THURBER ENGINEERING LTD.

MEMORANDUM
ADVANCE PILE LOAD TEST PROGRAM
PDA TESTING AND STATIC PILE LOAD TESTING
HIGHWAY 569 BLANCHE RIVER BRIDGE REPLACEMENT (SITE 47-038)
NEW LISKEARD DISTRICT, ONTARIO
LAT. 47.727947, LONG. -79.695757

GEOCRES No. 31M-125

Report

to

WSP

Date: January 18, 2021
File: 22459

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HIGHWAY 569 BLANCHE RIVER BRIDGE REPLACEMENT (SITE 47-038)
NEW LISKEARD DISTRICT, ONTARIO

To:	Rob Kleine, P.Eng. Andrew Hachborn, P.Eng. WSP	Date: January 18, 2021
From:	Keli Shi, P.Eng. (Reviewed by P.K. Chatterji, P.Eng.)	File: 22459

1. INTRODUCTION

This memorandum presents a summary of the results of an advance pile load test program carried out at Highway 569 Blanche River Bridge site. The purpose of the pile load test program was to investigate the axial geotechnical resistance and pile resistance setup behaviour (time-dependent strength gain) of steel H-piles (HP310x110) driven into deep clay deposit proposed for the replacement bridge. The replacement bridge will be supported on all friction piles embedded in the clay.

It is a condition of this report that Thurber's performance of its professional services is subject to the attached Statement of Limitations and Conditions.

2. SCOPE OF WORK

The pile load test program consisted of performing three static load tests (SLT) on one test pile and three PDA tests on a second test pile. Both SLT and PDA test piles were installed to a depth of 45 m below the existing ground surface at the south side of the Blanche River and immediately to the east of the existing Highway 569 south approach.

Bermingham Foundation Solutions Limited (BFS) from Hamilton, Ontario was retained to carry out the test pile installation and PDA/SLT testing under WSP/Thurber's supervision. A Berminghammer B-32 hammer was used to install two test piles, eight reaction piles for SLT testing, and one hammer warm-up pile for PDA testing. The tests piles and warm-up pile are steel HP310x110 piles and all anchor piles are 406 mm OD and 9.5 mm thick steel pipe piles. The two test piles were installed more than 10 m apart.

The test program also included monitoring of foundation porewater pressures and surface settlement during installation and testing of the SLT test pile. A cone penetration test with pore pressure measurements (CPTu) was conducted following installation of monitoring instruments. The CPTu results are included in Appendix E.

A test pad was prepared prior to pile installation and monitoring instrumentation. Drawing A-1 included in Appendix A shows the test pad extent and location relative to the existing bridge and preparation details. A site photo showing the completed test pad prior to commencement of any installation is included in Appendix A. Drawing A-2 illustrates the locations of test piles, reaction piles and monitoring instruments within the test pad. Drawing A-3 shows the setup details of the reaction frame for SLT testing. A site photo showing the completed reaction frame is also included in Appendix A.

3. PDA TESTING

EXP was retained by BFS to carry out the PDA testing. Three PDA tests were performed on the PDA test pile at the end of initial driving (EOID), 3 weeks after initial driving and 8 weeks after initial driving, respectively. One PDA test was performed on the SLT test pile 24 hours after initial installation of the SLT test pile and reaction piles prior to assembling the reaction frame. The test results for all PDA tests are included in Appendix B and summarized in the Table 1.

Table 1 – Summary of PDA Test Results

PDA Test Time	EOID	24-hour	3-week	8-week
Test Pile	PDA Pile	SLT Pile	PDA Pile	PDA Pile
Ultimate Shaft Resistance (kN)	228	520	1,316	1,557
Ultimate Toe Resistance (kN)	22	40	44	43
Total Ultimate Geotechnical Resistance (kN)	250	560	1,360	1,600

4. STATIC PILE LOAD TESTING (SLT)

Three static pile load tests were carried out on the SLT test pile at 3 weeks, 8 weeks and 7 months after initial driving, respectively. The SLT test results are included in Appendix C and summarized in the Table 2.

Table 2 – Summary of Static Pile Load Test Results

SLT Test Time	3-week	8-week	7-month
Assessed Ultimate Geotechnical Resistance (kN)	1,250	1,400	1,500

Based on the 8-week load displacement plot, the unfactored axial geotechnical resistance at ULS is assessed to be 1,400 kN for the 45 m long test pile. For a 40 m long production pile and with a resistance factor of 0.6, the factored axial geotechnical resistance at ULS is recommended to be 750 kN per pile. The factored axial geotechnical resistance at SLS is recommended to be 600 kN per pile.

The factored ULS geotechnical resistance of 750 kN per pile is applicable for both abutments and piers. The pile is anticipated to settle in the order of 4 to 5 mm at an SLS load of 600 kN per pile based on the 8-week SLT results.

The results of the 7-month SLT indicate that there is a further increase in the axial pile resistance after 8 weeks. The average rate of increase in pile resistance is found to be much lower than that between 3-week and 8-week tests. The reduction in the rate of increase in pile resistance is generally consistent with the magnitude and dissipation rate of the remaining excess pore pressure in the foundation clay after the 8-week test.

5. INSTRUMENTATION AND MONITORING

A total of six (6) vibrating wire piezometers (VWP) were installed with two VWPs each at depths of 15 m, 25 m and 40 m below the top of test pad and located at 1 m or 2 m radius from the SLT test pile. All VWPs were connected to a datalogger mounted on a timber post near the north limit of the test pad.

Two (2) surface settlement monitoring points (SMP) were installed each at 1 m or 2 m radius from the SLT test pile and each embedded approximately 0.5 m below the top of the test pad. One (1) standpipe piezometer (SSP) was installed near the north limit of the test pad just east of the VWP datalogger.

The locations of the monitoring instruments relative to the SLT test pile are illustrated on Drawing A-2 included in Appendix A and Figure 1 in Appendix D.

5.1. Excess Pore Pressures (EPP)

EPPs in the foundation clay induced by the initial pile installation and subsequent pile load testing were monitored at the VWPs with reference to the hydrostatic pore pressure fluctuation in the clay monitored at the SSP. The results of EPP monitoring data collected up to May 1, 2019 are included in Appendix D.

Based on a review of the EPP data, following observations are made:

- Maximum EPP response of 271 kPa was recorded at the VWP installed at 25 m depth and 1 m radius;
- Two VWPs installed at 15 m depth responded to driving of the closest anchor piles with a maximum EPP of 110 kPa;
- The magnitudes of maximum EPP were generally lower at 2 m radius than those at 1 m radius;
- The rates of EPP dissipation were fastest at 40 m depth and slowest at 25 m depth;
- The overall rates of EPP dissipation were generally identical at 1 m radius and 2 m radius;
- The residual EPPs in the foundation at 8 weeks after initial driving were similar between the two VWPs installed at the same depth irrespective of their distances from the SLT test pile.

5.2. Surface Settlement Monitoring

Ground surface settlements were monitored at the two SMP locations with respect to a non-yielding benchmark. The results of the settlement monitoring are attached in Appendix D.

The settlement data indicates that approximately 10 mm settlement has occurred at the 1 m radius and 13 mm at the 2 m radius from the SLT test pile. The higher settlement at 2 m radius is likely associated with disturbances from the construction traffic and piling equipment.

6. CLOSURE

The memorandum was prepared by Keli Shi, P.Eng. and reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

Thurber Engineering Ltd.

Keli Shi, P.Eng.
Senior Geotechnical Engineer

P.K. Chatterji, P.Eng.
Designated MTO Principal Contact

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STATEMENT OF LIMITATIONS AND CONDITIONS

1. STANDARD OF CARE

This Report has been prepared in accordance with generally accepted engineering or environmental consulting practices in the applicable jurisdiction. No other warranty, expressed or implied, is intended or made.

2. COMPLETE REPORT

All documents, records, data and files, whether electronic or otherwise, generated as part of this assignment are a part of the Report, which is of a summary nature and is not intended to stand alone without reference to the instructions given to Thurber by the Client, communications between Thurber and the Client, and any other reports, proposals or documents prepared by Thurber for the Client relative to the specific site described herein, all of which together constitute the Report.

IN ORDER TO PROPERLY UNDERSTAND THE SUGGESTIONS, RECOMMENDATIONS AND OPINIONS EXPRESSED HEREIN, REFERENCE MUST BE MADE TO THE WHOLE OF THE REPORT. THURBER IS NOT RESPONSIBLE FOR USE BY ANY PARTY OF PORTIONS OF THE REPORT WITHOUT REFERENCE TO THE WHOLE REPORT.

3. BASIS OF REPORT

The Report has been prepared for the specific site, development, design objectives and purposes that were described to Thurber by the Client. The applicability and reliability of any of the findings, recommendations, suggestions, or opinions expressed in the Report, subject to the limitations provided herein, are only valid to the extent that the Report expressly addresses proposed development, design objectives and purposes, and then only to the extent that there has been no material alteration to or variation from any of the said descriptions provided to Thurber, unless Thurber is specifically requested by the Client to review and revise the Report in light of such alteration or variation.

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a) Nature and Exactness of Soil and Contaminant Description: Classification and identification of soils, rocks, geological units, contaminant materials and quantities have been based on investigations performed in accordance with the standards set out in Paragraph 1. Classification and identification of these factors are judgmental in nature. Comprehensive sampling and testing programs implemented with the appropriate equipment by experienced personnel may fail to locate some conditions. All investigations utilizing the standards of Paragraph 1 will involve an inherent risk that some conditions will not be detected and all documents or records summarizing such investigations will be based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated and the Client and all other persons making use of such documents or records with our express written consent should be aware of this risk and the Report is delivered subject to the express condition that such risk is accepted by the Client and such other persons. Some conditions are subject to change over time and those making use of the Report should be aware of this possibility and understand that the Report only presents the conditions at the sampled points at the time of sampling. If special concerns exist, or the Client has special considerations or requirements, the Client should disclose them so that additional or special investigations may be undertaken which would not otherwise be within the scope of investigations made for the purposes of the Report.

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c) Design Services: The Report may form part of design and construction documents for information purposes even though it may have been issued prior to final design being completed. Thurber should be retained to review final design, project plans and related documents prior to construction to confirm that they are consistent with the intent of the Report. Any differences that may exist between the Report's recommendations and the final design detailed in the contract documents should be reported to Thurber immediately so that Thurber can address potential conflicts.

d) Construction Services: During construction Thurber should be retained to provide field reviews. Field reviews consist of performing sufficient and timely observations of encountered conditions in order to confirm and document that the site conditions do not materially differ from those interpreted conditions considered in the preparation of the report. Adequate field reviews are necessary for Thurber to provide letters of assurance, in accordance with the requirements of many regulatory authorities.

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The information, interpretations and conclusions in the Report are based on Thurber's interpretation of conditions revealed through limited investigation conducted within a defined scope of services. Thurber does not accept responsibility for independent conclusions, interpretations, interpolations and/or decisions of the Client, or others who may come into possession of the Report, or any part thereof, which may be based on information contained in the Report. This restriction of liability includes but is not limited to decisions made to develop, purchase or sell land.

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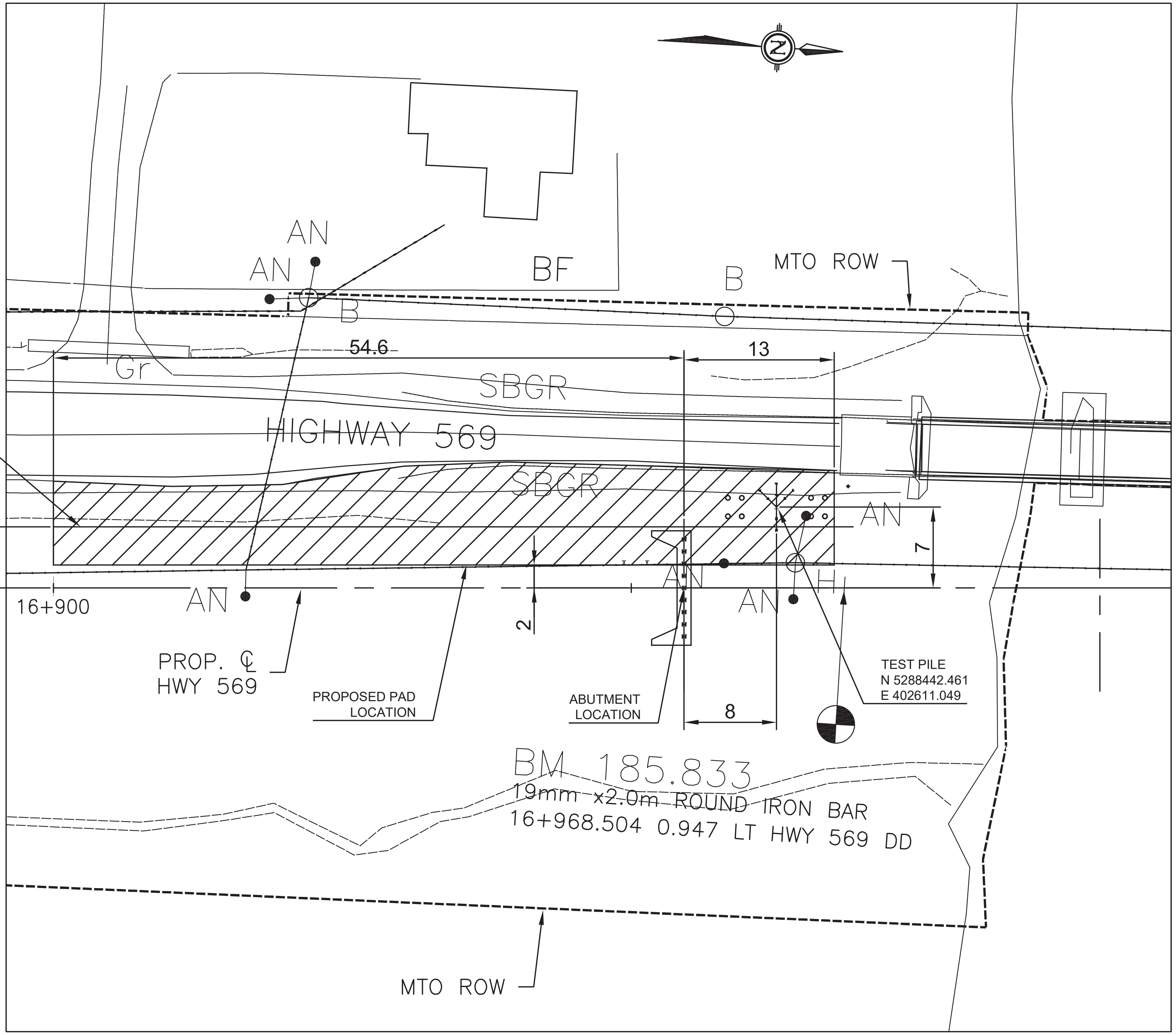
Appendix A

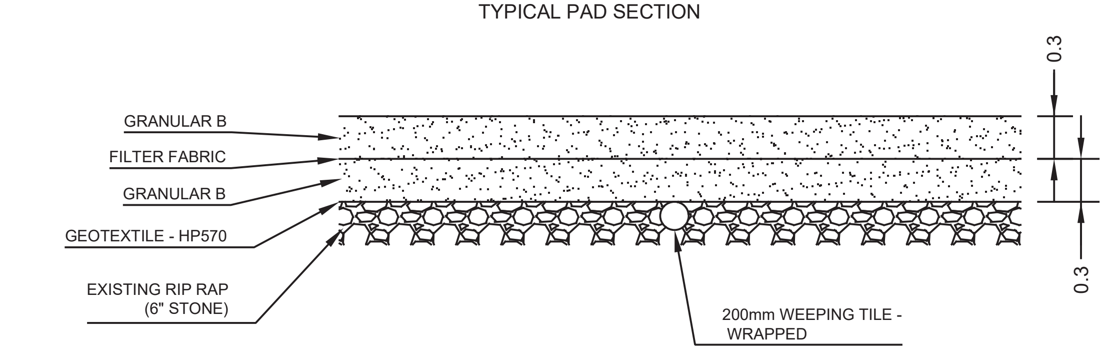
Drawings and Site Photos

PRELIMINARY

Drawing A-1



TYPICAL PAD SECTION



No.	Zone	Revision Description	Date	By
P4		Moved test pile 2m closer to road	13.08.18	DES
P3		Moved PDA and warm up pile	02.08.18	DES
P2		Added N&E for test pile	01.08.18	DES
		PRELIMINARY	31.07.18	DES

Designed: MR	Checked:
Drawn: DES	Date: 31.07.18 Scale:

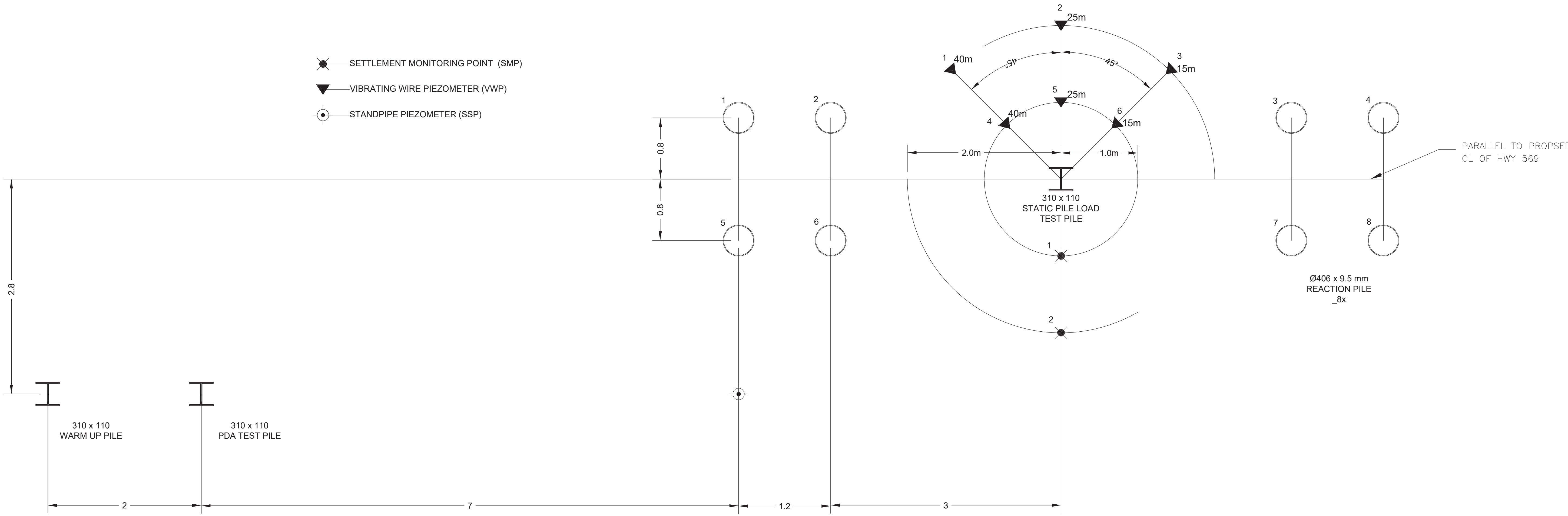
NOTE: UNLESS NOTED OTHERWISE ALL WELDS SHOWN OR IMPLIED ARE TO BE 8mm FILLET WELDS

BERMINGHAM
FOUNDATION SOLUTIONS LIMITED
HAMILTON · ONTARIO · CANADA

Project:

Title: C18–19 TEST PILE PAD DETAILS

Drawing No: Sheet 1 OF 1 Rev.: A



	Northing	Easting
Test Pile	5288442.583	402613.044
Reaction Pile 1	5288438.342	402612.503
Reaction Pile 2	5288439.540	402612.429
Reaction Pile 3	5288445.528	402612.062
Reaction Pile 4	5288446.726	402611.988
Reaction Pile 5	5288438.440	402614.100
Reaction Pile 6	5288439.638	402614.026
Reaction Pile 7	5288445.626	402613.659
Reaction Pile 8	5288446.824	402613.585
Warm up Pile	5288433.572	402616.402
PDA Pile	5288435.568	402616.280
SMP 1	5288442.644	402614.042
SMP 2	5288442.705	402615.040
VWP 1	5288441.085	402611.719
VWP 2	5288442.461	402611.048
VWP 3	5288443.908	402611.546
VWP 4	5288441.834	402612.382
VWP 5	5288442.522	402612.046
VWP 6	5288443.245	402612.295
SSP	5288438.562	402616.096

Drawing A-2

NOTE: UNLESS NOTED OTHERWISE ALL WELDS SHOWN OR IMPLIED ARE TO BE 8mm FILLET WELDS

No.	Zone	Revision Description	Date	By
P3		Moved warm up and PDA Pile out new abutment zone	02.08.18	DES
P2		Added N&E table	02.08.18	DES
A		PRELIMINARY	31.07.18	DES

Designed: MR | Checked:
Drawn: DES | Date: 31.07.18 | Scale:

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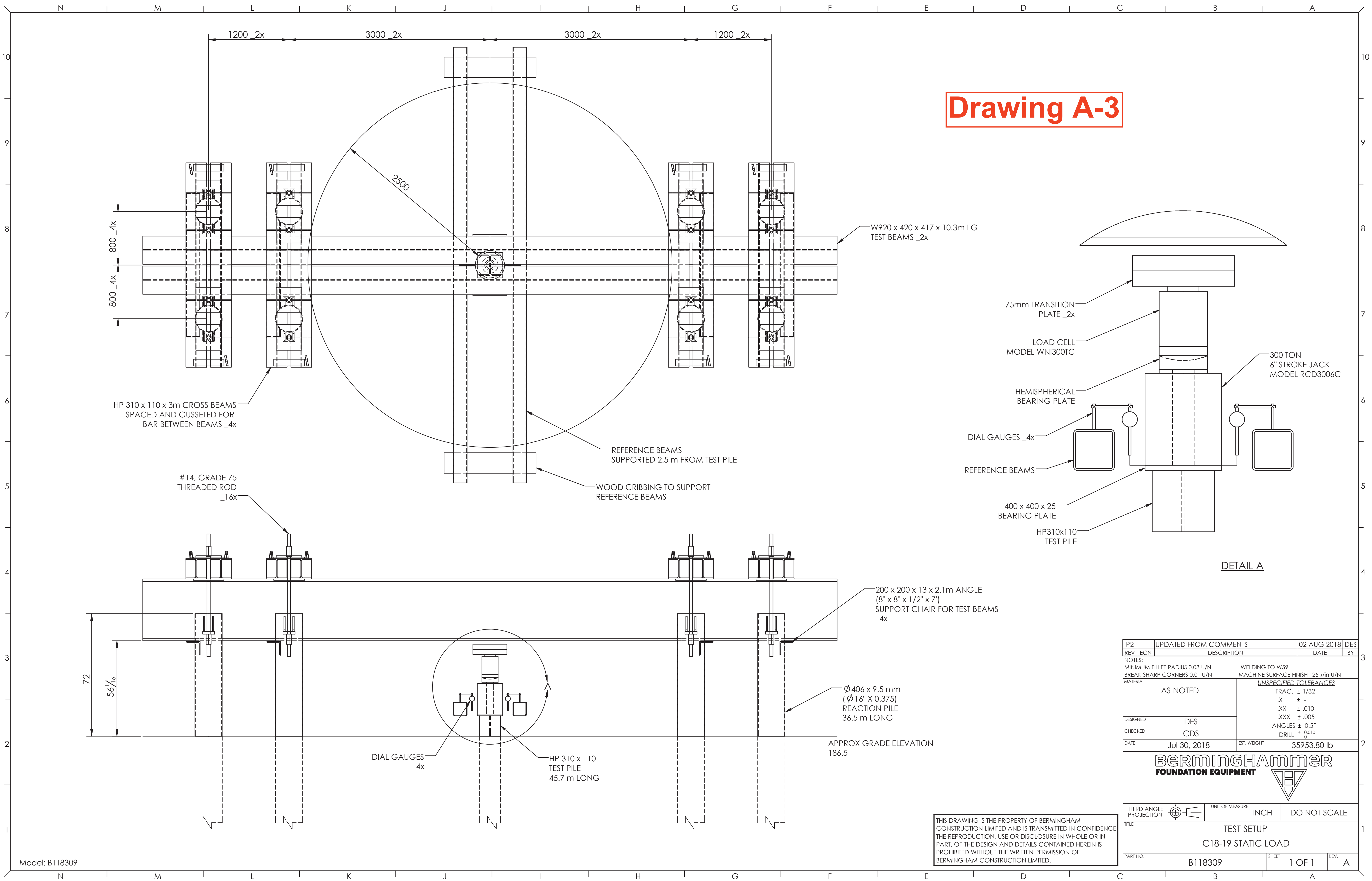
Project:
Title: C18-19 LOAD TEST INSTRUMENT DETAILS
Drawing No: | Sheet 1 OF 1 | Rev.: P2



Drawing A-3
1200 _2x
3000 _2x
3000 _2x
1200 _2x
2500
800 _4x
800 _4x
W920 x 420 x 417 x 10.3m LG TEST BEAMS _2x
HP 310 x 110 x 3m CROSS BEAMS SPACED AND GUSSETED FOR BAR BETWEEN BEAMS _4x
REFERENCE BEAMS SUPPORTED 2.5 m FROM TEST PILE
WOOD CRIBBING TO SUPPORT REFERENCE BEAMS
#14, GRADE 75 THREADED ROD _16x
75mm TRANSITION PLATE _2x
LOAD CELL MODEL WNI300TC
300 TON 6" STROKE JACK MODEL RCD3006C
HEMISPHERICAL BEARING PLATE
DIAL GAUGES _4x
REFERENCE BEAMS
400 x 400 x 25 BEARING PLATE
HP310x110 TEST PILE
DETAIL A
200 x 200 x 13 x 2.1m ANGLE (8" x 8" x 1/2" x 7') SUPPORT CHAIR FOR TEST BEAMS _4x
Ø406 x 9.5 mm (Ø16" X 0.375) REACTION PILE 36.5 m LONG
72
56 1/16
A
DIAL GAUGES _4x
HP 310 x 110 TEST PILE 45.7 m LONG
APPROX GRADE ELEVATION 186.5
Model: B118309
THIS DRAWING IS THE PROPERTY OF BERMINGHAM CONSTRUCTION LIMITED AND IS TRANSMITTED IN CONFIDENCE. THE REPRODUCTION, USE OR DISCLOSURE IN WHOLE OR IN PART, OF THE DESIGN AND DETAILS CONTAINED HEREIN IS PROHIBITED WITHOUT THE WRITTEN PERMISSION OF BERMINGHAM CONSTRUCTION LIMITED.
P2 | UPDATED FROM COMMENTS | 02 AUG 2018 | DES
REV | ECN | DESCRIPTION | DATE | BY
NOTES:
MINIMUM FILLET RADIUS 0.03 U/N
BREAK SHARP CORNERS 0.01 U/N
WELDING TO W59
MACHINE SURFACE FINISH 125 μ/in U/N
MATERIAL AS NOTED
UNSPECIFIED TOLERANCES
FRAC. ± 1/32
.X ± -
.XX ± .010
.XXX ± .005
ANGLES ± 0.5°
DRILL +0.010 -0
DESIGNED DES
CHECKED CDS
DATE Jul 30, 2018
EST. WEIGHT 35953.80 lb
BERMINGHAMMER FOUNDATION EQUIPMENT
THIRD ANGLE PROJECTION
UNIT OF MEASURE INCH
DO NOT SCALE
TITLE TEST SETUP C18-19 STATIC LOAD
PART NO. B118309
SHEET 1 OF 1
REV. A


JCB

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Appendix B

PDA Testing Results

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End of Initial Driving (EOID) PDA Test Report

Bermingham Foundation Solutions
600 Ferguson Avenue North
Hamilton, Ontario
L8L 4Z9

Pile Load Test Program – High-Strain Dynamic Test of Pile TP 2 Proposed Replacement of Blanche River Bridge Highway 569 in New Liskeard, Ontario

Project Number
BRM- 607254-A0

Prepared By:

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

Date Submitted
2018-09-18

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Tables

Appendices

1 Introduction

The Ministry of Transportation of Ontario (MTO) has commissioned Bermingham Foundation Solutions (Bermingham) to carry out a pre-construction pile load test program for the proposed replacement of the Blanche River Bridge (MTO Structure No. 47-38), located approximately 150 m north of the intersection of Hilliardton Road and Highway 569, near New Liskeard.

The primary purpose of the program is to examine the geotechnical resistances of steel pile foundations that have been proposed for the planned bridge. The area of the test pile program is the south-east corner of the bridge. The methods of examination include a series of static load tests of one pile (TP1) and a series of dynamic load tests on the second, adjacent pile (TP2).

Exp Services Inc. (EXP) was retained by Bermingham to monitor the Static Load Testing of Pile TP1 and to perform dynamic testing of the Pile TP2.

In this report is presented the results of the dynamic test that was carried out at the end of initial driving (EOID) of Pile TP2. The test was performed on September 11, 2018 and is the first of three (3) dynamic tests that are planned.

Pile TP2 is a 310 mm x 110 kg/m steel HP-section and was monitored using the Pile Driving Analyzer (PDA). It is made up of three 15.24 m (50 feet) long steel sections and driven to an embedded depth of 44.0 m. Dynamic load to the pile was provided by a Berminghammer B-32 open end diesel hammer. The manufacturer's maximum rated energy of the hammer is 110 kJ.

Subsurface characteristics of the site is described in the project Foundation Investigation Report (report Geocres No. 31M-120), and indicates that the site is underlain by a deposit of soft to firm varved clay that is more than 50 m thick.

2 Fieldwork and Engineering Analysis

On September 11, 2018, pile TP2 was monitored at the end-of-initial-driving (EOID) with the instrumentation from the Pile Driving Analyzer (PDA) attached. The purpose of the test was to examine the geotechnical resistance of the pile immediately after installation. Pile TP2 was driven to a depth of 44.0 m using a Berminghammer B-32 open end diesel hammer (110 kJ rated energy).

High-strain dynamic testing using the PDA was undertaken in general accordance with the ASTM D4945-12 procedures. The instrumentation for the PDA 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 PAX/PAK) 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 hammer blow from the end of initial driving of the tested pile was used to perform signal matching analyses using CAPWAP (CAse Pile Wave Analysis Program) in order to evaluate the ultimate resistance 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 also 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 fieldwork was carried out on September 11, 2018; Pile TP2 was instrumented with strain gauges and accelerometers, and monitored at the end of initial driving (EOID) with the Pile Driving Analyzer (PDA), and using a Berminghammer B-32 open ended diesel hammer (rated 110 kJ).

The results of the dynamic testing are presented on Table No. 1, and together with field observations are summarized below.

When Pile TP2 had penetrated 43.0 m into the ground, the diesel hammer was operating normally (i.e. with fuel being injected for engine ignition), and the penetration per blow was noted to be large; greater than ~10 mm per blow. In general, wave records with corresponding pile movements in excess of ~10 mm is considered to lead to an overestimation of dynamic testing results. As such, a request was made to halt the injection of fuel during testing.

At the end of driving, the penetration resistance was reported to be 38 mm for 5 blows (average rebound of ~3 mm) when fuel was <u>not</u> injected for the diesel engine ignition. When fuel was injected and with the diesel hammer operating at a speed of 43 blows per minute, the penetration resistance was found to be ~200 mm per 4 blows (average rebound of ~2 mm).

The energy transferred to the top of the pile TP2 was 5 kJ, with the hammer operating without fuel. The maximum forces at the instrumentation was calculated to be 1143 kN, which corresponds to a compressive stress of 81 MPa.

Pile TP2 was driven to a final depth of 44.0 m. The ultimate geotechnical resistance of Pile TP2 at the EOID was evaluated to be 250 kN.

3.2 CAPWAP Analysis Results

Signal matching analyses using CAPWAP was undertaken on a selected hammer blow from the EOID of Pile TP2. The Case Method Capacities and Pile Profile and Model tables, CAPWAP Force matches, Force-Velocity Wave forms, Resistance Distributions, and Simulated Compression Load Test Curves are presented in Appendix A.

The ultimate geotechnical resistance at the EOID of Pile TP2 was evaluated to be 250 kN, of which 228 kN was evaluated as shaft resistance and 22 kN was evaluated as toe resistance.

4 Summary

As part of a preconstruction test pile program for the proposed replacement of the Blanche River Bridge, dynamic load test using the PDA was carried out at the end of initial driving (EOID) on a HP310x110 pile (Pile TP2). The test at the EOID is the first of a series of three (3) dynamic load tests that are intended to examine the geotechnical resistance of the pile foundations. Subsequent tests are planned at 2 weeks and 8 weeks after pile installation.

Pile TP2 was driven to an embedment depth of 44.0 m below grade on September 11, 2018 using a Berminghammer B-32 diesel hammer. Field observations and records indicate that little effort (less than a total of 150 blows) was required to install the pile using the said hammer.

The mobilized geotechnical resistance at the EOID was evaluated to be ~250 kN, of which over 90% of the resistance was evaluated to be shaft resistance.

It should be noted that the evaluated geotechnical resistance of TP2 reflects the resistance at the time of testing i.e. *at the end of initial driving*. The geotechnical resistance is anticipated to increase with time. If pile TP2 is load tested at a later date, the proven geotechnical resistance is expected to be significantly higher than its geotechnical resistance at the end of initial driving.

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.

Michael W.K. Choy, P. Eng
Senior Geotechnical Engineer

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

MCHOY/I:\2003-Brampton\Projects\Geotechnical Engineering\0600000\607000\607200\607254-A0 Blanche River\REport\PDA EOID rev 0.doc

Table 1 – Summary of Pile Driving Analyzer Results

PILE NO.	EVENT	HAMMER	DEPTH BELOW GRADE (m)	REPORTED PENETRATION RESISTANCE DURING TEST		TRANSFERRED ENERGY		FORCE		EVALUATED ULT. GEOTECHNICAL RESISTANCE	REMARKS
				Blows / mm	Equiv. Blows/ 25 mm	Mean (kJ)	Speed (blows / min)	Max. (kN)	Stress (MPa)	(kN)	
Date of Test : September 11, 2018											
TP2	End of Initial Driving	Berminghammer B-32	44.0	5 bl. / 38 mm Note 2 4 bl. / 200 mm Note 3	3 to 4 less than 1	5 20	n/a 51	1143 2058	81 146	250 -	Length below PDA sensors = 45.0 m

Notes:

1. Grade refers to the existing ground level at the time of testing; approx. 186.5 m.
2. Hammer blows applied without fuel
3. Hammer blows applied with fuel.

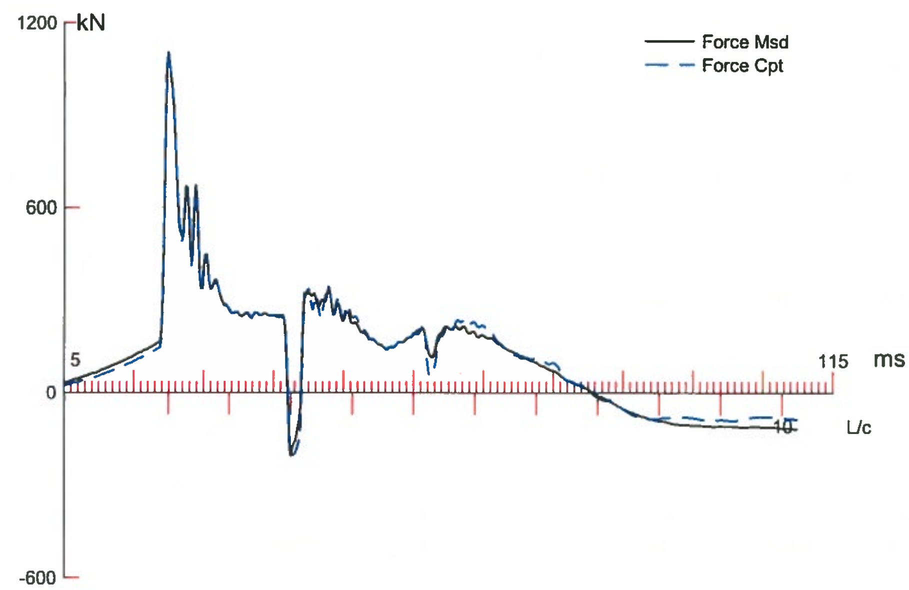
Table 2 - Summary of Signal Matching (CAPWAP) Analysis Results

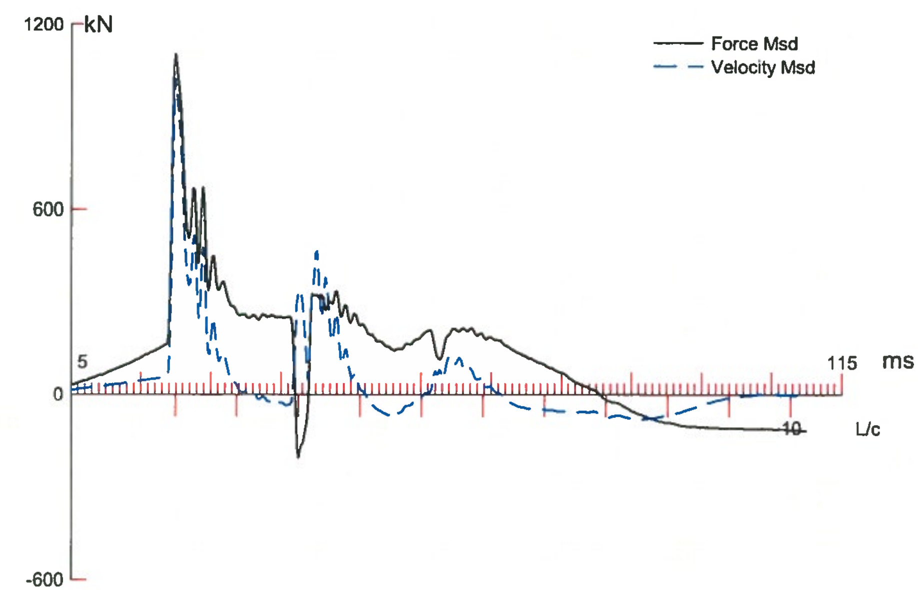
PILE NO.	EVENT	DEPTH BELOW GRADE	REPORTED PENETRATION RESISTANCE (blows/mm)	EVALUATED ULT. (MOB) GEOTECHNICAL RESISTANCE		
				Total	Shaft	Toe
TP2	BOR	45.0 m	5 blows / 38 mm	250 kN	228 kN	22 kN

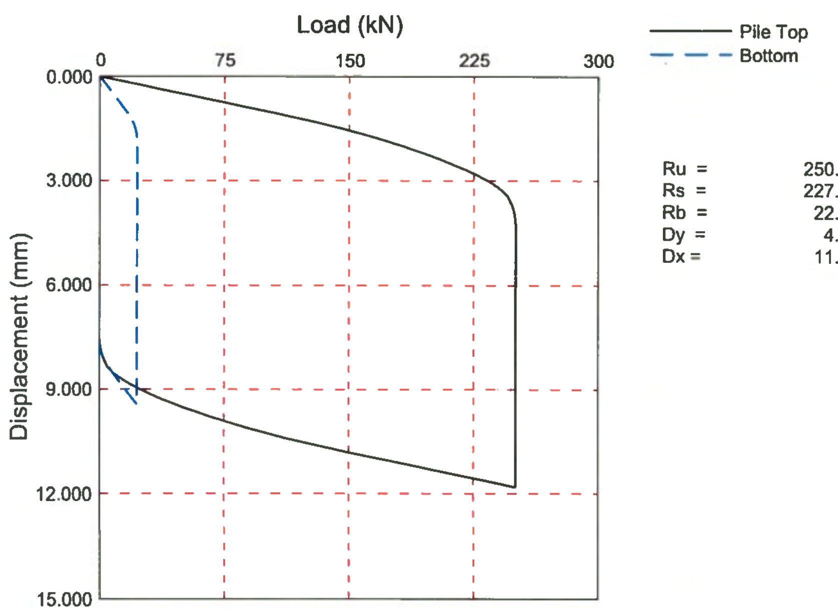
Appendix A
CAPWAP Tables and Figures for Test at Pile TP2 End of Initial Driving (September 11, 2018)

BLANCHE RIVER BRIDGE, NEW LISKEARD; Pile: TP2; HP310X110; Blow: 33 (Test: 11-Sep-2018 16:16:)
exp Services, Inc.

