Comparison of Field and Laboratory Measurements of Modulus of Deformation of a Clay

THOMAS H. HANNA, Sheffield University, England, and
JOHN I. ADAMS, The Hydro Electric Power Commission of Ontario, Canada

The results are presented of a field and laboratory study of the deformability of a natural clay from a site in southwestern Ontario tested under undrained loading conditions. It was found that the laboratory-determined modulus of deformation is very sensitive to the test load system used and the stress level applied as well as the magnitude of the undrained shear strength. Moduli of deformation determined in-place by plate loading tests and inferred from in-place ground movements resulting from construction work generally were different from and in most cases greater than the laboratory-determined values.

•WHEN a soil mass is loaded it will deform and work will be done. The work done may be conveniently divided into an irrecoverable component (plastic) associated with particle slippage and a recoverable component (elastic) associated with pressure changes. In routine testing the sum of the components is measured. Perfectly elastic materials have a linear stress-strain behavior. Ductile materials, however, can tolerate strain beyond the limit of elasticity following elastic-plastic behavior, and their deformation may be studied by "perfect" theories of plasticity. Soils are at the soft end of the range of engineering materials and have stress-strain characteristics with a definite peak or local failure curve of many forms depending on the method of test. Several rigorous attempts have been made to study the stress-strain behavior of soils (1, 2).

Clay soils comprise very fine plate-shaped particles surrounded by both adsorbed and free water. The geological processes common to the formation of a clay stratum can result in a preferred orientation of these particles as well as laminations and stratifications due to different rates of deposition. Changes near the ground surface resulting from desiccation and weathering can cause fissuring and cracking to considerable depths. From all the processes involved most clays possess, in situ, a system of shear stresses. The sampling and preparation of natural clays for laboratory testing invariably cause some damage to the arrangement of the clay particles and alteration of the stress system. The replacement of the estimated in-place field stress system in the laboratory specimen and the simulation of the additional stress changes that result during field loading cause deformations that are partly irrecoverable.

The immediate displacements of a clay stratum, both at ground surface and depth, may be important and under certain circumstances they can have a decisive control on design and construction. In many cases reliable prediction of ground movement is difficult and in such cases a valid approach is to measure the resulting field movements and compare them with computations based on laboratory test information. A site near Sarnia, Ontario, provided such an opportunity. Comprehensive deformability data for the firm-to-stiff clay deposit, obtained by laboratory triaxial testing, were complemented by several sets of field movements. This paper presents some of the test in-

Paper sponsored by Committee on Soil and Rock Properties and presented at the 47th Annual Meeting.

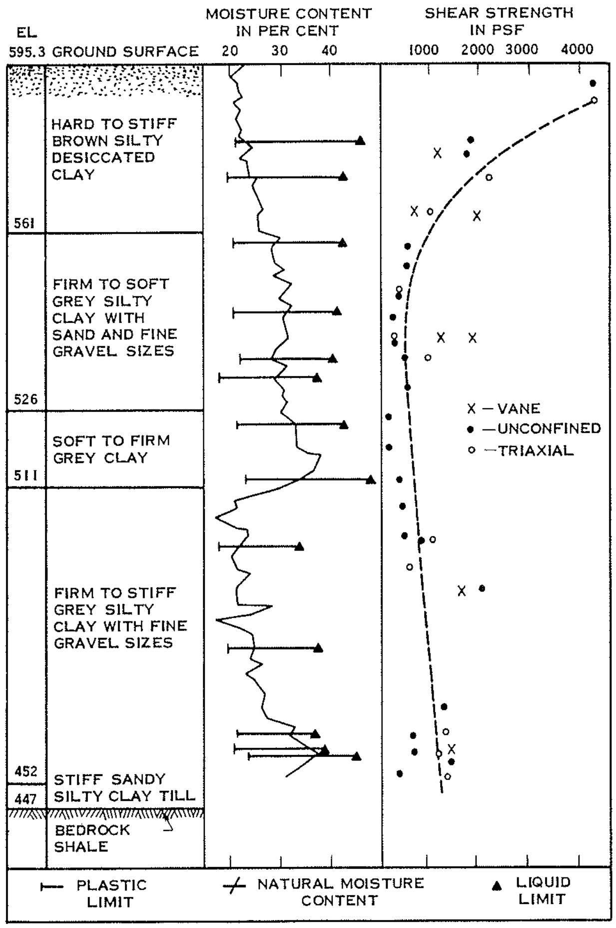

Figure 1. Geological profile.

formation obtained from this site. The data refer to natural clays from a large site where testing was predominantly commercial, and therefore the results are of a practical nature.

