December 4, 2017

FINAL STATIC PILE LOAD TEST REPORT

Highway 401 / Fletcher's Creek Bridges Highway 401 Widening from Highway 403/410 Interchange to the Credit River City of Mississauga, Region of Peel GWP 2150-01-00, Contract No. 2015-2018

Submitted to:
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GEOCRES No. 30M12-499

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FINAL REPORT

Golder Associates

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder), under subcontract to AECOM, was retained by the Ministry of Transportation, Ontario (MTO) to oversee and monitor a full-scale static pile load test during construction of the Highway 401-Fletcher's Creek culvert replacement as part of the widening of Highway 401 from the Highway 403/410 Interchange to the Credit River in the City of Mississauga, Regional Municipality of Peel, Ontario (Contract #2015-2018). The following sections of this report provide a summary of the details associated with the test pile location (see Figure 1 in Appendix A), installation, testing procedures and results, and a discussion on our interpretation of the results.

The intended purpose of the full-scale pile load test is to compare the actual geotechnical resistance achieved to the design geotechnical resistance value estimated by geotechnical analyses, the Hiley formula and dynamic (PDA) test results obtained during pile driving, and thus verify and optimize correlations between theory, measured and calculated dynamic test results after initial driving, and actual measured longer-term geotechnical resistance of the pile in compression. Depending on the results of the full-scale pile load test, consideration would then be given to using higher geotechnical resistance factors and consequently higher design geotechnical pile resistances at the site to optimize the next stage of pile installation to support the portions of the new bridge below the Highway 401 express lanes.

In addition, the results of the full-scale pile load test could be used to refine / confirm design assumptions and allow for an assessment of the development of geotechnical resistances with time for similar sandy / gravelly soils under flowing artesian pressures, for applicability to other highway bridges in similar conditions.

2.0 BACKGROUND

The Highway 401-Fletcher's Creek site is located approximately 1 km west of the intersection of Highway 401 and Mavis Road, and approximately 1 km east of the Credit River, in the City of Mississauga. Highway 401 in this area is currently a three-lane freeway in both eastbound and westbound directions. The current double-cell concrete box structure is being replaced with two single-span bridge structures (designated North Bridge Structure and South Bridge Structure as shown on Contract Drawing Sheet 510-2 in Appendix B) to accommodate Highway 401 and the proposed widening over Fletcher's Creek.

The detail design of the new bridges was carried out by AECOM between 2012 and 2015 and a driven steel H-pile (HP310x110) integral abutment design was selected for support of the new east and west abutments for the replacement bridges. Foundation recommendations for design of the new bridges are provided in the Foundation Investigation and Design Report (FIDR) for Fletcher's Creek Bridges, dated March 2013 prepared by Golder Associates Ltd. (GEOCRES No. 30M12-356). It is noted that flowing artesian conditions (i.e. high hydrostatic heads) were measured within the silty sand to sand and gravel till aquifer below the clayey cap at the site and the piles were designed to penetrate into the artesian aquifer. A sand filter blanket located below the pile cap is incorporated into the design to filter fine soil particles that could be carried along the pile-soil interface as a result of the unbalanced hydrostatic conditions.

The construction contract was awarded to Dufferin Construction Company (Contractor) and the foundation piling works were subcontracted to Anchor Shoring & Caissons Limited (Anchor Shoring). The Contract Administrator (CA) contract was awarded to AECOM. A copy of the General Arrangement and Foundation drawings for the

Contract for the North and South Bridge are included in Appendix B for reference. The initial stages of construction included widening Highway 401 to the south and north while maintaining traffic on the existing Highway 401 WB and EB core lanes. The pile foundations for the abutments at the northern portion of the North Bridge and the entire South Bridge were completed between Fall 2016 and early Winter 2017. It is understood that traffic will be diverted to the new widened portion of the bridges in Spring 2018 and the remaining abutment piles for the North Bridge will be completed in 2018.

During piling activities for the northern portion of the North Bridge and for the entire South Bridge foundations, the majority of driven HP310 x 110 piles achieved the required design geotechnical resistance upon driving to the design tip elevation, or on subsequent retapping, as confirmed using the Hiley formula test method. The retapping of piles shortly after initial installation measured both higher and lower ultimate resistance values than the ultimate resistance values measured at end of initial driving to the approximate pile tip design elevation, which raised some uncertainty to anticipated strength gain or the possibility of relaxation over time at this site.

It was decided to perform a full-scale static Pile Load Test on a test pile to provide data for comparison of geotechnical resistance with the results from PDA testing and Hiley testing to confirm strength gain over time, and to optimize correlations between the Hiley method, PDA test results and actual longer-term pile capacities under compression loads.

Based on the history of static pile load testing in similar soil conditions, it was anticipated that the full-scale pile load test (if conducted a reasonable time after pile installation) would measure a higher pile geotechnical resistance compared to the resistance calculated from dynamic testing performed during driving and upon final driving of the test pile to the design tip elevation. This phenomenon of anticipated set-up or strength gain over time is attributed to an increase in porewater pressure within the soil matrix surrounding the pile upon driving followed by dissipation of porewater pressures over time leading to a gain in effective stress and associated increase in geotechnical resistance. It is noted that the site of bridge construction and test pile assessment is known to historically have natural “springs” or areas of groundwater seepage at the ground surface, and “quick sand” conditions as posted on signs visible from Highway 401 warning the public of the unstable conditions. The flowing artesian conditions present within the sandy / gravelly till aquifer underlying a lower permeability clayey silt deposit created a unique condition for the pile-supported structures, and thus confirmation of design loads and the anticipated strength gain over time of the foundation elements was considered prudent.

In collaboration with MTO and the CA, Anchor Shoring installed a test pile near the west abutment of the new South Bridge (see Figure 1) on December 14, 2016. The pile was left in place over the winter and spring, and a Pile Load Test (in general accordance with ASTM D1143) was performed on May 16 and 17, 2017, approximately five months after initial pile installation. The following sections of this report summarize the details of the test pile location and installation, pile load test procedures and results. The last section of the report provides a discussion and our interpretation of the pile load test results.

3.0 SUBSURFACE CONDITIONS AT TEST PILE SITE

3.1 Regional Geology

The Fletcher's Creek crossing below Highway 401 is located within the Peel Plain physiographic region, near the transition to the South Slope physiographic region, as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984)[1].

The Peel Plain physiographic region covers the central portions of the Regional Municipalities of York, Peel and Halton. The general topography of this region consists of level to gently rolling terrain, sloping gradually southward toward Lake Ontario. A surficial till sheet, which generally follows the surface topography, is present throughout much of this area. The till, which is mapped in this area as the Halton Till, typically consists of clayey silt to silty clay, with occasional sand to silt zones. Shallow, localized deposits of loose sand and silt and/or soft clay can overlie this uppermost till sheet, and these represent relatively recent deposits, formed in small glacial melt water ponds scattered throughout the Peel Plain and concentrated near river valleys. The recent sand, silt and clay and uppermost till deposits in this area overlie and are interbedded with stratified deposits of sand, silt and clay.

3.2 Subsurface Conditions

As part of the foundation investigation for design of the new bridge structures, a total of fourteen boreholes (Boreholes FC-1 to FC-13 and FC-13A) were advanced in the vicinity of the Fletcher's Creek bridge structures site. The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of in situ and laboratory testing are provided in the Foundation Investigation Report (GEOCRES No. 30M12-356). A copy of the Foundation Drawings provided for the Contract (Sheet Nos. 511 to 513) and relevant borehole records are included in Appendix B for reference.

The approximate location of the test pile (i.e. Pile Load Test) has been added to Sheet No. 511 and 513 for reference, as shown in Appendix B. Based on the nearest boreholes (FC-10 and FC-12) to the test pile location, the subsurface conditions generally consist of a surficial layer of clayey silt fill to a depth of between about 0.7 m and 1.5 m below ground surface (Elevation 163.4 m and 162.4 m), underlain by a cohesive deposit consisting of firm to hard clayey silt with sand till to a depth of about 5.6 m (Elevation 158.3 m). Underlying the cohesive till layer, a non-cohesive till deposit of dense to very dense sand and silt to sand and gravel till was encountered. Based on Borehole FC-12 and the surrounding deeper boreholes (FC-6 and FC-11), the non-cohesive till deposit typically transitions from dense sand and silt to very dense sand and gravel between Elevation 156 m to 159 m. The very dense sand and gravel till contained cobbles/boulders and extends to the termination of the boreholes in this area that were drilled down to Elevation 151.8 m.

Flowing artesian groundwater conditions were encountered in Borehole FC-12 when the casing was advanced to a depth of 6 m below ground surface (Elevation 157.9 m) during borehole installation and water was flowing out of the top of the casing, to a height greater than 1.2 m above ground surface (Elevation 165.1 m). Artesian groundwater levels measured at the site during the foundation investigation were measured to be as high as Elevation 171.9 m (about 5 m above ground surface) in September 2012 at Borehole FC-13/13A.

[1] Chapman, L.J. and Putnam, D,F. 1984. The Physiography of Southern Ontario, Ontario Geological Survey, Special Volume 2, Third Edition. Accompanied by Map P. 2715, Scale 1:600,000.

During construction, a nested well (Well Tag# A208617) was installed by the Contractor near the west abutment of the South Bridge (see approximate location on Sheet 511) on September 19 and 20, 2016. A shallow and deep well were sealed at depths of 7.0 m (Elevation 157 m) and 12.2 m (Elevation 151.8 m), respectively, within the non-cohesive till deposit (i.e. aquifer). A copy of the Well Record is included in Appendix B. The groundwater level in the shallow and deep wells was measured at 0.9 m and 7.1 m, respectively, above ground surface (Elevation 164.9 m and Elevation 171.1 m) on September 27, 2016. The nested well was located about 15 m from the test pile location; however, the well was decommissioned at the end of 2016 and was not active during pile load test operations.

4.0 TEST PILE INSTALLATION

Golder, in collaboration with MTO, submitted a specific scope/work plan to be followed by the Contractor for installation of the test pile and execution of a full-scale pile load test. A copy of the scope/work plan titled "Full-Scale Pile Load Test at Highway 401-Fletcher's Creek South Structure, Highway 401 Widening from Highway 403/410 Interchange to Credit River, City of Mississauga, Region of Peel" is provided in Appendix C and was submitted to AECOM for distribution to the Contractor on November 16, 2016.

According to the work plan, the test pile location was to be near the east abutment of the South Bridge; however, access constraints and ongoing construction conflicts in this area resulted in the Contractor, in collaboration with the CA and MTO, installing the test pile near the west abutment of the South Bridge. Golder engineering staff were on site the morning of the scheduled test pile installation and observed welding of the driving shoe to the test pile and clarified installation requirements with the Contractor, AECOM, exp. Services Inc. (exp.) and MNA Engineering Ltd. (MNA). Exp. and MNA were retained by the Contractor to perform the PDA testing and Hiley testing, respectively, on the test pile.

The steel HP310x110 test pile (designated TP1) was installed in the afternoon of December 14, 2016 and was driven using a Delmag D19-42 diesel hammer with a maximum rated energy of about 66 kJ. The average energy transferred to the top of the pile as measured by the PDA testing was about 30 kJ with the hammer operating at a speed of about 38 blows per minute. The existing ground surface at the test pile location was at about Elevation 164.0 m, and the pile was driven to a final tip Elevation 154.4 m (i.e. final embedment depth of about 9.6 m below ground surface).

The "set" criteria established for the test pile to reach the design factored ultimate geotechnical resistance value (f-ULS) of 900 kN (for production piles at the South Bridge) was achieved at tip Elevation 154.4 m based on the results of the PDA testing and Hiley Dynamic Formula test (see Appendices D and E respectively). According to the work plan, Hiley and PDA testing were to be initiated at pile tip Elevation 154.5 m and then carried out in 0.5 m depth increments thereafter until the design geotechnical resistance was achieved. Coincidentally, and consistent with the production piles installed at the South Bridge location, the "set" criteria and measured geotechnical resistance using PDA and Hiley test procedures was achieved at the onset of the specified pile dynamic test procedure and there was no need to drive the test pile any deeper. Details of the PDA test procedure and results are included in the report titled "*Dynamic Analysis of Piles, Contract No. 2015-2018, Hwy 401 – Fletcher's Creek, South Structure, City of Mississauga, Ontario*", prepared by exp and dated January 5, 2017 (see Appendix D). The pile driving record and details of the Hiley test procedure prepared by MNA Engineering Ltd. are provided in Appendix E.

5.0 FULL-SCALE STATIC PILE LOAD TEST

5.1 Load Test Set-up

The reaction load system for the pile load test was designed, supplied and constructed by the Contractor / Anchor Shoring. Anchor Shoring provided pile test arrangement drawings (Drawing Nos. LT1, LT2 and LT3, revision dated April 27, 2017) that show two W920x420 steel reaction beams and timber cribs to support the reaction weight (to counteract jacking load) consisting of approximately 174 steel H-piles stacked in eight layers. Each row of steel piles acted as beams to support the next layer which was shorter and placed perpendicular to the row beneath for increased stability. A copy of the sealed drawings is provided as an enclosure in the memorandum titled "*Full Scale Pile Load Testing, Fletcher Creek, Pile ID INC249, 401 and Mississauga Road –MTO Contract 2015-2018*" prepared by SNC-Lavalin GEM Ontario Inc., dated May 29, 2017, included in Appendix F. As discussed below, SNC-Lavalin GEM Ontario Inc. (SNC-Lavalin) were retained by the Contractor to oversee and carry out the full-scale pile load test set-up and loading / measurement procedure.

A hydraulic cylinder jack was used to transfer the load between the top of the test pile and the reaction beam. Four dial gauges were set up radially on a reference frame to measure the vertical movements of the top of the pile as the test progressed. The dial gauge readings were used as the primary measurement system for pile axial movements and a series of additional survey points at the top of the pile were measured periodically by a licensed surveyor as a secondary measurement system during the test.

Calibration certificates for the hydraulic cylinder jack and dial gauges were provided by Anchor Shoring and copies of the certificates are included in the memorandum in Appendix F. As a load cell was not used for the test, the applied loads (discussed in the next section) were converted to an equivalent pressure for the hydraulic jack system and the pressure gauge continuously checked and recorded.

Golder confirmed the set-up was in general conformance with ASTM 1143 and select photographs of the pile load test set-up and operation are presented in Appendix G.

5.2 Load Test Procedure

The static load test was carried out in general accordance with ASTM D1143-07 using a modified Procedure A – Quick Test method as per the loading and measurement procedure outlined in a letter provided by Golder to the CA titled "*Highway 401 / Fletcher's Creek Pile Load Test, Proposed Loading and Measurement Criteria, Highway 401 Widening from Highway 403/410 Interchange to the Credit River, City of Mississauga, Region of Peel, G.W.P. 2150-10-00, Contract No. 2015-2018*", dated May 10, 2017. A copy of the letter is provided as Enclosure D in the SNC-Lavalin memorandum included in Appendix F.

In summary, loading was carried out in general accordance with the procedure which included seven incremental loadings of 400, 800, 1200, 1600, 2000, 2300 and 2600 kN. All loading increments were held for a minimum 20 minutes, or until the rate of displacement was measured to be less than 0.25 mm/hr up to a maximum of 2 hours. The maximum load was held for a total of 12 hours and then the test pile was unloaded in four increments at loadings of 1900, 1200, 500 and 0 kN, with each load held for one hour and a displacement reading taken prior to each subsequent unloading stage, with a final displacement reading taken 6 hours after removal of the total load.

5.3 Load Test Results

The pile load test was initiated in the morning of May 16, 2017 and was completed the morning of May 17, 2017 (about 24 hours duration). The weather was overcast with seasonal temperatures throughout the duration of the test, and construction operations in the vicinity of the test site were halted by the Contractor such that vibrations and/or construction activities were not influencing the test operations.

A representative from Anchor Shoring applied the load increments by adjusting the hydraulic jack pressures, in collaboration with the SNC-Lavalin representative on site who checked and verified loading increments, calculated and determined hold times, and recorded the pile displacements from the dial gauges (i.e. primary measurement system). The factual data and details of test load increments and measurements are presented in the memorandum prepared by SNC-Lavalin in Appendix F.

A surveyor retained by the Contractor (Hunt Surveys Inc.) recorded survey measurements of the pile displacement (secondary measurement system), and of several points on the reference beams and reaction frame to ensure that the set-up remained stable. According to the memorandum prepared by SNC-Lavalin, reference beam deflection was reported to be less than 1 mm through the duration of the test, and observations by Golder on site concluded that the reaction frame (i.e. stacked H-piles) remained stable throughout the loading procedure (i.e. by observing the interface between bottom row of stacked H-piles relative to the temporary supports).

The majority of the testing operation was monitored by a Golder representative to verify that set-up and load procedures were being followed according to the pre-established test procedure until the maximum load was applied; the Golder representative was not on-site during the unloading operations. After reaching the maximum test load and observing limited pile creep movement, consideration was given to increasing the load, but this was not attempted due to concerns that the reaction system could become unstable, considering that the reaction system was not designed to resist loads in excess of the maximum test load.

A summary of the results of the pile load test are shown on Figure 2 (Appendix A) and include plots of:

- i) applied load vs. time,
- ii) pile movement vs. time, and
- iii) pile movement vs. applied load.

The pile movement measurement shown on Figure 2 is based on the average of the four dial gauge readings. The applied load on the test pile was measured from the pressure gauge of the hydraulic jack and was continuously monitored to ensure constant load was applied to the test pile.

6.0 DISCUSSION

6.1 Design Resistance vs. Tested Pile Resistance

The foundation design report recommends that steel HP 310x110 piles driven to found within the “100-blow” very dense non-cohesive till (with anticipated tip Elevations 152.5 m and 153.5 m at the South Bridge and North Bridge respectively) be designed predominantly as end-bearing foundations with a design factored ultimate limit state value (f-ULS) of 900 kN and 1,000 kN at the South Bridge and North Bridge, respectively.

It was noted in the report that the presence of the "100-blow" soils was variable in depth and the presence of cobbles/boulders and artesian conditions within the non-cohesive till deposit created the potential for variable pile lengths. As a result, the Contract documents included the following notes on the foundation drawing (Sheet 516-1):

Note 6: For the North Structure (East and West Abutment): Piles to be driven in accordance with SS 103-11 using an ultimate geotechnical resistance of 2,000 kN per pile, but must be driven below Elevation 155.5 m and not below 153.5 m without approval of the engineer; and

Note 7: For the South Structure (East and West Abutments): Piles to be driven in accordance with SS 103-11 using an ultimate geotechnical resistance of 1,800 kN per pile, but must be driven below Elevation 154.5 m and not below 152.5 m without approval of the engineer; and

Note 8: The piles shall be driven to not more than 2.0 m above the recommended pile tip elevation and then the driving be monitored by employing the Hiley Dynamic Formula as per SS 103-11.

It is understood that the majority of production piles installed at the South Bridge and north portion of the North Bridge achieved the ultimate resistance upon initial driving as measured using the Hiley test method, with a resistance factor of 0.5 then being applied to estimate the factored ULS value. All production piles tested using the Hiley method upon re-tap met the design geotechnical resistance, and PDA testing was not performed on any production piles installed to date. The Hiley tests performed on production piles installed at the east and west abutments of the South Bridge (i.e. close to the test pile location) averaged an ultimate geotechnical resistance of about 2,400 kN and 3,000 kN (f-ULS = 1,200 kN and 1,500 kN), respectively.

Regarding the actual pile load test, the test pile is considered to have not reached failure at an ultimate load of 2,600 kN in compression (see Figure 2).

A comparison of the test pile TP1 ultimate and factored geotechnical resistance estimated from the PDA test, Hiley test and full-scale pile load test is provided below.

Test Pile No.	Test Pile Tip Elevation (m)	*Eq. Penetration Resistance (Blows/25 mm)	Test / Analysis Method	Set-Up Period from Pile Installation to Test (days)	Ultimate Resistance (kN)	Resistance Factor	f-ULS Resistance (kN)
TP1	154.4	5.1	Hiley Test	0	2,368	0.5	1,184
		4.3	PDA Test	0	2,200	0.5	1,100
		4.3	Full-Scale Pile Load Test	153	> 2,600	0.6	> 1,560

*Using Delmag D19-42 diesel hammer; penetration measured for a certain number of blows and then converted to equivalent blows per 25mm penetration.

The ultimate geotechnical resistance measured from the pile load test (i.e., >2,600 kN) 153 days after pile installation is much greater than the values measured from the Hiley test and PDA test performed on completion of initial pile driving. There is considered good agreement between the measured Hiley and PDA test results on initial completion of driving.

The factored geotechnical resistance values measured from the Hiley, PDA, and pile load tests are all greater than the design factored geotechnical resistance recommended in the FIDR and Contract Drawing Sheet 516-1 (i.e. f-ULS of 900 kN and 1,000 kN for South and North Bridges respectively), with the f-ULS resistance values from the Hiley and PDA test upon initial driving being only slightly higher (10% to 30%) than the design values.

The geotechnical resistance values at SLS (for 25 mm of settlement) recommended in the FIDR for design were 700 kN and 800 kN for the South and North Bridges respectively. The load-deformation characteristics of the pile load test measured 22 mm of deflection at the top of the pile under the maximum test load of 2,600 kN. The measured deflection includes elastic compression of the pile itself and upon unloading of the test pile, the net vertical deflection of the pile was measured to be about 11 mm. Based on the results of the static pile load test, the geotechnical resistance at SLS (for 25 mm of settlement) is greater than the factored geotechnical resistance at ULS.

The FIDR states that "*given the high artesian pressures within the cohesionless till soils near the estimated pile tip elevations, the resistances are lower than would typically be given for driven piles in similar soil conditions with similar pile lengths*". Based on the results of the pile load test, the design geotechnical resistance for predominantly end-bearing piles penetrating into the very dense zone of the flowing artesian groundwater till deposit may be increased for similar pile installation and soil and groundwater conditions, provided that sufficient elapsed time is available between the installation of piles and the loading of the foundations (i.e., construction of the bridge deck).

6.2 Anticipated Strength Gain with Time

This discussion and interpretation of the pile load test results are intended to provide MTO with supplementary information for assessment of remaining piling activities on site, to make informed decisions whether additional piles or piles installed to greater depths are warranted given that the geotechnical resistance measured during initial driving is expected to increase with time. The pile load test results and interpretations may also be referenced for future projects with similar site subsurface conditions and pile lengths. However, it is noted that the results of this static pile load test are specific to the pile, soil and groundwater conditions at the specific test pile site only and should be used with caution if a practitioner chooses to interpret the results for any other pile location or project. Those using the information contained in this report should make their own interpretation of the factual information as such interpretation may affect/impact design and estimated long-term geotechnical resistance of piles, pile layout and equipment selection, proposed construction methods, scheduling and the like.

For the test pile (TP1), comparing the results of the Hiley and PDA test performed upon completion of initial driving and the full-scale pile load test performed about 153 days after initial driving, there is greater than a 10% and 18% increase in the estimated ultimate geotechnical resistance (2,368 kN and 2,200 kN vs. 2,600 kN) over this approximately five-month timeframe. The actual increase in geotechnical resistance cannot be established from the results of the static pile load test as the pile was not tested to ultimate failure. The potential gain in geotechnical resistance over time increases substantially over the design geotechnical resistance considering that the resistance factor for a full-scale pile load test is 0.6, compared to a factor of 0.5 for Hiley and PDA testing.

As the re-tap of production piles at the South and North Bridges installed to date indicated both an increase and decrease in ultimate geotechnical resistance using the Hiley test method, the results of the pile load test suggest that set-up and long-term strength gain is achievable.

Based on the borehole information, pile driving record, and monitoring wells installed during construction, the test pile likely penetrated through the cohesive clayey cap and the pile tip terminated within the very dense non-cohesive (sand and gravel) till deposit under artesian groundwater pressures estimated to be about 4 m above ground surface (Elevation 168 m) relative to the pile tip level (Elevation 154.4 m).

Based on the test data, it is our opinion that the driven steel HP310x110 test pile experienced significant strength gain in geotechnical resistance over time, between the time of installation to the application of the compression test load.

Assuming similar soil and groundwater conditions throughout this site (i.e., presence of artesian pressures at/below pile tip), it is anticipated that the geotechnical resistance of surrounding piles in the immediate area of the test pile driven into the very dense non-cohesive till deposit will increase over time compared to the measured Hiley test and PDA test values upon completion of initial driving.

6.3 Future Pile Load Tests – General

The following suggestions / recommendations are provided for future pile load tests on similar MTO projects:

- Test Pile Installation: The designer(s) and owner should be on site or correspond / meet directly with the Contractor / CA team so that the target geotechnical resistance and installation procedure for the test pile is clearly communicated. The Hiley and PDA that results should be communicated to the designer/owner immediately to confirm anticipated and actual test results upon initial driving are acceptable prior to demobilizing equipment / crew from site.
- A secondary measurement system of survey points should be established to monitor pile displacement and tilting/movement of the load reaction system during the pile load test, and procedures should be clearly communicated and incorporated into the work plan.
- Consideration should be given to installing instrumentation to measure porewater pressures prior to, during, and after test pile installation, as applicable. This would improve understanding of initial porewater development and dissipation over time and could potentially be correlated to prediction models of potential strength gain over time or relaxation in different soil and groundwater conditions.
- For integral abutment design, consideration could be given to performing ASTM D3966/D3966M-07 “Standard Test Methods for Deep Foundations Under Lateral Load” on the test pile to predict lateral performance and estimate lateral subgrade reaction values of the surrounding soil to assess whether “standard” integral abutment pile design (i.e., the upper portion of the pile installed within CSP liners backfilled with loose sand) is required or can be negated.

7.0 CLOSURE

This Report was prepared by Mr. Matthew Kelly, P.Eng., and reviewed by Mr. Kevin Bentley, P.Eng., a geotechnical engineer and Associate with Golder, with technical input from Mr. Murty Devata, P.Eng., a specialist foundations consultant with Golder. Mr. Jorge Costa, P.Eng. and Ms. Lisa Coyne, P.Eng., Designated MTO Foundations Contacts for Golder, conducted an independent quality control review of this report.

