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Laboratory and field determination of preconsolidation pressures at Gloucester

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In recent literature several special test methods have been proposed to measure the preconsolidation pressure of a compressible clay soil. Five methods, in addition to the conventional oedometer test, were applied to the marine clays from the Gloucester test site. The preconsolidation pressures measured using these laboratory tests were compared with that mobilized *in situ* below the centre of a test embankment. The investigation showed that the preconsolidation pressure is directly related to the rate of strain and that special techniques such as constant rate of strain, controlled gradient, single-stage loading, and anisotropic triaxial consolidation tend to overestimate the *in-situ* preconsolidation pressure. The conventional oedometer test using a load increment ratio of 0.5 and a reloading schedule of 24 h applied to good-quality undisturbed samples produced preconsolidation pressures that compared best with the *in-situ* values.
Keywords: preconsolidation pressure, laboratory, *in situ*, strain rate effects, disturbance.

De nouvelles techniques expérimentales de laboratoire ont été récemment proposées pour déterminer la pression de préconsolidation des sols argileux compressibles. Sur les argiles marines du site d'essai de Gloucester, cinq de ces techniques, en plus de l'essai oedométrique conventionnel, ont été utilisées; les pressions de préconsolidation ainsi obtenues ont pu être comparées à celle qui est effectivement mobilisée *in situ* sous le centre du remblai d'essai. L'étude montre que la pression de préconsolidation est directement reliée à la vitesse de déformation du sol et que les essais oedométriques spéciaux à vitesse de déformation constante à gradient contrôlé, et à chargement unique ont tendance à surestimer la pression de préconsolidation *in situ*. Ce sont les résultats obtenus par essais conventionnels, avec un taux d'accroissement de charge de 0,5 et une durée d'application de la charge de 24 h, qui sont les plus proches des valeurs obtenues *in situ*.
Mots-clés: pression de préconsolidation, laboratoire, *in situ*, vitesse de déformation, remaniement.

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Introduction

Casagrande (1932, 1936) defined preconsolidation pressure, σ_p' as the maximum past vertical effective stress applied to the soil and correlated it to the change in slope of a curve of void ratio versus logarithm of the vertical effective stress σ_v' obtained from a one-dimensional consolidation test.

It became evident at the beginning of the sixties that σ_p' estimated with Casagrande's method was higher in some clays than the maximum vertical effective stress applied during its geological history. This discrepancy was attributed to a number of factors, including thixotropic effects, long-term secondary compression, and cementation effects (Leonards 1962; Bjerrum 1967). The term "preconsolidation pressure" is definitely misused in such cases. Although the exact origin of σ_p' is difficult to establish, the term has been extended to define the break of the $e - \log \sigma_v'$ curve. From a practical viewpoint engineers are interested in this threshold point beyond which important plastic deformations take place, particularly in sensitive clays where the normally consolidated branch of the compression curve is very steep.

The conventional test method for measuring σ_p' uses a consolidometer in which vertical loads are applied in steps such that each load increment equals the previous load (load-increment ratio of 1) with a load-increment duration of 24 h. This technique presents some disadvantages.

(1) In the $e - \log \sigma_v'$ diagram, the experimental points are widely spaced, making it difficult to draw the test curve and estimate the preconsolidation pressure.

(2) When the sample is loaded every 24 h the amount of secondary compression will vary for each load increment and soil sample.

(3) One to two weeks are required to complete the test and consequently the test is expensive.

To overcome some of these disadvantages, modifications to the conventional test were suggested (for example, Leonards 1962; Bjerrum 1973). In numerous laboratories in eastern Canada, a load increment ratio $\Delta P/P$ equal to 0.5 is used. Other techniques have also

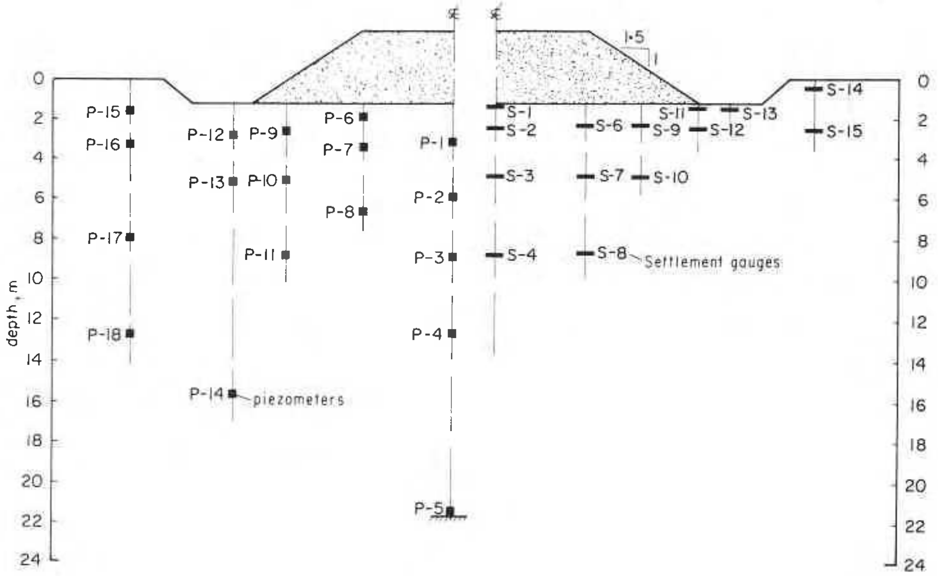

FIG. 1. Location of settlement gauges and piezometers—Gloucester test embankment.

been suggested such as: (1) the constant-rate-of-strain oedometer test (Hamilton and Crawford 1969; Crawford 1965; Smith and Wahls 1969; Wissa *et al.* 1971; Sällfors 1975), (2) the controlled-gradient oedometer test (Lowe *et al.* 1969), (3) the anisotropic triaxial consolidation test (Tavenas and Leroueil 1977), and (4) the single-stage-loading oedometer test (Leroueil *et al.* 1980).

In previous studies (Crawford 1964, 1965; Sällfors 1975) and in the experience of the authors, the preconsolidation pressures obtained from the above procedures were generally different from those measured in conventional tests. The problem was to determine which test procedure provided the best σ_p' for use in foundation design. Consequently a research program was undertaken (Samson *et al.* 1981) to compare σ_p' measured using all known test procedures with σ_p' mobilized in soft clay under a granular test embankment at the Gloucester site (Bozozuk and Leonards 1972).

Test site

In the fall of 1967 the Division of Building Research, National Research Council of Canada, constructed and instrumented a test fill at Gloucester, 21 km south of Ottawa. The embankment was constructed in an excavation 1.3 m deep which removed the desiccated crust from the site. The engineering performance during construction was described by Bozozuk and Leonards (1972) and the long-term settlement was analyzed by Lo *et al.* (1976). The geometry of the embankment and location of piezometers and settlement gauges are shown in Fig. 1.

Subsoil conditions

In the fall of 1979 three bore holes were drilled 2 m apart, about 30 m from the embankment by means of a modified 70 mm diameter Geonor piston sampler with an area ratio of 11% and inside clearance of 0.9%. A Nilcon vane sounding was also taken near the bore holes. In addition, a large-diameter (30 cm) block sample was taken at a depth of 4.1 m with the Sherbrooke sampler (Lefebvre and Poulin 1979). All samples were stored in a humid room at 10°C.

The subsurface conditions 30 m from the embankment are similar to those found by Bozozuk and Leonards (1972), as shown in Fig. 2. The soil profile consists of a thin layer of topsoil about 0.3–0.4 m thick overlying a layer of fine sand and silt 0.6–1.0 m thick. This formation is underlain by a deep deposit of highly sensitive silty clay with medium to high plasticity that can be subdivided into five strata. From 1.2 to 2.4 m the clay is grey-brown, oxidized, and stratified by thin seams of silt. It is stiff in the upper part and soft in the lower part of the layer with an undrained shear strength as low as 15 kPa. From 2.4 to 5.4 m the clay is soft, grey, and silty with an undrained shear strength varying from 14 to 22 kPa. At this level the preconsolidation pressure increases slightly from 55 to 65 kPa. From 5.4 to 7.0 m the clay is uniform with a decreasing undrained shear strength from 29 to 24 kPa. From 7.0 to 13.8 m the undrained shear strength and the preconsolidation pressure of the clay increase almost linearly with depth from 24 to 43 kPa and from 85 to 135 kPa respectively. From 13.8 to 18.3 m there is a layer of grey silty clay which is underlain with varved clay and glacial till overlying the bedrock.

