Full-scale pile load testing of cast-in-place caissons using Osterberg load-cell method – Anthony Henday Drive Southeast Ring Road case study

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ABSTRACT
Primary purposes of full-scale pile load testing are to obtain directly measured in-situ ground-pile bond resistance characteristics, and to measure the behaviour (including settlement) of the test pile under axial loading to provide refined input for the design and construction of the foundation elements. The geotechnical engineer, project team and owner will gain a greater reliance on the load carrying behaviour and performance of the foundation elements compared to that based on a typical site investigation alone.

The general benefit to the owner consist of an increase in reliability and a reduction in total piling costs as a result of a reduction in total pile length due to higher shaft friction and/or end-bearing values than initially anticipated, in addition to an increase in the specified limit state geotechnical resistance factor from 0.4 to 0.6. Full scale pile load testing is beneficial on projects when the potential cost benefits and/or risk reductions out weigh the cost of performing the test(s) itself.

Three full-scale pile load tests of cast-in-place caissons loaded to failure using the Osterberg Load-Cell method were undertaken for the Anthony Henday Drive Southeast Ring Road project in Edmonton, Alberta. The paper presents and discusses an overview of the site conditions, the Osterberg load-cell design and set-up, caisson installation and field conditions, test results and analysis, estimated project cost savings and conclusions.

RÉSUMÉ
Les objectifs principaux des essais de charge à grande échelle de pieux sont d'obtenir, par mesure directe, les caractéristiques in-situ de résistance ultime d'interface sol-pieu et roc-pieu (si applicable) ainsi que d'évaluer le comportement (incluant les tassements) d'un pieu d'essai soumis à une charge axiale dans le but de raffiner la conception et la construction d'éléments de fondation. L'ingénieur géotechnique, l'équipe de projet et le propriétaire bénéficieront d'une confiance accrue en la performance et le comportement sous charge des éléments de fondation versus une conception basée uniquement sur une investigation typique.

Pour le propriétaire, l'avantage principal consiste en une confiance accrue et la réduction des coûts totaux de la mise en œuvre des pieux entrainée par la réduction de la longueur totale des pieux en raison de valeurs de frottement latéral et/ou de résistance à la pointe plus grandes qu'anticipées à l'origine, en plus de l'augmentation de 0,4 a 0,6 du facteur de résistance nécessaire au calcul à l'état limite. Les essais de charge à grande échelle de pieux sont avantageux pour les projets où les coûts-avantages et/ou les réductions des risques surpassent le coût d'effectuer le(s) test(s).

Dans le cadre du projet “Anthony Henday Drive Southeast Ring Road” à Edmonton en Alberta, des pieux coulés sur place on été chargé jusqu'à la rupture lors de trois essais de charge à grande échelle utilisant la méthode de la cellule d'Osterberg. Cet article présente un sommaire des conditions spécifiques à l'emplacement du projet, la conception et le réglage de la cellule d'Osterberg, les conditions et l'installation des pieux coulés sur place, l'analyse et les résultats des essais de charge, l'estimation des économies réalisées lors du projet ainsi que les conclusions.

Key words: Pile foundations, in-situ pile load testing, caissons and Osterberg load cell

1 INTRODUCTION

Three full-scale caisson pile load tests were undertaken along the proposed eastern alignment of Anthony Henday Drive Southeast Ring Road (AHDSERR) in Edmonton, Alberta. The testing program comprised construction of three dedicated test piles, installation of an Osterberg cell (O-cell) to apply the compressive load to each pile, instrumentation to monitor the behaviour of each pile during loading, monitoring of the pile load test, and analysis, design implications and reporting of the test results. The pile load testing was undertaken during the construction of AHDSERR. The three test pile locations (17th Street, Hwy 14 ramps and 34th Avenue at AHDSERR) were strategically selected as representative sites (i.e., ground conditions and loading) for the remaining bridge structures to be constructed. The purpose of the full-scale load tests were to obtain directly measured in-situ, ultimate ground/caisson bond resistance characteristics, and to measure the behaviour of the test pile under axial loading to provide updated input for the design and construction of the foundations for the remaining proposed bridge structures.

