Field and Laboratory Studies of Modulus of Elasticity of a Clay Till

L. G. SODERMAN and Y. D. KIM, University of Western Ontario, and
V. MILLIGAN, H. Q. Golder and Associates Ltd., Cooksville, Ontario

As part of a settlement study of structures founded on clay till in southern Ontario, factors influencing the modulus of elasticity of the till have been investigated. In situ plate tests and laboratory tests on tube and block samples were carried out under repetitive loading cycles, during which pore pressures were measured. Remolded samples were also tested and the results are compared with the field and undisturbed laboratory tests.

The results of the study indicate that the modulus of elasticity is extremely sensitive to sample disturbance; that the modulus of elasticity should be defined for the specific level of stress considered; and that definite relationships seem to exist between the undrained shear strength, consolidation pressure, and modulus of elasticity for this specific clay till. Pore pressure behavior during undrained repetitive loading in the laboratory and in the field was similar.

•IN the prediction of both immediate and time-dependent deformations resulting from the application of loads to a soil mass, general use is made of the theory of elasticity, incorporating soil properties that must, in most cases, be arrived at from laboratory tests made on the "best samples" that can be obtained from the particular soil deposit. In addition to the inherent problem of sample disturbance, the inelastic and nonlinear load-deformation characterisitc of soils necessitates an arbitrary definition of modulus of elasticity that must be used in the classical elastic theory solutions for stresses and deformations.

The results reported in this paper relate specifically to the practical determination of a modulus of elasticity to be used in predicting the immediate settlement of a highway embankment underlain by a deep deposit of near-normally consolidated clay till. A very common and characteristic feature of this clay till deposit is a relatively stiff desiccated surface crust having a thickness of about 20 ft; below this depth, the deposit is considered to be normally consolidated. The significance of the relative stiffness of the surface layer in evaluating the stress distribution throughout the deposit led to a detailed study of elastic modulus determination for this layer. The results obtained are reported in terms of total stresses only, since these data are sufficient to emphasize the sensitivity of the modulus of elasticity as determined in the laboratory to such factors as (a) sample type and size, (b) test type and procedure, (c) stress ratio, (d) rate of strain, and (e) number of cycles of repetitive load applied. The structure has been instrumented to monitor settlements; consequently, the field average value of the modulus will be forthcoming.

The stress-strain modulus used most commonly in nondynamic soil engineering problems dealing with saturated clay is that obtained from an undrained compression test either unconfined or confined in a triaxial test cell. Practical justification for the undrained tests in problems having to do with deformation immediately following a load application is found in the fact that the permeability of the clay soil is such that significant drainage is precluded during the relatively short time interval involved during

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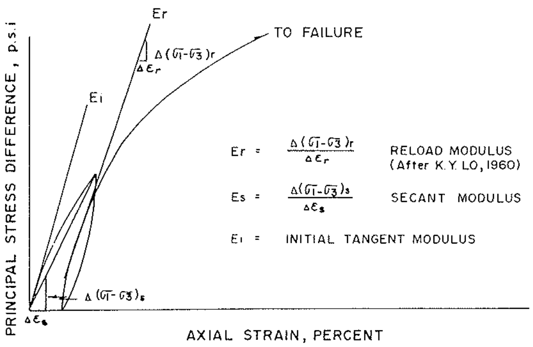

Figure 1. Definitions of moduli [$\Delta \epsilon_s$, $\Delta \epsilon_r$ = axial strain increment; Δ (σ_1 - σ_3)s, Δ (σ_1 - σ_3)r = principal stress difference increment].

loading. The most frequently defined moduli from such undrained compression tests (Fig. 1) and (a) initial tangent modulus E_i, taken as the slope of the virgin stress-strain curve, and (b) the secant modulus, E_s, taken as the ratio of stress to strain at a particular stress value on the virgin loading curve. It has been previously suggested (7) that the modulus defined by the virgin loading curve is a quantity that involves both pseudoplastic and elastic effects and is not a true elastic modulus. Cyclic or repetitive loading results in a more linear compression curve and the true modulus of elasticity may be calculated from this curve. This modulus is shown in Figure 1 and is termed "reload modulus" denoted by E_r. A number of investigators have compared moduli determined from first loading and reloading stress-strain curves on laboratory samples (2, 10), showing that the ratio of reloading modulus to initial loading modulus from laboratory compression tests varied from about 3 to 10. A notable exception to this is reported by Klohn (5), who states that the initial tangent modulus from unconfined compression tests and the reloading modulus from triaxial compression are sensibly the same for a dense glacial till. Moduli of elasticity determined from in situ plate loading tests were found to be "of the same general order of magnitude as those obtained from field observations," whereas laboratory compression test values varied from about one-half to one-tenth of those found from plate loading tests (5). The best agreement between plate test values and values obtained from structure performance was obtained by "using the average slope of the hysteresis loop that develops when the plate is subjected to cyclic loading." Simons (8), reporting on settlements of two structures in Norway, found that E values from Q tests gave values lower than those measured from actual settlement, while E values from R tests were too high. (Q test is an unconsolidated undrained triaxial test; R test is a consolidated undrained triaxial test.)

