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TESTING AND DESIGN OF A PILED FOUNDATION PROJECT A CASE HISTORY

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ABSTRACT: Design for a large refinery expansion was undertaken at a site reclaimed from a lake about 40 years ago. The natural soils consist of sand deposited on normally consolidated, compressible post glacial lacustrine clay followed by silty clay till on limestone bedrock found at about 25 m to 30 m depth below existing grade. The site will be raised an additional 1.5 m, which will cause long-term settlement. Some of the new units are 30 m to 70 m in height and will be supported on piles—several thousand in all. In anticipating negative skin friction to develop, the initial design called for subtracting the drag load from the allowable load determined from the pile capacity. Initial design also expected the piles to be constructed to bedrock. However, a review of the design made clear that a drag load is a problem for the axial structural strength of a pile and should not be subtracted from an allowable load based on bearing capacity. Moreover, analysis of the results of full-scale static and dynamic loading tests demonstrated that it was not necessary to reach bedrock, but the piles would develop adequate capacity in the clay till and they would not experience excessive down-drag due to the settling clay. The final, revised design resulted in a saving of close to 25 million dollars and considerable construction time.. The piles selected for the foundations were 457 mm (18inch) diameter bored piles installed to about 1.5 m into the glacial till. The paper presents site conditions, tests results, and the design principles employed.

Introduction

Design of foundations over reclaimed land usually faces problems with settlement necessitating supporting the foundations on piles. The settlement is due to past and future fill placed on the site, and the design has to ensure that the piles will not be adversely affected by this settlement. This means that the load transfer for the piles selected for the foundations needs to be determined, frequently by performing full-scale tests. This paper reports a case history on an investigation for a project involving several heavy and movement-sensitive industrial structures, some 30 m to 70 m in height, to be constructed at a mid-Western site in the United States. The site was reclaimed from a lake about 40 years ago by placing about3 m of undocumented coarse-grained fill over the area. About 1.5 m of new fill is expected to be placed across the site. Most structures will be supported on 450 mm diameter augercast piles—several thousand in all—though some structures are expected to require 600 mm diameter piles. Some of the proposed units have a footprint of about 15 m by 90 m and would impose a stress, if placed on a slab, over the footprint of about 200 KPa. The desired unfactored load on the 450mm piles is about 1,300 KN. The investigation for the final design consisted of additional boreholes and dynamic and static loading tests. Key issues for the design were if the site conditions considering drag load and downdrag would necessitate bearing the piles on or in the bedrock or if satisfactory design would be obtained with piles stopping within the glacial till above the bedrock.

Soil Profile

The soil conditions are quite alike across the project site. The uppermost layer consists of an about 3 m thick heterogeneous fill consisting of sand, miscellaneous debris, and slag.

The natural soils consist of about 9 m of sand with trace of fines deposited on 10 m to 16 m of firm, compressible, but slightly preconsolidated, post-glacial lacustrine sandy silty clay to a depth of about 18 m to 20 m. Below the clay lies an about 4 m thick layer of stiff silty clay deposited on a layer of about 2 m to 4 m of hard silty and sand clay till, starting at depths across the site ranging from about 20 m through about 35 m below existing grade. The till is deposited on limestone bedrock. In places, the deposit immediately above the bedrock consists of sand and gravel (outwash deposits) instead of the clay till. The groundwater table lies at about 2.0m depth and the pore water pressure is hydrostatically distributed. Figure 1 shows the soil profile at test location B, one of two test locations of the project, compiling soil layering determined from a CPTU sounding (only the q_t is shown) and N indices from a SPT test with Atterberg limits. Soil layer identification name and Janbu modulus numbers are shown to the right. This paper addresses the results of tests performed at two test locations named B and G, respectively. At both test locations, the depth to the glacial till is 22.5 m. The depth to the bedrock is 31 m at test location B and at 27m at test location G.

Calculations applying the planned fill and the compressibility parameters show that the project site will experience long-term settlement which at the ground surface will amount to about 50mm, reducing almost linearly to insignificant values at the top of the glacial till at about 25m depth.

Testing Programme

Two companion test piles, Piles B1 and B2, and G1 and G2, were constructed at each of two locations about 200 m apart on February 14, 2008 (Location B), and February 22, 2008 (Location G). The piles were 457mm (18 inches) diameter augercast piles and installed to depths of 25.6 m and 26.2 m at Locations B and G, respectively. One of each companion pair was equipped with a 230 mm diameter Osterberg bi-directional load cell (O-cell) placed 1.8 m above the pile toe, i.e., at depths of 23.8 m and 24.4 m, respectively.

