Summary and Evaluation of Symposium Papers

E. C. W. A. GEUZE, Rensselaer Polytechnic Institute

COMPARISON OF FIELD AND LABORATORY MEASUREMENTS OF MODULUS OF DEFORMATION OF A CLAY

•AUTHORS Hanna and Adams performed triaxial tests to obtain the modulus of deformation and the shear strength of the clay. These tests were either stress controlled or rate of strain controlled, consolidated or unconsolidated, with loading intervals as short as 1 min or as long as 24 hr. The authors attach considerable importance to the ratio of the modulus of deformation and the undrained shear strength of the soil. The modulus of deformation is taken as the secant of the stress-strain curve between the strain values corresponding to the deviator stresses of about 0.2 and 0.5 of the undrained strength, both at loading and unloading.

The authors do not give reasons for their choice of these stress levels. Since the deformation modulus of clay soils has a unique significance only in the range of time-independent strains, the choice of the upper stress level seems questionable. In fact, the presence of a creep or viscous component is clearly demonstrated in the test series shown in Figure 4 of the paper, where controlled stress increments were maintained over 1 min and over 24 hr. In the short interval tests, the viscous component affected the deformations at loading to a much smaller extent than in the long interval tests. As a consequence a much smaller spread in E/C ratios and a greater similarity between consolidated and unconsolidated behavior appears. The limit of the stress range yielding mostly time-independent strains represents a strength of the soil that has no relationship with the conventional shear strength obtained from constant rate of strain or stress controlled triaxial tests where indefinitely increasing strains at constant stress deviators cause failure of the particle structure.

Interesting results were obtained with samples preconsolidated at 20 psi over periods of 2, 13, and 40 weeks. As a result of the compaction, the undrained shear strength increased from an unknown value to 1440 psf, 1780 psf, and 1810 psf. The moduli of deformation were obtained at loading and unloading intervals of 1 min. If the results can be considered to have been obtained from identical samples, the undrained strength increased to 0.5, 0.618, and 0.629 times the consolidation pressure. The results in the diagrams of Figure 5 are shown in terms of the ratios between the stress differences at the peak values of the major principal stress and the principal stress differences at failure. From the similarity between the 13-weeks and 40-weeks tests it can be concluded that the deformation moduli at the various stress levels are proportional to the principal stress differences at failure after the samples had been subjected to extended consolidation periods.

It is also of interest to note that flow affected the strength of the unconsolidated material to a greater extent than the preconsolidated material. Whereas the dots in the diagram of Figure 10B are interspersed with the circles in the band of points, they appear to be shifted to the left in Figure 10D.

The results obtained by the authors raise several questions. It does not follow from these results that the E/C ratio would present any advantage over the use of the modulus of deformation itself, particularly not when the interpretation of the in situ measurements can only yield the values of the moduli. For reasons mentioned before, correlations between the modulus and undrained strength cannot be expected because the choice

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of the maximum principal stress difference is too arbitrary. In clay soils the contributions of the elastic and of the viscous deformations do not depend on the limiting strength. Building materials other than clay soils may behave more consistently in terms of E/C ratios, because the structural bonds are less affected by short-term viscosity effects. The choice of a deformation modulus in cohesive soils therefore would necessarily be an arbitrary one.

A second point of controversial nature is the anisotropy of the soil. The authors suggest that in the upper portion of the clay medium a near-passive in situ state of stress could exist. Under these conditions the undrained, constant-volume method of triaxial testing will yield the value of the deformation modulus in axial direction only. The interpretation of the plate load tests will yield an intermediate value of the moduli, if a theory based on isotropic material properties is used. The interpretation of the field data cannot successfully be achieved if the anisotropic properties of the material are taken into account, and as a consequence certain discrepancies between these results and the laboratory and in situ test data can be expected.