SITE LOCATION AND SOIL CONDITIONS

The site investigated is located in southwestern Ontario about 35 miles east of Detroit at the point where the MacDonald-Cartier Freeway crosses Tilbury Creek. The general geotechnical properties of the clay till deposit have been summarized by Soderman et al (9), and the gelogical origin of the till has been discussed by Dreimanis (3), who suggested that the till sheet was deposited during the Wisconsin Ice Age. The soil profile is shown in Figure 2. In general, the soil is of remarkably uniform composition and, with the exception of the desiccated upper crust, the deposit appears to be near-normally consolidated. The index properties and mineralogical composition, provided by R. M. Quigley, are given in Table 1. Occasional shale fragments of about ½-inch size were found throughout the entire deposit. The water table was found to be at a depth of 3½ ft and this coincided with the water level in Tilbury Creek. No artesian conditions were found either in the bedrock or overburden.

SAMPLING AND TESTING

The postconstruction investigation was carried out in two stages. The initial stage consisted of deep borings from which thin-walled Shelby tube (area ratio < 10 percent) samples were removed in diameters of 2 and 3 in. The boreholes were positioned on a 10-ft grid and undisturbed samples were taken so that the different diameter samples were from the same depth in adjacent boreholes. All samples were carefully wax-sealed immediately upon removal and stored in a humidity cabinet until tested.

TABLE 1

TILBURY CLAY TILL PROPERTIES

Property	Value
Natural moisture content, W	22 percent
Liquid limit, W_L	30 percent
Plastic limit, W_P	18 percent
Clay fraction (< 0.002)	42 percent
Activity	0.30
S_u (in desiccated crust depth 0-20 ft)	1.5-3.0 kips/sq ft
S_u (below desiccated crust)	0.7-1.2 kips/sq ft
Sensitivity	2-3
ϕ' = Angle of shearing resistance in terms of effective stress	26 deg
c' = Apparent cohesion in terms of effective stress	100-200 psf
pH—pore fluid	8.5
Salinity—pore fluid (equivalent NaCl)	2.7 gm/liter
Clay minerals—illite, chlorite	
Nonclay minerals—quartz, feldspar, calcite, dolomite	
Ratio—calcite to dolomite	1.3

