Prediction of Friction Pile Capacity in a Till

Prévision de Capacité des Pieux dans Argile Blocaux

G.G. MEYERHOF
J.D. BROWN
G.D. MOULAND

Department of Civil Engineering, Technical University of Nova Scotia and Jacques, Whitford and Associates Ltd., Halifax, Nova Scotia, Canada

SYNOPSIS A comparison is made between the results of pressuremeter tests, screw plate tests and laboratory tests for the determination of the undrained shear strength and modulus of deformation of a glacial till. The results are used to estimate the bearing capacity and settlement piles for comparison with the results of load tests on some piles driven into the till.

INTRODUCTION

The behaviour of foundations in glacial till is difficult to estimate from laboratory tests, and in-situ investigations are generally preferred. One of the promising field methods is the use of the pressuremeter, but experience with it in till is limited. Accordingly, in connection with the foundation of a bridge resting on piles driven into a glacial till, the soil properties were determined both by laboratory and in-situ tests. These results were employed to make estimates of the bearing capacity and settlement of the piles and checked by load tests on two test piles.

SITE AND SOIL CONDITIONS

The site for this investigation is located near one of the abutments at the south end of a new highway bridge over the Stewiacke River, Nova Scotia. This area of the province is generally underlain by till of Wisconsin Age, derived mainly from the Permian sandstone and shale bedrock which occurs extensively to the north, but containing some igneous and metamorphic rock from the Cobequid mountains (Hughes, 1956). The till is a well-graded reddish-brown, stiff to hard, intact silty clay of low plasticity with some sand, gravel, cobbles and occasional boulders. It is closely associated in colour, texture, and plasticity, with the tills which mantle bedrock over extensive areas of the province. At this site, the index properties and standard penetration tests of the till show some decrease in water content and increase in plasticity and strength with depth (Fig. 1).

Shear Strength

Thin wall tube samples of about 60mm dia. were taken from various depths in the till for consolidated-undrained triaxial compression tests, and some unconfined tests were also performed on split spoon samples. The test results show that the till behaves as a heavily-overconsolidated soil with the undrained shear strength increasing from an average of about 150 kPa at shallow depth to about 250 kPa at a depth of about 15 m (Fig. 2). The effective angle of internal friction was found to be 29 degrees, and the effective cohesion 16 kPa. It may be noted that the undrained shear strengths determined in two cyclic load tests, which were carried out to study the deformation modulus, were higher than the corresponding single load tests.

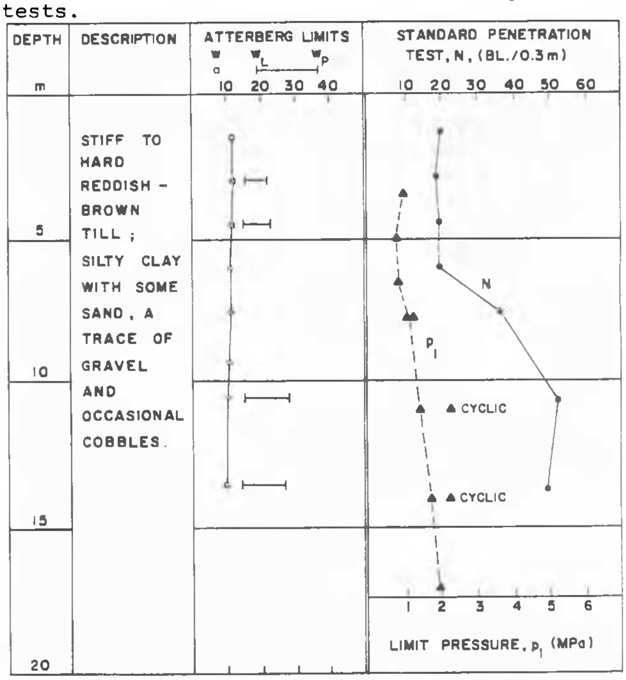

Fig. 1 Composite Borehole Record, Variation of Index Properties and Strength.

