Comparison of field consolidation with laboratory and *in situ* tests

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Settlement calculations for an earth embankment resting on soft, compressible Fraser River Delta sediments were made from laboratory consolidation tests and *in situ* tests using a piezocone and a flat dilatometer. The calculated values were compared with measured settlements. There was rather good agreement among the three methods of calculation, but the actual settlement was about 60% greater than the average calculated value. Calculated rates of settlement are also compared with observed values.

Key words: consolidation, settlements, piezocone, dilatometer, *in situ* tests, constrained modulus, pore-water pressure, settlement rate.

Des calculs de tassement pour un remblai de sol reposant sur des sédiments mous compressibles du delta du fleuve Fraser ont été faits à partir d'essais de consolidation et d'essais *in situ* au moyen d'un piézocone et d'un dilatomètre plat. Les valeurs calculées ont été comparées avec les tassements mesurés. Il y a une concordance assez bonne entre les trois méthodes de calcul, mais le tassement réel est d'environ 60% plus grand que la valeur moyenne calculée. Les vitesses de tassement ont également été comparées avec les valeurs observées.

Mots clés : consolidation, tassements, piézocône, dilatomètre, essais in situ, module confiné, pression interstitielle, vitesse de tassement.

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Introduction

The use of *in situ* tests to predict consolidation settlements is of considerable interest to foundation engineering practice. In recent years, theories and correlations have been developed in an effort to relate *in situ* test results to the magnitude and rate of soil consolidation under stress increases, but full-scale observations are required to assess their reliability. The primary purpose of this paper is to compare the consolidation parameters obtained from laboratory tests with those obtained from piezocone and flat dilatometer tests and to compare the predicted amount and rate of settlement with actual values measured beneath an embankment.

An earlier paper by Crawford and deBoer (1987) described the construction of the highway embankment shown in Fig. 1 and presented 15 years of settlement observations made at nine locations beneath the shoulders of the embankment. These observations showed that the settlements were directly related to the height of fill but were much greater than predictions based on laboratory consolidation tests. It was decided therefore to explore these discrepancies by carrying out new consolidation tests and to use the observed settlements to assess *in situ* test methods for predicting settlements.

The site

The site is located at Colebrook Road, approximately 25 km southeast of downtown Vancouver, British Columbia, on Highway 99A just north of its intersection with Highway 99. The natural ground is flat and poorly drained, with some surface deposits of peat lying over a soft, compressible silty clay with occasional sand layers.

The embankment, 24 m wide at the top with 2:1 side slopes, was built in stages during a 15-month period beginning in 1971. Sand drains, 0.4 m in diameter and 4.3 m on centres, were installed under the fill from the toe of slope at the abutment to station 68.5 (Fig. 2). The calculations and settlement measurements reported in this paper were made at station 67, where the final height is 3.8 m above original ground level and where surface peat was not reported in the original borings.

The original site investigation had established that the subsoils are reasonably uniform, with an average natural water content of 45%, a liquid limit of 36%, and a plasticity index of 11%. Artesian pressures were noted in some boreholes at the interface between the silty clays and the underlying dense sands and gravels. The recent tests indicate almost hydrostatic pore-water pressures in the upper regions of the silty clay, but when soundings reached the underlying sand and gravel there was evidence of slight artesian pressures. On the basis of all available information the silty clay is considered to have an average saturated unit weight of 17.8 kN/m^3 with the groundwater table at the surface and hydrostatic conditions below.

Field investigations

Sampling and testing was carried out during October and November 1988 within a test area, approximately 10 m square, located near the embankment but generally outside its influence on the subsoils (Fig. 2). The piers for the bridge structure are founded on end-bearing piles; the abutments are on spread footings. In December 1989 a second field vane test was made under the bridge near station 365.

The ground surface under the bridge is estimated to be about 0.5 m above the original ground surface, and the test area slopes upward to the west about 1 m farther to the shoulder of an adjacent roadway. Wet surface conditions in 1988 precluded the possibility of conducting all of the tests at the lower levels. The height above original ground level is indicated in each case.

Soil samples were recovered by the Ministry of Transportation and Highways of British Columbia from a borehole

