Settlement of an embankment on Leda clay

Burn, K. N.; Hamilton, J. J.

Publisher's version / Version de l'éditeur:

Canadian Geotechnical Journal, 5, 1, pp. 17-27, 1968-04-01

NATIONAL RESEARCH COUNCIL OF CANADA
CONSEIL NATIONAL DE RECHERCHES DU CANADA

SETTLEMENT OF AN EMBANKMENT ON LEDA CLAY

K. N. BURN and J. J. HAMILTON

Reprinted from
CANADIAN GEOTECHNICAL JOURNAL
Vol. V, No. 1, February 1968
pp. 17–28

Research Paper No. 361
of the
Division of Building Research

OTTAWA
April 1968

Price 25 cents

NRC 10082

SETTLEMENT OF AN EMBANKMENT ON LEDA CLAY

K. N. BURN and J. J. HAMILTON *Division of Building Research, National Research Council of Canada*

Before construction, standard consolidation tests indicated that the stresses imposed by a high embankment on a deep and relatively homogeneous deposit of Leda clay would exceed the preconsolidation stress and result in settlements of up to 18 in. Later, the results of more refined sampling and testing methods showed that the preconsolidation pressure had been underestimated by 1 ton/sq. ft. and that the imposed stresses would in fact fall within the recompression range. Time-settlement relationships, however, were not elastic; vertical strains continued to increase long after the embankment was completed. Field measurements of settlement are compared with values computed using compression moduli determined both from consolidation and compression tests in the laboratory.

Des essais de consolidation sur des échantillons obtenus avant le début des travaux avaient indiqué que les contraintes créés par un haut remblai érigé sur une épaisse couche d'argile Leda relativement homogène surpasseraient les pressions de préconsolidation et entraineraient des tassements allant jusqu'à 18 pouces. Les résultats de techniques d'échantillonage et de méthodes d'essais améliorées ont démontré par la suite que la pression de préconsolidation avait été sous-estimée d'une tonne au pied carré et qu'en fait les contraintes imposées seraient dans le domaine de recompression. Les rapports de tassements en fonction du temps, cependant, ne sont pas élastiques ; les déformations verticales ont continué d'augmenter longtemps après que le remblai fut érigé. Les mesures des tassements sur place sont comparées aux valeurs obtenues en utilisant les modules de compression basés sur les essais de consolidation et de compression en laboratoire.

Significant discrepancies have been found between predicted and measured settlements of structures founded on Leda clay. Crawford (1953) found that the actual settlement measured at the National Museum Building in Ottawa was less than one-half that predicted by the conventional consolidation theory using the sampling and testing techniques of fifteen years ago. Fortunately, these discrepancies tend to increase rather than reduce the safety factor in foundation designs. More recently the greater foundation loads from multistorey buildings or earth embankments have increased the need for a more reliable evaluation of the preconsolidation pressure and the prediction of recompression and virgin compression settlements.

Improvements in sampling equipment and laboratory apparatus have reduced

Canadian Geotechnical Journal, vol. V, no. 1, Feb. 1968. Printed in Canada.

the effects of disturbing the structure of this sensitive clay. Laboratory investigations, such as reported by Hamilton and Crawford (1959) and by Crawford (1964), have shown that testing technique can have a great influence on the measured preconsolidation pressure. Final evaluation of these variables must, however, be made from field studies.

An opportunity for further study of the field consolidation characteristics of Leda clay in the Ottawa area occurred in 1957, with the construction of a 23 ft. high earth embankment over a deep relatively homogeneous deposit of clay. The results of tests on samples obtained commercially had indicated that the increased subsoil stresses would be in the range slightly above the preconsolidation pressure and a total settlement of 1.5 ft. was predicted. Prior to construction, a more extensive field and laboratory soil investigation was begun and settlement gauges were installed.

Construction was carried out in several stages, but the bulk of the embankment was placed in October and December 1957. A height of 10 ft. was reached in October before heavy rains delayed the work. This is referred to as Stage I. Conditions in December permitted construction to continue and the embankment was raised to 19 ft. at the centre (Stage II). Smaller loads were added between March 1959 and August 1960 during the construction of the overpass and preparation of the subgrade for paving.

A plan of the completed interchange showing the location of the instrumentation is shown in Figure 1.