The trend of these results is confirmed by in-situ tests carried out with a Menard Type G pressuremeter of about 75 mm dia. (Baguelin et al, 1978) carefully installed at various depths in boreholes of the same diameter made by a continuous flight auger (Mouland, 1979). These tests were made in approximately ten equal load increments, each acting for one minute to obtain the corresponding creep curve for estimating the yield pressure p_y (Menard, 1975). Typical results are shown in Fig. 3. The limit pressure

P_l estimated from the pressuremeter curves according to Menard's method is plotted against depth in Fig. 1. The limit pressure may be used to determine the undrained shear strength C_u according to the equation:

$$P_l - P_o = C_u N_p \qquad (1)$$

where P_o = total horizontal "at rest" earth pressure, and N_p is a limit pressure factor analagous to the bearing capacity factor N_c.

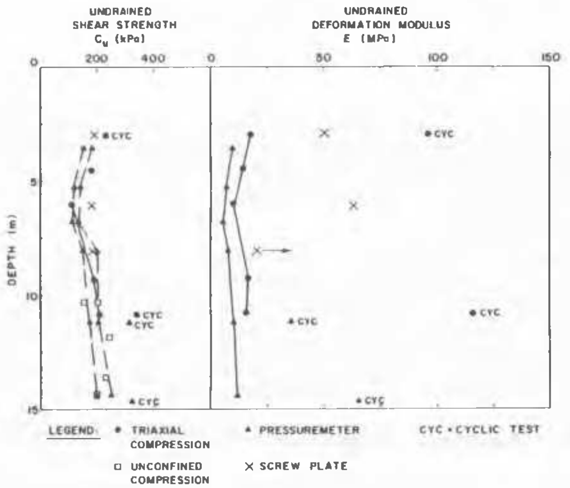

Fig. 2 Variation of Undrained Shear Strength and Undrained Deformation Modulus.

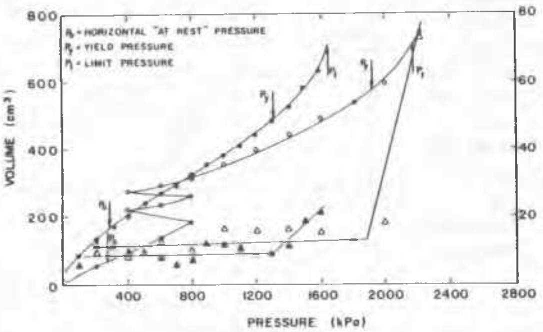

Fig. 3 Typical Results of Pressuremeter Tests.

In a comparison of pressuremeter and other tests in stiff London clay (Marsland and Randolph, 1977) a value of 6.2 was proposed for N_p. This value can be derived from theoretical considerations (Gibson and Anderson, 1961) and corresponds to the bearing capacity factor N_c = 9 derived from the same theoretical approach. On the other hand, tests performed on a wide range of U.S. midwest clay soils (Lukas and de Bussy, 1976) gave the experimental value of N_p = 5.1. Using these values as upper and lower limits, values of C_u have been obtained from the tests and these are represented by the narrow band in Fig. 2. A mean value of N_p = 5.7 could also be used.

Further information about the strength can be obtained from some buried screw plate load tests on a 160 mm dia. plate at various depths in the upper portion of the till (Fig.4). Usin a bearing capacity factor of 9.3 (Meyerhof, 1951) in the equation

$$q_f = C_u N_c \qquad (2)$$

the undrained shear strength may be deduced. The values obtained in this manner are also shown in Fig. 2.

The results of all the tests can be seen to compare quite well with one another, indicating that the results may be used with some confidence.

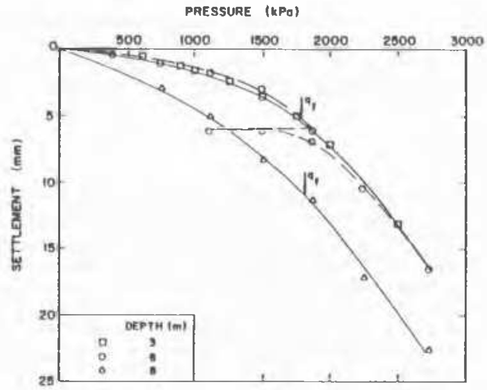

Fig. 4 Results of Screw Plate Load Tests.