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APPENDIX A

Figures

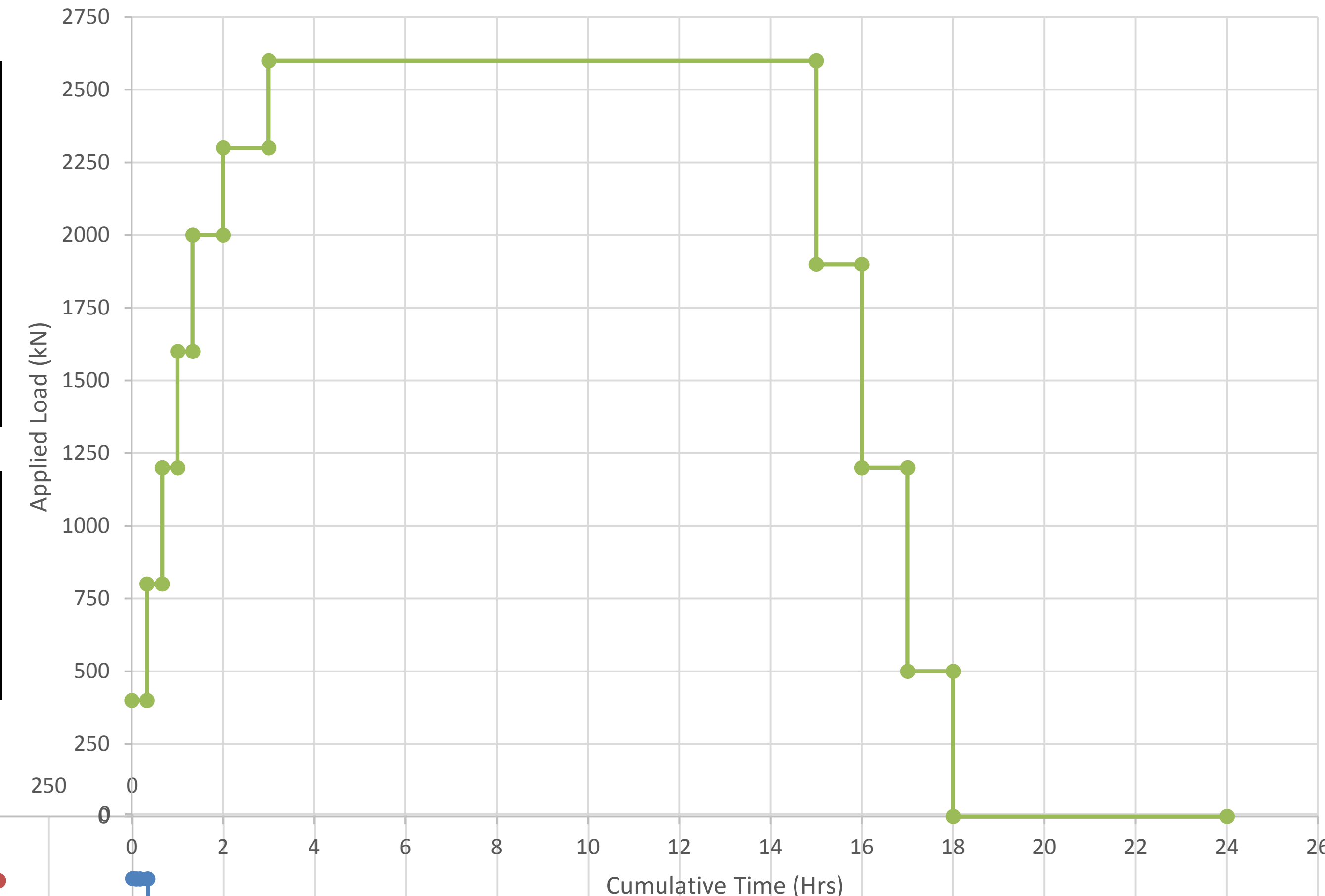
APPROXIMATE TEST PILE LOCATION (TP1)

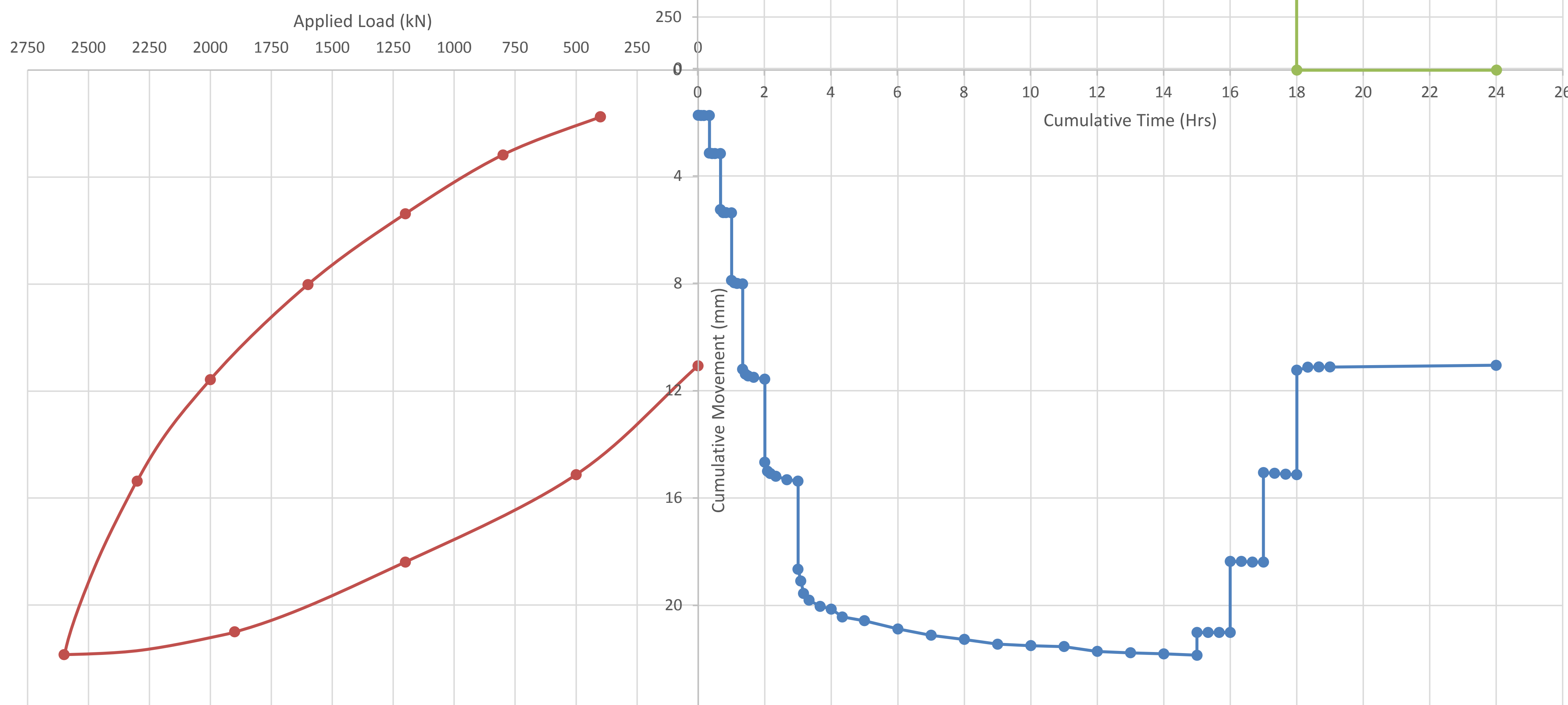
Reference: ©2017 Google – Image Digital Globe

CLIENT AECOM / MTO		PROJECT HIGHWAY 401 – FLETCHER'S CREEK BRIDGE MISISSAUGA, ONTARIO	
CONSULTANT Golder Associates	YYYY-MM-DD: 2017-08-31 PREPARED: MWK DESIGN: REVIEW: KJB APPROVED: JMAC	TITLE PILE LOAD TEST LOCATION PLAN	
		PROJECT No. 10-1111-0211	FIGURE 1

PILE ID	TPI	DATE DRIVEN	December 14, 2016
PILE TYPE		HP310x110	
SHOE DETAILS	Titus	Standard	'H' Bearing Point
FINAL LENGTH DRIVEN			16.8 m
LENGTH AFTER CUT-OFF			10 m
EMBEDDED LENGTH			9.6 m
CUT-OFF ELEVATION			164.4 m
TIP ELEVATION			154.4 m

LOAD TEST REULTS	
TEST No.	TPI
DATE	May 16/17 2017
MAX LOAD APPLIED	2600 kN
ESTIMATED FAILURE	>2600 kN





CLIENT
AECOM / MTO

PROJECT
HIGHWAY 401– FLETCHER'S CREEK BRIDGE
MISSISSAUGA, ONTARIO

CONSULTANT
Golder Associates

YYYY-MM-DD	2017-10-06
PREPARED	DH
DESIGN	DH
REVIEW	KJB
APPROVED	MSD/JMAC

TITLE
STATIC PILE LOAD TEST RESULTS

PROJECT No.

FIGURE
2

APPENDIX B

Relevant Contract Documents – Updated for Test Pile (TP1) and Monitoring Well Locations

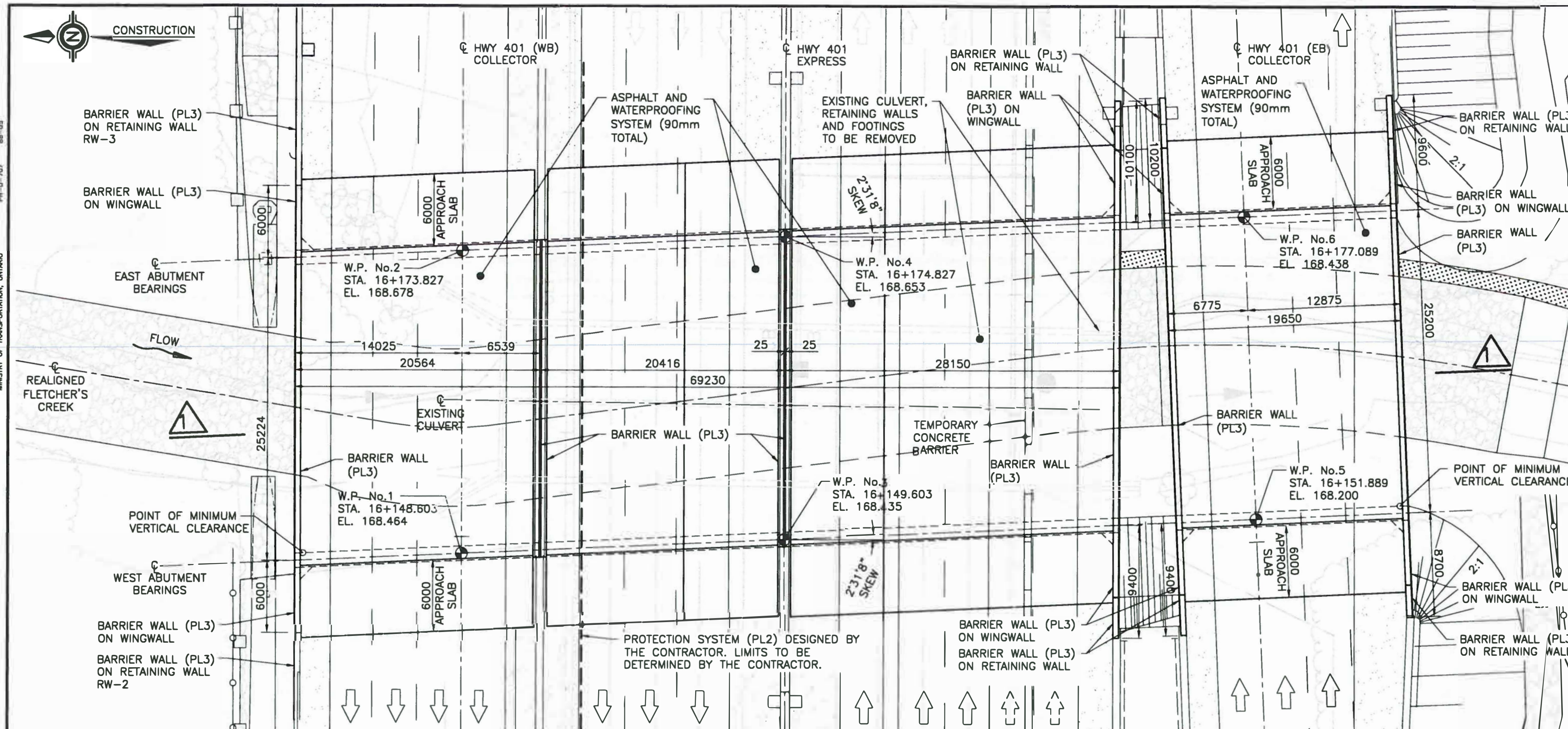
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

CONT No 2015–2018
GWP No 2150–01–00

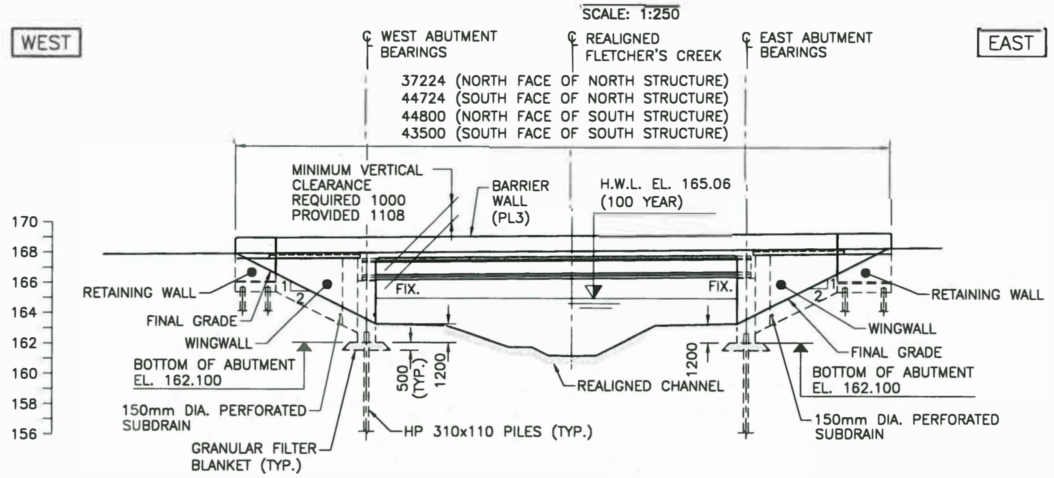
HIGHWAY 401
FLETCHER'S CREEK
CULVERT REPLACEMENT
GENERAL ARRANGEMENT

SHEET 510–2

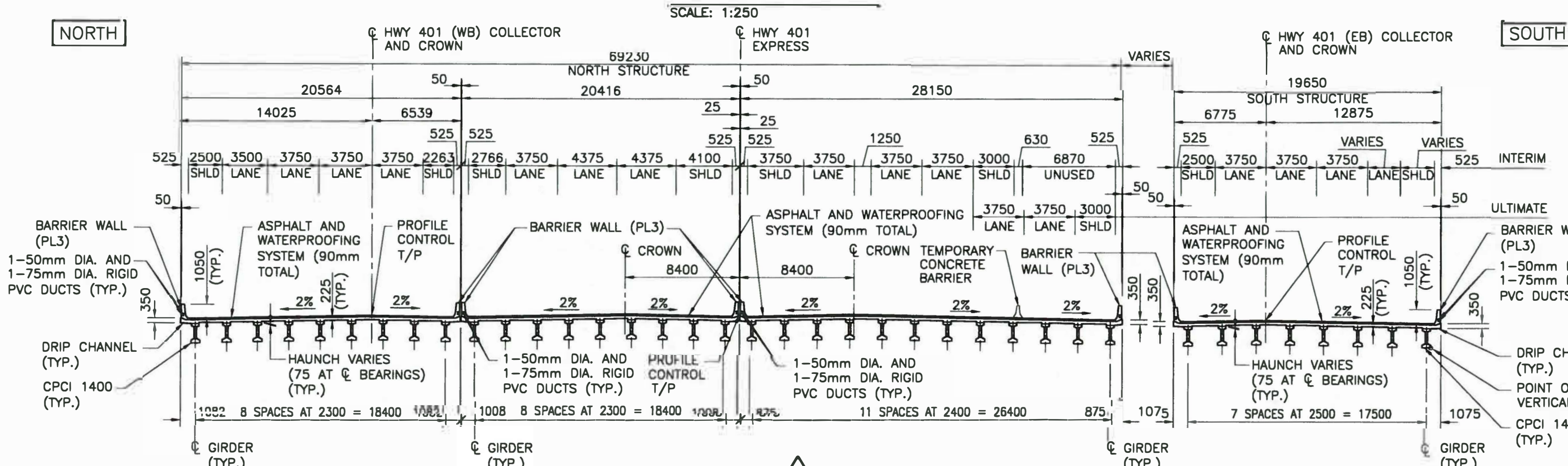
AECOM



PLAN
SCALE: 1:250

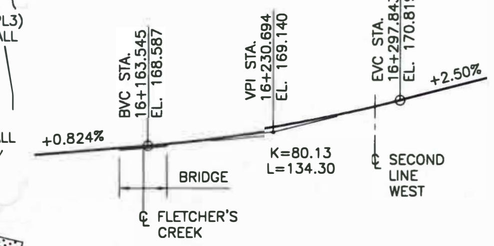


SOUTH ELEVATION
SCALE: 1:250

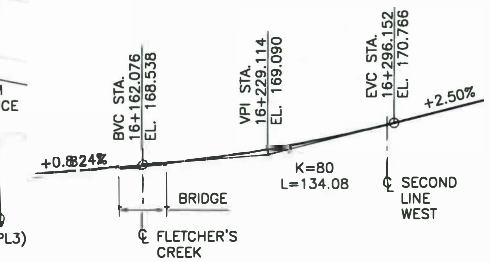


SECTION
SCALE: 1:250

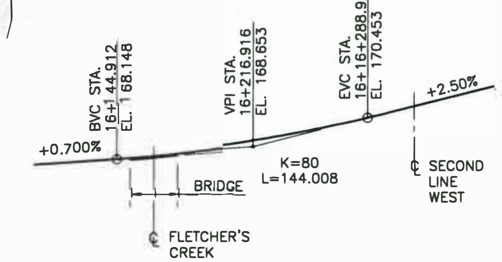
NOTE: DIMENSIONS ARE MEASURED PERPENDIULAR TO ℄ HWY 401 OR ℄ HWY 401 (EB) COLLECTOR.



HWY 401 (WB) COLLECTOR PROFILE
SCALE: N.T.S.



HWY 401 EXPRESS PROFILE
SCALE: N.T.S.



HWY 401 (EB) COLLECTOR PROFILE
SCALE: N.T.S.

GENERAL NOTES:

1. CLASS OF CONCRETE:

PRESTRESSED GIRDERS	50 MPa
REMAINDER	30 MPa

2. CLEAR COVER TO REINFORCING STEEL:

FOUNDATIONS		100 ± 25
DECK	TOP	70 ± 20
	BOTTOM	40 ± 10
REMAINDER		70 ± 20 UNLESS OTHERWISE NOTED

3. REINFORCING STEEL:

REINFORCING STEEL SHALL BE GRADE 400W UNLESS OTHERWISE SPECIFIED.

UNLESS SHOWN OTHERWISE, TENSION LAP SPLICES FOR REINFORCING STEEL BARS SHALL BE CLASS B.

STAINLESS REINFORCING STEEL SHALL BE TYPE 316LN OR DUPLEX 2205 AND HAVE A MINIMUM YIELD STRENGTH OF 500 MPa, UNLESS OTHERWISE SPECIFIED.

BAR MARKS WITH PREFIX 'S' DENOTE STAINLESS STEEL BARS.

BAR HOOKS SHALL HAVE STANDARD HOOK DIMENSIONS USING MINIMUM BEND DIAMETERS, WHILE STIRRUPS AND TIES SHALL HAVE MINIMUM HOOK DIMENSIONS. ALL HOOKS SHALL BE IN ACCORDANCE WITH THE STRUCTURAL STANDARD DRAWING SS12–1, UNLESS INDICATED OTHERWISE.

CONSTRUCTION NOTES:

1. THE CONTRACTOR SHALL ESTABLISH THE BEARING SEAT ELEVATIONS BY DEDUCTING THE ACTUAL BEARING THICKNESSES FROM THE TOP OF BEARING ELEVATIONS. IF THE ACTUAL BEARING THICKNESSES ARE DIFFERENT FROM THOSE GIVEN WITH THE BEARING DESIGN DATA, THE CONTRACTOR SHALL ADJUST THE REINFORCING STEEL TO SUIT.
2. THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS, DETAILS AND ELEVATIONS OF THE EXISTING STRUCTURE THAT ARE RELEVANT TO THE WORK SHOWN ON THE DRAWINGS PRIOR TO COMMENCEMENT OF THE WORK. ANY DISCREPANCIES SHALL BE REPORTED TO THE CONTRACT ADMINISTRATOR AND THE PROPOSED ADJUSTMENT OF THE WORK REQUIRED TO MATCH THE EXISTING STRUCTURE SHALL BE SUBMITTED FOR APPROVAL.
3. THE CONTRACTOR IS FULLY RESPONSIBLE FOR THE ADEQUATE PROTECTION OF UTILITIES, SERVICES, STRUCTURES, ROADWAYS, WATERCOURSES, ETC. DURING CONSTRUCTION OPERATIONS.
4. THE CONTRACT SHALL ADVISE ALL UTILITY COMPANIES IN WRITING OF HIS PROPOSED WORK. THE CONTRACTOR SHALL BE RESPONSIBLE AT HIS OWN EXPENSE FOR ANY DAMAGE TO UTILITIES BY THE CONTRACTOR.
5. BACKFILL SHALL NOT BE PLACED BEHIND THE ABUTMENTS UNTIL THE DECK HAS REACHED 75% OF ITS SPECIFIED CONCRETE COMPRESSIVE STRENGTH. BACKFILL SHALL BE PLACED SIMULTANEOUSLY BEHIND BOTH ABUTMENTS KEEPING THE HEIGHT OF BACKFILL APPROXIMATELY THE SAME. AT NO TIME SHALL THE DIFFERENCE IN ELEVATIONS BE GREATER THAN 500 mm.
6. CONSTRUCT ABUTMENTS AND WINGWALLS TO THE BEARING SEAT ELEVATIONS. THE CONTRACTOR SHALL SUPPLY TEMPORARY LATERAL BRACING FOR THE ABUTMENTS. FORMWORK AND LATERAL BRACING SHALL NOT BE REMOVED UNTIL THE DECK CONCRETE HAS REACHED 75% OF ITS SPECIFIED 28–DAY STRENGTH.
7. CONTRACTOR'S METHOD OF PROTECTION TO BE SUBMITTED TO CONTRACT ADMINISTRATOR FOR INFORMATION BEFORE PROCEEDING WITH THE WORK.

LIST OF DRAWINGS:

R1–1	GENERAL ARRANGEMENT
R1–2	BOREHOLE LOCATIONS
R1–3	SOIL STRATA
R1–4	SOIL STRATA
R1–5	CONSTRUCTION STAGING
R1–6	ACCESS AND PROTECTION
R1–7	FOUNDATIONS
R1–8	ABUTMENTS I
R1–9	ABUTMENTS II
R1–10	WINGWALLS
R1–11	RETAINING WALLS
R1–12	PRESTRESSED GIRDERS AND BEARINGS
R1–13	DECK I
R1–14	DECK II
R1–15	DECK III
R1–16	BARRIER WALLS I
R1–17	BARRIER WALLS II
R1–18	6000mm APPROACH SLAB
R1–19	PILE DRIVING CONTROL
R1–20	HOOK DIMENSIONS
R1–21	EMBEDDED WORK IN STRUCTURE

APPLICABLE STANDARD DRAWINGS:

OPSD 3101.150	WALLS – ABUTMENT, BACKFILL MINIMUM GRANULAR REQUIREMENT
OPSD 3190.100	WALLS – RETAINING AND ABUTMENT WALL DRAIN
OPSD 3370.100	DECK, WATERPROOFING, HOT APPLIED ASPHALT MEMBRANE WITH PROTECTION BOARD
OPSD 3370.101	DECK, WATERPROOFING, HOT APPLIED ASPHALT MEMBRANE AT ACTIVE CRACKS GREATER THAN 2mm WIDE AND CONSTRUCTION JOINTS
OPSD 3390.100	DECK DRIP CHANNEL
OPSD 3941.200	FIGURES IN CONCRETE – SITE NUMBER AND DATE LAYOUT
OPSD 3950.100	JOINTS – CONCRETE EXPANSION AND CONSTRUCTION ON STRUCTURE

LIST OF ABBREVIATIONS:

EB	DENOTES EAST BOUND
WB	DENOTES WEST BOUND
EL.	DENOTES ELEVATION
FIX.	DENOTES FIXED
SHLD	DENOTES SHOULDER
T/P	DENOTES TOP OF PAVEMENT
TYP.	DENOTES TYPICAL
W.P.	DENOTES WORKING POINT
H.W.L.	DENOTES HIGH WATER LEVEL

REFERENCE DRAWINGS:

- THE KING'S HIGHWAY NO. 401, MEADOWVALE CREEK CULVERT, DRAWINGS No. D–4003–1 AND D–4003–2, PREPARED BY THE DEPARTMENT OF HIGHWAYS ONTARIO, BRIDGE OFFICE – TORONTO, DATED NOVEMBER 1957.