SOIL PROPERTIES

An extensive program was designed to obtain *in situ* strengths and high quality samples for laboratory testing and to install sufficient instrumentation to determine the settlement of the fill and its foundation. Field vane tests throughout the entire depth of the deposit yielded information for embankment stability analysis. Tests were conducted at 1½ ft. intervals to a depth of 99 ft., using the equipment and procedures described by Eden and Hamilton (1956). The deposit was then sampled by means of a fixed-piston sampler, patterned after a Norwegian device. In this way samples were obtained up to 0.9 m in length in stainless steel, thin-wall tubes, 5.5 cm in diameter. Great care was exercised in obtaining these samples; recovery was excellent.

General properties

As seen from the geotechnical profile in Figure 2, the deposit is relatively uniform and homogeneous in comparison with other deposits in the Ottawa area (see, for example, Eden and Crawford 1957). The deposit shows a very slight decrease in water content with depth with typically wide scattering of values in contiguous samples.

The samples, when examined in the laboratory, showed fissuring or preferential planes of weakness to a depth of approximately 36 ft. There is no abrupt change in water content or marked change in appearance of the soil at this depth. In other deposits, this fissuring has been attributed to the effects of weathering and negative pore pressures induced by vegetation. In this case, however, the preconsolidation pressure does not show the characteristic

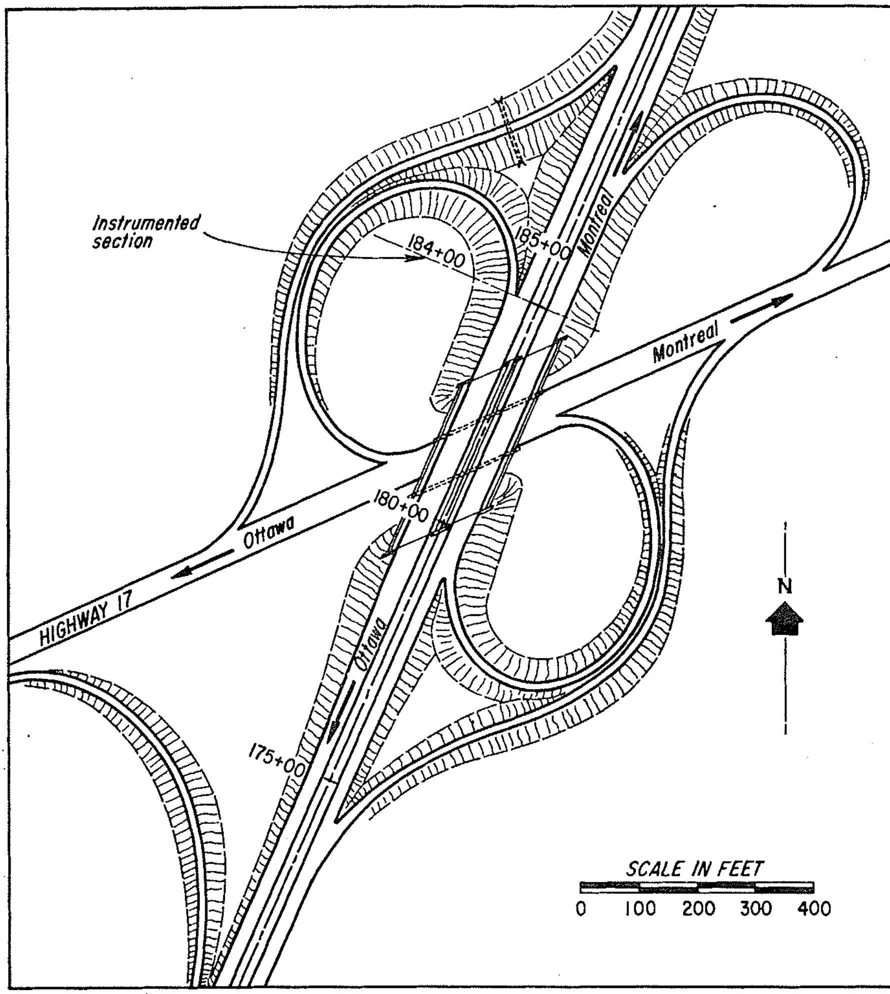


FIGURE 1. Ottawa Queensway–Highway 17 interchange

increase, but rather a substantial decrease from the 19 ft. depth to the 11 ft. depth. (This might be the result of softening and stress release associated with the removal of tree cover about 120 years ago.)

