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PILE LOADING TESTS AT GOLDEN EARS BRIDGE

Ali Amini, Trow Associates Inc., Burnaby, BC, Canada
Bengt H. Fellenius, Calgary, AB, Canada
Makram Sabbagh, AMEC, Burnaby, BC, Canada
Ernest Naesgaard, Trow Associates Inc., Burnaby, BC, Canada
Michael Buehler, Golden Crossing Constructors JV, Langley, BC, Canada

ABSTRACT
The Golden Ears Bridge is a new cable-stayed bridge over the Fraser River connecting Maple Ridge and Pitt Meadows to Langley and Surrey in BC, Canada. It includes a 970 m river crossing and a total length of over 2.4 km including approach structures. All structures are supported on either 2.5 m diameter bored piles or 0.35 m circular driven spun cast cylinder piles. All piles are shaft-bearing in soils consisting of normally consolidated to lightly overconsolidated soft to stiff clay and loose to medium sand. Four static pile loading tests were carried out as follows: (1) one Osterberg Cell test on a 2.5 m diameter, 74 m long strain-gage instrumented, bored pile in sand and clay, (2) one head-down loading test on a 2.5 m diameter, 32 m long strain-gage instrumented, bored pile in clay, (3) two head-down loading tests on 0.35 m diameter driven spun cast concrete cylinder piles in clay. The main focus of this paper is the shaft resistance of piles in clay. Shaft capacity was calculated using three methods: the effective stress (beta) method, two methods based on CPT and CPTU soundings (LCPC and EF methods), and the API recommendations. The API alpha method gave good agreement with test shaft capacities in clay for the bored piles and under-predicted the capacities for the driven pile. The CPT and CPTU methods underestimated the shaft resistance by 35 % to 60 % for bored piles and more so for driven precast concrete piles. Back calculated Beta values in clay ranged from 0.25 through 0.3 for bored piles.

RÉSUMÉ
“Golden Ears Bridge” est un pont suspendu traversant le fleuve Fraser, reliant Maple Ridge et Pitt Meadows à Langley et Surrey, en Colombie-Britannique, Canada. Il a une portée totale de 970 m au dessus du fleuve, et plus de 2.4 km de longueur avec les structures d’approche. Les structures sont fondées sur des pieux forés de 2.5 m de diamètre ou sur des pieux en béton battus de 0.35 m. La capacité axiale des pieux provient du frottement latéral extérieur développé dans les argiles molles à raides, normalement consolidées à légèrement surconsolidées, et des sables lâches à compacts. Quatre essais de chargement statiques ont été effectués de la façon suivante: (1) un essai de chargement utilisant des cellules Osterberg sur un pieu de 2.5m de diamètre par 74m de longueur, instrumenté avec des jauges de déformation, installé dans le sable et l’argile, (2) un essai utilisant la méthode classique de chargement sur un pieu de 2.5m de diamètre par 32m de longueur, instrumenté avec des jauges de déformation, installé dans l’argile, (3) deux essais utilisant la méthode classique de chargement sur des pieux en béton battus de 0.35m de diamètre. installés dans l’argile. L’objectif principal de cet article est de mettre en évidence le frottement latéral des pieux installés dans l’argile. La résistance du fût a été calculée à l’aide de trois méthodes: la méthode de contrainte effective (beta). deux méthodes basées sur les sondages CPT et CPTU (LCPC et EF). et les recommandations du API. La méthode API alfa a donné des résultats concordant aux résistances du fût dans l’argile déterminées des essais de chargement sur les pieux forés. et a sous-estimé la résistance des pieux battus. Les méthodes basées sur les CPT et CPTU ont sous-estimé la résistance du fût de 35% à 60% pour les pieux forés.

1. INTRODUCTION

The Golden Ears Bridge is an under-construction new cable-stayed bridge over the Fraser River connecting Maple Ridge and Pitt Meadows to Langley and Surrey in BC. Canada. The bridge is expected to be completed in 2009. It includes a 970 m river crossing and an over 2.4 km total length including approach structures. The main bridge has four marine piers each supported on a group of 12 bored piles of 2.5 m diameter and 75 m to 85 m embedment depths. The south approach structure and ramps are placed on 2.5 m diameter bored piles to 80 m embedment. Unfactored ultimate axial resistance of up to about 60 MN for each single pile was required for both marine and south approach piles. The north approach structures are supported on groups of 0.35 m circular driven spuncast concrete piles with embedment depths ranging from 12 m through 36 m. All piles are shaft-bearing in post-glacial normally consolidated (NC) to lightly over-consolidated (OC) soft to stiff silty clay occasionally with a surface layer of loose to dense sand. This is the first use of such long and large diameter shaft-bearing bored piles in this region.

Different methods of pile capacity calculation were used to estimate the pile capacity, such as the API approach, effective stress analysis, and methods based on results of cone penetrometer soundings (the LCPC CPT-method and the Eslami-Fellenius CPTU method).

The calculations resulted in a wide range of axial capacities. It was therefore necessary to calibrate the calculations to the specific pile construction methods and soil conditions, and to confirm the capacities of the proposed piles. One head-down static loading test and one O-Cell test (Osterberg.1989) were performed on each of two 2.5 m diameter strain-gage instrumented bored piles. In addition, head-down static loading tests were performed on a two uninstrumented 0.35 m diameter driven precast concrete piles. The analysis methods were correlated to the results and case-adjusted versions of the methods were prepared by fitting the methods to the results of the static loading tests.