GEOTECHNICAL PROPERTIES OF THE CLAY

The site investigation for Ontario Hydro's 2000-megawatt generating station near Sarnia, Ontario, where the overburden consists of lightly overconsolidated "glacial-lake" clays, provided a large number of 2-in. and 2.5-in. Shelby-tube samples recovered from augered boreholes. To obtain less disturbed samples, piston techniques were used and during excavation works several 1-ft cube samples were recovered. A general description of the soil strata is contained in a recent paper (3) and details of the variability of the engineering properties will be reported eslewhere (4).

A detailed geological profile for a sampled borehole is shown in Figure 1. The desiccated surface crust has resulted in a hardening of the upper clay layers as reflected in the strength profile on the right. Extensive data indicate that the clays forming this profile are of the same geological origin (5) and that the differences in engineering property are a function chiefly of the natural water content (4).

While the clay is geologically very uniform it does exhibit definite laminations and stratifications in places (3, 4) and near the ground surface it is severely fractured by surface drying. Thus there is a definite sequence of layers of clay, macroscopically homogeneous, which constitute the profile. It is known that the in-place stress system existing within this mass of clay is heterogeneous and indeterminate. From this brief summary of the general soil conditions prevailing, it is evident that the mass behavior of the in-place clay is very different from that of a small laboratory test specimen. Throughout the following report, this limitation must be kept in mind, because it provides an understanding of the differences in behavior of natural clays and the extremely idealized versions that are assumed for test data interpretation and subsequent deformation distribution computation.

THE LABORATORY-DETERMINED MODULUS OF DEFORMATION

Unconsolidated undrained triaxial compression tests, run at an axial deformation of 0.05 in. per min, provided values of the modulus of deformation (secant value), E, determined from the reload part of the stress/strain curve. (These values refer to a stress level of about half the undrained strength of the test specimens.) The modulus values, E, are graphed against the undrained shear strength, C, and Figure 2 shows that the

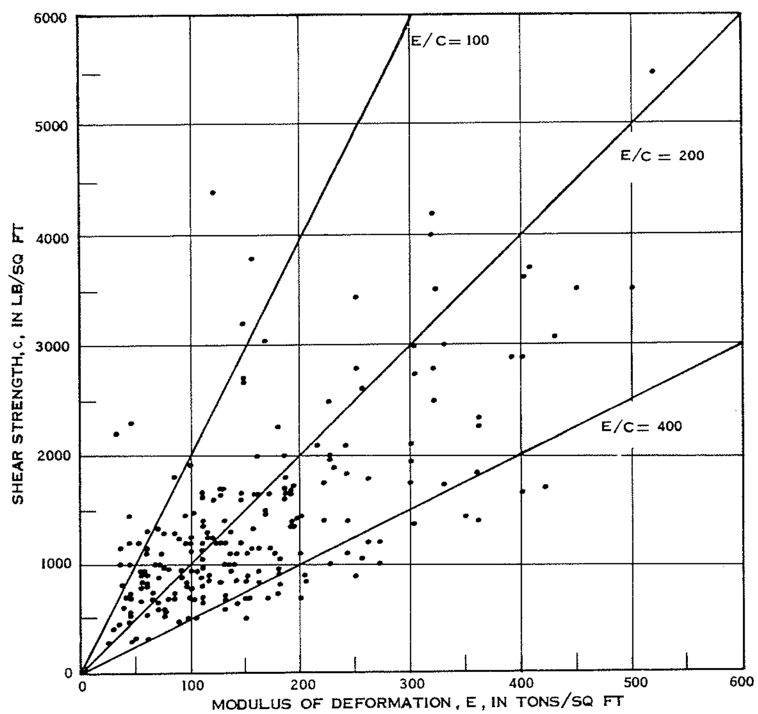

Figure 2. Relation of modulus of deformation, E, to undrained shear strength, C.

