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**SETTLEMENT OF A HIGH EMBANKMENT AND OVERPASS
STRUCTURES IN OTTAWA**

BY

K. N. BURN

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SETTLEMENT OF A HIGH EMBANKMENT AND OVERPASS STRUCTURES IN OTTAWA^{1,2}

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Settlement observations have been conducted over a 5-year period on a 20-ft-high earth embankment and 2 highway overpass structures founded on spread footings over a thick deposit of Leda clay. The applied stresses were considerably less than the overconsolidation pressure of the clay. In the five years since construction, settlements of approximately 12 cm have occurred, of which about 4 cm occurred immediately. The observed settlements are compared with settlement predictions on the basis of various types of laboratory tests. The observations indicate that significant settlements occur on a long-term basis and can be attributed to a slow yielding of the clay structure. Piezometers failed to detect any excess pore water pressure due to slow process of consolidation.

Des mesures de tassement ont été réalisées pendant une période de cinq ans sur un remblai de 20 pieds de hauteur et sur deux structures de ponts fondées sur semelles sur un dépôt épais d'argile Leda. Les contraintes appliquées étaient nettement inférieures à la pression de surconsolidation de l'argile. Cinq ans après la fin de la construction, des tassements d'environ 12 cm se sont produits, dont 4 cm immédiatement. Les tassements observés sont comparés aux calculs établis à partir de différents types d'essais de laboratoire. Les observations indiquent qu'une part importante du tassement se produit à long terme et peut être attribuée à un fluage lent de la structure de l'argile. Les piézomètres n'ont indiqué aucune pression interstitielle du fait de la lenteur du phénomène de consolidation.

INTRODUCTION

The prediction of settlements resulting from foundation stresses less than the measured preconsolidation stresses in the subsoil is an important engineering problem. Within this stress range, actual settlements may be small and relatively insignificant for structures founded on comparatively stiff soils (Crawford and Burn 1962) or they may be large and extremely critical when dealing with open-structured and sensitive materials such as Leda clay. Improved sampling and testing in recent years have led to better evaluation of engineering properties that appear to be more realistic in terms of field experience. Comparatively little is known, however, of the slow structural breakdown or creep distortions that accompany the application of stresses that are well below the overconsolidation stress.

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FIG. 1. Aerial view of Queensway.

Construction in 1958 of a 23-ft earth embankment over a deep deposit of Leda clay provided an opportunity to observe settlements and compare them with those determined from laboratory tests. This study showed that the measured settlements were not the result of primary consolidation. (The applied stress was about 45% of the overconsolidation stress, i.e. $\Delta p / (p_c - p_0) = 0.45$.) Nor were the settlements elastic: they continued to increase long after construction was complete (Burn and Hamilton 1968).

With the construction of Stage IV of the Ottawa Queensway in 1963–65, it became possible to study settlements occurring beneath a fairly uniform section of embankment 20 ft high and 600 ft long, at which the applied stress represents about 65% of the overconsolidation stress. Since construction was completed, vertical movements and piezometric levels in the deposit have been observed. Two overpass structures 1000 ft and 1600 ft west of this section were also instrumented for settlement observations.

The Ottawa Queensway in this area follows the right of way of a cross-town railway line that was removed specifically for its construction. It passes just south of two buildings on which earlier settlement studies were conducted: the National Museum Building (Crawford 1953), and the Tiffany apartment block (Legget *et al.* 1961). These two structures can be identified in Fig. 1.

SOIL PROPERTIES

In July 1962 continuous samples were obtained between the depths of 8 ft and 68 ft, using a thin-walled fixed piston sampler 5.5 cm in diameter. In situ shear strengths and sensitivities were measured with a field vane 4.3 by 8.6 cm.

The bore holes were located only a few feet from station 425+00 of the Queensway, which falls very close to the mid-point of the 600-ft-long embankment.

Laboratory and field test results are shown in the geotechnical profile in Fig. 2. Classification tests indicate that, apart from the surface covering of sand, the upper 30 ft (layer A) is a highly plastic silty clay with water contents generally at or slightly above the liquid limit. Field vane sensitivity is about 20.

The material between 30 ft and 60 ft (layers B and C) is a silty clay of low plasticity and higher sensitivity (20 to 40), with water contents well above the liquid limit. The liquidity index is generally around 2.

Below 66 ft (layer D) the material is a compact silt resting on a thin cover

of very dense sandy till lying on shale bedrock at a depth of 98 ft. It was impossible to penetrate this compact silt either with the piston sampler or the vane borer.