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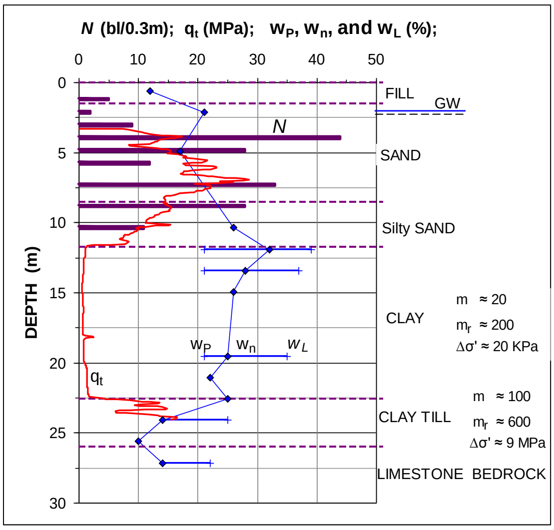

Fig.1 Soil profile at test Location B (Δσ' is the preconsolidation margin, i.e., the difference between the preconsolidation stress and the existing overburden stress).

The piles were instrumented with one strain gage pair placed 1.2 m below the O-cell and four levels of strain-gage pairs at distances of 1.8 m, 6.4 m, 10.4 m, and 17.0 m (Pile B1) and 1.5 m, 4.6 m, 10.0 m, and 14.9 m (Pile G1) above the O-cell. For both O-cell piles, the O-cell locations are at the interface between the silty clay and the glacial till. The O-cell assembly and strain-gage levels were attached to the center of the web of a 10HP42 steel beam and inserted into pile after completions of the grouting and removal of the auger. The buoyant weight of the piles at the O-cell level is 70 KN. The set-up times between construction and testing were 26 days and 19 days, respectively.

The companion piles at each test location (Piles B2 and G2) were tested by measuring the response to a dynamic impact using the GRL dynamic drop hammer testing system (Apple unit). The set-up times between construction and testing of the companions pile were 29 days and 22 days, respectively. To facilitate the testing, the piles were built up above ground with a section consisting of an 1.2 m long, 457 mm diameter, 9.5 mm wall steel shell filled with grout. The Pile Driving Analyzer (PDA) gages, four pairs of accelerometers and strain-gages, were attached to the build-up section. The dynamic tests were performed with a 135 MN drop hammer producing single drops with controlled height. In testing Pile B2, it was difficult to maintain concentric blows and the records are somewhat erratic. The height-of-fall was therefore not raised above 450 mm. Two heights-of-fall were used in testing Pile G2**:** 620 mm and 930 mm. A Case Pile Wave Analysis Program (CAPWAP) analysis was performed on each of the three blow records.

Results

O-cell tests

The load-movement response of the O-cell tests on Piles B1 and G1 are shown in Figures 2A and 2B, respectively. For both tests, the shaft above the O-cell reached ultimate resistance, which occurred at O-cell loads of 1,970 KN for Pile B1 and 1,640 KN for Pile G1.

The load-movements response of the portion below the pile toe followed a gently curving line and no indication of reaching any ultimate resistance can be observed. The shaft load-movement of Pile B1 developed strain softening beyond the peak load. The downward movements at the maximum load were 50mm and 30mm, respectively, corresponding to 9% and 15% of the nominal pile diameter.

The load-movement and O-cell expansion (not shown) records indicate that the O-cell level residual load in the test piles is approximately 300KN, i.e., about 200+KN larger than the pile buoyant weight.

It is customary to combine the measured upward and downward movement into equivalent pile head load-movement curves, which is shown in Figures 3A and 3B. The intended allowable load and the offset limit constructions are indicated in the graphs. The smallest combined maximum loads were 4,010 KN and 3,350 KN for Piles B1 and G1, respectively which loads are smaller than pile ultimate resistance—capacity—by a ratio larger than two. An allowable load of 1,300KN is therefore considered safe.

The separation of shaft (upwards records) and toe resistance (downward records) is an important result of a static loading test. In contrast, a head-down curve does not supply much information. However, assessment of pile test results needs to be addressed in term of resistance distribution, which is provided by analysis of the strain-gage records. In the analysis, the recorded strain changes are converted to load by multiplying strain, area, and 'elastic' modulus. The strain change values are the average strain of the steel and concrete cross section. While the steel area is well defined, the concrete area due to unavoidable variation of the pile diameter of the bored pile is not. The largest uncertainty rest with the modulus, which not only can vary between different concrete or grout compositions, it is also not a constant but a variable that changes with stress level. The difficulties in determining the load represented by the strain values can be overcome by applying the tangent modulus method of analysis (Fellenius 1989, 2009) in which the change of stress over change of strain is plotted versus the strain. When the ultimate resistance has been reached at a gage level, ideally, the data points plot along a sloping straight line and a linear regression will determine the slope "a" and ordinate intercept "b" of the line. The secant modulus, E_s, for the stress-strain relation of the data is then as shown in Eq. 1.

$$E_s = 0.5a\,\varepsilon + b \qquad (1)$$

where E_s = secant modulus of composite pile material
a = slope of the tangent modulus line
ε = measured strain
b = y-intercept of the tangent modulus line (i.e., initial tangent modulus)

The ideal condition for the tangent-modulus analysis is a shaft resistance that shows neither strain-softening nor strain-hardening response, i.e., has a well-defined peak value, and that other gage locations are where the soil provides increasing resistance to the continued loading, most typically toe resistance so that several points will plot the tangent-modulus line, allowing it to be well-defined.