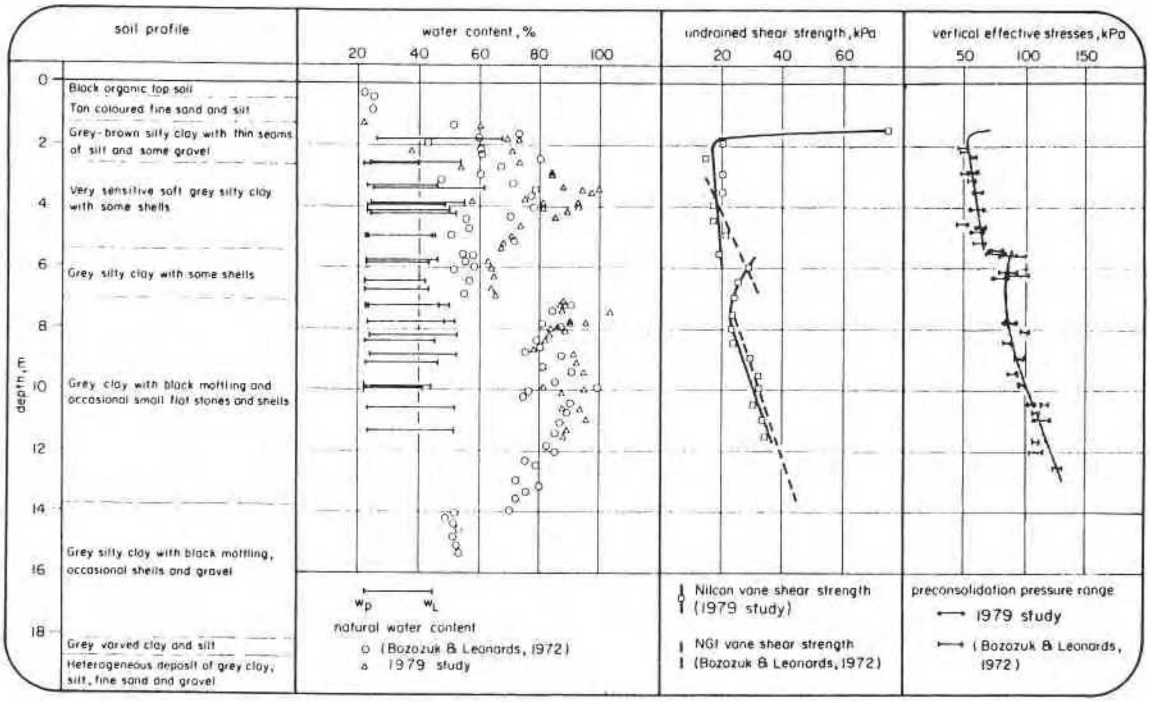

FIG. 2. Soil profile—composite of test data.

Eight reference piezometers were installed about 30 m northeast of the embankment to measure the natural pore water conditions in the ground. They showed that the water table rises to the ground surface in the spring and may drop to a depth of up to 2.0 m by the end of the summer.

Preconsolidation pressure

Figure 3 shows the possible range of vertical effective stress outside the embankment area, based on the variation of water levels observed with the reference piezometers. It also depicts the preconsolidation pressure profile obtained with conventional oedometer tests ($\Delta P/P = 0.5$).

The stress increase due to embankment loading was calculated by Lo *et al.* (1976) using a finite element method which took into account the excavation, construction, and stiffness of the embankment. The calculations compared well with measurements at the base of the embankment (Bozozuk and Leonards 1972). Depending on the position of the water table, the final vertical effective stress under the embankment should be located in the hatched area in Fig. 3.

The clay under the centre of the fill would become normally consolidated up to a depth of 5.4 m since the range of final effective stress exceeds the preconsolidation pressure profile determined from standard laboratory consolidation tests. At depths of 5.4–11 m the possible final effective stresses straddle the

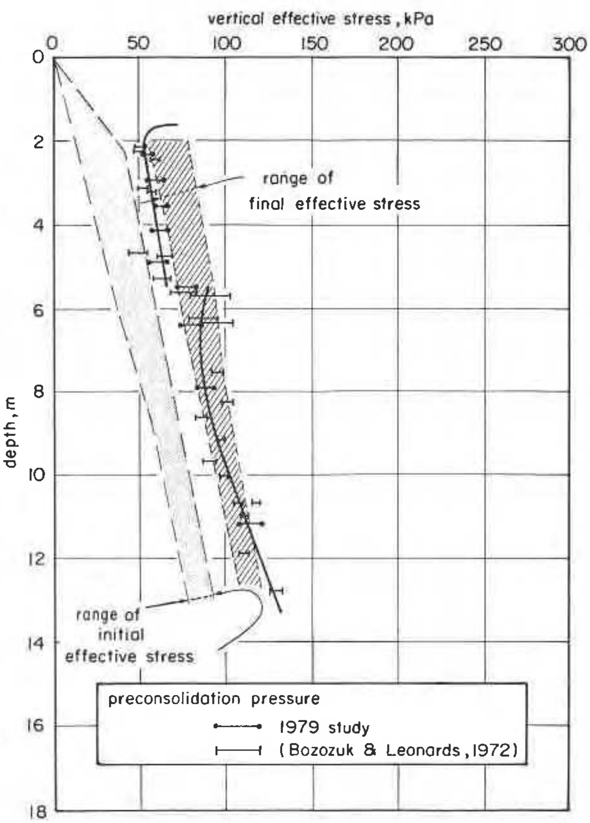

FIG. 3. Expected range of final vertical effective stress.

preconsolidation pressure profile, making it difficult to predict whether the clay will become normally consolidated. This case history is therefore ideal for comparing the preconsolidation pressure prevailing *in situ* with that measured in the laboratory.

Laboratory preconsolidation pressure of the clay

Conventional oedometer tests

Ten tests were performed on samples taken with the 70 mm diameter modified Geonor piston sampler. The test specimens, 12.7 mm high with a diameter of 64 mm, were loaded in steps using a load-increment ratio of 0.5 which was maintained for 24 h. The clay is sensitive and the preconsolidation pressure is generally well defined in $e - \log \sigma_v'$ and in $\epsilon_v - \sigma_v'$ plots, where $\epsilon_v = \Delta H/H_0$. The preconsolidation pressure was defined by the construction method shown in Fig. 4. The range of σ_p' obtained from these tests is shown in Figs. 2 and 3 together with those reported by Bozozuk and Leonards (1972). The results from both studies are similar.

Special consolidation tests

Three series of tests were performed to study the effect that test technique or method has on the measured laboratory preconsolidation pressure. Two were performed on samples taken with the 70 mm diameter modified Geonor piston sampler from depths of 3.4–3.9 m and 7.4–7.7 m. The third was performed on samples taken from a depth of 4.08–4.15 m using the 30 cm diameter Sherbrooke sampler.

The special oedometer tests (with the exception of the anisotropic triaxial consolidation test) were carried out in two identical, specially designed oedometer cells identified as "TUL" (Samson *et al.* 1981) in which a back pressure of 100 kPa was applied at the top end and the excess pore pressure was measured at the base of the specimen. The samples were 15 mm high and 55 mm in diameter.

The average strain or void ratio measured with time in constant-rate-of-strain and controlled-gradient consolidation tests was related to the average calculated effective stress σ_v' using the equation suggested by Smith and Wahls (1969) and Wissa *et al.* (1971):

$$\sigma_v' = \sigma_v - u_0 - \tfrac{2}{3}(u_b - u_0)$$

where σ_v = total vertical applied stress, u_0 = applied back pressure, and u_b = pore pressure measured at the base of the specimen.

The range of preconsolidation pressure in all special tests except those for single-stage loading was determined in the same way as for the conventional tests (Fig. 4). It should be noted that the preconsolidation pressure can also be obtained from an $\epsilon_v - \sigma_v'$ plot, which was used here to provide a clearer and more realistic representation of the behaviour of the clay.