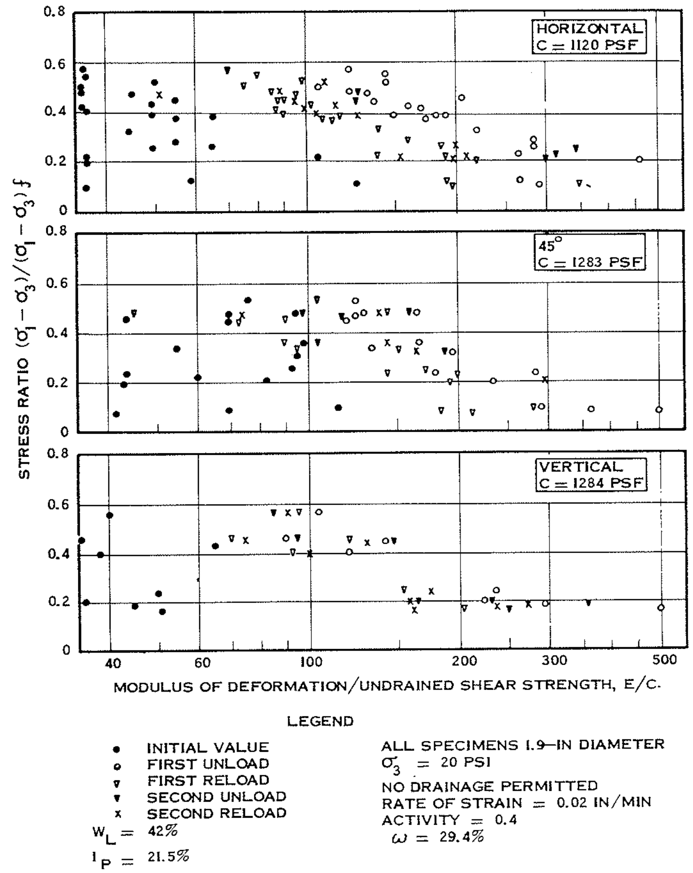

Figure 3. Modulus of deformation values for unconsolidated undrained triaxial specimens loaded at 0, 45, and 90° to the in situ vertical direction at constant strain loading.

data fall in the range 100 < E/C < 400, approximately. This information refers to specimens from the whole site. There are several disadvantages to the above system of modulus determination. Included are the strain-rate effect and the effective stress-loading system. To explore these factors in some detail, additional testing was performed on 1.9-in. specimens cut from 1-ft cubes of clay obtained at the 40-ft level in the side of a deep excavation. The testing comprised constant strain and constant stress methods of loading. In all tests, the top and bottom platens were lubricated with silicone grease to reduce the effect of end friction restraint on the specimen.

Constant Strain Tests

Samples were tested with the direction of the applied major principal stress inclined at 0, 45, and 90 deg to the in situ vertical direction, the load being cycled at stress levels between approximately 0.2 and 0.5 of the ultimate strength. An axial deformation rate of 0.02 in. per min was used and no drainage was permitted. Values of the secant modulus of deformation divided by undrained strength, E/C, are graphed against factor of safety of the specimen to failure in Figure 3. (Absolute values of E may be obtained from Figure 3 and subsequent Figures as values of c are quoted.) Several trends are evident: with increase in stress level, the modulus value decreased; the initial load modulus values were smallest while the unload values were greatest; and the moduli value for specimens tested with their axes horizontal were slightly greater than for specimens with their axes parallel to the in situ vertical. There was little difference between the 45° inclined and the horizontal specimens It was also found that the undrained strength measured with the direction of loading horizontal was slightly greater than the strength when loaded in the vertical direction (6).

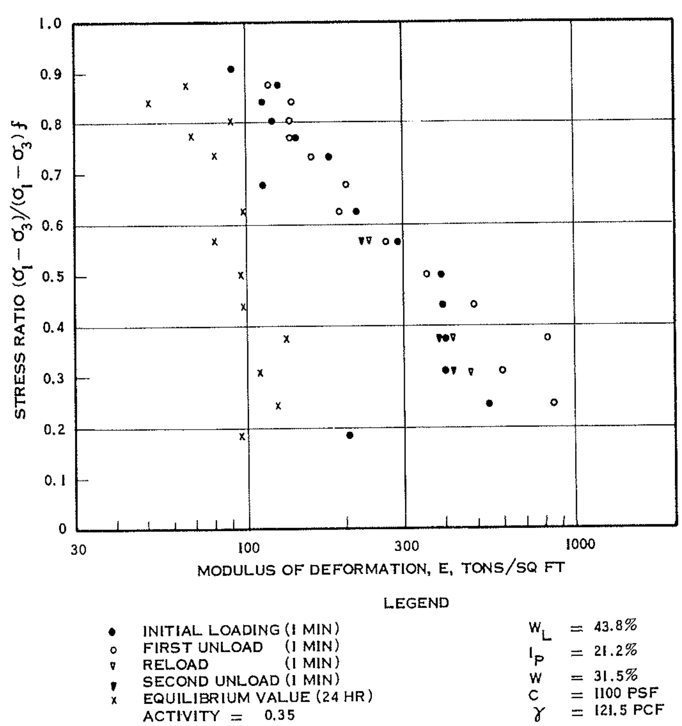

Figure 4. Modulus of deformation values for unconsolidated undrained triaxial specimen at constant stress loading.