FIELD AND LABORATORY STUDIES OF MODULUS OF ELASTICITY OF A CLAY TILL

Authors Soderman, Milligan, and Kim have performed field and laboratory studies of the properties of a clay till much along the same lines as the previous investigators. The reload modulus of deformation obtained from repeated cyclic loading between zero load and one-third of the expected maximum principal stress difference may have been affected by time-dependent strains to a smaller degree than those obtained from cyclic loading to one-half and to three-quarters of the expected maximum. The results shown in Figure 7 of the paper suggest little change in the reversible portions of the strains after four load cycles. The results shown in Figure 6 of the paper supply additional evidence of the negligible effect of the strain rate on the deformation modulus. A decrease of the deformation modulus with increasing principal stress ratio was found to occur regardless of the test type and mode of sampling. The authors attribute this decrease in modulus to a breakdown of the soil structure at increasing magnitudes of the shearing strains. This statement is of sufficient importance to cause a reconsideration of the value of the deformation modulus as a basis for predicting the deformations of a soil mass resulting from the application of loads. The elastic component of a clay soil under undrained conditions at increasing values of the stress deviator (1) shows a substantial decrease beyond the yield value of the shearing stress. The reason for this behavior is the occurrence of irreversible (viscous) strains increasing with time and with the magnitude of the shearing stress. It is therefore not possible to predict the magnitude of the deformations at values of the stress deviator exceeding the yield value of the clay without consideration of the irreversible, time-dependent strains. The actual breakdown of the soil structure takes place in the upper range of values of the stress deviator.

The plate tests yielded interesting information concerning the irreversible portions of the settlement at increasing plate loads (Fig. 3). Unfortunately the time effects are not shown in this diagram. The pore water pressure results suggest that at the load represented by the principal stress difference of about 17 psi the shearing stress at the point of measurement exceeded the yield value of the clay. It is doubtful, however, that the magnitude of the axial strain and of the principal stress difference in the diagram can be obtained by the simple application of the Boussinesq equation mentioned in the report, because of reasons discussed in the previous paper.

The authors are to be commended for the extensive research performed on the effects of sample size, modes of sampling, and types of testing on the test results. When the plate test results in Figure 13 are left out of consideration the comparison of results obtained with samples of different size may point not only to effects of sampling disturbance but also to the rate of equalization of pore pressures in the triaxial tests due to a nonuniform strain distribution in the samples. As could be expected, the consolidated undrained tests yielded higher values of the modulus than the unconsolidated undrained tests. Obviously, a comparison between these results is of little significance since they represent two different states of the material.

The authors did not find an acceptable correlation between modulus and undrained strength of the material. In view of the discussion on this point in the previous paper, this conclusion is noted without further comment.

COMPARISON OF LABORATORY AND FIELD VALUES OF c_v FOR BOSTON BLUE CLAY

Authors Bromwell and Lambe have studied the vertical displacements and the pore pressure changes in Boston blue clay due to two separate effects: the removal of a sand and gravel overburden to a depth of 14.5 ft, and the lowering of the water table in the sand and gravel deposit by about 12 ft.

During the first stage of the excavation about 7 ft were removed in 9 days. Four days later the well point system was put into operation and three days later another 7.5 ft were removed over a period of about 9 days. During the 38-day interval between the completion of the excavation and the pouring of concrete the water table was lowered from about 12 ft to about 13.5 ft.

The pore pressures in piezometers 2, 3, and 4, located in the lower two-thirds of the clay layer (Fig. 5) dropped by identical amounts during the first stage of the excavation. In their interpretation of the field data, however, the authors assumed that no dissipation of pore pressures had occured at the end of the second excavation period at the locations of piezometers 3 and 4. The initial excess pore pressure vs depth curve is based on this assumption. A shift of this curve in either direction obviously affects the value of the time factor in the early phase of pore pressure dissipation and, thereby, the magnitude of the coefficient of swelling c_{vs}.

When the effect of the anisotropic properties of the clay is taken into account, the simultaneous drop of the pore pressures at locations 2, 3, and 4 can be explained by making reasonable assumptions for the values of Poisson's ratios and assuming that the lateral strains are equal to zero. The immediate effect on the pore pressures was thus found to be in the order of one-half of the release of the vertical total stress during the first excavation period. It is interesting to note that at location 1, 7 ft below the top level of the clay, the pressure drop amounted to only a few feet as a result of the swelling which took place during the excavation period.