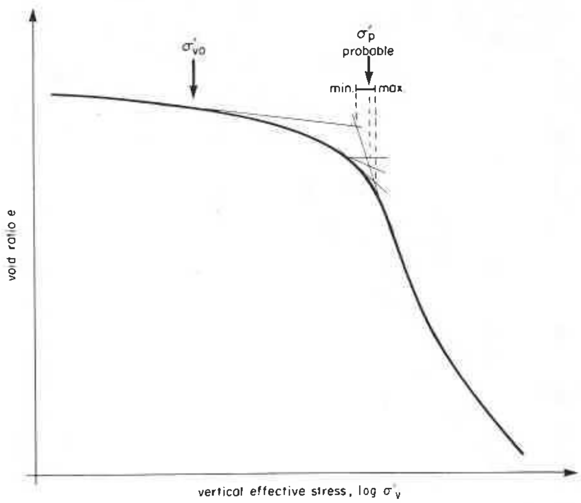

FIG. 4. Determination of range of preconsolidation pressure.

The special tests used to determine the preconsolidation pressure are as follows.

(a) Constant-rate-of-strain (CRS) tests

In these tests reported in Table 1, the strain rates varied from 4×10^{-3} to 5.1×10^{-1}%/min. The slower rate corresponds to the lower limit obtainable with the test apparatus.

The stress–strain curves obtained at different strain rates on samples taken from 3.45 to 3.90 m and 4.08 to 4.15 m are typical of the behaviour generally observed in sensitive clays (Figs. 5 and 6). The compressibility of the soil is very small at stresses below the preconsolidation pressure (in the overconsolidated range), increases considerably when stressed in the normally consolidated range just beyond the preconsolidation pressure, then decreases with greater stresses. The intersection of some curves was probably due to the unavoidable natural variability of the samples. The following general observations were made.

1. It was not possible to detect a strain rate effect on the compressibility of the soil in the overconsolidated range. This seems to contradict the results obtained on a Swedish clay by Sällfors (1975), who found that compressibility increased with decreasing strain rates. It is possible that the strain rate effects of the Gloucester clay specimens were masked by their natural variability.

2. The strain rate effect was more evident in the normally consolidated range. At a given strain, the vertical effective stress increased directly with the corresponding rate of strain $\dot{\epsilon}_v$. This is consistent with observations made by Crawford (1965), Sällfors (1975), and Vaid *et al.* (1979) on clays from Ottawa, Bäckebol, and St-Jean-Vianney respectively.

3. The preconsolidation pressure was considerably

TABLE 1. Summary of preconsolidation pressures measured in clay from the Gloucester test site using conventional and special test methods

	Preconsolidation pressure, σ_p', kPa		
Details of tests	Depth: 3.45–3.90 m; core size: 70 mm diam.	Depth: 4.08–4.15 m; core size: 30 cm diam.	Depth: 7.45–7.65 m; core size: 70 mm diam.
Conventional, $\Delta P/P = 0.5$	62– 68	63– 72	81– 91
CRS 4.0×10^{-3}%/min	71– 74	80– 84	96–102
1.2×10^{-2}%/min	75– 77		
2.7×10^{-2}%/min	82– 85	88– 90	
6.0×10^{-2}%/min	81– 86		
1.3×10^{-1}%/min	90– 97	88– 93	
5.1×10^{-1}%/min		97–107	
CGT, $\Delta u_b =$ 3.5 kPa	90– 93	89– 93	108–112
10.0 kPa	89– 92		124–128
15.0 kPa	90– 92		
20.0 kPa		94– 97	
35.0 kPa	102–104		
$(MSL)_p$	72– 80		100–112
$(MSL)_{24}$	63– 72		
SSL		105–110	
ATC	72– 80		105–115

NOTES: CRS: constant-rate-of-strain test; CGT: controlled-gradient test; SSL: single-stage-loading test; ATC: anisotropic triaxial consolidation; $(MSL)_p$: multiple-stage-loading test—end of primary; and $(MSL)_{24}$: multiple-stage-loading test—end of 24 h.

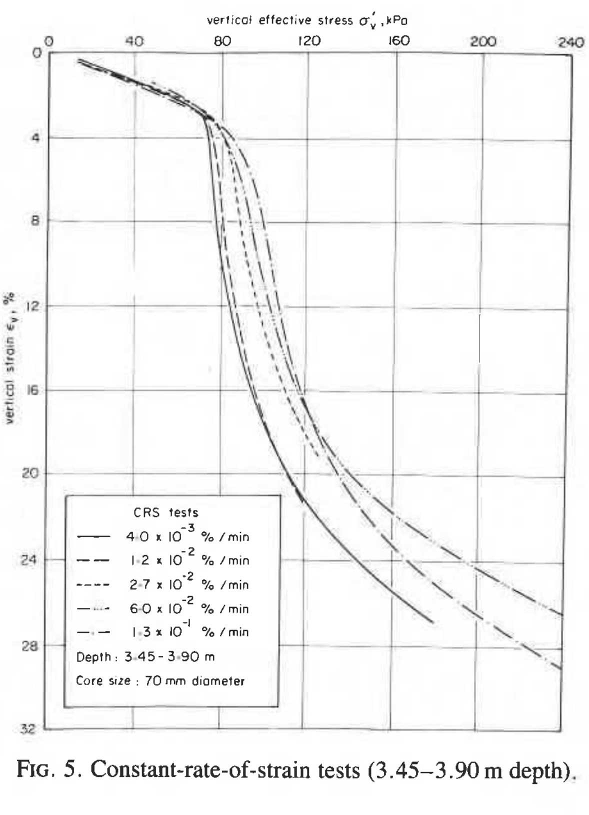

FIG. 5. Constant-rate-of-strain tests (3.45–3.90 m depth).

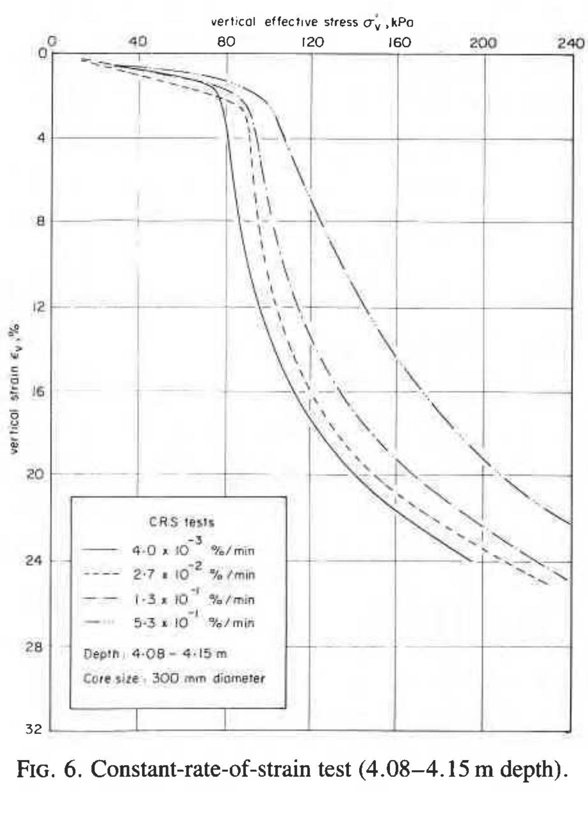

FIG. 6. Constant-rate-of-strain test (4.08–4.15 m depth).

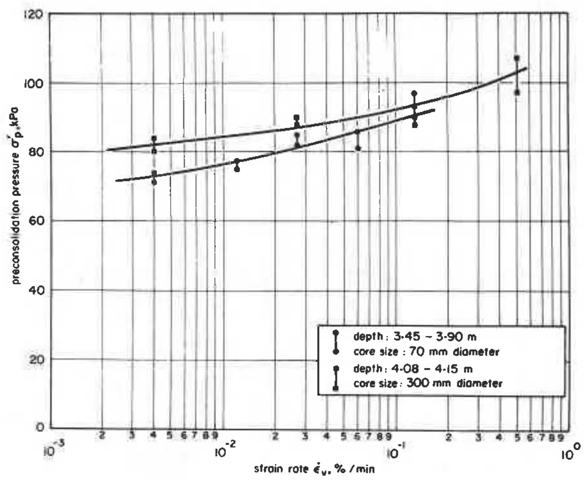

FIG. 7. Variation of preconsolidation pressure with strain rate in CRS tests.

