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**THE MODULUS OF ELASTICITY OF LEDA CLAY FROM
FIELD MEASUREMENTS**

by

M. BOZOUK

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THE MODULUS OF ELASTICITY OF LEDA CLAY FROM FIELD MEASUREMENTS

M. BOZOUK*

ABSTRACT

The construction of the Ottawa Sewage Treatment Plant provided an opportunity to measure bottom heave in two large excavations in clay. With gauges installed prior to excavation, elastic heave, swelling, and heave due to pile driving were measured in an excavation 31.5 ft. deep. Using a Poisson's ratio of 0.4, the modulus of elasticity determined from field measurements compared favourably with the initial tangent modulus from the best unconfined compression strength tests. In the other excavation a drop in the ground water table produced an elastic settlement that exceeded the elastic heave resulting from unloading by excavation.

SOMMAIRE

La construction d'une usine d'épuration des eaux à Ottawa nous a fourni une occasion de mesurer les soulèvements du fond de deux fouilles d'importance dans l'argile Léda. Avant le début de l'excavation, des jauges furent installées pour mesurer le soulèvement dû à la déformation élastique, au gonflement et au battage des pieux au fond d'une fouille de 31.5 pieds de profondeur. Le module d'élasticité calculé à partir des mesures sur le terrain et du coefficient de Poisson de 0.4 est comparable au module donné par la tangent à la partie initiale de la courbe contrainte-déformation obtenue par les meilleurs essais en compression simple. Dans l'autre fouille, un affaissement de la nappe phréatique a produit un tassement élastique de plus grande amplitude que le soulèvement dû à l'excavation.

When a soil is unloaded rapidly by excavation it rebounds elastically. Re-loading produces an elastic settlement of equal magnitude, and if greater loads are applied, additional settlement due to consolidation of the subsoil may occur. Elastic movements may be unimportant for many shallow excavations, but for deep excavations such as for basements of large buildings elastic rebound and subsequent elastic settlement may cause serious difficulties. It is often not possible to predict accurately the elastic movements because it is very difficult to measure the modulus of elasticity E for a soil material. The moduli determined from unconfined compression tests on undisturbed tube samples are usually much too low and field measurements are difficult to make.

The construction of the Ottawa Sewer Plant, with an initial capacity of 40 million gallons per day (mgd), was started in 1961. When completed in 1963, it will provide primary treatment of the 25 mgd of raw sewage now emptying into the Ottawa River through twenty-three separate outlets. When necessary, its capacity can be increased to 75 mgd to serve an ultimate population of 590,000. The design and construction details have been published by MacLaren *et al.* (1960). Large excavations were necessary for the construction of the main pumphouse of the plant, the digester tanks, and the primary settling tanks. The Division of Building Research of the National

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Research Council of Canada took the opportunity to measure bottom heave in two of the large excavations.

DESCRIPTION OF THE SITE AND SOILS

The sewer plant is situated on the eastern outskirts of the city of Ottawa on 320 acres of flat clay plain about 40 ft. above the Ottawa River (mean water elevation 134). The subsoils are lacustrine and marine clay (Gadd, 1963): stiff, fissured, brown-grey to a depth of 20 ft.; stiff grey clay to 50 ft.; becoming black-mottled and very silty to a depth of 86 ft., where glacial till is found. Grey limestone bedrock is at a depth of 96 ft.

Excavations for the three digestion tanks and the primary settling tanks are shown in Figure 1. For the digestion tanks the excavation was roughly

FIGURE 1. Location of heave gauges in excavations for digestion and primary settling tanks

L-shaped, with each leg about 320 ft. by 190 ft. by 31.5 ft. deep. For the primary settling tanks it was rectangular with sides 330 ft. by 240 ft. The depth varied, increasing from 12 ft. at the north and south ends to 21 ft. in the trench at the centreline. It was therefore possible to measure bottom heave in two excavations of different depths.

During excavation for the main pumphouse immediately north of the other excavations, large undisturbed block samples of soil were obtained at regular intervals for laboratory tests. These samples indicated a preconsolidation of more than 3 kg./sq. cm. (Crawford, 1961). The liquid limits of the material vary from 70 to 50, with the natural water content usually much higher. Grain size analyses indicated that 60 to 70 per cent of the soil particles were in the

FIGURE 2. Soil profile and summary of test results at the Ottawa Sewer Plant

clay size range. The soils are very sensitive, have a high initial void ratio, and liquidity indices between 1 and 3. Six unconfined compression tests performed on specimens trimmed from each undisturbed block sample produced shear strengths of 1.5 kg./sq. cm. For the initial tangent modulus the highest value from the unconfined tests on each block was used. A summary of the test results and a soil profile is plotted in Figure 2.