LICENSED PROFESSIONAL ENGINEER G.X. CHEN 100118885 PROVINCE OF ONTARIO
LICENSED PROFESSIONAL ENGINEER G. BAJLON PROVINCE OF ONTARIO

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

REVISIONS

DATE	BY	DESCRIPTION
SEP 2016	G.C.	② DELETED CSP's
APR 2016	G.C.	① REVISED LENGTHS OF NE AND NW RETAINING WALLS OF SOUTH STRUCTURE

DESIGN	G.C.	CHK	G.B.	CODE	CHBDC–06	LOAD CL–625–ONT	DATE OCT. 2015
DRAWN	D.L.	CHK	V.K.	SITE	24–129/C		DWG R1–1

MINISTRY OF TRANSPORTATION, ONTARIO PH–D–767 88–05

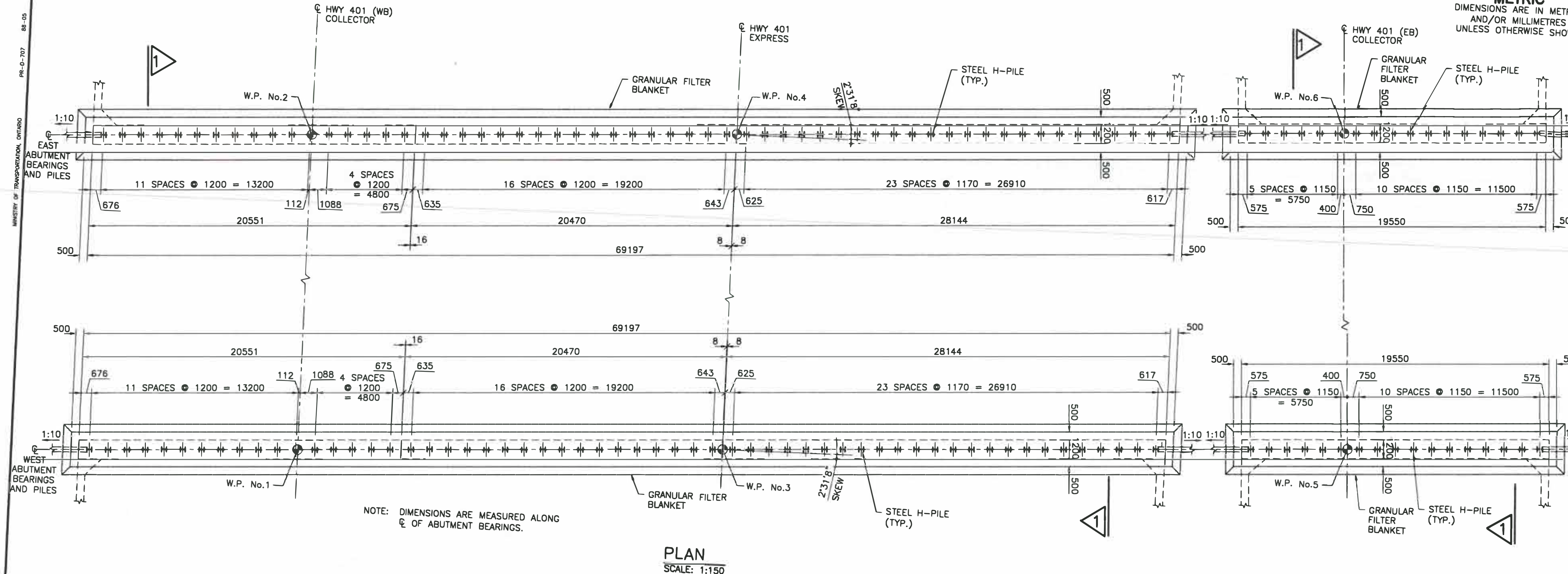
DRAWING NAME: 60213979_ST–24–0129–C–R1–1 GA.dwg
SAVED BY: lueckd SAVED DATE: 9/20/2016 1:48 PM PLOT DATE: 9/20/2016 1:52 PM

CONSTRUCTION

METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

CONT No 2015–2018 GWP No 2150–01–00	
HIGHWAY 401 FLETCHER'S CREEK CULVERT REPLACEMENT FOUNDATIONS	SHEET 516–1

AECOM



PLAN
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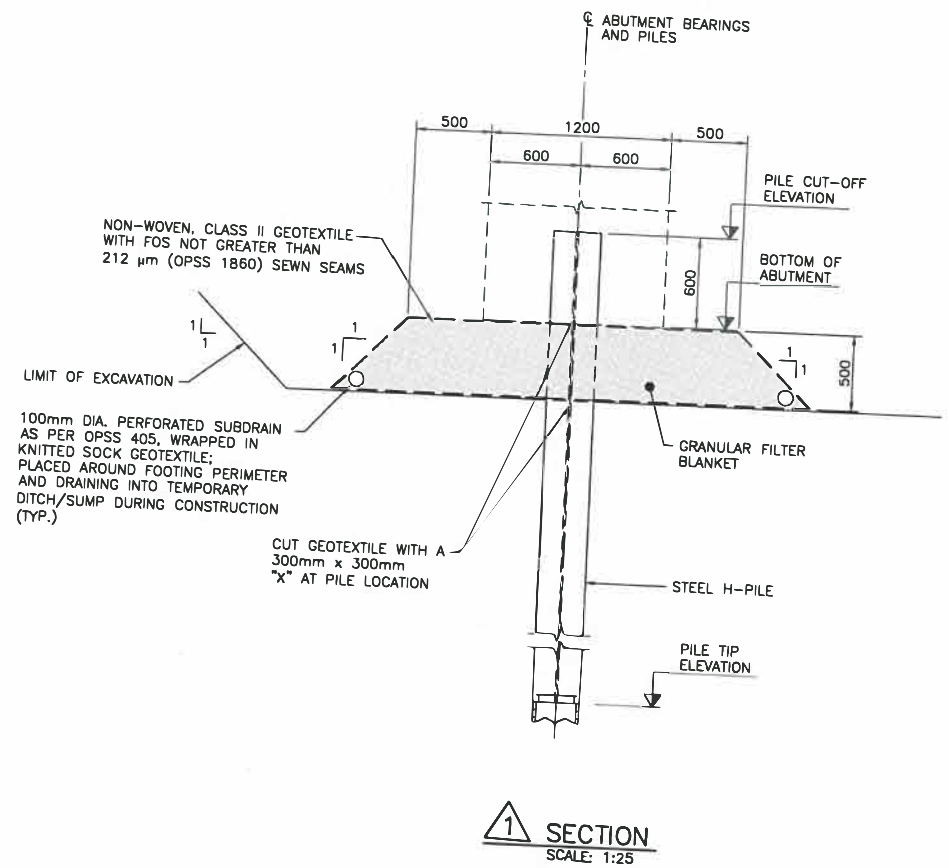


1 SECTION
SCALE: 1:25

NOTES:

1. GRANULAR FILTER BLANKETS SHALL BE PLACED PRIOR TO PILE DRIVING.
2. IF BLANKET IS DISTURBED DURING PILE DRIVING, THE BLANKET SHALL BE RESTORED TO THE DETAILS SHOWN ON THIS DRAWING AFTER THE COMPLETION OF THE PILE DRIVING.
3. STEEL H–PILES SHALL BE HP 310x110 FITTED WITH TITUS STANDARD 'H' BEARING POINTS OR APPROVED EQUAL.
4. PILE SPACINGS ARE MEASURED AT THE UNDERSIDE OF ABUTMENTS.
5. PILE LENGTHS SHOWN ARE THE THEORETICAL LENGTHS BELOW CUT–OFF.
6. FOR THE NORTH STRUCTURE (EAST AND WEST ABUTMENTS): PILES TO BE DRIVEN IN ACCORDANCE WITH SS 103–11 USING AN ULTIMATE GEOTECHNICAL RESISTANCE OF 2000 kN PER PILE, BUT MUST BE DRIVEN BELOW ELEVATION 155.5 m AND NOT BELOW 153.5 m WITHOUT APPROVAL OF THE ENGINEER.
7. FOR THE SOUTH STRUCTURE (EAST AND WEST ABUTMENTS): PILES TO BE DRIVEN IN ACCORDANCE WITH SS 103–11 USING AN ULTIMATE GEOTECHNICAL RESISTANCE OF 1800 kN PER PILE, BUT MUST BE DRIVEN BELOW ELEVATION 154.5 m AND NOT BELOW 152.5 m WITHOUT APPROVAL OF THE ENGINEER.
8. THE PILES SHALL BE DRIVEN TO NOT MORE THAN 2.0 m ABOVE THE RECOMMENDED PILE TIP ELEVATION AND THEN THE DRIVING BE MONITORED BY EMPLOYING THE HILEY DYNAMIC FORMULA AS PER SS 103–11.

PILE DESIGN DATA:

NORTH STRUCTURE:
AXIAL RESISTANCE AT SLS: 800 kN/PILE
FACTORED AXIAL RESISTANCE AT ULS: 1000 kN/PILE

SOUTH STRUCTURE:
AXIAL RESISTANCE AT SLS: 700 kN/PILE
FACTORED AXIAL RESISTANCE AT ULS: 900 kN/PILE

PILE DATA

LOCATION		BATTER	No. REQ'D	PILE CUT–OFF ELEVATION (m)	ESTIMATED PILE TIP ELEVATION (m)	ESTIMATED LENGTH (m)
NORTH STRUCTURE	EAST ABUTMENT	VERTICAL	56	162.700	153.500	9.200
		1:10	2	162.700	153.500	9.246
	WEST ABUTMENT	VERTICAL	56	162.700	153.500	9.200
		1:10	2	162.700	153.500	9.246
SOUTH STRUCTURE	EAST ABUTMENT	VERTICAL	15	162.700	152.500	10.200
		1:10	2	162.700	152.500	10.251
	WEST ABUTMENT	VERTICAL	15	162.700	152.500	10.200
		1:10	2	162.700	152.500	10.251

APPLICABLE STANDARD DRAWINGS:

OPSD 3000.150 FOUNDATION, PILES, STEEL H–PILE SPLICE

LICENSED PROFESSIONAL ENGINEER G.X. CHEN 100118885 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER G. BAJLON PROVINCE OF ONTARIO

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

REVISIONS			
SEP 2016	G.C.	1	DELETED CSP's; ROTATED STEEL H–PILES BY 90 DEGREES
DATE	BY		DESCRIPTION

DESIGN G.C. CHK G.B. CODE CHBDC–06 LOAD CL–625–ONT DATE OCT. 2015
DRAWN D.L. CHK V.K. SITE 24–129/C

DRAWING NAME: 60213979_ST–24–0129–C–R1–7 Foundations.dwg PLOT DATE: 9/20/2016 3:00 PM
SAVED BY: lunakd SAVED DATE: 9/20/2016 2:35 PM

MINISTRY OF TRANSPORTATION, ONTARIO PR–D–707 88–05

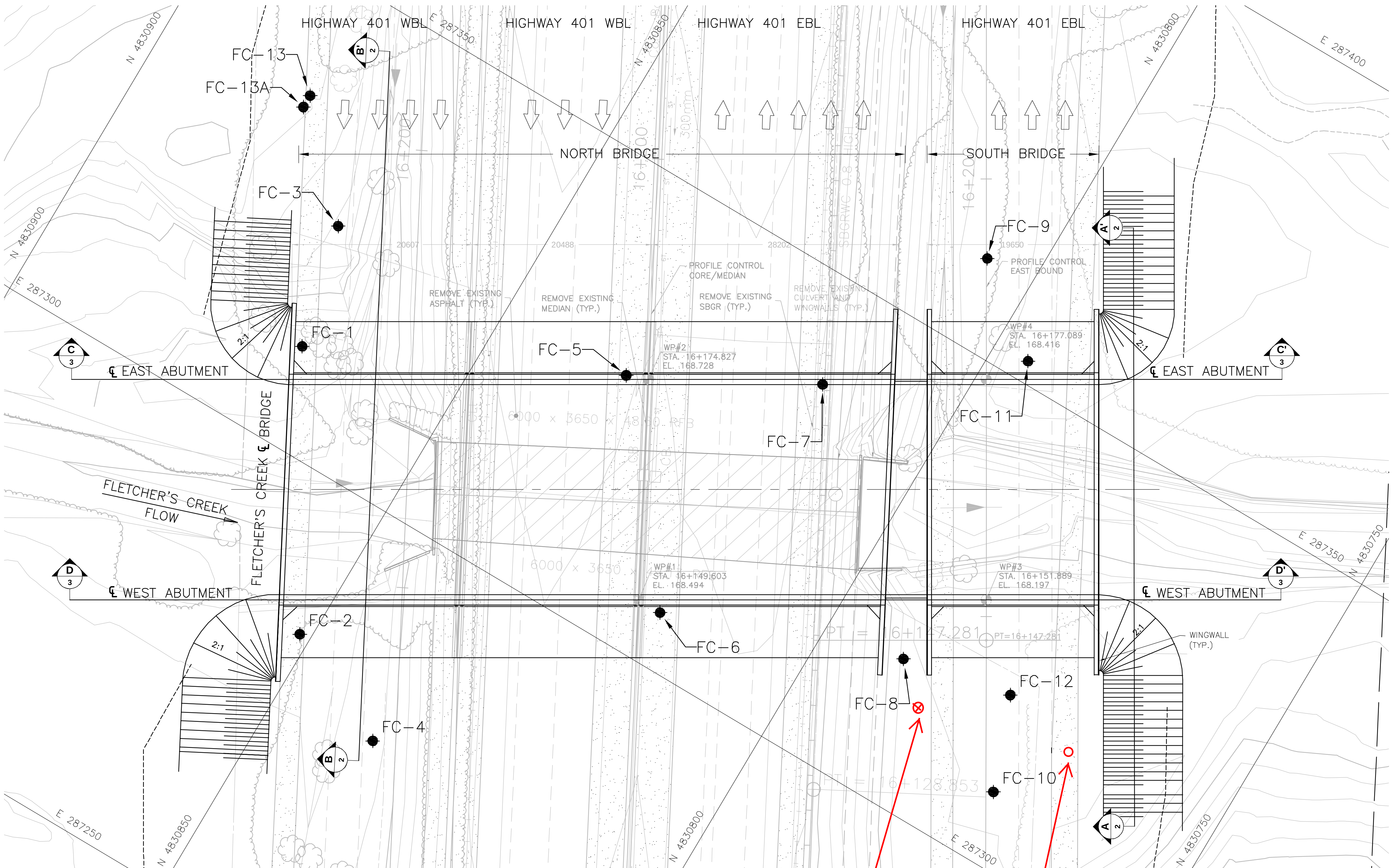
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 2150–01–00

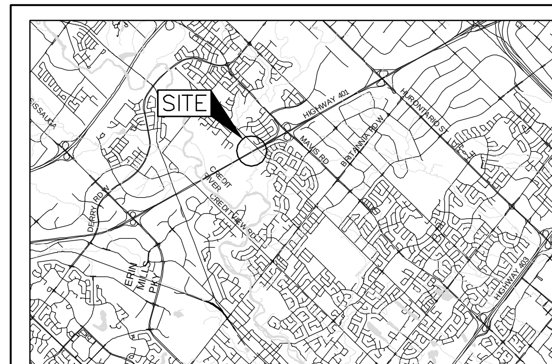
HIGHWAY 401
FLETCHER'S CREEK BRIDGES
BOREHOLE LOCATIONS

SHEET 511

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



PLAN
SCALE
5 0 5 10 m



KEY PLAN
SCALE
1.5 0 1.5 3 km

LEGEND

Borehole – Current Investigation

No.	ELEVATION	NORTHING	EASTING
FC–1	163.7	4830867.9	287311.4
FC–2	163.8	4830851.3	287283.0
FC–3	165.9	4830871.4	287325.3
FC–4	164.4	4830837.9	287276.8
FC–5	168.6	4830834.4	287327.5
FC–6	168.3	4830817.2	287306.2
FC–7	168.5	4830814.6	287338.1
FC–8	164.0	4830790.6	287315.9
FC–9	164.3	4830805.8	287360.1
FC–10	164.1	4830774.0	287308.1
FC–11	164.4	4830795.8	287352.4
FC–12	163.9	4830778.0	287318.6
FC–13	167.1	4830881.8	287336.5
FC–13A	167.1	4830881.8	287335.0

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

Base plans provided in digital format by Aecom, drawing file nos. "60213979_ST-24-0129-C-1 GA_30%.dwg", received September 25, 2012 and "X-60213979-C-CTR-HWY401_HALFm.dwg" received September 27, 2012.

REGISTERED PROFESSIONAL ENGINEER T. J. GARDE Feb. 7, 2013 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER T.V. AYETAN 100130071 Feb. 7, 2013 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 30M12–356
HWY. 401 | PROJECT NO. 10–1111–0211 | DIST.
SUBM'D. TVA | CHKD. TVA | DATE: Feb–2013 | SITE:
DRAWN: DD | CHKD. KJB | APPD. TG | DWG. R1-2

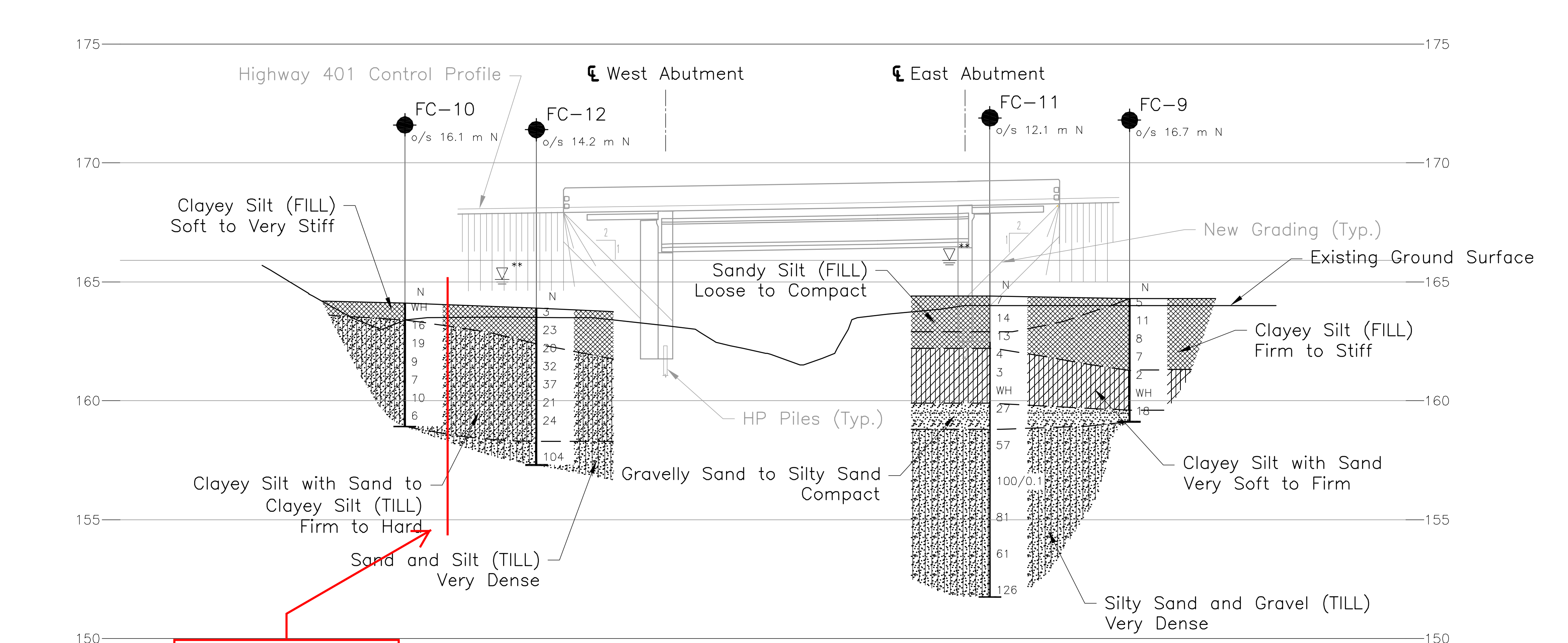
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 2150-01-00

HIGHWAY 401
FLETCHER'S CREEK BRIDGES
SOIL STRATA

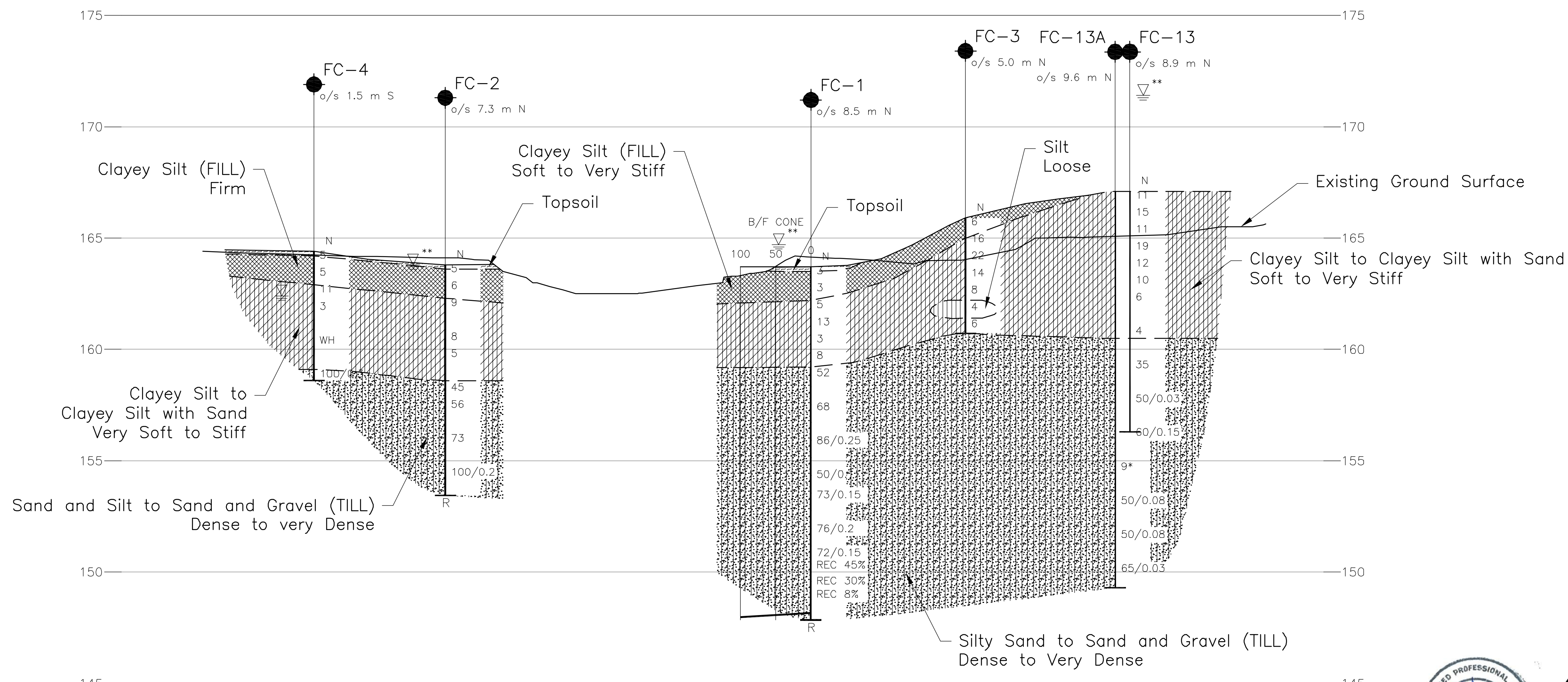
SHEET 512

Golder Associates
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



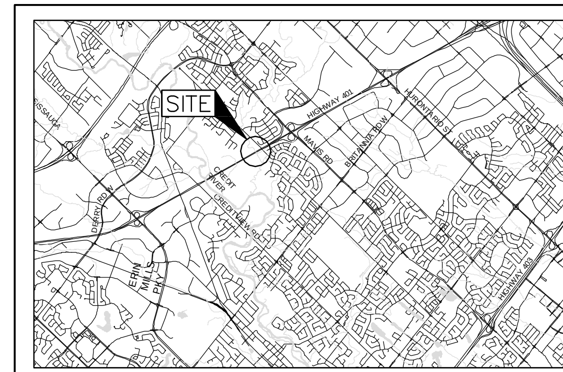
SOUTH PROFILE
A-A' 1

SCALE
HOR. 5 0 5 10 m
VER. 2.5 0 2.5 5 m



NORTH PROFILE
B-B' 1

SCALE
HOR. 5 0 5 10 m
VER. 2.5 0 2.5 5 m



KEY PLAN
SCALE
1.5 0 1.5 3 km

LEGEND

- ● Borehole – Current Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- ▽** Artesian WL conditions encountered upon completion of drilling
- ▽ WL upon completion of drilling
- R Refusal

No.	ELEVATION	NORTHING	EASTING
FC-1	163.7	4830867.9	287311.4
FC-2	163.8	4830851.3	287283.0
FC-3	165.9	4830871.4	287325.3
FC-4	164.4	4830837.9	287276.8
FC-9	164.3	4830805.8	287360.1
FC-10	164.1	4830774.0	287308.1
FC-11	164.4	4830795.8	287352.4
FC-12	163.9	4830778.0	287318.6
FC-13	167.1	4830881.8	287336.5
FC-13A	167.1	4830881.8	287335.0

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REFERENCE

Existing Ground Surface Cut and Profile obtained from digital files provided by Aecom, Drawing Files "60213979_ST-24-0129-C-1 GA_30%.dwg" received September 25, 2012 and "X-60213979-C-CTR-HWY401_HALFm.dwg" received September 27, 2012.