Subsequent to the first stage of the excavation the water table was lowered by about 12 ft. The authors do not specifically mention the magnitude of the resulting decrease of the pore pressure at the upper boundary of the clay layer. It is obvious, however, that it must be introduced to its full amount in the initial excess pore pressure. The second excavation involved the removal of 7.5 ft of sand and gravel amounting to a decrease of the vertical total stress of 0.42 kg/cm^2, a somewhat larger quantity than the decrease caused by the first excavation. If the effect of these stress releases on the pore pressures would be introduced at half their values and if the effect of lowering of the water table to its full extent would be introduced, the initial negative excess pore pressure at the upper boundary of the clay layer would be in the order of 25.5 ft and at the lower boundary in the order of 13.5 ft. The fact that the authors arrive at similar assumptions for the initial negative excess pore pressures is fortuitous.

It would seem that the introduction of the immediate elastic effect and of the time-load diagram would substantially affect the magnitude of the swelling coefficient.

The laboratory testing procedure is of major importance among the factors the authors hold responsible for the discrepancy between laboratory and field values. They refer to Taylor, Leonards, and Girault in connection with the effect of the pressure-increment ratio on c_v values. It should, however, be emphasized that the swelling process of clay differs substantially from the compaction process. As a result the swelling curves do not follow the semilogarithmic linear law. However, it does not seem impossible to simulate the load-time program over a period as short as 2 months.

One can entirely agree with the conclusions concerning the importance of anisotropic permeability and the three-dimensional nature of the problem. Both factors tend to accelerate the swelling process. In my opinion, however, preference is to be given to the solution of the points discussed in the evaluation of the paper.

CONSOLIDATION PROPERTIES OF AN ORGANIC CLAY DETERMINED FROM FIELD OBSERVATIONS

Settlements and pore pressures were measured in a soft, organic silty clay deposit underlying a roadway embankment during and after construction. The deposit was stabilized by using vertical sand drains and by the application of a surcharge. Values of compression indexes and consolidation coefficients were computed from settlement and pore pressure measurements as a function of the time-load program. These values were compared with laboratory test data of undisturbed samples taken previous to construction. Samples taken at the time when primary consolidation was nearly completed were used to measure the gain in shear strength of the clay.

Authors Schmidt and Gould give particular attention to the effects of displacement driving of sand drains as evidenced by conclusion No. 1, the second part of conclusion No. 3, and conclusion No. 4. From these conclusions it would follow that the early settlements are ascribed to the driving disturbance and the gas contained in the organic soil. Both factors would contribute to an increase of the c_v values. Part of the rapid initial settlement may, however, also be due to the instantaneous deformation of the clay foundation as a whole, considering the ratio between the thickness of the layer and the width of the embankment. The driving disturbance should have been considerable in view of the ratio of 1 to 5 between the diameter of the drains and their distance center-to-center. It was found that the rather small pore pressures at mid-distance between the drains had dissipated after about one month. Since the first placement of the fill occurred a few months after the driving operations were completed the clay soil had already gone through primary consolidation by that time. It therefore does not seem likely that the rapid initial settlement would be the result of the increase in the compressibility of the clay due to the driving disturbance. The instantaneous deformation of the clay foundation as a whole seems a more plausible explanation. One of the consequences of the reconsolidation of the clay in lateral directions is the reversal of the field stress conditions. The authors conclude that the c_v value computed for the rapid early settlement records cannot be used to extrapolate the rate of consolidation to determine a time for surcharge removal. This conclusion is supported in part by the preceding discussion. If, however, a major part of the initial settlement could be accounted for by the instantaneous effect of deformation a better approximation of the rate of settlement during the early phase might be possible. One can agree with the principal conclusion that the present state of knowledge does not warrant an accurate design, utilizing a carefully determined ratio of the vertical and horizontal coefficients of consolidation, with an allowance for smear or disturbance. The fact that they obtained a more than satisfactory agreement between predicted and observed settlement behavior of the embankment by using a conservative value of the average vertical coefficient of consolidation suggests a reasonable similarity between the laboratory test procedures and in situ behavior.

The authors are to be commended for the comprehensive presentation of the results of their investigations.

IN SITU PERMEABILITIES FOR DETERMINING RATES OF CONSOLIDATION

Weber investigated the effect of in situ measured permeabilities on the agreement between observed and predicted settlements of embankments. Using the in situ k values and the a_v values from laboratory pressure vs void ratio curves, the author obtained a much closer agreement between predicted and actual time-settlement curves than by using c_v values obtained from laboratory consolidation tests. With the latter method deviations mostly occurred in the time lags, though in two out of the five cases the theoretical curve also intersected the observed time-settlement curve. In the other three cases the "final" settlements agree rather well.