For the upward loading in an O-cell test, unless the pile length above the O-cell location is long, the latter condition is not available, and, at best, only a couple of readings are obtained with values on the tangent-modulus line before the test is over, and this mainly for the gage levels nearest the O-cell. Moreover, if the shaft shear shows a strain softening tendency, the last couple of gage readings will indicate larger strain changes than those representing the applied load increment. This is so because the loss of resistance along the pile portion between the strain-gage pair analyzed and the O-cell (or pile head jack in case of a conventional head-down test) will cause the load reaching the gage level to be larger than the applied load increment and, therefore, the measured strain will be larger than for the applied load increment. Despite these sometimes exasperating influences, the tangent-modulus approach is still the best way to determine the material modulus.

Figures 4A and 4B show the tangent-modulus plots for Piles B1 and G1. The stress values were obtained by dividing the applied load increments by the nominal pile cross section. **(**The gage levels are numbered from the pile toe to the pile head. Gage Level 1 is located below the O-cell and Gage Level 2 is the first gage level above the O-cell. The records from Gage Levels 3 and 2 in Piles B1 and G1, respectively, were erratic and have been excluded from the analyses**)**. Because of the ultimate shaft resistance developed suddenly and, also, due to the effect of the strain-softening, the tangent-modulus line is not well-defined and undefined in Figure. 4B, allowing no effect of variation of cross-section and stress-dependency to be discerned. The best estimate of the pile composite E-modulus is a constant value of 29 GPa.

The mentioned modulus was applied to the strain records and the nominal pile area to determine the distribution of the imposed loads. The resulting distributions for the two O-cell tests on piles B1 and G1 are shown in Figures 5A and 5B, respectively. The pile buoyant weight is subtracted from the start of the distribution at the O-cell. Note the wider separations between the load distribution curves for the last two increments. This is the effect of the strain-softening causing the load reaching the upper gage levels to be larger than the applied load increment when the shaft resistance is reduced.

Above the O-cells, the load distributions reflect the negative direction shaft resistance. A distribution showing the resistance distribution for an equivalent distribution of positive shaft resistance (equal to the negative direction resistance) is obtained by "flipping" the upward distribution curve. That is, the negative shaft resistance above the O-cell level is turned to positive shaft resistance rising from the O-cell load as indicated by the "flipped" curve in each figure, and the starting load values at the pile head are 4,010 KN for Pile B1 and 3,350 KN for Pile G1.

Dynamic tests

Three blow records from the dynamic tests on the companion piles, Piles B2 and G2 were analyzed in the CAPWAP program (Rausche et al. 1972).

The evaluated total, shaft, and toe resistances of the CAPWAP evaluation of the dynamic test records are compiled in Table 1. The table shows the CAPWAP determined resistances above and below the O-cell level to facilitate a comparison to the resistances determined for Piles B1 and G1 from the O-cell tests.

Neither shaft resistance nor toe resistance was fully mobilized in the dynamic test on Pile B2. For Pile G2, the impact was able to fully mobilize the shaft resistance, as indicated by the similarity between the shaft resistance values for Blows #3 and #4. The maximum toe movement calculated by the CAPWAP analysis of Blow #4 was 16 mm, which is about half to a quarter of the maximum downward movement of the O-cell test on the companion test piles.

A comparison between the CAPWAP determined shaft resistances and the directly measured resistances in the O-cell tests can neither be used to confirm an agreement between the methods—Pile B1—nor a disagreement—Pile G1. The two static tests show a 20 % difference between each other. A comparison between the methods can only be fully relevant when the tests are made on the same pile, not when made on a companion pile.

Table 1 CAPWAP Results and summary of O-cell results

	Pile B2 Blow #3	Pile G2 Blow #3	Pile G2 Blow #4
Shaft resistance above O-cell depth (KN)	1,480*)	**2,110**	**2,120**
Shaft resistance below O-cell depth (KN)	525*)	740	1,010
Toe resistance (KN)	205*)	290*)	670
Shaft plus toe resistance below O-cell (KN)	730*)	1,030*)	1,680
Total resistance (KN)	2,210*)	3,140*)	3,800**)

*) Not fully mobilized **) Toe movement = 16 mm

	Pile B1	Pile G1
O-cell upward (KN)	**1,970**	**1,640**
O-cell downward (KN)	2,040	1,710
O-cell total resistance (KN)	4,010	3,350

Table 2 Total Shaft Resistance from CPTU and CPT Methods

Method	R_s (KN)
Eslami and Fellenius (1997)	1,593
DeRuiter and Beringen (1979) "Dutch"	1,645
Bustamante and Gianeselli (1982) "LCPC"	1,504
Schmertmann (1978)	1,875