REGISTERED PROFESSIONAL ENGINEER T. J. GARDE Feb. 7, 2013 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER T.V. AYETAN 100130071 Feb. 7, 2013 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 30M12-356

HWY. 401	PROJECT NO. 10-1111-0211		DIST.
SUBM'D. TVA	CHKD. TVA	DATE: Feb-2013	SITE:
DRAWN: DD	CHKD. KJB	APPD. TG	DWG. R1-3

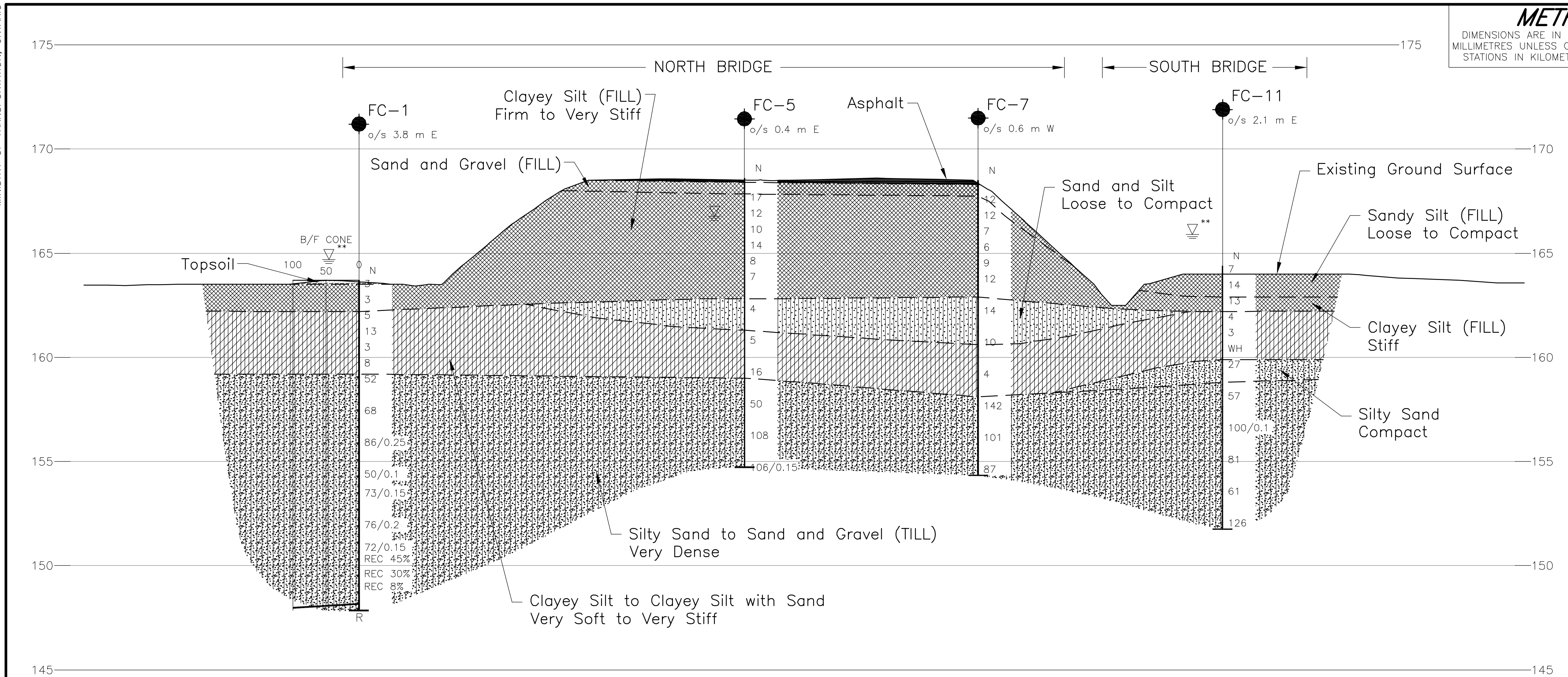
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 2150-01-00

HIGHWAY 401
FLETCHER'S CREEK BRIDGES
SOIL STRATA

SHEET 513

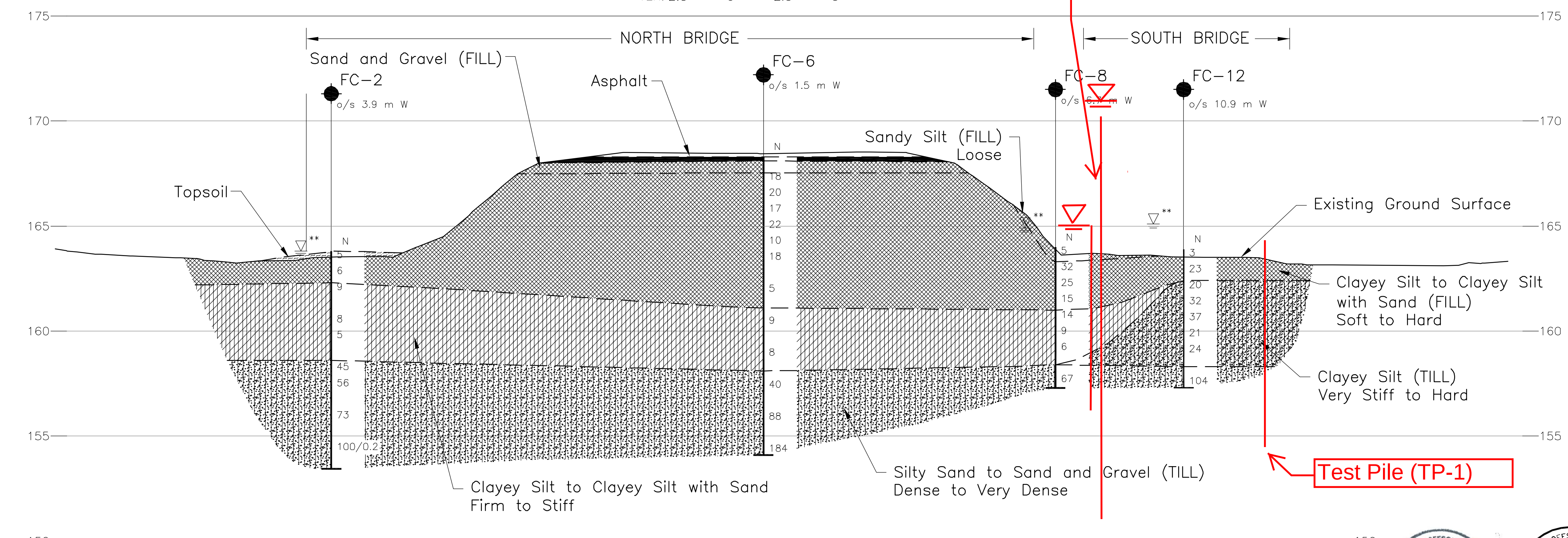
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



C-C' 1 EAST ABUTMENT

SCALE
HOR. 5 0 5 10 m
VER. 2.5 0 2.5 5 m

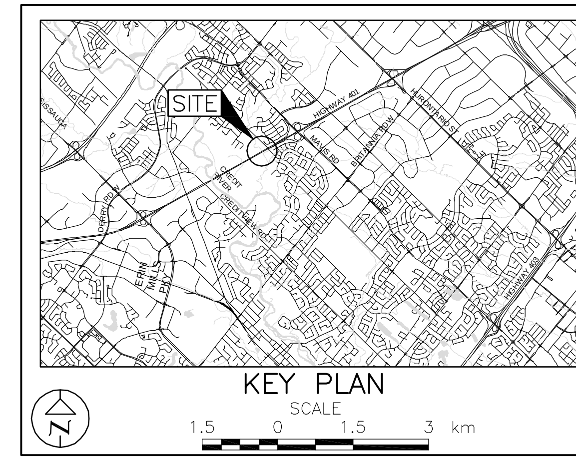
MW TAG # A208617



Test Pile (TP-1)

D-D' 1 WEST ABUTMENT

SCALE
HOR. 5 0 5 10 m
VER. 2.5 0 2.5 5 m



KEY PLAN
SCALE
1.5 0 1.5 3 km

LEGEND

- Borehole – Current Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- Artesian WL conditions encountered upon completion of drilling
- WL upon completion of drilling
- R Refusal

No.	ELEVATION	NORTHING	EASTING
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FC-8	164.0	4830790.6	287315.9
FC-11	164.4	4830795.8	287352.4
FC-12	163.9	4830778.0	287318.6

NOTES

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REFERENCE

Existing Ground Surface Cut obtained from digital files provided by Aecom, (Drawing File "X-60213979-C-CTR-HWY401_HALFm.dwg") received September 27, 2012.

NO.	DATE	BY	REVISION

Geocres No. 30M12-356

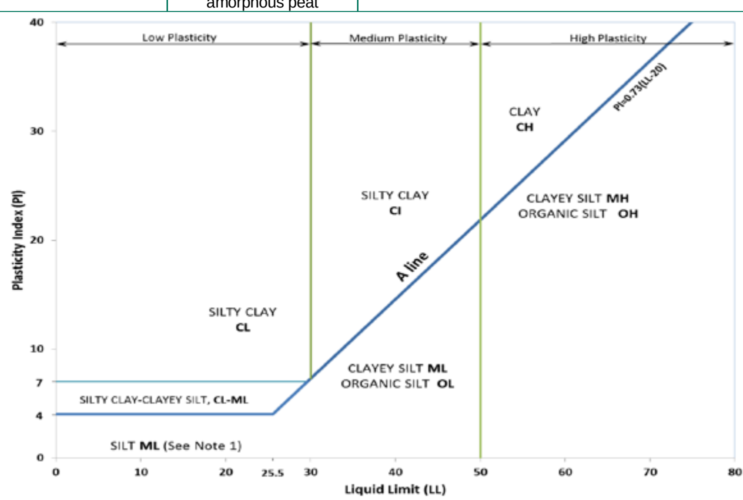
HWY. 401	PROJECT NO. 10-1111-0211		DIST.
SUBM'D. TVA	CHKD. TVA	DATE: Feb-2013	SITE:
DRAWN: DD	CHKD. KJB	APPD. TG	DWG. R1-4

METHOD OF SOIL CLASSIFICATION

The Golder Associates Ltd. Soil Classification System is based on the Unified Soil Classification System (USCS)

Organic or Inorganic	Soil Group	Type of Soil		Gradation or Plasticity	$Cu = \frac{D_{60}}{D_{10}}$	$Cc = \frac{(D_{30})^2}{D_{10} x D_{60}}$	Organic Content	USCS Group Symbol	Group Name
INORGANIC (Organic Content ≤30% by mass)	COARSE-GRAINED SOILS (>50% by mass is larger than 0.075 mm)	GRAVELS (>50% by mass of coarse fraction is larger than 4.75 mm)	Gravels with ≤12% fines (by mass)	Poorly Graded	<4	≤1 or ≥3	≤30%	GP	GRAVEL
				Well Graded	≥4	1 to 3		GW	GRAVEL
			Gravels with >12% fines (by mass)	Below A Line	n/a			GM	SILTY GRAVEL
				Above A Line	n/a			GC	CLAYEY GRAVEL
		SANDS (≥50% by mass of coarse fraction is smaller than 4.75 mm)	Sands with ≤12% fines (by mass)	Poorly Graded	<6	≤1 or ≥3		SP	SAND
				Well Graded	≥6	1 to 3		SW	SAND
			Sands with >12% fines (by mass)	Below A Line	n/a			SM	SILTY SAND
				Above A Line	n/a			SC	CLAYEY SAND

Organic or Inorganic	Soil Group	Type of Soil	Laboratory Tests	Field Indicators: Dilatancy	Field Indicators: Dry Strength	Field Indicators: Shine Test	Field Indicators: Thread Diameter	Field Indicators: Toughness (of 3 mm thread)	Organic Content	USCS Group Symbol	Primary Name
INORGANIC (Organic Content ≤30% by mass)	FINE-GRAINED SOILS (≥50% by mass is smaller than 0.075 mm)	SILTS (Non-Plastic or PI and LL plot below A-Line on Plasticity Chart below)	Liquid Limit <50	Rapid	None	None	>6 mm	N/A (can't roll 3 mm thread)	<5%	ML	SILT
				Slow	None to Low	Dull	3mm to 6 mm	None to low	<5%	ML	CLAYEY SILT
				Slow to very slow	Low to medium	Dull to slight	3mm to 6 mm	Low	5% to 30%	OL	ORGANIC SILT
			Liquid Limit ≥50	Slow to very slow	Low to medium	Slight	3mm to 6 mm	Low to medium	<5%	MH	CLAYEY SILT
				None	Medium to high	Dull to slight	1 mm to 3 mm	Medium to high	5% to 30%	OH	ORGANIC SILT
		CLAYS (PI and LL plot above A-Line on Plasticity Chart below)	Liquid Limit <30	None	Low to medium	Slight to shiny	~ 3 mm	Low to medium	0% to 30% **(see Note 2)**	CL	SILTY CLAY
			Liquid Limit 30 to 50	None	Medium to high	Slight to shiny	1 mm to 3 mm	Medium		CI	SILTY CLAY
			Liquid Limit ≥50	None	High	Shiny	<1 mm	High		CH	CLAY
HIGHLY ORGANIC SOILS (Organic Content >30% by mass)		Peat and mineral soil mixtures							30% to 75%	PT	SILTY PEAT, SANDY PEAT
		Predominantly peat, may contain some mineral soil, fibrous or amorphous peat							75% to 100%		PEAT



Note 1 – Fine grained materials with PI and LL that plot in this area are named (ML) SILT with slight plasticity. Fine-grained materials which are non-plastic (i.e. a PL cannot be measured) are named SILT.
Note 2 – For soils with <5% organic content, include the descriptor "trace organics" for soils with between 5% and 30% organic content include the prefix "organic" before the Primary name.

Dual Symbol — A dual symbol is two symbols separated by a hyphen, for example, GP-GM, SW-SC and CL-ML.
For non-cohesive soils, the dual symbols must be used when the soil has between 5% and 12% fines (i.e. to identify transitional material between "clean" and "dirty" sand or gravel.
For cohesive soils, the dual symbol must be used when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart (see Plasticity Chart at left).

Borderline Symbol — A borderline symbol is two symbols separated by a slash, for example, CL/CI, GM/SM, CL/ML.
A borderline symbol should be used to indicate that the soil has been identified as having properties that are on the transition between similar materials. In addition, a borderline symbol may be used to indicate a range of similar soil types within a stratum.

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

PARTICLE SIZES OF CONSTITUENTS

Soil Constituent	Particle Size Description	Millimetres	Inches (US Std. Sieve Size)
BOULDERS	Not Applicable	>300	>12
COBBLES	Not Applicable	75 to 300	3 to 12
GRAVEL	Coarse Fine	19 to 75 4.75 to 19	0.75 to 3 (4) to 0.75
SAND	Coarse Medium Fine	2.00 to 4.75 0.425 to 2.00 0.075 to 0.425	(10) to (4) (40) to (10) (200) to (40)
SILT/CLAY	Classified by plasticity	<0.075	< (200)

MODIFIERS FOR SECONDARY AND MINOR CONSTITUENTS

Percentage by Mass	Modifier
>35	Use 'and' to combine major constituents (*i.e.,* SAND and GRAVEL, SAND and CLAY)
> 12 to 35	Primary soil name prefixed with "gravelly, sandy, SILTY, CLAYEY" as applicable
> 5 to 12	some
≤ 5	trace

PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split-spoon sampler for a distance of 300 mm (12 in.).

Cone Penetration Test (CPT)
An electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (q_t), porewater pressure (u) and sleeve frictions are recorded electronically at 25 mm penetration intervals.

Dynamic Cone Penetration Resistance (DCPT); N_d:
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure
PM: Sampler advanced by manual pressure
WH: Sampler advanced by static weight of hammer
WR: Sampler advanced by weight of sampler and rod

SAMPLES

AS	Auger sample
BS	Block sample
CS	Chunk sample
DO or DP	Seamless open ended, driven or pushed tube sampler – note size
DS	Denison type sample
FS	Foil sample
GS	Grab Sample
RC	Rock core
SC	Soil core
SS	Split spoon sampler – note size
ST	Slotted tube
TO	Thin-walled, open – note size
TP	Thin-walled, piston – note size
WS	Wash sample

SOIL TESTS

w	water content
PL , w_p	plastic limit
LL , w_L	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, Gs)
DS	direct shear test
GS	specific gravity
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V (FV)	field vane (LV-laboratory vane test)
γ	unit weight

1. Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

NON-COHESIVE (COHESIONLESS) SOILS

Compactness[2]

Term	SPT 'N' (blows/0.3m)[1]
Very Loose	0 - 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	>50

1. SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects.
2. Definition of compactness descriptions based on SPT 'N' ranges from Terzaghi and Peck (1967) and correspond to typical average N_{60} values.

Field Moisture Condition

Term	Description
Dry	Soil flows freely through fingers.
Moist	Soils are darker than in the dry condition and may feel cool.
Wet	As moist, but with free water forming on hands when handled.

COHESIVE SOILS

Consistency

Term	Undrained Shear Strength (kPa)	SPT 'N'[1,2] (blows/0.3m)
Very Soft	<12	0 to 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	>200	>30

1. SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.
2. SPT 'N' values should be considered ONLY an approximate guide to consistency; for sensitive clays (e.g., Champlain Sea clays), the N-value approximation for consistency terms does NOT apply. Rely on direct measurement of undrained shear strength or other manual observations.

Water Content

Term	Description
w < PL	Material is estimated to be drier than the Plastic Limit.
w ~ PL	Material is estimated to be close to the Plastic Limit.
w > PL	Material is estimated to be wetter than the Plastic Limit.

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π	3.1416
ln x	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\,\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

* Density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)

(a) Index Properties (continued)

w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity index $= (w_l - w_p)$
w_s	shrinkage limit
I_L	liquidity index $= (w - w_p) / I_p$
I_C	consistency index $= (w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_α	secondary compression index
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
c_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction $= \tan\delta$
c′	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p′	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

Notes: 1 $\tau = c' + \sigma' \tan\phi'$
2 shear strength = (compressive strength)/2

Golder Associates

RECORD OF BOREHOLE No FC-6

SHEET 1 OF 2 — METRIC

PROJECT	10-1111-0211		
G.W.P.	2150-01-00	LOCATION	N 4830817.2 ;E 287306.2
DIST		HWY 401	
BOREHOLE TYPE	108 mm I.D. Hollow Stem Augers, NW Casing with Tricone, Wash Boring		
DATUM	Geodetic	DATE	May 13 and 14, 2012
ORIGINATED BY	SB	COMPILED BY	CC/TVA
CHECKED BY	KJB		

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
168.3	GROUND SURFACE				
0.0	ASPHALT				
0.2	Sand and gravel (FILL) Brown Moist				
167.5 / 0.8	Clayey silt, trace to some gravel, trace sand (FILL) Firm to very stiff Brown Moist	1	SS	18	
		2	SS	20	
		3	SS	17	
		4	SS	22	
		5	SS	10	
		6	SS	18	
		7	SS	5	
161.1 / 7.2	CLAYEY SILT, some sand, trace to some gravel, containing sandy silt interlayers Stiff Grey Moist	8	SS	9	7 25 55 13
		9	SS	8	
158.1 / 10.2	Silty SAND, trace to some gravel, trace to some clay (TILL) Dense Grey Wet	10	SS	40	10 59 22 9
156.6 / 11.7	SAND and GRAVEL, trace to some silt, trace clay, containing cobble (TILL) Very dense Grey Wet	11	SS	88	46 39 13 2
		12	SS	184	
154.1 / 14.2	END OF BOREHOLE				

Column headings: GROUND WATER CONDITIONS; ELEVATION SCALE (168–155); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED, 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³.

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity $\circ^{3\%}$ STRAIN AT FAILURE

PROJECT 10-1111-0211

RECORD OF BOREHOLE No FC-6

SHEET 2 OF 2

METRIC

G.W.P. 2150-01-00 LOCATION N 4830817.2 ;E 287306.2 ORIGINATED BY SB

DIST HWY 401 BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, NW Casing with Tricone, Wash Boring COMPILED BY CC/TVA

DATUM Geodetic DATE May 13 and 14, 2012 CHECKED BY KJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L / WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION --- CONTINUED FROM PREVIOUS PAGE ---	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	NOTE: 1. Given that Wash Boring techniques were used to advance the NW casing, the groundwater condition was not measured upon completion of drilling.										

$+^3, \times^3$: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

Golder Associates

Foundation Design

RECORD OF BOREHOLE No FC-10

SHEET 1 OF 1

METRIC

PROJECT 10-1111-0211

G.W.P. 2150-01-00

LOCATION N 4830774.0 ;E 287308.1

ORIGINATED BY SB

DIST HWY 401

BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers

COMPILED BY CC/TVA

DATUM Geodetic

DATE May 8, 2012

CHECKED BY KJB

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
164.1	GROUND SURFACE										
0.0	Clayey silt, trace to some sand, trace to some gravel, containing organics and rootlets (FILL) Soft Brown Moist	1	SS	WH		164					
163.4 / 0.7	CLAYEY SILT with SAND, trace gravel (TILL) Firm to very stiff Brown and grey Moist	2	SS	16		163					
		3	SS	19		162					
		4	SS	9				5	30	48	17
		5	SS	7		161					
		6	SS	10		160					
		7	SS	6		159		3	33	44	20
158.9 / 5.2	END OF BOREHOLE										

NOTE:

1. Open borehole dry upon completion of drilling.

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%)

$+^3, \times^3$: Numbers refer to Sensitivity ○ $^{3\%}$ STRAIN AT FAILURE

GTA-MTO 001 1011110211.GPJ GAL-GTA.GDT 2/8/13

Golder Associates

Foundation Design

RECORD OF BOREHOLE No FC-11

SHEET 1 OF 1

METRIC

PROJECT 10-1111-0211

G.W.P. 2150-01-00

LOCATION N 4830795.8 ;E 287352.4

ORIGINATED BY SB

DIST ______ HWY 401

BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, NW Casing, Wash Boring

COMPILED BY CC/TVA

DATUM Geodetic

DATE May 3 and 4, 2012

CHECKED BY KJB

ELEV / DEPTH (m)	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
164.4	GROUND SURFACE							
0.0	Sandy silt, trace to some clay, trace gravel, containing organics, rootlets, and wood fragments (FILL) Loose to compact Brown and grey Moist	1	SS	7				
		2	SS	14				
162.9 / 1.5	Clayey silt, some sand, trace to some gravel (FILL) Stiff Brown Moist	3	SS	13				
162.2 / 2.2	CLAYEY SILT with SAND, trace to some gravel Very soft to firm Grey Moist	4	SS	4	6	32	48	14
		5	SS	3				
		6	SS	WH	7	31	44	18
159.9 / 4.5	Silty SAND, trace gravel Compact Grey Wet	7	SS	27				
158.8 / 5.6	Silty SAND and GRAVEL, some silt, trace to some clay, contains sand and gravel layers (TILL) Very dense Grey Wet	8	SS	57	32	36	26	6
		9	SS	100/0.1				
		10	SS	81	49	35	13	3
		11	SS	61				
		12	SS	126				
151.8 / 12.7	END OF BOREHOLE							

Ground water conditions: ▽ ** (artesian)

** Artesian Conditions - see Note 1.

NOTE:

1. Water flowing from top of casing which was 1.5 m above ground surface (Elev. 165.9 m) when advanced to a depth of 12.2 m below ground surface (Elev. 152.2 m).

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (164 to 152); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20, 40, 60, 80, 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (10, 20, 30); UNIT WEIGHT γ kN/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

$+^3, \times^3$: Numbers refer to Sensitivity $\circ^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No FC-12

SHEET 1 OF 1 **METRIC**

PROJECT 10-1111-0211
G.W.P. 2150-01-00
LOCATION N 4830778.0 ;E 287318.6
ORIGINATED BY SB
DIST HWY 401
BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, NW Casing with Tricone, Wash Boring
COMPILED BY CC/TVA
DATUM Geodetic
DATE May 6 and 8, 2012
CHECKED BY KJB

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
163.9	GROUND SURFACE							
0.0	Clayey silt, trace to some sand, trace to some gravel, containing organics and rootlets (FILL) Soft to very stiff Brown and grey Moist	1	SS	3				
		2	SS	23				
162.4 / 1.5	CLAYEY SILT, some sand, trace to some gravel (TILL) Very stiff to hard Brown and grey Moist to wet	3	SS	20	4	22	58	16
		4	SS	32				
		5	SS	37				
		6	SS	21				
		7	SS	24				
158.3 / 5.6	SAND and SILT, some gravel, trace to some clay (TILL) Very dense Grey Wet	8	SS	104	17	46	30	7
157.3 / 6.6	END OF BOREHOLE							

** Artesian Conditions - see Note 2.

NOTES:

1. Water level inside casing at a depth of 2.7 m below ground surface (Elev. 161.2 m) at start of work day on May 8, 2012, when bottom of casing at a depth of 3.2 m below ground surface (Elev. 160.7 m).

2. Water flowing from top of casing which was 1.2 m above ground surface (Elev. 165.1 m) when advanced to a depth of 6.0 m below ground surface (Elev. 157.9 m).

Chart headings: SOIL PROFILE; SAMPLES; STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (163, 162, 161, 160, 159, 158); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%)

+3, ×3: Numbers refer to Sensitivity ○3% STRAIN AT FAILURE

Ontario | Ministry of the Environment and Climate Change

Well Tag No. *(Place Sticker and/or Print Below)*

Tag#:A 208617

Well Record

Regulation 903 Ontario Water Resources Act

Page ____ of ____

Measurements recorded in: ☐ Metric ☐ Imperial

Well Owner's Information

First Name	Last Name / Organization	E-mail Address	☐ Well Constructed by Well Owner
Ontario Ministry	of Transportation		

Mailing Address (Street Number/Name)	Municipality	Province	Postal Code	Telephone No. (inc. area code)
77 Wellesley St. West	Toronto	Ont	M7A 1Z8	

Well Location

Address of Well Location (Street Number/Name)	Township	Lot	Concession
HWY 401 (South Side)			

County/District/Municipality	City/Town/Village	Province	Postal Code
Peel	Mississauga	Ontario	

UTM Coordinates	Zone	Easting	Northing	Municipal Plan and Sublot Number	Other
NAD 83	17	603553	4830129		

Overburden and Bedrock Materials/Abandonment Sealing Record (see instructions on the back of this form)

General Colour	Most Common Material	Other Materials	General Description	Depth (m/ft) From	To
Grey	Fill			0	1
Brown	Sandy Clay			1	3.7
Grey	Clay	Gravel		3.7	6.7
Grey	Gravel			6.7	7.2
Grey	Clay	Gravel		7.2	9.8
Grey	Sand	Gravel		9.8	12.2

Annular Space

Depth Set at (m/ft) From	To	Type of Sealant Used (Material and Type)	Volume Placed (m³/ft³)
0	5.8	Holeplug Concrete	
5.8	6.1	Holeplug	
6.1	7	Silica Sand	
7	11.3	Holeplug	
11.3	12.2	Silica Sand	

Results of Well Yield Testing

After test of well yield, water was:
☐ Clear and sand free
☐ Other, specify ____

If pumping discontinued, give reason:

Pump intake set at (m/ft)

Pumping rate (l/min / GPM)

Duration of pumping ____ hrs + ____ min

Final water level end of pumping (m/ft)

If flowing give rate (l/min / GPM)

Recommended pump depth (m/ft)

Recommended pump rate (l/min / GPM)

Well production (l/min / GPM)

Disinfected? ☐ Yes ☐ No

Draw Down Time (min)	Water Level (m/ft)	Recovery Time (min)	Water Level (m/ft)
Static Level			
1		1	
2		2	
3		3	
4		4	
5		5	
10		10	
15		15	
20		20	
25		25	
30		30	
40		40	
50		50	
60		60	

Method of Construction

☐ Cable Tool ☐ Diamond
☒ Rotary (Conventional) ☐ Jetting
☐ Rotary (Reverse) ☐ Driving
☐ Boring ☐ Digging
☐ Air percussion
☐ Other, specify ____

Well Use

☐ Public ☐ Commercial ☐ Not used
☐ Domestic ☐ Municipal ☐ Dewatering
☐ Livestock ☐ Test Hole ☒ Monitoring
☐ Irrigation ☐ Cooling & Air Conditioning
☐ Industrial
☐ Other, specify ____

Construction Record - Casing

Inside Diameter (cm/in)	Open Hole OR Material (Galvanized, Fibreglass, Concrete, Plastic, Steel)	Wall Thickness (cm/in)	Depth (m/ft) From	To
3.75	Plastic	0.635	+1	11.3
3.75	Plastic	0.635	+1	6.1

Construction Record - Screen

Outside Diameter (cm/in)	Material (Plastic, Galvanized, Steel)	Slot No.	Depth (m/ft) From	To
3.75	Plastic	10	11.3	12.2
3.75	Plastic	10	6.1	7

Status of Well

☐ Water Supply
☐ Replacement Well
☐ Test Hole
☐ Recharge Well
☐ Dewatering Well
☒ Observation and/or Monitoring Hole
☐ Alteration (Construction)
☐ Abandoned, Insufficient Supply
☐ Abandoned, Poor Water Quality
☐ Abandoned, other, specify ____
☐ Other, specify ____

Water Details

Water found at Depth 6.7-7.2 (m/ft) ☐ Gas | Kind of Water: ☐ Fresh ☒ Untested ☐ Other, specify ____

Water found at Depth 9.8-12.2 (m/ft) ☐ Gas | Kind of Water: ☐ Fresh ☒ Untested ☐ Other, specify ____

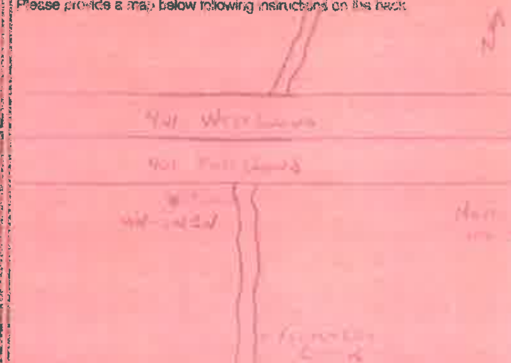
Water found at Depth ____ (m/ft) ☐ Gas | Kind of Water: ☐ Fresh ☐ Untested ☐ Other, specify ____

Hole Diameter

Depth (m/ft) From	To	Diameter (cm/in)
0	12.2	15

Map of Well Location

Please provide a map below following instructions on the back



Comments: [illegible] TSSA [illegible] Transmitters

Well Contractor and Well Technician Information

Business Name of Well Contractor	Well Contractor's Licence No.
Aquatech Dewatering	7341

Business Address (Street Number/Name)	Municipality
331 Rossland Road	Vaughan

Province	Postal Code	Business E-mail Address
Ont	L6A4P5	info@aquatd.com

Bus. Telephone No. (inc. area code)	Name of Well Technician (Last Name, First Name)
9059071700	[illegible], Michael

Well Technician's Licence No.	Signature of Technician and/or Contractor	Date Submitted
3436	[signature]	2016/08/27

Well owner's information package delivered	Date Package Delivered	Date Work Completed
☒ Yes ☐ No	2016/08/26	2016/08/19

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Audit No. Z233965

Received

0506E (2014/11)

Well Owner's Copy

APPENDIX C

Scope/Work Plan for Test Pile (TP1) Installation and Pile Load Test

Contract No.: 2015-2018

Full-Scale Pile Load Test at Hwy 401–Fletcher's Creek South Structure

Highway 401 Widening from Highway 403/410 Interchange to Credit River, City of Mississauga, Region of Peel

This document summarizes the scope of work for the Contractor to carry out a full-scale pile load test at or in the vicinity of the east abutment of the Highway 401 – Fletcher's Creek South Structure.