2 SUBSURFACE CONDITIONS

The subsurface and groundwater conditions at each test location were based on borehole drilling (soils) and coring (bedrock), in-situ and laboratory sample testing, cone-penetration testing and piezometer data within the proposed bridge structure foundation locations. The test pile locations were within 3 m to 17 m of the proposed foundation locations. The test pile auger holes were visually logged during drilling to confirm ground conditions and the design basis

2.1 17th Street at AHDSERR

The subsurface conditions encountered at the test pile location generally consist of a 3 m thick zone of silty clay fill, overlying a thin topsoil layer, over silty clay, all underlain by the main silty clay till unit, all over bedrock. Lenses of wet sand were encountered within the silty clay till unit at approximately 14 m and 24 m below ground surface. The silty clay unit and the silty clay till generally ranged from firm to very stiff with depth, with SPT 'N' values ranging from 4 to 14 blows per 300 mm penetration, with an average N-value of 10. The average undrained shear strength and CPT q_t (tip resistance) values generally ranged from 75 kPa to 125 kPa with depth and 1 MPa to 2.5 MPa with depth, respectively. The liquid limit (LL) and Plasticity Index (PI) of the silty clay till generally ranged from 28% to 42% and 16 to 28, respectively. Gravel and cobbles (sub-rounded to rounded) were encountered within the till unit and the sand content tended to increase with depth ranging from trace to with sand. Bedrock was not encountered at the test pile location and it is believed that the entire pile was located within the above described soil units.

2.2 Hwy 14 Ramps at AHDSERR

The subsurface conditions encountered at the test pile location generally consist of a 4 m thick layer of silty clay fill layer overlying a thin topsoil layer, all underlain by the main silty clay till unit, all over bedrock. Lenses of wet sand were encountered within the silty clay till unit from approximate depths of 3 m to 5 m, and at 13.5 m below ground surface. The silty clay till generally ranged from firm to stiff with depth, with the average undrained shear strength and CPT q_t (tip resistance) values generally ranged from 75 kPa to 125 kPa with depth and 1 MPa to 2.5 MPa with depth, respectively. The LL and PI values were 39% and 24, respectively. Gravel and cobbles were encountered within the till unit and the sand content tended to increase with depth ranging from trace to with sand. A stiff to very stiff bentonitic layer (LL = 81% and PI = 48), approximately 1 m thick, was encountered just above the bedrock at the test pile location at an approximate depth of 19 m. Bedrock was encountered at a corresponding approximate depth of 20 m below ground surface and included extremely weak to weak siltstone/claystone. The test pile was installed within the overlying soil units and the bedrock.

2.3 34th Avenue at AHDSERR

The subsurface conditions encountered at the test pile location generally consist of silty clay till. Lenses of wet sand were encountered within the silty clay till unit at approximate depths of 10.5 m to 11.5 m, and at 18.5 m below ground surface. The silty clay till generally ranged from stiff to very stiff with depth, with SPT 'N' values ranging from 7 to 15 blows per 300 mm penetration, with an average N-value of 11. The average undrained shear strength and CPT q_t (tip resistance) values generally ranged from 75 kPa to 125 kPa with depth and 1 MPa to 2.5 MPa with depth, respectively. The LL and PI values in the silty clay till were 29% and 16, respectively. Gravel and cobbles were encountered within the till unit and the sand content tended to increase with depth ranging from trace to with sand. Bedrock was not encountered at the test pile location and it is believed that the entire test pile was located within the silty clay till unit.

3 TEST PILE SETUP

An O-cell is a high capacity hydraulically activated jacking device that was attached to the rebar cage of each test pile and concreted into the caisson shafts. The O-cells used for the AHDSERR tests were capable of applying a maximum axial load of 10 MN (in both directions). During testing, the O-cell is pressurized to apply pressure upwards against the base of the upper portion of the shaft, and downwards against the top of the lower portion of the test shaft. Because the O-cell is installed within the test shaft, the reaction force required to carry out the test is derived from the ground/pile system (the upper portion provides the reaction force for the lower portion and vice-versa), thereby eliminating the need for large reaction beams and tension piles that are required for conventional pile load tests.

An O-cell load test is typically executed by increasing loads until one of the following occurs:

- The ultimate shaft friction capacity of the upper portion of the test pile is reached (i.e., the upper portion fails);
- The combined ultimate end-bearing and shaft friction capacity of the lower portion of the test pile is reached;
- The maximum O-cell load capacity is reached, or;
- The maximum O-cell expansion (jacking) distance is reached.