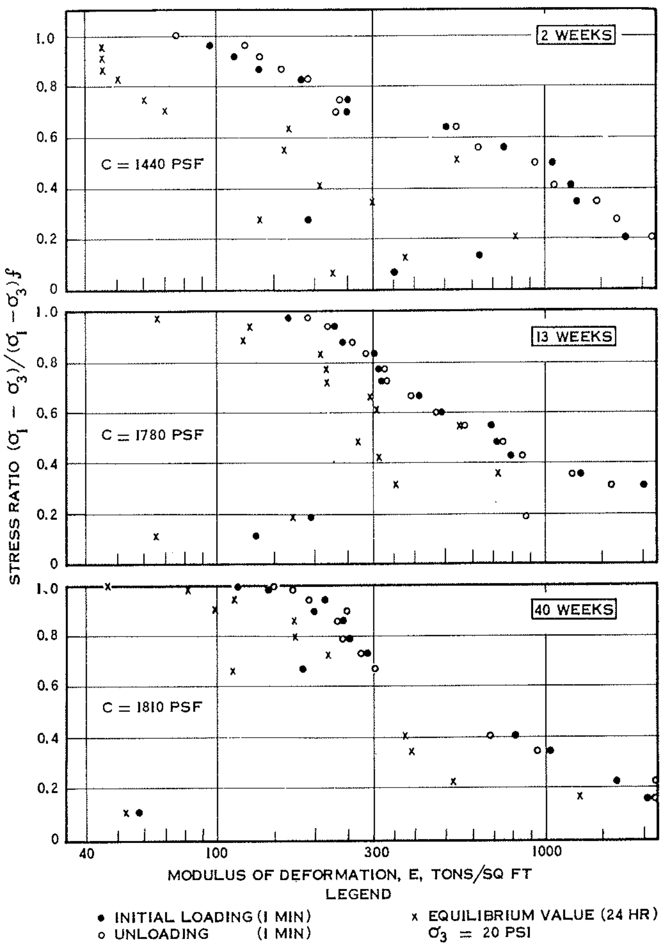

Figure 5. Modulus of deformation values for consolidated undrained triaxial specimens at 2, 13, and 40-week ages at constant stress loading.

Constant Stress Tests

Three variables were investigated: time of loading, isotropic consolidation, and stress level. All specimens were cut from a 1-ft cube of soil with their axes parallel to the in situ vertical. Loading was by deadweight to a hanger; generally each load increment was about 6 percent of the failure load and was maintained for a 24-hr period.

Two specimens were tested under unconsolidated undrained conditions with an applied cell pressure of 20 psi. Values of the modulus of deformation are graphed against stress level in Figure 4 for one test.

Three other "identical" specimens were consolidated by an isotropic effective stress of 20 psi with a back pressure of 40 psi. (In the present context, "identical" refers to specimens cut from adjacent parts of the cube sample. There are slight differences in plasticity, gradation, and natural water content.) Periods of consolidation of 2, 13, and 40 weeks ensued prior to the start of the load test. During this time the laboratory temperature was 68 ± 2 F. The drainage comprised a $^3/_8$-in. stone recessed in the top loading platen. Thus it was possible to use enlarged and lubricated end platens in all tests. The loading tests were similar to the unconsolidated undrained tests with a reload cycle at the start of each load increment. Values of the modulus of deformation are shown in Figure 5. Several trends emerged: the undrained shear strength increased with the period of consolidation, i. e., age; the initial loading and unloading moduli values were of the same order of magnitude; the moduli values decreased with stress level; and there was no significant difference between the modulus of deformation and the age of the specimen.

FIELD DEFORMATION MEASUREMENTS

In-Place Plate Bearing Tests

A number of in situ plate loading tests were performed against the walls of test trenches cut at several depths in the side of a deep excavation. The 9-in. diameter by 1-in. thick plate was stiffened by the 6-in. diameter loading strut, which contained the hydraulic loading jack. Reaction for loading was obtained from the opposite wall of the test trench. The loading plates penetrated the clay surface at 0.15 in. per min, giving a time to failure in the order of 10 min. The load was cycled at several stress levels. By use of the simplification that the vertical clay face is equivalent to an elastic half-space and that the clay is homogeneous, unfissured, and isotropic, the penetration of the rigid plate was used to estimate the modulus of deformation of the clay. A summary of the moduli values computed from the plate tests is given in Table 1.

Many criticisms may be directed to the method of test and its analytical interpretation, and the unconditional use of elastic theory has been queried recently (7). Accepting the limitations, it is seen from the data in Table 1 that the moduli values during loading are less than during unloading, but there is a wide range in value, suggesting that some of the deformation readings are in error.

In-Place Ground Movements

Complementary to the small-scale laboratory test program, simple field measurements were made to record in-place ground movements resulting from load changes in the clay mass. The work comprised the monitoring of vertical and horizontal movements at several convenient ground positions. Data typical of the movements recorded are presented.