Scope/Work Plan

- The Contractor shall conduct a full-scale Pile Load Test in accordance with ASTM D1143 (Standard test Methods for Deep Foundations Under Static Axial Compressive Load) at the location of the east abutment of the Highway 401-Fletcher's Creek South Structure, using an HP310x110 pile. The static pile load test may be carried out in the vicinity of the east abutment (east of the proposed pile cap) at either the central or northern portion of the proposed structure abutment foundation. The Contractor, in conjunction with the Contract Administrator, shall determine the location of the test site in a manner that minimizes interference with the local traffic, current construction staging and pile driving activities on site.
- It is understood that a conventional diesel hammer will be used to conduct the Pile Load Test to allow for Hiley testing.
- The test pile shall be driven to Elevation 154.5 m (i.e. 2 m above the design pile tip elevation) and then the driving be monitored by employing the Hiley Dynamic Formula as per SS 103-11 until the appropriate pile "set" value is achieved prior to the Pile Load Test. Given the variable elevation of the "100-blow" soil, allowance shall be made for the test pile to be driven up to 2 m deeper (about Elevation 150.5 m) than the design tip elevation upon consultation with MTO Foundations Section and the foundations sub-consultant (Golder Associates Ltd.) to achieve the appropriate pile "set" value prior to the Pile Load Test.
- The Contractor shall carry out Pile Driving Analyzer (PDA) testing during the installation of the test pile in combination with the Hiley method to confirm the ultimate geotechnical resistance, beginning at Elevation 154.5 m (consistent with the start of Hiley testing) and thereafter at 0.5 m intervals of depth.
- The Contractor shall design and install a reaction system for the Pile Load Test that is capable of providing a maximum load of 2,600 kN in compression to the test pile.
- The Contractor shall be responsible for providing all the equipment for the Pile Load Test in accordance with ASTM D1143, including but not limited to the reference beam, loading jack, gauges, etc. The reference beam shall be installed on an independent system at a sufficient distance away from the Pile Load Test setup to prevent any interference from the Pile Load Test.
- The Contractor shall prepare a detailed drawing showing the Pile Load Test setup and the reaction system.
- The full-scale Pile Load Test shall be carried out not earlier than 30 days after the installation of the test pile and reaction piles.
- The Pile Load Test shall be continued to a load equivalent to 2,600 kN or failure load, whichever occurs first.

Justification

The results from the full-scale Pile Load Test will be compared to the results from PDA testing and Hiley testing to allow for optimization in correlations between the PDA and Hiley test results and actual, longer-term pile capacities in compression.

Based on the soil and groundwater conditions at this site (i.e., presence of artesian pressures), it is anticipated that the full-scale Pile Load Test conducted 30 days after pile installation will measure higher pile capacities in the vicinity of the east abutment. In addition, the full-scale pile load test will allow for higher geotechnical resistance factors and consequently higher geotechnical pile capacities. This will allow for optimization of the next stage of pile installation at the west abutment of the South Structure and both east and west abutments of the North Structure.

In addition, the results of the full-scale Pile Load Test will be used to study the development of geotechnical resistances with time in non-cohesive soils under artesian pressures, for applicability to other highway bridges in similar conditions.

APPENDIX D

Dynamic Analysis of Piles, Contract No. 2015-2018, Hwy 401 – Fletcher's Creek, South Structure, City of Mississauga, Ontario, Project No. BRM-00603982 – AO, Report Dated May 01, 2017, prepared by exp.

Dynamic Analysis of Piles
Contract No. 2015-2018
Hwy. 401 – Fletcher's Creek
South Structure
City of Mississauga, Ontario

Prepared For:

Anchor Shoring & Caissons Limited
3445 Kennedy Road
Toronto, ON M1V 4Y3

Attn: Mr. Dave Winter

Project Number

BRM-00603982-A0

Prepared By:

exp Services Inc.
1595 Clark Boulevard
Brampton, Ontario L6T 4V1
Telephone: (905) 793-9800
Facsimile: (905) 793-0641

Date Submitted
2017-01-05

Table of Contents

Tables

Appendices

1 Introduction

Exp Services Inc. (**exp**) was retained to carry out dynamic testing of piles for the proposed Hwy. 401- Fletcher's Creek South Structure (Contract No. 2015-2018) located in the City of Mississauga, Ontario. This report presents the results of the dynamic testing of piles carried out at the site on December 14, 2016.

On this date, one test pile (TP1) was monitored with the instrumentation from the Pile Driving Analyzer (PDA) attached. The pile tested was a HP310 mm x 110 kg/m steel HP-section and was monitored while being driven with a Delmag D19-42 diesel hammer. The manufacturer's maximum rated energy of the D19-42 hammer is ~66 kJ.

It is our understanding the piles at the South Structure are designed to support a factored axial resistance at ULS of 900 kN/pile.

The purpose of the dynamic testing was to evaluate the ultimate geotechnical resistance of the pile tested.

2 Fieldwork and Laboratory Analysis

On December 14, 2016, one test pile at the South Structure was initially driven to ~9.6 m depth below grade to an assumed Toe Elev. of ~154.5 m (upon completion of the testing, the actual toe elevation was determined to be ~154.4 m). The instrumentation from the Pile Driving Analyzer (PDA) was then attached and the pile driven for an additional approximately fifteen (15) hammer blows. The pile tested was a HP310 mm x 110 kg/m steel HP-section and was monitored while being driven with a Delmag D19-42 diesel hammer.

The dynamic monitoring was undertaken in general accordance with the ASTM D4945-12 procedures. The instrumentation for the Pile Driving Analyzer consisted of two reusable strain gauges and two accelerometers securely bolted on the pile. For each hammer blow, electronic signals were fed into the pre-programmed Pile Driving Analyzer (Model PAK/PAX) and the basic measurements of strain and acceleration were converted into force and velocity parameters as a function of time.

From the force and velocity parameters, the ultimate (mobilized) bearing capacities were automatically computed. In addition, the maximum compressive and tensile forces, the developed energies and the hammer blow rate, etc., are some of the output data for the Analyzer. The force and velocity traces were continually observed in the field and their digital signals were recorded and stored in memory.

A selected representative hammer blow from near the end of initial driving was used to perform CAPWAP (CAse Pile Wave Analysis Program) analysis in order to evaluate the capacity of the pile and the corresponding CASE damping factors.

The CAPWAP program is an iterative method to analyze the static resistance and resistance distribution along a pile with the dynamic measurements obtained from the Pile Driving Analyzer Testing. In the CAPWAP analysis, the program utilizes the fact that the force and velocity are related to each other by the pile impedance, which is readily calculable by:

$$Z = \frac{EA}{C}$$

where

Z	=	impedance of pile
E	=	modulus of elasticity of pile
A	=	cross-sectional area of pile
C	=	speed of stress wave in the pile

In the CAPWAP program, the pile is divided into a number of mass points and springs. The soil reaction forces on these mass points are assumed to consist of elastoplastic (static) and linear viscous (dynamic) components. In the analysis, a measured force was used as input and by varying the ultimate static resistance, resistance distribution, quake, elastic soil deformation, soil damping constants, etc., a computed force or velocity is calculated.

When a good match is obtained by varying the above components, the pile-soil interaction is modeled and a solution for the ultimate static resistance along the pile can be calculated. Based on this calculated resistance, an estimate of the frictional resistance can be obtained.

Static computations can then be used to predict the load versus deformation characteristics of the pile, which is often referred to as a "simulated load test".

3 Test Results

3.1 Pile Driving Analyzer

The pile tested was a HP310 mm x 110 kg/m steel HP-section and was monitored while being driven with a Delmag D19-42 diesel hammer. The manufacturer's maximum rated energy of the D19-42 hammer is ~66 kJ. The results obtained from the dynamic pile testing are presented in Table No. 1 and are summarized below.

The average energy transferred to the top of the pile during monitoring was ~30 kJ with the D19-42 hammer operating at a speed of ~38 blows per minute (bpm).

The average maximum force at the instrumentation location was ~3040 kN which corresponds to a maximum stress of ~216 MPa.

The evaluated ultimate geotechnical resistance at the end of initial driving of the pile tested was ~2200 kN. The reported set per hammer blow at the end of initial driving was ~5.8 mm (equivalent of ~4.3 blows per 25 mm penetration). The depth of the pile tested was ~9.6 m below grade (~Toe Elev. 154.4 m).

3.2 CAPWAP Analysis Results

CAPWAP analysis was undertaken on a selected representative hammer blow from near the end of initial driving of the test pile.

A summary of the results is presented in Table No. 2. The Case Method Capacities and Pile Profile and Model tables, the CAPWAP Force matches, Force-Velocity Wave forms, Resistance Distributions, Simulated Compression Load Test Curves, etc. are presented in Appendix A.

The evaluated ultimate geotechnical resistance at the end of initial driving of the test pile was ~2200 kN of which 375 kN is evaluated as skin resistance and 1825 kN evaluated as toe resistance.

4 Summary

In accordance with the Ontario and Canadian Highway Bridge Design Codes (OHBDC and CHBDC), the ultimate geotechnical pile resistance evaluated by analysis using dynamic monitoring results, multiplied by a resistance factor of 0.5, is required to exceed the design pile factored load at ULS. For the HP310 mm x 110 kg/m steel 'H-piles' at this site with a maximum design factored load at ULS of 900 kN, the ultimate geotechnical pile resistance should therefore exceed 1800 kN.

The evaluated ultimate geotechnical resistance at the end of initial driving of the pile tested was ~2200 kN which is greater than the 1800 kN (min.) required. The reported set per hammer blow at the end of initial driving was ~5.8 mm (equivalent of ~4.3 blows per 25 mm penetration) and the depth of the pile tested was ~9.6 m below grade (~Toe Elev. 154.4 m).

It should be noted the reported evaluated ultimate pile resistance is the resistance of the pile at the time of testing. Pile resistance can increase with time (set-up) and also, although less common, decrease with time (relaxation). The pile should be monitored during restrike, after a suitable waiting period, to assess any soil setup or relaxation impact on the evaluated ultimate pile resistance.

We trust that the information contained in this report is satisfactory. Should you have any questions, please do not hesitate to contact this office.

exp Services Inc.

A. D. Maini P. Eng.
Project Manager

Stephen S. M. Cheng P. Eng.
Discipline Manager Geotechnical Division

ADM/am/I:\2003-Brampton\Projects\Geotechnical Engineering\0600000\603000\603982-A0 Fletchers Creek South PDA\Fl Creek S Structure PDA Report.docx

Table 1 - Summary of Pile Driving Analyzer Test Results
December 14, 2016
Hwy. 401 – Fletcher's Creek South Structure

PILE NO.	EVENT	HAMMER	DEPTH BEL. GRADE	REPORTED PENETRATION RESISTANCE		TRANSFERRED ENERGY		FORCE		EVALUATED ULTIMATE GEOTECHNICAL RESISTANCE	REMARKS
			(m)	Blows/mm	Equiv. Blows/ 25 mm	Mean (kJ)	Speed (bpm)	Max. (kN)	Stress (MPa)	(kN)	
TP1 (Vert)	EOID	Delmag D19-42	~9.6	~1 / 5.8	~4.3	30	38	3040	216	~2200	Toe Elev. ~154.4 m

EOID – End Of Initial Driving
bpm – blows per minute

Table 2 - Summary of CAPWAP Analysis Results

Pile I.D.	Emb. Depth (m)	Quake (mm)		Damping				Evaluated Ult. (Mob.) Geotechnical Resistance (kN)			Reported Penetration Resistance (blows/mm)
				Case		Smith (sec/m)					
		Skin	Toe	Skin	Toe	Skin	Toe	Total	Skin	Toe	
TP1 EOID – Dec. 14	~9.6	2.5	9.5	0.36	0.44	0.55	0.14	2200	375	1825	~1 / 5.8

EOID – End Of Initial Driving

Appendix A:
CAPWAP Tables and Figures

Pile No. TP1
EOID
December 14, 2016

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 2200.0; along Shaft 375.0; at Toe 1825.0 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m
				2200.0				
1	3.0	1.5	49.2	2150.8	49.2	32.80	26.54	0.545
2	5.0	3.5	54.1	2096.7	103.3	27.05	21.89	0.545
3	7.0	5.5	59.0	2037.7	162.3	29.50	23.87	0.545
4	9.0	7.5	88.4	1949.3	250.7	44.20	35.76	0.545
5	11.0	9.5	124.3	1825.0	375.0	62.15	50.28	0.545
Avg. Shaft			75.0			39.47	31.94	0.545
Toe			1825.0				19113.95	0.136

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(mm)	2.500	9.500
Case Damping Factor		0.359	0.436
Damping Type			Smith
Unloading Quake	(% of loading quake)	40	42
Reloading Level	(% of Ru)	100	100
Soil Plug Weight	(kN)		0.33

CAPWAP match quality = 3.19 (Wave Up Match) ; RSA = 0
Observed: final set = 5.800 mm; blow count = 172 b/m
Computed: final set = 5.804 mm; blow count = 172 b/m

max. Top Comp. Stress = 203.1 MPa (T= 20.5 ms, max= 1.028 x Top)
max. Comp. Stress = 208.8 MPa (Z= 3.0 m, T= 20.9 ms)
max. Tens. Stress = -27.63 MPa (Z= 5.0 m, T= 38.1 ms)
max. Energy (EMX) = 28.01 kJ; max. Measured Top Displ. (DMX)=16.23 mm

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress MPa	max. Tens. Stress MPa	max. Trnsfd. Energy kJ	max. Veloc. m/s	max. Displ. mm
1	1.0	2864.2	-272.6	203.1	-19.34	28.01	4.7	15.792
2	2.0	2905.2	-328.4	206.0	-23.29	27.46	4.6	15.184
3	3.0	2944.4	-371.3	208.8	-26.34	26.96	4.5	14.598
4	4.0	2820.8	-353.2	200.1	-25.05	24.74	4.5	14.027
5	5.0	2864.9	-389.6	203.2	-27.63	24.21	4.4	13.428
6	6.0	2732.9	-353.8	193.8	-25.09	21.86	4.3	12.825
7	7.0	2792.0	-355.6	198.0	-25.22	21.29	4.2	12.202
8	8.0	2632.3	-291.5	186.7	-20.68	18.88	4.5	11.583
9	9.0	2492.0	-295.2	176.7	-20.94	18.33	5.3	10.963
10	10.0	2095.6	-194.4	148.6	-13.79	15.16	5.7	10.355
11	11.0	2164.3	-204.3	153.5	-14.49	12.38	5.6	9.710
Absolute	3.0			208.8			(T =	20.9 ms)
	5.0				-27.63		(T =	38.1 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	2324.9	1999.6	1674.3	1349.0	1023.7	698.5	373.2	47.9	0.0	0.0
RX	2570.0	2487.0	2404.0	2321.1	2238.1	2155.1	2072.1	2026.1	2024.5	2022.9
RU	2324.9	1999.6	1674.3	1349.0	1023.7	698.5	373.2	47.9	0.0	0.0

RAU = 1842.5 (kN); RA2 = 2217.7 (kN)

Current CAPWAP Ru = 2200.0 (kN); Corresponding J(RP)= 0.04; J(RX) = 0.45

VMX m/s	TVP ms	VT1*Z kN	FT1 kN	FMX kN	DMX mm	DFN mm	SET mm	EMX kJ	QUS kN
4.66	20.50	2653.7	2924.2	2924.2	16.235	5.804	5.800	28.8	2609.7

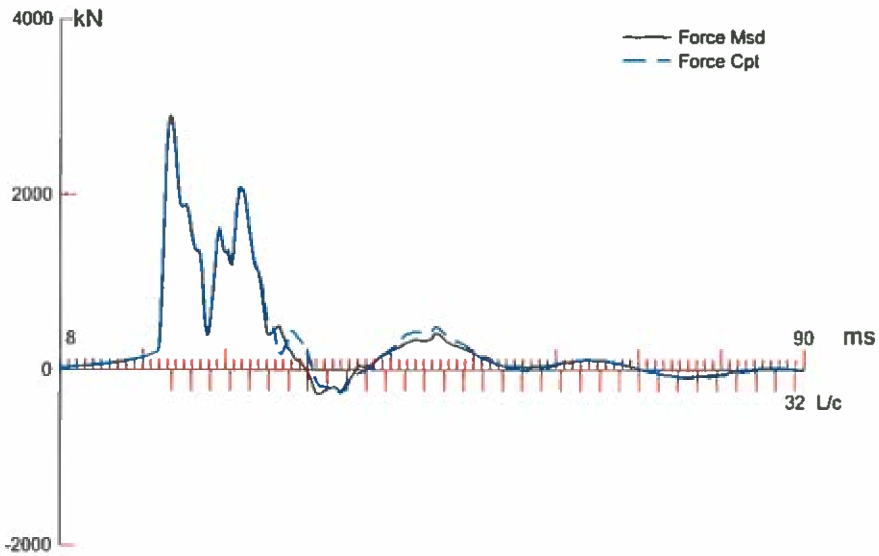
PILE PROFILE AND PILE MODEL

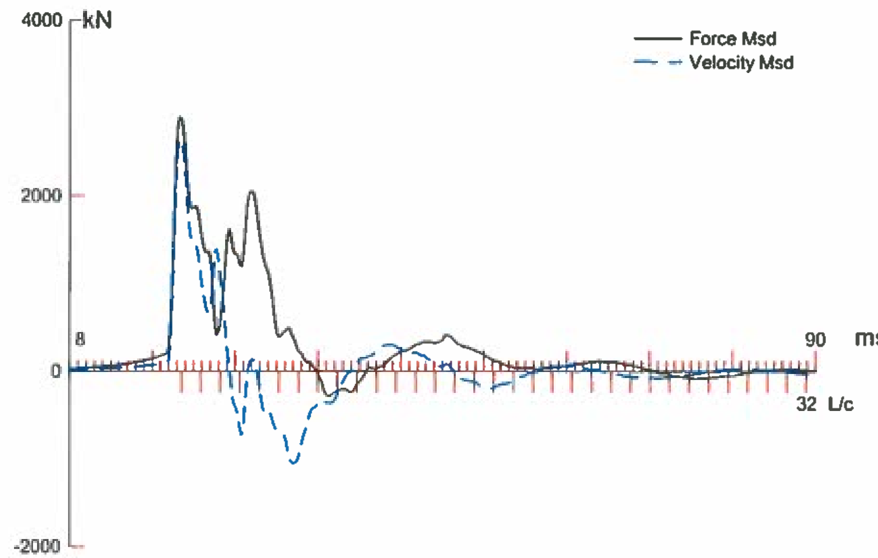
Depth m	Area cm^2	E-Modulus MPa	Spec. Weight kN/m^3	Perim. m
0.00	141.00	206842.7	77.287	1.236
11.00	141.00	206842.7	77.287	1.236

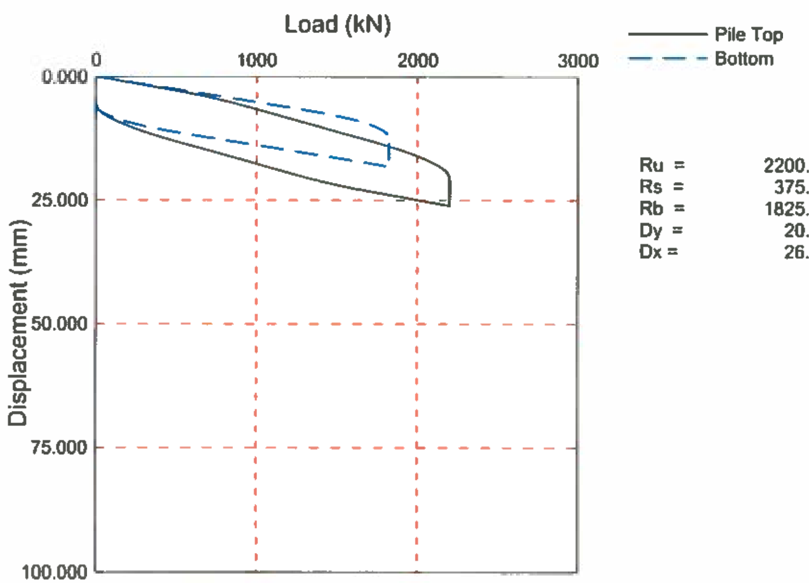
Toe Area 0.095 m^2

Segmnt Number	Dist. B.G. m	Impedance kN/m/s	Imped. Change %	Tension Slack mm	Tension Eff.	Compression Slack mm	Compression Eff.	Perim. m	Soil Plug kN
1	1.00	569.29	0.00	0.000	0.000	-0.000	0.000	1.236	0.00
2	2.00	569.29	0.00	0.000	0.000	-0.000	0.000	1.236	0.01
11	11.00	569.29	0.00	0.000	0.000	-0.000	0.000	1.236	0.01

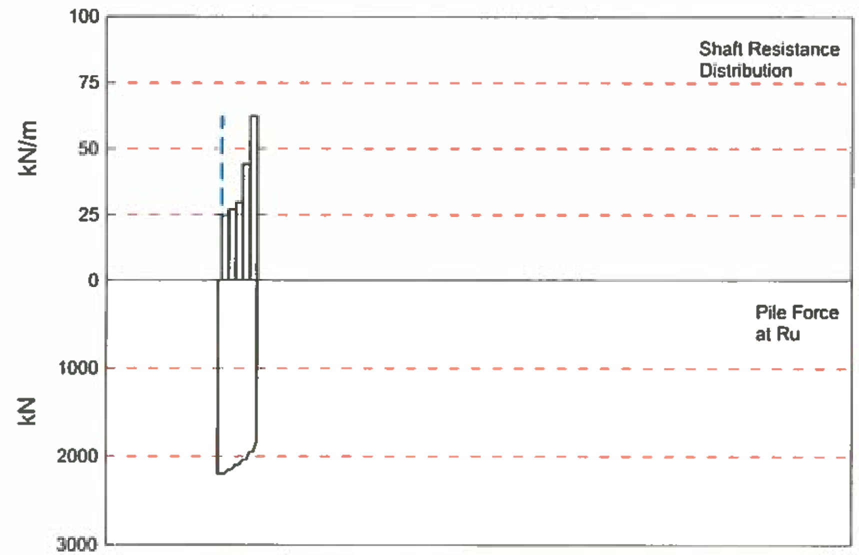
Pile Damping 1.0 %, Time Incr 0.195 ms, Wave Speed 5123.0 m/s, 2L/c 4.3 ms







Ru =	2200.0	kN
Rs =	375.0	kN
Rb =	1825.0	kN
Dy =	20.3	mm
Dx =	26.1	mm



APPENDIX E

Test Pile (TP1) Driving Record and Hiley Test Results Report Dated December 14, 2016, by MNA Engineering Ltd.

PILE ID: Test Pile	PILE DRIVING RECORD	DATE: December 14, 2016

PROJECT:	Hwy 401/Mavis	STRUCTURE	Fletcher's Creek Cukvert
CONTRACT NO.	2015-2018	STR LOC	South Abutment - Test Pile
LOCATION:	Fletcher's Creek Cukvert	PILE DETAIL	H 310 X110
GENERAL CONTRACTOR:	Dufferin Construction	HAMMER	D19-42
SUB CONTRACTOR:	Anchor Shoring	DRIVEN DIR	V

LENGTH IN GROUND (M)	PENETRATION BLOWS	LENGTH IN GROUND (M)	PENETRATION BLOWS	LENGTH IN GROUND (M)	PENETRATION BLOWS	LENGTH IN GROUND (M)	PENETRATION BLOWS	LENGTH IN GROUND (M)	PENETRATION BLOWS
0.25	5	5.25	4	10.25		15.25		20.25	
0.50	4	5.50	5	10.50		15.50		20.50	
0.75	4	5.75	5	10.75		15.75		20.75	
1.00	4	6.00	6	11.00		16.00		21.00	
1.25	4	6.25	14	11.25		16.25		21.25	
1.50	4	6.50	17	11.50		16.50		21.50	
1.75	4	6.75	17	11.75		16.75		21.75	
2.00	4	7.00	21	12.00		17.00		22.00	
2.25	4	7.25	19	12.25		17.25		22.25	
2.50	4	7.50	19	12.50		17.50		22.50	
2.75	4	7.75	20	12.75		17.75		22.75	
3.00	4	8.00	25	13.00		18.00		23.00	
3.25	3	8.25	34	13.25		18.25		23.25	
3.50	2	8.50	38	13.50		18.50		23.50	
3.75	2	8.75	36	13.75		18.75		23.75	
4.00	3	9.00	37	14.00		19.00		24.00	
4.25	3	9.25	36	14.25		19.25		24.25	
4.50	3	9.50	51	14.50		19.50		24.50	
4.75	3	9.75		14.75		19.75		24.75	
5.00	3	10.00		15.00		20.00		25.00	

DETAILS FOR FINAL 150mm OF PENETRATION	1	2	3	4	5	6
BLOWS PER 25mm						

	DATE	TIME	RESULT
GRAPH-1	14-Dec	14:00	Ok
GRAPH-2			
RETAB-1			

Length of pile	16.77
Cut off length	
Length below cut off	

CALCULATIONS

		GRAPH-1	GRAPH-2	RE-TAP
Pile length	L_p	16.77		
Rebound	C	12		
Penetration	S	5.8		
Eq-1	S+C/2	11.8		
Pile unit weight	w	110		
Pile weight -A	L_p*w	1844.7		
Block weight -B	W_b	605		
Pile+block weight ; P=A+B	P	2449.7		
RAM Weight	W_r	1820		
Restitution ; e	e	0.32		
Pe^2	Pe^2	251		
Eq-2	W_r+Pe^2	2,071		
Eq-3	W_r+P	4,270		
n=(Eq2) / (Eq3)	n	0.485		
Energy of hammer ft.lb	E	57600		
Eq-4	n x E x c_1	27937		
R = nE / (S+C/2)	Eq-4 / Eq-1	2368		

Prepared by : **Douglas Liu**

Test pile #1 SW Abut.