Section 5 presents a brief description for these methods. Figure 1 is a key plan showing the approximate location of the pile loading tests.

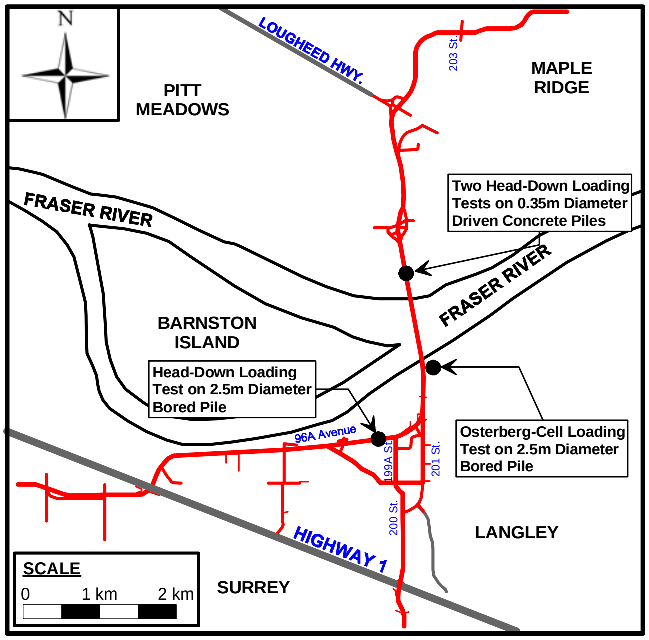

Figure 1- Site key plan and approximate location of test piles.

The purpose of this paper is to present a summary of the pile loading tests results and the methodology used for calibration of pile capacity calculation methods, in particular. in the clay layers. Each loading test is described briefly in a separate section followed by a discussion section that addresses the results from each site.

2. HEAD-DOWN PILE LOADING TEST ON 2.5 m DIAMETER, 32 m LONG BORED PILE

The soils at the location of head-down test pile consisted of 2.5 m of gravelly sand fill over 3 m of sandy gravel, overlying lightly to over-consolidated stiff clay to the maximum depth of exploration of 50 m (Figure 1) Some thin sand layers were encountered between 33 m and 37 m depth in one of the two CPTU soundings in the vicinity of the test pile. Water table was at 2.1 m depth. Artesian pressures of about 70 KPa were measured at 100 m depth in the clay (Deepest borehole was 118 m). The artesian pressures were assumed to linearly decrease to hydrostatic pressure at the underside of the sand fill.

Figure 2 shows profiles of Atterberg limits, water content, cone stress q_t, and undrained shear strength, S_u evaluated from Nilcon field vane tests at different boreholes at the site. An N_{KT}-value of 17, as defined in Equation 1, was obtained for the clay above the pile toe. The peak vane shear strength values were not corrected for strain rate effects. The S_u/σ'_{vo} ratio was 0.4 (σ'_{vo} is the effective vertical stress).

$$S_u = \frac{q_t - \sigma_{vo}}{N_{KT}} \qquad \text{Eq. 1}$$

Where σ_{vo} is the total vertical stress and q_t is the pore pressure corrected cone stress.

A strain-gage instrumented nominal 2.5 m diameter, 32 m long bored pile was constructed by Bilfinger Berger BOT GmbH from Germany on October 14. 2006 on the south bank of the Fraser River close to the project alignment. The construction consisted of advancing a 2.5 m O.D