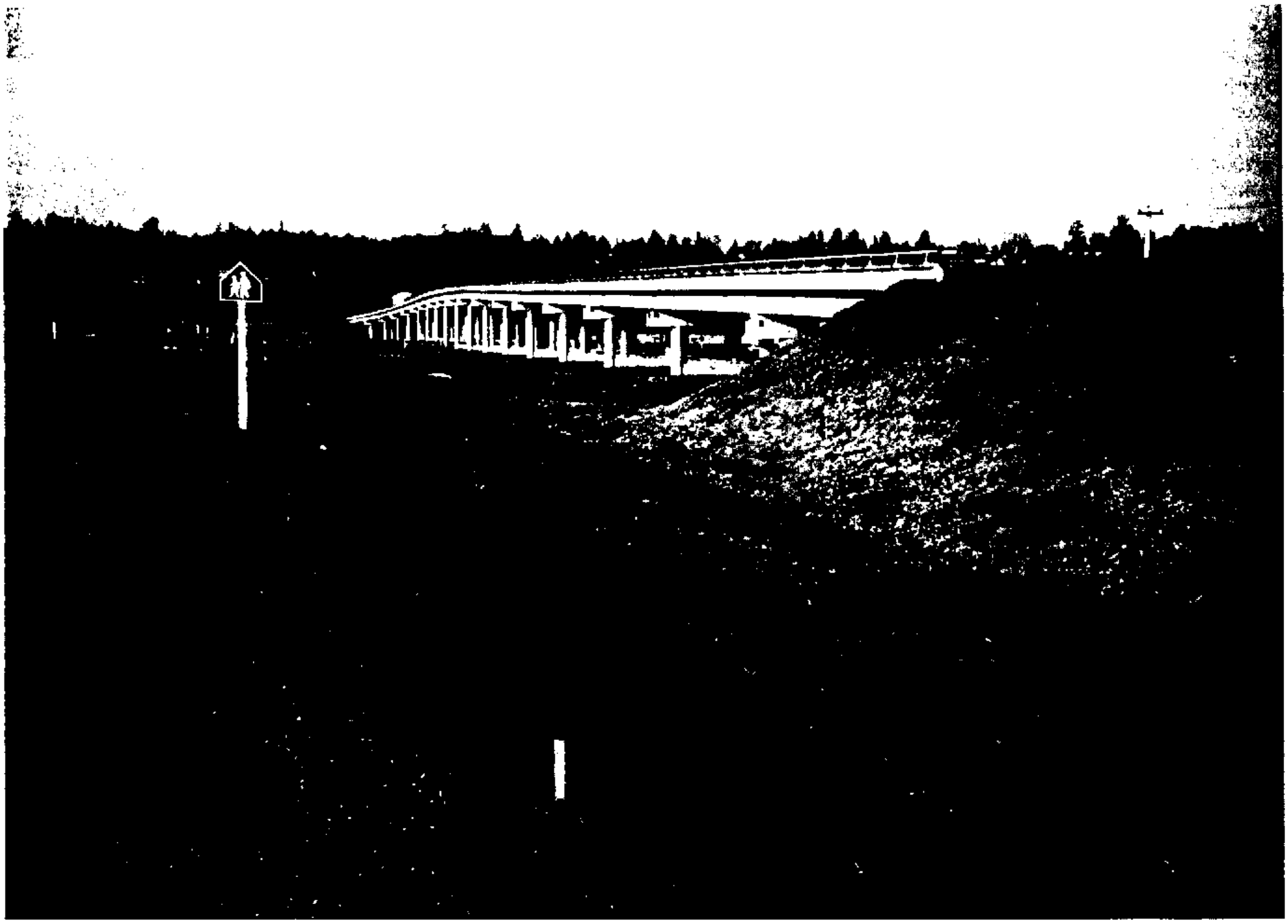

FIG. 1. Test area and embankment.

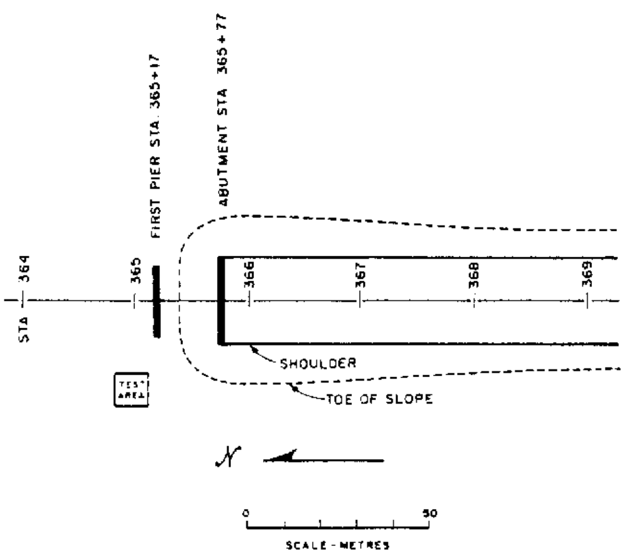

FIG. 2. Plan view of Colebrook site.

located about 1.4 m above the original surface using an 85 mm, hydraulically operated piston sampler. Consolidation tests were conducted on specimens cut from these samples. Field vane shear tests (FVST) were made near the first pier from a surface elevation 0.5 m above original grade.

A flat dilatometer test (DMT) and a piezocone penetration test (CPTU) were carried out from surface elevations approximately 1.4 m above original ground level. A BAT groundwater monitor (Torstensson 1984) was installed nearby at a depth of 10 m to measure the equilibrium pore-water pressure and the *in situ* permeability of the compressible clay.

Consolidation tests

Consolidation tests were conducted using a fixed-ring odometer with specimens 20.3 cm^2 in area and 2.5 cm high. The specimens were allowed to remain overnight at a nominal seating load of about 3 kPa and then compressed at a constant rate of strain (CRS) of about 0.65%/h while pore-water pressures were monitored at the base. Tests were completed in about 1 day.

Specimens for consolidation tests were influenced by the 1.4 m of fill over the original ground surface. This fill has been in place for many years as part of the original highway before construction of the bridge, and the subsoil may therefore be considered to be fully consolidated under its weight. The original effective stresses in the natural ground and the stresses at the borehole under 1.4 m of fill are shown in Fig. 3.

The preconsolidation stresses (σ'_p) interpreted from laboratory tests are also shown in Fig. 3. The large solid circles are σ'_p values obtained from well defined e – log σ' curves

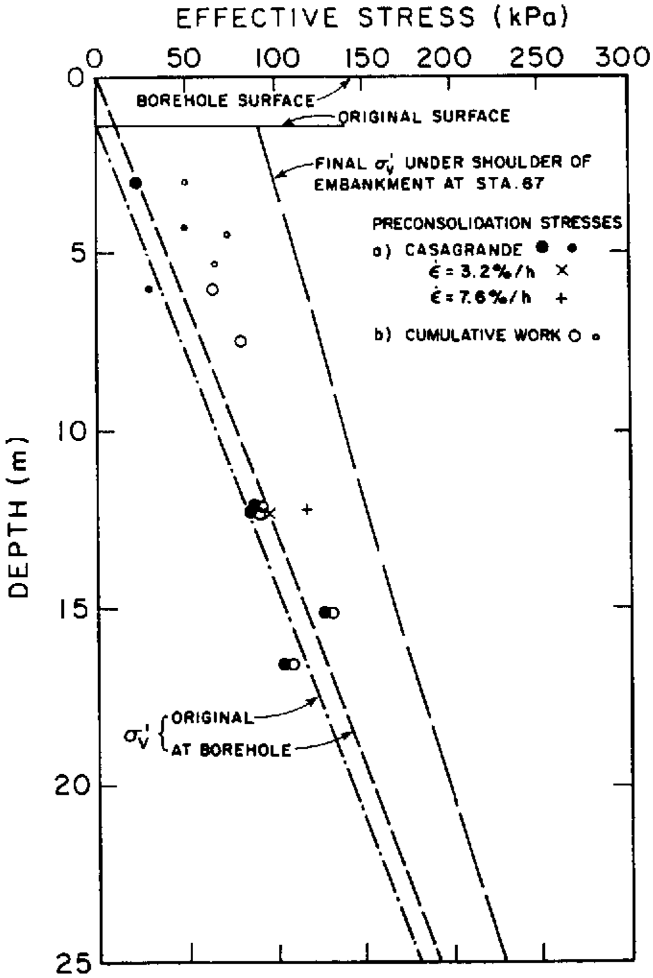

FIG. 3. Effective stresses related to depth.

