Time-dependent deformations in clay soils

Comportement dans le temps des sols argileux

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SYNOPSIS The time-dependent behavior of compressible soils, including both hydro-dynamic lag and creep effects, is an important problem in geotechnical practice. Time-dependent behavior can be an important design consideration, particularly with respect to earthen embankments, which impose relatively large foundation loads and often traverse deep deposits of highly compressible soils requiring staged construction procedures. Previous numerical models of time-dependent soil behavior under other than one-dimensional conditions have considered either hydro-dynamic lag or undrained creep separately. In this paper, a recently developed numerical model that accounts for both hydro-dynamic lag and creep effects is introduced. This model is used to analyze the behavior of the I-95 test embankment, outside of Boston, Massachusetts, USA. Results of the analysis indicate that undrained deviatoric creep can be a major contributor to embankment foundation deformations and that an immediate/delayed consolidation model may yield the same results as a primary/secondary consolidation model if the volumetric creep rate is slow enough.

INTRODUCTION

In 1967, construction of a highway embankment for a proposed extension of the I-95 interstate highway began across a marsh underlain by a deep deposit of soft clay north of the city of Boston. As part of the construction process, a test section was instrumented by the geotechnical group at M.I.T. and centerline settlements and pore pressures within the clay beneath the embankment were monitored. In 1973, after plans for the highway were abandoned, nine different groups of geotechnical engineers made predictions of the deformation and pore pressure response of the foundation soil due to an additional six feet of fill placed on top of the embankment at a symposium sponsored by M.I.T. (1975). Most of these predictions were made using undrained analyses, with parameters derived from laboratory test data and adjusted to account for the deformations that occurred between initial embankment construction in 1967 and the second phase of construction, for the M.I.T. symposium, in 1973. At that time, neither hydro-dynamic lag nor creep effects could be accounted for in two dimensional consolidation analyses. Duncan (1975) performed a one-dimensional consolidation analysis of centerline settlements and, noting that the one-dimensional analysis seriously underestimated consolidation settlements, concluded that creep deformations played an important role in determining the magnitude of centerline settlements. Tavenas, Mieussens, and Bourges (1979) drew a similar conclusion based upon the analysis of 21 case histories of embankment foundation deformation.

Since 1973 significant advances have been made in the ability of numerical analyses to consider time-dependent aspects of soil behavior. A variety of numerical models now exist to consider either creep or hydro-dynamic lag. Despite these advances, the authors know of no model that considers the combined effects of creep and hydro-dynamic lag on the consolidation behavior of compressible soils.

In this paper, a numerical model for the combined effects of creep and hydro-dynamic lag on clay soil behavior is introduced and used to analyze the behavior of the I-95 test section. The constitutive model used in the analysis is an extension of the modified Cam-clay model (Roscoe and Burland, 1968), and thus is subject to the same limitations as the Cam-clay model: it is applicable to normally consolidated to lightly overconsolidated saturated "wet"clays of low to intermediate sensitivity.

Based upon the work of Kavazanjian, Bonaparte and Mitchell (1984), the elliptical Cam-clay yield surface is used to describe the direction of both the time-independent and the time-dependent incremental plastic strains. The magnitude of the creep strains can be scaled using either C_α, the coefficient of secondary compression (C_α scaling) or the Singh and Mitchell (1968) creep function (SM scaling).

An immediate/delayed compression model, as suggested by Bjerrum (1967), is used to describe consolidation deformations. Results using both creep scaling laws are compared to an analysis in which creep strains are suppressed and to the observed field behavior of the I-95 test section.

Results indicate that time-dependent deviatoric strains are a major component of the total centerline settlement, and that unless C_α is very large or the rate of pore pressure dissipation very slow, the results from an immediate/delayed compression model may be identical to those from a conventional primary/secondary model.