TABLE 1

MODULUS OF DEFORMATION VALUES FOR IN-PLACE HORIZONTAL PLATE BEARING TESTS

Depth (ft)	q/q_f	C (psf)	North Side		South Side		Thickness of Surface Crust (ft)
			$\frac{E}{c}$ Load	$\frac{E}{c}$ Unload	$\frac{E}{c}$ Load	$\frac{E}{c}$ Unload	
8	0.36	1800	100	320	85	170	
	0.48		290	910	240	980	15
	0.66		520	420	430	420	
8	0.28	2340	160	190	140	210	
	0.37		275	195	285	230	
	0.59		150	145	155	190	15
	0.75		165	195	170	210	
19	0.23	4040	220	150	170	230	
	0.40		300	1120	380	1600	
	0.52		300	530	630	430	25
	0.71		320	600	240	520	
34	0.18	2010	340	—	100	1170	
	0.32		740	2400	—	—	
	0.46		940	740	690	630	
	0.51		1100	610	860	600	25
	0.76		330	570	170	510	
	0.86		500	—	380	500	

Note: The test was carried out within 6 hr of trench excavation. All values are secant values and refer to 1-min load readings. q = applied plate pressure; q_f = ultimate bearing capacity.

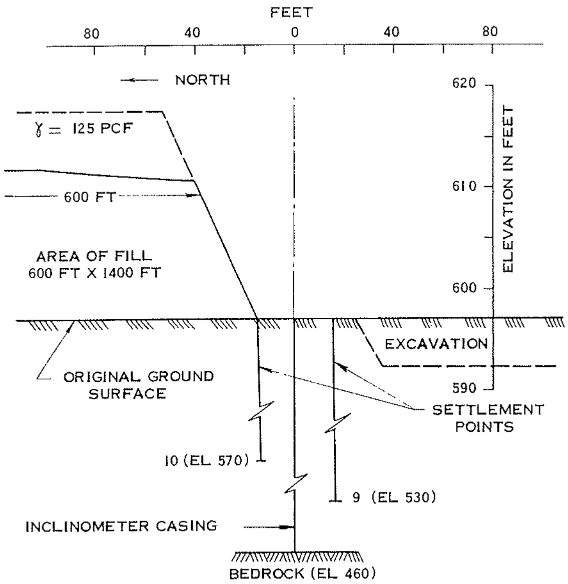

Figure 6. Profile through test fill and heave installation (solid line, profile 6/65; dashed line, profile 7/65).

Advantage was taken of a spoil area, and both lateral and vertical soil movements due to construction were measured. Two heave gages, similar in design to those reported by Bozozuk (8), were installed near the toe of the fill and their elevation determined to 0.001 ft by precise level prior and subsequent to fill placement. A profile through the fill, parallel to the heave gages, is shown in Figure 6. Also included are details of the heave gage positions and bedrock. The soil profile (Fig. 1) is near the heave gage installation and may be taken to be representative of the soils beneath the test fill. A slope indicator tube was installed midway between settlement gages 9 and 10 (Fig. 6) and anchored into bedrock. The space around the tube was filled with a medium sand placed by pouring through water. The slope of the casing was determined after

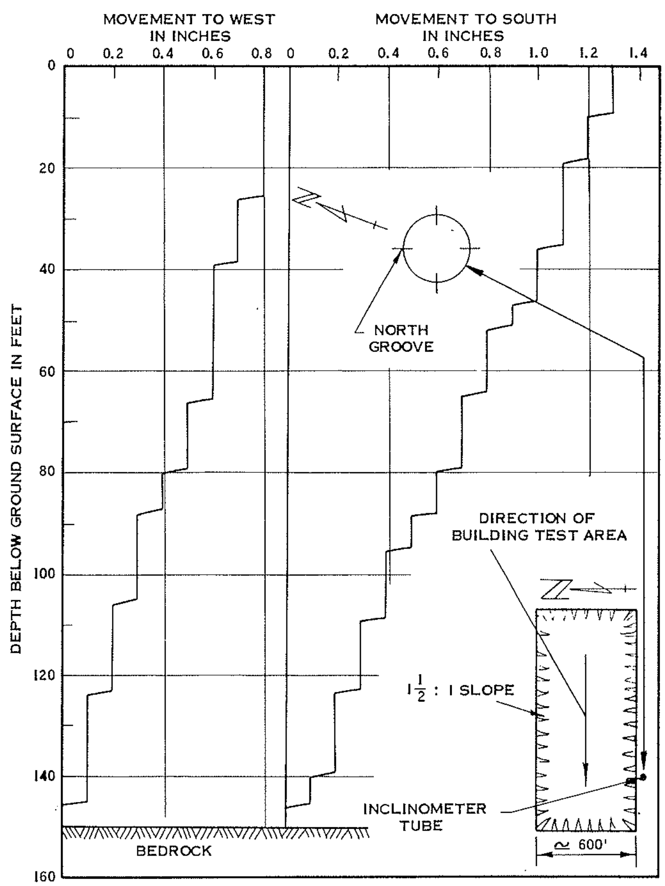

Figure 7. Lateral ground movement profiles adjacent to test fill.