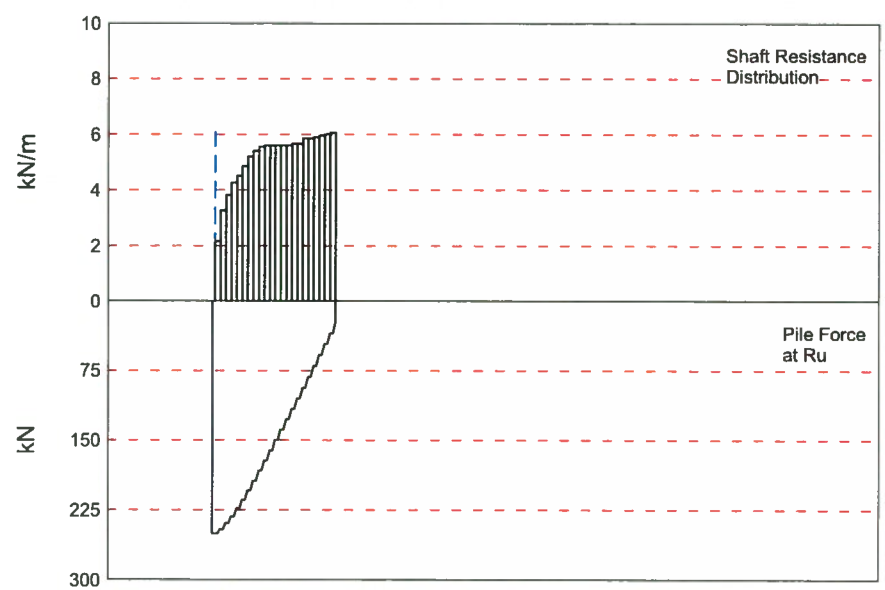
18-Sep-2018
CAPWAP(R) 2006-3







Ru =	250.2 kN
Rs =	227.7 kN
Rb =	22.5 kN
Dy =	4.2 mm
Dx =	11.8 mm



CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 250.2; along Shaft 227.7; at Toe 22.5 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
				250.2				
1	3.0	2.0	4.3	245.9	4.3	2.15	1.74	1.402
2	5.0	4.0	6.5	239.4	10.8	3.25	2.63	1.402
3	7.0	6.0	7.6	231.8	18.4	3.80	3.07	1.402
4	9.0	8.0	8.5	223.3	26.9	4.25	3.44	1.402
5	11.0	10.0	9.0	214.3	35.9	4.50	3.64	1.402
6	13.0	12.0	9.7	204.6	45.6	4.85	3.92	1.402
7	15.0	14.0	10.4	194.2	56.0	5.20	4.21	1.402
8	17.0	16.0	10.8	183.4	66.8	5.40	4.37	1.402
9	19.0	18.0	11.1	172.3	77.9	5.55	4.49	1.402
10	21.0	20.0	11.2	161.1	89.1	5.60	4.53	1.402
11	23.0	22.0	11.2	149.9	100.3	5.60	4.53	1.402
12	25.0	24.0	11.2	138.7	111.5	5.60	4.53	1.402
13	27.0	26.0	11.2	127.5	122.7	5.60	4.53	1.402
14	29.0	28.0	11.2	116.3	133.9	5.60	4.53	1.402
15	31.0	30.0	11.3	105.0	145.2	5.65	4.57	1.402
16	33.0	32.0	11.3	93.7	156.5	5.65	4.57	1.402
17	35.0	34.0	11.7	82.0	168.2	5.85	4.73	1.402
18	37.0	36.0	11.7	70.3	179.9	5.85	4.73	1.402
19	39.0	38.0	11.8	58.5	191.7	5.90	4.77	1.402
20	41.0	40.0	11.9	46.6	203.6	5.95	4.81	1.402
21	43.0	42.0	12.0	34.6	215.6	6.00	4.85	1.402
22	45.0	44.0	12.1	22.5	227.7	6.05	4.89	1.402
Avg. Shaft			10.4			5.18	4.19	1.402
Toe			22.5				235.65	1.313

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(mm)	1.004	1.417
Case Damping Factor		0.561	0.052
Unloading Quake	(% of loading quake)	30	78
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	34	

CAPWAP match quality = 2.40 (Wave Up Match) ; RSA = 0
Observed: final set = 7.600 mm; blow count = 132 b/m
Computed: final set = 7.768 mm; blow count = 129 b/m

max. Top Comp. Stress = 78.3 MPa (T= 20.3 ms, max= 1.006 x Top)
max. Comp. Stress = 78.8 MPa (Z= 3.0 m, T= 20.7 ms)
max. Tens. Stress = -21.70 MPa (Z= 33.0 m, T= 31.4 ms)
max. Energy (EMX) = 5.09 kJ; max. Measured Top Displ. (DMX)=10.79 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	1104.0	-230.6	78.3	-16.35	5.09	1.8	11.237
2	2.0	1107.4	-210.8	78.5	-14.95	5.09	1.8	11.161
5	5.0	1106.6	-190.2	78.5	-13.49	4.98	1.8	10.951
8	8.0	1075.8	-209.6	76.3	-14.86	4.66	1.7	10.791
11	11.0	1064.2	-215.5	75.5	-15.29	4.47	1.7	10.650
14	14.0	1022.7	-241.6	72.5	-17.14	4.08	1.6	10.513
17	17.0	1008.2	-249.8	71.5	-17.72	3.87	1.6	10.379
20	20.0	959.8	-282.9	68.1	-20.06	3.44	1.5	10.254
23	23.0	943.3	-284.1	66.9	-20.15	3.23	1.5	10.132
26	26.0	894.1	-290.4	63.4	-20.59	2.81	1.5	10.023
29	29.0	879.3	-271.6	62.4	-19.27	2.60	1.4	9.909
32	32.0	830.7	-266.7	58.9	-18.91	2.25	1.4	9.800
35	35.0	817.9	-244.1	58.0	-17.31	1.97	1.5	9.736
38	38.0	772.4	-245.9	54.8	-17.44	1.53	1.6	9.688
39	39.0	778.2	-173.7	55.2	-12.32	1.51	1.5	9.671
40	40.0	752.4	-251.9	53.4	-17.86	1.34	1.5	9.657
41	41.0	757.1	-233.1	53.7	-16.53	1.28	1.5	9.644
42	42.0	724.0	-219.6	51.3	-15.57	1.03	1.8	9.633
43	43.0	674.7	-144.0	47.9	-10.21	1.03	2.0	9.621
44	44.0	434.8	-89.3	30.8	-6.34	0.77	2.2	9.613
45	45.0	138.0	-7.8	9.8	-0.55	0.50	2.2	9.603
Absolute	3.0			78.8			(T =	20.7 ms)
	33.0				-21.70		(T =	31.4 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	843.0	706.4	569.8	433.2	296.6	160.0	23.3	0.0	0.0	0.0
RX	843.0	706.4	569.8	433.2	317.9	273.8	232.1	231.9	231.7	231.4
RU	918.8	789.8	660.8	531.7	402.7	273.6	144.6	15.6	0.0	0.0

RAU = 215.5 (kN); RA2 = 302.9 (kN)

Current CAPWAP Ru = 250.2 (kN); Corresponding J(RP)= 0.43; J(RX) = 0.56

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
m/s	ms	kN	kN	kN	mm	mm	mm	kJ	kN
1.87	20.11	1065.9	1143.4	1143.4	10.794	7.674	7.600	5.3	572.0

PILE PROFILE AND PILE MODEL

Depth	Area	E-Modulus	Spec. Weight	Perim.
m	cm^2	MPa	kN/m^3	m
0.00	141.00	206842.7	77.287	1.236
45.00	141.00	206842.7	77.287	1.236

Toe Area 0.095 m^2

Top Segment Length 1.00 m, Top Impedance 569.29 kN/m/s

Pile Damping 1.0 %, Time Incr 0.195 ms, Wave Speed 5123.0 m/s, 2L/c 17.6 ms

Appendix B
Reference Drawings, Documents and Subsurface Information

BERMINGHAMMER
FOUNDATION EQUIPMENT

Model B-32

Clean Series 2005

Features

- ☐ Remote Throttle - infinitely controlable energy
- ☐ Clean Combustion- Low Emissions
- ☐ Fuel injection
- ☐ Easy Start in soft driving
- ☐ Available with hydraulic trip
- ☐ Free-standing operation
- ☐ Specialty driving adapters
- ☐ Optional Kinetic Energy Monitor
- ☐ Optional Energy Control System (patented)
- ☐ Environmentally friendly (no-drip operation, bio-fuels and oils)

Operational Specifications

Ram mass:	7,050 lbs (3 200 kg)
Rated Energy:	81,080 ft▪lbs (110 kJ)
Stroke at Rated Energy:	11.5 ft (3.5 m) 35 blows per minute
Maximum Physical Stroke:	13.0 ft (4.0 m)
Range of Operation:	4.5-11.5 ft (1.4-3.5 m) 56-35 blows per minute
Kinetic Energy at Rated Stroke:	50,040 ft-lbs (67.8 kJ)
Hammer Weight - bare hammer:	14,110 lbs (6 400 kg)
Weight with Typical USA-Style Box Lead Guides:	14,570 lbs (6 610 kg) 26 in (660 mm) guides
Typical Direct-Drive Housing:	1,850 lbs (840 kg) 21 in (530 mm) opening
Total Typical Operating Weight:	16,420 lbs (7 450 kg) (with guides, trip, and drive housing)
Fuel Tank Capacity:	19.0 US Gal. (72 L)
Oil Tank Capacity:	6.5 US Gal. (25 L)
Overall Length:	20.1 ft (6.1 m)
Length including Direct-Drive Housing:	21.7 ft (6.6 m)
Minimum Box Lead size:	26 in (660 mm)

English Units

B-32	7,050 lb Piston		
BPM	Stroke (ft)	Potential Energy (ft•lb)	Velocity (ft/s)
35	11.8	83,190	22.5
36	11.2	78,960	22.0
37	10.6	74,730	21.5
38	10.0	70,500	21.0
39	9.5	66,980	20.5
40	9.1	64,160	20.0
41	8.6	60,630	19.5
42	8.2	57,810	19.0
43	7.8	54,990	18.5
44	7.5	52,880	18.0
45	7.2	50,760	17.5
46	6.9	48,650	17.0
47	6.6	46,530	16.5
48	6.3	44,420	16.0
49	6.0	42,300	15.5
50	5.8	40,890	15.0
51	5.6	39,480	14.6
52	5.4	38,070	14.2
53	5.2	36,660	13.8
54	5.0	35,250	13.4
55	4.8	33,840	13.0
56	4.6	32,430	12.6

SI Units

B-32	3 200 kg Piston		
BPM	Stroke (m)	Potential Energy (kJ)	Velocity (m/s)
35	3.60	113	6.9
36	3.41	107	6.7
37	3.23	101	6.6
38	3.05	95.7	6.4
39	2.90	91.0	6.3
40	2.77	87.0	6.1
41	2.62	82.2	5.9
42	2.50	78.5	5.8
43	2.38	74.7	5.6
44	2.29	71.9	5.5
45	2.20	69.1	5.3
46	2.10	65.9	5.2
47	2.01	63.1	5.0
48	1.92	60.3	4.9
49	1.83	57.4	4.7
50	1.77	55.6	4.6
51	1.71	53.7	4.5
52	1.65	51.8	4.3
53	1.59	49.9	4.2
54	1.52	47.7	4.1
55	1.46	45.8	4.0
56	1.40	43.9	3.8

Stroke height is a function of soil resistance and may not be attainable in certain driving conditions.
Standard Operating Range.

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

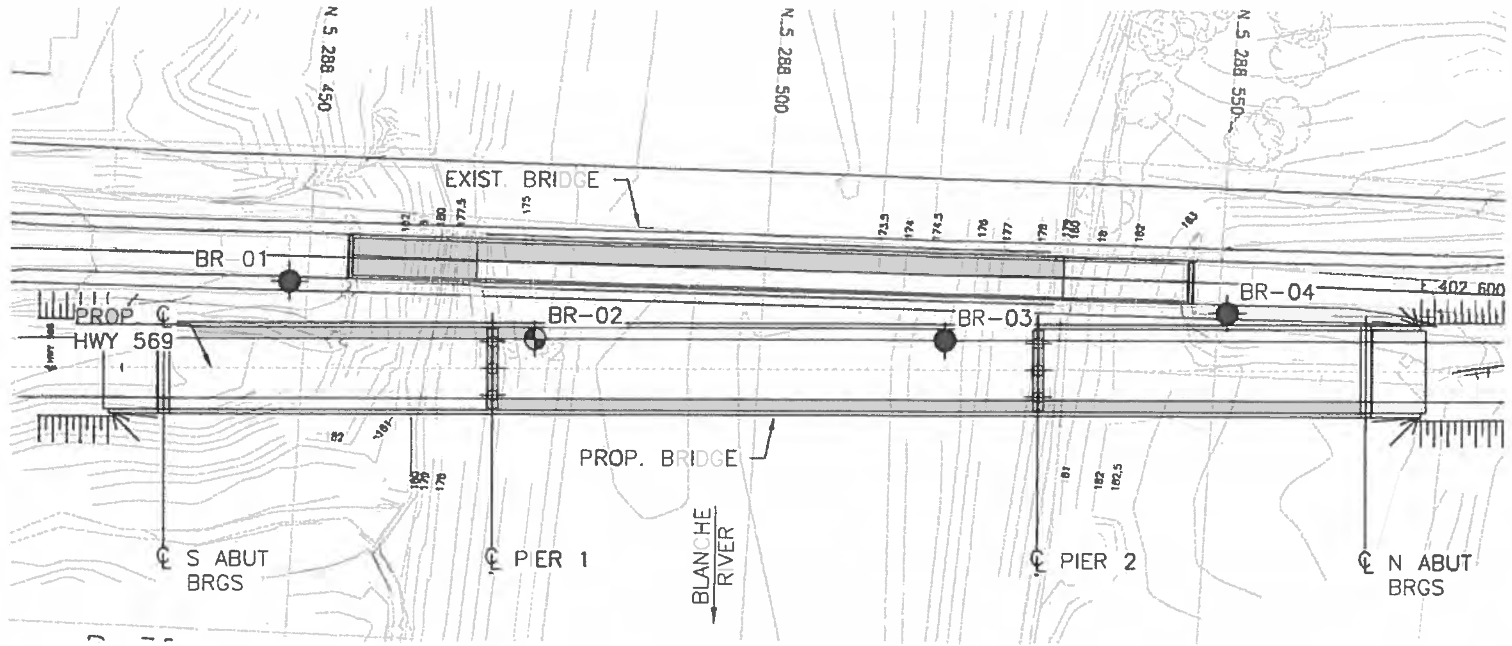
CONT No
WP No 5163-13-00

HIGHWAY 569
BLANCHE RIVER BRIDGE
REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

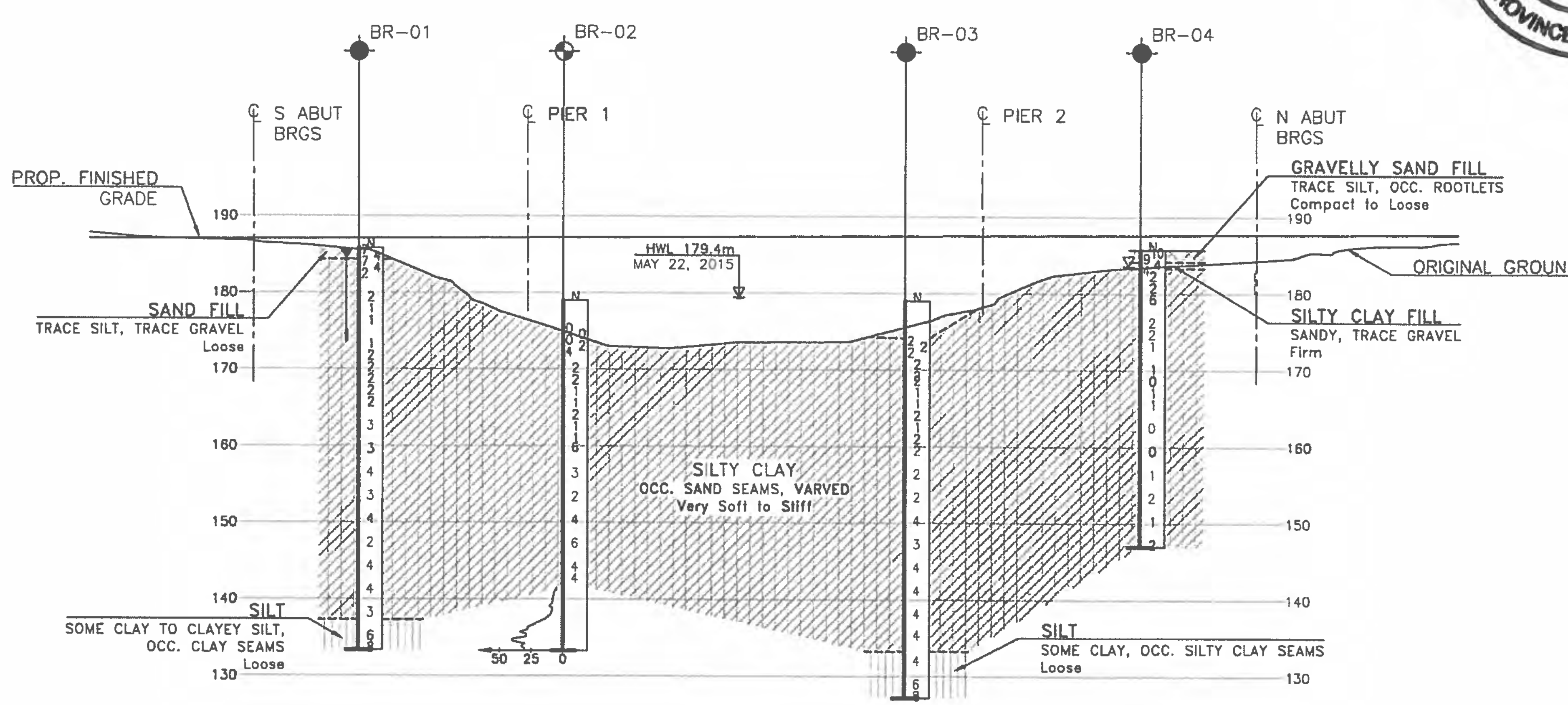
MMM GROUP

THURBER ENGINEERING LTD.