CONSTITUTIVE EQUATIONS

It is assumed in the constitutive model that the deformation behavior of "wet" clays can be separated into a time-independent immediate component and a time-dependent delayed component, as described by Bjerrum (1967). The time-independent model is an elasto-plastic strain-hardening model whose yield surface, F, is the ellipsoid of the modified Cam clay model:

$$F = F(\sigma'_{ij}, p'_c) = \frac{q^2}{M^2} + p'(p' - p'_c) = 0 \qquad (1)$$

where σ'_{ij} is the effective stress tensor, q and p' are the deviatoric and volumetric stress state variables (defined subsequently), M is the slope of the critical state line in a (q, p') space, and p'_c is the preconsolidation pressure. This yield surface also serves as the plastic potential surface by the associative flow rule.

A three-dimensional generalization from the Cam-clay model is made by defining the stress and strain state parameters with invariants to encompass all the components of the stress and strain tensors σ'_{ij} and ε_{kl}. The effective volumetric stress, p', is defined as

$$p' = \sigma'_{oct} = \frac{1}{3} I'_1 \quad (2)$$

where σ'_{oct} is the effective octahedral normal stress and I'_1 is the first principal effective stress invariant.

The deviatoric stress, q, is defined as

$$q = \frac{3}{\sqrt{2}} \tau_{oct} = \sqrt{3J_2} \quad (3)$$

where τ_{oct} is the octahedral shear stress and J_2 is the second deviatoric stress invariant.

The volumetric strain, ε_v, is defined as

$$\varepsilon_v = 3\varepsilon_{oct} = I_{\varepsilon 1} \quad (4)$$

where ε_{oct} is the octahedral normal strain and $I_{\varepsilon 1}$ is the first principal strain invariant.

The deviatoric strain is defined as

$$\gamma = \sqrt{2}\, \gamma_{oct} = [\frac{4}{3} J_{\varepsilon 2}]^{1/2} \quad (5)$$

where γ_{oct} is the octahedral shear strain and $J_{\varepsilon 2}$ is the second deviatoric strain invariant. Note that these definitions reduce to the conventional definitions of q, p', and ε_A for undrained triaxial stress states, facilitating evaluation of the model parameters from the results of standard laboratory tests.

Although the validity of the above generalization is yet to be verified in three-dimensional applications, it has been shown (Roscoe and Burland, 1968; Bonaparte, Mitchell, and Kavazanjian, 1984) that this scheme accurately predicts the stress-strain behavior of "wet" clays not only in the "triaxial" (axisymmetric) compression case but under plane strain conditions as well.

The elastic initial tangent modulus and the trace of the Cam-clay model on the q-γ plane are used to represent time-independent deviatoric behavior. The rules for loading, unloading and reloading along this trace follow those described by Kavazanjian, Bonaparte and Mitchell. For most soft clays, this approach will overestimate the deviatoric stiffness (underestimate deviatoric strains). Only by accounting for plastic deviatoric strains beneath the yield surface can this Cam-clay type model accurately model the immediate deviatoric stiffness of a typical soft clay. Unfortunately, this results in a non-associative flow rule and a non-symmetric stiffness matrix, greatly increasing computational difficulty.

Time-dependency is incorporated by allowing creep strains and quasi-preconsolidation to develop. The effect of quasi-preconsolidation is to make the size p'_c of the yield surface grow with time. The time-dependent rate of change of p'_c with respect to time, $\dot{p}'_c$, is given by the equation:

$$\dot{p}'_c = \frac{\psi p'_c}{(\lambda - \kappa) t_v} \quad (6)$$

where ψ is the coefficient of secondary compression on a ln(t) scale ($\psi = 0.43\, C_\alpha$), λ and κ are the slopes of the virgin and recompression lines respectively on the e - ln(p') plot, and t_v is the volumetric age of the soil.

To obtain the direction of the creep strain rate tensor the normality rule is employed on the equivalent yield surface associated with the current stress state (p',q) if the soil were normally consolidated. To completely define creep strains, the tensor normal to the yield surface is "scaled" either by prescribing the creep strain component along the volumetric axis (volumetric or C_α scaling) or along the deviatoric axis (deviatoric or SM scaling). The prescribed creep strain components are evaluated using "age variables" t_v and t_d.

If C_α scaling is used, the delayed strain rate tensor, $\dot{e}^t_{kl}$, is given by:

$$\dot{\varepsilon}^t_{kl} = [\frac{\psi}{(1 + e)(2p' - p'_c)t_v}] \frac{\partial F}{\partial \sigma_{kl}} \quad (7)$$

where e is the void ratio.