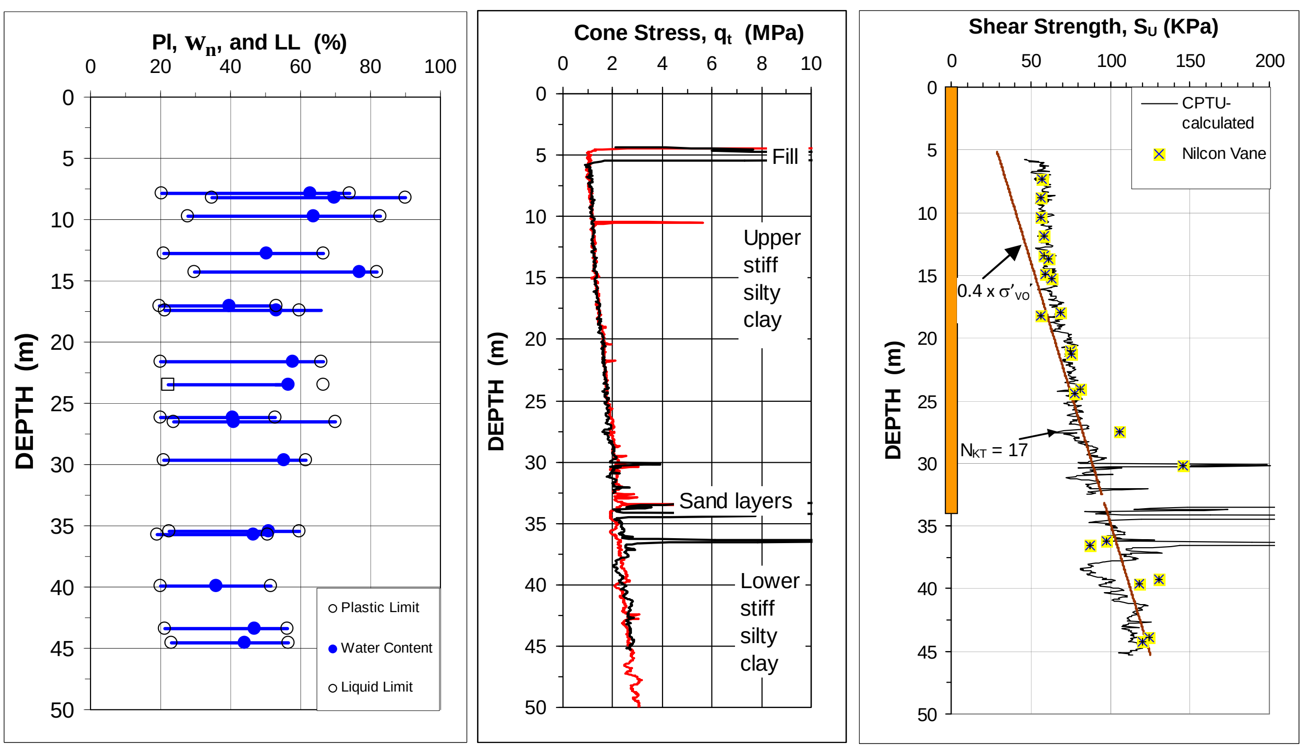

Figure 2 Soil conditions and shaft resistance profile for head-down pile loading test on a 2.5m diameter, 32m long bored pile.

steel temporary casing a few metre ahead of excavation with a special spherical grab. The grab weighed about 20 tonnes and had a diameter of 2.5 m when fully opened. The casing was withdrawn during the concrete placement. Installation and withdrawal of the temporary casing were carried out using oscillatory rotating motion of the casing. No drilling mud was used. Oscillatory rotation of casing in conjunction with oversized casing bits was expected to create a spiral shape macro-fabric on the shaft wall resulting in some enhancement in the shaft resistance. The purpose of the test was to assess the effect of this installation methodology on the shaft capacity.

The finished pile head was 2.4 m below the ground surface and the pile toe was at a depth of 34.4 m. The test pile was installed midway between two reaction piles; 2.5 m diameter, 50 m long bored piles supporting an abutment wall. The reaction force was provided by the weight of the abutment wall (~6MN) and the uplift resistance of the two piles. The center to center spacing between the test pile and reaction piles was about three times the pile diameter. The pile was tested on January 18, 2007, 96 days after construction. The instrumentation included 20 single vibrating wire strain gages in the piles, 4 displacement gages at the pile head, and 2 displacement gages on each side of the abutment wall.

Figure 3 presents the load-movement of the pile head. For bored piles with capacity greater than 10 MN. O'Neill and Reese (1999) recommended that the load at a displacement equal to 5 % of the pile diameter, i.e., 130 mm, be considered as the axial pile capacity, if plunging cannot be achieved. Figure 3 shows that the pile plunged at 16 MN load at a movement of 30 mm, well before reaching 5 % displacement.

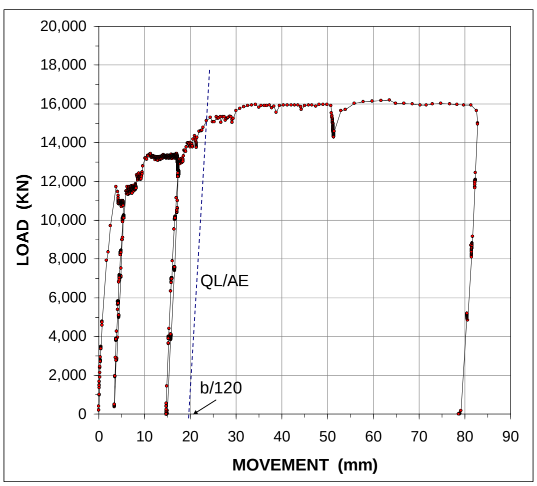

Figure 3 Load-movement of pile head for the head-down loading test on the 2.5m diameter, 32 m long bored pile.

The distribution of shaft resistance was interpreted from the strain gages and is shown in Figure 4 (solid circle symbols). An average back-calculated Beta coefficient of 0.32 was found for the shaft resistance in the clay. The maximum toe resistance was interpreted as 2.6MN (i.e., the 16.0 MN failure load plus 2.4 MN pile buoyant weight minus 15.8 MN of interpreted shaft resistance. Residual load not

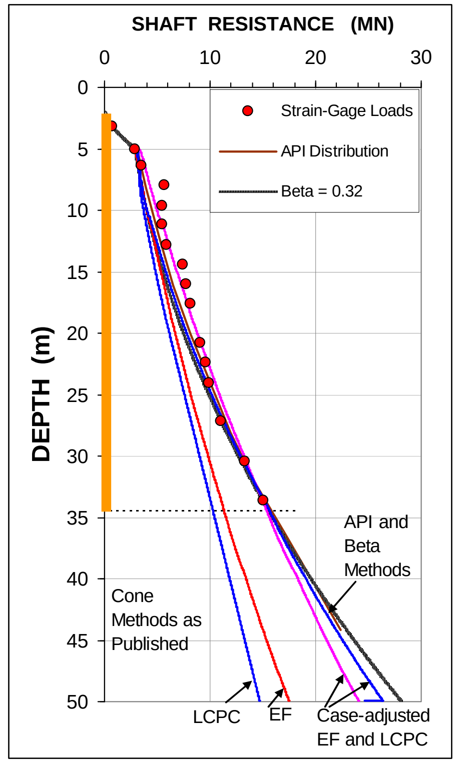

Figure 4 Load distribution for the head-down loading test on the 2.5m diameter, 32 m long bored pile.

included). This toe resistance value is smaller than the toe capacity, R_t, calculated according to Equation 2.

$$R_T = 9 S_u A_{toe}$$
$$R_T = 9(100kPa)(4.9m^2) = 4.4MN \qquad \text{Eq. 2}$$

Equation 2 does not include the buoyant weight, 2.4 MN, of the pile. The 2.5 m nominal diameter is used. For discussion regarding the cone sounding analyses (LCPC and EF methods) and API methods, see Section 5.3.