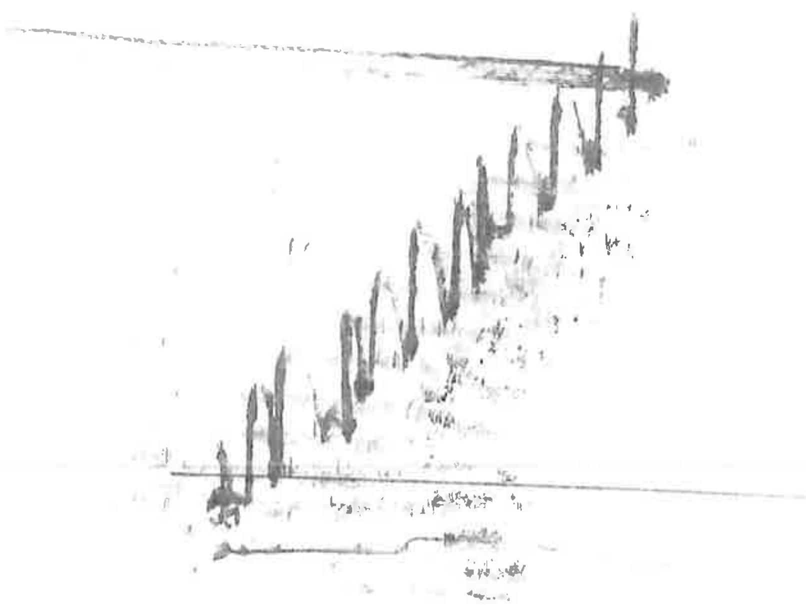
Dec 14, 2016

@ 2:00

$S = \frac{58}{10} = 5.8$

$C = 12$

$S + \frac{C}{2} = 11.8$



APPENDIX F

Memorandum: Full Scale Pile Load Testing, Fletcher Creek, Pile ID INC249, 401 and Mississauga Road –MTO Contract 2015-2018 Reference No. 17-1031-03, dated May 29, 2017, prepared by SNC–Lavalin GEM Ontario Inc.

SNC-Lavalin GEM Ontario Inc.
401 Hanlan Road
Vaughan, Ontario, Canada, L4L 3T1
905.851.0090 905.851.0091

May 29, 2017

MEMORANDUM – Our Ref. 17-1031-03

To: Muhammad Rashid
Senior Quality Control Administrator
Dufferin Construction Company, a Division of CRH Canada Group Inc.

From: Dylan Hill, P.Eng
Project Manager

Subject: Full Scale Pile Load Testing
Fletcher Creek, Pile ID INC249
401 and Mississauga Road –MTO Contract 2015-2018

As requested, SNC-Lavalin GEM Ontario Inc. ("SNCL") is pleased to present the following memorandum outlining the observations made during full scale pile load testing carried out at Fletcher Creek, beginning on May 16th and ending May 17th, 2017. It should be noted that this memo is a presentation of observations and factual results – no engineering interpretation of results was included in SNCL's scope of work.

1.1 Test Apparatus and Procedure

Testing was carried out on one (1) HP310x110 sized steel pile, identified as INC249, in general accordance with the requirements of ASTM D1143, however with the modified procedure as outlined by Golder Associates in Instruction Notice #249, which is enclosed with this memorandum. The pile was loaded with a 600 ton Dudgeon jack, which was placed on a 900 x 900 x 50 mm steel plate on the top of the pile. Four (4) Mitutoyo Corporation model 3428S-19 analogue dial gauges were placed on each corner of the plate in order to make displacement observations during loading.

Loading was carried out in six (6) increments of 400, 800, 1200, 1600, 2000 and 2300 kN, followed by a 12 hour creep test under a load of 2600 kN, and four (4) unloading increments of 1900, 1200, 500 and 0 kN. All loading increments were held for a minimum of 20 minutes, or until the rate of displacement was observed to fall below 0.25 mm per hour. Measurements were taken at 20 minute intervals for 60 minutes during all unloading increments, including one (1) additional measurement at 6 hours following complete unloading.

Figure 1 – Test setup during testing procedure

Secondary survey measurements were also recorded on the top of the loading plate and on several points on the reference beams by the Client's surveying staff, and provided to SNCL upon completion of the testing. Reference beam deflection was observed to be less than 1 mm throughout the testing procedure. Verification surveying of the reaction frame in order to ensure it remained plumb and stable was also carried out by the Client's surveying staff.

Figure 2 - Reaction frame, test area tarped during testing

General arrangement drawings of the reaction frame and reference beam construction are presented in Enclosure B at the end of this memorandum. Calibration certificates for the four (4) displacement gauges and the 600 t jack/hydraulic gauge which were provided to SNCL by Dufferin Construction Company (the “Client”) prior to the start of the testing are presented in Enclosure C at the end of this memorandum.

1.2 Results

A graphical presentation of loading versus displacement for the entire test sequence for the mean of all displacement gauge readings, as well as the secondary survey measurements are presented in Enclosure A at the end of this memorandum. As well, graphical presentations of load versus time for the loading and unloading phases, and load versus log-time for the creep phase of the testing are also presented, both for each individual gauge and the mean of the readings from all gauges. Gauges are identified as 1 through 4, at the north-west, north-east, south-west and south-east corners of the plate respectively.

1.3 Closure

We trust this meets your present requirements. Please do not hesitate to contact us if there are any questions or comments.

SNC-Lavalin Inc.

Enclosure A – Test Results- (7 pages)
Enclosure B – Test Apparatus Drawings (4 pages)
Enclosure C – Provided Calibration Certificates (6 pages)
Enclosure D – Modified Testing Procedure - Contract Instruction Notice #249 (3 pages)

N:\17 DocMgr\1001-1200\1031 CRH Canada (Dufferin Construction)\17-1031-03 - 401 and Mississauga Road Pile Testing\Reports\17-1031-03 - Mississauga Road Pile Testing - 05-29-2017.pdf

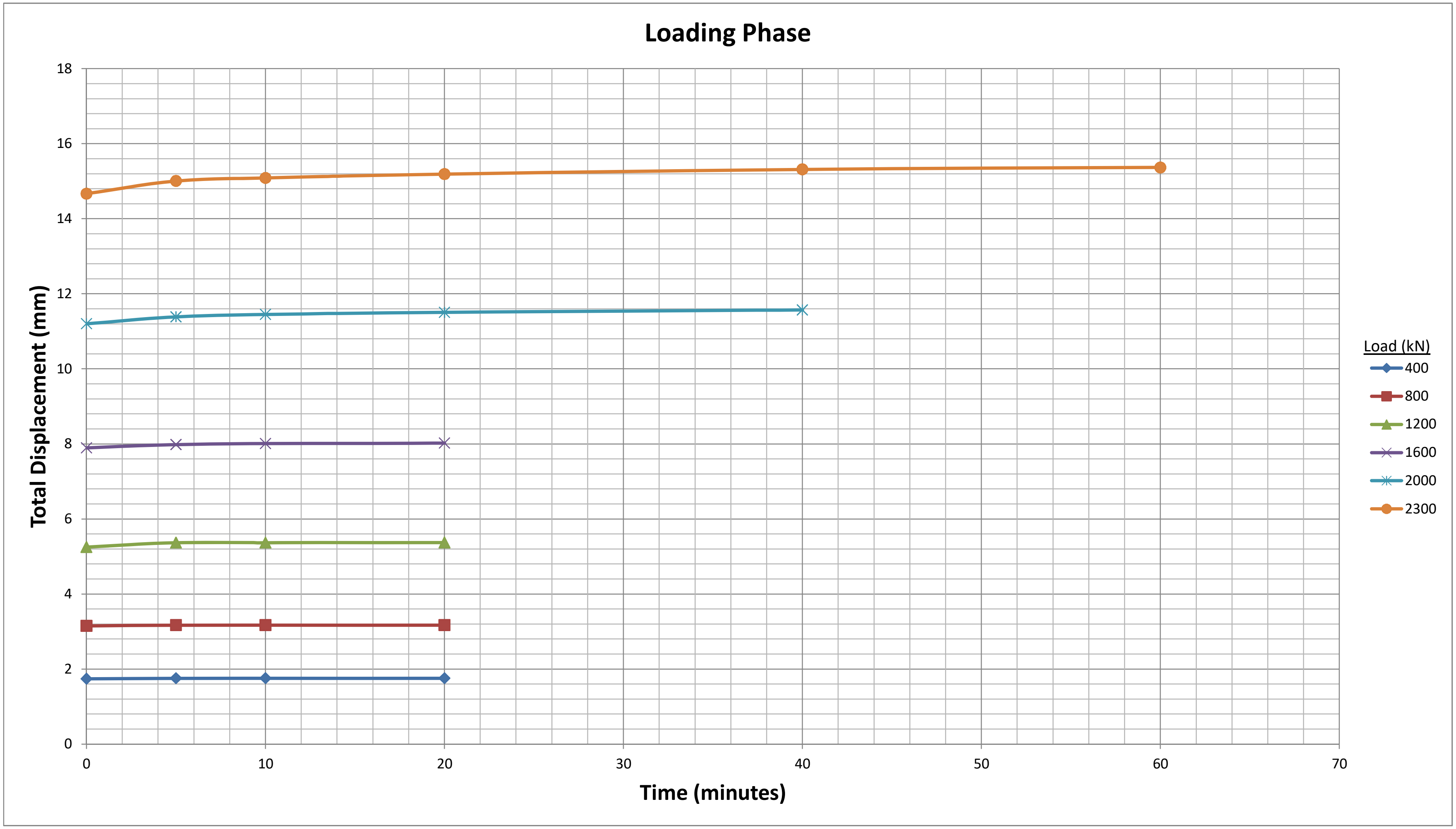
Enclosure A

Test Results (7 pages)

SNC·LAVALIN

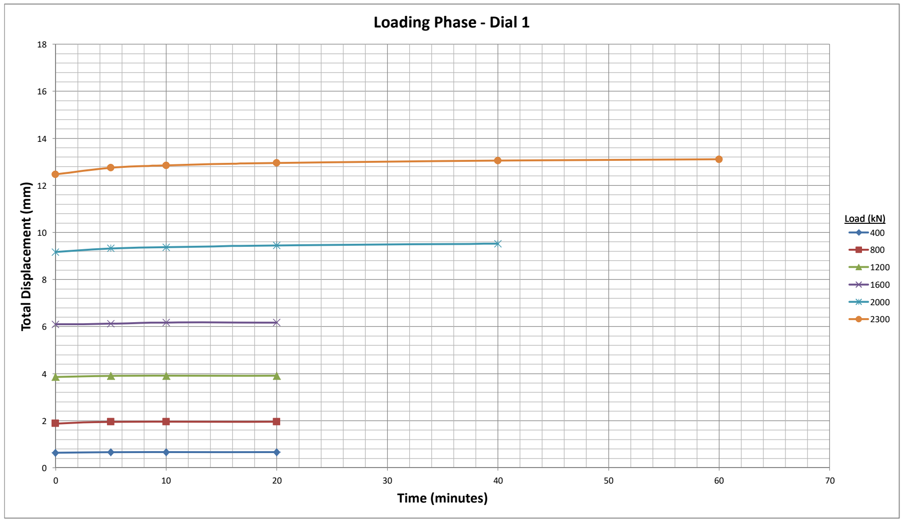
Combined Loading, Creep and Unloading Phases - Load vs. Displacement
Displacement (mm)
25
20
15
10
5
0
0
500
1000
1500
2000
2500
3000
Load (kN)
Secondary Measurements
Dial Gauges

SNC·LAVALIN

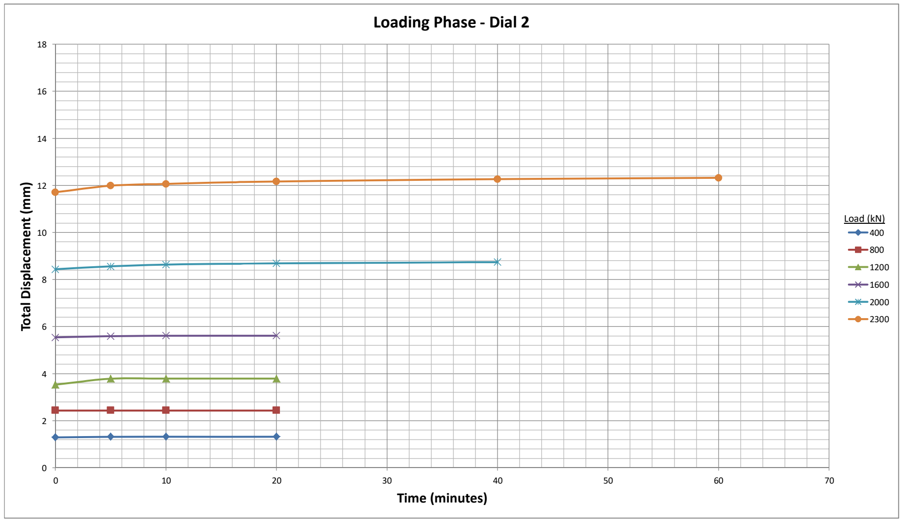


Loading Phase
Total Displacement (mm)
Time (minutes)
0
2
4
6
8
10
12
14
16
18
0
10
20
30
40
50
60
70
Load (kN)
400
800
1200
1600
2000
2300

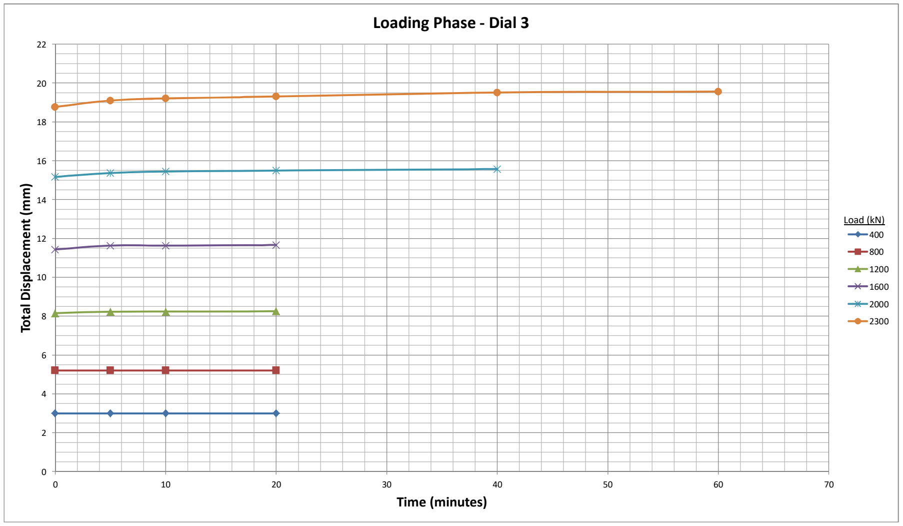
SNC·LAVALIN



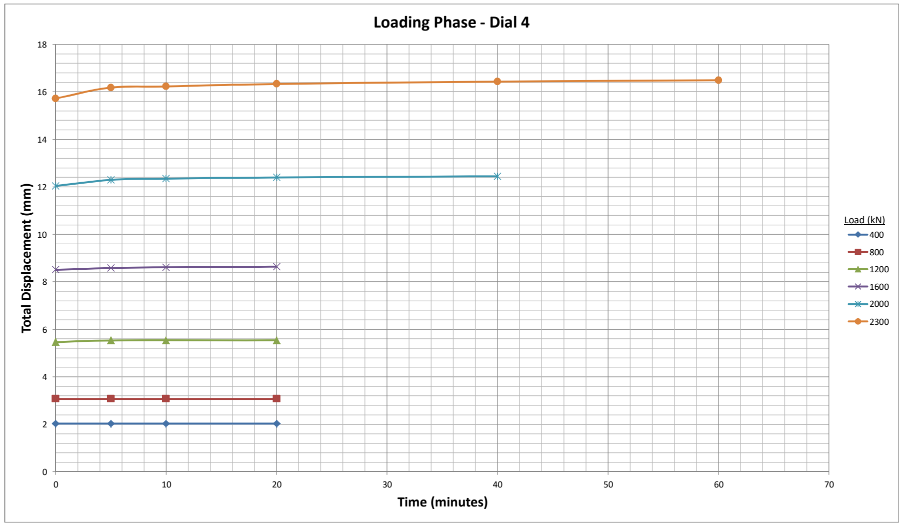
Loading Phase - Dial 1
Total Displacement (mm)
Time (minutes)
Load (kN)
400
800
1200
1600
2000
2300



Loading Phase - Dial 2
Total Displacement (mm)
Time (minutes)
Load (kN)
400
800
1200
1600
2000
2300

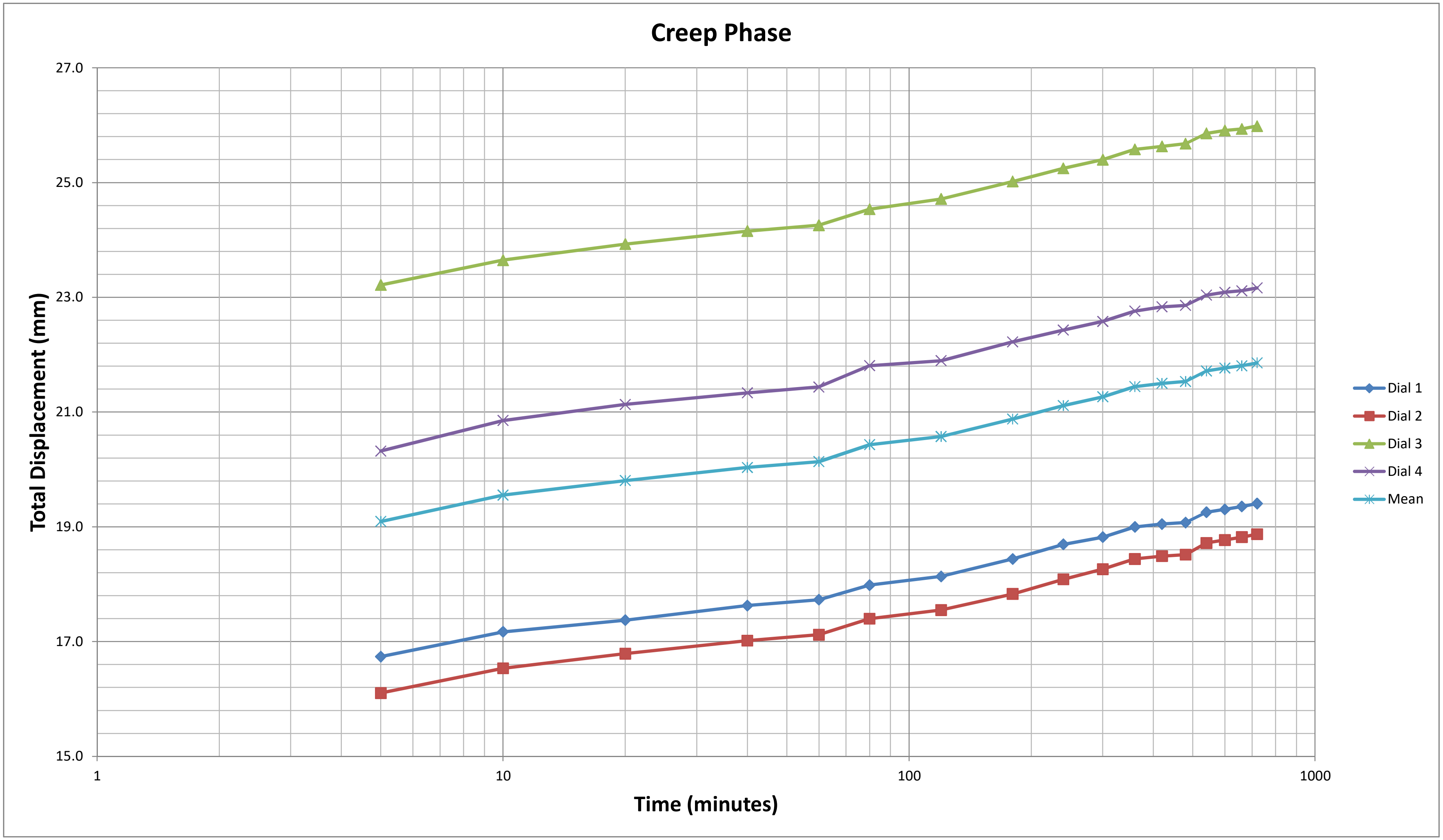


Loading Phase - Dial 3
Total Displacement (mm)
Time (minutes)
Load (kN)
400
800
1200
1600
2000
2300



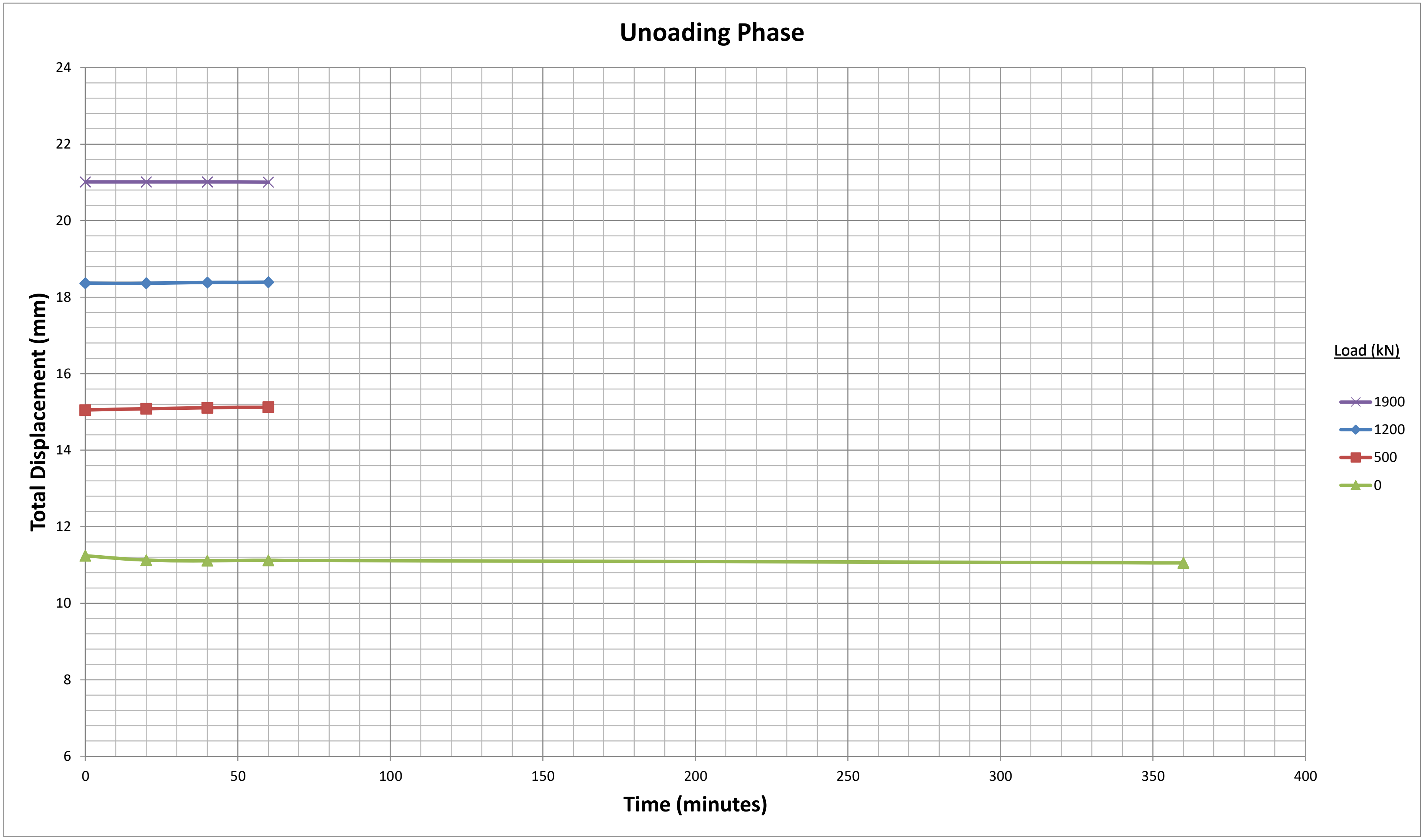
Loading Phase - Dial 4
Total Displacement (mm)
Time (minutes)
Load (kN)
400
800
1200
1600
2000
2300

SNC·LAVALIN



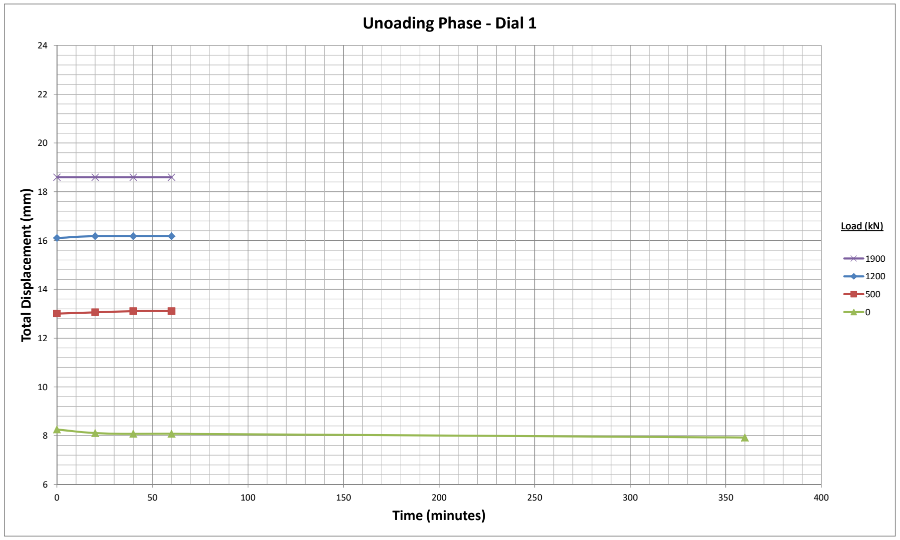
Creep Phase
27.0
25.0
23.0
21.0
19.0
17.0
15.0
Total Displacement (mm)
1
10
100
1000
Time (minutes)
Dial 1
Dial 2
Dial 3
Dial 4
Mean

SNC·LAVALIN

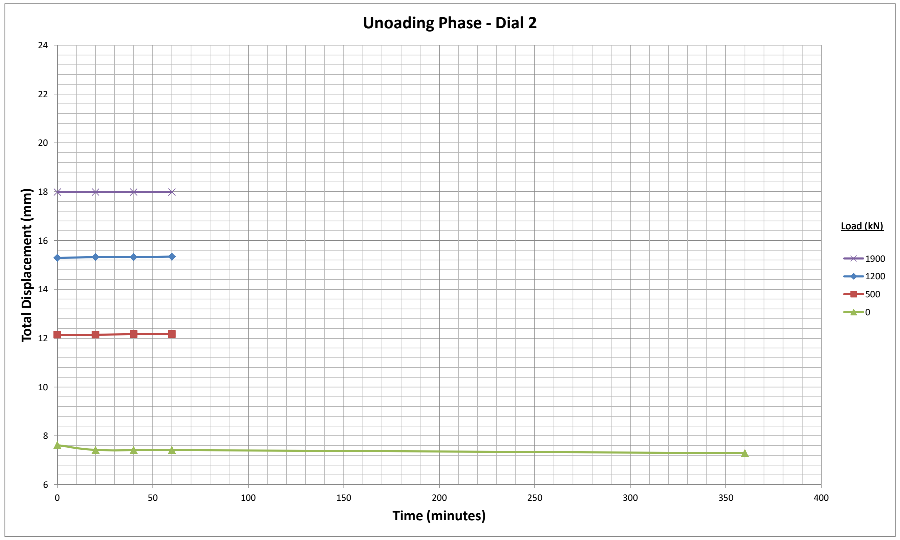


Unoading Phase
Total Displacement (mm)
24
22
20
18
16
14
12
10
8
6
0
50
100
150
200
250
300
350
400
Time (minutes)
Load (kN)
1900
1200
500
0

