Osterberg load cell testing results and analysis for rock socket design validation – bridge over Beauharnois Canal, Autoroute 30, Montréal

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ABSTRACT
A major component of the Autoroute 30 project near Montréal is a 2km long bridge over the Beauharnois Canal. Foundation support includes groups of 1.85m diameter drilled shafts socketed into very strong and abrasive sandstone. An initial load testing program was implemented to validate the design and promote a cost-effective foundation solution. This consisted of two sacrificial, heavily-instrumented 1.18m diameter shafts tested using Osterberg load cells. Telltales were also installed in the test piles and in the adjacent rock mass to monitor possible movement. The results and subsequent analysis of these tests, which are presented in this paper, culminated in a revision to the initial design and provided a greater understanding of the cone uplift mechanism in rock. Insights gained are discussed and economies of the load testing program presented.

RÉSUMÉ
Un élément majeur de l'autoroute 30 près de Montréal est un pont de 2 km de long sur le canal de Beauharnois. Les supports de fondation comprennent des groupes de pieux forés de 1,85 m de diamètre ancrés dans du grès très solide et abrasif. Un programme d'essais de charge initiale a été mis en œuvre pour valider la conception et pour promouvoir une solution de fondation économique. Ce programme consistait de deux pieux forés sacrificiels fortement instrumentés de 1,18m de diamètre, testés en utilisant la cellule d'Osterberg. Des inclusions diagnostiques ont été installées dans le massif rocheux adjacent pour surveiller les mouvements possibles. Les résultats et l'analyse ultérieure de ces essais, qui sont présentés dans le présent document, ont abouti à une révision de la conception initiale et ont permis une meilleure compréhension du mécanisme de soulèvement du cône dans la roche. Les enseignements acquis et les économies du programme d'essais de charge initiale sont présentés dans ce document.

1 INTRODUCTION

Construction work for the extension of Autoroute 30 southwest of the City of Montréal is well underway. This work includes a major bridge crossing over the Beauharnois Canal, a manmade waterway serving to provide both hydroelectric power generation and a navigation channel for the Saint Lawrence Seaway. The bridge structure, extending over land and water for a total length of 1.8 km, will be supported on 44 piers including abutment piers. Of these, the eastern abutment pier and 18 land and water piers will be founded on bedrock using groups of drilled shafts. These include an 8-shaft group supporting the eastern abutment, 6-shaft groups supporting 13 water piers crossing the hydroelectric portion of the canal, 14-shaft groups supporting piers at each end of the navigation span over the Saint Lawrence Seaway shipping channel, and 6-shaft groups supporting three of the western land approach piers. In total, construction of 138 individual drilled shafts socketed into bedrock are required. Integral with design and construction of the drilled shafts was an initial load testing program on two instrumented sacrificial drilled shafts to verify design assumptions and assess performance.

This paper serves to describe the design approach and load testing undertaken, and the subsequent design and construction adjustments implemented. It highlights some key considerations to be aware of when utilizing the Osterberg cell (O-cell) for load testing, as related to rock socket design and construction.

2 GEOLOGIC AND SUBSURFACE CONDITIONS

Bedrock geology at the bridge site is regionally described as Cambrian and Ordovician strata, overlain by superficial (drift) deposits that reflect the glacial history of the region. Bedrock encountered during the site-wide ground investigation typically consisted of strong to very strong, light to dark grey, fine to medium grained, quartzitic sandstone with siliceous cement. The sandstone was generally horizontally bedded, with moderately close, occasionally wide joint spacing, with occasional thin sand and gravel lenses. Fine grained dolomotic dark grey sandstone with black shale laminations and dolomitic cement was also encountered, locally alternating with the quartzitic sandstone. Material overlying bedrock encountered during the ground investigation varied with location but generally comprised of fill and Champlain clay deposits with underlying glacial till present at some locations.