3. OSTERBERG-CELL LOADING TEST ON A 2.5 m DIAMETER 74 m LONG BORED PILE

The soil profile at the location of the O-cell tested pile consisted of 17 m of loose to medium silty sand to sand overlying 21 m of medium to dense fine to medium sand overlying stiff NC to lightly OC silty clay with intermittent thin silty sandy layers to depth beyond 100 m. The groundwater table was at 3 m depth below ground surface. The pore water pressure in the upper sand units was assumed hydrostatically distributed. The artesian pressures were assumed to linearly decrease to hydrostatic pressure at the underside of the sand layer.

Figure 5 shows profiles of Atterberg limits and water content, pore pressure corrected cone resistance, and undrained shear strength, obtained using Equation 1. No vane shear values were available at the location of the test pile and the same N_{KT} value of 17 (see Figure 2) has been used with the CPTU sounding at the site to estimate S_u.

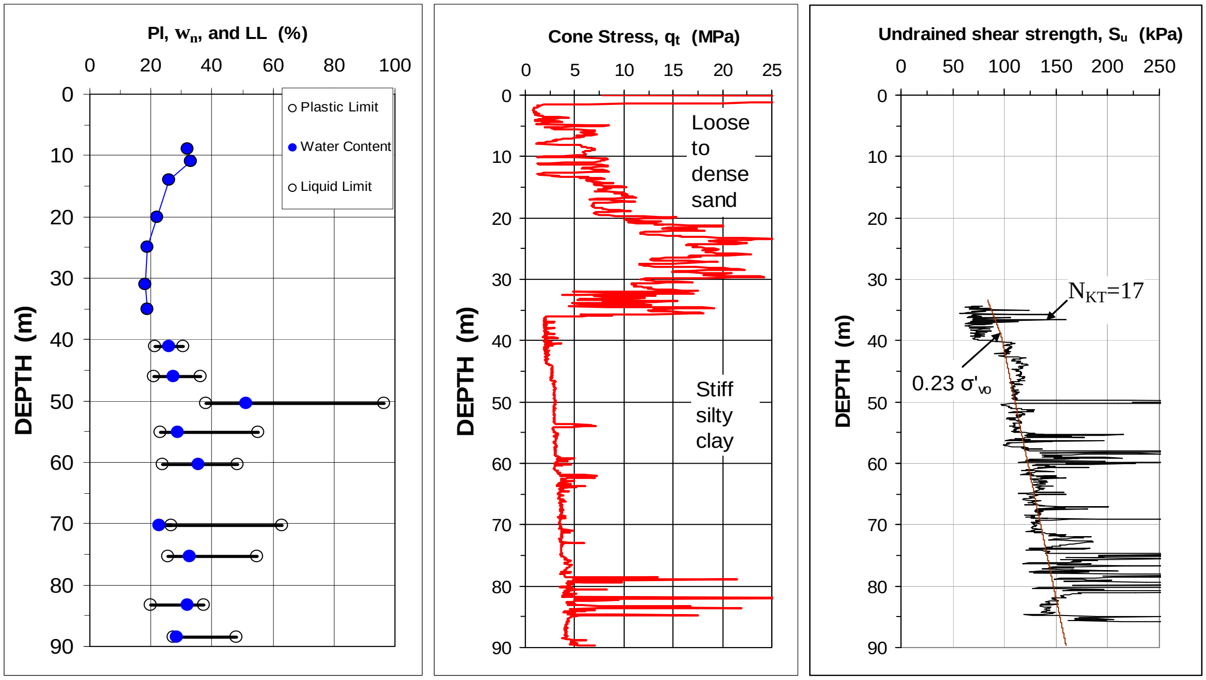

Figure 5 Soil conditions and shaft resistance profile for O-cell pile loading test on a 2.5 m diameter, 74.5 m long bored pile.

On May 19, 2006, a nominal 2.5 m diameter, 74.5 m long bored pile was constructed on the south bank of the Fraser River next to the project alignment. The pile construction included a permanent 2.5 m diameter steel casing vibrated to a depth of 21 m into the ground and extending 6.75 m above the ground surface. The same spherical grab described in Section 2 was used for excavation of the shaft Polymer slurry with a positive head of about 7.5 m above the water table was maintained to help stabilizing the shaft walls below the casing. During and after pile excavation, sonar caliper tests were performed to obtain a three-dimensional shape of the excavated hole. An average shaft diameter of 2.6 m and a general inclination of about 1 % were found. Two O-cell assemblies and corresponding instrumentation were attached to the reinforcing steel cage by Loadtest Inc., Florida. The lower O-cell assembly was placed at 70.5 m depth and the upper O-cell assembly at 44 m depth. The lower assembly had three O-cells with a total capacity of 18.7 MN and the upper had three O-cells with a total capacity of 48 MN.

