A Quasi-Preconsolidation Clay Model

Modèle d'Argile Soumise à Quasi-Préconsolidation

J.L.DAVIDSON Ass. Prof., Civil Engineering Department, University of Florida, U.S.A.

SYNOPSIS In order to predict pore pressures and deformations during primary consolidation of a clay soil which exhibits a quasi-preconsolidation effect, a constitutive model reflecting this behavior is needed. Such a model is formulated from drained triaxial testing of soil specimens in which a quasi-preconsolidation effect has been induced by maintaining the specimens under a constant hydrostatic stress for a period of time. From the test results a three-dimensional effective stress 'yield' surface, representing the rupture of the quasi-preconsolidation bonding, is developed. The model has been incorporated into a finite element program to yield solutions to realistic boundary value problems.

INTRODUCTION

When loading of a clay specimen is continued after a period of time during which the soil has been subject to a constant effective stress, it has been found that compression does not occur along the virgin curve until a substantial load increment had been added. The stress at which a sharp increase in slope occurs (Figure 1) has been termed the quasi-preconsolidation pressure $\bar{\sigma}_{cq}$ (Leonards and Altschaeffl, 1964). The magnitude of $\bar{\sigma}_{cq}$ has been found, for a wide variety of different soils, to be approximately 1.4 times the effective stress acting during the rest period. An appreciation of this phenomenon, found in most clays (Bjerrum,1972), may be decisive in selecting a satisfactory foundation. Research on the quasi-preconsolidation effect (Leonards and Ramiah 1960, Leonards and Altschaeffl 1964, Narain et.al. 1969) has been carried out almost exclusively in the oedometer apparatus under conditions of one-dimensional loading and drainage. In 1972 Bjerrum introduced a one-dimensional procedure for predicting settlements in soft clay taking this effect into account. Since in many instances in the field, conditions are not one-dimensional, a need existed for an investigation of the quasi-preconsolidation phenomenon and its effects in a more generalized form. The objective of this research was to experimentally define a simple constitutive model for soil having a quasi-preconsolidation behavior, which might then be incorporated into a finite element solution procedure to investigate deformations, stresses and pore pressures under realistic, field, boundary and loading conditions. This paper reports primarily on the formulation of such a model. A future publication will present in detail the analytical study.

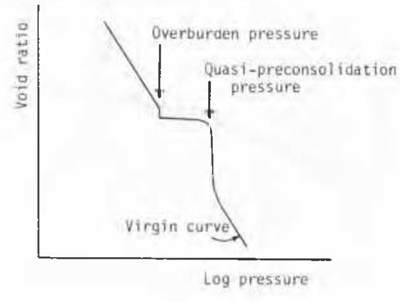

Fig. 1 The quasi-preconsolidation effect

LABORATORY TESTS

In order to define a soil model which would reflect the characteristics of quasi-preconsolidation behavior in a general manner, laboratory triaxial testing was performed on a commercially available soil, Grundite. The clay, primarily Illite, has the following properties:-

$$w_l = 56\% \quad w_p = 32\% \quad \%\ \text{clay} = 68 \quad G = 2.79$$

The clay samples to be ultimately tested under triaxial conditions were initially prepared from a slurry, by a one-dimensional sedimentation process. Loads of 0.8 kg/sq. cm. were applied over a three week period in a loading frame designed by Altschaeffl (1960). The resulting 9 cm. diameter specimens were approximately 10 cm. long, with a unit weight of 1.8 gm/cc. and a void ratio of 1.57. Test specimens of standard dimension were cut from the sedimented soil, placed in triaxial cells and stressed isotropically for a period of time (Figure 2) under a constant cell water pressure, termed the q-consolidation pressure, q_c.

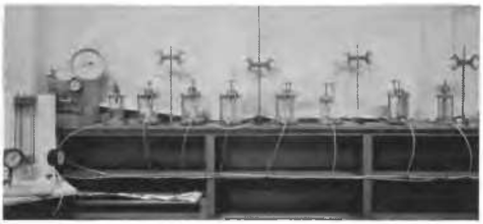

Fig. 2 Set-up for q-consolidation period

During this time a quasi-preconsolidation pressure effect was generated. This procedure eliminates the disturbance due to stress release and sample preparation experienced when triaxial specimens were trimmed from blocks of soil in which the quasi-preconsolidation effect was generated in a consolidometer type apparatus.
At the end of the q-consolidation period, drained triaxial tests along four different types of stress path (Figure 3) were carried out.