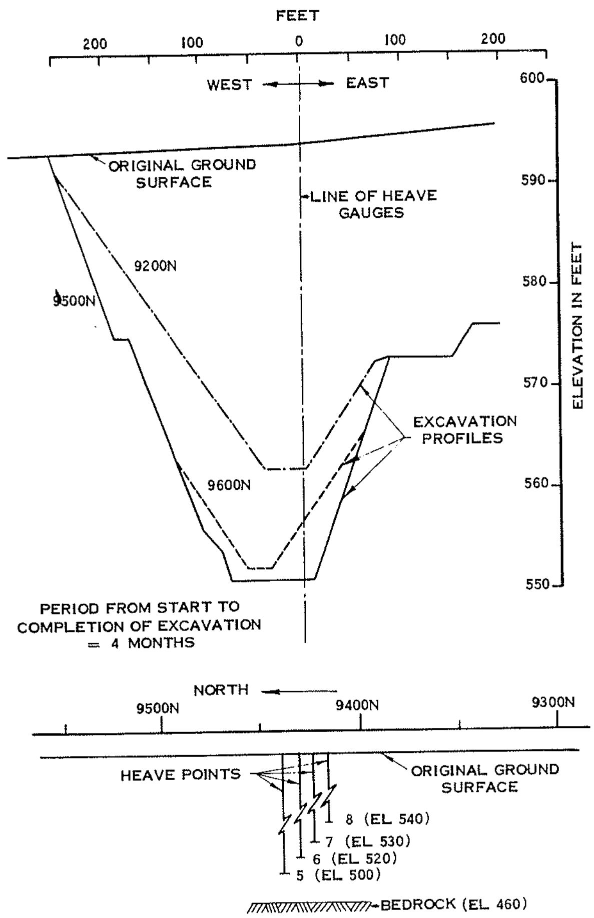

Figure 8. Profiles through excavation and heave points.

installation by use of a Wilson slope indicator series 200B instrument (9). Similar readings were taken subsequent to completion of the test fill construction. Subtraction of the deflection components for any particular depth gives the lateral components of clay movement in perpendicular directions. A profile showing the lateral ground movements resulting from the placement of the fill is shown in Figure 7. The movement away from the side of the test fill is about twice the magnitude of the movement parallel to the side of the fill. This observation is compatible with the order of fill placement, which proceeded from east to west.

A large excavation up to 45 ft deep was instrumented, with four heave gages at different depths below final grade. Profiles normal and parallel to the heave gages, both before and after excavation, are shown in Figure 8. This excavation was dug during a period of 4 months. Prior to the start of construction, the water table was about elevation 585, some 15 ft below ground surface. With excavation, the water level in the clay was lowered, but detailed readings were not taken in the vicinity of the heave points in the deep excavation. Adjacent to the test fill, four Geonor-type piezometers recorded slight increases in water pressure during fill placement.

TABLE 2

MODULUS OF DEFORMATION VALUES DETERMINED FROM FIELD DATA

Heave Gage No.	Observed Movement (ft)		Modulus of Deformation (tons/sq ft)		Remarks
	Settlement	Heave	Loading	Unloading	
5		—		—	Impossible to locate gage due to obstruction in hole.
6		0.076		1100	For details of layout see Fig. 8.
7		0.089		1265	Readings taken about 2 weeks after completion of excavation.
8		0.118		1320	
9	0.054		276		For details of layout see Fig. 6.
10	0.022		850		Readings taken at end of construction.

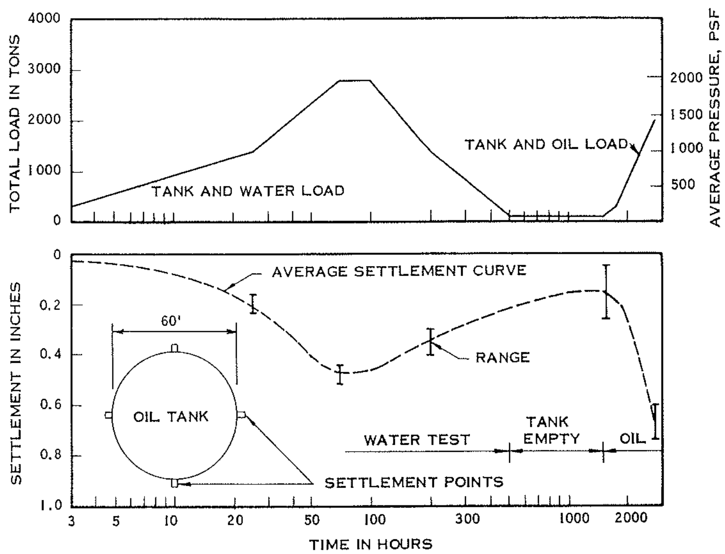

Figure 9. Oil tank load settlement data.