If SM scaling is used, the deviatoric component of the creep strain rate is given by

$$\dot{\varepsilon}^t_{kl} = \frac{3}{2} A\exp(\bar{\alpha}\, \bar{D}) t_d^{-m} [\frac{\partial F}{\partial \sigma_{ij}} \frac{\partial F}{\partial \sigma_{ij}} - \frac{1}{3}(\frac{\partial F}{\partial p'})^2]^{-1/2} \frac{\partial F}{\partial \sigma_{kl}} \quad (8)$$

where A, $\bar{\alpha}$, and m are the Singh-Mitchell parameters evaluated from triaxial compression tests and $\bar{D}$ is the deviatoric stress level, defined as

$$\bar{D} = \frac{q}{q_{ULT}} \quad (9)$$

The deviatoric stress at failure, q_{ULT}, is uniquely related to the void ratio, e:

$$q_{ULT} = \frac{M p'_c}{2^{(1-\kappa/\lambda)}} \quad (10)$$

The volumetric age t_v is evaluated implicitly by measuring the e- distance of the state point (e, p',q) from the state boundary surface, and the deviatoric age t_d is similarly obtained by measuring the γ-distance of the state point (q, γ) from the trace of the Cam-clay model on the 1-α plane.

A complete description of the constitutive equations and their development is presented by Borja (1984).

NUMERICAL MODEL

The constitutive equations were coded into a finite element consolidation program SPIN2D by Borja (1984). This program is capable of performing drained, undrained, and consolidation analyses using the above constitutive model. The program also has the ability to model linear elastic and hyperbolic stress-strain behavior. A coupled deformation-consolidation scheme proposed by Carter, Booker, and Small (1979) is used to model pore pressure dissipation. All required soil parameters are readily determined from the results of standard triaxial and one-dimensional laboratory tests.

Borja has used the model to perform parametric studies of the behavior of San Francisco Bay mud. Soil properties taken from the literature were used to predict the behavior of Bay mud in drained and undrained plane strain and triaxial tests and for creep and stress relaxation conditions. Comparison of the predictive behavior with actual test results showed excellent agreement, the only shortcoming being that the model predicted a stiffer deviatoric response then the observed behavior for most of the tests.

CASE HISTORY

The soil profile at the I-95 site is shown in Figure 1. The value of the initial overconsolidation ratio within the Boston Blue Clay (BBC) is taken from the data package provided the symposium predictors. The value of the Cam-clay parameters used to model BBC behavior are the same values used by Wroth to make his symposium prediction (M.I.T., 1975). The value for C_α was taken from the results of Ladd and Preston (1965). The Singh-Mitchell creep parameters were evaluated from the results of two creep tests performed by Duncan for the symposium. The clay soil was represented in the finite element mesh by quadrilateral elements with nine displacement nodes and four pore pressure nodes (Q9P4 elements).

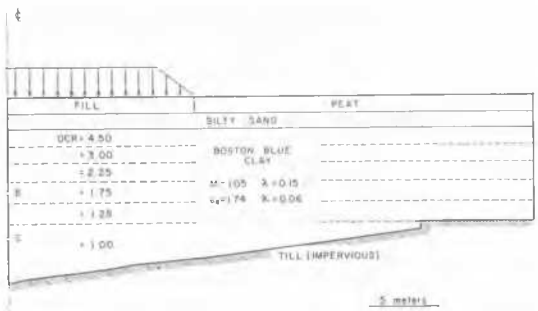

Fig. 1. Soil Profile at the I-95 Embankment Site

The silty sand and embankment fill beneath the ground surface were modeled as linear elastic fully drained materials using the average bulk and shear moduli used by Duncan in his symposium predictions. This soil was represented with nine noded quadrilateral elements (Q9P0) in the finite element mesh. The peat at the ground surface was modeled as a drained Cam-clay material using parameters for a soft organic clay of similar water content. As the peat beneath the embankment had been excavated and replaced with silty sand fill, the material properties of the peat were not considered critical to results of the analysis.