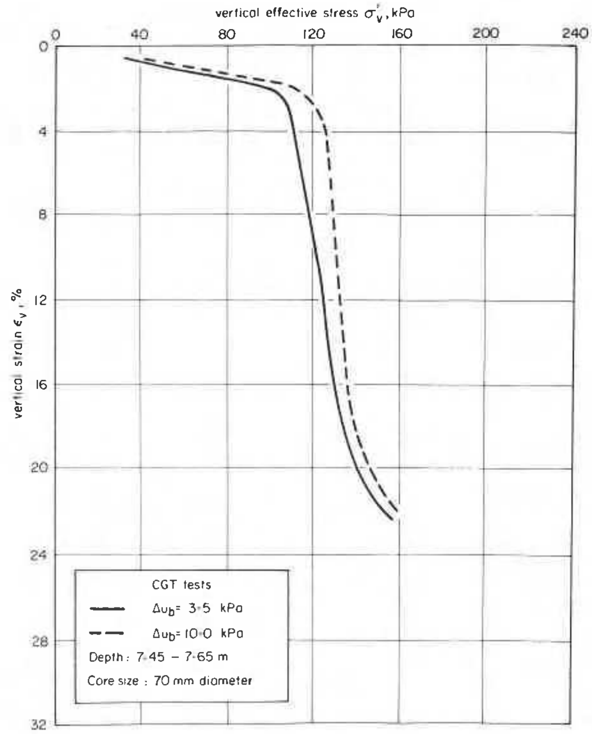

FIG. 8. Controlled-gradient tests (7.45–7.65 m depth).

strain-rate dependent, as can be seen in Fig. 7 in which the σ_p' values are plotted against strain rate $\dot{\epsilon}_v$. Over the range of strain rates (4×10^{-3} to 5×10^{-1}%/min), the variation in preconsolidation pressure was about 12% per logarithm cycle; nevertheless, the preconsolidation pressures obtained from the tests (71–107 kPa) were significantly greater than those measured in conventional tests (62–72 kPa) at these depths (Table 1).

(b) Controlled-gradient (CGT) tests

These tests were performed in the TUL cell with the confining piston carefully positioned on top of the sample. The pressure in the fluid above the piston was increased until the pore water pressure at the base reached a predetermined value related to the desired gradient. During consolidation the pressure difference between the top and bottom of the sample was kept constant by means of a servo-control mechanism. The cell pressure and piston displacement were recorded continuously during the test.

The pore pressure differences used in the tests $\Delta u_b = u_b - u_0$ ranged between 3.5 and 35 kPa. The results are given in Table 1.

Figure 8 shows the stress–strain test curves obtained from samples at depths of 7.45–7.65 m using pore pressure differences of 3.5 and 10 kPa. The preconsolidation pressure is well defined for both curves. The test values for σ_p' are about 110 kPa ($\Delta u_b = 3.5$ kPa) and 125 kPa ($\Delta u_b = 10$ kPa); this indicates that the preconsolidation pressures increase directly with the hydraulic gradient. These values are also considerably greater than those obtained from the conventional tests which are in the 81–91 kPa range at this depth (Table 1).

The preconsolidation pressures plotted versus Δu_b in Fig. 9 show that the gradient effect is less pronounced at depths of 3.45–3.90 m and 4.08–4.15 m. The preconsolidation pressure rises slightly, however, as the gradient increases, and again the observed test values of σ_p' (89–104 kPa) are greater than those measured with conventional tests (62–72 kPa) at this depth (Table 1).

The test with $\Delta u_b = 10$ kPa (Fig. 8) is replotted in Fig. 10 to show the average vertical effective stress and the average strain rate with the logarithm of time. The results are typical of generally observed behaviour. The vertical effective stress increases rapidly at the beginning of the test, shows a well-defined step as the soil passes its preconsolidation pressure, and then increases again. The strain rate is fairly constant at the beginning, increases to reach a maximum when the clay reaches the preconsolidation pressure, and then lessens as the length of time increases.

(c) Multiple-stage-loading (MSL) tests

Apart from the fact that drainage is permitted only at the top and that pore pressure is measured at the bottom, these tests are similar to conventional tests. The duration of loading was either the time required for primary consolidation to occur $(MSL)_p$ or 24 h $(MSL)_{24}$.

The tests were performed in the TUL cell with a stress increment ratio of 0.5, a back pressure of 100 kPa, and drainage at the top. Figure 11 shows the stress–strain curves for the samples taken at a depth of 3.45–3.90 m. The $(MSL)_p$ test was reloaded after the excess pore

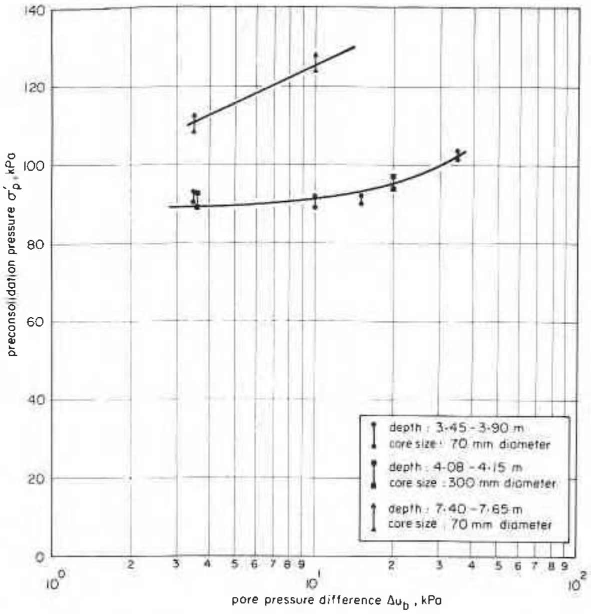

FIG. 9. Variation of preconsolidation pressure with pore pressure difference in CGT tests.

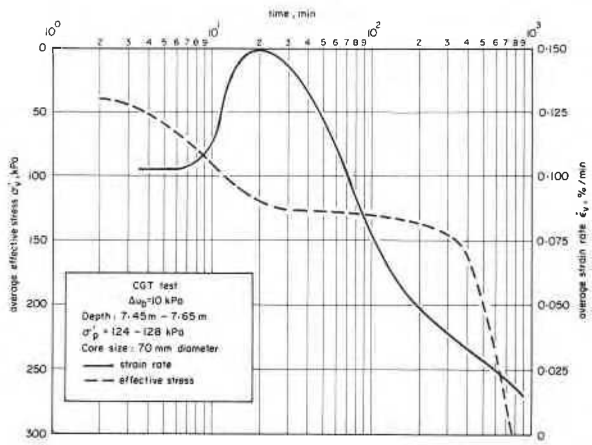

FIG. 10. Average effective stress and average strain rate in a CGT test.

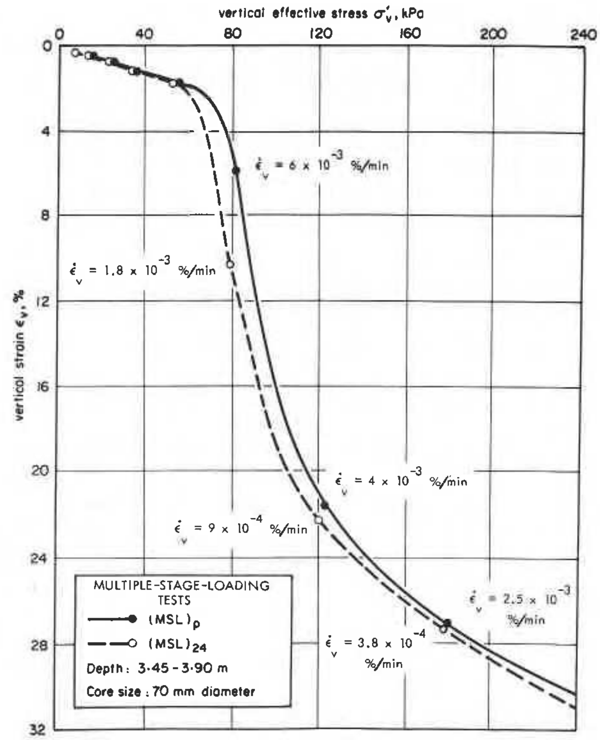

FIG. 11. Multiple-stage-loading tests (3.45–3.90 m depth).

pressure had fully dissipated. The dissipation took 8–30 min in the overconsolidated range and 6–9 h in the normally consolidated range. The $(MSL)_{24}$ test was reloaded every 24 h.