Unconfined compressive strength data from laboratory testing performed on intact rock samples obtained during the ground investigation are summarized in Table 1. The Rock Quality Designation (RQD) data recorded during ground investigation indicated over 80% of RQD values in

Table 1. Unconfined Compressive Strength (q_u) of Rock

Data Set	Range (MPa)	Median (MPa)	Median – Standard Deviation (MPa)	Design Value (MPa)
All Data (51 tests)	116-390	200	130	130

excess of 60%, with a general increase in RQD values apparent with depth. Total rock core recovery (TCR) varied between 30% and 100%. The lower RQD and TCR values recorded were generally associated with more weathered surficial rock.

3 FOUNDATION DESIGN

3.1 Basis of Design

Design of the drilled shafts was undertaken in accordance with the Canadian Highway Bridge Design Code (CSA, 2006). This code adopts a limit state design approach in which the factored resistance (either structural or geotechnical) must equal or exceed each and every factored load combination. At the ultimate limit state, the factored geotechnical resistance is obtained by multiplying the ultimate geotechnical resistance, calculated using unfactored soil/rock parameters, by a resistance factor. The resistance factors specified in the code are presented in Table 2, being a function of both loading mode (compression or tension) and how the ultimate geotechnical resistance is determined (i.e. from a static analysis estimate or measured directly by a static load test).

When applied to drilled shaft design, the resistance factors in Table 2 entertain the possibility of reducing shaft lengths by approximately 30% if load testing is performed (presuming the load test is successful in verifying the geotechnical parameters used in the design), which translates to cost savings. There is also the possibility that further cost savings can be achieved if the results of the load tests justify the use of higher ultimate values of the geotechnical parameters. Load tests also provide an opportunity to select design parameters commensurate with the chosen construction method and provide greater confidence in the overall performance of the bridge foundations.

Given such opportunities and the scale of the drilled shaft work, higher resistance factors were applied in the design of the drilled shafts on the basis of a load test program being undertaken prior to production work.

3.2 Design Parameters

In establishing design parameters, various load demands on the drilled shafts are considered. Of particular significance are the lateral effects of seismicity and ice loading on the bridge piers which require sufficient embedment of drilled shafts into rock to provide adequate fixity for restraint against lateral loading. Associated overturning tendencies on the bridge piers also required consideration of uplift (tension) resistance as a result of the "push-pull" action mobilized in the shaft groups.

Table 2. Resistance Factors for Deep Foundation Design (CSA, 2006)

Loading Mode	Resistance Factor	
	Static Analysis Estimate	Static Load Test Measurement
Compression	0.4	0.6
Tension	0.3	0.4

Table 3. Design Parameters and Ultimate Values Prior to Load Testing Program

Parameter	Ultimate Value (MPa)
Side Shear Resistance	2.0
End Bearing Resistance	12.1
Rock Mass Tensile Strength	0.53

Compressive resistance featured throughout, but was especially prevalent at the navigation span support piers where compression loading is the highest as a result of the longer spans. Design parameters of relevance in this paper were as follows:

1. Compression Resistance:
   a. Shaft resistance: Ultimate side resistance mobilized at the rock-shaft interface.
   b. Toe resistance: Ultimate end bearing resistance mobilized at the base of the shaft.
2. Tension Resistance:
   a. Shaft resistance: Ultimate side resistance mobilized at the rock-shaft interface.
   b. Rock cone resistance: Ultimate tensile strength of the rock mass at the boundary of a cone of rock mobilized by drilled shaft.

The design parameters were assessed considering several sources (CGS, 2006; USDOT, 1999; AASHTO, 2006; NCHRP, 2006) and are summarized in Table 3, as well as comparison to New York and Hong Kong design guides for drilled shafts in similarly strong bedrock. In the design calculations, both rock and concrete failure conditions were considered. The computer program RocLab (Rocscience, 2007) was used to estimate the ultimate tensile strength of rock mass according to the work of Hoek and Brown (1997), based on m_i = 17 and geological strength indices (GSI's) of 65 and 70.

A 2.0m diameter drilled shaft section in soil reducing to 1.85m diameter shaft sections for the rock socket length was adopted for design. A minimum socket length of 4m into competent rock was specified to provide adequate lateral restraint, corresponding to a minimum length to diameter (L/D) ratio of approximately 2.