The O-cells used for the AHDSERR pile load tests were instrumented with a pressure gauge and a vibrating wire pressure transducer to record both pressure (applied axial load) and expansion. Three telltales consisting of 6.4 mm (1/4 inch) diameter rods installed within 12.5 mm (1/2 inch) diameter steel pipes were used to independently monitor the movement of the top and bottom of each O-cell. In addition to the instruments installed at the load cell, two LVDTs, per test pile, were used to monitor the movement of the top of the test pile with respect to a fixed reference beam, and a total of 10 and 12 vibrating wire strain gauges were installed on the rebar in pairs at five and six levels, respectively, within the test piles at 17th Street/34th Avenue and Hwy 14 sites, respectively, to monitor strains for the interpretation of pile axial loads. A digital survey level was used to monitor movement of the reference beams during each test. A representative example of the locations of the O-cells and the various instruments installed within each test pile are indicated on the schematic test pile sections shown for the Hwy 14 site on Figure 1.

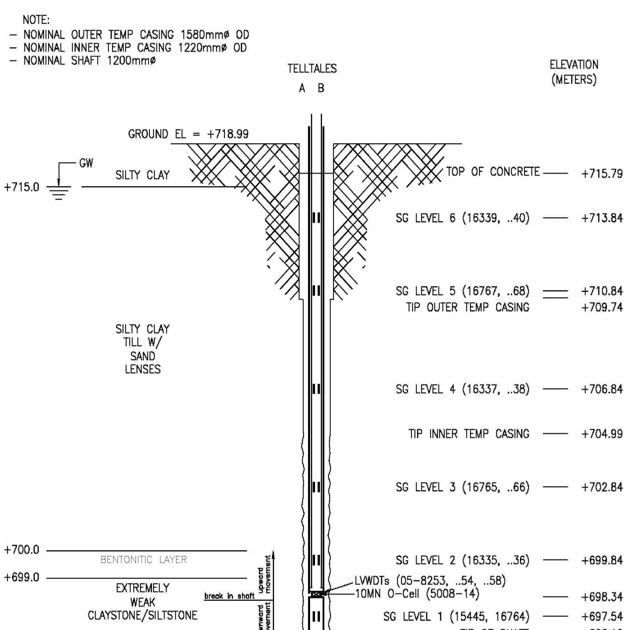

Figure 1. Schematic Section of Test Pile at Hwy 14 Site (modified from Loadtest 2005 – not to scale)

4 TEST PILE INSTALLATION

For each test pile, loose soil was removed from the base of the pile hole using the piling auger. The shaft wall and base condition was confirmed prior to placing steel and concrete by visual inspection and either 'sounding' or video camera inspection. Seepage was noted just below the level of the lower steel casing at one test pile location (17th Street) during the installation of the test pile, but no significant sloughing or collapse of the sidewalls was evident at the two other sites. The top of concrete being placed was kept above the bottom elevation of the steel casing(s) as they were withdrawn from the hole to reduce contamination of the concrete and enhance good contact between the concrete and ground. The pile concrete (high early strength mix) was tremmied during placement for the entire pile length and was tested during and after pile installation and found to meet the project specifications.

The test pile at the 17th Street site was an approximate 1200 mm diameter, straight shaft, cast in place concrete pile. The test pile extended from existing ground surface to a depth of approximately 25.2 m. Temporary steel casings (upper and lower) with nominal outside diameters of 1510 mm (upper) and 1230 mm (lower) were used over the upper 15.5 m and between 15.5 m and 22.5 m (depth), respectively, of the test pile to cut off seepage and sloughing into the test hole.

The test pile at the Hwy 14 site was an approximate 1200 mm diameter, straight shaft, cast in place concrete pile. As can be seen on Figure 1, the test pile extended to a depth of approximately 22.8 m. The top of the test pile was located approximately 3.2 m below top of ground in order to adequately install the lower portion of the pile shaft within the bedrock material. Temporary steel casings (upper and lower) with nominal outside diameters of 1580 mm (upper) and 1220 mm (lower) were used over the upper 6.05 m and between 6.05 m and 10.8 m (depth), respectively, of the test pile to cut off seepage and sloughing into the test hole.

The test pile at the 34th Avenue site was an approximate 1500 mm diameter, straight shaft, cast-in-place concrete pile. The test pile extended from existing ground surface to a depth of approximately 25 m. A temporary steel casing with a nominal outside diameter of 1580 mm was used over the upper 14 m of the test pile to cut off seepage into the test hole.

5 PILE LOAD TESTING AND RESULTS

The pile load tests were carried out generally in accordance with the Quick Load Test for Individual Piles (ASTM D1143 Standard Test Method for Piles under Static Axial Load). Each load increment was maintained for a total of eight minutes and a data logger was used to record instrument readings at 30 second intervals throughout each load test.