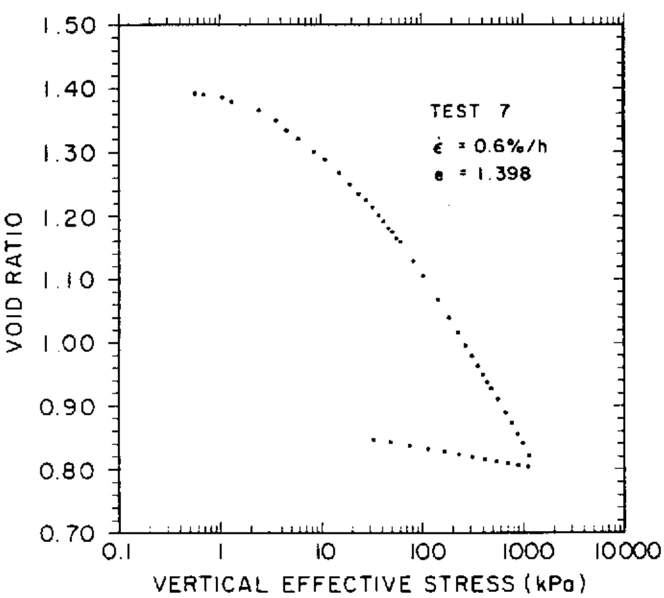

FIG. 4. Consolidation curve (test 7).

using the usual Casagrande construction. The smaller solid circles were obtained from e – log σ' curves that were difficult to interpret, probably owing to disturbance. Several of the test curves were so rounded that the σ'_p values could

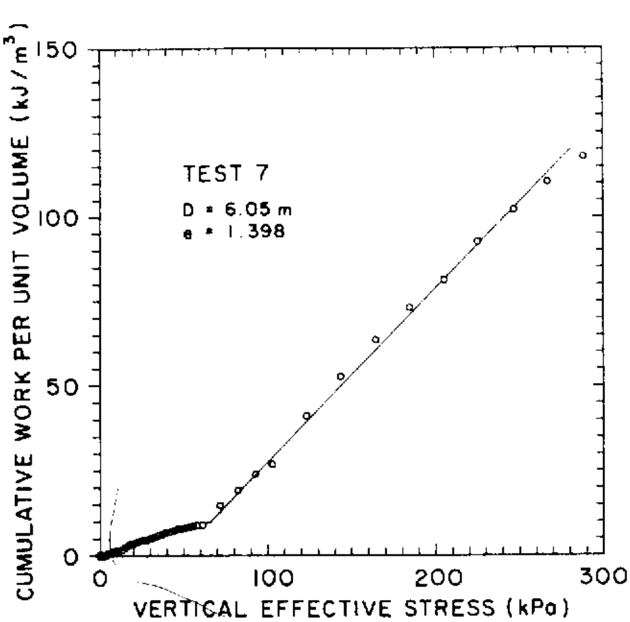

FIG. 5. Cumulative work curve (test 7).

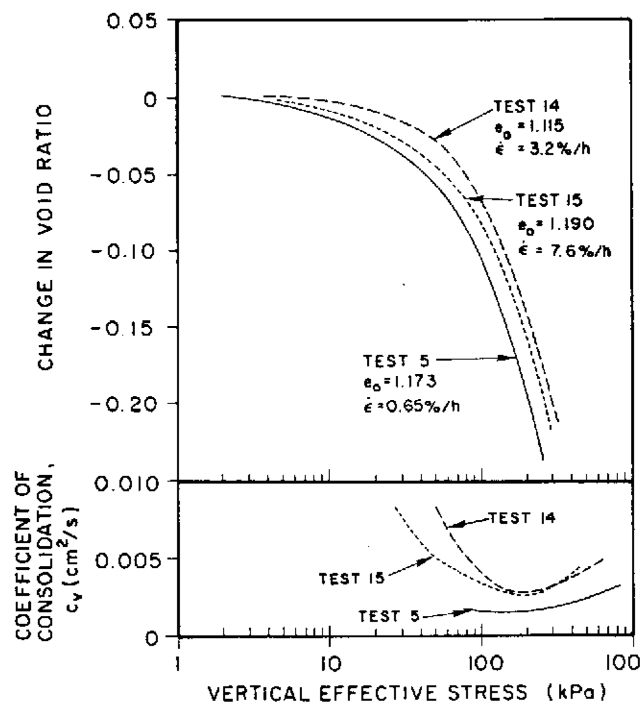

FIG. 6. Consolidation curves (tests 5, 14, and 15).

not be estimated from the e – log σ' curves, so cumulative work curves were used following the method proposed by Becker *et al.* (1987). These are shown as open circles where the larger circles indicate well-defined intersections.

The cumulative work concept has been questioned by Morin (1988), but it did give definitive values when the Casagrande construction was clearly unsatisfactory. Figure 4 shows the e – log σ' curve and Fig. 5 shows the cumulative work curve for test 7. The modulus method (Janbu 1967) was also tried but was not satisfactory for these tests.

It appears from Fig. 3 that the subsoils at the site are normally to slightly overconsolidated. The cumulative work method of interpretation suggests considerable overconsolidation in the upper 6 m, but the e – log σ' curves do

TABLE 1. Consolidation test results

Test No.	Depth (m)	Preconsolidation stress (kPa) Casagrande	Preconsolidation stress (kPa) Cumulative work	e_0	c_c	K (cm/s × 10^{-7})	c_v (cm^2/s × 10^{-3})	c_h (cm^2/s × 10^{-3})[a]	M (kPa)[b]
12	3.0	22	50	1.877	0.55	1.2	0.6	1.5	600
2	4.3	50		1.292	0.29				
11	4.5		75	1.026	0.16				4000
10	5.4		67	1.114	0.22	1.0	0.9	2.2	2200
7	6.0	30	67	1.398	0.28	1.2	0.3	0.7	1800
6	7.5		81	1.119	0.24	0.8	2.0	5.0	2800
4	12.1	88	90	1.218	0.34	1.2	2.8	7.0	2400
5	12.2	86	91	1.173	0.34	0.7	1.4	3.5	2000
14	12.3	96	93	1.115	0.30	0.9	2.8	7.0	2600
15	12.3	117	120	1.190	0.37	0.9	2.0	6.2	2600
8	15.1	128	131	1.389	0.52	0.9	1.5	3.7	2100
9	18.1	105	110	1.485	0.45	0.7	1.9	4.7	2100
Average				1.285	0.34	1.0	1.6	4.0	2300

[a] c_h is assumed to be 2.5 × c_v.
[b] $M = 1/m_v$ at average stress, $(\sigma_0 + \sigma_f)/2$.

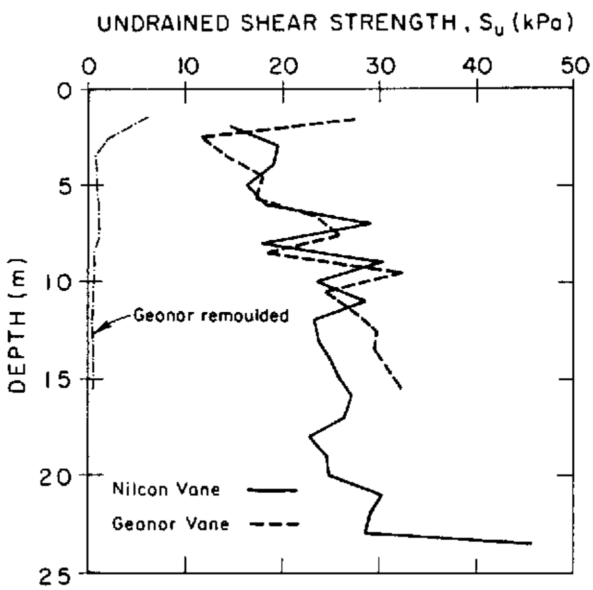

FIG. 7. Field vane shear tests (Colebrook).