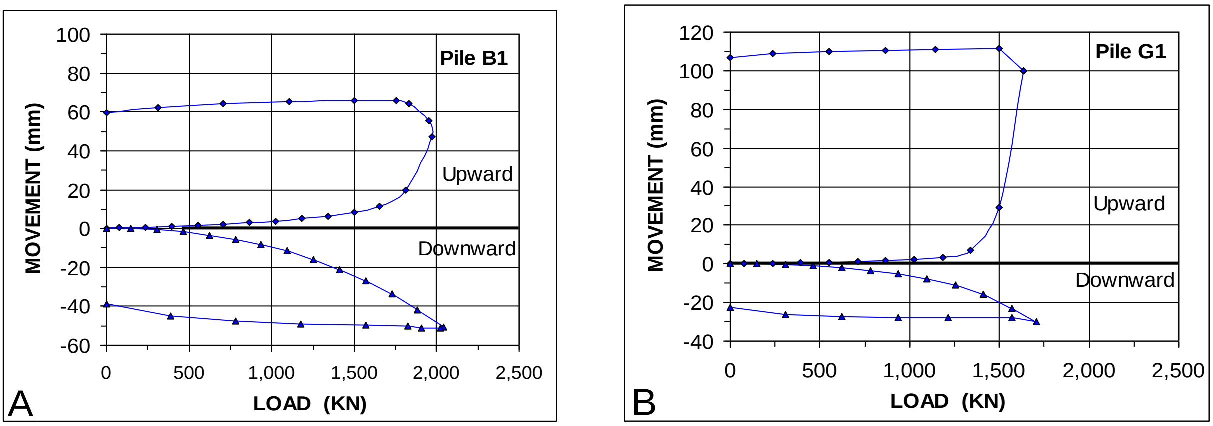

Fig.2 Load-movement curves from O-cell tests on Piles B1 and G1

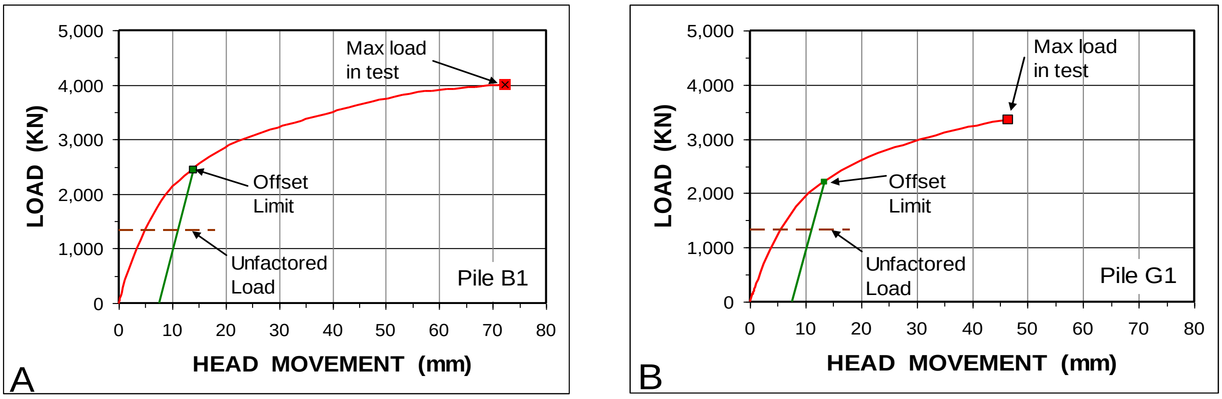

Fig.3 Equivalent head-down load-movement curves for Piles B1 and G1.