Sampling and testing programs conducted by the consultants indicate that the soil deposits under the two overpass structures are similar with respect to materials and thicknesses and succession of strata.

COMPRESSION TESTS

Consolidation Tests

Three different loading procedures were followed in the consolidation tests conducted. The normal procedure of loading in small increments was used, with some refinements concerning (a) terminating load increments (Hamilton and Crawford 1959; Crawford 1964); (b) simulating field stress changes, as suggested by Skempton and Bjerrum (1957); and (c) loading just short of preconsolidation stress, releasing to overburden stress and reloading, all in small load increments.

The results of all of these tests, and the line A-A representing the most probable preconsolidation stress with depth are shown in Fig. 2. In relation to the effective overburden stress, line B-B, this represents an overconsolidation stress of 2.0 kg/cm² in layer A, diminishing to 1.5 kg/cm² at the base of layer C. (Downward drainage of groundwater, particularly in layers B and C, significantly increases the in-place effective stresses in these layers.)

Shear Tests

In addition to the in-situ vane tests, shear strengths were measured in the unconfined compression and undrained triaxial tests. The results are shown in Fig. 2.

Compression Moduli

From the stress-strain relationships of all the consolidation and shear tests, compression moduli were calculated and are shown graphically in Fig. 3.

The shaded area M_c denotes the range of initial tangent moduli obtained from the unconfined and triaxial compression tests. Little significant difference was found between the values calculated for the two types of test. (Confining pressures ranged between 0.4 and 1.6 kg/cm².)

M_L denotes the line of average values of compression moduli determined from an arithmetic plot of the initial loading curve for the three types of consolidation tests. These curves were normally quite linear, but where they were not, the slope was calculated for that stress change equivalent to the field load. These results also indicated no significant differences between the values of modulus obtained from one type of test or another and they are approximately equal to those derived from the shear strength tests M_c .

Mean values of M_c and M_L for the layers A and (B + C) are shown as M_{Lc} .

A third modulus M_R calculated using the reloading branch of the consolidation test curves gives values appreciably higher than the others. In the upper layer the mean value of M_R is 310 kg/cm², which is more than twice M_{Lc} , and in layers (B + C), grouped together because there now appears no significant difference in material, the mean value of M_R is 840 kg/cm², which is 3½ times M_{Lc} .

FIG. 3. Field and laboratory compression moduli.

EMBANKMENT

Stress Distribution

The Boussinesq distribution of stress in the deposit below the centerline at station 425+00, as calculated using the Osterberg influence charts for a trapezoidal embankment, is shown in Fig. 2 for two stages of loading. The first

stage is for that stress increase accompanying the construction of the embankment, which proceeded at a fairly uniform rate for 38 days, and the second represents the distribution of stress under the final load when the sand cushion and paving were complete. These stages represent vertical stress increases that range from 0.5 to 0.75 of the measured overconsolidation stress.

Computed Settlements

From the mean stress increase in each of the two layers A and (B + C) and the mean compression moduli M_{Lc} , elastic compressions under full load were calculated and yielded values of 8.7 cm for layer A and 5.1 cm for layer (B + C), making a total of 12.8 cm. No estimate was possible for the compact silt below 60 ft (layer D), since no undisturbed samples were obtained. It was considered to be relatively incompressible.

Using the mean values of M_R , these estimated vertical movements were reduced to 3.9 cm for layer A and 1.9 cm for layer (B + C), making a total of 5.8 cm.

Instrumentation and Field Measurements

Three settlement gauges consisting of vertical pipes attached to spiral anchor plates screwed down to depths of 30, 47, and 66 ft, and one remote reading electrical settlement gauge (Burn 1959) were installed to measure compression in layers A, B, C, and D. A deep bench-mark was also installed within a few feet of the settlement gauges. As the embankment was being constructed, 5 ft extension rods and casing to prevent frictional loading of the gauges were added and elevations were read optically to the nearest 0.001 ft or 0.3 mm. It seems inevitable that some errors will develop and some damage will occur while heavy equipment is constantly passing, and this installation was no exception. The total error in vertical movements is estimated to be of the order of 0.01 ft or 3.0 mm.

Concrete pads were placed at the ground surface beneath the median and the side slopes for future use in measuring the pattern of transverse settlement. When paving was completed, holes were bored and vertical pipes within casings were installed with their tips resting on the concrete.

In order to determine whether excess pore water pressures would be generated during loading, piezometers were also installed, the tips of which were located at the mid-depth of the compressible layers. Detailed location of these instruments is shown in Fig. 4.