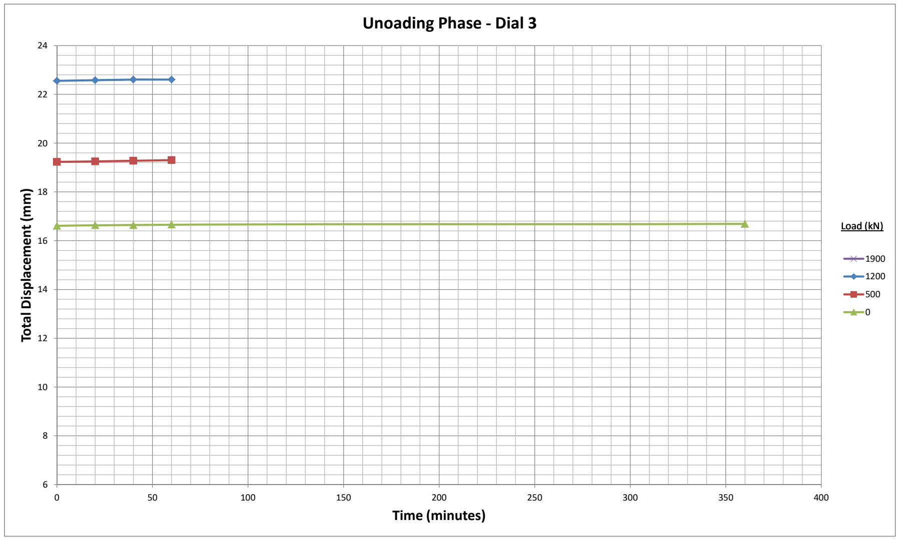
SNC·LAVALIN



Unoading Phase - Dial 1
Total Displacement (mm)
Time (minutes)
Load (kN)
1900
1200
500
0



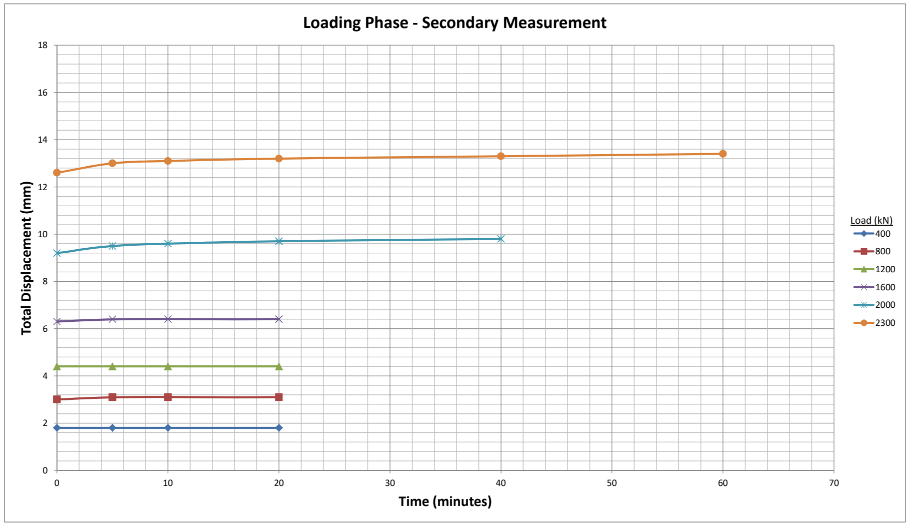
Unoading Phase - Dial 2
Total Displacement (mm)
Time (minutes)
Load (kN)
1900
1200
500
0



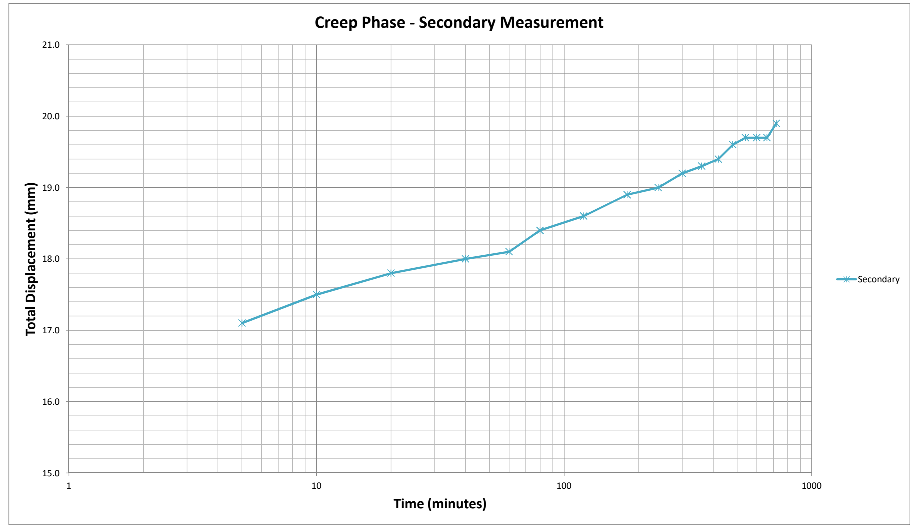
Unoading Phase - Dial 3
Total Displacement (mm)
Time (minutes)
Load (kN)
1900
1200
500
0

Unoading Phase - Dial 4
Total Displacement (mm)
Time (minutes)
Load (kN)
1900
1200
500
0

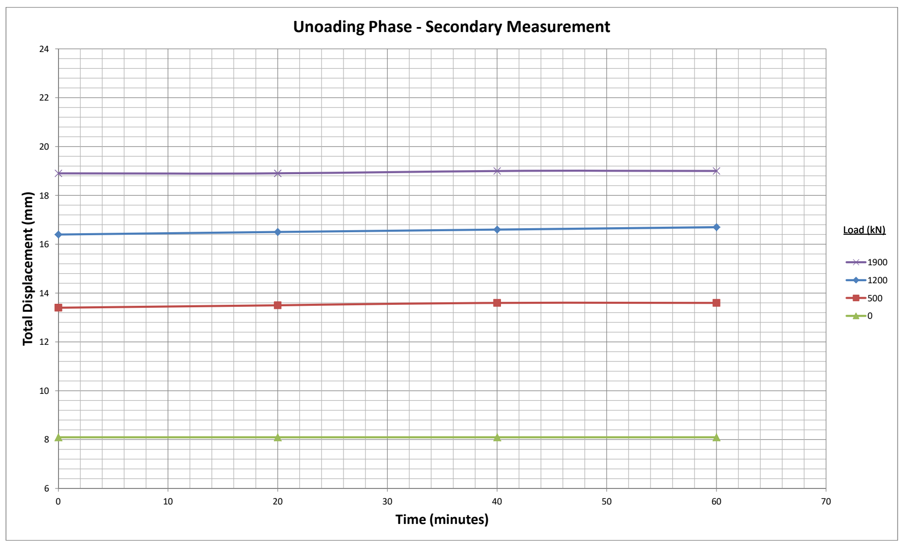
SNC·LAVALIN



Loading Phase - Secondary Measurement
Total Displacement (mm)
0
2
4
6
8
10
12
14
16
18
Time (minutes)
0
10
20
30
40
50
60
70
Load (kN)
400
800
1200
1600
2000
2300



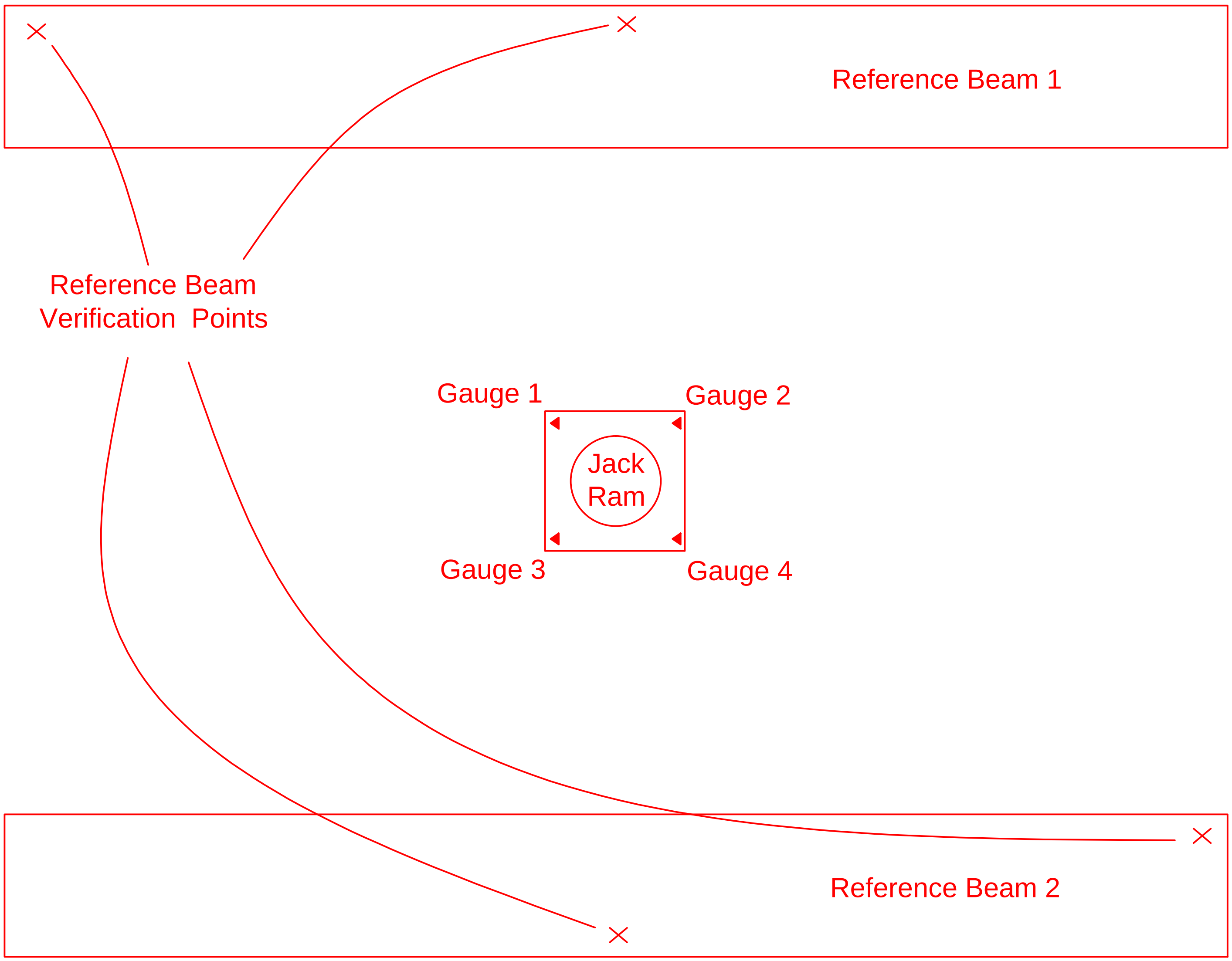
Creep Phase - Secondary Measurement
Total Displacement (mm)
15.0
16.0
17.0
18.0
19.0
20.0
21.0
Time (minutes)
1
10
100
1000
Secondary



Unoading Phase - Secondary Measurement
Total Displacement (mm)
6
8
10
12
14
16
18
20
22
24
Time (minutes)
0
10
20
30
40
50
60
70
Load (kN)
1900
1200
500
0

Enclosure B

Test Apparatus Drawings (4 pages)



NOTES

1. All features are approximate.

2. Drawing should be viewed in conjunction with memo ref. 17-1031-03, dated May 29, 2017.

NO.	DESCRIPTION	DATE

SNC·LAVALIN

CLIENT: Dufferin Construction, a Division of CRH Canada

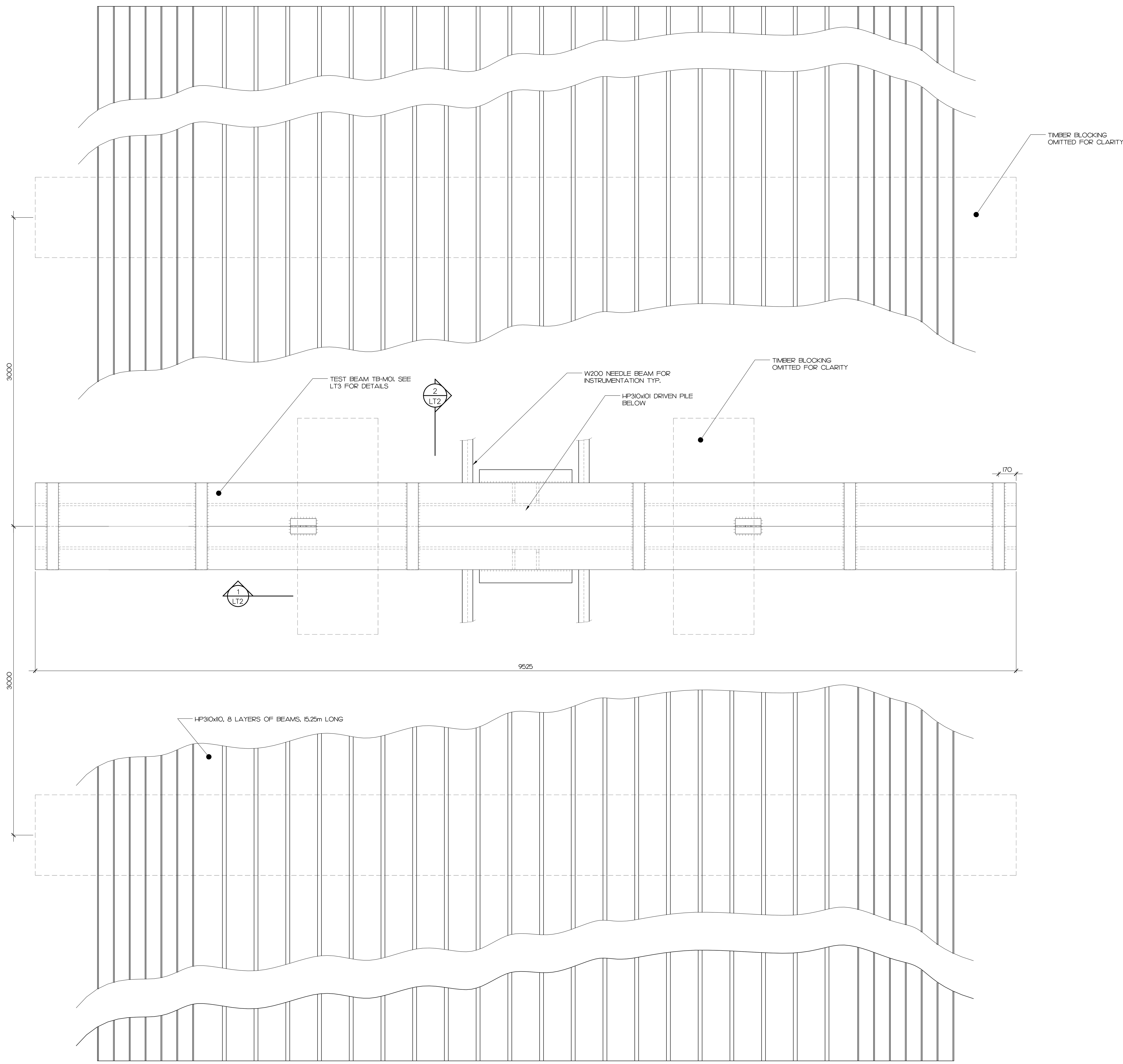
PROJECT: Pile Load Testing
MTO Contract No. 2015-2018

LOCATION: Fletcher Creek, Pile ID: INC249

TITLE: Test Apparatus Configuration

SCALE: NTS

DATE:	FILE:	DIV.	DRAWING:
May 2017	17-1031-03	00	1



1 LT1 **PLAN**
SCALE 1:20

No.	DESCRIPTION	Date
	ISSUED FOR REVIEW	17/04/27

NOTES

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LICENSED PROFESSIONAL ENGINEER 34891507 T.H. O'ROURKE 2017/APR/27 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER PROVINCE OF ONTARIO

Client

ANCHOR SHORING & CAISSONS LTD.

TORONTO ONTARIO

Consultant

T.H. O'Rourke
structural consultants

P.O. Box 599 Stouffville, ON L4A 7Z7
Telephone (905)640-6665, Fax (905)640-6655, Cell (416) 937-6999

Project

HWY 401
FLETCHER'S CREEK CULVERT
STRUCTURE REPLACEMENT

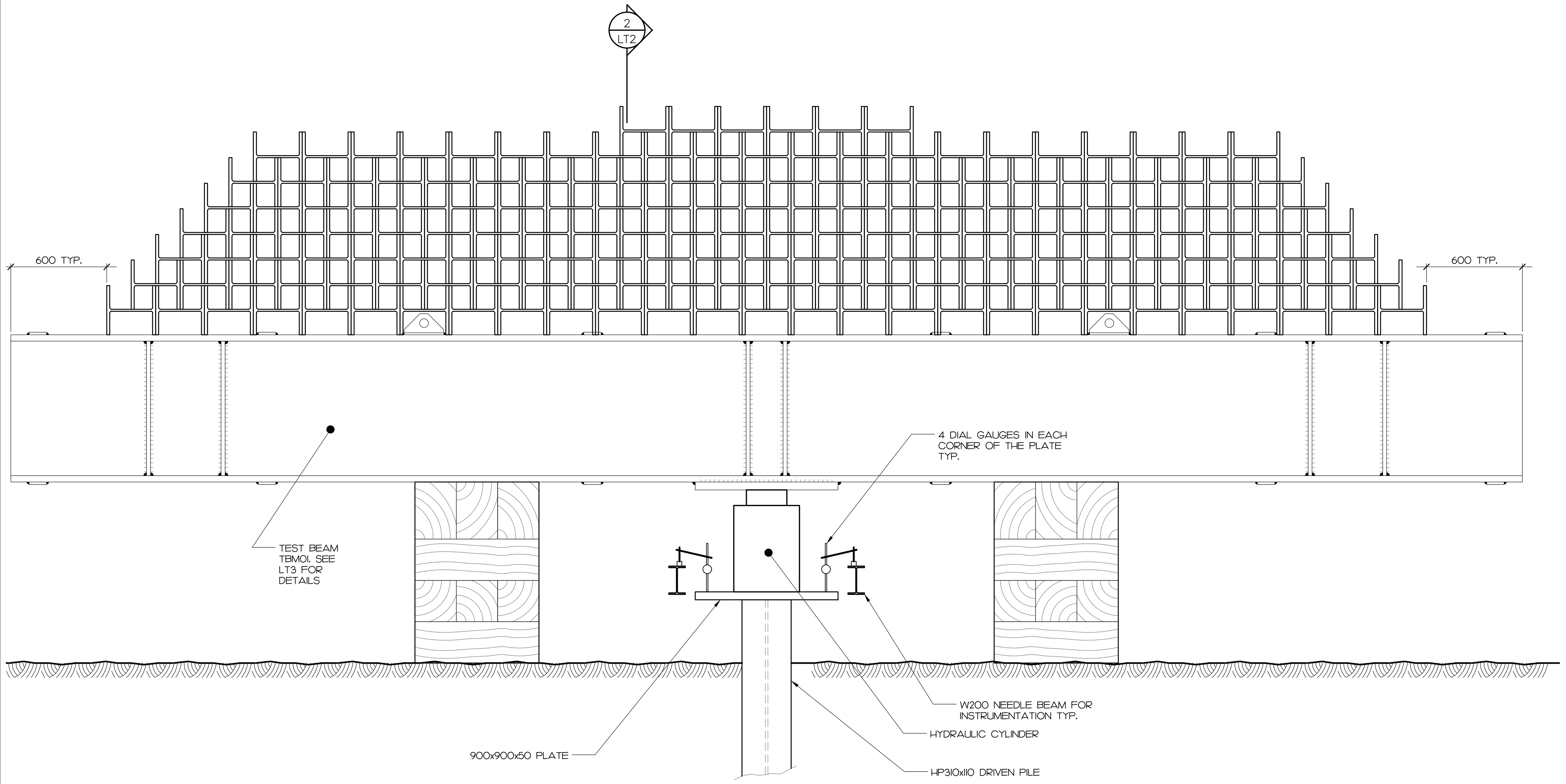
MISSISSAUGA ONTARIO

Drawing Title

PILE TEST ARRANGEMENT

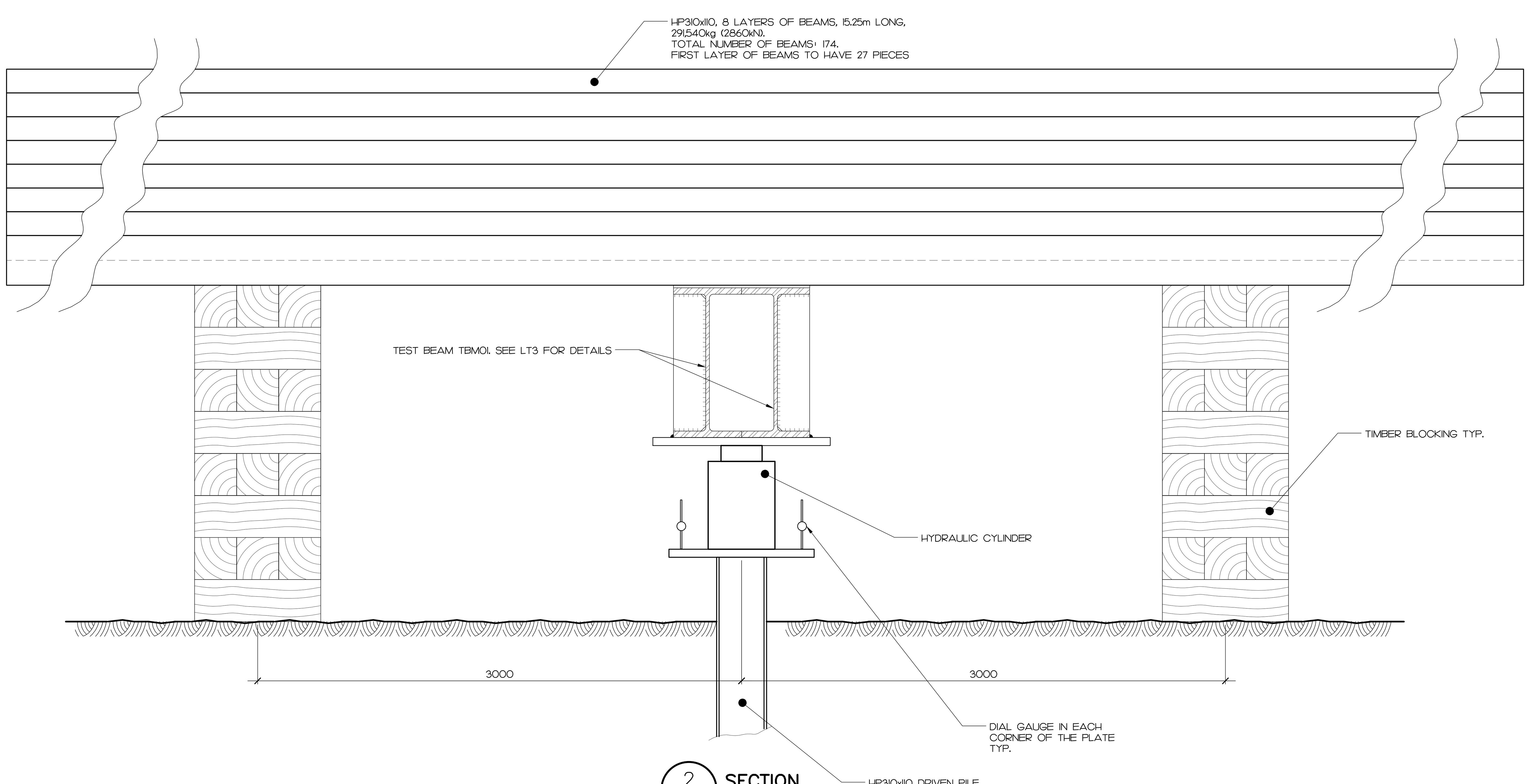
Drawn:	JC	Scale	AS NOTED
Checked:	TOR	Date	FEB., 2017
Project No.	1628-17	Drawing Number	LT1

April 27, 2017 4:57:59 PM



NOTE:
FOR TEST PROCEDURE AND NOTES SEE THIS DRAWING

1
LT1
ELEVATION
SCALE 1:20



2
LT1
SECTION
SCALE 1:20

A. TEST

1. All aspects of the test must comply with the O.B.C. and the Occupational Health and Safety Act.

2. The test pile is to be tested to a maximum load of 2,600 kN.

3. Refer also to the Full-Scale Pile Load Test Specification for this project: FletcherÆs Creek South Structure, Contract No.: 2015-2018. Comply with ASTM D1143.

B. REFERENCES

4. Design is in accordance with the structural requirements of the Ontario Building Code and The Canadian Highway Bridge Design Code. All work is to be carried out in accordance with the Occupational Health and Safety Act.

5. Geotechnical Report:
Project No. 10-1111-0211-02, by Golder Associates, dated October 23, 2013.

C. MATERIALS:

6. Structural steel design, connections, fabrication and erection is to conform to requirements of CSA S16.1 and S136.

7. Structural steel to conform to CSA G40.20-13/G40.21-13, grade 350W for wide flanges, channels and hollow structural sections and grade 300W min. for plates and all other shapes. Steel to be fabricated and erected by a shop certified by the Canadian Welding Bureau to the requirements of CSA W47.1-09. Sheet piles are to comply with ASTM A328 or CSA G40.21 with a minimum yield of 345MPa.

8. Welding to conform to CSA W59-13. Welders to be qualified to CSA W47.1-09.

9. Alternative sections or grades of equivalent strength may be substituted subject to approval by T.H. O'Rourke Structural Consultants.

10. Concrete materials, mixing, handling, design, formwork, rebar, placement, cutting and finishing to comply with CSA A23.1,2 & 3, unless modified in writing by the engineer.

11. *Drill holes to sizes and depths indicated employing liners, mud drilling and/or other methods as required to avoid the ingress of soil or groundwater. Install piles plumb and to line. Fill holes with concrete strength as specified above.*

D. PROCEDURE

12. Build up or cut down test pile to ensure that the reaction point is flat and level. Weld on plate to top of test pile.

13. Place the hydraulic jack and all required tubes and wiring. Ensure everything is in proper working order.

14. Assemble needle beams and their supports in a solid and secure configuration. Assemble strain gauges placing one on each corner of the Test Pile top plate.