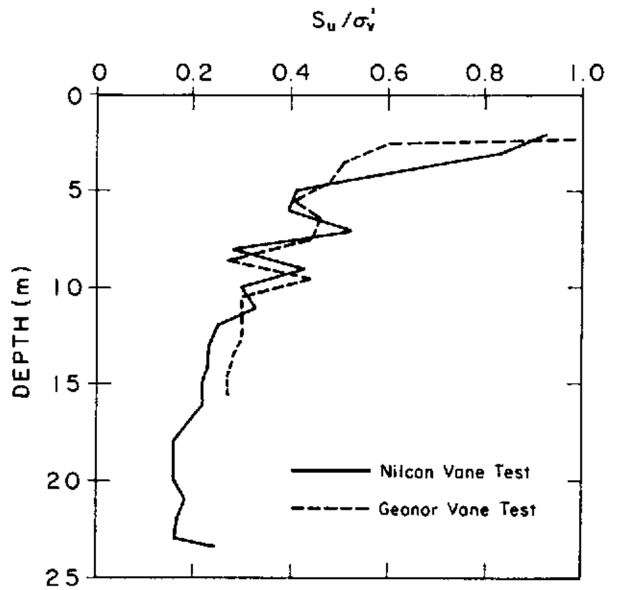

FIG. 8. Relationship of vane strength to vertical effective stress (Colebrook).

not confirm this interpretation. There is little indication of overconsolidation at lower depths. There is some geological evidence (Williams 1988) that the sea level may have been considerably lower than at present, and this would be expected to cause some overconsolidation, but the age of the sediments in relation to sea levels is not well established.

The effect of strain rate on test results has not been adequately explored, but there is evidence that it has some influence on these soils. All of the specimens were strained at a rate of 0.65%/h, except at a depth of 12 m one specimen (shown with an x in Fig. 3) was strained at a rate of 3.2%/h and another (shown with a +) was strained at 7.6%/h. It is seen that the interpreted value for σ_p' increases with increasing strain rate, but when test results for the three specimens are plotted as $e - \log \sigma'$ curves (Fig. 6), the most rapid test result falls between the other two. This may be attributed to variations in the soil because the specimens were obtained from slightly different levels. It was noted that at the preconsolidation stress the base pore pressure for specimen 5 was about 7% of the applied stress, and 15 and 30%, respectively, for specimens 14 and 15. As a result of the high pore-pressure gradients, the calculated effective stresses for the two rapid tests may be less reliable.

The coefficients of consolidation (c_v) and vertical permeability (k_v) were determined assuming parabolic distribution of pore pressure above the base. The value of c_v for specimen 5, plotted in Fig. 6, is reasonably constant during the test. The approximate value for c_v is 1.6 × 10^{-3} cm^2/s and for k_v is 10^{-7} cm/s. At faster rates (tests 14 and 15) the computed values are high during recompression and at a minimum at stresses just beyond the preconsolidation stress.

The consolidation test results are shown in Table 1. The depth of specimen is measured from the top of the borehole, which is 1.4 m higher than the original ground level. Taking elevations into account, these test results are very similar to those obtained in 1969 before the embankment was built.

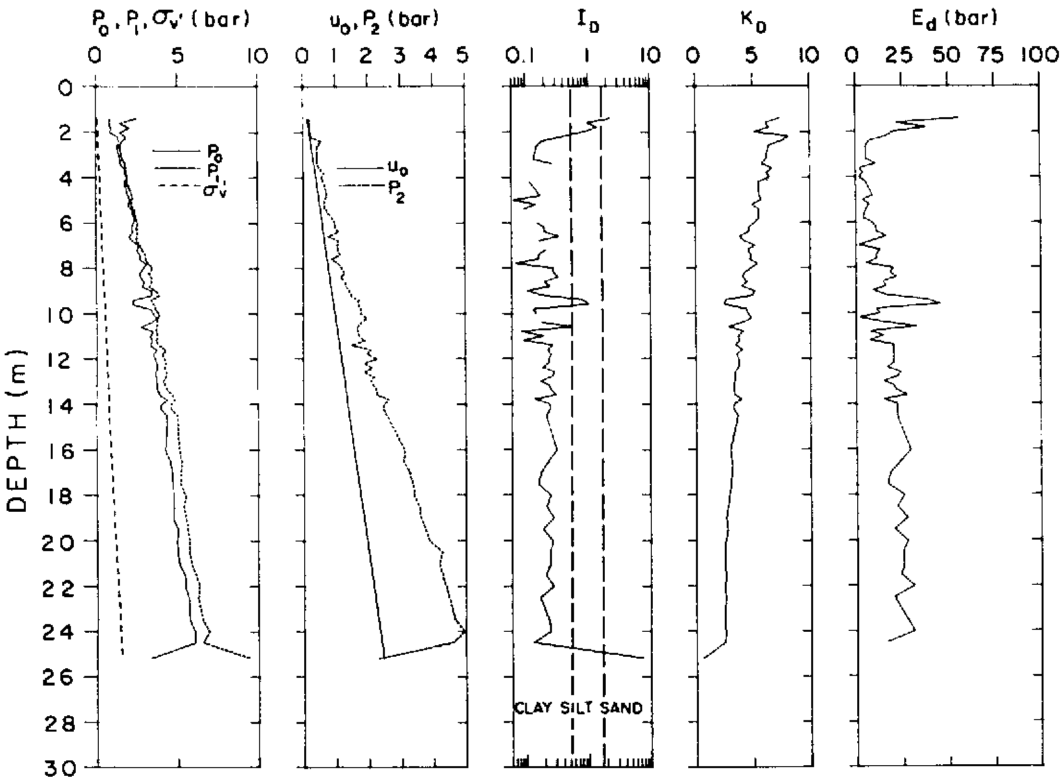

FIG. 9. Flat dilatometer test, DMT (Colebrook) (1 bar = 100 kPa).

Field vane shear test

The surface at the location of the first vane boring is about 0.5 m above the original ground level. As shown in Fig. 7, the measured undrained shear strength (S_u) averages about 20 kPa through the upper 8 m and about 27 kPa below 8 m. The ratio of undrained shear strength to vertical effective stress (S_u/σ_v') shown in Fig. 8 is very high in the upper 5 m and decreases steadily to the bottom of the boring. This indicates that the upper part of the soil is lightly overconsolidated and the lower soils are normally consolidated. However, at upper levels the cone log (Fig. 12) indicates the presence of sandy lenses that could cause vane strengths and consequently the ratio S_u/σ_v' to be too great relative to more uniform soils.