The vertical ground movements were translated into average moduli of deformation as follows: From the geometry of the excavation or the test fill, with respect to the heave gauge, the change in vertical stress was estimated by influence charts according to Osterberg (10). Assuming that the clay is elastic and disregarding the influence of the initial anisotropic stress system acting, the average modulus of deformation (E_{av}) for the clay between the heave gage and bedrock is

$$E_{av} = \sum \frac{\sigma v \cdot dh}{\Delta}$$

where Δ is the vertical movement of the heave gage, and σ_v is the vertical stress change in the element of height dh.

Values of the computed moduli of deformation are given in Table 2. Also included are the heaves measured and other pertinent details.

The test loading of a 60-ft diameter steel oil storage tank provided immediate settlement data during the load and unload sequence. Settlements were taken by geodetic leveling procedures to four peripheral settlement points with respect to a rock bench mark. The tank, of welded-steel construction, rested on a 5-ft thick granular pad. Details of the test load program along with the settlement data are shown in Figure 9. Values of the estimated modulus of deformation are shown in Figure 10(F).

COMPARISON OF LABORATORY-DETERMINED AND FIELD-INFERRED MODULI

Close agreement between laboratory-determined and field-inferred moduli of deformation is unlikely, except in special cases. The following sources of disagreement are believed to be important: (a) limitations of stress distribution theory, (b) inexact knowledge of the initial in-place effective stress system in the clay mass, (c) damage caused to small specimens during recovery from the ground and during setting-up for testing, and (d) the variability of the clay mass.

Bearing in mind these factors, and referring to the test data previously presented, one finds a very large range in modulus value. Generally the field-inferred moduli are larger than the corresponding laboratory values. The range of laboratory values was wide and depended on the sequence of loading, the applied stress level, and consolidation. To isolate some of the variables present and reduce the data to a generalized

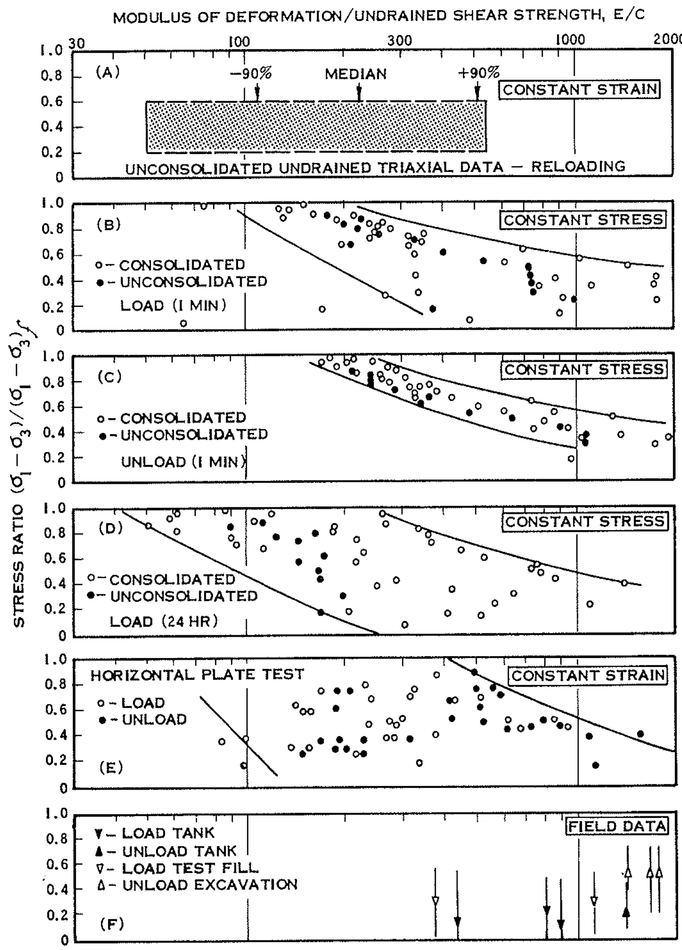

Figure 10. Comparison of field and laboratory-determined moduli.