4 TEST SHAFT PROGRAM

4.1 Locations

Test shaft locations were established based on review of the site-wide ground investigation undertaken in late 2008 for the final design of the bridge structure. This review identified locations on the north and south banks of the canal, where each location was considered representative of the different rock types encountered, namely the quartzitic sandstone and dolomitic sandstone geologic units. These locations were considered to best address possible differences in rock and hence drilled shaft behaviour. Each test shaft was located adjacent to bridge pier locations, namely Pier 30 on the north bank location, situated between the Saint Lawrence Seaway and the Beauharnois hydroelectric canal, and Pier 44 (eastern abutment pier) at the south bank location. The two test shaft locations are identified by pier location herein.

4.2 Dimensions and Instrumentation

The diameter of each test shaft was 1.18m, with an embedment in rock of approximately 2.4m, corresponding to an L/D ratio of approximately 2. This maintained geometric similitude with production drilled shafts to promote consistency in behaviour and also aimed to provide a 2m deep socket into competent bedrock.

A single 870mm diameter 27MN O-cell was placed approximately 0.15m above the base of each test shaft, resulting in a maximum possible unit side resistance of 3.5MPa. Each O-cell was instrumented with three linear vibrating wire displacement transducers (LVWDTs) to measure the expansion between the upper and lower plates of the assembly. Two telltale casings were attached to each rebar cage to measure the movement of the upper plate of the load cell. Four levels of vibrating wire strain gauges were embedded within the shaft, both within the rock socket as well as through the upper overburden soils, to evaluate the load transfer along the shaft. In an effort to identify the possibility of an inverted cone (wedge) breakout mechanism in the rock mass during the load tests, two telltale casings were installed in boreholes drilled 1m from the edge of the test shafts, extending from the ground surface and terminating 300mm below the top of rock. Figure 1 shows the elevation view of a test shaft.

4.3 Rock Conditions

During the ground investigation undertaken in late 2008 for final design, a single borehole was performed at each pier location. The 2008 boring available at Pier 30 (prior to load testing) was situated approximately 20m away from the actual load testing location. This reported an RQD of 53% right above the O-cell level, and 100% below O-cell level. At Pier 44, the 2008 boring was situated approximately 40m from the load testing location, with a reported RQD of 53% along the shaft and 99% at O-cell level. Immediately prior to implementation of the field load testing program in the fall of 2009, borings were drilled directly at each test shaft location. At Pier 30, the 2009 boring showed an RQD of 83% in the first meter of core, but only 44% directly above and below the O-cell level. At Pier 44, the 2009 borings showed an RQD of 64% at the O-cell level.