PLAN

SCALE 1:800

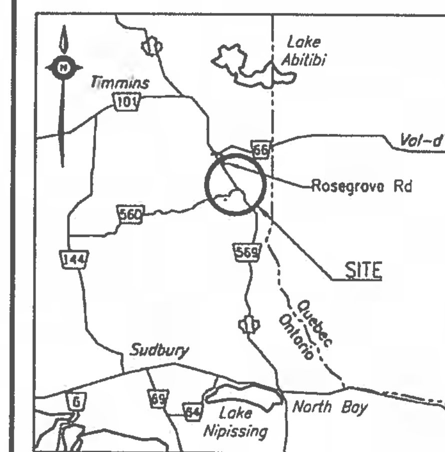


PROFILE ALONG ℄ HWY 569

H 1:800
V 1:200

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
Jun 9/17
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
K. L. SHI
100141639
Jun 9/17
PROVINCE OF ONTARIO



KEYPLAN

LEGEND

⯁	Borehole
⯐	Borehole and Cone
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
▽	Water Level
⫯	Head Artesian Water
	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING (MTM)	EASTING (MTM)
BR-01	185.8	5 288 448.0	402 608.3
BR-02	179.0	5 288 475.4	402 612.8
BR-03	179.0	5 288 520.3	402 610.0
BR-04	185.7	5 288 551.0	402 605.2

-NOTES-

1) The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
2) This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

GEOCRES No. 31M-120

REVISIONS	DATE	BY	DESCRIPTION

DESIGN AMP	CHK KS	CODE	LOAD	DATE JUN 2017	
DRAWN AN	CHK	SITE	STRUCT	DWG 1	

FILENAME: H:\Drafting\19\5161\2658\ted12658-BoreholePlan&Profile (Blanche River).dwg
PLOTDATE: 6/8/2017 1:41 PM

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No BR-01

METRIC

W.P. 5163-13-00 | LOCATION Blanche River Bridge N 5 288 448.0 E 402 608.3 | ORIGINATED BY GA

HWY 569 | BOREHOLE TYPE NW Casing | COMPILED BY AN

DATUM Geodetic | DATE 2015.11.24 - 2015.11.26 | CHECKED BY AMP

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa / SENSITIVITY	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
185.8	GROUND SURFACE						
0.0	**TOPSOIL:** (50mm)						
	SAND, trace silt, trace gravel, Loose, Brown, Moist (FILL)	1	SS	7			
		2	SS	4	185		1 95 4 (SI+CL)
184.4 / 1.4	Silty **CLAY**, occasional sand seams, varved, Firm to Stiff, Grey, Wet	3	SS	7	184		
		4	SS	4	183		0 0 44 56
		5	SS	2	182	+ 4.0	
		1	TW		181		0 0 28 72
		6	SS	2	180 / 179	+ 3.0	
		7	SS	1	178 / 177		
		8	SS	1	176		0 0 20 80

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m^3

Continued Next Page

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

15 ⊕ 5 (20 above, 10 below): (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No BR-01

METRIC

W.P. 5163-13-00
LOCATION Blanche River Bridge N 5 288 448.0 E 402 608.3
ORIGINATED BY GA

HWY 569
BOREHOLE TYPE NW Casing
COMPILED BY AN

DATUM Geodetic
DATE 2015.11.24 - 2015.11.26
CHECKED BY AMP

SOIL PROFILE: ELEV DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) (20 40 60 80 100)	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page										
	Silty **CLAY**, occasional sand seams, varved Firm to Stiff Grey Wet							4.0			
							175				
			2	TW							
							174				
			9	SS	1				○		
							173	5.0			
			10	SS	2		172		○		
							171				
			11	SS	2				○		
							170	5.0			
			12	SS	2		169		○		
							168				
			13	SS	2				○		
							167	4.0			
							166				

Continued Next Page

+ 3, × 3 Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

ONTMT4S 19-5161-265B.GPJ 2015TEMPLATE(MTO).GDT 5/11/16

RECORD OF BOREHOLE No BR-01

METRIC

W.P. 5163-13-00	LOCATION Blanche River Bridge N 5 288 448.0 E 402 608.3		ORIGINATED BY GA
HWY 569	BOREHOLE TYPE NW Casing		COMPILED BY AN
DATUM Geodetic	DATE 2015.11.24 - 2015.11.26		CHECKED BY AMP

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	SHEAR STRENGTH kPa (FIELD VANE)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Silty **CLAY**, occasional sand seams, varved Firm to Stiff Grey Wet	14	SS	2		
		15	SS	3		0 0 35 65
					+ 4.0	
		16	SS	3		
					+ 4.0	
		17	SS	4		

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (165, 164, 163, 162, 161, 160, 159, 158, 157, 156); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3; STRAT PLOT.

Continued Next Page

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

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No BR-01

METRIC

W.P.	5163-13-00	LOCATION	Blanche River Bridge N 5 288 448.0 E 402 608.3	ORIGINATED BY	GA
HWY	569	BOREHOLE TYPE	NW Casing	COMPILED BY	AN
DATUM	Geodetic	DATE	2015.11.24 - 2015.11.26	CHECKED BY	AMP

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa (+ FIELD VANE)	WATER CONTENT (%)	UNIT WEIGHT γ kN/m3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Silty **CLAY**, varved Firm to Stiff Grey Wet							4.0			
							155				
							154				
			18	SS	3						
							153	4.0			
							152				
							151				
			19	SS	4						
							150	3.0			
							149				
							148				
			20	SS	2						
							147	4.0			
							146				

Continued Next Page

+3, ×3 Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No BR-01

METRIC

W.P. 5163-13-00 LOCATION Blanche River Bridge N 5 288 448.0 E 402 608.3 ORIGINATED BY GA

HWY 569 BOREHOLE TYPE NW Casing COMPILED BY AN

DATUM Geodetic DATE 2015.11.24 - 2015.11.26 CHECKED BY AMP

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page							
	Silty **CLAY**, varved Firm to Stiff Grey Wet				145			
		21	SS	4				0 0 32 68
					144	+ 3.0		
					143			
					142			
		22	SS	4				
					141			
					140			
					139			
		23	SS	3				
					138			
137.3								
48.5	**SILT**, some clay to clayey silt, occasional clay seams Loose Grey Wet				137			
					136			

Continued Next Page

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

GROUND WATER CONDITIONS; STRAT PLOT

+³, ×³: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No BR-01

METRIC

W.P.	5163-13-00	LOCATION	Blanche River Bridge N 5 288 448.0 E 402 608.3	ORIGINATED BY	GA
HWY	569	BOREHOLE TYPE	NW Casing	COMPILED BY	AN
DATUM	Geodetic	DATE	2015.11.24 - 2015.11.26	CHECKED BY	AMP

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
	SILT, trace to some clay Loose Grey Wet	24	SS	6	135				
	Silty clay seam at 51.8m depth				134				
133.4 52.4		25	SS	8		0	0	63	37

END OF BOREHOLE AT 52.4m.
BOREHOLE OPEN TO 52.4m AND WATER LEVEL AT 0.6m.
Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.

WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
Nov26/2015	2.0	183.8
Nov27/2015	1.4	184.4
Nov28/2015	1.4	184.4

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³

+ 3, × 3 Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

24-Hour Restrike PDA Test Report

Dynamic Testing of Pile TP1 (1 Day Restrike)

Introduction

The MTO has commissioned Bermingham to undertake a preconstruction test pile program for the proposed replacement of the Blanche River Bridge. Exp was retained by Bermingham to perform a dynamic load test at the end of initial driving on pile TP2.

Separately and with approval from the MTO, Exp initiated to perform a dynamic load test on pile TP1 that is located adjacent to Pile TP2. This work is not part of the contract and results are provided for academic and information purposes only.

Field Work and Construction Records

Pile TP1 was driven to a depth of 44.0 m on September 10, 2018; it consists of three steel HP 310x110 sections, each measuring ~15.2 m (50 feet). Dynamic testing was performed on Pile TP1 on September 11, 2018 using instrumentation from the Pile Driving Analyzer (PDA) and in accordance with ASTM D1143. Dynamic loads for the test were provided by a Berminghammer B-32 open end diesel hammer (rated at 110kJ).

Results of Dynamic Testing

The dynamic test was performed 1 day after pile installation. The first five blows to the pile was delivered with the hammer operating without added fuel. The penetration resistance was reported to be 7 mm for 5 blows (average rebound of 7 mm). The subsequent 5 blows to the pile was delivered, with the fuel was injected and with the hammer operating at speeds of ~50 bpm; the penetration resistance was found to be ~55 mm per 5 blows (average rebound of 6 mm).

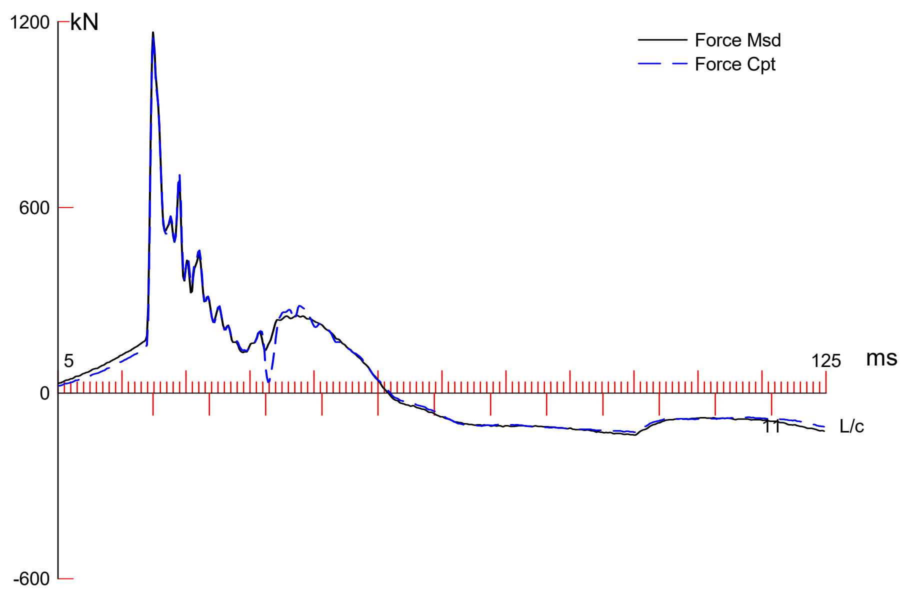
The energy transferred to the top of the pile was 4 kJ, with the hammer operating ***without*** fuel. The maximum force at the instrumentation location was calculated to be 1219 kN, which corresponds to a compressive stress of approximately 87 MPa.

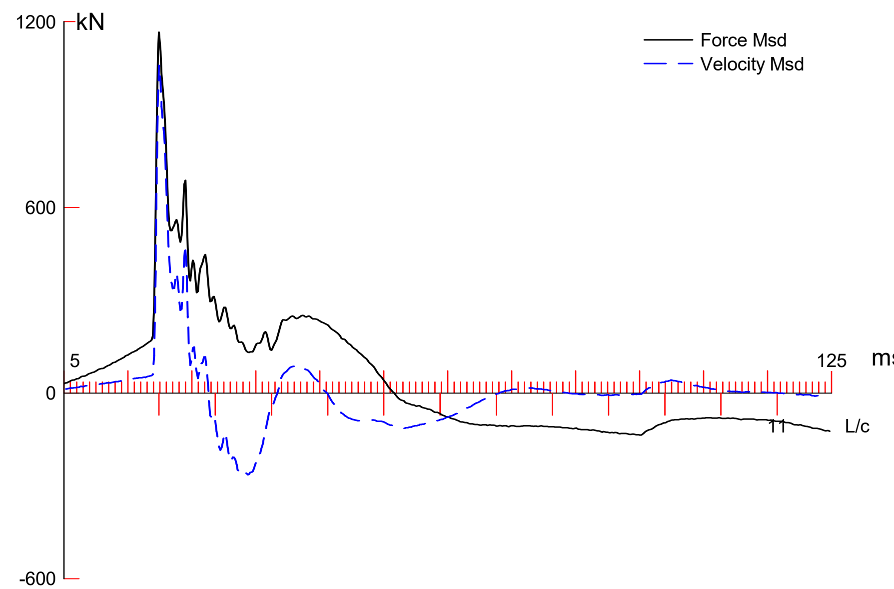
Blow no. 5 was selected for CAPWAP analysis. The mobilized geotechnical resistance of Pile TP1 (1 day Restrike) was evaluated to be 560 kN. The shaft resistance was evaluated to be 520 kN and the toe resistance was evaluated to be 40 kN.

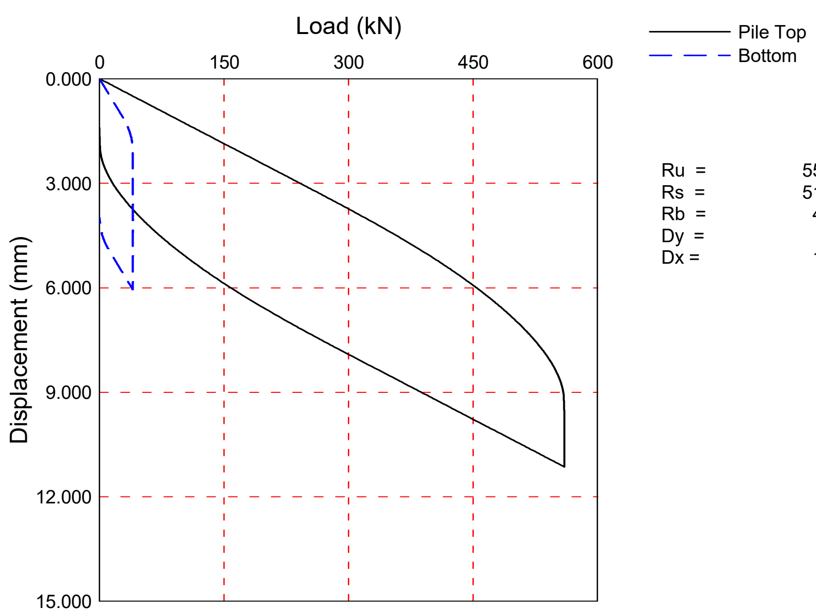
In comparison, the geotechnical resistance of pile TP2, tested at the EOID, was evaluated to be 250 kN. The results provide an indication of the short term increase in geotechnical resistance of HP310x110 at the site. In the longer term, the geotechnical resistance to expected to increase further and this is expected to be proven in the upcoming load tests. The results also attest to the value of the PDA as a means to assist in the planning of construction schedules and contracts, and a means to monitor the performance and control the quality of driven piles.

We trust that the above is acceptable. If there are any questions, please feel free to contact us.

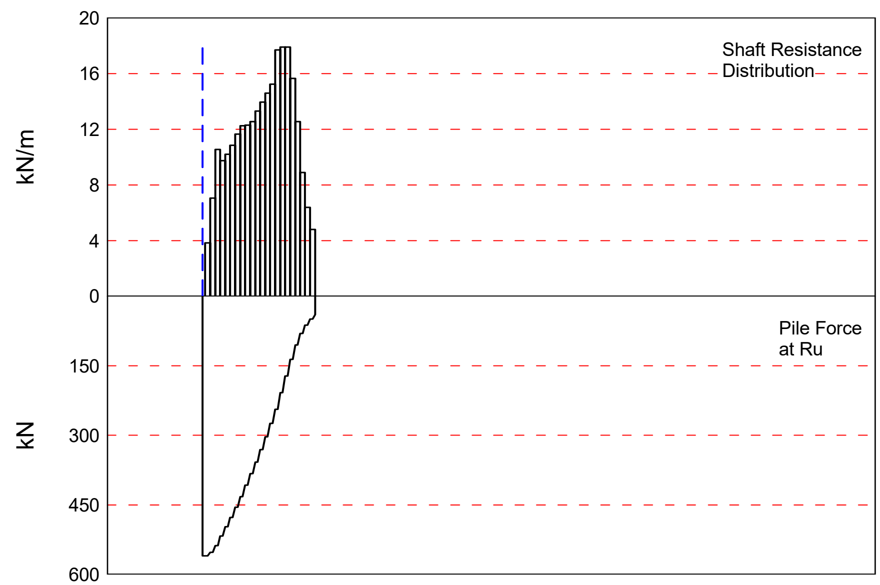
Michael Choy
Exp Services Inc.
Brampton







Ru =	559.8	kN
Rs =	519.8	kN
Rb =	40.0	kN
Dy =	9.7	mm
Dx =	11.1	mm



CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 559.8; along Shaft 519.8; at Toe 40.0 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	Quake mm
				559.8					
1	3.0	2.1	7.7	552.1	7.7	3.67	2.97	1.437	3.751
2	5.0	4.1	14.1	538.0	21.8	7.05	5.70	1.437	3.753
3	7.0	6.1	21.1	516.9	42.9	10.55	8.54	1.437	3.753
4	9.0	8.1	19.5	497.4	62.4	9.75	7.89	1.437	3.753
5	11.0	10.1	20.4	477.0	82.8	10.20	8.25	1.437	3.753
6	13.0	12.1	21.7	455.3	104.5	10.85	8.78	1.437	3.753
7	15.0	14.1	23.3	432.0	127.8	11.65	9.43	1.437	3.753
8	17.0	16.1	24.5	407.5	152.3	12.25	9.91	1.437	3.753
9	19.0	18.1	24.6	382.9	176.9	12.30	9.95	1.437	3.753
10	21.0	20.1	25.1	357.8	202.0	12.55	10.15	1.437	3.753
11	23.0	22.1	26.6	331.2	228.6	13.30	10.76	1.437	3.703
12	25.0	24.1	27.9	303.3	256.5	13.95	11.29	1.437	3.679
13	27.0	26.1	29.2	274.1	285.7	14.60	11.81	1.437	3.691
14	29.0	28.1	30.5	243.6	316.2	15.25	12.34	1.437	3.626
15	31.0	30.1	35.4	208.2	351.6	17.70	14.32	1.437	3.548
16	33.0	32.1	35.8	172.4	387.4	17.90	14.48	1.437	3.444
17	35.0	34.1	35.8	136.6	423.2	17.90	14.48	1.437	3.416
18	37.0	36.1	31.3	105.3	454.5	15.65	12.66	1.437	3.380
19	39.0	38.1	25.1	80.2	479.6	12.55	10.15	1.437	3.355
20	41.0	40.1	17.8	62.4	497.4	8.90	7.20	1.437	3.753
21	43.0	42.1	12.8	49.6	510.2	6.40	5.18	1.437	3.254
22	45.0	44.1	9.6	40.0	519.8	4.80	3.88	1.437	3.225
Avg. Shaft			23.6			11.79	9.54	1.437	3.613
Toe			40.0				418.94	1.436	1.577