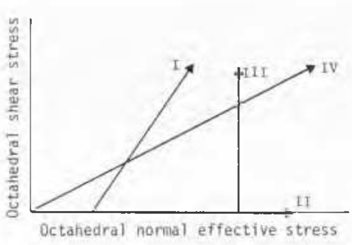

Fig. 3 Triaxial test stress paths

Type I tests were standard constant strain rate, constant confining pressures, triaxial tests. Type II were isotropic loading tests in which the confining pressure was increased incrementally. Type III were incremental loading tests in which the confining pressure was decreased and the axial pressure increased so that a constant octahedral normal stress was maintained.

Type IV were incremental loading tests in which the axial and confining stresses were altered so as to maintain a constant octahedral shear stress - normal stress ratio.
Details of the sample preparation and test procedure are given in Davidson (1973). Stress and strain parameters used, as recommended by Roscoe and Burland (1968), were the octahedral normal effective stress $\bar{\sigma}_{oct}$, the octahedral shear stress τ_{oct}, the compressive volumetric strain increment δv, and the deviatoric (shear distorsional) strain increment $\delta\varepsilon$.

TEST RESULTS

Twenty-seven triaxial test specimens were prepared and were isotropically loaded under three different confining pressures, q_c = 1.17, 1.69, and 2.11 kg/cm^2 for periods of approximately twenty-three days. Drained triaxial tests of the types described were then performed. Stress-strain plots for all tests indicated an initial steep curve, the soil exhibiting low compressibility, then a fairly abrupt change to a more shallow, more compressible slope. All plots have the appearance of two straight lines with a transition curve, until large strains when failure occurred. Figure 4 illustrates the results from one test, typical of all.

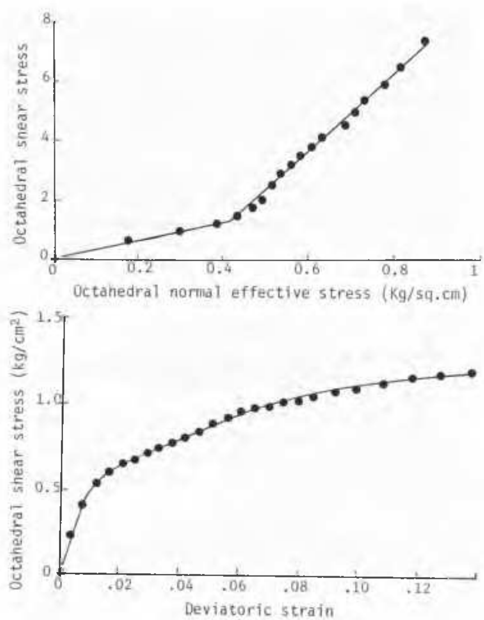

Fig. 4 Typical stress-strain curves

The intersection of the two straight line portions of the curve was termed the q-stress point, from which $\Delta\bar{\sigma}_{oct\ q}$ and $\Delta\tau_{oct\ q}$ were calculated. This represents the effective stress state at which the temporary bonding which had developed during the consolidation period was assumed to rupture. Bond breaking probably occurred over some range of stresses, explaining the transition curve. Conlon (1966), observing similar shaped curves for cemented marine and esturine clays, explained the shape as the sum of two effects, the normal shearing resistance and the interparticle bonding strength. This is shown schematically in Figure 5.

Results from the twenty-seven triaxial tests are given in Table I.