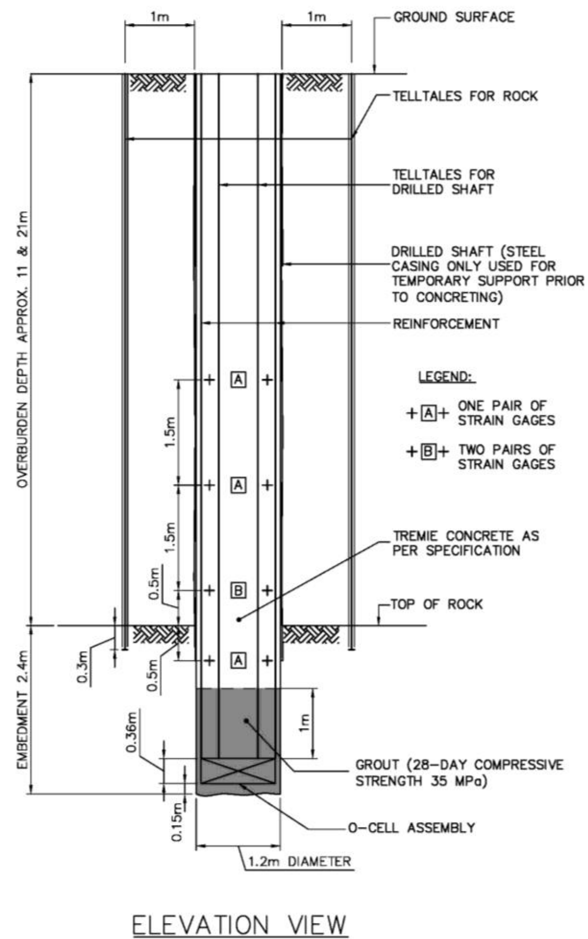

Figure 1. Elevation View of Test Shaft Configuration

4.4 Construction Method

Construction of the test shafts followed the same procedures as used for construction of the land-based production shafts, with the exception of the use of grout to form the lower portion of the test shaft section. Shaft drilling consisted of installation of a starter casing then use of augers, drilling buckets or core barrels to drill down to top of rock with support fluid. Following installation of a temporary casing to top of rock (if required) and replacement of support fluid with water, construction of the rock socket proceeded using a rotary drill and steel core barrel drilling technique. Cleaning of the toe was undertaken with cleaning bucket and airlifting equipment.

The use of grout to form the lower portion of the test shaft section in lieu of concrete was necessary given insufficient space for tremie pipe installation and need to avoid disturbing the O-cell.

4.5 Loading Procedure

The loading procedures were performed in general accordance with ASTM D1143. Each test shaft was to be loaded in three cycles corresponding to 25%, 60%, and 100% of the maximum anticipated O-cell load. At the maximum load of each cycle, the load was to be maintained for 6 hours to measure creep effects. Each load increment within each load cycle was to be maintained at the specified value until a rate of movement not exceeding 0.05mm in 15 minutes was satisfied. Prior to the beginning the next load cycle, the test drilled shaft was allowed to recover until the rebound did not exceed 0.1mm/hour in a period of not less than 30 minutes.

5 LOAD TEST RESULTS

The O-cell testing at the Beauharnois Bridge site was carried out by LoadTest of Gainesville, Florida USA in the fall of 2009. While the two test shaft sockets were of essentially equivalent diameter and length, the behaviour of each during the load testing program was quite different. The test shaft at Pier 44 was loaded as originally scheduled, but with an additional (fourth) quick load cycle mobilizing a maximum base load of approximately 25MN. In comparison, significant rock mass uplift movements and an inability to apply sustained load prevented completion of scheduled loading at Pier 30, limiting the maximum test load at that location to approximately 20MN mobilized in the fourth load increment of the third load cycle. Further details are provided in the following subsections, and a summary of key design parameters resulting from the load testing program at Piers 30 and 44 is provided in Table 4.

5.1 Side Shear

Plots of measured unit side resistance at the shaft-rock interface versus pile net upward movement (relative to the surrounding rock mass) are provided in Figure 2 for the two test shafts. The maximum measured unit side resistance at the Pier 44 test shaft was 2.60 MPa at a net upward movement of 7 mm, while at pier 30 the maximum value reached was 1.53 MPa, which occurred at essentially the same net upward shaft movement. Hence, both the shaft-rock interface strength and shear stiffness measured at Pier 30 were approximately 60 percent of the values measured at the Pier 44 test shaft. Since the concrete/grout used in each test shaft was of equivalent mix and strength, the only logical explanation is to attribute the difference in behaviour to different rock mass conditions, as suggested by the RQD results discussed previously. The telltales installed to monitor rock mass movement showed upward movement for Pier 30 which supports this notion.

5.2 End Bearing

At Pier 44, the O-cell reached the maximum load of 25 MN under 8 mm of net downward base movement. In comparison, when the O-cell at Pier 30 achieved the load of 16 MN at the end of the second load cycle, a higher net downward base displacement of 16.5mm was measured.

Plots of shaft base load versus pile base net downward movement for each shaft are shown in Figure 3. For both shafts the end bearing resistance increased approximately linearly with displacement, and neither shaft was loaded to base failure. However, the base of the test shaft at Pier 30 did experience up to 3mm of creep movement at each of the 11, 13.5, and 16 MN load increments during the second load cycle, with corresponding hold times of 230, 300, and 370 minutes respectively. Creep movements at the base of the Pier 44 test shaft were negligible in comparison. Inspection of the base load versus displacement curves presented in Figure 3 for both shafts suggests that the base stiffness of the rock mass directly under the Pier 30 test shaft is only 40 percent of the base stiffness exhibited under the Pier 44 test shaft.