A second FVST was made in December 1989 using a Geonor vane that has an outer rod to eliminate friction on the torque rod. The result, plotted in Figs. 7 and 8, is in reasonable agreement with the Nilcon test except at the lower levels where the strength is about 25% higher. These results support the view that soils in the upper 5–7 m are lightly overconsolidated where S_u/σ_v' is in excess of 0.3 (Robertson and Campanella 1983).

The remoulded strengths measured with the Geonor vane gave an average sensitivity of 25 from depths of 2–8 m and an average of 80 below 8 m. This remarkably high sensitivity agrees with measured liquidity indices of 1.5–2 and is probably caused by leaching of pore-water salts owing to the slight artesian condition.

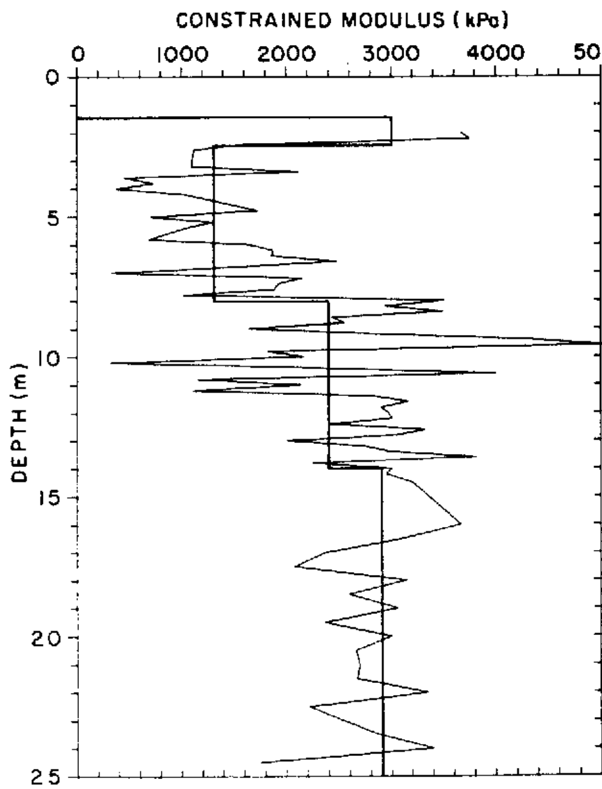

FIG. 10. DMT constrained modulus (Colebrook).

Flat dilatometer test

The dilatometer test was carried out in accordance with the proposed standard method (Schmertmann 1986) as part of the University of British Columbia research on *in situ* testing methods (Campanella and Robertson 1983; Robertson *et al.* 1988). The empirical relationships for dilatometer modulus, $E_D = 34.7\ (p_1 - p_0)$, horizontal stress index, $K_D = (p_0 - u_0)/\sigma_v'$, and material index, $I_D = (p_1 - p_0)/$

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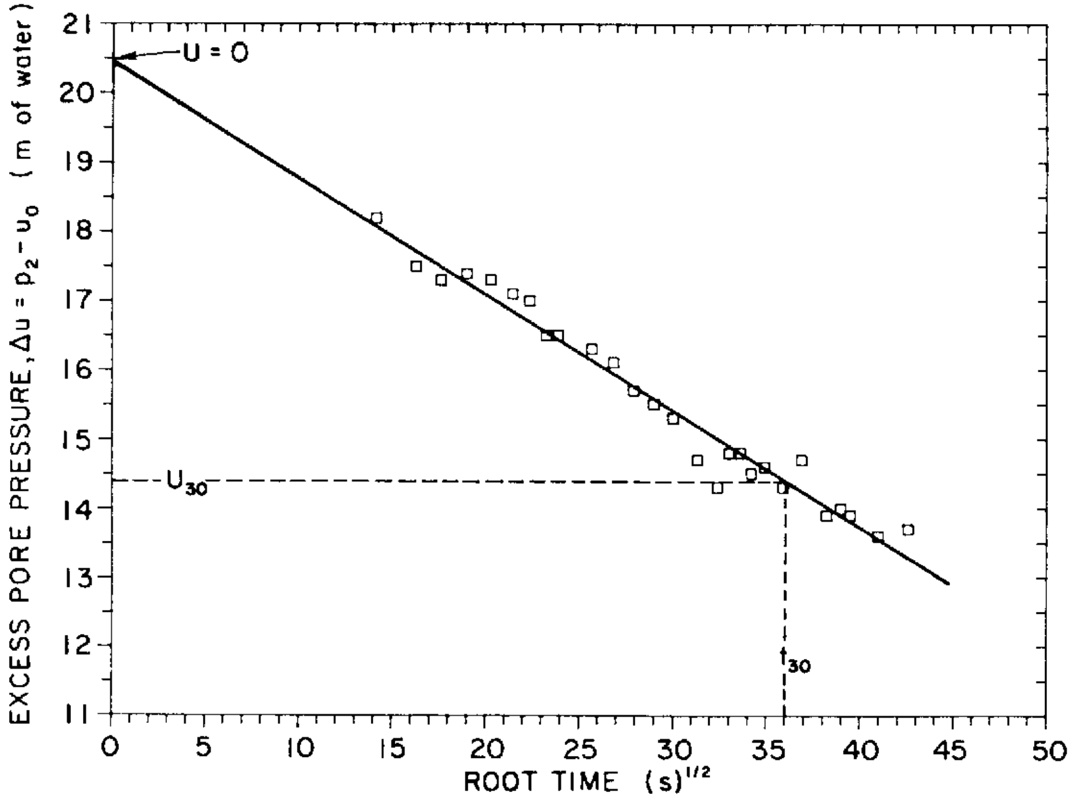

FIG. 11. DMT dissipation test at 15 m depth (Colebrook).

$(p_0 - u_0)$, are those developed by Marchetti (1980)[1] with some slight modifications in recent years. The calculations for these relationships are carried out using the DILLYS FORTRAN program developed by GPE Incorporated, Gainesville, Florida. The test results are shown in Fig. 9.

The vertical constrained modulus, M, which is required for settlement calculations, is obtained from the dilatometer modulus, E_D, by the relationships $M = R_M \times E_D$ where the ratio R_M is derived from I_D and K_D (Marchetti 1980). The modulus, M, is plotted in Fig. 10, and average values through four discrete layers are indicated by vertical lines. Note that the original surface relative to the borehole is at a depth of 1.4 m.