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		1.312	0.101
Unloading Quake	(% of loading quake)	58	30
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	76	
Resistance Gap (included in Toe Quake) (mm)			0.573
Soil Plug Weight	(kN)		1.04

CAPWAP match quality = 1.95 (Wave Up Match) ; RSA = 0
Observed: final set = 1.400 mm; blow count = 714 b/m
Computed: final set = 1.655 mm; blow count = 604 b/m

max. Top Comp. Stress = 81.7 MPa (T= 20.3 ms, max= 1.011 x Top)
max. Comp. Stress = 82.6 MPa (Z= 3.0 m, T= 20.7 ms)
max. Tens. Stress = -12.97 MPa (Z= 40.0 m, T= 30.3 ms)
max. Energy (EMX) = 4.21 kJ; max. Measured Top Displ. (DMX)= 6.37 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	1152.6	-126.5	81.7	-8.97	4.21	1.9	6.679
2	2.0	1158.9	-126.8	82.2	-8.99	4.18	1.9	6.547
5	5.0	1161.1	-121.9	82.3	-8.64	4.01	1.8	6.141
8	8.0	1095.4	-96.1	77.7	-6.82	3.54	1.7	5.734
11	11.0	1075.8	-89.9	76.3	-6.38	3.26	1.7	5.331
14	14.0	999.6	-83.0	70.9	-5.89	2.82	1.6	4.940
17	17.0	978.4	-58.0	69.4	-4.11	2.56	1.5	4.565
20	20.0	895.6	-45.6	63.5	-3.23	2.17	1.4	4.233
23	23.0	874.9	-86.1	62.0	-6.10	1.97	1.3	3.937
26	26.0	792.8	-94.8	56.2	-6.72	1.66	1.3	3.984
29	29.0	770.0	-31.5	54.6	-2.23	1.51	1.2	3.980
32	32.0	681.1	-136.4	48.3	-9.67	1.19	1.1	3.928
35	35.0	653.1	-139.5	46.3	-9.89	1.03	1.0	3.854
38	38.0	565.4	-179.3	40.1	-12.71	0.74	1.0	3.837
39	39.0	574.1	-138.3	40.7	-9.81	0.73	1.0	3.833
40	40.0	537.6	-182.9	38.1	-12.97	0.65	0.9	3.835
41	41.0	543.4	-175.3	38.5	-12.44	0.63	0.9	3.837
42	42.0	518.3	-180.5	36.8	-12.80	0.55	0.9	3.864
43	43.0	523.3	-136.2	37.1	-9.66	0.49	1.2	3.877
44	44.0	460.7	-96.7	32.7	-6.86	0.42	1.3	3.887
45	45.0	307.5	-40.0	21.8	-2.84	0.33	1.4	3.901
Absolute	3.0			82.6			(T =	20.7 ms)
	40.0				-12.97		(T =	30.3 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	1255.8	1149.0	1042.2	935.4	828.5	721.7	614.9	508.1	401.3	294.4
RX	1255.8	1149.0	1042.2	935.4	828.5	721.7	614.9	508.1	401.3	294.4
RU	1410.3	1318.9	1227.5	1136.1	1044.8	953.4	862.0	770.6	679.3	587.9

RAU = 211.9 (kN); RA2 = 370.0 (kN)

Current CAPWAP Ru = 559.8 (kN); Corresponding J(RP)= 0.65; J(RX) = 0.65

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
m/s	ms	kN	kN	kN	mm	mm	mm	kJ	kN
1.94	20.11	1104.8	1219.2	1219.2	6.374	1.566	1.400	4.3	1108.3

PILE PROFILE AND PILE MODEL

Depth	Area	E-Modulus	Spec. Weight	Perim.
m	cm^2	MPa	kN/m^3	m
0.00	141.00	206842.7	77.287	1.236
45.00	141.00	206842.7	77.287	1.236

Toe Area 0.095 m^2

Top Segment Length 1.00 m, Top Impedance 569.30 kN/m/s

Pile Damping 1.0 %, Time Incr 0.195 ms, Wave Speed 5123.0 m/s, 2L/c 17.6 ms

3-Week Restrike PDA Test Report

Bermingham Foundation Solutions
600 Ferguson Avenue North
Hamilton, Ontario
L8L 4Z9

Pile Load Test Program
~ High-Strain Dynamic Test of Pile TP 2
(3 Week Restrike)
Proposed Replacement of Blanche River Bridge
Highway 569 in New Liskeard, Ontario

Project Number
BRM- 607254-A0

Prepared By:

exp Services Inc.
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Brampton, Ontario L6T 4V1
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Facsimile: (905) 793-0641

Date Submitted
2018-10-23

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Appendices

1 Introduction

The Ministry of Transportation of Ontario (MTO) has commissioned Bermingham Foundation Solutions (Bermingham) to carry out a pre-construction pile load test program for the proposed replacement of the Blanche River Bridge (MTO Structure No. 47-38), located approximately 150 m north of the intersection of Hilliardton Road and Highway 569, in the town of Hilliardton near the district of New Liskeard.

The primary purpose of the program is to examine the geotechnical resistances of steel pile foundations that have been proposed for the planned bridge. The area of the test pile program is the south-east corner of the bridge. The methods for examination include a series of two static load tests of one pile (TP1) and a series of three dynamic load tests on the second, adjacent pile (TP2).

Exp Services Inc. (EXP) was retained by Bermingham to monitor the Static Load Testing of Pile TP1 and to perform dynamic testing of the Pile TP2.

In this report is presented the results of the dynamic test that was carried out at the beginning of restrike (BOR) of Pile TP2. The test was performed on October 4, 2018 (i.e. 23 days after the end of initial driving (EOID)); it is the second of a series of three (3) dynamic load tests that have been planned. The results of the earlier dynamic load test have been published under a separate cover.

Pile TP2 is a 310 mm x 110 kg/m steel HP-section and was monitored using the Pile Driving Analyzer (PDA). It is made up of three 15.24 m (50 feet) long steel sections and driven to an embedded depth of 44.0 m. Dynamic load to the pile was provided by a Berminghammer B-32 open end diesel hammer. The manufacturer's maximum rated energy of the hammer is 110 kJ.

Subsurface characteristics of the site is described in the project Foundation Investigation Report (report Geocres No. 31M-120), and indicates that the site is underlain by a deposit of soft to firm varved clay that is more than 50 m thick. A the request of the Owner, Bermingham had also commissioned cone penetrometer testing (CPTu) at the site. Full results were published by others under a separate cover and an extract is appended to the report for reference.

2 Fieldwork and Engineering Analysis

On October 4, 2018, pile TP2 was monitored at the Beginning of Restrike (BOR) with the instrumentation from the Pile Driving Analyzer (PDA) attached. The primary purpose of the test was to examine the geotechnical resistance of the pile after a time equivalent to 3 weeks after pile installation. Pile TP2 was originally driven to a depth of 44.0 m using a Berminghammer B-32 open end diesel hammer (110 kJ rated energy) on September 11, 2018.

High-strain dynamic testing using the PDA was undertaken in general accordance with the ASTM D4945-12 procedures. The instrumentation for the PDA 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 PAX/PAK) 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 hammer blow from the end of initial driving of the tested pile was used to perform signal matching analyses using CAPWAP (CAse Pile Wave Analysis Program) in order to evaluate the ultimate resistance 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 also 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 fieldwork was carried out on October 4, 2018; Pile TP2 (HP 310x110) was instrumented with strain gauges and accelerometers, and monitored at the beginning of restrike with the Pile Driving Analyzer (PDA), and using a Berminghammer B-32 open ended diesel hammer (rated 110 kJ).

The purpose of the test was to determine the ultimate geotechnical resistance of Pile TP2 at the beginning of restrike, at approximately 23 days the end of initial drive on September 11.

The results of the dynamic testing are presented on Table No. 1, and together with field observations are summarized below.

Prior to the start of the test, the diesel hammer was warmed up by driving a warm-up pile that was located some 2 m away from pile TP2. The warm-up pile is a pipe pile (406 mm dia. 9.5 mm thick) and was advanced over 3 m.

The energy transferred to the top of the pile TP2 was approximately 24 kJ, with the hammer operating at 46 blows per minute (bpm). The penetration resistance was 2 mm per blow, with a temporary compression of approx. 7 mm.

The maximum compressive force at the instrumentation location was calculated to be 2681 kN, which corresponds to a compressive stress of 190 MPa.

At the end of the test, pile TP2 had been advanced to a final depth of approximately 44.3 m. The mobilized geotechnical resistance of Pile TP2 at the beginning of the 23-day restrike was evaluated to be 1360 kN.

3.2 **Signal Matching Analysis Results**

Signal matching analyses using CAPWAP was undertaken on a selected hammer blow from the EOID of Pile TP2. The Case Method Capacities and Pile Profile and Model tables, CAPWAP Force matches, Force-Velocity Wave forms, Resistance Distributions, and Simulated Compression Load Test Curves are presented in Appendix A.

The ultimate geotechnical resistance at the EOID of Pile TP2 was evaluated to be 1360 kN, of which 1316 kN was evaluated as shaft resistance and 44 kN was evaluated as toe resistance.

4 Summary

As part of a preconstruction test pile program for the proposed replacement of Blanche River Bridge on Highway 569, a dynamic load test using the PDA was carried out on Test Pile TP2 at the beginning of the 23-day restrike (BOR_{23}). Dynamic load was provided by a diesel hammer (Berminghammer B-32, rated at 110kJ). The 23-day restrike test is the second the series of three dynamic load tests. The third dynamic load test is planned for November 2018.

Pile TP2 (HP310x110) was originally driven to an embedment depth of 44 m below grade on September 11, 2018 and then dynamically tested at the end of initial driving (EOID) using the PDA. The penetration resistance at the EOID was reported to be 50 mm per blow with the hammer operating at ~43 bpm; the transferred energy to the pile was ~34 kJ. The ultimate geotechnical resistance at the EOID was evaluated to be 250 kN.

An adjacent test pile, Pile TP1 (HP310x110, 44 m long), was been driven at about the same time as pile TP2; it was dynamically tested at the beginning of its 1-day restrike (BOR_1) with equipment from the PDA. The ultimate geotechnical resistance at the BOR_1 was evaluated to be 560 kN.

On October 4, 2018, Pile TP2 was dynamically load tested at the beginning of the 23-day restrike. The penetration resistance was reported to be 2 mm per blow with the hammer operating at ~46 bpm and the transferred energy to the pile was ~24 kJ. The mobilized geotechnical resistance was evaluated to be 1360 kN, of which over 90% of the resistance was evaluated to be shaft resistance.

It should be noted that the evaluated geotechnical resistance of TP2 reflects the resistance at the time of testing i.e. *at the beginning of the 23-day restrike*. Given that the pile is predominantly embedded in clay it is possible that the resistance may continue to increase further with time.

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.

Michael W.K. Choy, P. Eng
Senior Geotechnical Engineer

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

Table 1 – Summary of Pile Driving Analyzer Results

PILE NO.	EVENT	HAMMER	DEPTH BELOW GRADE (m)	REPORTED PENETRATION RESISTANCE DURING TEST		TRANSFERRED ENERGY		FORCE		EVALUATED ULT. GEOTECHNICAL RESISTANCE	REMARKS
				Set (mm per blow)	Equiv. Blows/ 25 mm	Mean (kJ)	Speed (blows / min)	Max. (kN)	Stress (MPa)	(kN)	
Date of Test : September 11, 2018											
TP2	Beginning of Restrike (23 day)	Berminghammer B-32	~ 44.2	2 mm	12 to 13 blows	23.8	46.4	2681	190.1	1360	Length below PDA sensors = 45.0 m. Length below grade after testing 44.3 m

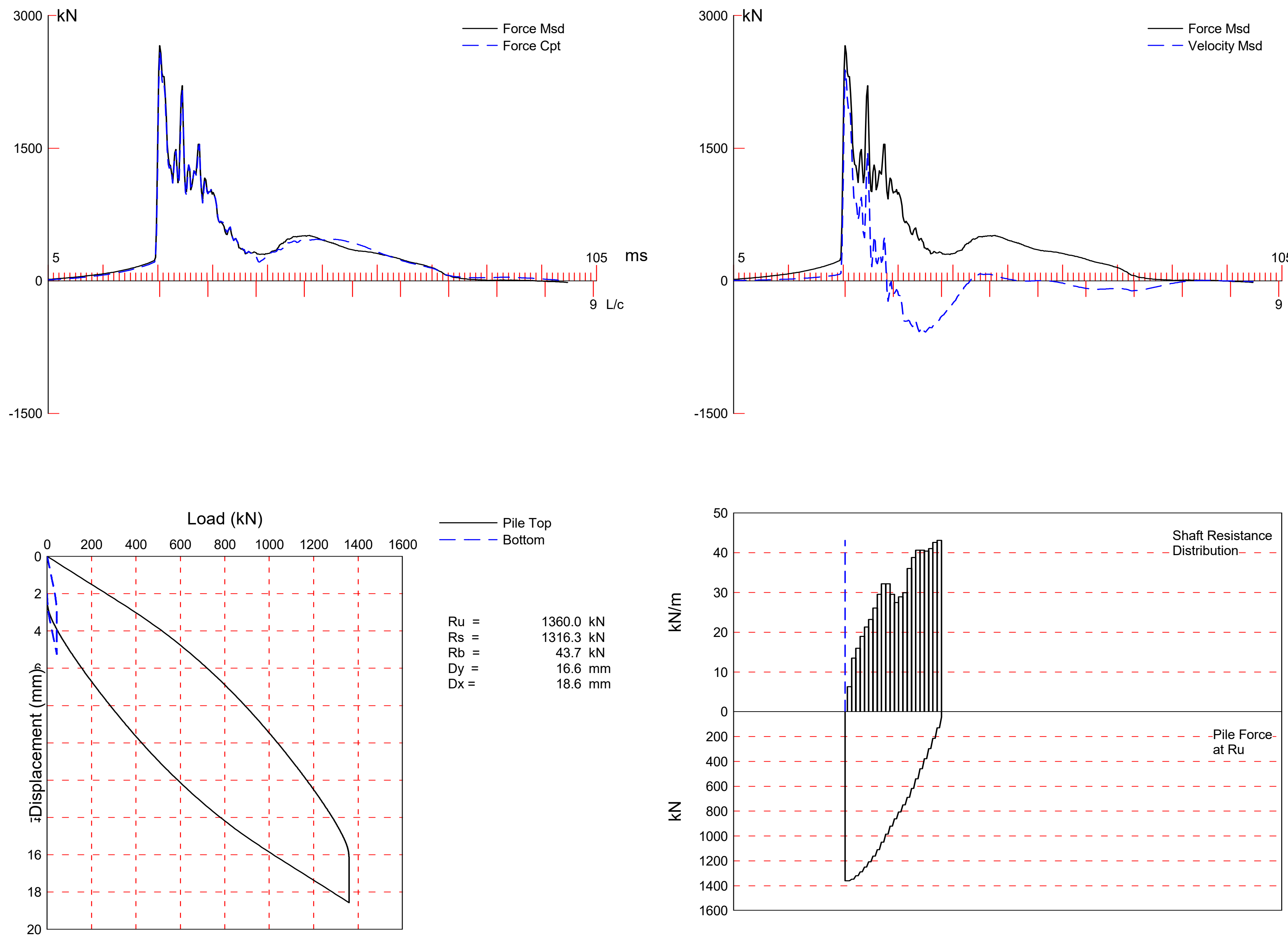
Notes:

1. Existing ground level is understood to be approx. 186.5 m.

Table 2 - Summary of Signal Matching (CAPWAP) Analysis Results

PILE NO.	EVENT	LENGTH BELOW GAUGES/SENSORS	REPORTED PENETRATION RESISTANCE (blows/mm)	EVALUATED ULT. (MOB) GEOTECHNICAL RESISTANCE		
				Total	Shaft	Toe
TP2	Beginning of Restrike (23 day)	45.0 m	1 blow / 2 mm	1360 kN	1316 kN	44 kN

Appendix A
CAPWAP Tables and Figures
for Test at Pile TP2
Beginning of Restrike (23-Day)
October 4, 2018



Ru = 1360.0 kN
Rs = 1316.3 kN
Rb = 43.7 kN
Dy = 16.6 mm
Dx = 18.6 mm

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 1360.0; along Shaft 1316.3; at Toe 43.7 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	Quake mm
				1360.0					
1	3.0	2.2	12.6	1347.4	12.6	5.73	4.63	1.231	2.500
2	5.0	4.2	27.0	1320.4	39.6	13.50	10.92	1.231	2.500
3	7.0	6.2	31.9	1288.5	71.5	15.95	12.90	1.231	2.500
4	9.0	8.2	37.9	1250.6	109.4	18.95	15.33	1.231	2.500
5	11.0	10.2	42.6	1208.0	152.0	21.30	17.23	1.231	2.500
6	13.0	12.2	46.4	1161.6	198.4	23.20	18.77	1.231	2.500
7	15.0	14.2	52.2	1109.4	250.6	26.10	21.12	1.231	2.500
8	17.0	16.2	59.1	1050.3	309.7	29.55	23.91	1.231	2.500
9	19.0	18.2	64.3	986.0	374.0	32.15	26.01	1.231	2.500
10	21.0	20.2	64.3	921.7	438.3	32.15	26.01	1.231	2.500
11	23.0	22.2	59.1	862.6	497.4	29.55	23.91	1.231	2.500
12	25.0	24.2	54.9	807.7	552.3	27.45	22.21	1.231	2.500
13	27.0	26.2	57.8	749.9	610.1	28.90	23.38	1.231	2.500
14	29.0	28.2	59.9	690.0	670.0	29.95	24.23	1.231	2.500
15	31.0	30.2	72.0	618.0	742.0	36.00	29.13	1.231	2.500
16	33.0	32.2	77.6	540.4	819.6	38.80	31.39	1.231	2.500
17	35.0	34.2	81.3	459.1	900.9	40.65	32.89	1.231	2.500
18	37.0	36.2	81.2	377.9	982.1	40.60	32.85	1.231	2.500
19	39.0	38.2	80.8	297.1	1062.9	40.40	32.69	1.231	2.500
20	41.0	40.2	82.1	215.0	1145.0	41.05	33.21	1.231	2.274
21	43.0	42.2	85.1	129.9	1230.1	42.55	34.43	1.231	2.012
22	45.0	44.2	86.2	43.7	1316.3	43.10	34.87	1.231	1.882
Avg. Shaft			59.8			29.78	24.09	1.231	2.414
Toe			43.7				457.69	1.303	2.500

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		2.847	0.100
Unloading Quake	(% of loading quake)	30	30
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	0	
Soil Plug Weight	(kN)		1.33

CAPWAP match quality = 2.19 (Wave Up Match) ; RSA = 0
Observed: final set = 2.000 mm; blow count = 500 b/m
Computed: final set = 2.592 mm; blow count = 386 b/m

max. Top Comp. Stress = 183.8 MPa (T= 25.8 ms, max= 1.013 x Top)
max. Comp. Stress = 186.2 MPa (Z= 3.0 m, T= 26.2 ms)
max. Tens. Stress = -3.15 MPa (Z= 44.0 m, T= 39.0 ms)
max. Energy (EMX) = 23.60 kJ; max. Measured Top Displ. (DMX)=13.59 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	2591.5	0.0	183.8	0.00	23.60	4.3	14.246
2	2.0	2605.9	0.0	184.8	0.00	23.38	4.3	13.861
5	5.0	2618.6	0.0	185.7	0.00	22.09	4.1	12.695
8	8.0	2402.3	0.0	170.4	0.00	18.96	3.8	11.543
11	11.0	2349.8	0.0	166.7	0.00	16.97	3.5	10.416
14	14.0	2065.1	0.0	146.5	0.00	13.68	3.3	9.318
17	17.0	2003.3	0.0	142.1	0.00	11.80	2.9	8.251
20	20.0	1686.0	0.0	119.6	0.00	8.79	2.6	7.251
23	23.0	1596.1	0.0	113.2	0.00	7.29	2.3	6.303
26	26.0	1355.0	0.0	96.1	0.00	5.45	2.1	5.522
29	29.0	1296.7	0.0	92.0	0.00	4.66	1.9	4.936
32	32.0	1096.0	0.0	77.7	0.00	3.37	1.7	4.351
35	35.0	1029.9	0.0	73.0	0.00	2.70	1.5	3.785
38	38.0	825.9	0.0	58.6	0.00	1.74	1.3	3.443
39	39.0	848.9	0.0	60.2	0.00	1.72	1.2	3.331
40	40.0	750.1	0.0	53.2	0.00	1.34	1.2	3.230
41	41.0	771.8	0.0	54.7	0.00	1.33	1.1	3.134
42	42.0	678.8	-16.7	48.1	-1.18	0.99	1.1	3.057
43	43.0	708.9	0.0	50.3	0.00	0.99	1.0	2.979
44	44.0	613.7	-44.4	43.5	-3.15	0.64	1.2	2.933
45	45.0	484.3	0.0	34.3	0.00	0.20	1.4	2.913
Absolute	3.0			186.2			(T =	26.2 ms)
	44.0				-3.15		(T =	39.0 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RX	2909.1	2691.3	2473.5	2255.7	2037.9	1820.1	1602.3	1384.4	1166.6	948.8
RU	3348.8	3174.9	3001.1	2827.2	2653.4	2479.5	2305.7	2131.8	1958.0	1784.1

RAU =   434.3 (kN); RA2 =   945.1 (kN)

Current CAPWAP Ru = 1360.0 (kN); J(RX) = 0.71

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
m/s	ms	kN	kN	kN	mm	mm	mm	kJ	kN
4.27	25.57	2428.8	2658.5	2680.7	13.591	2.000	2.000	23.8	3055.4

Possible Pile Damage at 0.26 L Below Gages?