5.3 Rock Mass Tensile Strength

Changes in the rock surface elevation were monitored with telltales during each test. These measurements were not only used to calculate net upward movement in the shaft (relative to the rock mass into which it was embedded), but were also able to identify the potential for rock wedge (cone) uplift movement as a limiting design consideration. For the Pier 44 test shaft, the upward rock mass movement at the maximum test load of 25 MN was measured at just over 10mm. In comparison, the upward rock mass movement at the Pier 30 test shaft under 16 MN of load was 35 mm, increasing to 58 mm at the rapidly-applied load of 20 MN. It is important to note that when the O-cell in each test shaft was loaded to approximately 16 MN, the difference in upward rock mass movement between the two shafts was nearly thirty-fold (35mm at Pier 30 and only 1.3mm at Pier 44). On the basis of these measurements, it was concluded that the maximum applied load at Pier 30 was limited by the tensile capacity of the surrounding rock mass in uplift.

It was concluded that poorer quality of rock both above and below the O-cell level attributed to the poorer performance of the Pier 30 test shaft. Indeed, subsequent construction inspection of production shafts at this location confirmed the existence of an anomaly up to 400mm in thickness within the competent rock at the approximate O-cell elevation.

6 IMPLICATIONS OF LOAD TEST RESULTS ON FINAL SHAFT DESIGN

6.1 Design Parameters

The results of the load test at Pier 30 required a re-examination of the original (pre-load test) design of the rock socketed shafts in terms of shaft side shear resistance, end bearing resistance, and rock mass tensile strength.

6.1.1 Side Shear

Prior to load testing, the ultimate side shear resistance adopted in the design was 2.00 MPa. While the load test at Pier 44 proved a value of 2.60 MPa, the measurement at Pier 30 of 1.53 MPa was lower, and subsequently was adopted in final shaft design. Applying the resistance factors in Table 2 resulted in factored side shear resistance values of 0.92 MPa in compression and 0.61 MPa in tension.