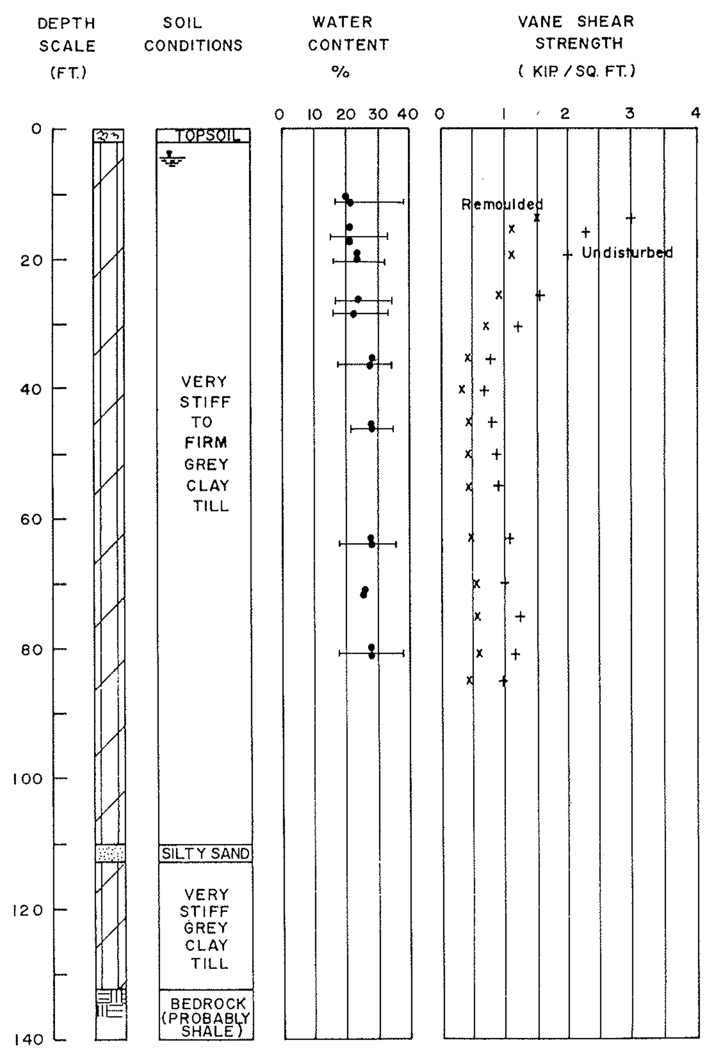

Figure 2. Generalized soil conditions, Tilbury test site.

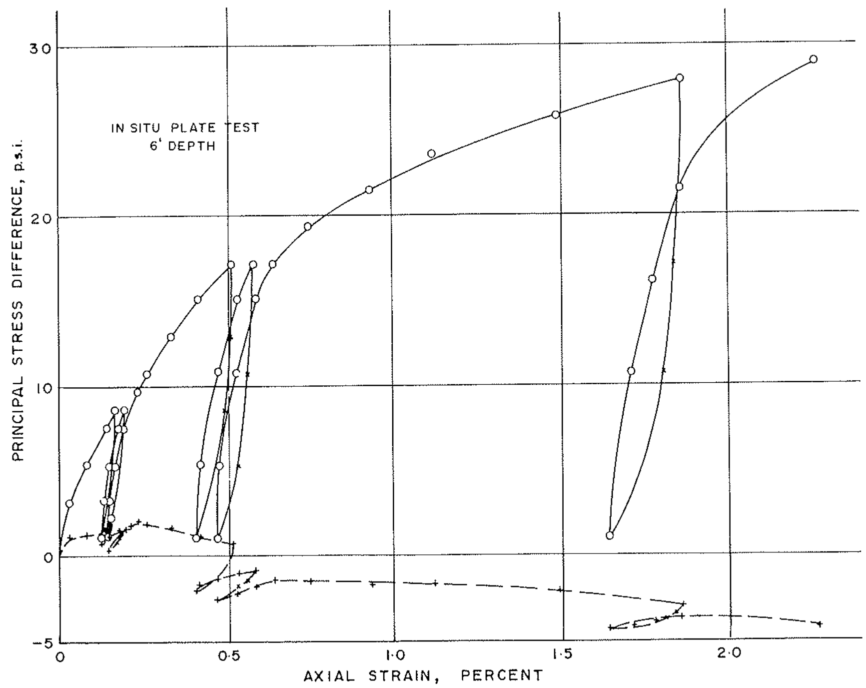

Figure 3. Stress-strain-pore water pressure relationship, 6-in. diameter plate test.

The second stage of the investigation was limited to the upper crust material and consisted of a test trench excavation so that horizontal benches at depths of 6, 10, and 14 ft resulted. Samples were taken at each bench elevation using thin-walled Shelby tubes about 18 in. long with diameters varying from 2 to 4 in. These sampling tubes were driven or pushed into the undisturbed soil to a depth of 15 in. and then dug out, rather than being rotated and pulled out. The inside clearance was zero percent for the tubes used (10), allowing no expansion of sample in the tubes. Both horizontal and vertical tube samples were taken. Plate loading tests using a cyclic loading procedure (Fig. 3) were carried out at each bench elevation both in a horizontal and vertical direction. The plate tests were limited to a single plate having a diameter of 6 in. The