The instrumentation included vibrating wire displacement transducers positioned between the lower and upper plates of both O-cell assemblies and vibrating-wire strain gage pairs at nine levels in the pile. Details of the results of the strain measurements are not included in this paper. One steel pipe, extending from the pile head to the bottom plate of each O-cell assembly, was installed to vent the break in the pile formed by the expansion of the O-cells. The pipes were filled with water prior to the start of the test. Immediately after the reinforcing cage had been lowered into the shaft, the shaft was concreted through a tremie pipe.

The O-cell loading test was performed 30 days after the pile was completed. The loading procedure was by adding increments of load the O-cell assembly every ten minutes according to the following planned schedule.

(1) Expand lower O-cell assembly to fail the lower segment of the pile (the 4.0 m long segment below the lower O-cell) in downward direction to determine toe capacity.

(2) open lower O-cell assembly to let it drain while expanding upper O-cell assembly (at 44.0 m depth) to fail the segment between upper and lower O-cell levels in downward direction to determine shaft capacity of the middle segment.

(3) close the lower O-cell assembly while expanding upper O-cell to fail the upper segment (the segment above upper O-cell) upward to determine its shaft capacity.

The observed upward and downward load-movements measured in Stage 1 are presented in Figure 6. As shown, when increasing the O-cell load from 7.1 MN (65 mm downward movement), large differential expansion of the O-cells indicated that the 4.0 m long section below the O-cell level started to tilt. Attempts to adjust the tilt were not successful, and the cells were unloaded from a maximum load of 8.0 MN at 140 mm downward movement.

As also indicated, the downward load-movement curve suggests that prior to the start of the test, an about 3.5 MN residual (locked in) load existed at the lower O-cell level. The locked-in load is smaller than the 5.7 MN buoyant weight of the pile at the O-cell level.

The Stage 2 upward and downward load-movement curves from the upper O-cell level are shown in Figure 7.

The pile was loaded in 20 increments to a maximum O-cell load of 29.0 MN. At increment No. 18, the lower segment became engaged due to seizure of lower O-cells, transferring some load to the lower segment. This was likely caused by differential movements of the lower O-cells due to that the short lower pile segment had tilted toward the end of Stage 1.

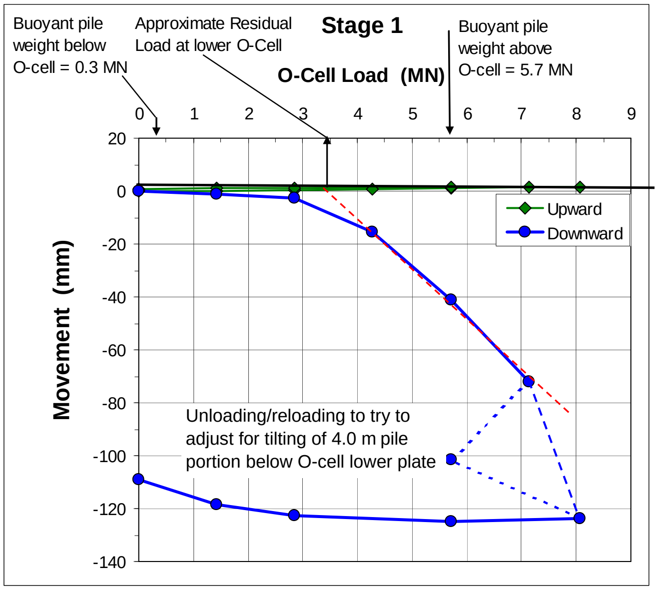

Figure 6 Stage 1, lower O-cell load-movements for the Golden Ears test pile.

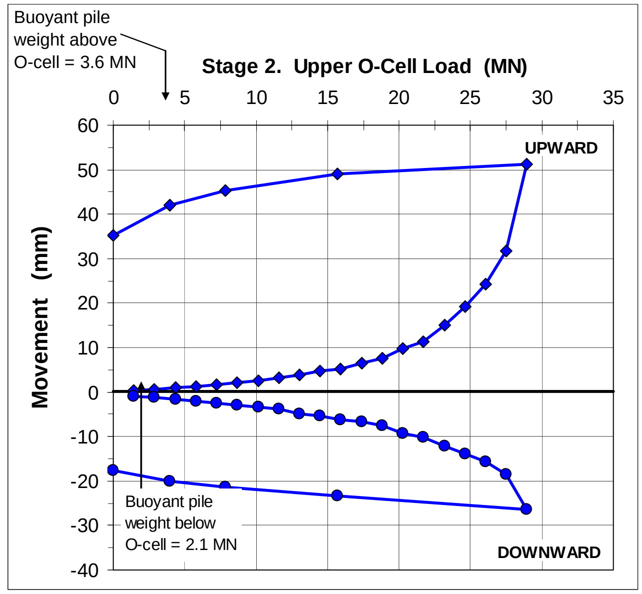

Figure 7 Stage 2. upper O-cell load-movements for the Golden Ears test pile.

Both the upper and the middle segments are considered to have reached the ultimate shaft resistance. Therefore, the planned next test stage, Stage 3, was cancelled. The upper segment is considered to have reached the ultimate resistance at the 29.0 MN maximum load minus the 3.6 MN buoyant weight, i.e., the shaft resistance was 25.4 MN. The upward movement was then 50 mm. The shaft resistance of the middle segment was interpreted as the O-cell load measured before the cells engaged with the lower segment plus buoyant weight of middle segment, i.e., 28.2 MN (26 MN plus 2.2 MN). The downward movement of the middle segment was then 25 mm. The pile shaft resistance at depths 44 m, 70.5 m and 74.5 m were thus interpreted as 25.4 MN, 53.6 MN, and 58.1 MN, respectively.