Sensitivity values determined with a laboratory vane apparatus are also shown on Figure 2. Considering the large range in salt concentration, the measured variation is small. The sensitivities of these samples are, in general, in the lower range of past experience. The relationship between the logarithm of sensitivity and liquidity index agrees reasonably well with the results previously reported by Bjerrum (1954), and Eden and Hamilton (1956).

Compressibility

In order to calculate settlements, it is necessary to obtain good estimates of the preconsolidation pressure and the stress-strain-time relationships in the loading range from the overburden pressure to the maximum pressure after

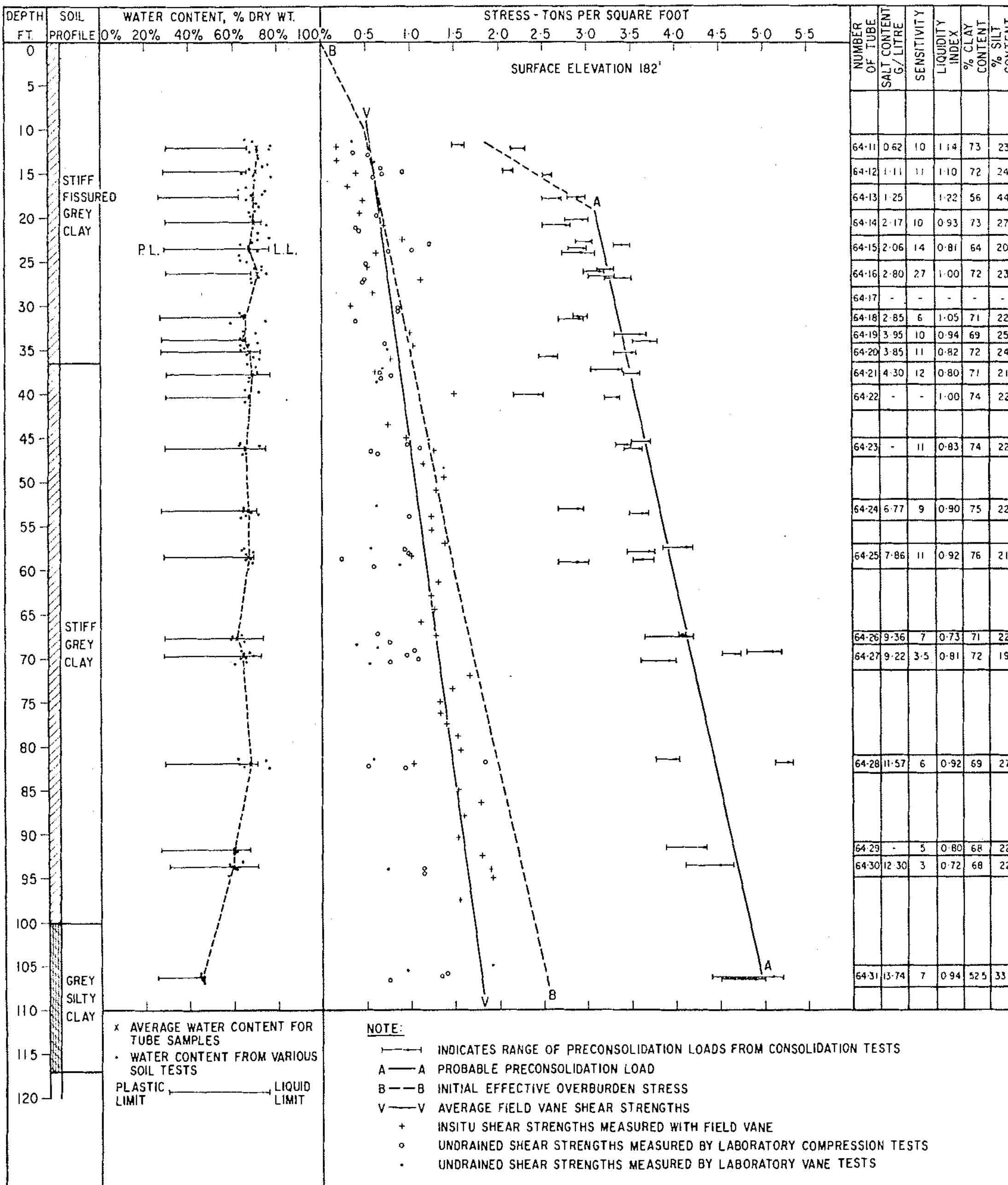


FIGURE 2. Geotechnical profile

construction. Unfortunately, the usual test that has an increment ratio of unity does not fulfil either requirement. Reducing the size of the load increment provides a curve from which a better estimate of the preconsolidation pressure may be made, but the end of primary compression as determined by a curve-fitting operation becomes less definite and results in less reliable time-compression predictions. The size of the increment ratio appears to have no effect on the total amount of compression for a given pressure increment in the virgin compression range, provided the method of terminating load increments is based on a selected creep rate as already discussed (Hamilton and Crawford 1959). More recent research has shown that when load increments are not terminated on a creep rate basis, the pressure void ratio can be affected by the duration of the tests (Crawford 1964).