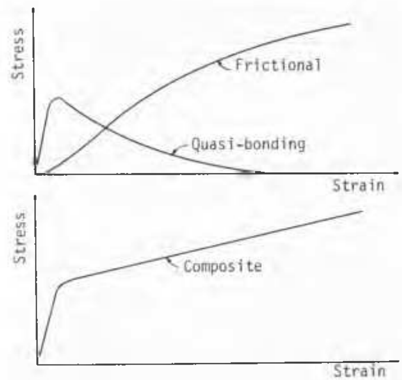

Fig. 5 Shape of quasi stress-strain curves

Test Number	q-Consolidation Pressure (kg/cm^2)	Octahedral Normal Stress At Yield (kg/cm^2)	Octahedral Shear Stress At Yield (kg/cm^2)
1	1.17	0.193	0.273
2	1.17	0.197	0.278
3	1.17	0.176	0.249
4	1.17	0.429	0.000
5	1.17	0.204	0.288
6	1.17	0.457	0.000
7	1.17	0.443	0.000
8	1.17	0.192	0.271
9	1.17	0.283	0.252
10	1.17	0.366	0.219
11	1.69	0.577	0.219
12	1.69	0.633	0.000
13	1.69	0.264	0.373
14	1.69	0.000	0.362
15	1.69	0.105	0.380
16	1.69	-0.337	0.084
17	1.69	0.314	0.345
18	1.69	0.422	0.323
19	2.11	0.368	0.520
20	2.11	0.625	0.387
21	2.11	0.328	0.463
22	2.11	0.809	0.000
23	2.11	0.359	0.506
24	2.11	0.809	0.149
25	2.11	0.057	0.478
26	2.11	0.202	0.506
27	2.11	0.794	0.199

Table I Results from triaxial tests

The yield octahedral stress increments $\Delta\bar{\sigma}_{oct\ q}$ and $\Delta\tau_{oct\ q}$ are plotted, along with stress paths, in Figures 6, 7 and 8; each figure corresponding to a specific q-consolidation pressure.

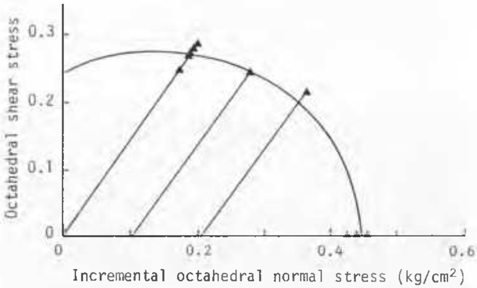

Fig. 6 The 1.17 kg/cm^2 q-yield curve

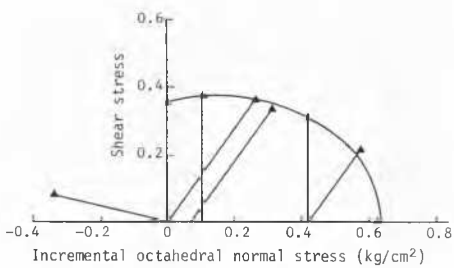

Fig. 7 The 1.69 kg/cm^2 q-yield curve

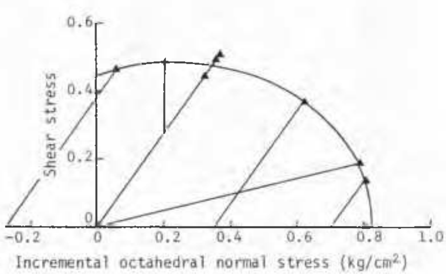

Fig. 8 The 2.11 kg/cm^2 q-yield curve

Best fit polynomials for the three sets of quasi data points were obtained by a step-wise regression program and yielded ellipses, displaced along the abscissa. Combining these curves, a three-dimensional effective stress yield surface was defined (shown schematically in Figure 9) with an equation of the form

$$y^2 = a\,q^2 + b\,q\,x + c\,x^2$$

where

q - q-consolidation pressure
x - q-octahedral normal stress increment
y - q-octahedral shear stress increment
a,b,c - constants

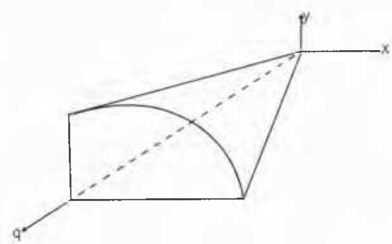

Fig. 9 Effective stress yield surface

Figure 10 from a paper by Hoeg et. al. (1969) on the behavior of a circular test embankment, illustrates the variations in undrained excess pore pressures at different locations with surface pressure. The slope change in each plot indicates yielding of the soil adjacent to that piezometer. Stress paths and an approximate insitu yield curve, of shape similar to those found in the laboratory study, were backfigured from this data and are shown in Figure 11.