It has been observed (Campanella *et al.* 1985) that when the membrane is returned to the closed position, the measured pressure p_2 represents the water pressure on the membrane. It is possible therefore to observe the rate of pore-pressure dissipation by measuring p_2 at intervals after stopping the penetration of the dilatometer. This was done at depths of 15 and 20 m. The dissipation of excess pore pressures $(p_2 - u_0)$ at a depth of 15 m is shown in Fig. 11 with respect to the square root of time. Based on the observed linear relationship it is taken that the initial excess pore pressure was 20.5 m. The final value would be at zero, and at 30% dissipation the value is 14.4 m and t_{30} = 1300 s.

The horizontal coefficient of consolidation can be estimated from the relationship, $c_h = 600\ T/t\ \mathrm{mm}^2/\mathrm{min}$, suggested by Schmertmann (1988) in which T is a "time factor" at time t after the start of dissipation. In this case, at a depth of 15 m and for a value of E/S_u = 100, the time factor after 30% dissipation is T_{30} = 0.47 and t_{30} = 1300 s (Fig. 11). Therefore, the calculated value for $c_h = (600/100) \times (0.47/1300) = 2.2 \times 10^{-3}\ \mathrm{cm}^2/\mathrm{s}$. At a depth of 20 m the estimated $c_h = 1.7 \times 10^{-3}\ \mathrm{cm}^2/\mathrm{s}$. These values are obtained in soil that has been overconsolidated by the insertion of the DMT, and the values for virgin consolidation are considered to be much smaller. For soils ranging from normally consolidated to lightly overconsolidated, Schmertmann (1988) suggests dividing the DMT value by 5, and the field value is therefore estimated at $c_h = 0.4 \times 10^{-3}\ \mathrm{cm}^2/\mathrm{s}$.

Piezocone tests (CPTU)

The Hogentogler Super (Hog Super) piezocone with pore pressures measured just behind the cone was used in this investigation. The cone was pushed in 1-m increments at a rate of 20 mm/s, and observations at 25-mm intervals during penetration are shown in Fig. 12. Pore-pressure dissipation tests were carried out at depths of 4, 6, 10, 16, 23, and 25 m. The equipment, test procedures, and correlations used for interpretation are described in Robertson and Campanella (1983).

As an example, several characteristics of the subsoil at a depth of 12 m were calculated from the test results. At this depth the overburden stress (σ_{v_0}) is approximately equal to 12 m × 17.8 kN/m^3 = 213 kPa, and the pore-water pressure (u_0) is 118 kPa, so the vertical effective stress (σ'_{v_0}) is 95 kPa. The cone bearing q_c = 5.92 bar (592 kPa), the friction ratio, R_f = 0.7%, and the pore pressure, u = 3.14 bar (314 kPa), were measured just

[1] p_0, membrane lift-off stress; p_1, stress to expand the membrane 1 mm; σ'_v, vertical effective stress before insertion of the DMT blade; and u_0, pore-water pressure before insertion of the blade.

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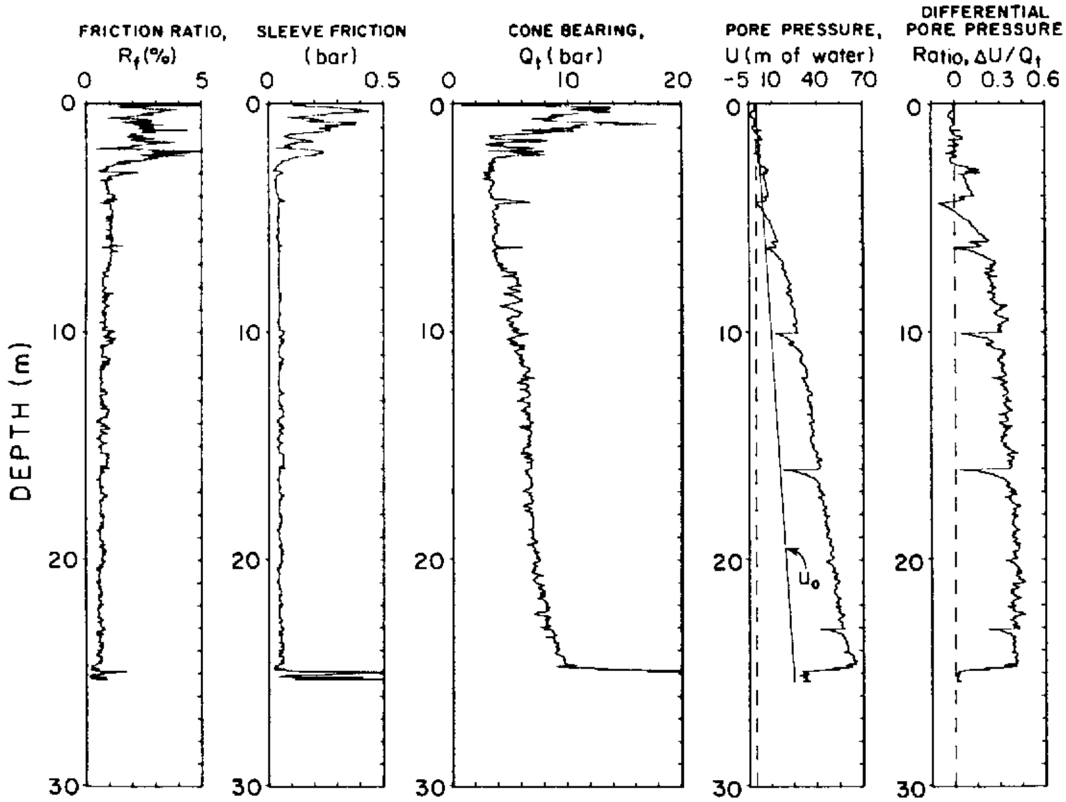

FIG. 12. Piezometer test (CPTU) (1 bar = 100 kPa).

behind the cone tip. The corrected cone bearing $q_T = q_c + u(1 - a) = 5.92 + 3.14(0.2) = 6.55$ bar (655 kPa), where $a = 0.8$ for the cone used (a is the net area ratio for pore effects; Campanella *et al.* 1983).

The undrained shear strength may be estimated from cone tests by the empirical relationship $S_u = (q_T - \sigma_{v_0})/N_{KT}$ where N_{KT} is a bearing-capacity factor obtained from site-specific correlations. Such a correlation was made at a depth of 12 m where the shear strength measured by the Geonor field vane was 28.6 kPa. The calculated value for $N_{KT} = (655 - 213)/28.6 = 15$. The average N_{KT} value from five tests in the upper 5 m of the profile is 21, and the average of 11 tests from a depth of 5–15 m is 14. The value of $N_{KT} = 14$ is the same as that reported by Greig *et al.* (1988) from a series of tests at Cloverdale, a few kilometres to the east.