The first series of consolidation tests was run on samples from various depths representing the entire profile. The results of these preliminary tests were used as a guide in designing later tests. They indicated that the preconsolidation pressure was considerably higher than that obtained in the preliminary investigations. Subsequent tests were performed to investigate the influence of increment size and duration on total compression and on the preconsolidation pressure, the effects of load cycling and various degrees of rebound on the recompression branch, and methods to determine the end of primary consolidation. Some of the conclusions of these and other related studies were reported by Hamilton and Crawford (1959). These factors were taken into account in establishing the most probable preconsolidation pressure in the natural soil as shown by line *A-A* in Figure 2.

Shear tests

Three types of shear strength tests were performed on a large number of samples: unconfined compression, unconsolidated undrained triaxial, and vane tests. Results are shown in Figure 2.

Compression moduli

Values of compression moduli calculated both from consolidation and compression tests are listed in Table I. They were determined in the following ways.

Modulus M_R is calculated from the slope of the recompression branch of the arithmetic pressure-void ratio plots, after correction for compressibility of apparatus. It is equivalent to the coefficient of compressibility in the recompression range. $M_{\Delta P}$ is defined as the recompression modulus calculated from the stress-strain relationship for a single load increment close to the pressure increase caused by the embankment. Both moduli were calculated from one-dimensional consolidation test results.

Several load-rebound cycle tests in the load ranges slightly below and slightly above the preconsolidation pressure were conducted on samples from various depths. These showed that the values for M_R were reproducible as long as rebound was not allowed to proceed as far as a zero load condition with free water available. If this was allowed, the recompression moduli

TABLE I
Comparison of compression moduli (calculated from laboratory tests)

Tube No.	64-11	64-12	64-13	64-14	64-15	64-16	64-17
Depth	10'-7" to 13'-4"	13'-4" to 16'-2"	16'-2" to 19'-1"	19'-1" to 22'-0"	22'-0" to 24'-11"	24'-11" to 27'-10"	27'-10" to 30'-5"
Sample No.							
M_R (psi)	6: 1,180 8: 1,390			6: 2,180 8: 2,090		5: 4,420 6: 3,550	
$M_{\Delta P}$ (psi)	6: 1,010 8: 1,200	7: 2,590 9: 5,430	6: 2,800 7: 1,820	6: 2,170 8: 2,410	4: 2,200 6: 5,960 8: 2,450 10: 2,910	6: 2,490 8: 1,090 9: 2,840	
M_{50} (psi)	σ_3 10: 9.25 — 600 11: 18.5 — 800	σ_3 5: 10.0 — 4,400 6: 0 — 1,190 8: 30.0 — 1,780 10: 0 — 3,800	σ_3	σ_3 10: 12.5 — 8,700 11: 25.0 — 800	σ_3 5: 12.5 — 6,800 7: 0 — 1,770 9: 45.0 — 1,320 11: 0 — 1,720	σ_3 10: 13.6 — 1,800 11: 27.2 — 2,400	σ_3 1: 0 — 1,140 2: 0 — 2,040

Tube No.	64-18	64-19	64-20	64-21	64-22	64-23	64-24
Depth	30'-5" to 32'-4"	32'-4" to 34'-3"	34'-3" - 36'-5"	36'-5" to 38'-11"	38'-11" to 41'-10"	45'-0" to 47'-2"	52'-0" to 54'-5"
Sample No.							
M_R (psi)	5: 4,510 6: 8,020		6: 3,360 8: 1,224		6: 2,550	3: 5,020 5: 5,430 5: * 2,715	6: 8,120 8: 3,100
$M_{\Delta P}$ (psi)	5: 2,960 6: 4,440	5: 2,160 8: 2,840	6: 3,520 8: 1,300	5: 1,980 7: 5,310	6: 1,250 8: 2,330	3: 5,230 5: 5,630 5: * 2,940 7: 3,400	6: 1,460 8: 2,960
M_{50} (psi)	σ_3 7: 12.0 — 1,150	σ_3 9: 15.5 — 2,950	σ_3	σ_3 6: 17.0 — 4,900 8: 34.0 — 2,540 9: 0 — 3,600	σ_3	σ_3 4: 19.6 — 2,880 6: 40.0 — 3,800 8: 0 — 1,300 9: 0 — 1,350	σ_3 10: 26.7 — 1,230