Embankment construction was simulated by applying vertical loads to the nodes at the ground surface. The time history of loading for the embankment was based upon information derived from the symposium data package. Predictions were made of centerline settlement at the ground surface and of center line pore pressures at points B and C in Figure 1 using a 126 element, 553 node mesh. Three sets of predictions were made; one each for C_α and SM scaling, and one suppressing all delayed deformations. Figures 2 and 3 compare the results of these predictions to the field behavior.

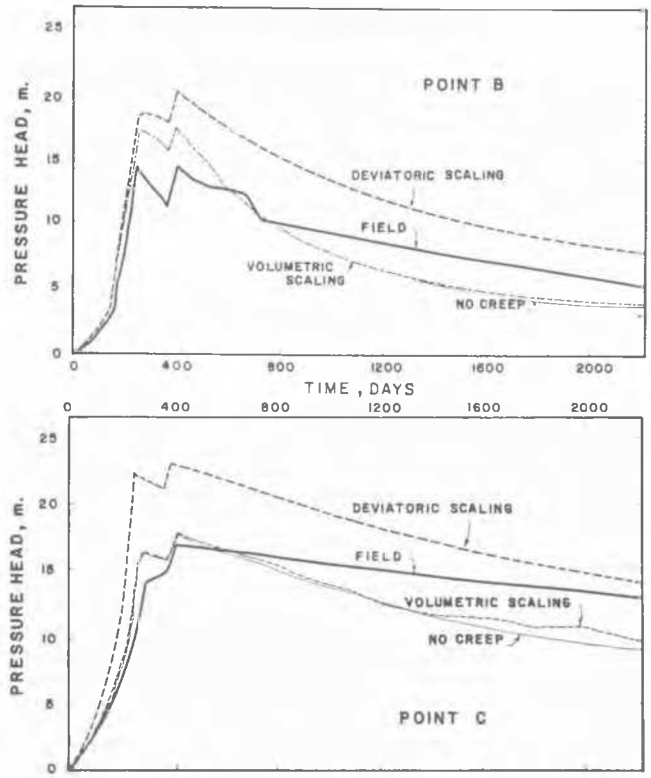

Fig 2. Pore Pressure Head at Points B and C

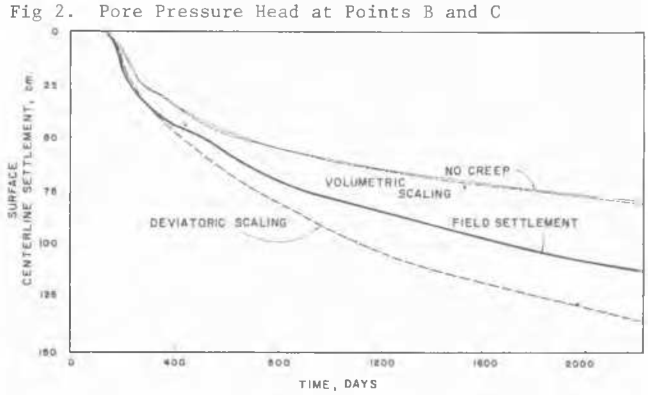

Fig. 3. Surface Centerline Settlement for the I-95 Embankment

ANALYSIS OF RESULTS

The following observations can be made based upon the observed and predicted behavior shown in Figures 2 and 3. Creep can play a major role in the magnitude of consolidation settlements of compressible foundations. The analysis which did not account for creep effects under-predicted the magnitude of observed consolidation settlements by a steadily increasing margin, resulting in an error of more than 1 ft., or over 40%, in a period of 6 years.

Since C_α scaling had little effect on consolidation settlements, the increased centerline surface settlement predicted in the SM scaling analysis can clearly be attributed to the increased lateral spreading induced by the deviatoric creep component rather than by time-dependent volumetric strains. This can be seen from the displacement vectors shown in Figure 4 for the end of the consolidation period from the no creep and SM scaling analyses. These results are in substantial agreement with the conclusions of Tavenas et al. (1979).