The determination of a constrained modulus, M, from the cone test is estimated from correlations. The usual relationship is of the form $M = 1/m = \alpha q_c$ where α is related to soil type. For clays of low plasticity and $q_c < 7$ bar (700 kPa), Sanglerat (1979) suggests that α lies between 3 and 8. Since q_c is about 6 (near the upper end of the range), a value of 3–5 would be appropriate. Using an average of 4, the value of $M = 4q_c = 24$ bar (2400 kPa). The variation of M with depth is shown as a solid line in Fig. 13, and for comparison the values obtained with the DMT and from consolidation tests are also shown.

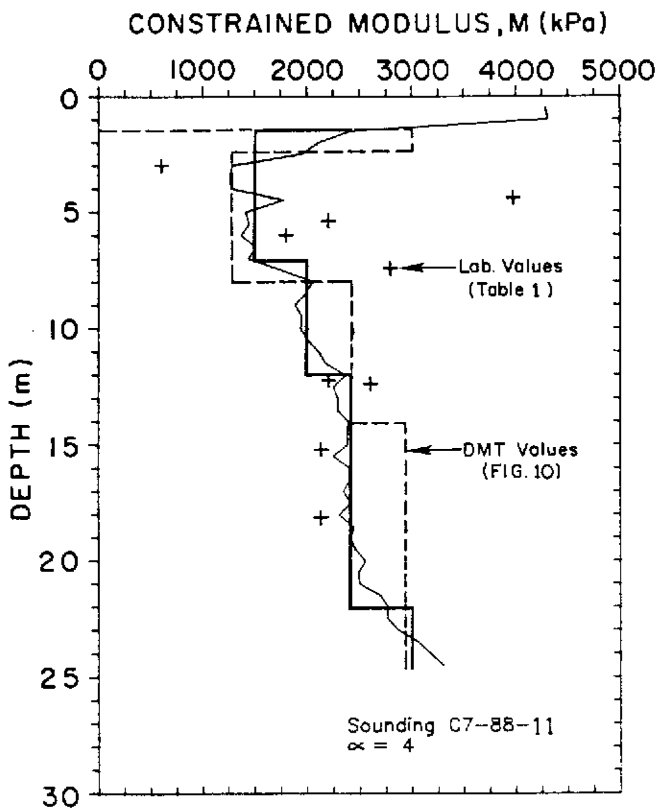

FIG. 13. CPTU constrained modulus (Colebrook).

The pore-pressure dissipation observed at the 10-m depth varies directly with the square root of time as shown in Fig. 14. From this curve the $t_{50} = 23^2 = 530$ s. On the basis of methods reviewed by Robertson and Campanella (1983) the coefficient of consolidation in the horizontal direction is $c_h = R^2T_{50}/t_{50}$, where R is the radius of the cone (1.8 cm) and T_{50} is a dimensionless time factor. A correlation attributed to Baligh and Levadoux (1980) and shown

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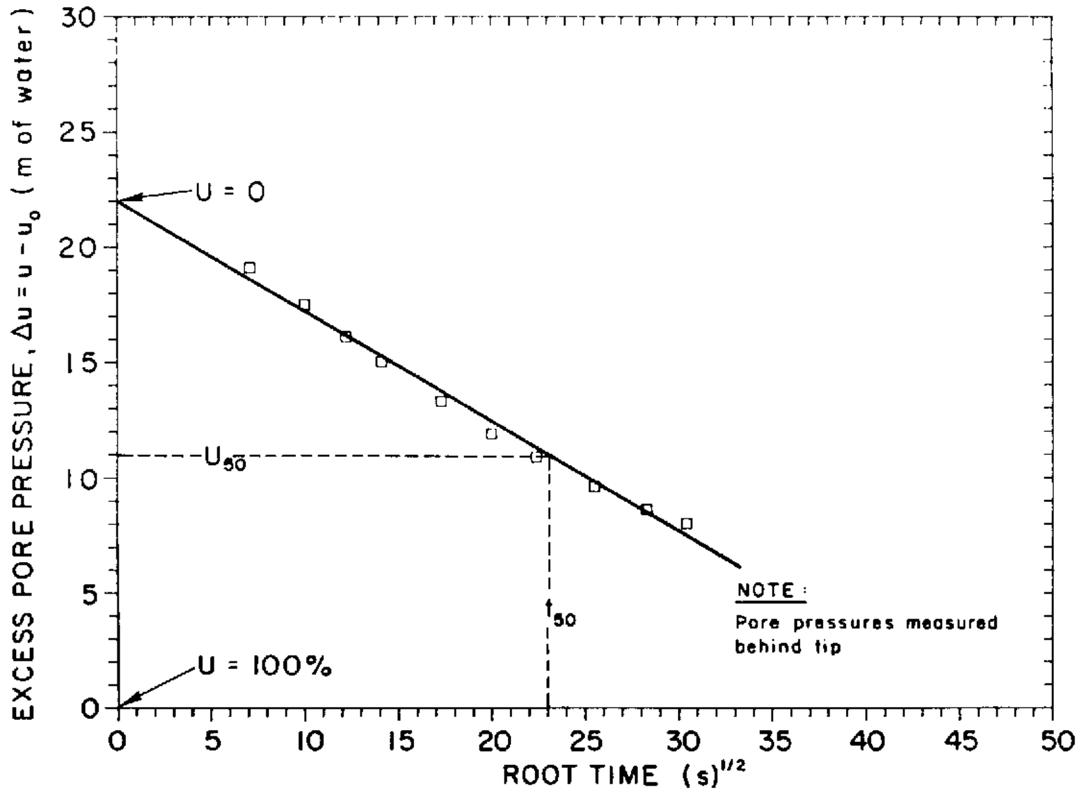

FIG. 14. CPTU dissipation test at 10 m depth (Colebrook).

as Fig. 4 in Robertson and Campanella (1983) gives a value for $T = c_h t_{50}/r^2 = 3.3$ for $\Delta u/\Delta u_0 = 0.50$. Therefore, $c_h = 3.3 r^2/t_{50} = 3.3(1.8)^2/530 = 20 \times 10^{-3}$ cm^2/s.

For dissipation less than 50% the consolidation occurs primarily in the recompression mode. To obtain the equivalent value for the normally consolidated (NC) mode, Jamiolkowski *et al.* (1983) suggest c_h (NC) = 0.15 c_h (CPTU), but Campanella *et al.* (1983) showed that for normally consolidated compressible clay silt in the Fraser River Delta the c_h (NC) = 0.25 c_h (CPTU) = $0.25 \times 20 \times 10^{-3} = 5.0 \times 10^{-3}$ cm^2/s. Values of c_h at depths of 16 and 23 m are 1.8 and 2.3 $\times 10^{-3}$ cm^2/s, respectively. The average value is about 3×10^{-3} cm^2/s.