The results of the load test at each structure are too numerous to present in their entirety in this paper. Thus, the measured load movement curve separated into top and bottom portions of the test piles (i.e., above and below the O-cell) and the inferred load distributions within the test pile at various load increments for the Hwy 14 site are presented in Figures 2 and 3, respectively, as a representative example.

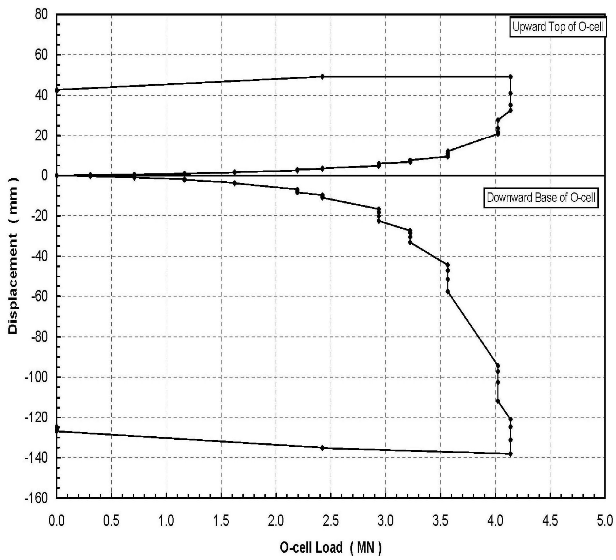

Figure 2. Osterberg Cell Load – Movement at Hwy 14 Site (modified from Loadtest 2005)

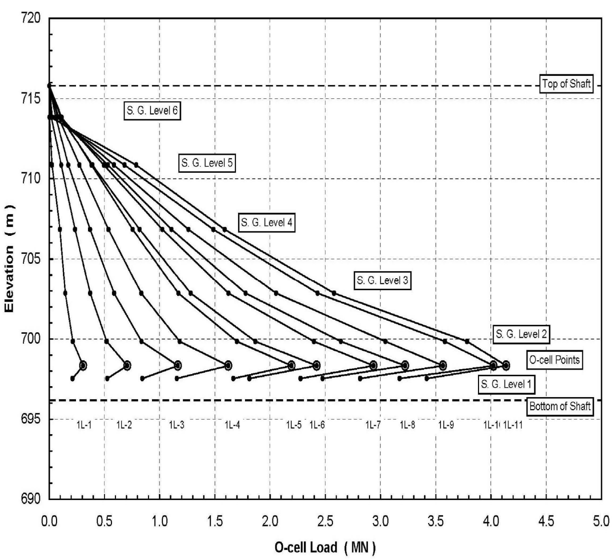

Figure 3. Strain Gauge Load Distribution at Hwy 14 Site (modified from Loadtest 2005)

In order to estimate the axial load carried by shaft friction during each test, the loads carried in each section of the pile were calculated using the strain gauge data collected during the test. As can be seen in Figure 2, the loads are a maximum at the point of application at the O-cell. Moving away from the load cell, each successive set of strain gauges measures less load. The difference in load between two sets of strain gauges is the amount of load resisted through shaft friction over the respective relevant pile length.

The end-bearing resistance of each test pile was estimated using the data from the lower portion of the pile (from the O-cell to the pile tip). Using the shaft friction values back-calculated from strain gauge measurements, the total shaft load carried by friction acting on the lower portion pile was estimated. The total load carried in end-bearing during the test was then calculated by subtracting the estimated shaft friction load from the total load applied to the bottom portion of the pile.

5.1 17th Street at AHDSERR

A total of 17 load increments were applied up to a maximum bi-directional load of 3.4 MN. Loading was halted after the 17th increment as the combined shaft friction and end-bearing capacity of the lower portion of the test pile was reached, and additional loading could not be maintained. The test pile was unloaded in one decrement of load. The measured maximum movements of the upper and lower sections of the pile during the load test were 8.4 mm and 149.5 mm, respectively.

During the test, the loads applied by the O-cell were sufficient to partially mobilize the shaft friction in the upper pile section and fully mobilize the combined shaft friction and end-bearing resistance in the lower pile section. The average mobilized shaft friction values for the silty clay till calculated using the load test data are presented in Table 1.

Table 1 Average Mobilized Shaft Friction

Depth From Existing Ground Surface	Mobilized Average Shaft Friction
0 to 12 m	27 kPa
12 m to 17 m	63 kPa [1.]
17 m to 24 m	77 kPa [1.]