The results for both tests are similar to those reported in the literature. Crawford (1964), for example, showed a greater accumulated strain after 24 h than at the end of primary consolidation. These results are due to additional secondary deformations occurring between the end of primary and 24 h. Consequently, the σ_p' of 72–80 kPa obtained from $(MSL)_p$ is greater than the 63–72 kPa measured with $(MSL)_{24}$ tests. The latter values are comparable with the 62–68 kPa obtained from conventional tests, whereas the $(MSL)_p$ values are larger (Table 1).

(d) Single-stage-loading (SSL) tests

In this test a unique load corresponding to 1.5–3 times the anticipated preconsolidation pressure is applied to the test specimen. The pore pressure is recorded at the base of the specimen during the entire consolidation process and the vertical effective stress is plotted concurrently against the logarithm of time. According to Leroueil *et al.* (1980) the resulting curve develops a step at an effective stress equal to the preconsolidation pressure.

One test carried out on a sample taken at a depth of 4.08–4.15 m with the 30 cm diameter sampler is shown in Fig. 12. An initial load equivalent to a vertical stress of 200 kPa was applied instantaneously. After 40 min, when the entire sample was normally consolidated, the vertical total stress was increased by 37 kPa. This increase modified the behaviour of the soil at the end of the test without affecting the measured preconsolidation pressure.

The figure shows the average strain ϵ_v (%) and vertical effective stress σ_{vb}' at the base of the specimen versus the logarithm of time. The $\sigma_{vb}' - \log t$ curve developed a well-defined step 5–10 min after loading, at an effective stress of 105–110 kPa. This stress range, which corresponds to the preconsolidation pressure of

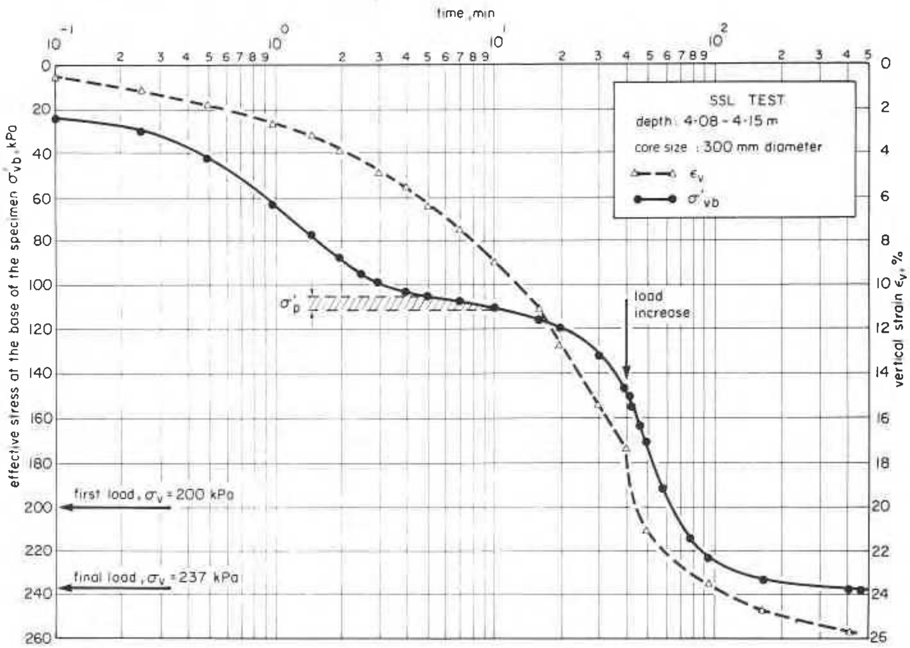

FIG. 12. Single-stage-loading test (4.08–4.15 m depth).

the clay specimen (Leroueil *et al.* 1980), is significantly higher than the 63–72 kPa obtained in conventional tests (Table 1).

(e) Anisotropic-triaxial-consolidation (ATC) tests

The consolidation pressures were applied in increments, each having a duration of 24 h. A stress ratio of $\sigma_h'/\sigma_v' = 0.5$ was used which is similar to the coefficient of earth pressure at rest of normally consolidated clays with a friction angle of about 30°. Yielding under these conditions is obtained at a vertical effective stress equal to the preconsolidation pressure (Tavenas and Leroueil 1977) and is determined at the break in the $e - \log \sigma_v'$ or $\epsilon_v - \sigma_v'$ curve, similar to the method applied to oedometer tests.

The anisotropic triaxial tests were performed in conventional triaxial cells on specimens 76 mm high and 38 mm in diameter. Drainage was permitted at both ends and radially to the circumference of the sample.

Two tests were carried out. Figure 13 shows the vertical effective stress–strain curve for the specimen from a depth of 3.45–3.90 m. The preconsolidation pressure is also relatively well defined and has a value between 72 and 80 kPa. At 7.45–7.65 m, σ_p' was 105–115 kPa. Both values are about 20% higher than those measured with conventional tests (Table 1).

Relation between preconsolidation pressure and strain rate

The above tests show that an intact sample of clay does not have a unique preconsolidation pressure, but, on the contrary, may have as many values as testing techniques. In view of the tendency to replace the conventional oedometer test by other generally faster test methods, it is important to realize that the σ_p' values obtained using the new methods may be different. In general, they are higher than those produced in the conventional test.

This observation gives rise to two questions: (1) Why does the clay behave in this manner? (2) Which σ_p' value should be considered when designing foundations? (The second question will be answered in the following section in which σ_p' measured in the laboratory is compared with σ_p' mobilized *in-situ*.)

To answer the first question, consider the strain rate $\dot{\epsilon}_v$ during the test. The CRS tests (Fig. 7) have effectively shown that a relationship exists between the strain rate and the preconsolidation pressure of the clay. In the CGT tests (Fig. 10) the strain rate $\dot{\epsilon}_v$ varies; however, it is possible to associate σ_p' with the strain rate at the moment when the soil passes its preconsolidation pressure. The corresponding values for $\sigma_p' - \dot{\epsilon}_v$ are shown in Fig. 14 together with the CRS test results. Although there is some scatter, probably due to the natural variability of the samples, a good correlation can be found between both series of test results.

Although the analysis for the multiple-stage-loading tests $(MSL)_p$ and $(MSL)_{24}$ is not straightforward due to the fact that $\dot{\epsilon}_v$ varies continuously during each loading step (the stress–strain curves shown in Fig. 11 are based only on the deformation measured at the end of the loading periods), it is nevertheless possible to determine σ_p'. The latter is highly dependent on the normally

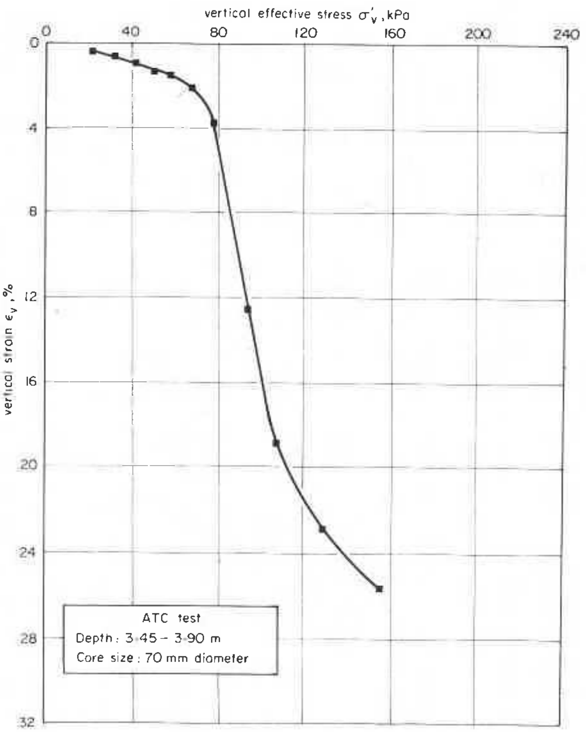

FIG. 13. Anisotropic-triaxial-consolidation test (3.45–3.90 m depth).

consolidated branch of the stress–strain curve, especially on the two or three test points just exceeding the sharp break in the curve; consequently, it must be associated with the strain rate range observed at the end of the loading periods at these points (Fig. 11). The corresponding values for $\sigma_p' - \dot{\epsilon}_v$ are also shown in Fig. 4. These results correlate well with those from CRS and CGT tests at the three depths, thus showing that a unique relationship exists between the preconsolidation pressure measured in the laboratory and the strain rate. In fact the strain rate alone explains the differences in σ_p' observed with the different test methods. Among the special tests, multiple-stage-loading tests have the lowest strain rates and consequently give the lowest σ_p' values.