One can agree with the conclusion that the permeameter measures primarily the lateral permeability whereas the rate of consolidation in the (routine) laboratory test depends mostly on the vertical permeability. In three cases the author found a considerable discrepancy between permeabilities with ratios varying between 100 and 5000; in five cases between 5 and 40; and in twelve cases between 1 and 5. Nine measurements belonging to the latter group were obtained in a rather uniform soft silty clay. The majority of the given ratios can be expected to occur in normally consolidated sediments. Ratios in the order of hundreds and thousands, however, cannot be explained by preferred orientation of soil particles resulting from normal consolidation in the vertical direction.

The use of the extremely high in situ values has resulted in a better agreement between predicted and observed rates of consolidation in all cases. The "final" settlement of the embankments proved to be in close agreement with the 100 percent consolidation compression, except in one case. This fact would seem to indicate that the a_v values obtained from the pressure-void ratio curves were appropriately chosen.

The fact remains, however, that the effect of the two-dimensional nature of the deformation and flow problem cannot be neglected in a prediction of long-term settlements. Although piezometers were placed at significant depths, there are no indications in the paper that the piezometric measurements were used to check the theoretical pore pressure vs time curves. Such comparisons would have served the purpose of evaluating the validity of the assumptions used in the prediction of the settlements.

SOIL BEARING TESTS USING A SPHERICAL PENETRATION DEVICE

Authors Butt, Demirel, and Handy investigated the use of a spherical shape loading surface in penetration techniques. It was found that reproducibility of test results with this device was far better than of results obtained with other penetration devices. Satisfactory agreement was also obtained between the spherical bearing value and the unconfined compressible strength in clay soils. The ultimate bearing capacity obtained from Terzaghi's equation for shallow footings turned out to be close to the sphere bearing value. The authors did not include cone penetrometer results in their comparative studies.

CONCLUSIONS

A summary of conclusions of the Symposium papers is not a simple task in view of the different objectives set by the authors. On this basis a distinction can be made between the studies of Hanna and Adams and of Soderman, Milligan, and Kim on the one hand, and those of Schmidt and Gould and of Weber on the other. Whereas the first two papers attempted to determine the properties of clay deposits through the immediate effects of the release and application of loads, the last two papers are concerned with long-term prediction of the time-settlement behavior of embankment loads. The third paper, by Bromwell and Lambe, dealt with the case of short-term effects due to excavation and water table lowering at the top level of a clay layer.

A point common to four out of five papers is the assumption of the one-dimensional nature of the problems. Considering the agreement found by Schmidt and Gould and Weber, the effect of two-dimensional deformation and flow on the vertical rate of consolidation would not have been a significant factor. In the work reported by Bromwell and Lambe the effect of the three-dimensional nature of the problem could have been stronger. In all cases, conclusions concerning the magnitude of these effects on the basis of agreements or differences between predicted and observed behavior would be premature because of the influence of other significant factors.

The lack of similarity between laboratory or in situ testing procedures and deformation processes in the field may well be one of the most important factors. It is well-known that small loading increments in consolidation tests on clays may yield compression indexes varying from one-tenth to one-twentieth of the value obtained by the standard increment procedures. In several instances the authors have not found an acceptable relationship between the elastic deformation properties of clays and the undrained strength of clays. In other instances no noticeable increase of the undrained strength was found in the field at the end of the primary consolidation period. These instances demonstrate a basic fallacy in the definition of strength and the methods used for the measurement of this quantity.

If effects of the multidimensional nature of problems are to be included in the prediction of the rate of consolidation we have to be aware of the small strain type solutions presently available. Test results obtained from large incremental load programs are not applicable to this kind of solution.

It therefore seems that progress in the prediction of settlement behavior in the near future requires improvement of testing techniques, both in the laboratory and in situ, before more sophisticated techniques of computation are attempted.

Comparisons between observed and predicted behavior as achieved by the authors of the Symposium papers are badly needed to realize the progress made in these attempts.

A similar problem exists regarding the use of a deformation modulus for the computation of instantaneous deformations under no drainage conditions. If a modulus is required it should take the initial in situ stress conditions into account.

REFERENCE

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