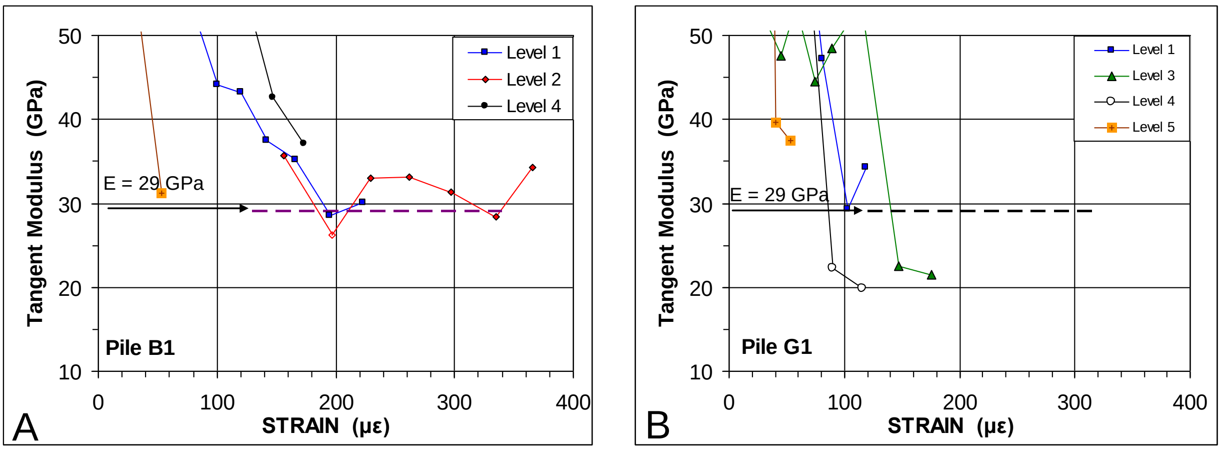

Fig.4 Tangent-modulus plots for Piles B1 and G1

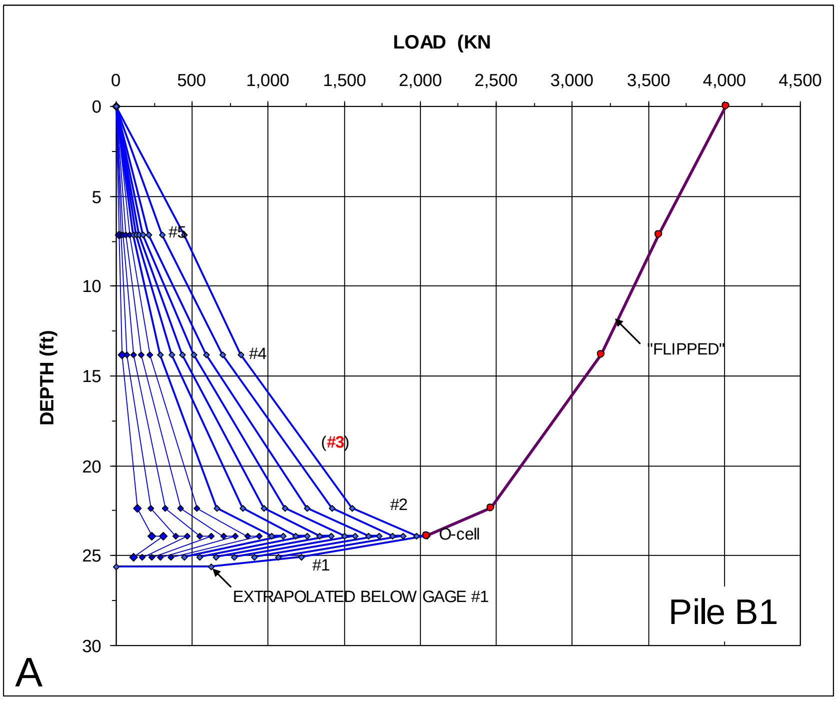

Fig.5A Load distribution in Pile B1 with the distribution for the maximum load "flipped" (Gage Level 3 records were not usable)

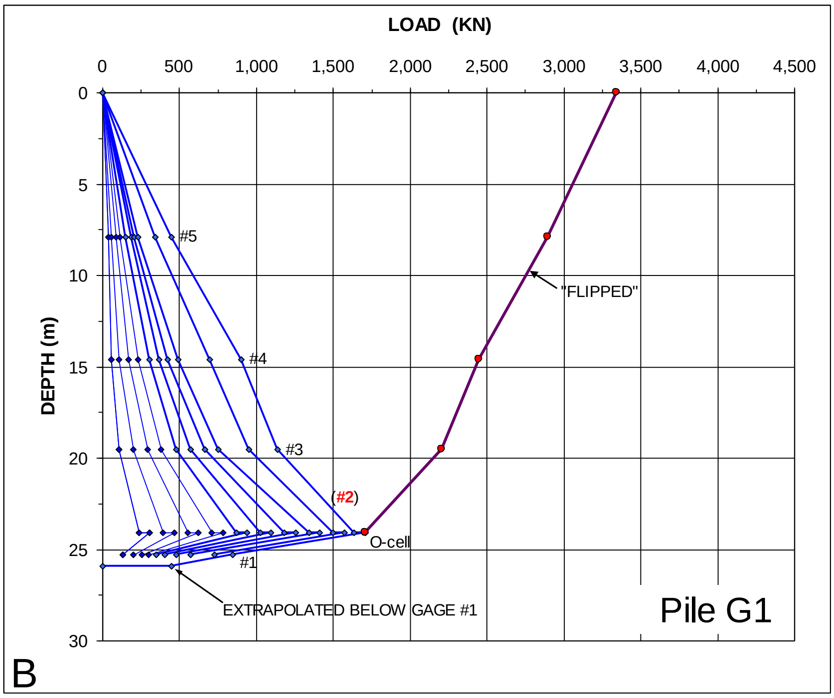

Fig.5B Load distribution in Pile G1 with the distribution for the maximum load "flipped" (Gage Level 2 records were not usable).

Results Compilation

The shaft resistance distributions determined from the strain-gage values of the O-cell tests and the CAPWAP analyses are shown in Figure 6. Also included is the shaft resistance above the O-cell level calculated from the CPTU sounding at test location B, using the CPTU method proposed by Eslami and Fellenius (1997). The q_t-resistance distribution is indicated in the figure as reference to the soil profile. Table 2 shows the total shaft resistance values determined by also three additional cone sounding methods, CPT-methods.