Calculated settlements

In 1969 the primary consolidation under the shoulder of the embankment at station 67 was predicted from laboratory tests to be 1.02 m. Subsequent observations revealed that primary consolidation was completed in about 2 years (owing to sand drains), and the settlement was about 1.83 m (Crawford and deBoer 1987). During the following 13 years a further 0.12 m of secondary consolidation has occurred. New samples were obtained in 1988 and tested in the manner described earlier in this paper, giving results shown in Table 1.

Laboratory tests

The calculated primary consolidation settlement under the shoulder of the embankment at station 67, assuming normally consolidated subsoil for the entire depth and using the 1988 values in Table 1, is 1.10 m. A simple one-dimensional settlement analysis with elastic stress distribution was used. The more sophisticated sampling and testing has therefore not significantly improved the discrepancy between measured and calculated values, and the observed primary settlement (1.83 m) is much greater than the calculated value.

Dilatometer test

The calculated settlement is the summation of the contributions of individual layers having the constrained moduli shown in Fig. 10 and under the applied stresses shown in Fig. 3. This results in a calculated settlement of 0.75 m (Table 2). However, the DMT test was made at a location where the natural soil was consolidated under a stress of 26 kPa owing to the original road, and consequently the measured moduli will be greater than those that would have been measured in virgin ground. To correct for this it is assumed that the calculated settlement is due to an applied stress increase of only 90 − 26 = 64 kPa, and the corrected settlement would be (90/64) × 0.75 = 1.05 m.

Piezocone test

The calculated settlement using the constrained moduli shown in Fig. 13 and the stress increases shown in Fig. 3 is 0.80 m, as shown in Table 3. Applying the same reasoning used with DMT results, the corrected settlement would be (90/64) × 0.80 = 1.13 m. It should be noted that the constrained modulus calculated from cone tests varies directly with α, which is based on crude correlations with q_c. In this case, $\alpha = 4$ was considered appropriate, but if $\alpha = 5$ had been selected the calculated settlement would be 0.64 m, and the corrected value would be only 0.90 m. On the other hand, if $\alpha = 3$ had been chosen, the corrected value for settlement would be 1.50 m.

In summary the settlements calculated from DMT and CPTU tests agree reasonably well with the value using laboratory tests, but the average value is still only about 60% of the observed settlement.

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TABLE 2. Settlement estimates from DMT results

Layer	ΔH (m)	$\Delta\sigma$ (kPa)	M (kPa)	Settlement (m) Calculated	Settlement (m) Corrected
1	1.1	88	3000	0.03	0.04
2	5.5	82	1300	0.35	0.49
3	6	70	2400	0.17	0.24
4	11	52	2900	0.20	0.28
Total				0.75	1.05

TABLE 3. Settlement estimates from piezocone results

Layer	ΔH (m)	$\Delta\sigma$ (kPa)	M (kPa)	Settlement (m) Calculated	Settlement (m) Corrected
1	5.6	84	1500	0.31	0.44
2	5	75	2000	0.19	0.27
3	10	59	2400	0.26	0.36
4	3	37	3000	0.04	0.06
Total				0.80	1.13

Rate of consolidation settlement

The calculated rate of consolidation settlement depends on the measured value of the coefficient of consolidation and on the assumed drainage conditions. In this case sand drains were used, so the drainage, except near the surface of the clay, must be horizontal and the length of the drainage path is known. It is possible therefore to compare the actual coefficient of consolidation derived from settlement observations with values measured in the laboratory and *in situ*.

In the earlier study (Crawford and deBoer 1987) it was shown that approximately 90% of the primary consolidation at station 67 was completed 600 days after loading began. But taking into account the rate of load application, an observed time of 500 days would be more appropriate, and the back calculated value for c_h is 2.7×10^{-3} cm^2/s.

The new consolidation test results shown in Table 1 give an average c_v of 1.6×10^{-3} cm^2/s. The value for c_h will be larger than c_v, but the amount is unknown. If the ratio is between 2 and 3 (a common assumption), the average c_h is 4×10^{-3} cm^2/s, and the calculated time for 90% consolidation, assuming lateral drainage into the sand drains, is about 340 days. The average laboratory value for c_h is therefore about 50% greater than the observed value. The observed value may have been reduced owing to smearing of the clay during sand-drain installation, but the drains were installed by the Stang Jet method, which is thought to minimize this effect.

The average value for c_h quoted earlier for the CPTU tests was about 3×10^{-3} cm^2/s, or about 10% greater than the estimated *in situ* value. The average value quoted for the DMT tests was $c_h = 0.4 \times 10^{-3}$ cm^2/s or about one-sixth of the *in situ* value.

Summary and conclusions

The agreement between calculated one-dimensional primary consolidation settlement using laboratory tests and *in situ* (DMT and CPTU) tests is remarkably good at this site, but the calculated values are all considerably less than the measured values under the earth embankment. The reasons for this discrepancy are not known. There are some indications that the subsoil is slightly overconsolicated, but if this is true the calculated settlement using laboratory tests would be even less and the discrepancy would be greater. On the other hand, there may have been some thin layers of compressible peat under the embankment, although such deposits were not reported in the original soil investigations.

It is very likely that a significant part of the measured settlement may have been due to lateral spreading under the shoulders of the fill in the very sensitive subsoil, but there are no measurements to confirm this possibility. An attempt was made to measure lateral movement on the surface during construction by observing a line of stakes on either side of the embankment, but there was no discernible movement. This does not preclude the possibility of subsurface spreading, but more elaborate instrumentation would be required to detect such movements.

Another possibility is that the construction of the sand drains caused disturbance of the subsoil and that this resulted in increased consolidation settlement. There is some evidence to support this view because areas adjacent to the sand drains appeared to settle a little less (Crawford and deBoer 1987). Unfortunately, it is not possible to compare the two areas precisely because the areas without sand drains were surcharged and the degree of consolidation was not known when the surcharge was removed.

Although there is good agreement among the prediction methods, the comparison with observed settlement is not satisfactory. Further field research is needed to determine how movement occurs, both vertically and laterally, and laboratory studies on better soil samples are required to improve understanding of the material properties.

The coefficient of consolidation measured with the CPTU is quite satisfactory. It is in better agreement with the value derived from settlement observation than either the laboratory or DMT values.

Many engineers will be skeptical of the rationalism of predicting long-term consolidation settlements from very short-term tests, and a measure of caution is certainly appropriate. It is, for example, not possible using rapid *in situ* tests to assess the influence of time-related structural breakdown in very sensitive soils. Furthermore, the DMT and CPTU values are obtained from correlations that are site specific and extrapolations to other sites may be misleading. For this reason it is necessary to compare these kinds of predictions with full-scale observations in a variety of soil regions to determine the reliability of the methods and to build confidence when this is warranted.

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