PILE PROFILE AND PILE MODEL

Depth	Area	E-Modulus	Spec. Weight	Perim.
m	cm^2	MPa	kN/m^3	m
0.00	141.00	206842.7	77.287	1.236
45.00	141.00	206842.7	77.287	1.236

Toe Area   0.095 m^2

Top Segment Length   1.00 m, Top Impedance   569.29 kN/m/s

Pile Damping   1.0 %, Time Incr 0.195 ms, Wave Speed   5123.0 m/s, 2L/c 17.6 ms

8-Week Restrike PDA Test Report

Bermingham Foundation Solutions
600 Ferguson Avenue North
Hamilton, Ontario
L8L 4Z9

Proposed Replacement of Blanche River Bridge
Highway 569 in New Liskeard, Ontario
Pile Load Test Program
~ High-Strain Dynamic Test of Pile TP 2
(8 Week Restrike)

Project Number
BRM- 607254-A0

Prepared By:

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

Date Submitted
2018-11-27

Table of Contents

Tables

Appendices

1 Introduction

The Ministry of Transportation of Ontario (MTO) has commissioned Bermingham Foundation Solutions (Bermingham) to carry out a pre-construction pile load test program for the proposed replacement of the Blanche River Bridge (MTO Structure No. 47-38), located approximately 150 m north of the intersection of Hilliardton Road and Highway 569, in the town of Hilliardton near the district of New Liskeard.

The primary purpose of the program is to examine the geotechnical resistances of steel pile foundations that have been proposed for the planned bridge. The area of the test pile program is the south-east corner of the bridge. The methods for examination include a series of two static load tests of one pile (TP1) and a series of three dynamic load tests on the second, adjacent pile (TP2).

Exp Services Inc. (EXP) was retained by Bermingham to monitor the Static Load Testing of Pile TP1 and to perform dynamic testing of the Pile TP2.

In this report is presented the results of the dynamic test that was carried out at the beginning of restrike (BOR) of Pile TP2. The test was performed on November 9, 2018 (i.e. approximately 3 weeks after the end of initial driving (EOID)); it is the third of a series of three (3) dynamic load tests that had been planned. The results of the earlier dynamic load tests have been published under separate covers.

Pile TP2 is a 310 mm x 110 kg/m steel HP-section and was monitored using the Pile Driving Analyzer (PDA). It is made up of three 15.24 m (50 feet) long steel sections and driven to an embedded depth of 44.0 m. Dynamic load to the pile was provided by a Berminghammer B-32 open end diesel hammer. The manufacturer's maximum rated energy of the hammer is 110 kJ.

Subsurface characteristics of the site is described in the project Foundation Investigation Report (report Geocres No. 31M-120), and indicates that the site is underlain by a deposit of soft to firm varved clay with a thickness that is greater than 50 m. At the request of the Owner, Bermingham had also commissioned cone penetrometer testing at the site. Full results were published by others under a separate cover and an extract is appended to the report for reference.

2 Fieldwork and Engineering Analysis

On November 9, 2018, pile TP2 was monitored at the Beginning of Restrike (BOR) with the instrumentation from the Pile Driving Analyzer (PDA) attached. The primary purpose of the test was to examine the geotechnical resistance of the pile at approximately 8 weeks after pile installation. Pile TP2 was originally driven to a depth of 44.0 m using a Berminghammer B-32 open end diesel hammer (110 kJ rated energy) on September 11, 2018.

High-strain dynamic testing using the PDA was undertaken in general accordance with the ASTM D4945-12 procedures. The instrumentation for the PDA 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 PAX/PAK) 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 hammer blow from the end of initial driving of the tested pile was used to perform signal matching analyses using CAPWAP (CAse Pile Wave Analysis Program) in order to evaluate the ultimate resistance 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

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 also 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 fieldwork was carried out on November 9, 2018; Pile TP2 (HP 310x110) was instrumented with strain gauges and accelerometers, and monitored at the beginning of restrike with the Pile Driving Analyzer (PDA), and using a Berminghammer B-32 open ended diesel hammer (rated 110 kJ).

The purpose of the test was to determine the ultimate geotechnical resistance of Pile TP2 at the beginning of restrike, at approximately 8 weeks after the end of initial drive on September 11, 2018.

The results of the dynamic testing are presented on Table No. 1, and together with field observations are summarized below.

Prior to the start of the test, the diesel hammer was warmed up by driving a warm-up pile that was located some 2 m away from pile TP2. The warm-up pile is a pipe pile (406 mm dia. 9.5 mm thick) and was advanced over 3 m.

The energy transferred to the top of the pile TP2 was approximately 23 kJ, with the hammer operating at 46 blows per minute (bpm). The penetration resistance was ~1.5 mm per blow.

The maximum compressive force at the instrumentation location was calculated to be 2640 kN, which corresponds to a compressive stress of 187 MPa.

The mobilized geotechnical resistance of Pile TP2 at the beginning of the 23-day restrike was evaluated to be 1600 kN.

3.2 Signal Matching Analysis Results

Signal matching analyses using CAPWAP was undertaken on a selected hammer blow from the BOR of Pile TP2. The Case Method Capacities and Pile Profile and Model tables, CAPWAP Force matches, Force-Velocity Wave forms, Resistance Distributions, and Simulated Compression Load Test Curves are presented in Appendix A.

The ultimate geotechnical resistance at the EOID of Pile TP2 was evaluated to be 1600 kN, of which 1557 kN was evaluated as shaft resistance and 43 kN was evaluated as toe resistance.

4 Summary

As part of a preconstruction test pile program for the proposed replacement of Blanche River Bridge on Highway 569, a dynamic load test using the PDA was carried out on Test Pile TP2 at the beginning of the restrike at approximately 8 weeks after the pile was installed. Dynamic load was provided by a diesel hammer (Berminghammer B-32, rated at 110kJ). The 8-week restrike test is the third of a series of three dynamic load tests.

Pile TP2 (HP310x110) was originally driven to an embedment depth of 44 m below grade on September 11, 2018 and then dynamically tested at the end of initial driving (EOID) and at restrike after approximately three weeks. The ultimate geotechnical resistances were evaluated to be 250 kN and 1360 kN, respectively.

On November 9, 2018, Pile TP2 was dynamically load tested at the beginning of the restrike. The penetration resistance was reported to be ~1.5 mm per blow with the hammer operating at ~46 bpm and the transferred energy to the pile was ~23 kJ. The mobilized geotechnical resistance was evaluated to be 1600 kN, of which over 90% of the resistance was evaluated to be shaft resistance.

It should be noted that the evaluated geotechnical resistance of TP2 reflects the resistance at the time of testing i.e. *at the beginning of the 8-week restrike*. Given that the pile is predominantly embedded in clay it is possible that the resistance may continue to increase further with time.

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.

Michael W.K. Choy, P. Eng
Senior Geotechnical Engineer

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

Table 1 – Summary of Pile Driving Analyzer Results

PILE NO.	EVENT	HAMMER	DEPTH BELOW GRADE (m)	REPORTED PENETRATION RESISTANCE DURING TEST		TRANSFERRED ENERGY		FORCE		EVALUATED ULT. GEOTECHNICAL RESISTANCE	REMARKS
				Set (mm per blow)	Equiv. Blows/ 25 mm	Mean (kJ)	Speed (blows / min)	Max. (kN)	Stress (MPa)	(kN)	
Date of Test : September 11, 2018											
TP2	Beginning of Restrike (8-week)	Berminghammer B-32	~ 44.3	~1.5 mm	12 to 13 blows	23	46	2640	187	1600	Length below PDA sensors = 45.0 m.

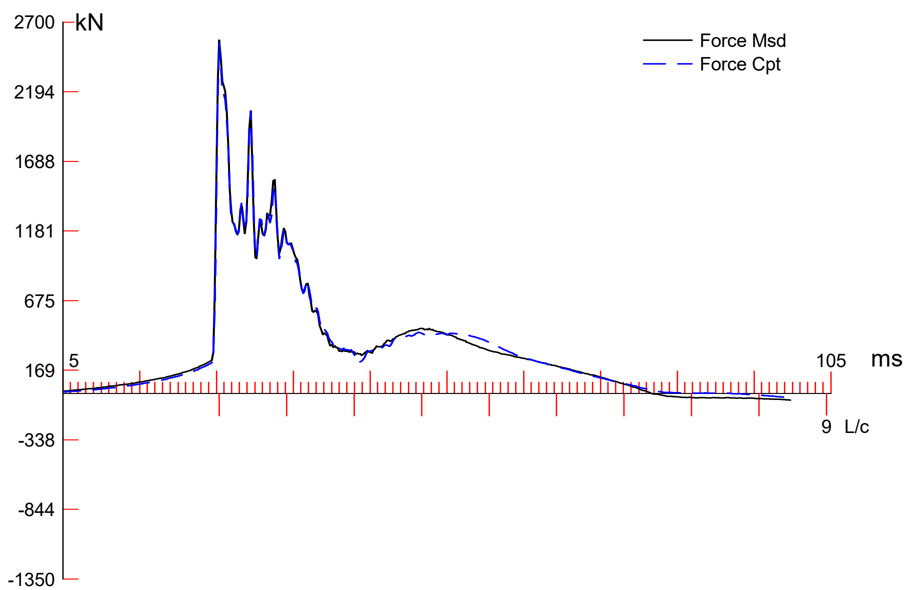
Notes:

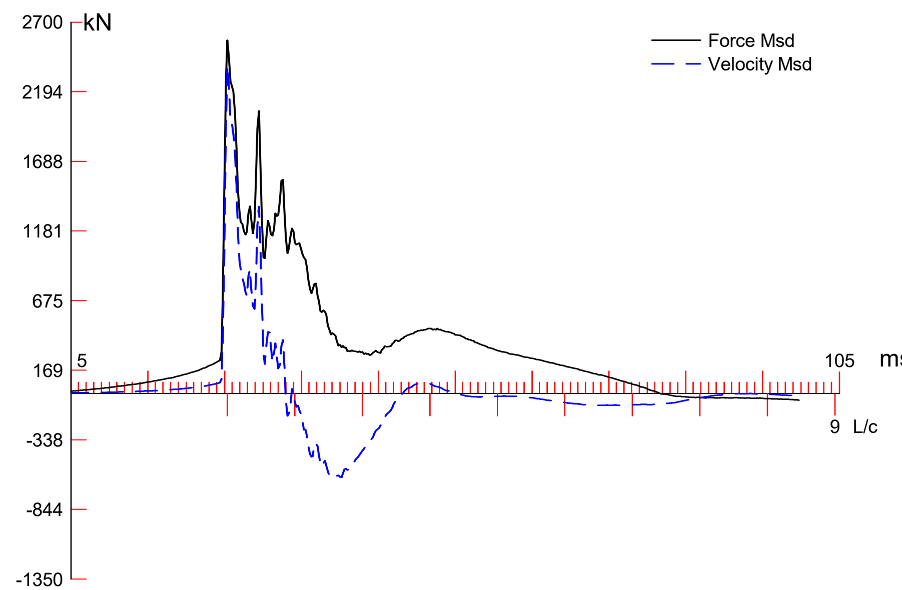
1. Existing ground level is understood to be approx. 186.5 m.

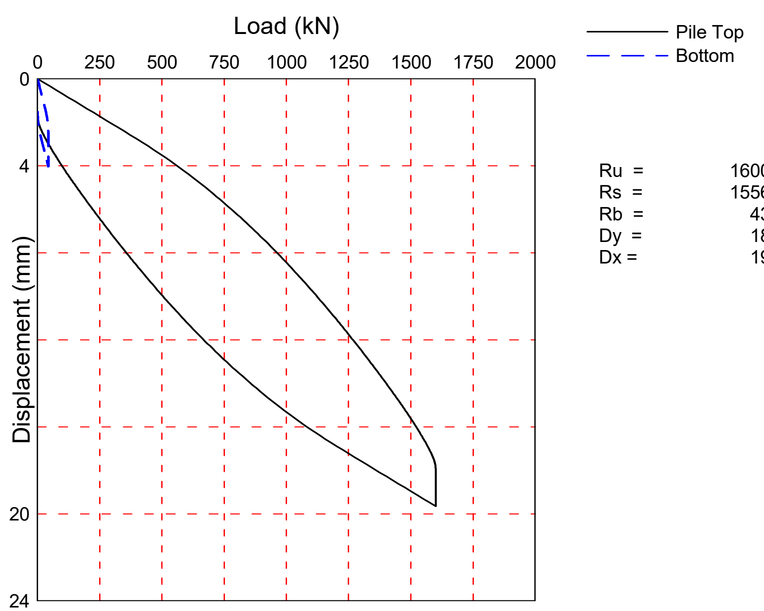
Table 2 - Summary of Signal Matching (CAPWAP) Analysis Results

PILE NO.	EVENT	LENGTH BELOW GAUGES/SENSORS	REPORTED PENETRATION RESISTANCE (blows/mm)	EVALUATED ULT. (MOB) GEOTECHNICAL RESISTANCE		
				Total	Shaft	Toe
TP2	Beginning of Restrike (8-week)	45.0 m	1 blow / 1.5 mm	1600 kN	1557 kN	43 kN

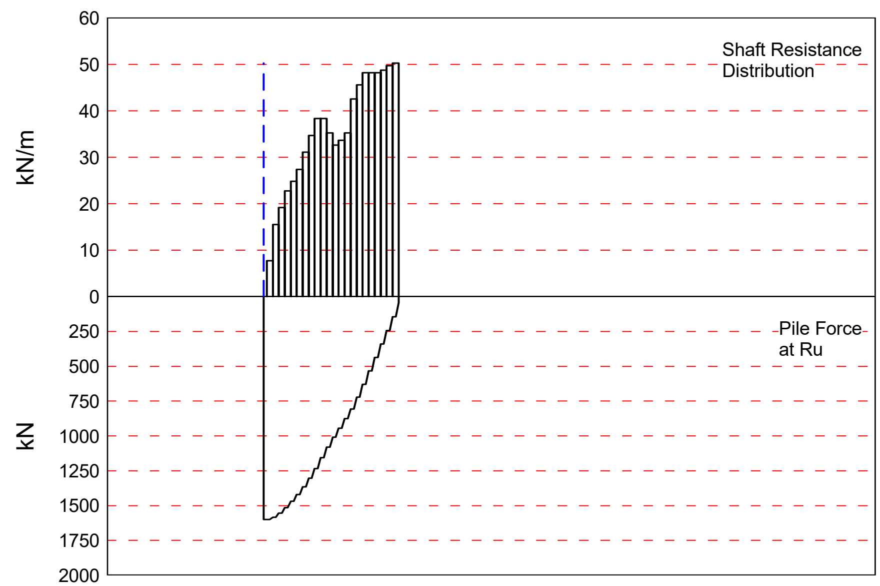
Appendix A
CAPWAP Tables and Figures
for Test at Pile TP2
Beginning of Restrike (8 Week)
November 9, 2018







Ru =	1600.0	kN
Rs =	1556.9	kN
Rb =	43.1	kN
Dy =	18.1	mm
Dx =	19.6	mm



CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 1600.0; along Shaft 1556.9; at Toe 43.1 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	Quake mm
				1600.0					
1	3.0	2.4	15.5	1584.5	15.5	6.46	5.23	1.058	2.370
2	5.0	4.4	31.1	1553.4	46.6	15.55	12.58	1.058	2.370
3	7.0	6.4	38.4	1515.0	85.0	19.20	15.53	1.058	2.370
4	9.0	8.4	45.6	1469.4	130.6	22.80	18.45	1.058	2.370
5	11.0	10.4	49.7	1419.7	180.3	24.85	20.11	1.058	2.370
6	13.0	12.4	54.8	1364.9	235.1	27.40	22.17	1.058	2.370
7	15.0	14.4	62.2	1302.7	297.3	31.10	25.16	1.058	2.370
8	17.0	16.4	69.4	1233.3	366.7	34.70	28.07	1.058	2.370
9	19.0	18.4	76.7	1156.6	443.4	38.35	31.03	1.058	2.370
10	21.0	20.4	76.7	1079.9	520.1	38.35	31.03	1.058	2.370
11	23.0	22.4	70.5	1009.4	590.6	35.25	28.52	1.058	2.370
12	25.0	24.4	65.3	944.1	655.9	32.65	26.42	1.058	2.370
13	27.0	26.4	67.4	876.7	723.3	33.70	27.27	1.058	2.370
14	29.0	28.4	70.5	806.2	793.8	35.25	28.52	1.058	2.370
15	31.0	30.4	85.1	721.1	878.9	42.55	34.43	1.058	2.370
16	33.0	32.4	91.2	629.9	970.1	45.60	36.89	1.058	2.370
17	35.0	34.4	96.4	533.5	1066.5	48.20	39.00	1.058	2.370
18	37.0	36.4	96.4	437.1	1162.9	48.20	39.00	1.058	2.370
19	39.0	38.4	96.4	340.7	1259.3	48.20	39.00	1.058	2.147
20	41.0	40.4	97.5	243.2	1356.8	48.75	39.44	1.058	1.917
21	43.0	42.4	99.5	143.7	1456.3	49.75	40.25	1.058	1.716
22	45.0	44.4	100.6	43.1	1556.9	50.30	40.70	1.058	1.685
Avg. Shaft			70.8			35.07	28.37	1.058	2.242
Toe			43.1				451.40	1.306	1.947