Tube No.	64-25	64-26	64-27	64-28	64-29	64-30	64-31
Depth	57'-0" to 59'-11"	67'-0" to 68'-4"	68'-4" to 71'-3"	81'-0" to 83'-0"	91'-0" to 92'-4"	92'-4" to 94'-10"	105'-0" to 107'-6"
Sample No.							
M_R (psi)		3: 2,560 4: 2,025		4: 6,480	3: 2,915		
$M_{\Delta P}$ (psi)	3: 3,235 5: 4,190 9: 3,360 11: 1,520	3: 2,730 4: 2,060	5: 2,650 6: 2,320 10: 1,475	4: 3,130	3: 3,570	6: 1,315	6: 2,850
M_{50} (psi)	σ_3 4: 23.6 — 2,640 6: 0 — 3,350 8: 47.0 — 1,300 13: 23.6 — 1,100	σ_3 2: 35.0 — 1,360 6: 45.0 — 2,600	σ_3 4: 27.0 — 2,950 7: 61.0 — 3,700 8: 0 — 3,600 11: 0 — 7,200	σ_3 5: 34.0 — 6,600 7: 0 — 1,370 8: 50.0 — 2,800	σ_3	σ_3 4: 34.4 — 2,500 5: 0 — 400 7: 65.0 — 300 10: 34.4 — 3,150	σ_3 4: 38.3 — 5,200 5: 0 — 3,550 8: 75.0 — 4,200 9: 0 — 3,100

obtained dropped considerably below those calculated for the first laboratory loading through the recompression range. For example, see calculated values of M_R and $M_{\Delta P}$ for sample 64-23-5 marked with an asterisk in Table I.

A third modulus, M_{50}, or secant modulus, was calculated from unconsolidated-undrained triaxial compression tests having various confining pressures with no control of lateral strain.

Calculations were also made on an "instantaneous modulus" from consolidation test results and an initial tangent modulus from compression test results. Corrections for compressibility of the apparatus in the former were of the same magnitude as the "instantaneous" compression in the sample. This resulted in a wide range of calculated moduli which in some instances were negative. Interpretation of initial tangent modulus in the latter was also difficult and subject to considerable judgement.

SETTLEMENT ANALYSIS

Stress distribution

The distribution of applied stresses was computed from the Boussinesq theory using Newmark (1942) charts. For comparison, the stresses beneath the centreline were determined from the Westergaard equations. The maximum computed stress shown in Figure 3 is well below the estimated preconsolidation stress of the subsoil. The influence of shape of the loaded area was also investigated. Although it differed considerably from a model having a uniform trapezoidal cross-section of infinite length, this does not appear to have altered the distribution of stresses significantly.