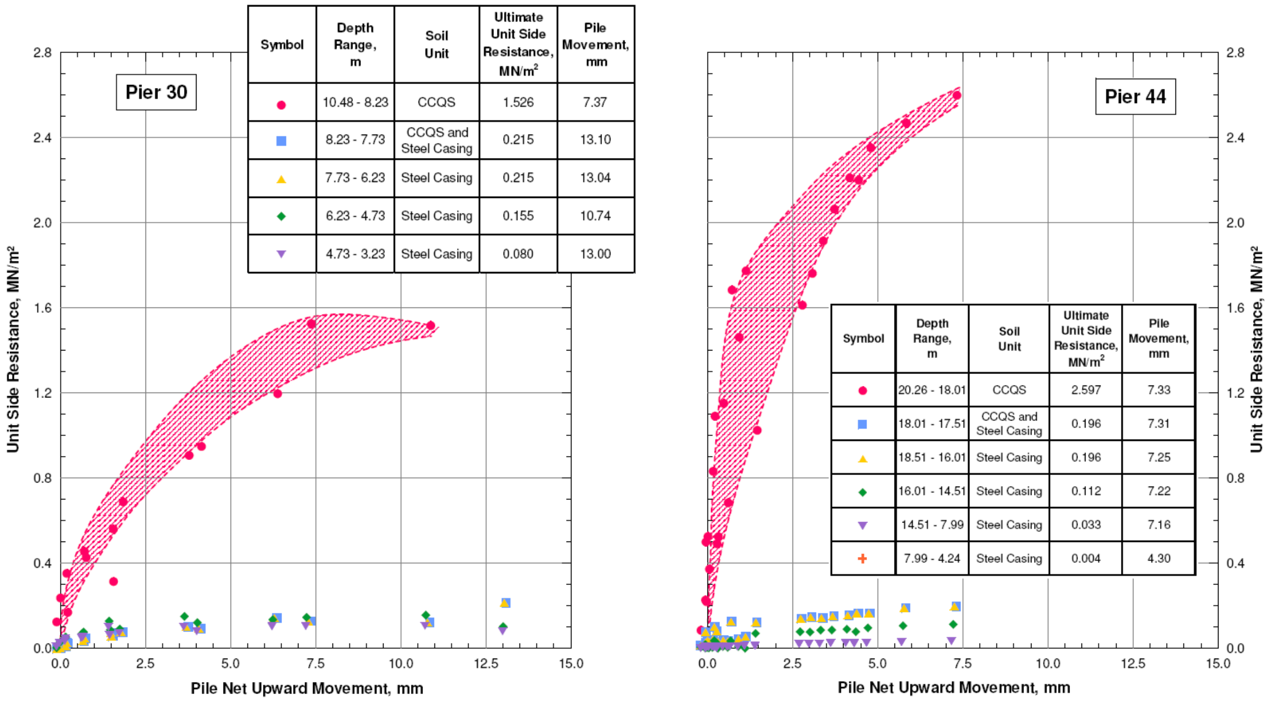

Figure 2. Measured Unit Side Resistance Versus Pile Net Upward Movement, Pier 30 & 44 Test Shafts

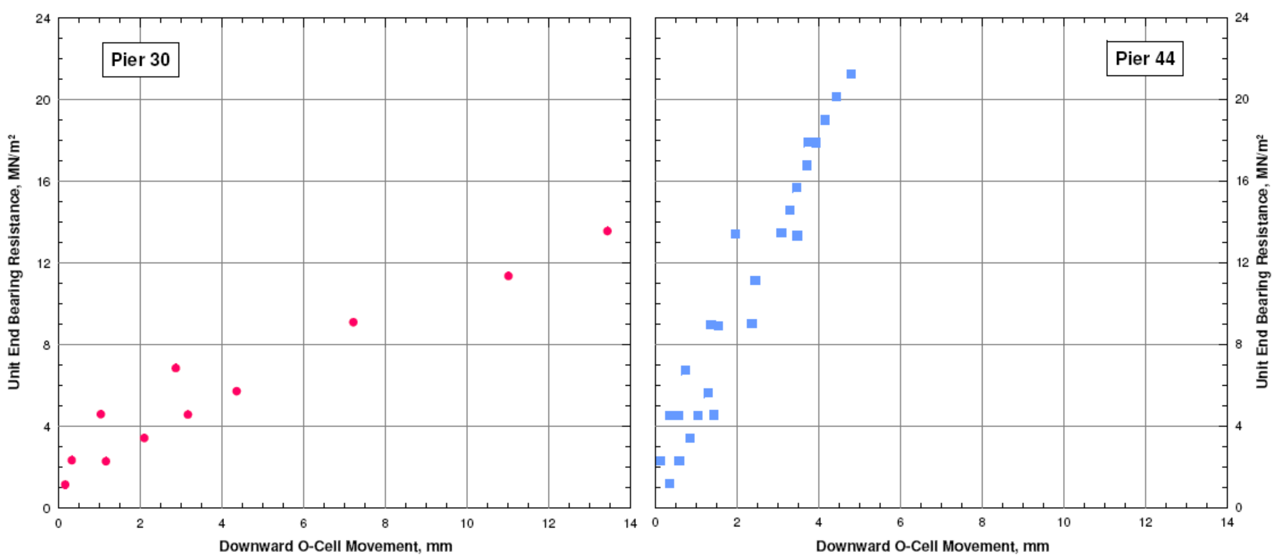

Figure 3. Measured Unit End Bearing Resistance Versus Downward O-Cell Movement, Pier 30 & 44 Test Shafts

Table 4. Summary of Design Parameters – Original Design, O-Cell Test Results, and Revised Design Values

