Measured Strengths under Fills on Sensitive Clay

Résistance Mesurée d'Argile Sensible sous Remblais

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SYNOPSIS A series of field vane strength measurements conducted in the subsoil of four fills of different geometry and constructed on Champlain Sea clays near Ottawa, Canada, gave values the same as those found prior to construction. On the other hand, laboratory triaxial compression tests conducted on undisturbed piston samples taken at the same time as the vane measurements showed a definite increase in shear resistance when tested at stress conditions comparable to the effective stresses existing under the fills. The possible factors contributing to this discrepancy are discussed.

INTRODUCTION

When soft, sensitive marine clays are loaded, settlements occur over long periods of time owing in part, it is believed, to a breakdown in structure and resulting increase in pore water pressure (Crawford and Burn, 1976). Under such circumstances stage construction is somewhat hazardous if used to achieve a gain in strength and an increase in stability with time. A number of fills in the Ottawa region have been under observation over a period of several years. This paper reports some of the long-term observations, the measured strengths under the fills (using field and laboratory techniques), and discusses factors affecting the measurement of strength when the field vane is used.

SITES AND FILLS

Four fills constructed at three sites on Champlain Sea clays in the Ottawa region are considered. The general characteristics of the subsoils involved are similar to those reported earlier by Crawford (1961). At each site, subsoils were sampled with the 54-mm dia NGI piston sampler (Bjerrum, 1954) or the Osterberg 127-mm dia sampler (Osterberg, 1952). Routine classification, triaxial and consolidation tests were carried out; and the results, together with the applied vertical pressures, are summarized in Figures 1 to 3. The fills were instrumented extensively and some of the measurements were reported elsewhere (Eden and Poorooshasb, 1968; Bozozuk and Leonards, 1972).

Two fills were constructed in the fall of 1967 near South Gloucester, 21 km south-east of Ottawa, Canada. One, a test embankment constructed in an excavation 1.2 m deep, is a granular fill 3.7 m high, 9.2 m wide and 36.6 m long at the top, with 1.5:1 side slopes. A plane strain condition prevails at its middle section where the bulk of the instrumentation was installed. The second fill, built of materials from the excavation, is circular, about 2.4 m high and 22.4 m in diameter at the top, with 1.5:1 side slopes. The two fills were sufficiently far apart to avoid mutual influence.

A third fill is located at Kars Bridge, 40 km south of Ottawa. This is an approach embankment, built in stages, of granular soil. The first lift was placed in November 1959 to a height of 6.1 m and was brought to its final elevation of 8.0 m 20 months later. The top width of the fill is 15.2 m and the side slopes 2:1.

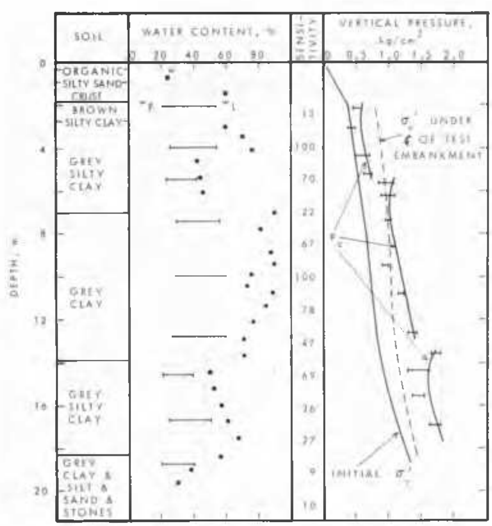

Figure 1 Geotechnical Profile at Gloucester

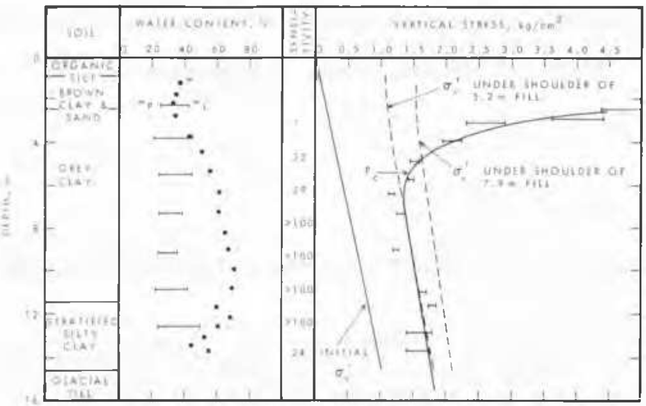

Figure 2 Geotechnical Profile at Kars Bridge

The fourth fill is situated about 6.5 km east of the Gloucester site on Boundary Road where it crosses Highway 417. This was constructed in June 1970 as a two-level test embankment with heights of 2.7 and 4.3 m, respectively. It was graded a year later to form an approach fill with a maximum height of 4.3 m.