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		2.892	0.099
Unloading Quake	(% of loading quake)	30	30
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	0	
Soil Plug Weight	(kN)		1.44

CAPWAP match quality = 1.78 (Wave Up Match) ; RSA = 0
Observed: final set = 1.500 mm; blow count = 667 b/m
Computed: final set = 1.474 mm; blow count = 678 b/m

max. Top Comp. Stress = 180.9 MPa (T= 25.8 ms, max= 1.014 x Top)
max. Comp. Stress = 183.5 MPa (Z= 3.0 m, T= 26.2 ms)
max. Tens. Stress = -3.73 MPa (Z= 44.0 m, T= 35.9 ms)
max. Energy (EMX) = 22.32 kJ; max. Measured Top Displ. (DMX)=13.54 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	2551.0	-30.7	180.9	-2.17	22.32	4.2	13.838
2	2.0	2566.9	-30.6	182.0	-2.17	22.08	4.2	13.439
5	5.0	2575.0	-29.3	182.6	-2.08	20.73	4.0	12.228
8	8.0	2357.0	-25.2	167.2	-1.78	17.62	3.7	11.030
11	11.0	2297.5	-22.9	162.9	-1.62	15.60	3.4	9.857
14	14.0	2010.1	-18.6	142.6	-1.32	12.39	3.2	8.717
17	17.0	1946.4	-15.5	138.0	-1.10	10.53	2.8	7.612
20	20.0	1616.6	-10.7	114.7	-0.76	7.65	2.5	6.580
23	23.0	1537.5	-8.4	109.0	-0.60	6.21	2.2	5.609
26	26.0	1285.5	-4.5	91.2	-0.32	4.47	2.0	4.759
29	29.0	1239.6	-3.4	87.9	-0.24	3.75	1.8	4.162
32	32.0	1025.2	-0.1	72.7	-0.01	2.59	1.6	3.513
35	35.0	977.7	0.0	69.3	0.00	2.01	1.4	2.943
38	38.0	765.5	0.0	54.3	0.00	1.26	1.2	2.587
39	39.0	801.5	0.0	56.8	0.00	1.24	1.1	2.454
40	40.0	691.4	0.0	49.0	0.00	0.95	1.1	2.334
41	41.0	724.8	0.0	51.4	0.00	0.92	1.0	2.204
42	42.0	622.0	-47.5	44.1	-3.37	0.69	1.0	2.118
43	43.0	654.7	0.0	46.4	0.00	0.67	0.9	2.045
44	44.0	577.0	-52.5	40.9	-3.73	0.44	1.1	2.010
45	45.0	457.5	0.0	32.4	0.00	0.13	1.2	2.001
Absolute	3.0			183.5			(T =	26.2 ms)
	44.0				-3.73		(T =	35.9 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RX	2879.5	2660.1	2440.6	2221.2	2001.8	1782.4	1563.0	1343.5	1124.1	904.7
RU	3391.2	3222.9	3054.7	2886.4	2718.2	2549.9	2381.7	2213.4	2045.2	1876.9

RAU = 418.0 (kN); RA2 = 882.0 (kN)

Current CAPWAP Ru = 1600.0 (kN); J(RX) = 0.58

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
m/s	ms	kN	kN	kN	mm	mm	mm	kJ	kN
4.27	25.57	2433.4	2640.3	2640.3	13.536	1.500	1.500	22.6	3012.1

PILE PROFILE AND PILE MODEL

Depth	Area	E-Modulus	Spec. Weight	Perim.
m	cm^2	MPa	kN/m^3	m
0.00	141.00	206842.7	77.287	1.236
45.00	141.00	206842.7	77.287	1.236

Toe Area 0.095 m^2

Top Segment Length 1.00 m, Top Impedance 569.29 kN/m/s

Pile Damping 1.0 %, Time Incr 0.195 ms, Wave Speed 5123.0 m/s, 2L/c 17.6 ms

THURBER

Appendix C

Static Pile Load Testing Results

THURBER

3-Week Static Load Test Report

Bermingham Construction
600 Ferguson Avenue North
Hamilton, Ontario
L8L 4Z9

Proposed Replacement of Blanche River Bridge
Highway 569 in New Liskeard, Ontario
Pile Load Test Program
~ Static Load Testing of Pile TP1
(3 Weeks After Initial Driving)

Project Number
BRM- 607254-A0

Prepared By:

exp Services Inc.
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Brampton, Ontario L6T 4V1
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Date Submitted
2018-10-23

Table of Contents

1 Introduction

The Ministry of Transportation of Ontario (MTO) has commissioned Bermingham Foundation Solutions (Bermingham) to carry out a pre-construction pile load test program for the proposed replacement of the Blanche River Bridge (MTO Structure No. 47-38), located approximately 150 m north of the intersection of Hilliardton Road and Highway 569, in the town of Hilliardton near the district of New Liskeard.

The primary purpose of the program is to examine the geotechnical resistances of steel pile foundations that have been proposed for the planned bridge. The area of the test pile program is the south-east corner of the bridge. The methods for examination include a series of two static load tests of one pile (TP1) and a series of three dynamic load tests on the second, adjacent pile (TP2).

Exp Services Inc. (EXP) was retained by Bermingham to monitor the Static Load Testing of Pile TP1 and to perform dynamic testing of the Pile TP2.

In this report is presented the results of the static load test that was carried out on the Pile TP1 from October 2 to 3, 2018 (approximately 3 weeks days after the end of initial driving). This test is the first of two static load tests that have been planned. The results of the dynamic tests and second static load test (planned for November 6, 2018) will be published under a separate cover.

Pile TP1 is a 310 mm x 110 kg/m steel HP-section and is made up of three 15.2 m (~50 feet) long steel sections and driven to an embedded depth of 44.0 m. The pile was statically load tested in general accordance with ASTM D1143; static load to the pile was imposed by jacking the test pile against a steel reaction frame. The steel frame was designed and constructed by Bermingham and supported in by eight (8) open-ended pipe piles (406 mm dia. x 9.5 mm thick) that are embedded 33 m into the ground.

Subsurface characteristics of the site is described in detail in the project Foundation Investigation Report (report Geocres No. 31M-120); it indicates that the site is underlain by a deposit of soft to firm varved clay that is more than 50 m thick. At the request of the Owner, Bermingham had also commissioned a cone penetrometer test (CPTu) at the site. Full results were published by under a separate cover by others. Selected subsurface information (borehole logs, CPTu plots) are shown on Appendix B.

2 Fieldwork – Pile Installation and Testing

Pile Installation

On September 10, 2018, pile TP1 was driven to an embedded length of approximately 44.0 m. On September 11, it was it was re-tapped and dynamically load tested. On October 4, 2018, pile TP1 was statically load tested to examine its geotechnical resistance at approximately 3 weeks after installation and re-tapping.

Pile TP1 is a steel H-pile (HP310x110) and it was driven with a Berminghammer B-32, open-ended diesel impact hammer. The hammer is rated at 110 kJ and has a maximum physical stroke of 3.5 m at the rated energy (35 blows per minute).

The pile installation record indicates that the effort required to drive the pile into the soft to firm varved clay deposit was low (less than 150 blows in total). The penetration resistance at the end of driving was reported to be 0.25 m per 2 blows. It is understood the diesel engine did not combust during installation due to the relatively large pile movement per blow.

The following day after installation, pile TP1 was re-tapped and dynamically tested. Prior to the beginning of the re-tapping, the hammer was warmed up by striking an adjacent pile. The penetration resistance during the beginning of the 1-day re-tap was found to be approximately 7 mm for 5 blows (average rebound of 7 mm); the energy transferred to the pile was approximately 4 kJ, as calculated by the Pile Driving Analyzer.

Load Test Procedure

On October 2, 2018 (at 3 weeks after the re-tap on September 11, 2018), Pile TP1 was statically load tested in general accordance with ASTM D1143-07 Standard Test Methods for Deep Foundations Under Static Axial Compressive Load. Procedure 'B' of the ASTM D1143, with modifications by the Owner, was indicated as the procedure for the test and is summarized below.

The static load test comprised a single load–unload cycle where axial compressive load were to be imposed at 150 kN increments until the Maximum Test Load of 1500 kN was reached or until the pile head had displaced by 15 % of its width (i.e. 46.5 mm for a HP310x110 pile).

Load increments were to be maintained on the pile until the pile movement had stabilised (movement less than 0.25 mm per hour) or for a maximum of 2 hours. The Maximum Test Load was to be maintained for 24 hours.

Pile movement under each load increment were measured after load placement and at intervals of 5, 10 and at every 20 minutes thereafter. At the Maximum Test Load, readings were to be taken as above for the first 2 hours, at every hour from the 2^{nd} to 12^{th} hour and at every 2 hours from the 12 to 24^{th} hour after load placement. When the Maximum Test Load had been maintained for 24 hours or when the pile had moved by 15% of the pile width, the load on the pile was to be removed.

Removal of the imposed loads was carried out in decrements of 25% of the maximum imposed load and at 1 hour intervals. Readings were taken at zero load until all readings had stabilized (i.e. every 20 mins for the first hour and at 12 hours after fully unloading).

Equipment for Static Load Test and Test Setup

Loads were imposed to the test pile by jacking it against a reaction frame using a hydraulic jack. The reaction frame comprised a steel test beam and anchor piles that were designed and constructed by Bermingham. Eccentric loading was limited through the use of a hemispherical bearing and imposed loads were measured using a load cell in conjunction with a digital pressure gauge. The layout and setup of the test are shown on Appendix A**.**

Pile head movement was measured using two (2) dial gauges mounted at approximately equidistance from the centre and on opposites sides of the pile top. Gauge stems were parallel to the direction of the load application The gauges recorded vertical displacement relative to two (2) self-supporting reference beams. The true movement of the pile was taken to be the average of the two (2) deflections measured on the gauges. The true, imposed load was taken to be the readings from the load cell.

The load cell, hydraulic jack, pressure gauge and dial gauges were calibrated prior to the commencement of the test. Calibration certificates are shown in Appendix C.

3 Observations and Test Results

During the test, loads were imposed incrementally at approximately 150 kN increments. When the imposed load was being increased from 1200 kN to 1350 kN (prior to reaching the intended Maximum Test Load of 1500 kN), "plunging" failure of the pile was observed.

Attempts to maintain the load increment of 1350 kN on the pile resulted in continuous, downward movement of the pile. The results of the Static Load Test (at ~3 weeks after pile installation) for Pile TP1 are presented on Drawing 1 and indicate that:

a. At a load of 755 kN (~50% of the intended Maximum Test Load), the pile head movement was approximately 5.3 mm;

b. At a load of approximately 1215 kN, the increment prior to plunging failure of the pile, the pile head movement was approximately 12.5 mm; and

c. The incremental load of 1350 kN could not be maintained on the pile and resulted in "plunging" failure of the pile.

When the total (gross) downward movement of the pile head reached ~47 mm, the pile was unloaded in decrements of approximately 25% of its maximum imposed load.

Upon fully unloading, the pile head movement had recovered; the displacement was 36.1 mm from its original position. At 12 hours after full unloading, the pile head movement had further recovered; its displacement was found to be 34.9 mm from its original position.

4 Closure

As part of a preconstruction test pile program for the proposed replacement of Blanche River Bridge on Highway 569, New Liskeard, a static load test was carried out on Test Pile TP1 at about 3 weeks after it was installed. The 3-week static test is the first of a series of two planned static load tests. The second test is planned for November 2018.

Pile TP1 was originally driven to an embedment depth of 44 m below grade on September 10, 2018 and then re-tapped and dynamically at the beginning of its 1-day restrike (BOR_1) with equipment from the PDA. The ultimate geotechnical resistance at the BOR_1 was evaluated to be 560 kN.

On October 2, 2018, Pile TP1 was statically load tested at approximately 21 days after the re-tap. The test comprised a single load-unload cycle and was carried out by jacking the test pile against a reaction frame in general accordance with ASTM D1143, Procedure B, with modifications by the Owner.

The intended Maximum Test Load was 1500 kN. During the course of the test, it was found that Pile TP1 was not able to maintain the incremental load of 1350 kN, which resulted in its plunging failure.

The results of the static load test also confirm that the geotechnical resistance of Pile TP1 had increased since it was installed on September 10, 2018 and since its re-tapping on September 11, 2018.

The behaviour and evaluated geotechnical resistance of TP1 reflects the pile resistance at the time of testing i.e. *at 21 days after re-tapping*. Given that the pile is predominantly embedded in clay it is possible that the resistance may continue to increase further with time.

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.

Michael W.K. Choy, P. Eng
Senior Geotechnical Engineer

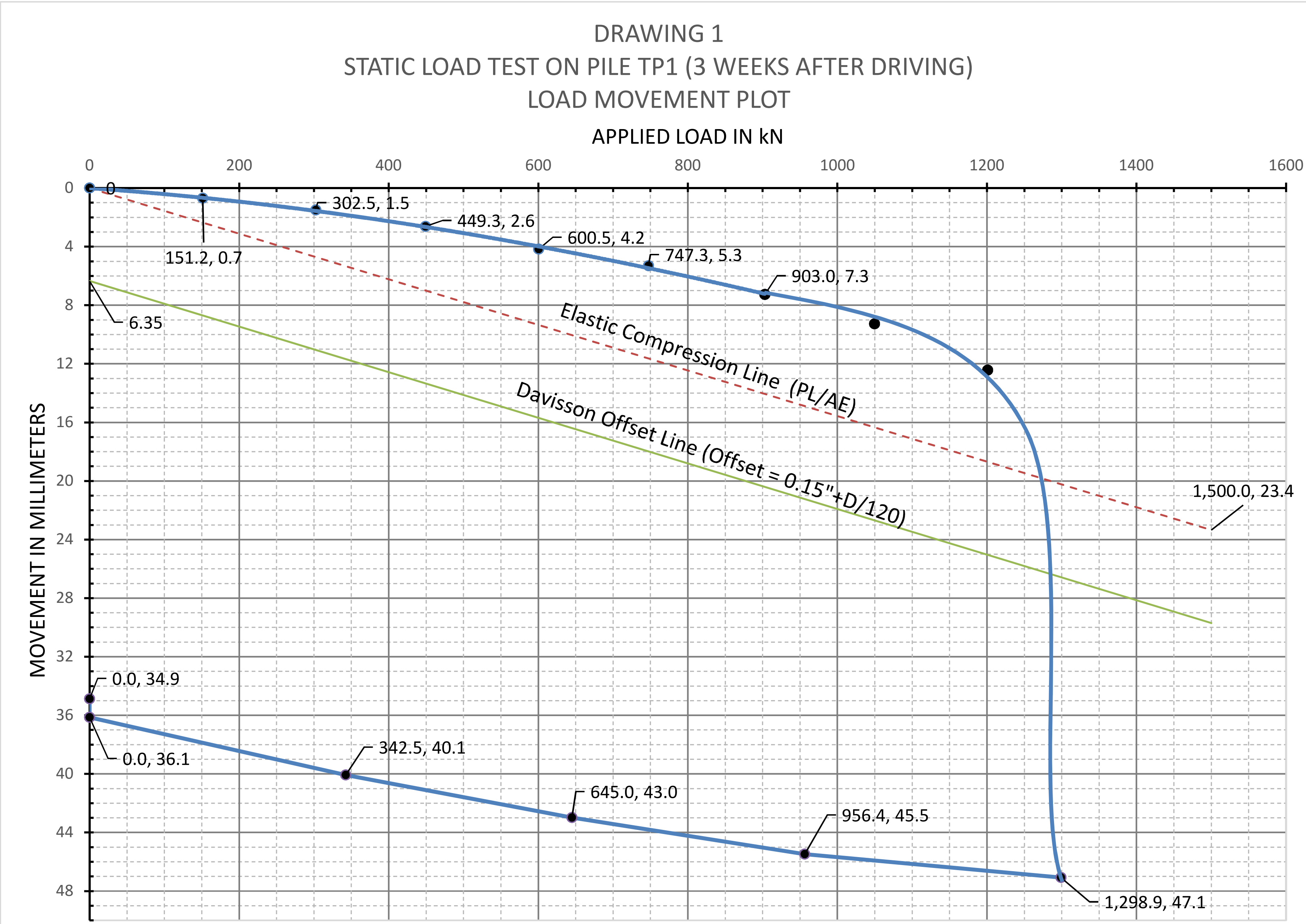
Stephen S.M. Cheng, P. Eng
Discipline Manager,
Geotechnical Division

MCHOY/I:\2003-Brampton\Projects\Geotechnical Engineering\0600000\607000\607200\607254-A0 Blanche River\Scope 3 - SLT\SLT Report 3 Wks -rev. 0.doc

exp.

Appendix A

Drawings


DRAWING 1
STATIC LOAD TEST ON PILE TP1 (3 WEEKS AFTER DRIVING)
LOAD MOVEMENT PLOT
APPLIED LOAD IN kN
MOVEMENT IN MILLIMETERS
0
151.2, 0.7
302.5, 1.5
449.3, 2.6
600.5, 4.2
747.3, 5.3
903.0, 7.3
6.35
Elastic Compression Line (PL/AE)
Davisson Offset Line (Offset = 0.15"+D/120)
1,500.0, 23.4
0.0, 34.9
0.0, 36.1
342.5, 40.1
645.0, 43.0
956.4, 45.5
1,298.9, 47.1

Appendix B

Reference Drawings, Documents and Subsurface Investigations

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

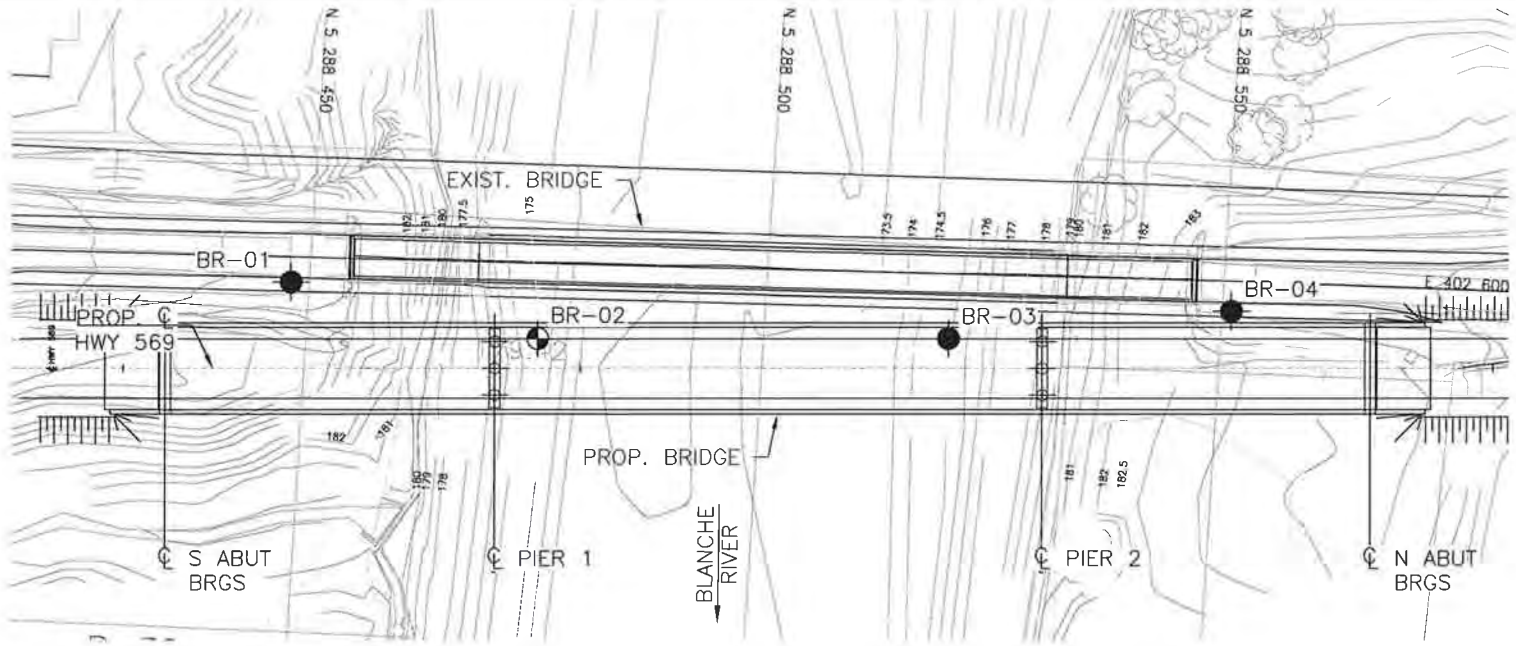
CONT No
WP No 5163-13-00

SHEET

HIGHWAY 569
BLANCHE RIVER BRIDGE REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

MMM GROUP

THURBER ENGINEERING LTD.