A simple bi-linear constitutive behavior law can now be formulated. For low stress levels, falling within the defined yield surface, the soil modulus is high. However, if the stresses cross the yield surface, due to additional loading or increased effective stresses during consolidation, rupture of the temporary quasi bonding occurs and a lower modulus must be assigned. Such a model has been incorporated into a finite element solution procedure yielding results in general agreement with reported field observations.

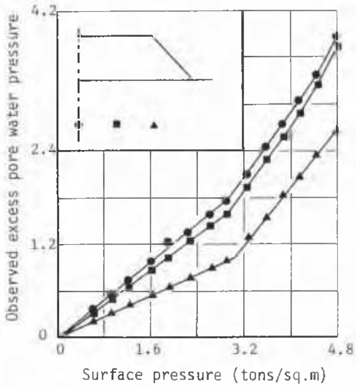

Fig. 10 Observed excess pore pressures (Hoeg et. al. 1969)

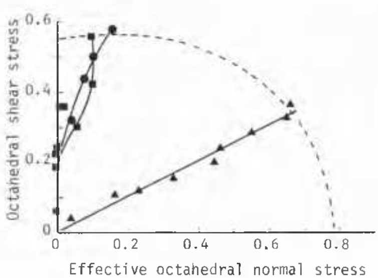

Fig. 11 Field yield curve

CONCLUSIONS

1 A quasi-preconsolidation effect can be induced in a clay specimen by consolidating it in a triaxial cell under a hydrostatic confining pressure.

2 Drained triaxial tests performed on such specimens along different stress paths, show essentially similar stress-strain behavior.

3 From the triaxial test results a soil model may be formulated with the following features:
   a. the stress-strain behavior can be represented by two straight lines intersecting at a point, termed the q-stress point.
   b. for specimens prepared under the same confinement pressure, the q-stress points define a curve which can be approximated by an ellipse
   c. these ellipses form cross-sections of an elliptic cone with apex at the zero confinement origin

4 The soil model derived from the laboratory test results for a clay soil which exhibits a quasi-preconsolidation effect, can be introduced into the finite element procedure.

5 Results from the finite element study indicate that effective stresses from construction loads which do not exceed the insitu effective yield surface result in small settlements and rapid dissipation of excess pore pressures. If yielding does occur, due to loading or consolidation, then the corresponding settlements are large and excess pore pressure dissipation slow. These are in agreement with field observations reported in the literature.

REFERENCES

Bjerrum,L.(1972), "Embankments on Soft Ground," Proc. A.S.C.E. Specialty Conference on 'Performance of Earth and Earth-Supported Structures', Vol.2, pp. 1-54.

Conlon,R.J.(1966), "Landslide on the Toulnustouc River, Quebec," Canadian Geotechnique, Vol.15,No.2, pp. 161-173.

Davidson,J.L.(1973),"The Effect of Quasi-Preconsolidation on Compression of Clay Soils," Ph.D. Thesis, Purdue University, Lafayette, Indiana.

Hoeg,K., O.B. Andersland, and E.N.Rolfsen, (1969), "Undrained Behavior of Quick Clay under Load Test at Asrum," Geotechnique, Vol. 19, No.1, pp. 101-115.

Leonards,G.A. and A.G.Altschaeffl,(1964), "Compressibility of Clay,"Proc. A.S.C.E., Vol.90,No.SM5, pp.133-155

Leonards,G.A. and B.K. Ramiah,(1960), "Time Effects in the Consolidation of Clays," A.S.T.M. Special Technical Publication, No. 254, pp.116-130.

Narain,J. et.al.,(1969),"Quasi-Preconsolidation Effects and Pore Pressure Dissipation during Consolidation," Proc. 7th Int. Conf. S.M.F.E., Vol. 1, pp.311-315.

Roscoe,K.H. and J.B.Burland,(1968), "On the Generalized Stress-Strain Behavior of 'Wet' Clay," from Engineering Plasticity, Cambridge University Press, pp. 535-609.