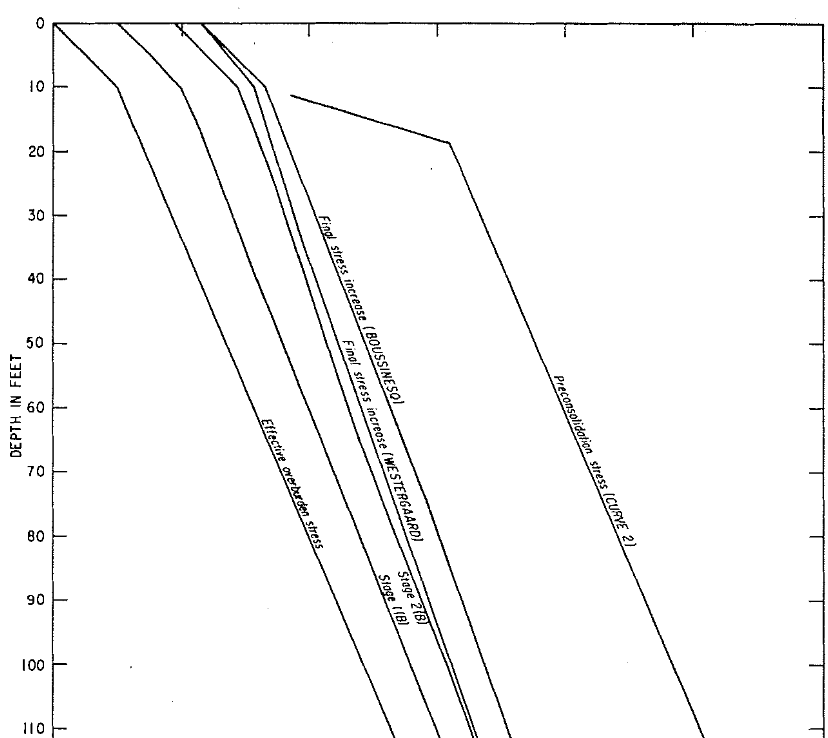
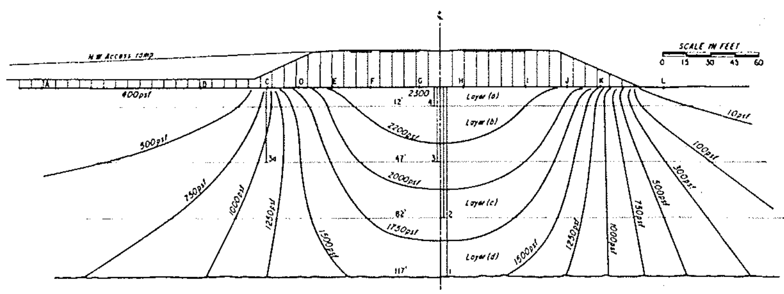


FIGURE 3. Stresses beneath the centreline at station 184+00

When compared, no difference was found east of the centreline, but the access ramps, and shallow fill within the northwest loop which was added to reverse the natural drainage, produced small additional stresses that affected everything west of the centreline (Figure 4a). No appreciable increase in stress on this section was caused by the other access ramps.



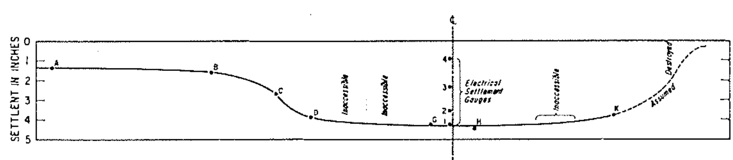


FIGURE 4a. Distribution of stress increase at instrumented cross-section (Boussinesq)
FIGURE 4b. Transverse total settlements, June 1962

Computed settlements

Laboratory values of compression moduli inherently comprise the effects of time-dependent strains in proportions that vary with applied rates of loading. Consequently, when they are used to calculate elastic field settlements, the resulting values should be expected to be erroneously high. In this study, a value of 5.6 in. was obtained when the average values from each layer were used in the calculation (Table III). When only the highest values for moduli were used, realizing that this property was very sensitive to test seating errors but that these higher values were probably more realistic, an elastic settlement of 3.1 in. was obtained.

Actual settlements

In order to measure settlements beneath this high embankment several special remote reading gauges were installed (Burn 1959). These consisted of measuring heads "floating" at the natural ground surface and centred over the tops of vertical pipes that were secured in place by earth anchors at their lower ends (Figure 5). Compression is measured by a simple device that changes electrical resistance with relative displacement between "floating" head and anchor. Total compression between the surface and the lower limit of the four layers *A*, *B*, *C*, and *D* are plotted against time in Figure 6. The compression of individual layers is determined from the differences. (The broken lines repre-