The total shaft resistance calculated from the cone sounding methods appears to be close to the measured values. However, Figure 6 cone resistance curve indicates that an overestimation in the sand that is compensated by an underestimation in the clay.

To obtain a general relation for use in the design, the O-cell distribution of shaft resistance has been correlated to an effective stress distribution. The fit of measured and calculated distributions is shown in Figure 7 along with the distribution of beta-coefficient producing the fit. In the upper 3.0 m, the undocumented fill, the beta-coefficient is 0.7. In the sand layer from here to a depth of 12m depth, the coefficient reduces to 0.3, Hereunder, in the clay layer, the value is 0.25. In the stiff clay layer from 19 m to the glacial till, the value increases to 0.4 at the O-cell level. In the glacial till, the beta-coefficient is 0.8. The back-calculated beta-coefficients agree well with the general ranges mentioned in the Canadian Foundation Engineering Manual (2006) and by Fellenius (2008).

The piled foundation design was carried out employing the principle of the "Unified Design" (Fellenius 1984, 2004, 2006, 2009), which considers three main requirements, as follows.

(**1**) the pile capacity must be larger with a margin (factor of safety of load and resistance factors) than the sum of sustained (dead) and transient (live) loads,

(**2**), the sum of sustained load and drag load must be smaller with a margin than the pile structural strength, and

(**3**) the settlement of the piled foundation must not be more than the maximum acceptable value. The settlement of the piled foundation is governed by the settlement at the neutral plane, which is the location of the pile force equilibrium and where the pile and the piles settle equally (the settlement equilibrium).

The Unified Design method is accepted in many standards and codes, such as the Canadian Foundation Engineering Manual (Canadian Geotechnical Society 2006), the Canadian Highway Design Code (2006), the Australian Piling Standard (1995), the US Federal Highway Design Manual (Hannigan et al. 2006), and the Government of Hong Kong Design Guide (2006). The main tenet of the method is that the location of the neutral plane, and, therefore, the drag load and the pile settlement is a function of the load-movement response of the pile toe to the applied load and to any downdrag caused by soil settlement. The load-movement response is best obtained from direct testing, such as an O-cell test, but lacking test data, it can also be calculated from general principles.