In the SSL test, the sudden application of a high initial stress (200 kPa) most probably creates a state of strain that varies from point to point in the test specimen. Consequently, the preconsolidation pressure defined locally at the base of the specimen cannot be correlated with the average strain or be compared directly with the results in Fig. 14. The strain rates are very high at the beginning of the test (the average strain rate is 0.5%/min when the preconsolidation pressure is measured at the base of the specimen), which implies high values of preconsolidation pressure. The 105–110 kPa obtained at a depth of 4.08–4.15 m is therefore comparable to the $\sigma_p' - \dot{\epsilon}_v$ relationship shown by other tests.

The ATC test is different from the oedometer tests. The stress path is different and radial and vertical drainage is allowed. Therefore, σ_p' measured in these tests cannot be compared except on a strictly empirical basis. This test procedure provided values of σ_p' that were approximately 20% higher than those determined using conventional tests (Table 1).

Effect of soil disturbance due to sampling on consolidation curve

The clays used in the two series of tests (from depths of 3.45–3.90 m and 4.08–4.15 m) were sampled by different methods, providing an opportunity to compare the degree of disturbance on the consolidation curve associated with these techniques. One of the major effects of sampling disturbance is to reduce the stiffness of the clay in the overconsolidated range (Bozozuk 1971; La Rochelle *et al.* 1980). This disturbance can be evaluated by considering the accumulated strain at the preconsolidation pressure. These strains varied between 1.5 and 2.3% for samples taken with the 30 cm sampler (Fig. 6), compared with 3 and 4% for those taken with the 70 mm diameter sampler (Fig. 5). It appears that the sampling disturbance was greater with the 70 mm diameter sampler than with the 30 cm diameter sampler, a behaviour similar to that reported for a 75 mm sampler at Yamaska by La Rochelle *et al.* (1980).

The average preconsolidation pressure profiles shown in Figs. 2 and 3 indicate that the difference in σ_p' for both series of tests, considering the difference in depth, would be less than 3 kPa. The $\sigma_p' - \dot{\epsilon}_v$ curves shown in Fig. 14(*a* and *b*) are almost identical, indicating that the difference in degree of disturbance due to the sampling method had an insignificant effect on σ_p' values. However, Bozozuk (1971) found a considerable difference when he compared samples taken on the same site with 54 and 124 mm diameter samplers.

In-situ preconsolidation pressure of the clay

On the basis of conventional oedometer tests, Fig. 3 indicates that the clay under the centre of the embankment should become normally consolidated from a depth of 1.4–5.4 m beneath the original ground surface. At greater depths the final stress range straddles the preconsolidation pressure profile, which makes it difficult to determine whether the clay will become normally consolidated.

The vertical effective stresses under the centre of the embankment were deduced from the vertical total stresses computed by Lo *et al.* (1976) and from the measured pore pressures. It was assumed that the total vertical stress remains constant throughout the entire life of the embankment, which is not quite correct since there are water content variations in the fill as well as

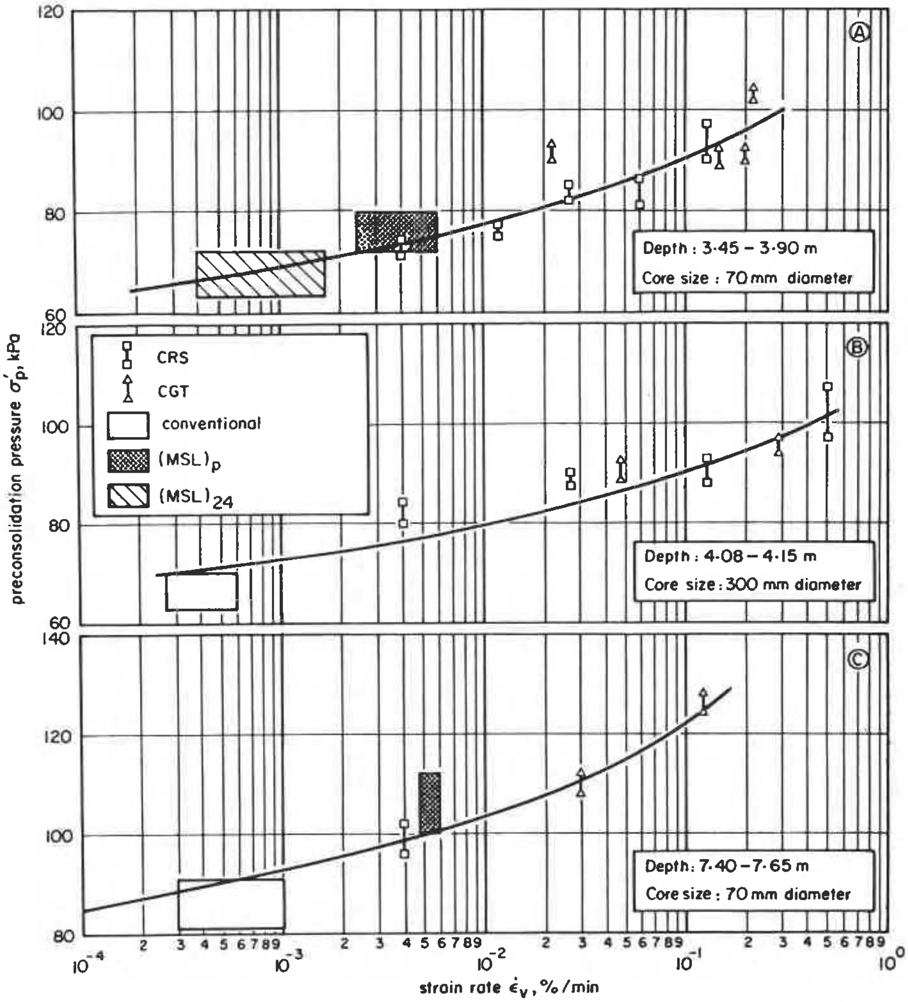

FIG. 14. Variation of preconsolidation pressure with strain rate in oedometer tests.

settlement and arching. However, as the settlements are relatively small (17 cm under the centre after 1.5 years, 30 cm after 12 years), these changes are not significant and will not be considered further. The head of water measured in piezometers P1 to P4 is shown in Fig. 15. After the embankment was constructed, the water levels continued to increase for about 10 days due to factors such as creep-induced pore pressures and time lag in the hydraulic piezometers. The pore pressures then gradually decreased, becoming quasi-stationary within 1.5 years after construction.

Figure 16 shows the average consolidation strain of the layers of soil defined by settlement gauges S1 to S4 under the centre of the embankment. In the layers 1.5–2.4 m and 2.4–4.9 m deep, the settlements are greater than in the layers further below. The consolidation strains in these layers 1.5 years after construction are 5.8 and 3.0% respectively. These strains increased to 8.0 and 5.6% respectively after 12 years, thus indicating that between depths of 1.5 and 4.9 m the clay was effectively normally consolidated. The consolidation strains at greater depths were less than 0.5% after 1.5 years and less than 1.0% after 12 years, indicating that the *in-situ* preconsolidation pressure was probably not exceeded. The consolidation strains at 1.5 and 12 years under the embankment are shown in Fig. 17*b*.