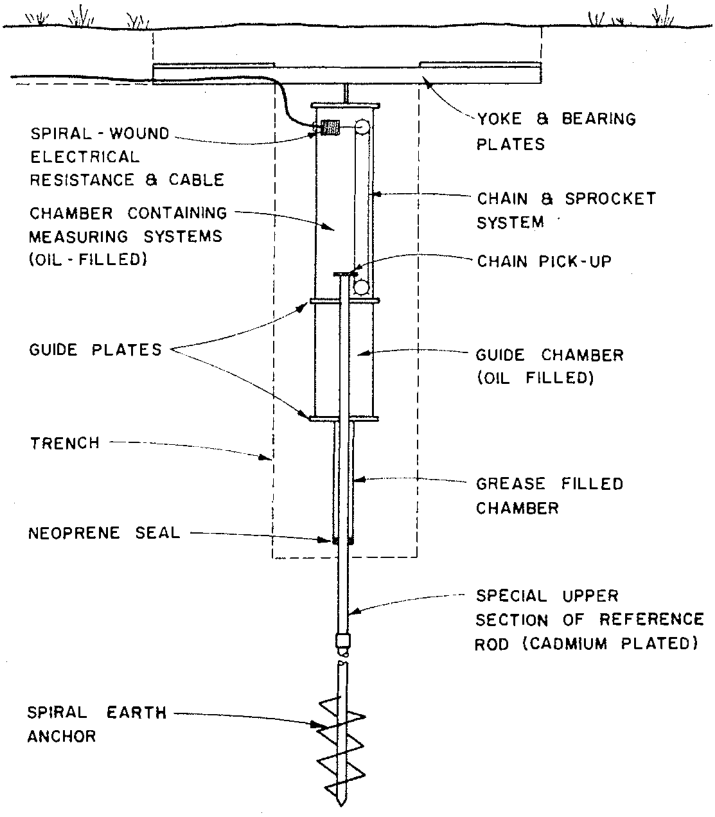


FIGURE 5. Schematic drawing of the settlement gauge showing its components

sent a period during which the gauges could not be read because of flooding in the northwest access loop.)

Less than half of the total settlement observed at the end of 1964 occurred when the embankment was first constructed (Stages I and II). Some further settlement developed at a slow rate in the 1½ years that elapsed before the building of the overpass and the backfilling behind the abutments. This added load caused a rapid settlement followed by a slower rate which accelerated briefly during the placing of a sand cushion on top of the clay fill. Settlement continued to increase and four years after the eastern section of the Queensway was opened, a further settlement of 1 in. had taken place.

Before construction began, several concrete pads were cast just below natural ground surface in a line normal to the centreline of the Queensway at the location of the electrical settlement gauges. They were placed so that it would be possible to make borings down to them from between the paved surfaces of the completed roadway—in order to determine the variation of total settlement from one side of the embankment to the other. The levels obtained in one such survey showed the effect of stress increase on the section and that of the shallow fill in the northwest access loop (Figure 4b).

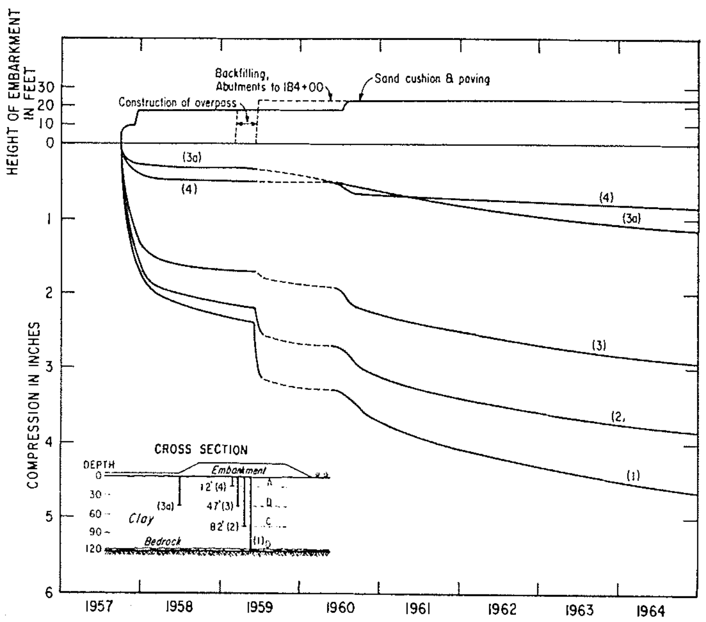


FIGURE 6. Compression with time

COMPARISON OF LABORATORY AND FIELD RESULTS

Values of compression moduli were calculated for each layer by assuming that vertical movements observed during the loading (Stages I and II) were essentially elastic (Table II). Small creep movements no doubt are included so that these values may be expected to be a little low. Except for those in layer *B*, however, they are several times greater than the average laboratory values. When used to calculate the elastic settlement under full load the field moduli yield a value of only 2.05 in. (Table III). It follows that about 3 in. of vertical movement is attributable to creep.

The values of compression modulus obtained by field measurements for layer *C* agree quite closely with the value reported by Bozozuk (1963) in a study of the rebound of the equivalent layer when a deep excavation was made ½ mile from this site.