Notes: [1.] Considered to be ultimate value.

These skin friction values correspond to Beta values ranging from approximately 0.35 to 0.4 for the silty clay materials.

The maximum sustained load applied to the bottom portion of the pile during the load test was 3.4 MN at an average displacement of 149.5 mm. Using the shaft friction values back-calculated from strain gauge measurements, a total end-bearing load of approximately 1.0 MN was estimated, which for a 1200 mm diameter pile corresponds to a unit end-bearing stress of 884 kPa. The lower portion of the test pile failed in combined shaft friction and end-bearing before the upper portion failed in shaft friction. Thus, the ultimate end-bearing capacity was considered to be reached during the test.

In addition, the measured test data for the bottom portion of the pile was used to estimate the ultimate failure load of the section of the pile below the O-cell using both the Brinch-Hansen and Chin Failure Criteria, as outlined in the Canadian Foundation Engineering Manual (CFEM 2006). These estimated ultimate (failure) loads, together with the measured resistance due to shaft friction on the relevant pile sections were then used to estimate ultimate end-bearing resistance. These estimates are summarized in Table 2.

Table 2 Estimated Ultimate End-Bearing Resistance

Method Used	Estimated Ultimate End-Bearing
Maximum measured end-bearing resistance	884 kPa
Estimate using load-displacement curve for bottom section	800 kPa
Estimate using Brinch-Hansen Criteria	829 kPa
Estimate using Chin Criteria	1067 kPa
Average	895 kPa

Based on the measured test loads, as well as estimates of the ultimate pile capacity extrapolated using the various methods for interpretation identified above, it is estimated that the ultimate end-bearing resistance of the test pile in the silty clay till was approximately 895 kPa. This end-bearing value corresponds to N_t values of approximately 7 and 3 for the undrained (total stress) and drained (effective stress) cases, respectively.

5.2 Hwy 14 Ramps at AHDSERR

The upper test section was constructed mainly in the silty clay till and the lower test section was constructed in the bedrock (Figure 1). A total of 11 load increments were applied up to a maximum bi-directional load of 4.1 MN (Figure 2 and 3). Loading was halted after the eleventh increment as the shaft friction capacity of the upper portion, and the combined shaft friction and end-bearing capacity of the lower portion of the pile were reached, and additional loading could not be maintained. Thus, both the ultimate shaft friction resistance and end-bearing capacity were reached during the test. The test pile was unloaded in two decrements of load. The measured maximum movement of the upper and lower sections of the pile during the load test were 49.1 mm and 138.1 mm, respectively.

During the test, the loads applied by the O-cell were sufficient to fully mobilize the shaft friction in both the upper and lower pile sections and the end-bearing resistance in the lower pile section. The ultimate shaft friction values calculated using the load test data, together with the weighted average estimated over a particular shaft length are presented in Table 3.

Table 3 Average Mobilized Ultimate Shaft Friction

Depth From Existing Ground Surface	Mobilized Average Shaft Friction
0 to 12 m	44 kPa [1.]
12 m to 16 m	61 kPa
16 m to 19 m	102 kPa
19 m to 20.7 m	58 kPa [2.]
20.7 m to 21.5 m	238 kPa [3.]

Note: [1.] Upper 3 m to 5 m section failed early in test and not considered representative of production pile behaviour.
[2.] Bentonitic zone
[3.] Bedrock

These skin friction values correspond to Beta values ranging from approximately 0.45 to 0.6 for the silty clay materials, and an unconfined compressive strength of the upper weathered bedrock of 1 MPa.

The maximum sustained load applied to the bottom portion of the pile during the load test was 4.1 MN at an average displacement of 138.1 mm. Using the shaft friction values back-calculated from strain gauge measurements, a total end-bearing load of approximately 2.2 MN was estimated, which for a 1200 mm diameter pile corresponds to a unit end-bearing stress of 1945 kPa.

In addition, the measured test data from the bottom portion of the pile were used to estimate the ultimate failure load of the section of the pile below the O cell using both the Brinch-Hansen and Chin Failure Criteria, as outlined in the CFEM (2006). These estimated ultimate (failure) loads, together with the measured resistance due to shaft friction on the relevant pile sections were then used to estimate ultimate end-bearing resistance for the bedrock. These estimates are summarized in Table 4.