The computed vertical effective stresses on completion of construction at the level of the piezometer tips are shown (black dots) in Fig. 17*a*. The consolidation in terms of settlement is restricted at this time and the clay is clearly in the overconsolidated range or at its limit (Leroueil *et al.* 1978). Consequently, the vertical effective stress developed in the soil was less than or

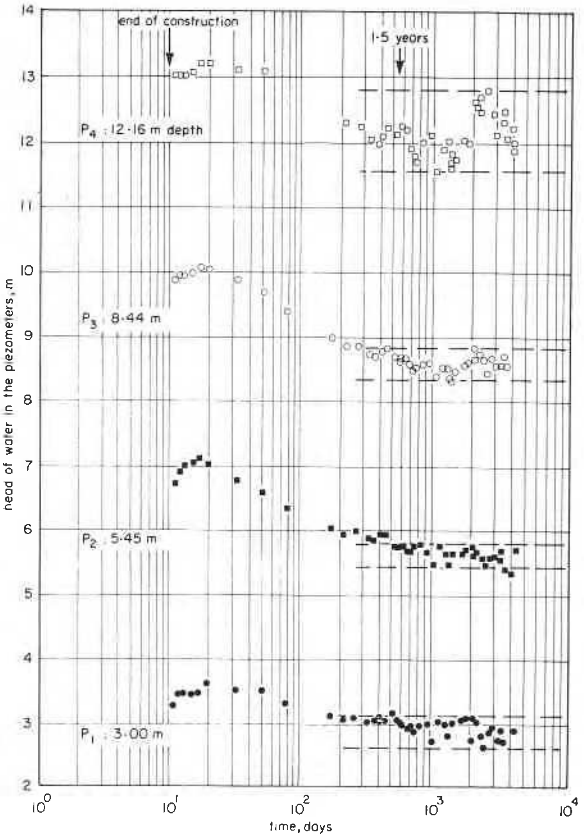

FIG. 15. Head of water in piezometers under centre of test embankment.

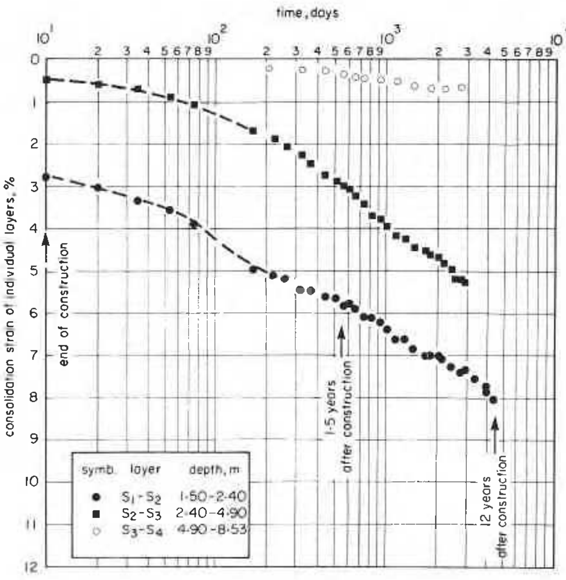

FIG. 16. Settlement of different layers under centre of test embankment.

equal to the *in-situ* preconsolidation pressure. The black dots corresponding to this state of stress represent lower limits of the *in-situ* preconsolidation pressure.

The range of vertical effective stresses, 1.5–12 years after construction are indicated by horizontal bars on Fig. 17*a*. Twelve years after construction, the consolidation of soil deeper than 4.9 m is less than 1%, clearly indicating that the *in-situ* preconsolidation pressure is not exceeded. The vertical effective stresses 1.5–12 years after construction at piezometers P3 and P4 are therefore considered as lower limits of the *in-situ* preconsolidation pressure. In the formation above 4.9 m at P1 the consolidation strains are about 3.0 and 5.6% after 1.5 and 12 years respectively (Fig. 17*b*), thus indicating that the *in-situ* preconsolidation pressure is definitely exceeded. Consequently, the corresponding state of stress is considered as an upper limit of the *in-situ* preconsolidation pressure. The *in-situ* σ_p' at this depth is well defined between these lower and upper limits and has a value of between 55 and 66 kPa. Because the piezometer tip P2 intersects the boundary between the two different soil formations (Fig. 17*a*), it is difficult to establish whether the clay at this depth reached the *in-situ* σ_p'.

The range of *in-situ* preconsolidation pressures at P1 can also be defined by plotting the consolidation strain for the layer 2.4–4.9 m deep versus the vertical effective stress computed at different times from the piezometer observations (Fig. 18). After excavation and immediately before construction of the embankment, the effective stress was about 21 kPa. It increased to 55 kPa 10 days after construction and the observed consolidation strain reached 0.6%. Thereafter, the consolidation strains increased to 5.6% after 12 years during which time the excess pore water pressures decreased (Fig. 15) and the vertical effective stress increased slightly. Figure 18 displays a very rigid field behaviour in the overconsolidated range, and shows that σ_p' *in-situ* is between 55 and 60 kPa.

Comparison of *in-situ* and laboratory preconsolidation pressures

The *in-situ* preconsolidation pressure at piezometer P1 (3 m) is between 55 and 60 kPa. According to Fig. 17*a*, the estimated preconsolidation pressure obtained from conventional consolidation tests was 62 kPa, which is very close to the range determined *in-situ*. In view of the uncertainty in developing the σ_p' profile and in calculating the effective stresses, the conventional oedometer test gave a good estimate of the *in-situ* preconsolidation pressure. As discussed before, the effective stresses at piezometers P3 (8.4 m) and P4 (12.2 m) from 1.5–12 years after construction provided

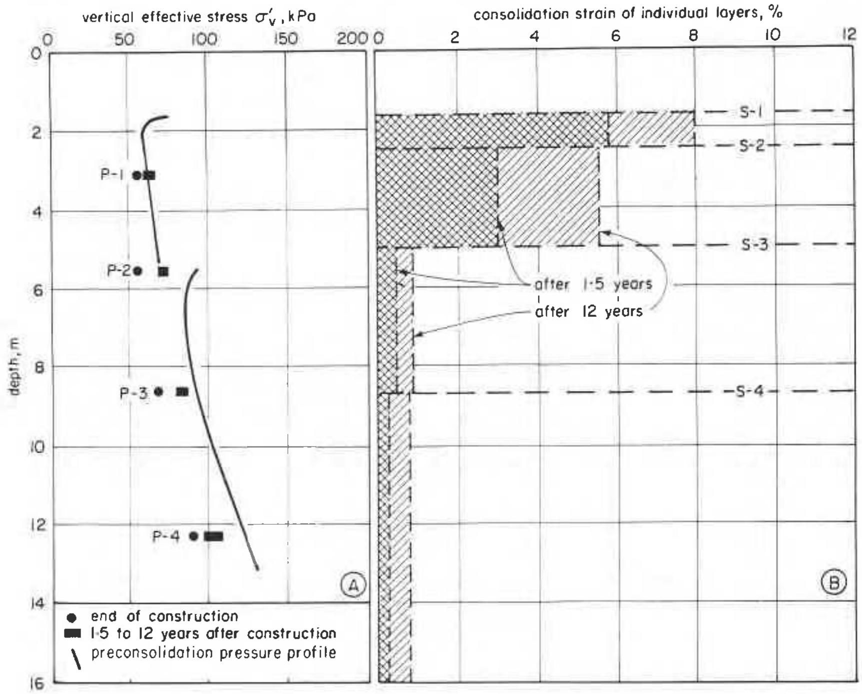

FIG. 17. Stresses and strains in clay foundation.

lower limits of the *in-situ* preconsolidation pressure. The fact that they are also lower than the conventional preconsolidation pressure profile (Fig. 17*a*) is consistent with the behaviour observed at P1.

As a result of the strong correlation between the preconsolidation pressures determined *in-situ* and those measured with conventional consolidation tests, as well as the fact that special consolidation tests produced higher values of σ_p' than the conventional tests, the following conclusions can be made.

(1) CRS (at usual strain rates) CGT, SSL, and even (MSL_p) tests overestimated the preconsolidation pressure mobilized *in-situ*.

(2) The high values obtained from the special tests were due to strain rate effects. It would be invaluable to correlate the behaviour observed in the laboratory with the behaviour *in-situ*.