INSTRUMENTATION

At the excavation for the digestion tanks initial instrumentation consisted of three sets of heave gauges, installed to depths of 15 ft. and 31.5 ft. below the ground surface, and a deep bench mark. Unfortunately, the 15 ft. gauges were lost during excavation without any possibility of their being read. Four heave gauges and a deep bench mark were also installed at the site of the primary settling tanks. The depths of the gauges at stations (12 + 47), (13 + 00), (13 + 05), (13 + 10) were 14.5 ft., 13.5 ft., 17.5 ft., and 23.5 ft. respectively. The locations of the heave gauges and bench marks are shown in Figure 1. The purpose of this arrangement of gauges was to observe the bottom heave near the edge and towards the middle of the excavations.

The heave gauges were patterned after those used in Norway (Norwegian Geotechnical Institute, 1962). They consist of four 0.25 in. thick steel fins welded together to form a vane 3.5 in. diameter and 12 in. long supporting a 0.75 in. thick plate as detailed in Figure 3. A slotted keyhole was prepared in the centre of the plate to facilitate installation.

FIG. 3. Details and installations of heave gauge

The heave gauges were lowered with drill rods down a clean augered 4 in. diameter borehole and forced into the undisturbed soil. The rods were disengaged and removed, leaving the gauge in place. The borehole was then pumped full from the bottom with a bright red bentonite slurry that would gel (about 800 per cent water content and coloured with Erythrosine dye

Colour Index 773, Figure 3). The bentonite gel protected the borehole from caving and from filling from the top with loose soil during excavation, and the bright red colour assisted in locating boreholes while excavation was in progress. Readings on the gauges were made by sounding through the gel and taking measurements to ± 0.001 ft. with an engineer's precise level.

The deep bench marks consist of an iron pipe with a steel foot driven into the glacial till. The iron pipe, which is the reference point, is protected from the surrounding soil by an oil-filled casing. Details of this instrumentation and the installation procedure have been described by Bozozuk *et al.* (1963).

FIELD MEASUREMENTS

Digestion Tanks

Excavation for the digestion tanks was started August 1, 1961. The contractor excavated uniformly over the area with scrapers and pushers to a depth of 3 ft. to 4 ft., but this equipment bogged down and the excavation had to be completed with drag lines and shovels. As shown in Figure 1 the drag lines excavated to the full 31.5 ft. depth in one stage, starting at the southwest end and completing the excavation at the north end on September 8. The floor was then covered with 1.5 ft. of crushed stone fill to provide a firm working base for construction equipment. Pile driving for the individual tanks commenced at the west end of the excavation on September 25 and was completed over the site on November 3, 1961. An interesting part of the pile-driving operation was the ease with which the piles were driven. Once they were positioned and tapped with the hammer a couple of times they were pushed to the full depth of 50 ft. (the boom length of the pile driver) in one quick stroke.

Because of the method of excavation, it was impossible to obtain readings on the heave gauges at various stages of unloading as had originally been planned. Readings were obtained only after the excavation had been completed over the various gauges.

As is shown in Figure 4, bottom heave occurred in three stages: (i) instantaneous or elastic heave, (ii) time-dependent heave or swelling, and (iii) heave due to pile driving. Elastic heave was least near the edge of the excavation, becoming greater in the middle. As the material was always fully saturated (the water table was found in the crushed stone fill) swelling also took place. Before swelling was complete pile driving was begun and caused additional heaving. At the end of pile driving the total combined heave ranged from 1.8 in. at the edge of the excavation to 2.0 in. at the centre. About half of this total could be attributed to pile driving.

Primary Settling Tanks

The heave gauges at the primary settling tanks were installed December 14, 1961. Excavation proceeded between January 23 and February 26, 1962. The top 5 ft. to 6 ft. were removed by scrapers; because of freezing temperatures the contractor was forced to rip the frozen soil so that the scrapers could be push loaded. Finally drag lines were used to complete the job. A 1.5 ft. layer of crushed stone fill was then placed on the floor of the excavation.

FIGURE 4. Bottom heave in 31.5-ft. deep excavation for digestion tanks

FIGURE 5. Bottom heave in 12-ft. deep excavation for primary settling tanks

Satisfactory measurements on the heave gauges were difficult to obtain. At the start of excavation the contractor chose to use the area where the deep bench mark was located as a spoil area for stockpiling the excavated soil. A large concrete foundation for one of the nearby structures was therefore used as a reference datum, but it soon became evident that it was not stable, probably because of frost action. When the frost disappeared in the spring, it was possible to locate the original bench mark and use it for the final survey. This indicated that a small net settlement had occurred at the 23.5 ft. gauge. All preceding surveys in which the concrete foundation had been used as a datum were referenced, therefore, to the 23.5 ft. gauge and the results plotted in Figure 5.

A small heave of about 0.1 in. occurred at the edge of the excavation. Towards the centre, the bottom settled about 0.3 in. initially, reducing to 0.15 in. with subsequent swelling of the soil. Lowering of the ground water table, caused by pumping from the adjacent 31.5 ft. excavation and drainage from the 21 ft. deep trench, apparently caused an elastic settlement that exceeded the elastic heave due to unloading.