SETTLEMENT AND PORE PRESSURE RECORDS

Figures 4 to 6 show the vertical compression and excess pore pressure measured in the soil formations in which the imposed load exceeded the preconsolidation pressure below the centreline of the fills. Substantial compression (4 to 8 per cent) has taken place, with the actual amount depending on the intensity, geometry, and duration of loading. These are similar to those observed beneath the shoulder of the fills. The ratios of surface settlement below the shoulder to that below the centre are 62 per cent at the Gloucester circular fill, 92 per cent at Kars Bridge and 86 per cent at the Boundary Road.

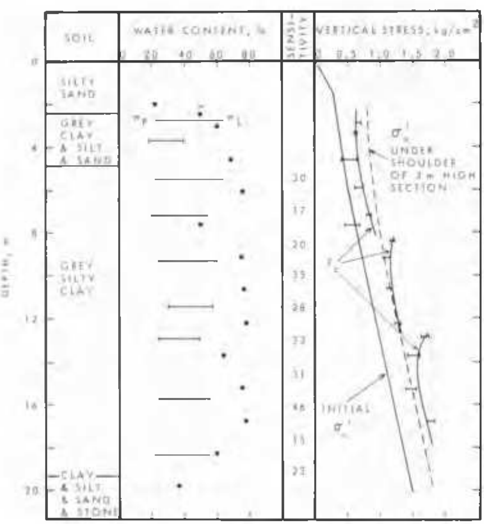

Figure 3 Geotechnical Profile at Boundary Road

Excess pore water pressure generally developed immediately on loading. Following construction, it dissipated relatively rapidly at first, then slowly dropped to a value of less than 1 m of water head. This resulted in a vertical effective stress exceeding the preconsolidation pressure in all cases.

FIELD VANE SHEAR TESTS UNDER FILLS

During 1974 and 1975, series of field vane tests were performed at the four fills. At Gloucester, one boring was made through the centre of the test embankment and three were made through the circular fill at varying distances from the centre. At the approach embankment of Kars Bridge two vane borings were performed through the shoulder where thicknesses of fill were 7.9 and 5.2 m, respectively. The induced stresses at these locations were significantly different. For undrained conditions (short-term loading) no zone of local failure (region in which the soil is strained beyond the peak strength) was detected in the lower section, but an extensive zone was found beneath the higher section (Law, 1974). The last vane boring was carried out through the shoulder of the Boundary Road fill at a section where thickness was about 3.0 m.

The results of the vane tests are shown in Figures 7 to 9. Also plotted on these figures are the pre-construction in situ field vane strengths measured with the same equipment. In general, there is little difference between the two sets of results, implying that the sustained loading and resulting large compression did not effect the shear strength of the soil.

For the circular fill at Gloucester there was no

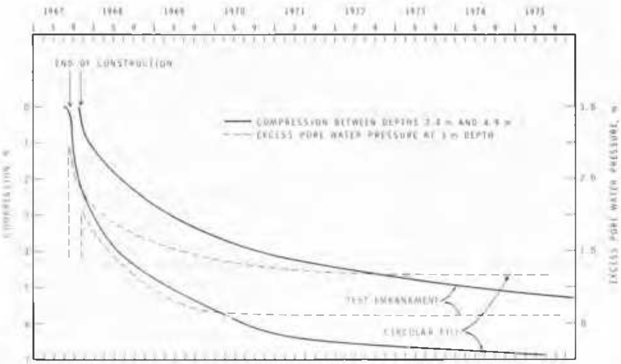

Figure 4 Records of Settlement and Excess Pore Water Pressure Under Centreline of the Fills at Gloucester

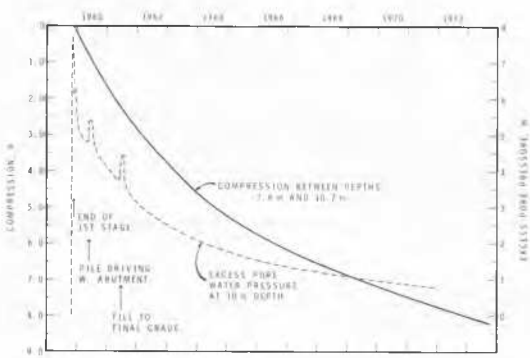

Figure 5 Compression and Pore Pressure Records at Centre line, Kars Bridge Fill

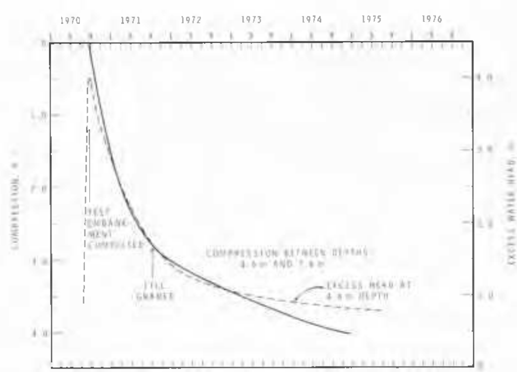

Figure 6 Settlement and Excess Water Head Under Shoulder of Embankment at Boundary Road