Field Observations

During the initial stages of construction the cable leading from the electrical settlement gauge to the monitoring point at the crib wall was broken, and it was not possible to measure total settlements until the embankment was complete and the settlement gauge was reached by constructing a manhole above it.

Actual settlements are plotted against time in Fig. 5, together with increase in load at the centerline and the elevations of water levels observed in the piezometers. The total settlements occurring while the electrical settlement gauge was inoperative were extrapolated from the shape of settlement curve S1.

Settlements developed fairly rapidly during construction and some months afterward, although the rate had diminished before the second stage of loading

FIG. 4. Location of instrumentation, Ottawa Queensway Station 425+00.

again speeded it up. At the present time the rates of compression except for layer D are appreciable; the entire deposit is going down at about 1.5 cm per year. The rates are decreasing only slowly and it is evident by extrapolating the present trend, that final total settlement could be of the order of 20 cm. More than half of the measured settlement has occurred in layer A.

FIG. 5. Compression with time, Main Street embankment.

Settlements of the concrete pads were measured on two occasions, verifying the values given by the electrical settlement gauge at the centerline and showing the usual dish-shaped pattern of displacement across the embankment section. The settlements below the shoulders were approximately two-thirds of that at the centerline and diminished almost to zero at the toe on the south side. No measurements were made at the crib wall.

The piezometer levels indicate an increase in pore water pressure that in part is a reflection of the rise in water level in the embankment during construction. Copious quantities of water were continuously added to aid in compaction of the sandy fill. The water levels quickly dropped once the embankment was completed, and the two piezometers that were still operating showed that by May of 1964, less than 5 months after the end of construction, the water levels were essentially the same as those measured just before construction began. The results in this short period could represent either a dissipation of pore water pressures or a reduced rate of drainage from the embankment during the winter. There was no perceptible increase in water levels when the embankment was raised about 4 ft in June 1964. The next 4 years show a further gradual lowering of water levels below the initial condition that could reflect a slow dissipation of pore water pressure—a general lowering of the water table—or the result of reading the water levels in June and July during the past three summers when they should be expected to be near their lowest seasonal level.

Field Compression Moduli

During the initial loading stage of 38 days there was a fairly linear relationship between height of fill and recorded settlement for the three gauges that were functioning during the period. Assuming an elastic response to stress increase it is possible to compute field compression moduli for the layers, A, B, and C. As indicated earlier, total settlements for this critical period were estimated and could be in error by an appreciable amount. The values calculated in this manner for layers B and C were found to be sensitive to settlement corrections of 0.002 or 0.003 ft, and the two layers were therefore treated as a single stratum. These field moduli E_f are shown graphically in Fig. 3, where they are compared to the moduli determined from the laboratory tests. They are more than 2.5 times higher than M_R , in each case, and 6 and 10 times higher than M_{Lc} for layers A and (B + C) respectively. The field modulus for the compact silt of layer D was unexpectedly low.

When the values of field moduli calculated from the observations of compression during the initial stages of loading are used to compute compression of the layers under final load, they yield values that total only about 25% of what may be eventually expected. Use of the average moduli determined from initial loading in the laboratory M_{Lc} gives amounts of compression that are a little larger than those observed after 5 years but less than may be ultimately expected. When the highest values of moduli determined in the laboratory, M_R , are used, the estimated settlements are only half of those observed recently. They do, however, agree fairly closely with the amounts of compression measured in late 1964 at the time the curves began to exhibit fairly constant time-settlement rates.

OVERPASS STRUCTURES

The overpass structures are both of reinforced concrete rigid-frame construction, resting on shallow footings. The one spanning Metcalfe Street is a simple structure on a section of the Queensway without ramps, and is therefore symmetrically loaded. It is 101 ft wide, with 60 ft between abutment walls. Six points, three on each abutment wall, were observed for settlements.

The Elgin Street overpass is not symmetrical, having a longer abutment wall on one side to accommodate the widening lanes leading to entry and exit ramps just west of Elgin Street. One abutment wall is 166 ft and the other is 153 ft. The span is 66 ft. Settlement observations were made at eight points, four on each wall.

To ensure reliable measurements deep bench-marks were installed immediately to the north of both overpasses.