The mentioned shaft resistance values were used to calibrate effective stress (Beta) analysis, CPT and CPTU calculations, and API methods. Figure 8 shows the O-cell loads and loads interpreted from the strain-gage values. Also shown are the distributions calculated from the case-adjusted sounding methods (LCPC and EF) and the API recommendations (see Section 5, below). The effective stress calculations used saturated unit weights of 20 KN/m^3 and 17.5 KN/m^3 for the sand and clay, respectively, and the back-calculated Beta coefficients were 0.25, 0.40, and 0.25 for the upper silty sand with permanent steel casing, and the underlying sand and clay, respectively.

For discussion regarding the cone sounding analyses (LCPC and EF methods) and API method, see Section 5.3.

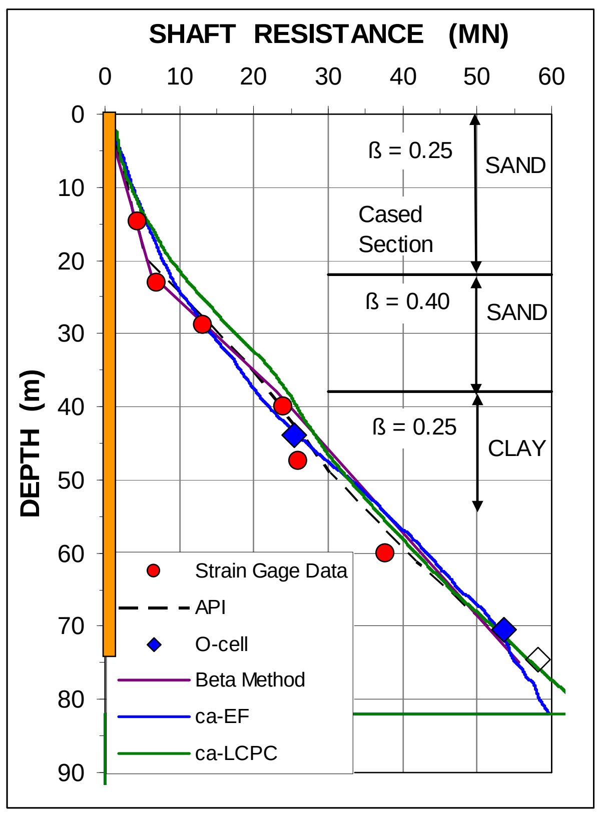

Figure 8 Shaft resistance distribution for the O-cell test. "Cased Section" is pile length within the permanent casing. "Strain-gage loads" are preliminary evaluation of the measurements (no consideration of residual loads). "Beta" and "API" are effective stress method and API method fitted to the load data. and "ca-EF" and ca-LCPC are case-adjusted CPT and CPTU methods, respectively.

4. HEAD-DOWN PILE LOADING TEST ON 357 mm DIAMETER DRIVEN SPUN CAST CYLINDER PILES

The subsoils at this site consisted of 12 m of soft NC silty clay/clayey silt, overlying lightly OC to OC silty clay with S_u/σ'_{vo} ~0.4. Figure 9 shows profiles of Atterberg limits, water content, cone stress, q_t, and undrained shear strength S_u obtained cone stress using an N_{KT}-value of 17. No vane shear values were available at the location of the test piles

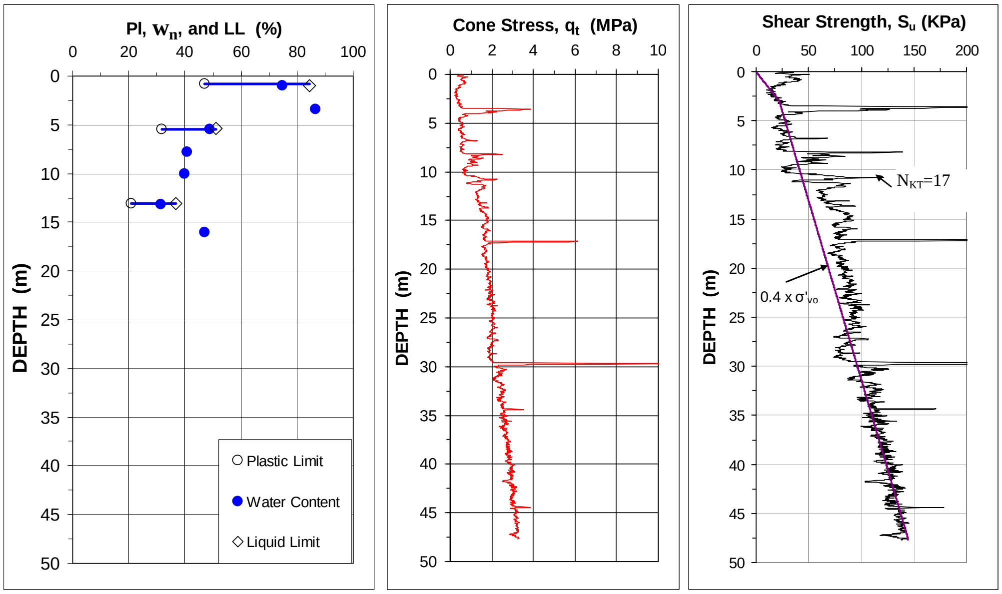

Figure 9 Soil conditions and shaft resistance profile at the site of the loading test on 357 mm diameter cylinder pile.

and an N_{KT} value of 17 (see Figure 2) was used based on vane shear tests in the vicinity.