Design or Testing Stage	Side Shear Resistance			End Bearing Resistance		Rock Mass Tensile Strength	
	Ultimate (MPa)	Factored Ultimate Compression (MPa)	Factored Ultimate Tension (MPa)	Ultimate (MPa)	Factored Ultimate (MPa)	Ultimate (MPa)	Factored Ultimate (MPa)
Original Design	2.00	1.20	0.80	12.10	7.26	0.53	0.21
Pier 30 Test	1.53	0.92	0.61	8.15	4.90	0.30	0.12
Pier 44 Test	2.60	1.56	1.04	> 13.4[a]	> 8.04[a]	> 0.45[a]	> 0.18[a]
Revised Design (All piers except 28 & 29)	1.53	0.92	0.61	8.15	4.90	0.30	0.12
Revised Design Piers 28 & 29				10.2[b]	6.10[b]		

(a) Values reported are the maximum values achieved (without apparent failure) during the Pier 44 O-cell test
(b) 25% increase in end bearing resistance permitted at Piers 28 & 29 due to higher RQD values at the base of these longer shafts

6.1.2 End Bearing

Prior to load testing, an ultimate end bearing resistance of 12.1MPa was adopted in the design. This value implicitly considered displacement compatibility with ultimate side resistance.

In general, the ultimate side shear resistance of a pile is mobilized prior to its ultimate end bearing resistance. While an O-cell test loads a shaft from the base in an upward fashion, a production shaft under compression is loaded from the top downward. As such, displacement compatibility under axial compression must be considered in socket design. The O-cell test results permit direct consideration of displacement-compatible axial load transfer between the rock socket base and shaft.

It has already been stated that at Pier 30, the maximum unit side shear resistance of 1.53 MPa was mobilized at 7mm net upward shaft movement. At this same net downward displacement, the corresponding base load was 9.2 MN (unit end bearing resistance of 8.15 MPa). Considering a resistance factor of 0.6, the typical factored ultimate end bearing resistance after load testing was 4.90 MPa.

At locations where the results of the O-cell testing required a lengthening of the rock sockets (namely, at main span support piers 28 and 29, as described in Section 6.2 of this paper), these values of end bearing resistance were increased by 25% to take into account the improvements in RQD with depth, consistent with the higher end bearing resistance proven during the Pier 44 O-cell test.

To provide assurance that the loads would be transferred through the base, a regime of concrete-rock interface verification coring and subsequent base grouting (where required) was implemented.

6.1.3 Rock Mass Tensile Strength

Prior to load testing, an ultimate rock mass tensile strength of 0.53MPa was adopted in the design. However, the load test at Pier 30 was terminated prematurely because of rock wedge (cone) uplift mechanism. The rock mass tensile strength was re-evaluated on the basis of this knowledge. Based on an assumed rock cone angle of 45 degrees, the ultimate rock mass tensile strength was recalculated to be 0.30 MPa, which was the value adopted in the final design of the production shafts. As the rock wedge (cone) uplift mechanism corresponds to tension loading, a resistance factor of 0.4 applies, resulting in a factored ultimate rock mass tensile strength value of 0.12 MPa.

6.2 Shaft Lengths

The initial design of the shafts prior to testing always assumed that the testing would be completed and therefore took advantage of the higher load factors. The typical minimum shaft length of 4m in competent rock was governed by lateral load considerations (Length/Diameter of 2). The compression demand for these shafts was significantly higher than its tension demand. Even with the lower adopted design parameters which resulted from the load tests (Table 4), the typical maximum shaft length required solely for tension design remained at 4m in competent rock or less. In general, there was no change in length to the 4m long shafts.

The shafts at Piers 28 and 29, which support the main span over the shipping channel, were each 5m length in competent rock prior to load testing, governed by tension loading (rock wedge/cone uplift mechanism). The lower results from the O-cell load tests required that the shafts at these two piers be lengthened by 2.1m (to 7.1m) in competent rock to satisfy both tension and compression demands.