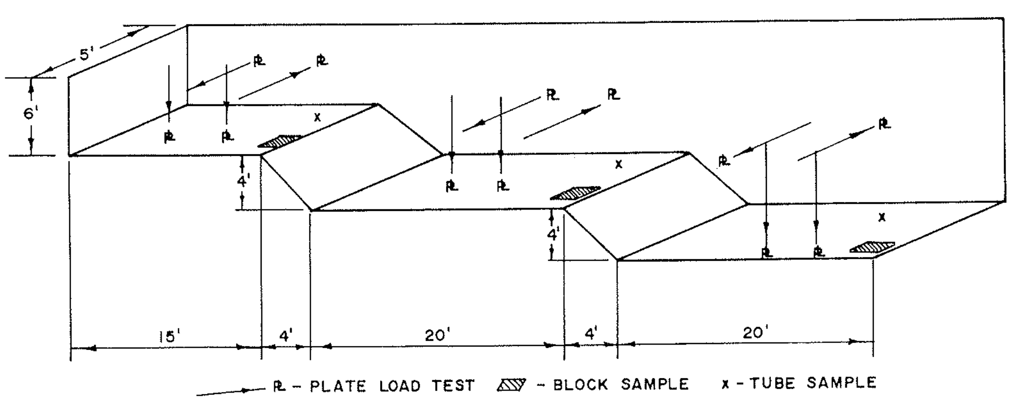

Figure 4. Schematic diagram of location samples in test pit.

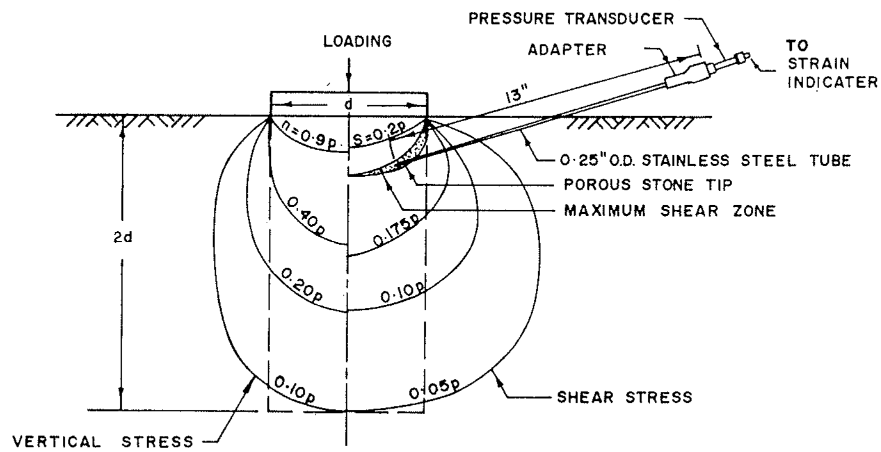

Figure 5. Distribution of stresses under a circular footing (after Jurgenson, 1934).

test trench details are shown in Figure 4. A transducerized porous-tipped needle was inserted into the theoretical maximum shear zone below the loading plate (Fig. 5) and pore water pressures were continuously recorded during the cyclic plate load tests (Fig. 3). All tube samples were wax-sealed immediately upon recovery and stored with the borehole samples until laboratory tested. In addition to the tube sampling and plate testing in the trench, block samples approximately 12 by 12 by 8 in. were carefully taken at each bench elevation. These were sealed with alternate layers of wax and cheesecloth immediately upon recovery.

Triaxial Testing

In addition to routine index tests performed on all samples recovered, a special cyclic triaxial testing program consisting of both unconsolidated undrained and consolidated undrained tests ($\overline{Q}$ and $\overline{R}$ tests) with pore water pressures measured at the base of the sample using pressure transducers was carried out. Back pressures varying from 40 to 70 psi were used for all tests. The cyclic loading procedure consisted of loading the undisturbed specimens to ⅓ the maximum expected principal stress difference and unloading to zero load. This was repeated three times, the next reloading cycle was taken to a principal stress difference of about 50 percent of the estimated maximum and two unload-reload cycles were carried out. This repetitive procedure was also carried out at about 75 percent of estimated failure stress and at failure stress.

Strain Rate

All compression tests were carried out as strain-controlled tests, and the rate of strain used varied from about 2.5 percent to 4.5 percent per hour. Ladd (6) points out that as much as a threefold variation in modulus has been reported when strain rates vary from 0.12 percent per hour to 60 percent per hour. A series of tests carried out on undrained consolidated specimens using remolded and undisturbed material (Fig. 6) showed

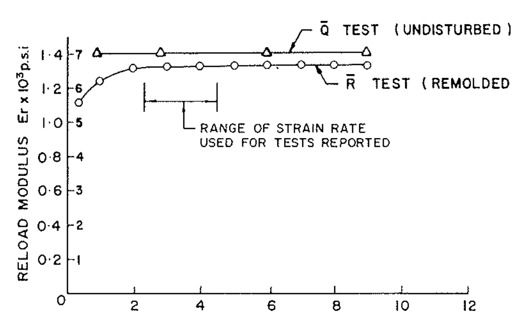

Figure 6. Variation in reload modulus with rate of strain.