Undrained Modulus of Deformation

Each of the three types of test mentioned above permits a modulus of deformation to be obtained. In the triaxial compression tests, the initial tangent modulus E_t has been computed for the single loading, and the slope of the curve during the final cycle has been used to compute the cyclic modulus. In the pressuremeter tests, the slope of the pressuremeter curve mid-way between P_o and P_y has been used to find the modulus of deformation E_p for the single loading tests, and the slope of the last load cycle has been used to compute the cyclic modulus.

The plate load test can also be analysed to determine the initial modulus of deformation E_s from the equation for settlement of a buried plate (Marsland and Randolph, 1977)

$$S = \frac{q\ B}{E_s} (1 - \mu^2) \frac{\pi}{4} I_s \qquad (3)$$

where q = average pressure on a plate of width B, I_s = influence value = 0.85 for a plate at depths greater than 6 times the plate diameter, and Poisson's ratio μ = 0.5.

The deduced values of deformation modulus for the three methods of test are shown in Fig. 2. The plate test E_s values are much greater than the corresponding pressuremeter and triaxial values of E_p and E_t, respectively, under single loading and compare fairly well with the pressuremeter cyclic modulus.

ANALYSIS OF PILE LOAD TESTS

The precast reinforced concrete piles were a Herkules 420 hexagonal pile (12.7 cm each side) driven to a depth of 12 m and a Herkules 800 hexagonal pile (17.8 cm each side) driven to a depth of 13 m. The piles were tested a few days after driving and the failure load was reached in 3 to 4 hours with the results given in Fig. 5.

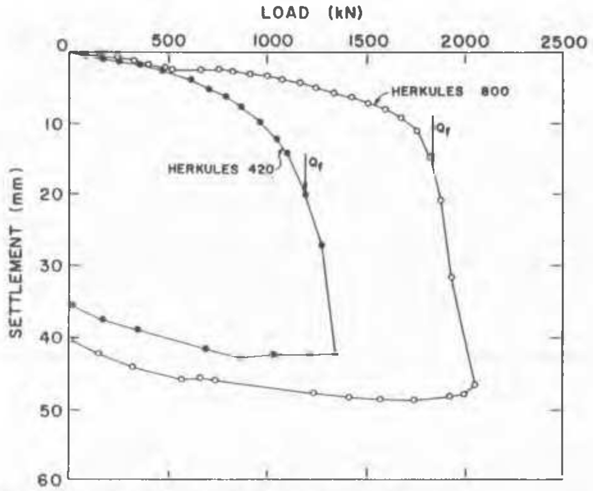

Fig. 5 Results of Pile Load Tests.

Ultimate Bearing Capacity

The ultimate point resistance of piles driven into cohesive soil can be estimated from equation (2) using a bearing capacity fact r of 9.3. From the present tests, C_u = 220 kPa at the pile tip.

The shaft capacity can be estimated from a shaft adhesion usually expressed as (Tomlinson, 1957)

$$C_a = \alpha C_u \text{ (av)} \qquad (4)$$

For very stiff clays, α = 0.45 (Can. Found. Manual, 1978) and, from the present tests, the average C_u = 170 kPa.

A more rational estimate of shaft capacity may be made from the drained shear strength of the remoulded soil, wherein the effective unit skin friction is taken as (Burland, 1973, Meyerhof, 1976).

$$f_s = K_s \bar{p} \tan \phi' \qquad (5)$$

where earth pressure coefficient on shaft K_s = 1.5 K_o, approximately, for driven piles, $\bar{p}$ = average overburden pressure along shaft and ϕ' = effective angle of internal friction of clay. The value of K_o, the coefficient of earth pressure at rest, can be found by using the horizontal earth pressure measured in the pressuremeter tests, although the procedure is open to serious criticism (Baguelin et al, 1978) on the ground of its inherent inexactitude. A crude estimate of K_o may also be made on the basis of plasticity, ϕ' and overconsolidation ratio (Brooker and Ireland 1965), using in this case an overconsolidation ratio of 8 to 10. Both methods give an average K_o equal to about 1.3, and so this value may be used with some confidence.