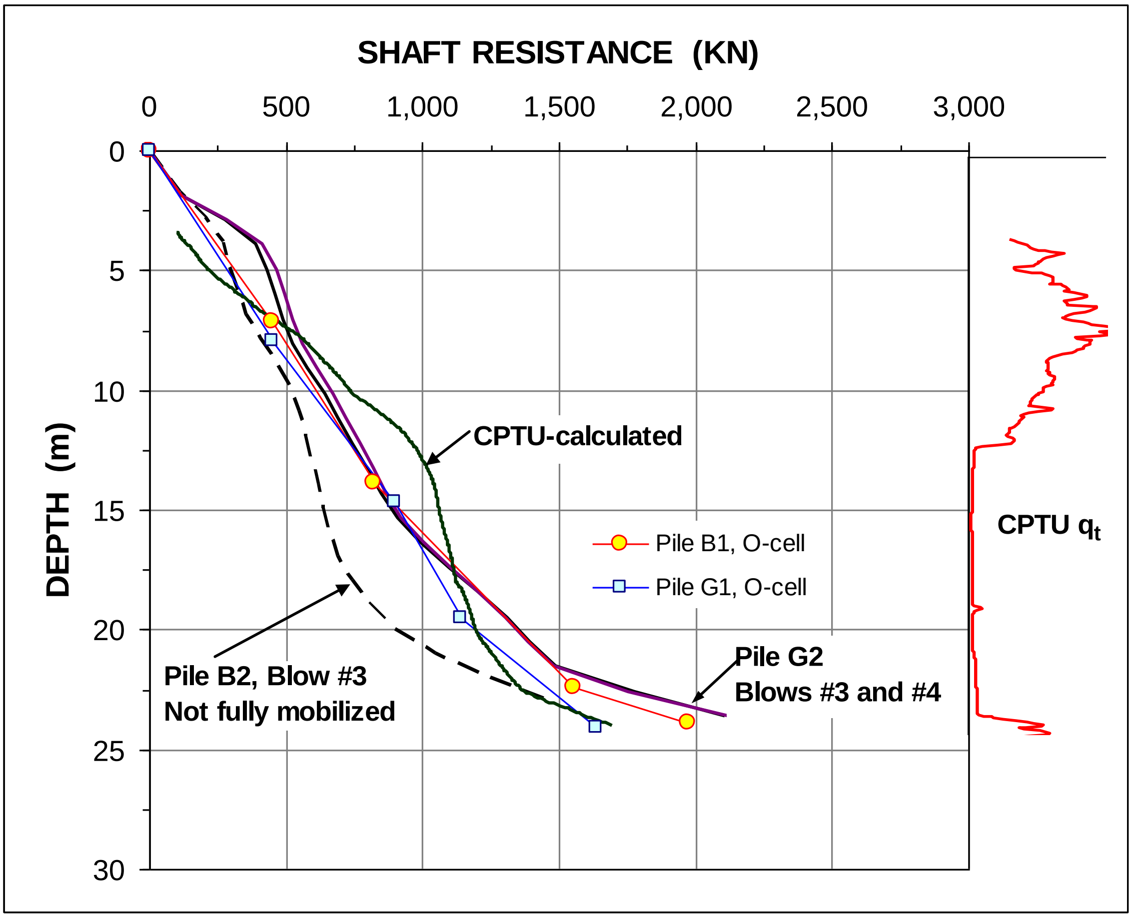

Fig.6 Load distributions above the O-cell level from the O-cell tests on Piles B1 and G1, the CAPWAP analyses on impacts on Piles B2 and G2, and calculated from the CPTU sounding at test location B. The q_t-diagram serves as reference to the soil layering

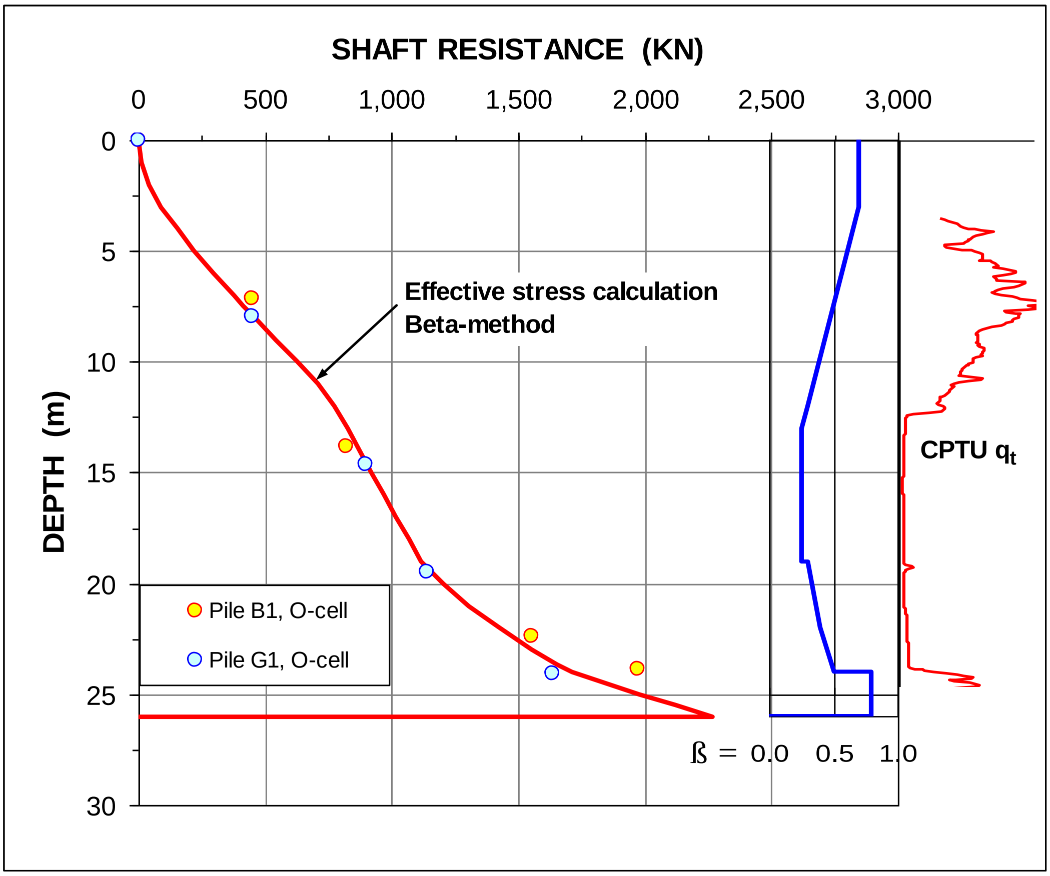

Fig.7 Load distributions above the O-cell level in Pile B1 and G1 and strain-gage values fitted to an effective stress analysis for the indicated Beta-coefficients.The q_t-diagram serves as reference to the soil layering.