ANALYSIS OF RESULTS

The elastic settlement of the corner of a rectangular uniformly-loaded area located at the ground surface was given by Steinbrenner (1934) and Terzaghi (1943):

$$(1) \quad \rho = q \frac{B}{E} I_p$$

where ρ = elastic settlement

q = surcharge

B = width of loaded area

E = elastic modulus

I_p = influence factor which depends upon size and shape of the loaded area, Poisson's ratio (μ) and depth of compressible soil.

The influence factor, I_p , may be obtained from:

$$(2) \quad I_p = (1 - \mu^2)F_1 + (1 - \mu - 2\mu^2)F_2$$

where F_1 and F_2 are constants that may be obtained from the graphs given by Steinbrenner (1934) or Terzaghi (1943).

The modulus of elasticity may be determined from heave measurements by re-arranging equation (1) into the form:

$$(3) \quad E = \frac{\Sigma q B I_p}{\rho}$$

where ρ = elastic heave

q = amount of unloading.

The excavation for the digestion tanks was subdivided into rectangles, with the common corner located over each heave gauge. The influence factor was

calculated from the original surface of the ground and corrected for the depth of the excavation. The depth of soil subject to elastic heave would increase from the edge of the excavation toward the centre. Assuming that this "depth" increased linearly from the toe of the slope by 45°, the heave gauges at (12 + 70) and (14 + 00) would be affected by the edge of the excavation. At (14 + 75) the gauge was 66 ft. from the toe of the slope. Consequently, the depth of soil subject to elastic heave was limited by the depth of clay overlying the glacial till at this location, 55 ft.

The modulus of elasticity at (14 + 75) was calculated for Poisson's ratios of 0.3, 0.4, and 0.5 and summarized in Table I. For comparison, the average initial tangent modulus of elasticity E_1 for the soil profile, obtained from laboratory strength tests, was also included. The laboratory value for E_1 of 750 kg./sq. cm. compared favourably with the field results of 720 kg./sq. cm. for a Poisson's ratio of 0.4.

TABLE I
Summary of Calculations: Modulus of Elasticity Determined from Heave Measurements in Excavation for Digestion Tanks

Location	E = Modulus of Elasticity, kg/sq. cm.			
	$\mu = 0.3$	$\mu = 0.4$	$\mu = 0.5$	E_1^*
14 + 75	822	720	591	753

*Average maximum initial tangent modulus for the soil profile, obtained from the best unconfined strength tests on specimens trimmed from large undisturbed block samples.

Field measurements of bottom heave at the primary settling tanks showed that a net settlement occurred in the middle of the excavation; this was attributed to a lowering of the ground water table. Using an E value of 720 kg./sq. cm. measured in the deep excavation, the elastic settlement due to a drop in ground water table of 28 ft. and the elastic heave due to unloading were calculated for Poisson's ratios of 0.3, 0.4, and 0.5. The results given in Table II show that the calculated net settlement of 0.02 ft. for Poisson's ratios of 0.3 and 0.4 is very close to the measured value of 0.03 ft.

TABLE II
Comparison of Measured with Calculated Bottom Heave in the Excavation for the Primary Settling Tanks

E kg/sq. cm.	μ	ρ , due to drop in ground water table, ft.	ρ , due to unloading, ft.	Net ρ , ft.	ρ Measured, ft.
720	0.3	-.069	+.048	-.021	-.03
	0.4	-.059	+.040	-.019	
	0.5	-.046	+.030	-.016	

From the measurements in these excavations, the Poisson's ratio for Leda clay appears to be 0.4 or slightly less. It substantiates the use of this value

for estimating elastic settlements of buildings in Ottawa (Legget *et al.*, 1961) and supports arguments against the arbitrary assumption that the Poisson's ratio for saturated soils is 0.5 (Crawford and Burn, 1963). A more realistic value for Poisson's ratio of saturated soils may be between 0.3 and 0.4.

The observations also show that elastic heave may be compensated by elastic settlement due to a lowering of the ground water table (Serota and Jennings, 1959).

CONCLUSIONS

The bottom heave in an excavation 31.5 ft. deep in Leda clay near Ottawa occurred in three stages; (a) instantaneous elastic heave; (b) heave due to swelling with time; and (c) heave due to pile driving (this accounted for one-half the total heave).

Field measurements of the modulus of elasticity, E , with a Poisson's ratio of 0.4 compared with the best values of initial tangent modulus E_1 obtained from unconfined laboratory strength tests on specimens trimmed from large undisturbed block samples.

The elastic settlements resulting from lowering the ground water table exceeded the elastic heave due to unloading the soil. This caused a net settlement in the bottom of the excavation for the primary settling tanks, as is shown by field measurements and verified by calculations.

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