Two head-down static loading tests were performed on 357 mm diameter, 36 m long, closed-toe circular spun cast concrete piles four months after driving. In one test, each load increment was held for 10 minutes, while it was held for 2 hours in the second test. Both piles were loaded to the maximum capacity of the hydraulic jack, 2.5 MN and the pile capacity was not reached for either test. Figure 10 shows the load-movement for the 2-hour increment-duration test. By visual extrapolation, an approximate pile capacity in the range of about 2,800 to 3,000 KN was interpreted. The value is essentially all shaft resistance, as the pile toe resistance in the clay is considered very small (Equation 2 returned an estimated pile toe capacity of 100 KN).

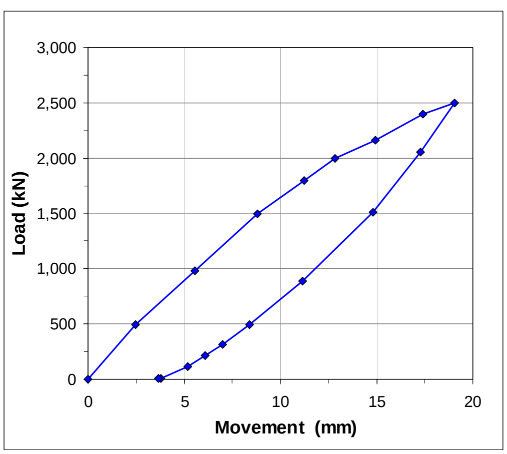

Figure 10 Pile loading tests with 2 hour increment duration for each load interval.

Calculations of pile shaft resistance by the two cone sounding methods. LCPC and EF. showed values of 1,100 KN and 1, 600 KN. respectively. Applying the case-adjusted values obtained for the two bored piles, the calculated shaft resistances become 2,000 KN and 2,300 KN, both somewhat shy of what can be intuitively extrapolated from the load-movement curve shown in Figure 10. An effective stress calculation using a beta-coefficient of 0.25, as found for the bored piles, gave a calculated shaft resistance of 1,400 KN.

5. DISCUSSION

5.1 Calibration of Alpha method

The Alpha method is the term for the total stress analysis, which uses the undrained shear strength, S_u, times a coefficient, α, as equal to the unit shaft resistance, r_s, in cohesive soils according to Equation 3.

$$r_s = \alpha \cdot S_u \qquad \text{Eq. 3}$$

Despite its simplicity, the alpha-value is expected to account for the behavior of the cohesive soils as well as the complex effects of pile installation on shaft capacity. Effects of installation of bored piles include soil disturbance, stress relief during excavation, increase in stress due to concreting, possible water migration from wet concrete to the interface soil, possible formation of mud-cake, etc. (O'Neill 2001). On the other hand, pile driving generally results in increase in lateral stresses and a higher level of soil disturbance. Alpha values for bored piles are generally expected to be smaller than those for driven piles, depending on the pile construction method and soil conditions. FHWA (1999) recommends α-values for bored piles in cohesive soils varying from 0.40 to 0.55 as a function of S_u. API (2000) recommends α-values ranging from 1.0 through 0.4 as a function of S_u/σ'_{vo} for piles in cohesive soils, as presented by Equations 4 and 5. This recommendation is based on Randolph and Murphy (1985) interpretation of driven pile loading test data base with majority of tests on steel pipe piles.

$$\alpha = 0.5 \cdot \left(\frac{S_u}{\sigma'_{vo}} \right)^{-0.5} \qquad \text{if } S_u/\sigma'_{vo} < 1 \qquad \text{Eq. 4}$$

$$\alpha = 0.5 \cdot \left(\frac{S_u}{\sigma'_{vo}} \right)^{-0.25} \qquad S_u/\sigma'_{vo} > 1 \qquad \text{Eq. 5}$$

Alpha-values back-calculated from the results of the tests on the two bored piles and the vane-shear calibrated cone resistances closely matched the API α–values. For the middle segment of the O-cell test with S_u/σ'_{vo} about 0.20, the average back calculated α-value is unity. For the head-down test, where the S_u/σ'_{vo} was about 0.4, the back calculated alpha-value was 0.6 to 1.0. It may be coincidental that API α-values, which were developed based on driven steel pipe piles, are similar to those found for the subject tests on the bored piles.

In contrast, the back-calculated alpha-values are considerably greater than those recommended by FHWA (1999) for bored piles. However, the S_u-values used in the back-calculations were obtained from field vane shear tests, whereas FHWA (1999) data base is based on UU tests. The differences between the back calculated and recommended values become even greater considering that S_u-values determined from field vane data are generally larger than UU test determined values (FHWA 2006). It is probable that the procedures used to construct the two bored piles are the main reasons for larger back-calculated α-values.

FHWA (1999) recommended correlation for α is based on S_u and cannot consider the effect of OCR properly. For example, an NC clay with high S_u-value would be treated the same as an OC clay with a similarly high S_u-value.

5.2 Effective stress (Beta) method

The effective stress method relates the unit shaft resistance to the in-situ vertical effective stress through a proportionality coefficient, the beta-coefficient, as presented in Equation 6.

$$r_s = \beta \cdot \sigma'_{v0} \qquad \text{Eq. 6}$$

As indicated in Figures 3 and 5, the back-calculated beta-coefficient in the clay for the two bored piles were 0.32 and 0.20. These values are larger than the beta-coefficients ranging from 0.15 through 0.20 found from back-calculations of static loading tests on driven steel pipe piles in the area (Fellenius 2008).