A shaft design without the benefit of load testing or base interface coring and base grouting during construction likely would have resulted in a design which neglected the base resistance. This fact, along with the required use of lower resistance factors without load testing, would have resulted in typical rock socket lengths on the order of 5m to 8m (rather than the 4m length employed after execution of the O-cell test program.) At main span support piers 28 and 29, the rock sockets would have likely been nearly 12m in length, significantly less than the 7.1m length adopted after O-cell testing.

Clearly, the execution of the O-cell testing program, combined with base interface coring (and remedial grouting when necessary) during construction, contributed to a significant reduction in the cumulative length of rock socket required for the bridge foundations.

6.3 Comments Regarding Use of Scaled Test Shaft Geometry

The 1.18m diameter test shafts were smaller than the production rock socket diameter of 1.85m, and the length was also shortened to maintain a consistent L/D ratio. A smaller diameter shaft was selected to reduce the cost of test shaft drilling, rebar cage fabrication, concrete volume, and O-cell equipment. The corresponding shortening of the test socket length can have the effect of testing only the upper perhaps more fractured or weathered rock rather than the often more competent rock below. This may have been a contributory factor in the results measured in Pier 30 where poorer rock was encountered in the boring at that location.

Scaling of shaft geometry to maintain a similar L/D ratio is commonplace as it allows cost effective load tests and, as in this case, keeps test loads within achievable bounds, but these benefits should be considered alongside the issues of over-emphasis of near-surface rock mass features of weathering, joints, or other anomalies.

7 ECONOMIES ACHIEVED THROUGH LOAD TESTING PROGRAM

The load testing program indicated lower ultimate values of side shear, end bearing, and rock mass tensile strength values than originally estimated but allowed the use of higher resistance factors. The combined net effect resulted in the following impacts on the design parameters (relative to a design with no load test):

- 15% increase in factored ultimate side shear resistance in compression;
- negligible increase in factored ultimate side shear resistance in tension;
- 26% increase in factored ultimate end bearing resistance at main span piers 28 & 29 (negligible increase at all other piers)
- proportion of axial load taken at the base under ULS conditions increased to a maximum value of 38% on the basis of displacement compatibility analysis of O-cell test results. Without load testing, this value would have typically been limited to approximately 20%, provided that interface coring at each base was performed to verify adequacy of base contact, with remedial base grouting being performed where deemed necessary. Without load testing and without such base interface verification, the proportion of axial load taken at the base under ULS conditions would have been limited to no more than 10%, and may have been neglected altogether.

Relative to the no load test condition, execution of the Beauharnois Bridge load testing program, combined with base interface coring and remedial base grouting (when necessary) during construction, resulted in a reduction in the drilling length required in competent rock. This savings is considerable given the high strength of the rock and the construction rate of progress that was achieved.

8 SUMMARY AND CONCLUSIONS

This paper has summarized the design of the rock-socketed drilled shaft foundations for the Beauharnois Bridge – part of the ongoing project to complete the Autoroute 30 ring road to the south and west of Montréal – considering the results of two sacrificial test shafts loaded by Osterberg load cells.

The behaviour of the two sacrificial shafts was quite different, with the test shaft at Pier 44 achieving the maximum anticipated test load while the load at the Pier 30 test shaft being limited by an apparent rock wedge/cone uplift mechanism as a result of a higher degree of fracturing.

O-cell testing is a cost effective alternative to conventional testing, particularly for testing large diameter drilled shafts in rock. However, it should be recognized that such tests on test shafts with rock sockets shorter than the production shafts will more likely be governed by fractures or other anomalies in the rock mass at shallow depth, promoting the rock wedge/cone uplift mechanism. In these cases, consideration should be given to conventional compressive tests, or testing shafts with longer rock sockets, although, as in this case, such alternatives can be prohibitively expensive where high rock strength and high shaft friction values require high test loads.

The results of the O-cell testing program were less favourable than originally expected. Nevertheless, execution of the testing program, along with base interface coring and remedial grouting, if necessary, resulted in a reduction of the cumulative length of rock socket required for the Beauharnois Bridge relative to the no load test condition. Combined with subsequent engineering analysis and implementation of quality control measures during production rock socket construction, it yielded a cost-effective foundation solution for the bridge.

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