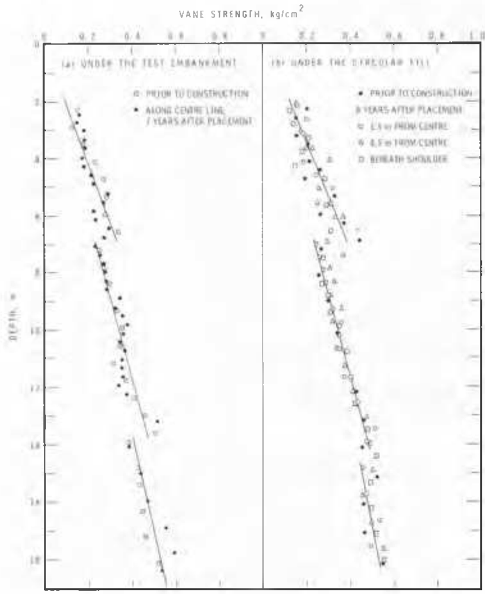

Figure 7 Field Vane Strengths at Gloucester (All Tests Restored to Pre-construction Elevations)

appreciable difference in measured shear strength beneath the centre and the shoulder, implying that the rotation of principal stresses beneath the shoulder did not change the vane strength. At Kars Bridge also, the results indicate no change in undrained shear strength, whether or not local failure conditions exist.

TRIAXIAL COMPRESSION TESTS

A program of undrained triaxial compression tests was carried out on the soils from Kars Bridge and Gloucester. There were three series of tests: (1) CIU and CAU tests performed at different consolidation pressures on undisturbed samples obtained prior to construction (pre-construction soils); (2) same as (1) except on undisturbed soil samples taken from under the fills (post-construction soils) at the time of vane shear testing; (3) CAU tests performed on post-construction soil samples consolidated to stress conditions comparable to those existing under the fills. The results of the tests are shown in Figures 10 to 12.

The strength envelope depicted in Figures 10 and 11 is similar to that described by Eden and Jarrett (1971). It has a pronounced curve at low effective pressures and is linear at higher pressures. This envelope remains practically unchanged for the post-construction soil.

The undrained strengths deduced from the field vane and triaxial tests corresponding to pre-construction and 1975 conditions are shown on Figures 10 and 11. A sizable strength increase can be noted from the triaxial tests.

A further comparison of the triaxial strengths of the Kars Bridge soil is presented in Figure 12. Results

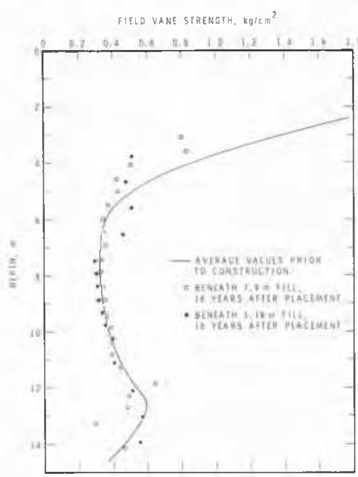

Figure 8 Field Vane Strengths Under Kars Bridge Fill (All Tests Restored to Pre-construction Elevations)

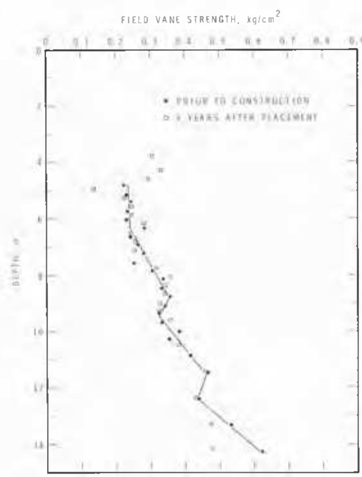

Figure 9 Field Vane Strengths Under Boundary Road Fill (All Tests Restored to Pre-construction Elevations)

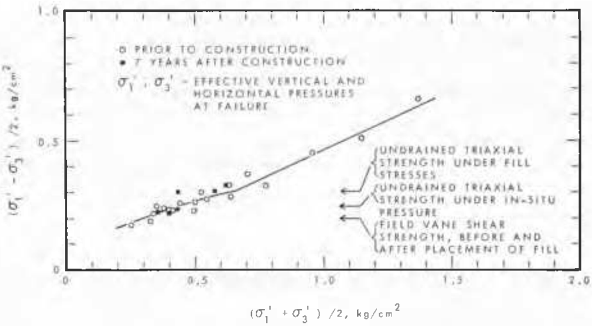

Figure 10 Strength Envelope for Soil Layer Between Depths 2.3 m and 5.3 m (Gloucester)