5.3 Cone penetration methods

Two cone penetration methods for correlating cone sounding results to shaft resistance are considered. The CPT-based LCPC method (Bustamante and Gianeselli 1982, CGS 2006), and the CPTU-based EF-method (Eslami and Fellenius 1997).

The main principle of the LCPC method is shown in Equation 7, determining the unit shaft resistance as the uncorrected cone stress, q_c, times a parameter, α_{LCPC}. Upper limits restrict the calculated resistances. Both the parameter and the limits depend on pile type, construction method, soil type, and ranges of the uncorrected cone stress, q_c.

$$r_s = \alpha_{LCPC} \cdot q_c \qquad \text{Eq. 7}$$

The CPTU EF method applies the q_t stress directly and, then, reassigns the q_t-value to a value denoted q_E by subtracting the measured pore pressure. (When applied to a CPT sounding, the EF method calculates the q_t-value using the neutral pore pressure and then subtracts the neutral pore pressure from q_t to obtain the q_E-value). As indicated in Equation 8, the shaft resistance is calculated by applying a coefficient, C_s, to the q_E-value that ranges from 0.02 through 0.08 in clay and silts, depending on the soil type, as characterized from the cone stress and sleeve friction values.

$$r_s = C_s \; q_E \qquad \text{Eq. 8}$$

Applying the LCPC method to the cone data produced shaft resistance values for the two bored piles that were much smaller than the values found in the tests. The method was therefore adjusted to fit the test data by removing the imposed limits of the published method and applying the pore pressure-corrected cone stress, q_t, instead of q_c. Then, a multiplier was applied to the so calculated shaft resistance to arrive at a case-adjusted LCPC distribution. For the head-down test, the multiplier was 1.35 in the clay. For the O-cell test, the multiplier for the LCPC method was 1.0 in the sand and 1.6 in the clay.

Also the EF-method underestimated the shaft resistances of the two bored test piles. For the head-down test, a case-adjusted fit to the test results was obtained by multiplying the C_s-coefficient with 1.4. For the O-cell pile, the shaft resistance in the sand and clay above 44 m depth also required a multiplier 1.4. However, for the section below 44 m, the adjustment had to be more than doubled.

Pile capacities in the region calculated from the two cone sounding methods usually agree quite well with the results from static loading tests (Fellenius 2008). However, those results are from tests on steel pipe piles, which may exhibit smaller shaft resistance than found for tests on concrete piles.

5.4 Driven piles test results

The pile capacity parameters obtained from driven piles are usually higher than those for bored piles. This is mainly because of the different installation method, which increases the lateral stresses due to driving of displacement piles

The driven piles were much more flexible than the 2.5 m bored piles. Flexible piles with displacements large enough to take the soil into its post-peak strains cause progressive failure and reduce the total shaft capacity. Using Randolph (2003) simplified relationship, a reduction factor of about 1.0 and 0.85 would apply to the bored test piles and the driven precast concrete test piles, respectively. It may be argued that the unit shaft resistance parameters obtained from flexible driven piles can be increased by a factor of about 1.2 to obtain unit shaft resistance parameter for an equivalent rigid driven pile.

6. SUMMARY AND CONCLUSIONS

One O-cell and one head-down loading test on 2.5 m diameter bored piles, and two head-down loading tests on 0.35 m diameter precast concrete piles were performed in the thick clay deposit at Golden Ears project site. As no dense/hard bearing layer existed at the site, the pile were dominantly shaft bearing (the toe resistance was very small). The main focus of this paper is to present the test results and discuss the ultimate shaft resistance in the clay at this site. It should be noted that the correlations and back-calculated shaft resistance parameters presented in this paper are for the specific construction methodologies and site conditions, and they may not apply to other sites and construction projects.

1. Alpha values recommended by FHWA (1999) significantly underestimated the shaft capacity of bored piles in clay at this site. It is believed that the pile construction procedures used at Golden Ears Bridge project resulted in shaft resistances significantly higher than bored piles in the FHWA data base. In addition, FHWA (1999) correlates alpha-values to S_u from UU test and this correlation cannot properly account for overconsolidation.
2. Alpha values recommended by API (2000) matched the bored pile test results with a calibration multiplier of about 1. This close match may be coincidental as API α-values were developed based on a data base consisting of mostly driven steel pipe piles. However, the API values agreed with the test results at both bored test pile sites with different depths and S_u/σ'_{vo} values. This agreement is attributed to API (2000) correlation of α to S_u/σ'_{vo}, which allows it to consider the effect of overconsolidation.
3. Difference in bored piles construction methods (with and without oscillatory temporary casings) had little effect on shaft capacities.
4. API (2000) alpha method under-predicted the shaft capacity of precast concrete piles driven in the clay.
5. The back-calculated beta-coefficients ranged from 0.25 through 0.3 for bored piles, which is larger than observed from back-calculated tests on driven steel pipe piles in the area.
6. Both the CPT (LCPC) an the CPTU (EF) cone sounding methods underestimated the pile shaft resistance. The methods were fitted to the test results (case-adjusted). The fit of the LCPC method was achieved by using q_t cone stress instead of q_c and disregarding all imposed limits on the shaft resistance, plus applying a multiplier of 1.35 to 1.6. The fit of the EF method was achieved by a multiplier of 1.4 above 44 m depth and more than 2 below.
7. The static loading tests on the instrumented bored piles showed the piles as a result of the construction method to have a shaft resistance larger than would have been considered available without the results of the tests.

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