form, the bulk of the test data was recompiled (Fig. 10). Six dimensionless plots of E/C against stress ratio are shown. (It should be noted that the data in Figure 10A refer to specimens from Shelby-tube samples, while the data in Figures 10B, C, and D refer to specimens trimmed from 12-in. cubes of soil carefully cut from the side of a deep excavation.) Figure 10A shows the routine test information obtained from over 200 unconsolidated undrained triaxial specimens reloaded once at a stress ratio about 0.5 ± 0.1. The median value and $\pm$ 90 percent range are indicated. Figures 10B, C, and D present the constant stress test information, showing the initial load values (Fig. 10B), the unloading data (Fig. 10C), and the 24-hr data (Fig. 10D). Upper and lower-bound envelopes have been drawn. The unconsolidated data are distributed throughout the consolidated data. Figure 10E shows the plate load test results. The lower graph (Fig. 10F) presents the field results obtained by taking an average undrained strength of 1500 psf for the soil profile. The stress ratio has been estimated from stability considerations and is approximate. The field data fall in two blocks, the loading results and the unloading results, with unload moduli somewhat larger than the load values. Comparison of the field data with the laboratory data shows the field results to be near the upper-limit laboratory results. Generally the constant stress tests are in better agreement with the field measurements than the constant strain tests. The importance of stress level is clearly evident when comparing test results.

It is believed that the elastic-plastic behavior of clay can account for the large range of modulus value determined by laboratory testing. On initial loading both elastic and plastic deformations occur. With increase in stress level the plastic component of deformation increases. Therefore modulus values determined by loading to high stress levels are less than those for loading to lower stress levels. Unloading causes a near-elastic rebound which gives a high modulus value. On reload the deformations are less than during initial loading but greater than the unload values. The marked reduction in modulus value obtained from the 24-hr reading shows creep to be significant. It is considered that the stress-controlled tests (1min reading) measure a near-elastic movement, particularly at low stress levels.

CONCLUSION

The summary of the field and laboratory-determined moduli shown in Figure 10 shows the laboratory triaxial test to underestimate the in-place modulus required for computing immediate ground movements. Several trends emerge—

1. Constant stress testing provides moduli values greater than constant strain testing, and the former are in better agreement with mass behavior than the latter.
2. Moduli determined from the loading and unloading part of the stress/strain relationship are different, the unloading value being the larger. Data inferred from the field movements show a similar trend.
3. The applied stress level had a marked influence, the modulus decreasing with stress ratio increase. This is believed to be due to the elastic-plastic behavior of soils.

ACKNOWLEDGMENT

Thanks are due to the Project Design and Construction Departments of the Generation Projects Division of the Hydro Electric Power Commission for their assistance with field installation and recording of data. The oil tank instrumentation was performed by the Applied Mechanics Section of the Structural Research Division.

REFERENCES

1. Rowe, P. W. The Stress-Dilatancy Relation for Static Equilibrium of an Assembly of Particles in Contact. Proc. Royal Soc. (London), Series A, Vol. 269, pp. 500-527, 1962.
2. Roscoe, K. H., Scoffield, A., and Wroth, C. P. On the Yielding of Soils. Geotechnique, Institution of Civil Engineers, London, Vol. 8, No.1, pp. 22-53,1958.
3. Hanna, T. H. Engineering Properties of Glacial-Lake Clays Near Sarnia, Ontario. Ontario Hydro Res. Quart., Vol. 18, No. 3, 1966.
4. Hanna, T. H., and Adams, J. I. Variation of the Engineering Properties of a Glacial-Lake Clay. (In preparation.)
5. Wu, T. H. Geotechnical Properties of Glacial Lake Clays. Jour. Soil Mech. and Found. Div., ASCE, Vol. 84, No. SM3, Proc. paper 1732, 1958.
6. Hanna, T. H. Discussion of Shear Strength Properties of Two Stratified Clays by Kwan Yee Lo and Victor Milligan. Jour. Soil Mech. and Found. Div., ASCE, Vol. 93, No. SM3, pp. 183-188, 1967.
7. Hanna, T. H. Discussion of The Elastic Properties of a Dense Glacial Till Deposit by E. J. Klohn. Canad. Geotech. Jour., Vol. 2, No. 2, pp. 129-131, 1965.
8. Bozozuk, M. The Modulus of Elasticity of Leda Clay From Field Measurements. Canad. Geotech. Jour., Vol. 1, No. 1, pp. 43-51, 1963.
9. Wilson, S. D., and Hancock, C. W. Horizontal Displacements of Clay Foundations. Proc. First Pan-American Conf. on Soil Mech. and Found. Eng., Mexico City, 1959.
10. Osterberg, J. O. Influence Values for Vertical Stresses in a Semi-Infinite Mass Due to an Embankment Loading. Proc. Fourth Internat. Conf. on Soil Mech. and Found. Eng., Vol. 1, pp. 393-394, London, 1957.