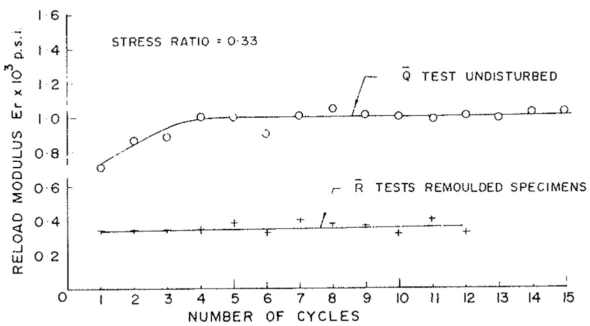

Figure 7. Variation in reload modulus with number of cycles.

that the value of reload modulus (E_r) was sensibly constant when the strain rate varied from about 0.5 percent per hour to 13 percent per hour. A similar insignificant sensitivity to strain rate was reported for the Sunnybrook till, Toronto (2).

Number of Load Cycles

Use of the recompression curves in the stress-strain diagram to define the modulus of elasticity has been discussed by a number of investigators (2, 6, 7, 10, 11). In addition to eliminating initial seating errors in the test setup, the use of the recompression curve has the advantage of causing fissures and partings in undisturbed samples of overconsolidated clays to close, thereby minimizing deformation that is not true strain (10). In order to determine the influence of a number of cycles on reload modulus, a series of $\overline{Q}$ and $\overline{R}$ tests was carried out on both undisturbed and remolded specimens with cycles varying from 1 to 15. The data obtained are shown in Figure 7. Three cycles were used for most of the laboratory and all of the plate load tests reported. Crawford and Burn (2) found that about 20 cycles were required to result in constant value of E_r, whereas Lo (7) found that recompression curves were parallel for all cycles up to two-thirds of the peak stress in each cycle.

Percentage Unloading

Wilson and Dietrich (11) have defined a static modulus of elasticity (E_{static}) obtained from repetitive load tests during which small on-off load increments were used. This procedure eliminates creep effects but has the disadvantage that a stiffening effect results when successively smaller off-on loads are used. This effect is demonstrated in Figure 8, which shows that the reload modulus obtained from a 25 percent unloading is about twice the value obtained from 100 percent unloading. Undoubtedly the modulus defined by Wilson and Dietrich is the modulus that should be used in some practical problems, but it does not apply directly to the problem of immediate deformation under a highway embankment loading.

Modulus From Plate Load Tests

The three most commonly used moduli for applied static loadings are shown in Figure 1. The reload modulus determined after two to three cycles of repetitive loadings at a specific stress ratio is the one chosen for this comparative study to demonstrate the influence of sample disturbance on the modulus value. The use of in situ plate load tests to evaluate the modulus of elasticity has been discussed by Burmister (1) and the method suggested in his discussion has been adopted. The Bousinesq equation for a rigid circular bearing plate with a uniformly distributed contact pressure has been used. This equation is as follows:

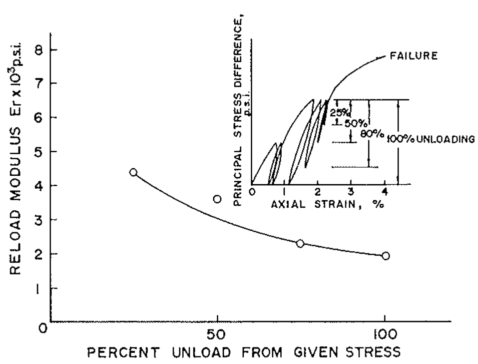

Figure 8. Variation in reload modulus with percent unload.