The $\sigma_p' - \dot{\epsilon}_v$ results obtained on samples from depths of 3.45–3.90 m and 4.08–4.15 m (Fig. 14 (*a* and *b*)) are summarized in Fig. 19. The scatter in test data is relatively small and the observed decrease of σ_p' with strain rate is definitely representative of the laboratory behaviour of the clay layer limited by settlement plates S2 and S3 at depths of 2.40 and 4.90 m respectively.

The development of consolidation strain with time of the soil formation at a depth of between 2.40 and 4.90 m is shown in Fig. 16. The strain rate was approximately 2×10^{-5}%/min at the end of construction and about 2×10^{-6}%/min after 1.5 years. Since the preconsolidation pressure was exceeded during this period, the strain rate associated with the *in-situ* preconsolidation pressure must be between these limits.

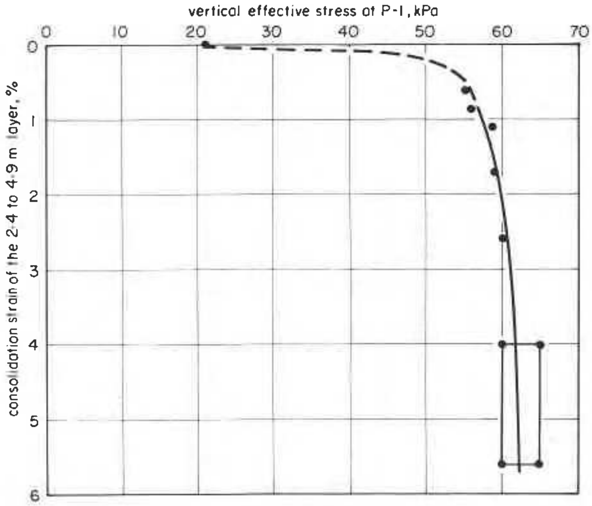

FIG. 18. Consolidation strain of clay layer from 2.4 to 4.9 m depth.

The pore water pressure for the depth at which the samples were tested (3.45–4.15 m) was not measured directly, but was estimated from observations made with piezometers P1 (3 m) and P2 (5.45 m). It was therefore possible to calculate the effective stresses at the end of

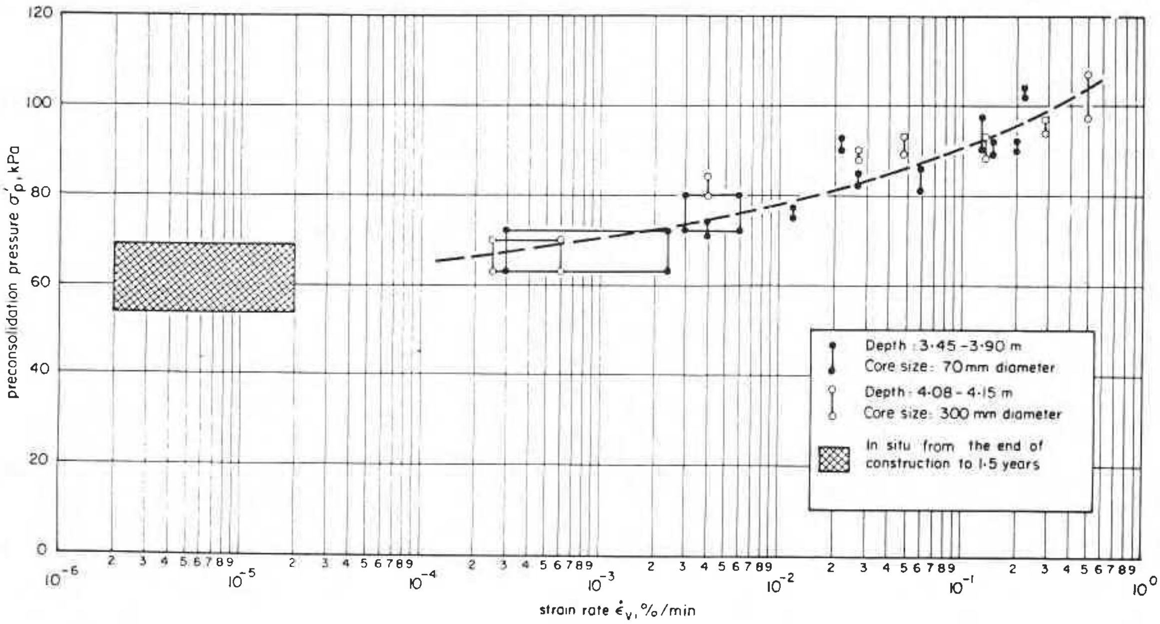

FIG. 19. Preconsolidation pressure – strain-rate relationship in laboratory and *in-situ*.

construction and 1.5 years later. These stresses, which are shown in Fig. 19 along with the corresponding range of strain rates from consolidation tests, established that the lower and upper limits of the *in-situ* preconsolidation pressure were between 54 and 69 kPa at this depth. The area representative of *in-situ* conditions is outside the range of strain rates obtained in laboratory tests but appears to lie within the extrapolated boundaries defined in laboratory results. There are indications which need, however, to be verified at other sites that a unique strain rate – preconsolidation pressure relationship exists for a given clay.

The shape of the average strain rate – preconsolidation pressure curve (Fig. 19) shows that the effect on σ_p', which is very significant at high laboratory strain rates, should be less significant at the lower strain rates prevailing *in-situ*. This would explain the high correlation between the *in-situ* preconsolidation pressures and that obtained with conventional tests which correspond to relatively low strain rates. On the other hand, σ_p' obtained from the special tests overestimated the *in-situ* σ_p' and are not directly applicable in engineering practice. These values must be corrected by a factor representative of the strain rate for the type of test carried out. The correction factor for the clays at the test site would be about 0.75 for a CRS test performed at a strain rate of 0.02%/min and about 0.55 for a SSL test.

To define correction factors for the various consolidation tests that would be of practical value, it is necessary to supplement these data with many more similar case studies. This was treated in subsequent papers.[1]

Conclusions

During the past 15 years the following special test methods have been suggested for determining the compressibility and, in particular, the preconsolidation pressure of clays: constant-rate-of-strain oedometer test, controlled-gradient oedometer test, single- and multiple-stage-loading oedometer tests, and anisotropic triaxial consolidation. These special techniques, together with the conventional oedometer test, were applied to the clays at the Gloucester test site, and the results were compared with the preconsolidation pressure effectively mobilized *in-situ* under a test embankment. The following conclusions were made.

1. All oedometer data are consistent and show that a unique preconsolidation pressure – strain rate relationship exists for a given clay at a given depth. Of all tests performed, the conventional oedometer test corresponds to the slowest rate of strain and consequently yields the lowest σ_p'. The special consolidation tests yield much higher values, consistent with the higher rates of strain.

2. The preconsolidation pressure was determined

[1](*a*) Morin, P., Leroueil, S., Samson, L. Preconsolidation pressure of Champlain clays, Part I: *In-situ* determination. (*b*) Leroueil, S., Tavenas, F., Samson, L., Morin, P. Preconsolidation pressure of Champlain clays, Part II: Laboratory determination. Both papers have been accepted for publication by the Canadian Geotechnical Journal.

in-situ from pore water pressure and settlement observations over a 12-year period. The *in-situ* σ_p' correlated best with that determined from conventional tests, but was overestimated significantly by the special tests. The *in-situ* $\sigma_v' - \dot{\epsilon}_v$ conditions seemingly confirmed the existence of a unique $\sigma_p' - \dot{\epsilon}_v$ relationship for the clay, and indicated that the strain-rate effect was more significant at the higher laboratory strain rates than at the lower rates prevailing *in-situ*.

3. Similar studies should be extended to other sites to verify these conclusions and to establish a strain-rate correction factor so that some of the suggested special consolidation tests can be used to derive the *in-situ* preconsolidation pressure.

4. The structure of the clay was altered by soil sampling and loading. In the overconsolidation range, the intact clay displayed a very rigid behaviour *in-situ* which seemed to be well preserved when large-diameter block sampling was used. The 70 mm diameter sampler partially disturbed the clay and made it more compressible in the overconsolidation range.

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