The over-estimate of surface settlement by the SM analysis at longer times could be due to an overestimate of the creep-induced lateral deformations, or it could be that the over-estimation of initial excess pore pressure development shown in Figure 3 accelerated the rate of consolidation. An accelerated consolidation rate will not only increase the rate of volume change, but may also increase the rate at which the overconsolidated foundation soils become normally consolidated, accelerating lateral creep displacements, as discussed by Tavenas et al. (1979). The overprediction of the initial pore pressure response by the SM analysis may be due to the overestimation of immediate deviatoric stiffness, discussed previously, inherent in the Cam-clay model.

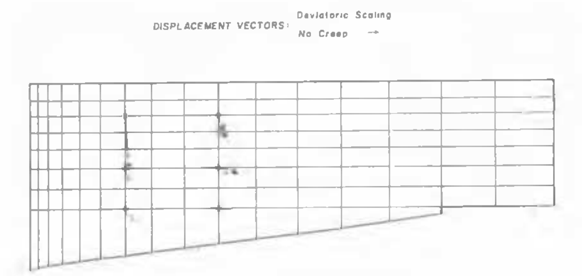

Fig. 4. Finite Element Mesh and Displacement Vectors for the I-95 Embankment

The result that C_α scaling had little influence on consolidation behavior was surprising. It was anticipated that the immediate/delayed compression model used herein would yield a greater rate of consolidation deformation than a conventional primary/secondary compression model, yet the inclusion of volumetric creep had little effect on these analyses. This is probably because the rate of delayed, C_α-induced strains is slower than the immediate strain rate due to pore pressure dissipation. This result is significant because it indicates that unless the ratio ψ/λ is large or the rate of pore pressure dissipation very small an immediate/delayed compression model will give the same results as a primary/secondary model. It is worth mentioning here that in addition to being preferable to the authors from a theoretical point of view, the immediate/delayed model is computationally simpler than a primary/secondary model because the age variables t_v and t_d can be evaluated implicitly.

It should be noted that once pore pressures dissipate, SM scaling will result in erroneous predictions of the rate of secondary compression. On the critical state line the volumetric commpression is undefined using SM scaling because the incremental strain rate tensor is vertical. For elements near the critical state line, SM scaling underpredicts delayed volumetric strains and because the Singh-Mitchell equation generally underpredicts creep deformations for $\overline{D} > 0.9$. For isotropic compression, SM scaling overpredicts delayed volumetric strains.

Analysis by Borja (1984) has shown that even in the range where the Singh-Mitchell equation best describes soil behavior ($0.2 < \overline{D} < 0.8$), secondary compression is underpredicted by SM scaling. Thus it would appear that a model containing both C_α and SM components is required to accurately describe time-dependent behavior over both the primary and secondary compression range. Such a model would be expected to give essentially the same results as the SM scaling analysis performed herein until pore pressures dissipated and the influence of C_α became important. Unfortunately, such a model would be non-associative.

CONCLUSION

An elasto-plastic finite element model for the combined effects of creep and hydro-dynamic lag on the consolidation behavior of soft clays has been developed. Two options for scaling creep strains, one using the coefficient of secondary compression and one using the Singh-Mitchell creep equation, are available. The model has been used to analyze a documented case history of the deformation of a compressible embankment foundation. Results of the analyses show that

1) Time dependent deformations play a major role in determining the magnitude of the consolidation settlement of compressible embankment foundations.
2) Lateral spreading due to time-dependent deviatoric deformations is the major contributor to the component of centerline settlement attributable to time-dependent effects.
3) Unless the rate of secondary compression is large compared to the virgin compressibility or the rate of pore pressure dissipation is very slow, an immediate/delayed consolidation model will yield essentially the same results as a conventional primary/secondary model.
4) Singh-Mitchell (deviatoric) scaling can account for lateral displacements during consolidation, but a non-associative model including both Singh-Mitchell and C_α creep components and accounting for plastic deviatoric strains beneath the Cam-clay yield surface is required to provide an accurate representation of compressible soil behavior in the secondary compression range.

ACKNOWLEDGEMENT

The work described in this project was part of a continuing National Science Foundation funded research program into "Development of a Numerical Method for Predicting Time-Dependent Behavior of Geotechnical Structures on Cohesive Soils" at Stanford University.

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