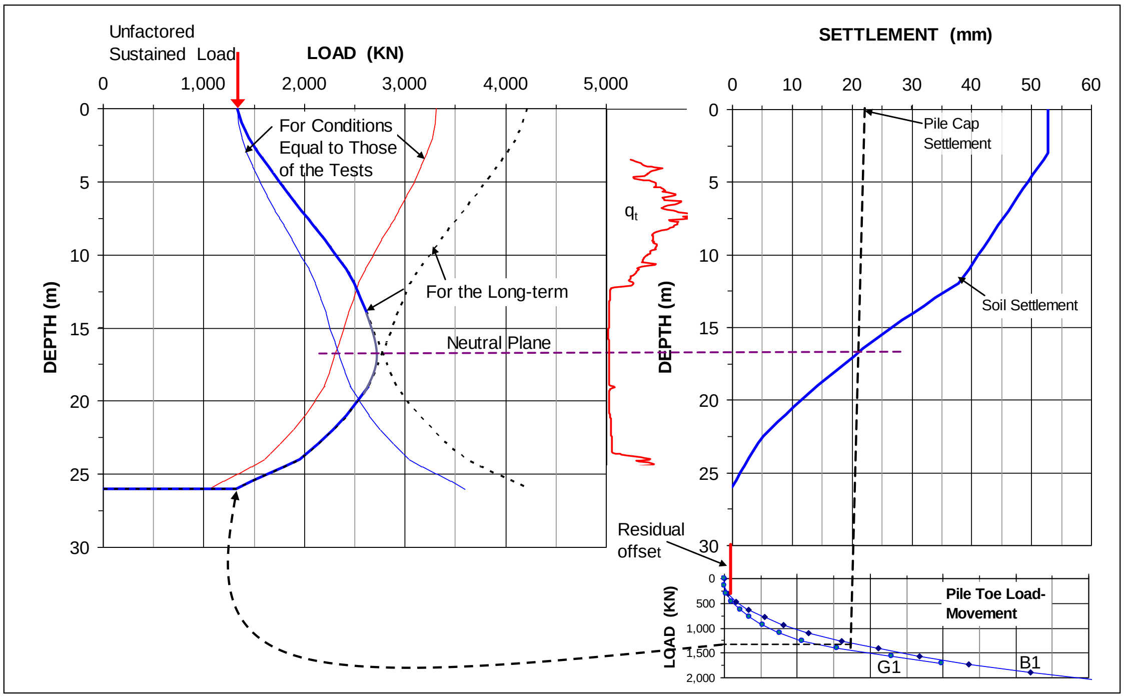

Fig.8 Distribution of load and settlement showing matching the pile toe load and pile toe movement to the pile toe load-movement response.

Use of the Results in the Design

The design of the subject case is illustrated in Figure 8, above, showing the long-term load distribution in the pile starting from the applied sustained load and increasing downward due to the accumulated loads from negative skin friction, assumed fully mobilized. The figure shows both the curves back-calculated from the test results and the long-term conditions after additional fill is placed on the ground. At the neutral plane, a transition from negative to positive direction occurs, and the load decreases down to the pile toe where the pile toe load is determined by the residual load prior to constructing the pile (shown as "residual offset" in the figure). The long-term curve descending from the sustained load value represents the load-transfer, as the negative skin friction gradually adds drag load and the sustained load and drag load gradually work their way down, and the pile penetration into the soil, so forced by the soil settlement. The calculated distribution of soil settlement is shown in the diagram to the right. As indicated, the relative penetration of the pile toe into the soil must correspond to the assigned pile toe load used for determining the location of the force equilibrium. If an initial design shows a lack of agreement in this regard, the analysis needs to be repeated until movement and force agree.

Figure 8 shows the conditions for force and movement equilibrium using the test results and the assumed conditions. As mentioned, the pile capacity is satisfactory for the 1,300 KN load sustained load from the structure to be supported by the piles. The maximum load in the pile occurs at the neutral plane and is estimated to be about 2,700 KN, which is about twice the sustained load, but well within the axial structural strength of the pile. The settlement at the neutral plane is about 20 mm and the 'elastic' shortening of the pile will be just a few millimetre. The acceptable maximum long-term foundation settlement for the project is about 25 mm (one inch). Should the actual settlement become larger than the estimated value, this would cause an increase of the pile penetration into the glacial till and a rapid increase of the pile toe force with a subsequent lowering of the neutral plane, which would counter the effect of the larger soil settlement. Thus, the testing and the design analysis indicate that there is no need for having the piles constructed through the glacial till to bearing on or in the bedrock.

Conclusions

(1) The O-cell tests indicate that the capacity of a pile construction to bearing in the glacial till below the sedimentary deposits will be larger than 3,000 KN, about 2.5 times the allowable load.

(2) The back-analysis of the shaft resistance distribution indicate beta-coefficients in the surficial sands and in the clay that agree well with published values, and they can serve as calibrated values for the pile design at the project site.

(3) The dynamic tests appear to agree well with the static tests. However, the dynamic tests were carried out on companion piles and the results of the two static test vary quite a bit from each other. Therefore, the comparison dynamic versus static is inconclusive.

(4) The design analysis according to the Unified Method indicate that the maximum load (sustained load plus drag load) is well within acceptable limits for the pile structural strength and that the expected settlement of the piled foundations will be smaller than the assigned limit of 25 mm. Therefore, the performed tests prove that the project piles can be constructed to bearing in the glacial till and do not need to be taken onto or into the bedrock.

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