Typical time-settlement curves are plotted in Fig. 6. They are of the same general shape as those for the embankment section: a fairly rapid rate of vertical movement extending for some time after loading had ceased (although in both cases the rate of loading was much slower than that for the embankment), followed by a fairly constant rate of compression with time that exhibits very little tendency to diminish. Assuming that the rates will decrease in the same manner as secondary consolidation rates decrease, settlements in 50 years will be of the order of 16 cm for the Elgin Street overpass and 18 cm for that at Metcalfe Street. As in the case of the embankment, the field moduli give values of compression that are very much lower than the observed values: M_R gave values that approximate the observed settlements just before the

FIG. 6. Settlements with time, overpass structures.

constant rate phase develops, and M_{LC} yielded values of compression that are similar to those observed 5 years after the start of construction. (In each case the field modulus was used for layer D. The actual time-settlement curve for this layer beneath the embankment shows a pattern similar to the other layers,

FIG. 7. Vertical stresses with depth for various stages of construction. (a) Station 4, Metcalfe Street overpass; (b) Station 3, Elgin Street overpass.

and it can be assumed that there will be a larger contribution to the total settlements beneath the overpass structures than is estimated using the field modulus.)

Stress distributions for these settlement estimates were calculated using the Newmark charts (Newmark 1942), and typical stress changes with depth under centerline are shown in Figs. 7a and b. Settlements were observed following the casting of pins in the footings, and only the stress changes occurring after this were used in the calculations. As would be expected, stresses at the points on centerline were of the same magnitude; those toward the ends of the abutment walls were lower. The presence of the long retaining walls and ramps that altered the shape of the embankment, together with a slight change in initial and final grades across Elgin Street, produced greater variations in total stress increases beneath the observed points on this structure. With one or two exceptions, the pattern of settlements across the abutment walls were dish-shaped as expected, but there are no signs of distress in the concrete. The differential settlement is approximately 1 cm.

Only the average stress increases in each layer were used in estimating settlements; however, as may be seen in Figs. 7a and b, particularly in the case of the Metcalfe Street overpass, the stresses immediately below the footings begin to approach the overconsolidation stress.

In this range the strain response is no longer linear and the compression modulus calculated for the range of stresses just below this can no longer be used with the same validity. Actual compression in the few feet directly below the footings is no doubt larger than that estimated.

The stress increases at Metcalfe Street are appreciably higher than at Elgin Street, primarily because this overpass structure rests on narrower footings.

DISCUSSION

The patterns of settlement with time for the deposit beneath the embankment and the overpass structures are similar. In all cases, vertical displacements continued at a fairly rapid rate for some time after construction was finished.

For a period of months the rate of settlement diminished; then for the following 3 or 4 years they continued at significant and almost constant rates. The most recent set of observations (June 1968) indicates a slight reduction in these rates of compression.

Observations of water levels in piezometers reflected the increasing height of the water table as the level of sand embankment rose and was 'wetted down' to aid compaction. A large proportion of these pressures was dissipated immediately after construction was complete and the remainder in another 4 months. Observations over the last 3 years show a small gradual drop below initial conditions.

These settlements would appear to be the result of a dual process—an essentially elastic one that is operative during the period of loading, and another that may also be operative from the beginning and is not greatly different from the consolidation process in that structural adjustment of the soil skeleton continues as the deposit accommodates the increased load. The difference comes in the time scale and perhaps the nature of the particle bonds that yield under these relatively low loads. Some small increase in pore pressures would be consistent with this picture but the field observations in this case have not been conclusive.

The field modulus seems useful in an engineering sense only in predicting those vertical movements that occur during the loading period, if it is not prolonged. Use of the modulus M_{Lc} gives estimates of settlement closest to those now expected, but would appear to underestimate them by between 30 and 40%. Where the ratio of the applied stress to the overconsolidation stress was about 0.45 (Burn and Hamilton) use of a modulus based on essentially the same criteria showed fairly good agreement between the estimated and observed settlements. For three apartment buildings on floating foundations in this part of Ottawa (Legget *et al.* 1961) where soil conditions are similar and the ratio of $\Delta p/p_c - p_0 = 0.37$ (Crawford 1968), this approach led to estimates of settlement that were considerably larger than the actual settlement.

CONCLUSIONS

The tests conducted in the laboratory indicate that the average stress applied by the embankment is well below the overconsolidation stress, i.e. $\Delta p/p_c - p_0 = 0.65$.

Settlements are not, therefore, caused by primary consolidation. No significant excess pore water pressures were detected over the period of observation.

The settlement records, however, show that besides initial 'elastic' compressions significant vertical movements beyond the time of application of full load occurred. The continuing settlements are attributed to slow yielding of the clay structure.

Use of the average laboratory compressed modulus M_{Lc} in estimating long-term settlements results in underestimating the expected amounts by about 40%.

These observations are also largely valid for the overpass structures at Elgin and Metcalfe Streets.

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