naylor, D.J. (1971). The Adaptation of Critical State Soil Mechanics Theory for Use in Finite Elements, Proceedings, Roscoe Memorial Symposium, p. 537-547.