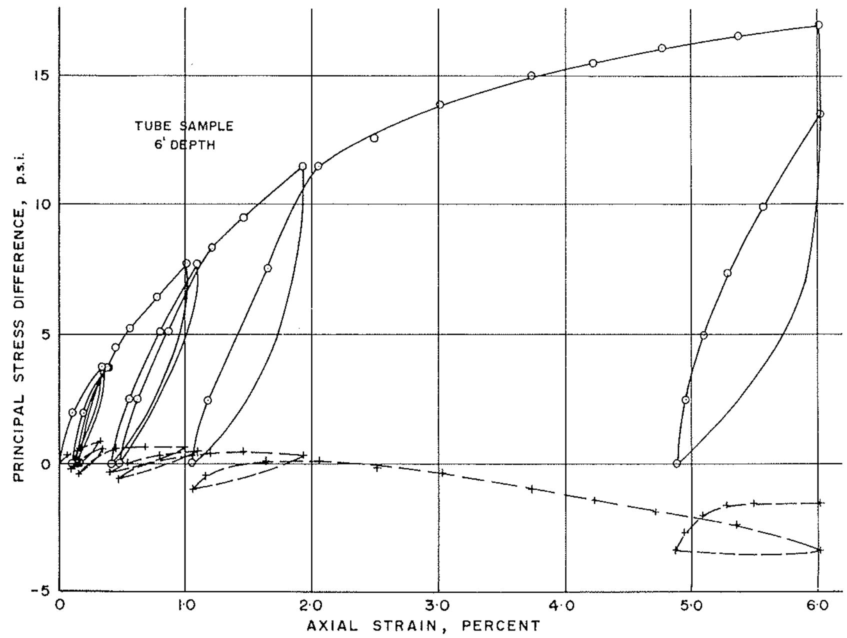

Figure 9. Stress-strain-pore water pressure relationship $\overline{Q}$ test.

$$\Delta = \frac{\pi}{2} \frac{(1 - \mu^2)}{E} pr$$

where μ = 0.5, p = contact pressure, r = plate radius, Δ = deflection, and E = modulus of elasticity.

In order to develop a stress-strain curve as shown in Figure 3, the depth of soil below the loading plate assumed to be deformed by the surface plate load has been taken as two plate diameters (Fig. 5). The principal stress difference is determined from the elastic equation using the surface contact pressure and a restraint factor as discussed by Burmister (1).

The small diameter plate has been used because (a) the small plate minimized equipment weight; (b) the variation in modulus with depth in the crust will have less influence for small plates than larger plates; and (c) the length of laboratory samples tested varied from 4 to 8 in., which is reasonably close to the depth of soil influenced by 6-in. diameter plate. Pore water pressures were measured at a point in the zone of estimated maximum shear stress below the loaded plate using a porous insert and a pressure transducer. The stress-strain-pore water pressure curves from a cyclic triaxial laboratory test on an undisturbed specimen and an in situ cyclic plate load test are compared in Figures 3 and 9. The pattern of pore water pressure behavior is remarkably similar in both tests.

RESULTS

The results obtained from the first stage of the investigation using undisturbed borehole samples are shown in Figure 10. This figure shows that

1. The reload modulus E_r was found to be about 50 percent greater than the secant modulus for stress ratios varying from 0.2 to 0.7 (Fig. 10).
2. The reload modulus as determined from 3-in. diameter samples was consistently greater than values determined from 2-in. diameter samples. At low stress ratios the modulus for 2-in. undisturbed samples was only about 20 percent larger than that found

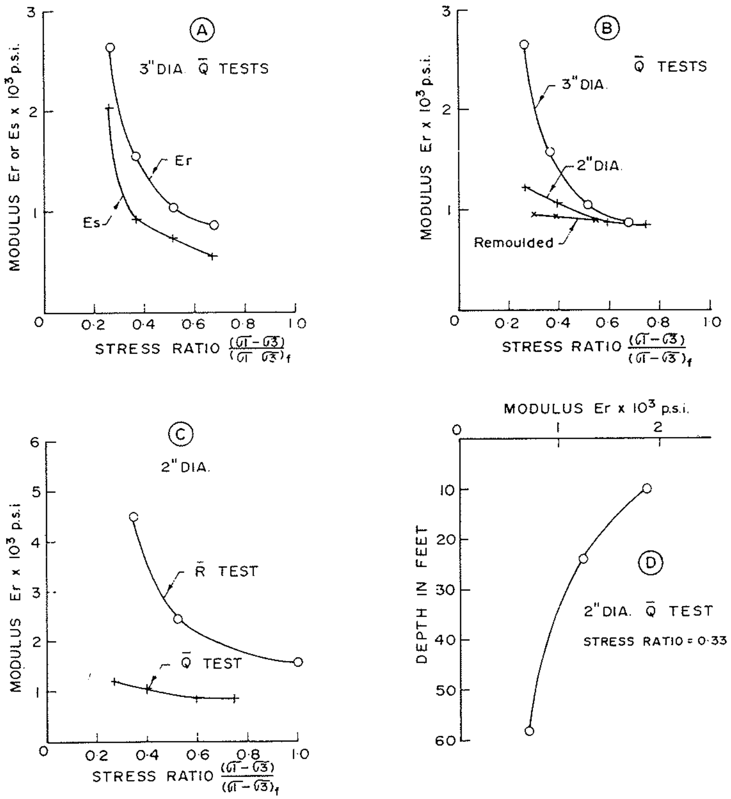

Figure 10. Influence of sample size and test type on moduli shown using borehole samples [$(\sigma_1 - \sigma_3)$ = applied principal stress difference; $(\sigma_1 - \sigma_3)$ f = principal stress difference at failure].