15. Assemble the pile of HP310x110 sections that will be used for a reaction mass.

16. Check that all parts of the Test Beam assembly are level, plum and in alignment.

17. Apply a 10% load increment to ensure that all parts are bedded in properly and everything is functioning properly.

18. Survey the tops of the test beam before starting the test. Check at 50% load level and after the application of 100% of the test load.

19. Conduct the test as per the Engineer's instructions and according to ASTM D1143.

20. Ensure the assembly stays in alignment and does not distort or buckle during the test.

21. Check reaction load to ensure it is stable throughout the test.

22. Record the jack hydraulic pressures, the applied load and the strains measured at each load increment during the test. Present the results in a clear, legible and descriptive report.

23. Measure strains and reaction caisson elevations when all stresses have been removed.

E. CONTRACTOR

24. Prepare the test area so that the test pile may be driven and the pile test conducted without encountering obstructions.

25. Ensure that test area is always drained with no standing water.

26. Ensure that all services in or near the test area are identified. Ensure the pile driver and the pile tester is made aware of such services.

No.	DESCRIPTION	Date
	ISSUED FOR REVIEW	17/04/27

NOTES

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LICENSED PROFESSIONAL ENGINEER
34891507
T.H. O'ROURKE
2017/APR/27
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
C.D. WINTER
2017/APR/27
PROVINCE OF ONTARIO

Client

ANCHOR SHORING & CAISSONS LTD.

TORONTO ONTARIO

Consultant

T.H. O'Rourke
structural consultants
P.O. Box 599 Stouffville, ON L4A 7Z7
Telephone (905)640-6665, Fax (905)640-6655, Cell (416) 937-6999

Project

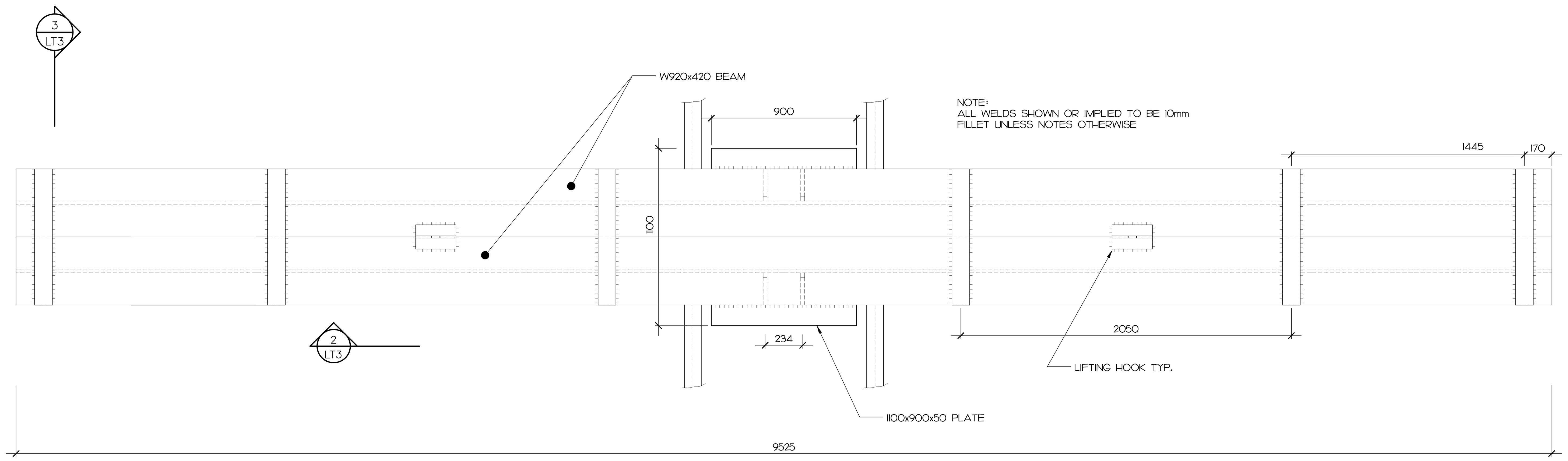
HWY 401
FLETCHER'S CREEK CULVERT
STRUCTURE REPLACEMENT

MISSISSAUGA ONTARIO

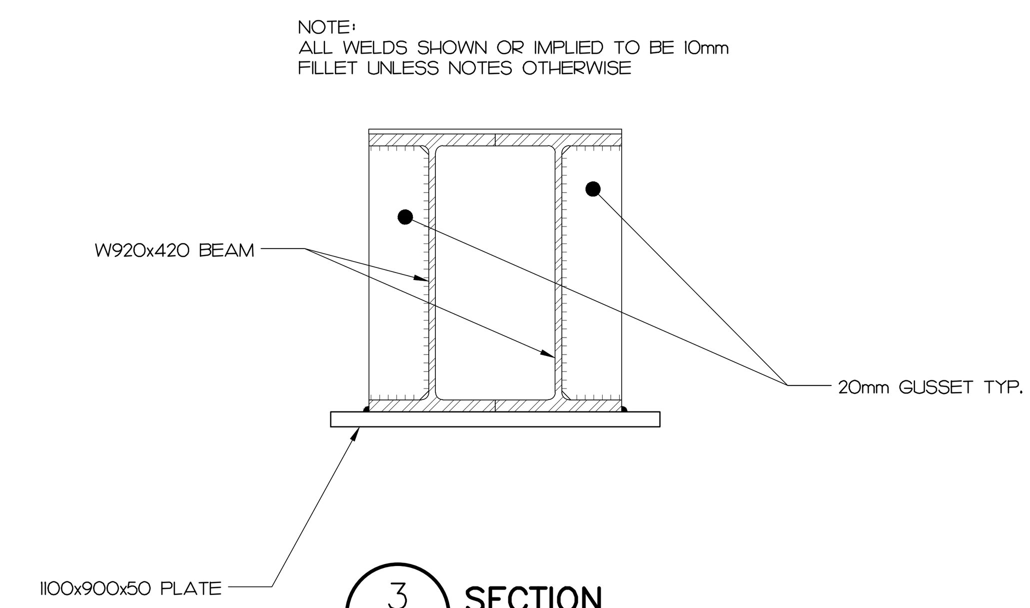
Drawing Title

PILE TEST ARRANGEMENT

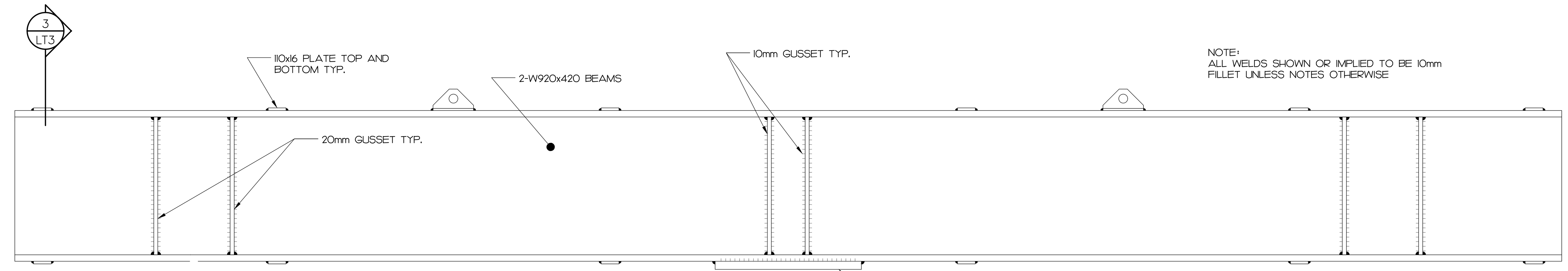
Drawn:	JC	Scale	AS NOTED
Checked:	TOR	Date	FEB., 2017
Project No.	1628-17	Drawing Number	LT2



1 LT3 **PLAN**
SCALE 1:20



3 LT3 **SECTION**
SCALE 1:20



2 LT3 **ELEVATION**
SCALE 1:20

TEST BEAM: TBM01	
MAIN BEAMS	W920x420
TOTAL WIGHT, kg	16502

No.	DESCRIPTION	Date
	ISSUED FOR REVIEW	17/04/27

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LICENSED PROFESSIONAL ENGINEER 34891507 T.H. O'ROURKE 2017/04/27 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER PROVINCE OF ONTARIO

Client

ANCHOR SHORING & CAISSONS LTD.

TORONTO ONTARIO

Consultant

T.H. O'Rourke
structural consultants
P.O. Box 599 Stouffville, ON L4A 7Z7
Telephone (905)640-6665, Fax (905)640-6655, Cell (416) 937-6999

Project

HWY 401
FLETCHER'S CREEK CULVERT
STRUCTURE REPLACEMENT

MISSISSAUGA ONTARIO

Drawing Title

TEST BEAM TBM01 DETAILS

Drawn:	JC	Scale	AS NOTED
Checked:	TOR	Date	FEB., 2017
Project No.	1628-17	Drawing Number	LT3

April 27, 2017 5:08:28 PM

Enclosure C

Provided Calibration Certificates (6 pages)

Canadian BBR Inc.
3450 Midland Avenue
Agincourt Ontario

Calibration of Hydraulic Components

600 t DUDGEON

Jack No.2

Ram area (sq.in)= 139 **Friction= 1.013**

Calibrated with digital pressure gauge
Enerpac Model DGB/ 10,000 psi
And load cell BBR No.2

Gauge pressure (psi)	Load (kips)	Load (kips)	Load (kips)		Average Load (kips)
1000	135.2	135.4	136.1		135.5
2000	271.6	274.4	269.5		271.8
3000	411.0	410.1	409.0		410.0
4000	550.8	547.7	547.6		548.7
5000	685.9	684.1	687.2		685.7
6000	824.6	822.2	825.0		824.0
7000	964.3	958.4	962.4		961.7
8000	1099.4	1093.7	1096.5		1096.5
8500	1167.9	1163.7	1166.2		1165.9

LICENSED PROFESSIONAL ENGINEER
G. EICHLER
12886015
Mar 14, 17
PROVINCE OF ONTARIO

P.O. Box 37, Agincourt, ON M1S 3B4
3450 Midland Ave., Agincourt, ON M1V 4V4

Tel :(416) 291-1618
Fax :(416) 291-9960

CERTIFIED TEST REPORTS

Reference No. : 039

Date : APRIL 21, 2017

Gauge Type : SOLFRUNT 0 - 10,000 PSI

Machine : Deadweight Tester Mansfield & Green

Traceability To : National Bureau Standards

Dead Weight Pressure	Gauge Indicated Pressure
1000 psi	1075 psi
2000 psi	2050 psi
3000 psi	3000 psi
4000 psi	4000 psi
5000 psi	5000 psi
6000 psi	6000 psi
7000 psi	7000 psi
8000 psi	8000 psi
9000 psi	8975 psi

Signature : Al [illegible]

MRM PRECISION INSTRUMENTS INC.
Embracing Quality, Innovation and Technology
An ISO 17025 Accredited Company
1 Regan Road, Unit 1
Brampton, Ontario, Canada
L7A 1B8

Tel. (905) 595-1000
Fax. (905) 595-1200
Calibration@MRM2.ca
www.MRM2.ca

CALIBRATION CERTIFICATE

CERTIFICATE No. WBH789-2017

INSTRUMENT: Indicator, Dial
CAL. DATE: May 15, 2017

SERIAL_ASSET No. WBH789
CAL. DUE DATE: **May 15, 2018**

MODEL No. 3428S-19
CUSTOMER: ANCHOR SHORING & CAISSONS LTD
ADDRESS: 3445 Kennedy Road
Toronto

GRADUATION: 0.001 in

ACCURACY: ± 0.001" (First 2.5 Rev.) ; ± 0.005" (Rest of Range)

RANGE: (0 to 4) in
TEMPERATURE: 20 ± 2 °C
HUMIDITY: (30 to 60) %

MANUFACTURER: Mitutoyo Corporation
METHOD USED: Comparison

CAL. PROCEDURE: CP-315
UNIT OF MEASUREMENT: Inch (in)
LOCATION: IH-1

PARAMETER	NOMINAL	AS FOUND	AS LEFT	MIN.	MAX.	TOLERANCE (AS LEFT)
Inward	0	0.00000	0.00000	-0.00091	0.00091	IN
	1	1.00050	1.00050	0.99509	1.00491	IN
	2	2.00075	2.00075	1.99509	2.00491	IN
	3	3.00100	3.00100	2.99509	3.00491	IN
	4	4.00125	4.00125	3.99509	4.00491	IN
Outward	2	2.00075	2.00075	1.99509	2.00491	IN
	0	0.00000	0.00000	-0.00091	0.00091	IN

TRACEABLE REFERENCE STANDARD:	
INSTRUMENT	ASSET No.
Indicator Calibration System	MRM-1060

RECEIVED CONDITION: Operational
FINAL CONDITION:

Uncertainty of Measurement: ± 0.0003 in

Uncertainty of Measurement is recognized in statements of compliance according to Decision Rule 4.2 of ASME B89.7.3.1-2001

The reported expanded uncertainty of measurement is stated as the standard uncertainty of measurement multiplied by the coverage factor k=2, which for a normal distribution corresponds to a coverage probability of approximately 95%

Reference Standards are traceable to SI through NIST, NRC or other recognized NMI.

Calibration results are only related to the instrument specified on this certificate.

Calibration Technician: M100

ilac-MRA
ACCREDITED

Rafik Mohamed, Technical Manager

MRM PRECISION INSTRUMENTS INC.
Embracing Quality, Innovation and Technology
An ISO 17025 Accredited Company
1 Regan Road, Unit 1
Brampton, Ontario, Canada
L7A 1B8

Tel. (905) 595-1000
Fax. (905) 595-1200
Calibration@MRM2.ca
www.MRM2.ca

CALIBRATION CERTIFICATE

CERTIFICATE No. WBH790-2017

INSTRUMENT: Indicator, Dial

CAL. DATE: May 15, 2017

SERIAL_ASSET No. WBH790

CAL. DUE DATE: **May 15, 2018**

MODEL No. 3428S-19

CUSTOMER: ANCHOR SHORING & CAISSONS LTD
ADDRESS: 3445 Kennedy Road
Toronto

GRADUATION: 0.001 in

ACCURACY: ± 0.001" (First 2.5 Rev.) ; ± 0.005" (Rest of Range)

RANGE: (0 to 4) in

TEMPERATURE: 20 ± 2 °C

HUMIDITY: (30 to 60) %

MANUFACTURER: Mitutoyo Corporation

METHOD USED: Comparison

CAL. PROCEDURE: CP-315

UNIT OF MEASUREMENT: Inch (in)

LOCATION: IH-1

PARAMETER	NOMINAL	AS FOUND	AS LEFT	MIN.	MAX.	TOLERANCE (AS LEFT)
Inward	0	0.00000	0.00000	-0.00091	0.00091	IN
	1	1.00000	1.00000	0.99509	1.00491	IN
	2	2.00025	2.00025	1.99509	2.00491	IN
	3	3.00025	3.00025	2.99509	3.00491	IN
	4	4.00050	4.00050	3.99509	4.00491	IN
Outward	2	2.00025	2.00025	1.99509	2.00491	IN
	0	0.00000	0.00000	-0.00091	0.00091	IN

TRACEABLE REFERENCE STANDARD:	
INSTRUMENT	ASSET No.
Indicator Calibration System	MRM-1060

RECEIVED CONDITION: Operational

FINAL CONDITION:

Uncertainty of Measurement: ± 0.0003 in

Uncertainty of Measurement is recognized in statements of compliance according to Decision Rule 4.2 of ASME B89.7.3.1-2001

The reported expanded uncertainty of measurement is stated as the standard uncertainty of measurement multiplied by the coverage factor k=2, which for a normal distribution corresponds to a coverage probability of approximately 95%

Reference Standards are traceable to SI through NIST, NRC or other recognized NMI.

Calibration results are only related to the instrument specified on this certificate.

Calibration Technician: M100

ilac-MRA A2LA ACCREDITED

Rafik Mohamed, Technical Manager

MRM PRECISION INSTRUMENTS INC.
Embracing Quality, Innovation and Technology
An ISO 17025 Accredited Company
1 Regan Road, Unit 1
Brampton, Ontario, Canada
L7A 1B8

Tel. (905) 595-1000
Fax. (905) 595-1200
Calibration@MRM2.ca
www.MRM2.ca

CALIBRATION CERTIFICATE

CERTIFICATE No. WBH792-2017

INSTRUMENT: Indicator, Dial
CAL. DATE: May 15, 2017

SERIAL_ASSET No. WBH792
CAL. DUE DATE: **May 15, 2018**

MODEL No. 3428S-19
CUSTOMER: ANCHOR SHORING & CAISSONS LTD
ADDRESS: 3445 Kennedy Road
Toronto

GRADUATION: 0.001 in

ACCURACY: ± 0.001" (First 2.5 Rev.) ; ± 0.005" (Rest of Range)

RANGE: (0 to 4) in
TEMPERATURE: 20 ± 2 °C
HUMIDITY: (30 to 60) %

MANUFACTURER: Mitutoyo Corporation
METHOD USED: Comparison

CAL. PROCEDURE: CP-315
UNIT OF MEASUREMENT: Inch (in)
LOCATION: IH-1

PARAMETER	NOMINAL	AS FOUND	AS LEFT	MIN.	MAX.	TOLERANCE (AS LEFT)
Inward	0	0.00000	0.00000	-0.00091	0.00091	IN
	1	1.00000	1.00000	0.99509	1.00491	IN
	2	2.00050	2.00050	1.99509	2.00491	IN
	3	3.00075	3.00075	2.99509	3.00491	IN
	4	4.00090	4.00090	3.99509	4.00491	IN
Outward	2	2.00050	2.00050	1.99509	2.00491	IN
	0	0.00000	0.00000	-0.00091	0.00091	IN

TRACEABLE REFERENCE STANDARD:

INSTRUMENT	ASSET No.
Indicator Calibration System	MRM-1060

RECEIVED CONDITION: Operational
FINAL CONDITION:

Uncertainty of Measurement: ± 0.0003 in

Uncertainty of Measurement is recognized in statements of compliance according to Decision Rule 4.2 of ASME B89.7.3.1-2001

The reported expanded uncertainty of measurement is stated as the standard uncertainty of measurement multiplied by the coverage factor k=2, which for a normal distribution corresponds to a coverage probability of approximately 95%

Reference Standards are traceable to SI through NIST, NRC or other recognized NMI.

Calibration results are only related to the instrument specified on this certificate.

ilac-MRA
ACCREDITED
CERT # 4194.01

Calibration Technician: M100

Rafik Mohamed, Technical Manager

MRM PRECISION INSTRUMENTS INC.
Embracing Quality, Innovation and Technology
An ISO 17025 Accredited Company
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L7A 1B8

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Fax. (905) 595-1200
Calibration@MRM2.ca
www.MRM2.ca

CALIBRATION CERTIFICATE

CERTIFICATE No. SXZ180-2017

INSTRUMENT: Indicator, Dial
CAL. DATE: May 15, 2017

SERIAL_ASSET No. SXZ180
CAL. DUE DATE: **May 15, 2018**

MODEL No. 3428S-19
CUSTOMER: ANCHOR SHORING & CAISSONS LTD
ADDRESS: 3445 Kennedy Road
Toronto

GRADUATION: 0.001 in

ACCURACY: ± 0.001" (First 2.5 Rev.) ; ± 0.005" (Rest of Range)

RANGE: (0 to 4) in
TEMPERATURE: 20 ± 2 °C
HUMIDITY: (30 to 60) %

MANUFACTURER: Mitutoyo Corporation
METHOD USED: Comparison

CAL. PROCEDURE: CP-315
UNIT OF MEASUREMENT: Inch (in)
LOCATION: IH-1

PARAMETER	NOMINAL	AS FOUND	AS LEFT	MIN.	MAX.	TOLERANCE (AS LEFT)
Inward	0	0.00000	0.00000	-0.00091	0.00091	IN
	1	1.00000	1.00000	0.99509	1.00491	IN
	2	2.00025	2.00025	1.99509	2.00491	IN
	3	3.00050	3.00050	2.99509	3.00491	IN
	4	4.00050	4.00050	3.99509	4.00491	IN
Outward	2	2.00025	2.00025	1.99509	2.00491	IN
	0	0.00000	0.00000	-0.00091	0.00091	IN

TRACEABLE REFERENCE STANDARD:	
INSTRUMENT	ASSET No.
Indicator Calibration System	MRM-1060

RECEIVED CONDITION: Operational
FINAL CONDITION:

Uncertainty of Measurement: ± 0.0003 in

Uncertainty of Measurement is recognized in statements of compliance according to Decision Rule 4.2 of ASME B89.7.3.1-2001

The reported expanded uncertainty of measurement is stated as the standard uncertainty of measurement multiplied by the coverage factor k=2, which for a normal distribution corresponds to a coverage probability of approximately 95%

Reference Standards are traceable to SI through NIST, NRC or other recognized NMI.

Calibration results are only related to the instrument specified on this certificate.

ilac-MRA ACCREDITED

Calibration Technician: M100

Rafik Mohamed, Technical Manager

Enclosure D

Modified Testing Procedure – Contract Instruction Notice #249 (3 pages)

Ontario Ministry of Transportation

INSTRUCTION NOTICE TO CONTRACTORS

Contractor:	Dufferin Construction Company			**Instruction Notice #:**	249
Contract No:	2015-2018	**Hwy:**	401/Mavis	**District:**	Central

Location: Hwy 401 Widening from East of Credit River to Highway 403/410 Interchange

Instruction Title: INC No 241- Pile Load Test Loading and Measurement Readings Procedure

Instruction To: Dufferin Construction Company

Location of Work: Hwy 401 Widening from East of Credit River to Highway 403/410 Interchange

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Furthermore to our meeting of April, 11, 2017, Revised Change Order # 107 and our discussions regarding pile load testing measurement readings, procedure and load increments and timing, please see attached letter provided by Golder Associates after discussions with Ministry to capture such requirements which form part of this Instruction Notice.

Also, please advise us of the anticipated testing date minimum two days in advance so the MTO and Golder Associates can attend the site.

Please contact us if you require any clarifications.

Three (3) pages inclusive

Issued to/
Received by: ______________________

Sal Rustico / Aaron Toth
Contractor Representative

Issued by: ______________________

Zaman Alikhani
Sr. Contract Administrator

Time & Date: ______________________

Time & Date: WED MAY 10, 2017

DISTRIBUTION:
CONTRACTOR
AREA CONTRACTS ENGINEER
CONTRACT SERVICES ADMINISTRATOR
CONTRACT ADMINISTRATOR

PH-CC-709 Aug-09

Golder Associates

May 10, 2017

Project No. 10-1111-0211

Mr. Zaman Alikhani,
AECOM
5080 Commerce Blvd.
Mississauga, Ontario
L4W 4P2

HIGHWAY 401 / FLETCHER'S CREEK PILE LOAD TEST
PROPOSED LOADING AND MEASUREMENT CRITERIA
HIGHWAY 401 WIDENING FROM HIGHWAY 403/410 INTERCHANGE TO THE CREDIT RIVER
CITY OF MISSISSAUGA, REGION OF PEEL
G.W.P. 2150-01-00, CONTRACT NO. 2015-2018

Dear Sir:

As requested in your May 4, 2017 e-mail and as indicated on the pile load test working drawing (Dwg. No. LT2 provided by Dufferin / Anchor Shoring, dated April 27, 2017), this letter summarizes the loading and measurement readings procedure that must be followed by the Contractor.

Referring to the April 11, 2017 meeting minutes (team meeting held at AECOM office), the duration of the pile load test was estimated to be less than one week. The proposed loading and measurement procedure outlined below is estimated to be less than 1.5 days to complete.

Referring to Section 8.1.2 in ASTM D1143 and after discussions with MTO Foundations, it was agreed that a modified *Procedure A: Quick Test* will be followed for this project. As per the original work plan, unless failure occurs first, the pile will be loaded to a maximum maintained load of 2,600 kN on the single test pile. The load will be applied in increments of about 25% of the design load and each load increment will be maintained for 20 minutes or until the rate of axial movement does not exceed 0.25 mm (0.01 inches) per hour, with a maximum time of 2 hours per load increment. After the maximum load is reached, if failure does not occur, the maximum load will remain on the pile for 12 hours. If failure occurs, maintain the failure load, or maximum load possible, until the total axial movement equals 50 mm (2 inches). After completing the final load increment, remove the load in increments of about 25% of the maximum test load with 1 hour between decrements, as indicated below.

SUMMARY OF LOAD INCREMENTS AND MEASUREMENT READINGS

Load (kN)	Time of Recording Readings after Load is reached	Notes
100	Confirm test setup and equipment / measurement devices are working properly, etc.	As per working drawings, load to about 10% of design load to ensure everything is functioning properly.
400	Immediately, 5, 10, and 20 minutes. (every 20 minutes thereafter up to a maximum of 2 hours, as required)	Adding load: if rate of movement is less than 0.25 mm/hr after 20 minutes, continue to next load increment, otherwise, continue recording measurements every 20 minutes until rate of movement is less than 0.25 mm/hr up to a maximum of 2 hrs.
800	Same as above	Same as above
1200	Same as above	Same as above
1600	Same as above	Same as above
2000	Same as above	Same as above
2300	Same as above	Same as above
2600	Immediately, 5, 10, 20 minutes, and every 20 minutes thereafter up to 2 hours, then every hour from 2 to 12 hours.	Maximum test load: if pile failure occurs, also take readings immediately before removing the first load increment
1,900	Immediately, 20, 40 and 60 minutes	Removing load: record measurements for total of 1 hour (i.e. 60 minutes)
1,200	Same as above	Same as above
500	Same as above	Same as above
0	Immediately, 20, 40, 60 minutes and 6 hours after removing all load	Final reading taken 6 hours after removal of all load

We trust this is sufficient for planning and completion of the pile load test. It is noted that the hydraulic jack / load cell and measurement gauges will need to be calibrated and copies of calibration records will need to be provided with the pile load test report.

Yours truly,

GOLDER ASSOCIATES LTD.

M Devata

Kevin J. Bentley, P.Eng.
Geotechnical Engineer

Murty Devata, P.Eng.
Senior Foundation Consultant

KJB/MD/lcc/rb

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APPENDIX G

Photographs #1 to #6 – Pile Load Test (TP1) December 14, 2016

Photograph 1: Titus Standard “H” Bearing Point installed on tip of test pile

Photograph 2: Static pile load test reaction frame set-up (stacked H-piles)

Photograph 3: Dial gauge(s) on independent reference beam

Photograph 4: Hydraulic jack set-up

Photograph 5: Hydraulic jack and dial gauge set-up

Photograph 6: Static load testing underway (note installation of level to monitor stability of reaction support beam)

At Golder Associates we strive to be the most respected global company providing consulting, design, and construction services in earth, environment, and related areas of energy. Employee owned since our formation in 1960, our focus, unique culture and operating environment offer opportunities and the freedom to excel, which attracts the leading specialists in our fields. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees who operate from offices located throughout Africa, Asia, Australasia, Europe, North America, and South America.

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