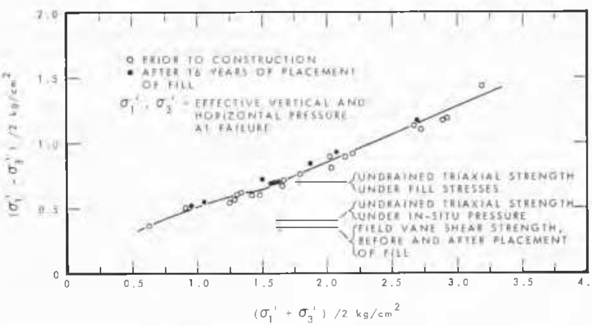

Figure 11 Strength Envelope for Soil Layer Between Depths 7.0 m and 11.3 m (Kars)

are shown of CIU tests on the pre- and post-construction soils at various depths, both consolidated to the same pressure close to the effective

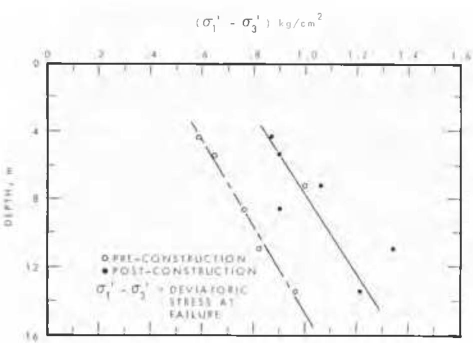

Figure 12 Comparison of Shear Strengths of Pre- and Post-construction Soils (Kars)

overburden load. A distinct strength increase may again be observed.

DISCUSSION

To resolve the question of whether there is any strength increase beneath fills, two more observations may be noted. (1) There is a marked reduction in moisture content in the subsoil beneath the fills. This reduction is compatible with the observed compression (Table I), indicating that lateral

TABLE I COMPARISON OF OBSERVED AND DEDUCED COMPRESSIONS

Site	Depth m	Observed Compression per cent	Compression deduced from change of water content per cent
Gloucester	2.4 to 5.0	5.20	4.64
Kars	7.6 to 10.7	8.14	7.72

yielding had little influence on compression. The subsoils beneath the fills had reached, therefore, a denser state. (2) Based on measurement of lateral pressure beneath the Gloucester test embankment (Bozozuk, 1974), the deduced maximum shear stress exceeded the vane strength down to a depth of 5.3 m. At 3 m for instance, shear stress is 50 per cent in excess of vane strength. These observations tend to throw additional support to the belief that there is actually a strength increase.

Reasons why the field vane failed to measure any strength gain include:

(1) Essentially, the field vane measures shear resistance along a vertical plane, which corresponds to the strength of a horizontal specimen (Lo and Milligan, 1967). As the soil displays strength anisotropy (Mitchell, 1970), there may not be an increase of strength along the vertical plane within the vertical pressure intensity encountered in these cases.

(2) By recognizing that vane strength corresponds to that along a vertical plane, one can argue that the strength change depends largely on the change in effective horizontal pressure, $\Delta\sigma_h'$. Bozozuk (1974) showed by means of hydraulic fracture tests that $\Delta\sigma_h'$ is approximately equal to zero beneath the Gloucester test embankment. Numerical analyses reveal that under the other fills the final effective horizontal pressure is less than 60 per cent of the vertical preconsolidation pressure. Hence, in all these cases $\Delta\sigma_h'$ is rather small. This may largely explain the anomaly between triaxial and vane strengths.

(3) Preliminary numerical analyses carried out for this study show that intrusion of the vane blade into this particular soil induces a shear strain that can exceed the strain at failure along the prospective cylindrical failure surface. Thus the vane would measure a strength intermediate between the peak and the residual. How this intermediate strength varies with confining pressure is not clear, but it would contribute to the observed discrepancy.

CONCLUSIONS

Field and laboratory measurements of shear strength under four fills constructed on Champlain Sea clays lead to the following conclusions:

(1) Field vane strength does not change with (a) time up to 16 years after placement of fill; (b) loading geometry corresponding to either plane strain or axisymmetric conditions; (c) load intensity to values moderately beyond the preconsolidation pressure; (d) rotation of principal stresses existing beneath the shoulder of fill.

(2) Triaxial tests show that there has been strength increase beneath the fills.

(3) Field evidence and preliminary analyses support the view that there is strength gain under sustained loading.

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