The capacities predicted by these methods, and using the strengths obtained by the in-situ and laboratory tests, are given in Table 1, together with the pile load test results.

TABLE 1

Measured and Predicted Pile Capacities

Pile	Measured Ultimate Load(kN)	Predicted Capacity (kN)		
		Point	Shaft Adhesion (Eqn. 4)	Shaft Friction (Eqn. 5)
Herkules 420	1160	85	700	1210
Herkules 800	1780	165	1060	1850

Comparison shows that the point resistance represents only about 8 percent of the pile capacity, so that the shaft makes the more significant contribution. The shaft adhesion approach gives values of the shaft capacity which are much too small. Agreement can only be attained if a value of α = 0.7 is used, which is much higher than previous experiments would warrant. On the other hand, the shaft friction approach gives estimates which compare well with the observed values.

Elastic Settlement

The settlement of an elastic pile in an elastic medium can be computed by means of the following equation (Mattes and Poulos, 1969):

$$S = \frac{P}{E_s L} I_p \qquad (6)$$

where P = pile load, E_s = elastic modulus of the soil, L = pile length, and I_p = influence factor, a function of the elastic moduli of the soil and the pile, the length and diameter of the pile, and Poisson's ratio.

Applying this equation to the prediction of short-term settlements for each pile under a working load of 445 kN, the curves shown in Fig. 6 are obtained. These curves provide a convenient means to illustrate the settlements which would be predicted using the undrained elastic moduli measured in the various methods of tests, and to compare these values with the observed settlements. Of most direct interest are the screw plate moduli, which compare relatively well, and the pressuremeter and triaxial moduli, which are much too low to explain the observed pile behaviour.

The undrained modulus back-calculated from this observed pile settlement is in the range of 65-80 MPa. Data on tills and overconsolidated clays (Radhakrishna and Klym, 1974; D'Appolonia et al 1971) indicate that this value is probably reasonable, hence the pressuremeter values are

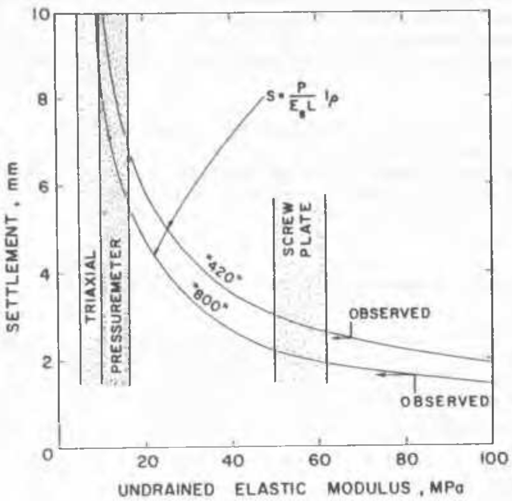

Fig. 6 Comparison of Observed and Predicted Pile Settlements.

too low, in absolute terms. It is tentatively concluded that the low pressuremeter and triaxial test results are due to softening and disturbance associated with the procedures followed. The cyclic test data are insufficient to permit drawing definite conclusions, but appear to be more promising for short-term settlement estimates.

CONCLUSIONS

The undrained shear strength of the low plasticity clay till at this site can be obtained from the limit pressure obtained in pressuremeter tests, by using a limit pressure factor, N_p equal to 5.7.

Estimates of pile capacity employing the concept of effective unit skin friction provide good agreement with observed values. The use of undrained shear strength and an adhesion factor is much less satisfactory. Values of elastic modulus vary over a wide range. Short-term pile settlements predicted by using the screw plate modulus are approximately 30 percent greater than the observed values. The initial pressuremeter modulus is quite inaccurate in the present case, but cyclic values would give fairly reasonable settlement estimates.

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