In attempting to determine the elastic settlement from maximum laboratory values of compression moduli, it was found that the contributions of the individual layers did not resemble those later observed in the field, and such a procedure could not be considered reliable. On the other hand, the average laboratory values yield predicted elastic settlements that are almost twice those observed but they are approximately in the same proportions of the total as those observed (Table III). By coincidence, the magnitude of the elastic settlement calculated from average laboratory moduli (5.56 in.) approximates the total observed settlement (4.63 in.). This, of course, includes the rather large proportion of creep deformations.

TABLE II
Field and laboratory compression moduli

		Compression modulus from settlement readings and calculated stress increase (lb./sq. in.)						
		"immediate"				Modulus from laboratory tests		
		Stage I		Stage II				
Layer	Depth (ft.)	Wester-gaard	Boussi-nesq	Wester-gaard	Boussi-nesq	Average lb./sq. in.	Highest value lb./sq. in.	No. of tests in group
A	0–12	4300	4500	4300	4500	1000	1400	4
B	12–47	4800	5600	4200	4900	3100	8700	62
C	47–82	8600	11,000	10,250	13,200	3100	8100	31
D	82–117	17,800	23,900	26,600	35,300	3600	5200	12

CONCLUSION

The earth embankment described in this paper is founded on moderately overconsolidated Leda clay. The actual amount of overconsolidation was underestimated in the preliminary investigation and the predicted settlement consequently was much greater than observed. The tests reported here show that the applied load is entirely in the recompression range of stress and that the settlements are not caused by primary consolidation. The measured settlements are of particular interest because the stresses fall within a realistic range of those induced by building foundation loads.

Although the test results show that field stress increases occur within the recompression range, the records show that settlements increase long after the application of full load. These settlements, which are attributable to "creep," are of the same magnitude as immediate elastic settlements.

Compression moduli calculated from loading conditions during the first few weeks of construction and the corresponding settlement readings, are several times greater than the average values derived from the laboratory compression

TABLE III
Observed and computed settlements

		Compression of layers under final load					
		*Calculated using				Observed October 1964	
Layer	Depth (ft.)	Lab modulus	(avg.)	Field modulus			
A	0–12	2.04″	(37%)	0.49″	(24%)	0.85″	(18%)
B	12–47	1.54″	(28%)	1.06″	(52%)	2.07″	(45%)
C	47–82	1.21″	(22%)	0.37″	(18%)	0.97″	(21%)
D	82–117	0.77″	(14%)	0.13″	(6%)	0.74″	(16%)
Total	0–117	5.56″	(100%)	2.05″	(100%)	4.63″	(100%)

*Using Boussinesq stress distribution.

tests for each layer. Use of average laboratory "compression moduli" leads to overestimating the immediate settlement, but gives close agreement with observed total settlement, even though creep effects are significant.

ACKNOWLEDGMENT

This paper is a contribution from the Division of Building Research, National Research Council of Canada, and is published with the approval of the Director of the Division.

REFERENCES

Bjerrum, L., 1954. Geotechnical properties of Norwegian marine clays. *Géotechnique 4*, 2: 49–69.

Bozozuk, M., 1963. The modulus of elasticity of Leda clay from field measurements. This Journal 2, 1: 43–51.

Burn, K. N., 1959. Instrumentation for a consolidation study of a clay deposit beneath an embankment. *Géotechnique* 9, 3: 136–42.

Crawford, C. B., 1953. Settlement studies on the National Museum Building, Ottawa, Canada. *Proc. 3rd Int. Conf. Soil Mech. and Found. Eng.* (Zurich) I: 338–45.

Crawford, C. B., 1964. Interpretation of the consolidation test. Conference on "Design of Foundations for Control of Settlement." *Proc. ASCE*: 93–108.

Eden, W. J., and Hamilton, J. J., 1956. The use of a field vane apparatus in sensitive clay. ASTM STP 193: 41–53.

Eden, W. J., and Crawford, C. B., 1957. Geotechnical properties of Leda clay in the Ottawa area. *Proc. 4th Int. Conf. Soil Mech. and Found Eng.* (London) I, a/6: 22–7.

Hamilton, J. J., and Crawford, C. B., 1959. Improved determination of preconsolidation pressure of a sensitive clay. ASTM STP 254: 254–71.

Newmark, N. M., 1942. Influence charts for computation of stresses in elastic foundations. *University of Illinois Bull. 40*, 12.

Westergaard, H. M., 1938. A problem of elasticity suggested by a problem in soil mechanics: Soft material reinforced by numerous strong horizontal sheets. Stephen Timoshenko 60th Anniversary volume (Macmillan, New York).