Table 4 Estimated Ultimate End-Bearing Resistance (Bedrock)

Method Used:	Estimated Ultimate End-Bearing
Maximum measured end-bearing resistance	1945 kPa
Estimate using load-displacement curve for bottom section	2000 kPa
Estimate using Brinch-Hansen Criteria	1905 kPa
Estimate using Chin Criteria	2260 kPa
Average	2028 kPa

Based on the measured test loads, as well as estimates of the ultimate pile capacity extrapolated using the various methods for interpretation identified above, it is estimated that the ultimate end-bearing resistance of the test pile was approximately 2000 kPa. This end-bearing value also corresponds to an unconfined compressive strength of the upper weathered bedrock of 1 MPa.

5.3 34th Avenue at AHDSERR

A total of 11 load increments were applied up to a maximum bi-directional load of 5.1 MN. Loading was halted after the eleventh increment as the shaft friction capacity of the upper portion of the pile was reached, and additional loading could not be maintained. Thus, the upper portion of the test pile failed in shaft friction before the lower portion failed in combined shaft friction and end-bearing resistance (i.e., ultimate end-bearing capacity was not reached). The test pile was unloaded in four decrements of load. The measured maximum movement of the upper and lower sections of the pile during the load test were 37.9 mm and 10 mm, respectively.

During the testing, the loads applied by the O-cell were sufficient to mobilize the shaft friction in the upper pile section and partially mobilize the shaft friction and end-bearing resistance in the lower pile section. The averaged mobilized shaft friction values are presented in Table 5.

Table 5 Average Mobilized Shaft Friction

Depth From Existing Ground Surface	Mobilized Average Shaft Friction
0 to 13.5 m	35 kPa
13.5 m to 17.5 m	112 kPa [1.]
17.5 m to 24 m	104 kPa [1.]

Note: [1.] Considered to be ultimate value.

These skin friction values correspond to Beta values ranging from approximately 0.35 to 0.5 for the silty clay materials

The maximum sustained load applied to the bottom portion of the pile during the load test was 5.1 MN at an average displacement of 10 mm. Using the shaft friction values back-calculated from strain gauge measurements, a total end-bearing load of approximately 1.56 MN was estimated, which for a 1500 mm diameter pile corresponds to a unit end-bearing stress of 882 kPa.

In addition, the measured test data from the bottom portion of the pile were used to estimate the ultimate failure load of the section of the pile below the O-cell using both the Brinch-Hansen and Chin Failure Criteria, as outlined in the CFEM (2006). These three estimated ultimate (failure) loads, together with the measured resistance due to shaft friction on the relevant pile sections were then used to estimate the mobilized end-bearing resistance for the silty clay till. These estimates are summarized in Table 6.

Table 6 Estimated End-Bearing Resistance

Method Used	Estimated End-Bearing
Maximum measured end-bearing resistance	882 kPa
Estimate using load-displacement curve for bottom section	1340 kPa
Estimate using Brinch-Hansen Criteria	1050 kPa
Estimate using Chin Criteria	1516 kPa
Average	1300 kPa

Based on the measured test loads, as well as estimates of the ultimate pile capacity extrapolated using the various methods of interpretation identified above, it is estimated that the end-bearing resistance of the test pile in the silty clay till was approximately 1300 kPa. This end-bearing value corresponds to N_t values of approximately 8.5 and 4.5 for the undrained (total stress) and drained (effective stress) cases, respectively.

6 Discussion

The test piles were constructed by an experienced piling contractor using appropriate equipment for the ground conditions encountered. The geotechnical parameters discussed above are based upon the results of the full-scale pile load tests discussed in this paper. It was implicitly assumed that production piles would be of a construction quality similar to that of the test piles.

The geotechnical parameters derived from the pile load testing apply to the locations and subsurface conditions that were encountered during the test pile installations. The parameters were also considered applicable at structures adjacent, and in close proximity, to the test pile locations and with subsurface conditions considered representative of those at the test pile locations. The values were found to be relatively consistent with the parameters used in the original design (i.e. ultimate axial capacity predicted was slightly less than that measured). Given this, together with the increased geotechnical resistance factor (0.6 versus 0.4), the load test results allowed the design axial resistance of a single pile to increase by 50 %.

The overall final bridge designs at the locations influenced by the pile load test results were modified after the testing was completed such that a direct cost comparison could not be completed. However, it is estimated that the pile load testing saved the project approximately 15% on the foundation cost for each associated structure. Further, it is felt that had the testing program been undertaken at an earlier stage of the project (schedule permitting) there would have been potential for additional project cost savings.

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