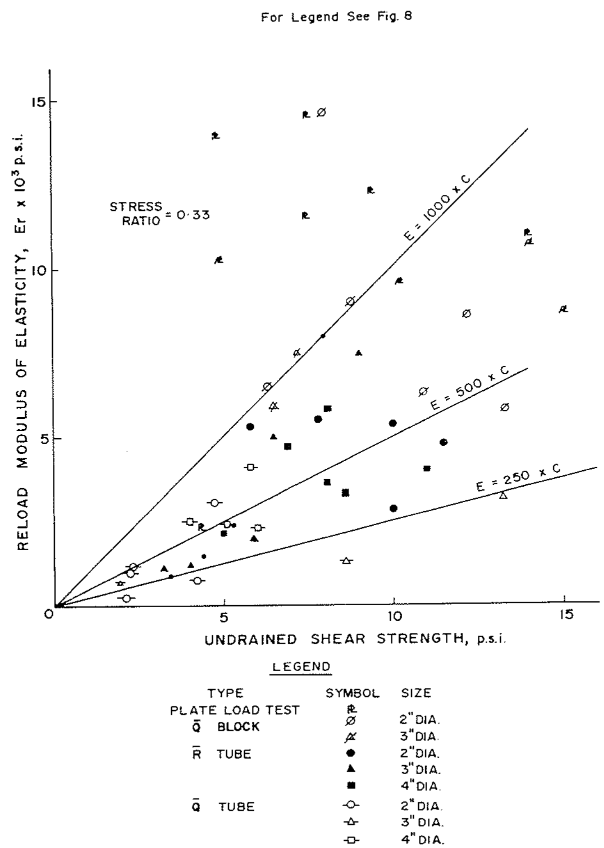

Figure 11. Strength vs modulus relationship.

from tests on remolded specimens; at high stress levels all results were similar (Fig. 7).

3. For the same size undisturbed samples the reload modulus as determined from consolidated undrained tests (R tests) varied from 2 to 4 times the value obtained from Q tests (Fig. 8).

4. The reload modulus was found to decrease markedly with depth through the crust zone and in a manner similar to the decrease in strength with depth. This is presumably related to desiccation (Fig. 6).

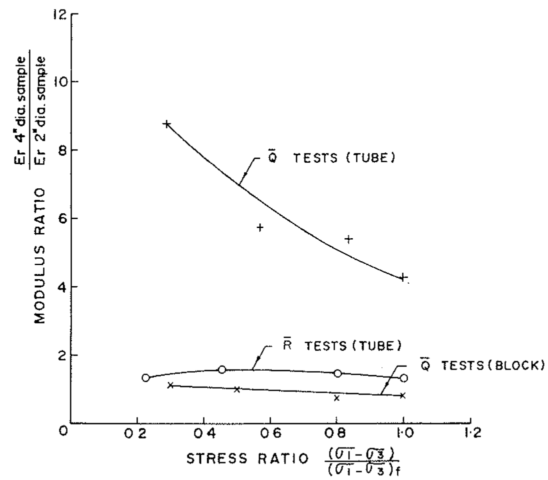

Figure 12. Influence of sample diameter and test type on reload modulus.

An attempt was made to determine the E_r/S_u ratio but no order was found in the data at hand (Fig. 11). These points are not new and similar patterns have been reported by others (4, 6).

The results of the second stage of the investigation are shown in Figures 12 and 13, which indicate that the influence of sample disturbance vs stress ratio for any one test type is shown by the ratio of reload modulus for 4-in. and 2-in. samples:

5. For $\bar{Q}$ tests on tube samples this modular ratio, $\frac{\text{Er-4}}{\text{Er-2}}$, varies by 100 percent for the stress ratio range of 0.25 to 1.0.

6. For $\bar{R}$ tests, which are recommended by Ladd (6), on tube samples the modular ratio varies from 1.5 to 2.0 for a stress ratio range of 0.25 to 1.0.

7. For $\bar{Q}$ tests on specimens cut from block samples the modular ratio was only slightly above 1 for a stress ratio range of 0.25 to 1.0.

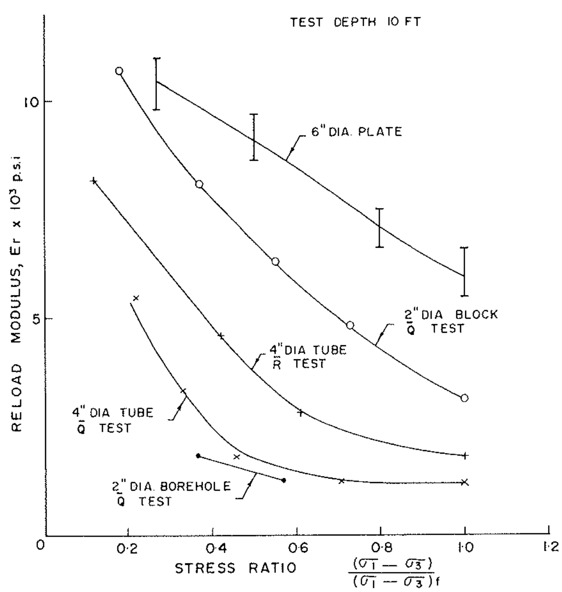

Figure 13. Reload modulus vs stress ratio for sample size and test type shown.

The significant findings of the complete study have been summarized in Figure 13. It was found that

8. All types of samples, regardless of test type, show a decrease in modulus with increase in stress ratio. This decrease is about 50 percent for all tests when the stress ratio varies from about 20 to 100 percent.

9. The largest value of reload modulus was obtained from in situ plate bearing tests for all stress ratios.

10. The plate loading tests gave values of reload modulus generally twice the value obtained from $\bar{R}$ tests on 4-in. diameter tube samples.

11. The $\bar{Q}$ tests on specimens trimmed from block samples gave reload modulus values nearest those obtained from plate loading tests.

The variation in modulus vs depth for the crust zone is shown in

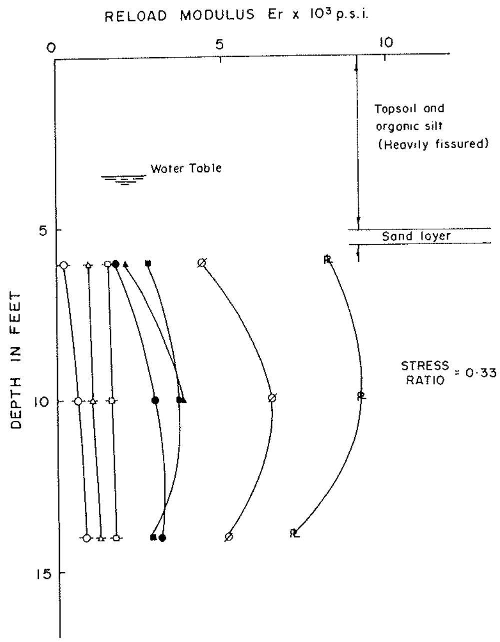

Figure 14. Reload modulus vs depth (see Fig. 11 for legend).

Figure 14. This shows clearly the wide discrepancy between moduli determined from laboratory tests on undisturbed samples and moduli determined from in situ plate bearing tests.

CONCLUSIONS

1. A decrease in the value of reloading modulus (E_r) with increase in stress ratio occurs regardless of test type and sample type. For Tilbury clay till this decrease was found to be about 50 percent when stress ratio was increased from 20 percent to 100 percent. This decrease in modulus is believed to be due to a breakdown of the soil structure as the specimen develops large strains near the failure stress ratio of unity.

2. The comparative tests on samples subjected to various degrees of sampling disturbance show clearly that carefully trimmed specimens from block samples subjected to unconsolidated undrained triaxial tests ($\overline{Q}$ tests) gave values of E_r nearest to values obtained from in situ cyclic plate load tests. The E_r values from consolidated undrained tests ($\overline{R}$ tests) on tube samples were about 50 percent of the values found from cyclic plate loading tests.

3. The presentation of data obtained in terms of undrained shear strength vs reload modulus or any other modulus of elasticity commonly used did not show an acceptable correlation between modulus and strength. In general the value of E_r/S_u was markedly higher for plate test data than for laboratory test data.

4. The pore water pressure developed in the zone of maximum shear stress below the bearing plate during the in situ cyclic loading exhibited a behavior pattern similar to the water pressures measured during cyclic triaxial tests on tube samples in the laboratory.

ACKNOWLEDGMENTS

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