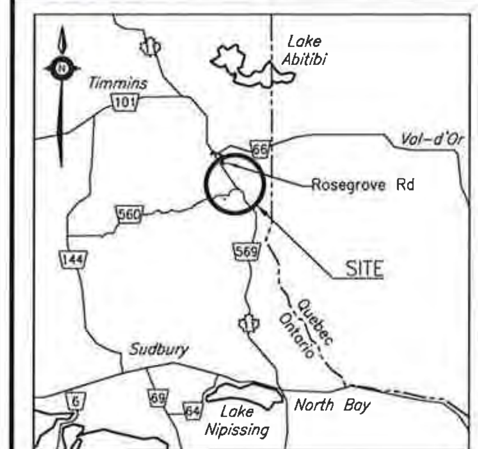


PLAN

SCALE 1:800

LICENSED PROFESSIONAL ENGINEER
P. R. CHATTERJI
Jun 9/17
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
K. L. SHI
100141639
Jun 9/17
PROVINCE OF ONTARIO

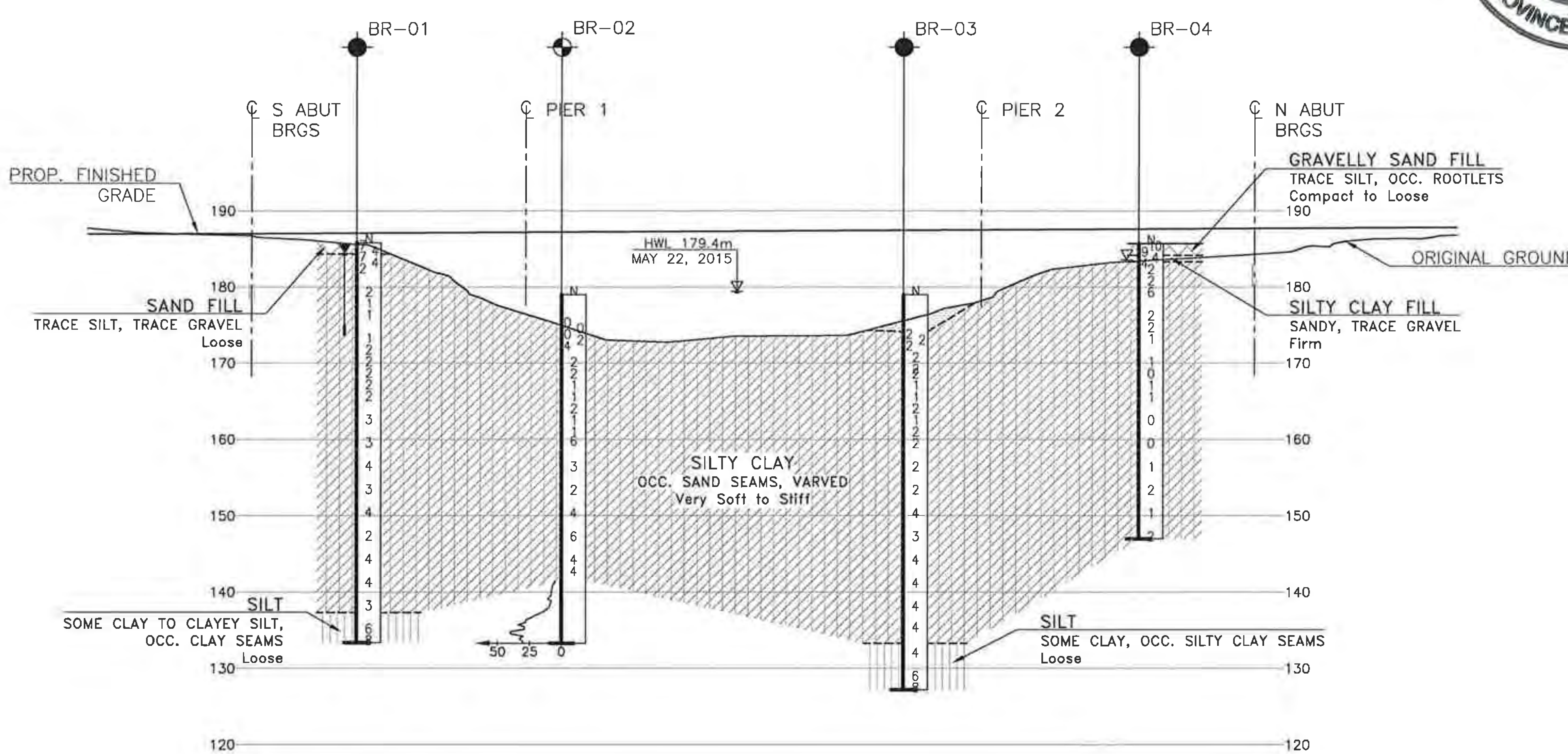


KEYPLAN

LEGEND

⯁	Borehole
⯁	Borehole and Cone
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
▽	Water Level
▼	Head Artesian Water
┃	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING (MTM)	EASTING (MTM)
BR-01	185.8	5 288 448.0	402 608.3
BR-02	179.0	5 288 475.4	402 612.8
BR-03	179.0	5 288 520.3	402 610.0
BR-04	185.7	5 288 551.0	402 605.2



PROFILE ALONG ℄ HWY 569

H 1:800

V 1:200

-NOTES-

1) The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
2) This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

GEOCRES No. 31M-120

REVISIONS	DATE	BY	DESCRIPTION

DESIGN AMP	CHK KS	CODE	LOAD	DATE JUN 2017
DRAWN AN	CHK	SITE	STRUCT	DWG 1

FILENAME: H:\Drafting\19\5161\2658\-BoreholePlan&Profile (Blanche River).dwg
PLOTDATE: 6/8/2017 1:41 PM

MINISTRY OF TRANSPORTATION, ONTARIO

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No BR-01

METRIC

W.P. 5163-13-00 | LOCATION Blanche River Bridge N 5 288 448.0 E 402 608.3 | ORIGINATED BY GA

HWY 569 | BOREHOLE TYPE NW Casing | COMPILED BY AN

DATUM Geodetic | DATE 2015.11.24 - 2015.11.26 | CHECKED BY AMP

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	GR	SA	SI	CL
185.8	GROUND SURFACE										
0.0	**TOPSOIL:** (50mm)										
	SAND, trace silt, trace gravel Loose Brown Moist (FILL)	1	SS	7							
					185						
		2	SS	4				1	95	4 (SI+CL)	
184.4 1.4	Silty **CLAY**, occasional sand seams, varved Firm to Stiff Grey Wet	3	SS	7	184						
		4	SS	4	183			0	0	44	56
		5	SS	2	182						
						4.0					
		1	TW		181			0	0	28	72
					180						
		6	SS	2	179						
						3.0					
		7	SS	1	178						
					177						
		8	SS	1	176			0	0	20	80

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%); STRAT PLOT; GROUND WATER CONDITIONS.

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

ONTMT4S 19-5161-265B.GPJ 2015TEMPLATE(MTO).GDT 5/11/16

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No BR-01

METRIC

W.P. 5163-13-00 | LOCATION Blanche River Bridge N 5 288 448.0 E 402 608.3 | ORIGINATED BY GA

HWY 569 | BOREHOLE TYPE NW Casing | COMPILED BY AN

DATUM Geodetic | DATE 2015.11.24 - 2015.11.26 | CHECKED BY AMP

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa (FIELD VANE)	WATER CONTENT (%)	UNIT WEIGHT kN/m3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Silty **CLAY**, occasional sand seams, varved Firm to Stiff Grey Wet					+ (4.0)			
					175				
		2	TW						
					174				
		9	SS	1			○		
					173				
						+ (5.0)			
					172				
		10	SS	2			○		
					171				
		11	SS	2			○		
					170				
						+ (5.0)			
					169				
		12	SS	2			○		
					168				
		13	SS	2			○		
					167				
						+ (4.0)			
					166				

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100
PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BR-01

METRIC

W.P.	5163-13-00	LOCATION	Blanche River Bridge N 5 288 448.0 E 402 608.3	ORIGINATED BY	GA
HWY	569	BOREHOLE TYPE	NW Casing	COMPILED BY	AN
DATUM	Geodetic	DATE	2015.11.24 - 2015.11.26	CHECKED BY	AMP

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	STRAT PLOT	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Silty **CLAY**, occasional sand seams, varved Firm to Stiff Grey Wet		14	SS	2				
							165		
							164		
							163		
			15	SS	3				0 0 35 65
							162	4.0 (field vane)	
							161		
							160		
			16	SS	3				
							159	4.0 (field vane)	
							158		
							157		
			17	SS	4				
							156		

Column headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³

Continued Next Page

+ 3 , × 3 : Numbers refer to Sensitivity

20 / 15 ◐ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BR-01

METRIC

W.P. 5163-13-00	LOCATION Blanche River Bridge N 5 288 448.0 E 402 608.3	ORIGINATED BY GA
HWY 569	BOREHOLE TYPE NW Casing	COMPILED BY AN
DATUM Geodetic	DATE 2015.11.24 - 2015.11.26	CHECKED BY AMP

SOIL PROFILE: ELEV DEPTH | DESCRIPTION | STRAT PLOT
SAMPLES: NUMBER | TYPE | "N" VALUES
GROUND WATER CONDITIONS | ELEVATION SCALE
DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100)
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE (20 40 60 80 100)
PLASTIC LIMIT w_P | NATURAL MOISTURE CONTENT w | LIQUID LIMIT w_L — WATER CONTENT (%) (20 40 60)
UNIT WEIGHT γ kN/m3
REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued From Previous Page

Silty **CLAY**, varved
Firm to Stiff
Grey
Wet

Number	Type	"N" Values
18	SS	3
19	SS	4
20	SS	2

Elevation scale: 155, 154, 153, 152, 151, 150, 149, 148, 147, 146

Field vane sensitivity values: 4.0, 4.0, 3.0, 4.0

Continued Next Page

+3, ×3: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No BR-01

METRIC

W.P.	5163-13-00	LOCATION	Blanche River Bridge N 5 288 448.0 E 402 608.3	ORIGINATED BY	GA
HWY	569	BOREHOLE TYPE	NW Casing	COMPILED BY	AN
DATUM	Geodetic	DATE	2015.11.24 - 2015.11.26	CHECKED BY	AMP

Continued From Previous Page

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT kN/m³	GR	SA	SI	CL
	Silty **CLAY**, varved Firm to Stiff Grey Wet				145							
		21	SS	4			w ≈ 44, wP ≈ 18, wL ≈ 41		0	0	32	68
					144	Field vane ≈ 50, sensitivity 3.0						
					143							
					142							
		22	SS	4			w ≈ 57					
					141							
					140							
					139							
		23	SS	3			w ≈ 45					
					138							
137.3 48.5	**SILT**, some clay to clayey silt, occasional clay seams Loose Grey Wet				137							
					136							

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 / 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No BR-01

METRIC

W.P. 5163-13-00 LOCATION Blanche River Bridge N 5 288 448.0 E 402 608.3 ORIGINATED BY GA

HWY 569 BOREHOLE TYPE NW Casing COMPILED BY AN

DATUM Geodetic DATE 2015.11.24 - 2015.11.26 CHECKED BY AMP

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILT, trace to some clay Loose Grey Wet	24	SS	6	135	
	Silty clay seam at 51.8m depth				134	
133.4		25	SS	8		0 0 63 37
52.4	END OF BOREHOLE AT 52.4m. BOREHOLE OPEN TO 52.4m AND WATER LEVEL AT 0.6m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.					

WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
Nov26/2015	2.0	183.8
Nov27/2015	1.4	184.4
Nov28/2015	1.4	184.4

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m3

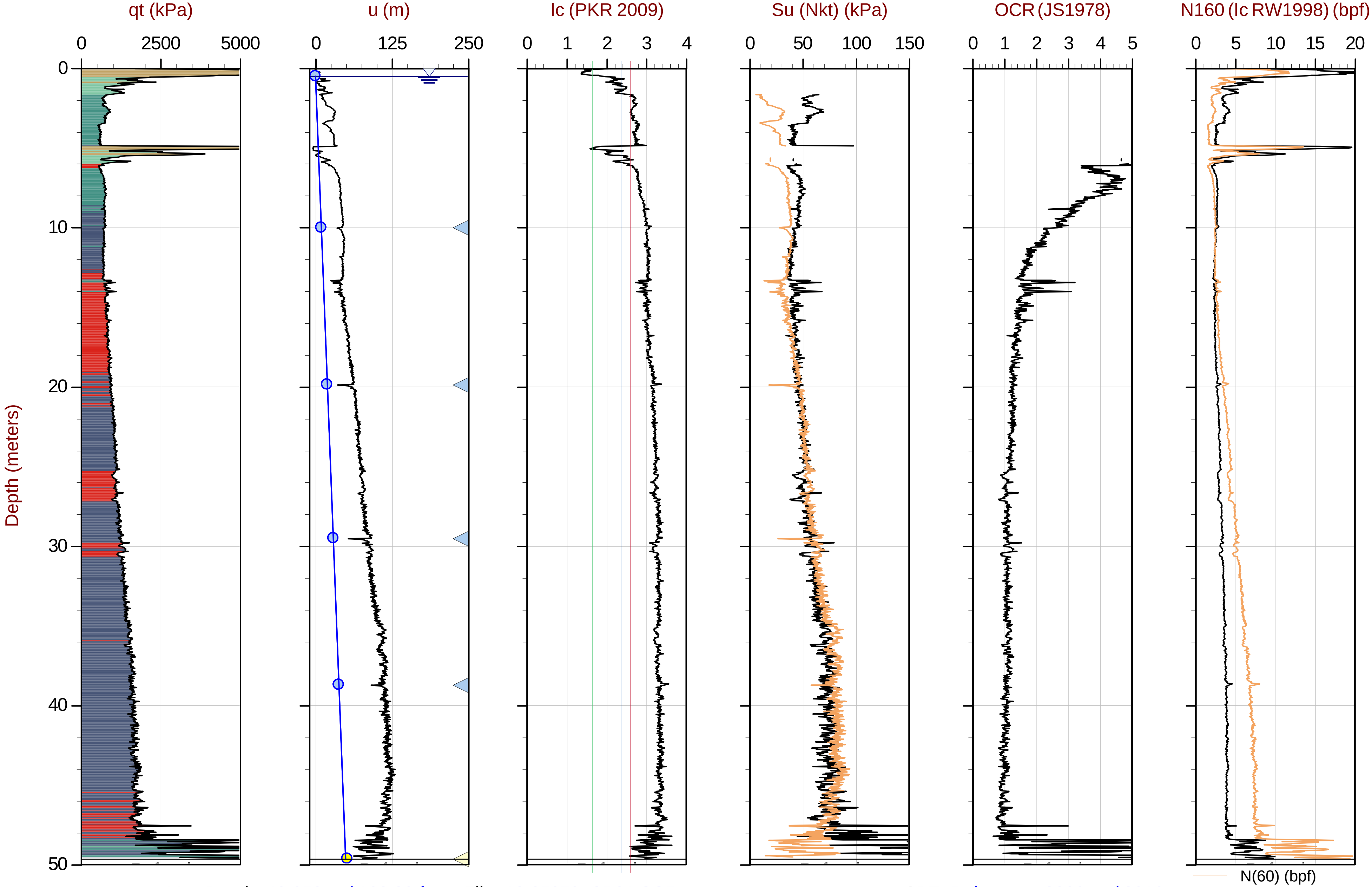
+ 3, × 3: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

CONETEC

Bermingham Foundation Solutions

Job No: 18-05053
Date: 2018-09-04 09:19
Site: Hwy 569 and Hilliardton Rd, New Liskeard, ON
Sounding: CPT18-01
Cone: 428:T1000F10U500



Max Depth: 49.650 m / 162.89 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 18-05053_CP01.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 12.5 / 9.0

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 5286865m E: 597774m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Hydrostatic Line Dissipation, Ueq assumed

Appendix C

Calibration Certificates

Equipment Corps Inc./
United Machine Tool Corp

October 22, 2018

Dear Andy,

Calibration of a jack system and load cell was conducted by Equipment Corps prior to dispatching the equipment to site. The calibration was conducted using a test frame in which all equipment is third party certified and calibrated to standards ASTM E4-16 AND/OR CSA A23.2-14. Traceable to NIST to the International Systems of Units (SI Units)

The calibration test was conducted with all of the following components together:

- Digital Pressure Gauge (SN1A00RLH2430)
- Hydraulic jack & powerpack (RCD3006C, SN# 22T031)
- Hemisherical bearing
- Digital load cell (SN21M301)

The following information was recorded at each pressure interval:

- PSI - Pressure applied to the jack being calibrated, as indicated by the digital pressure gauge
- Cylinder Output (i.e. measured force), in lbs as measured by the calibrated test frame
- Expected Output (i.e theoretical force), in lbs calculated using the jack specifications
- Load Cell Output (i.e measured force), in lbs as displayed by the load cell

At each pressure interval the load is maintained through the full stroke of the ram. Variance is calculated as the difference between the Cylinder Output and Expected Output, divided by the Expected Output:

$$Variance = \frac{(Cylinder\ Output - Expected\ Output)}{Expected\ Output}$$

The results are tabulated on the following page, followed by the equipment specifications for the jack and load cell.

Regards,

DAVE KISEL
EQUIPMENT CORPS/
UNITED MACHINE TOOL LTD
1256 ARVIN AVENUE, STONEY CREEK, ON L8E 0H7
P 905-545-1234 F 905-545-1270 C 905-730-1117
www.equipmentcorps.com

Certificate #	Date & Time	Surface Area	Tested By	NOTES
761	2018-09-14 13:26	91.5	SHANE ADORANTI	FULL VISUAL INSPECITON

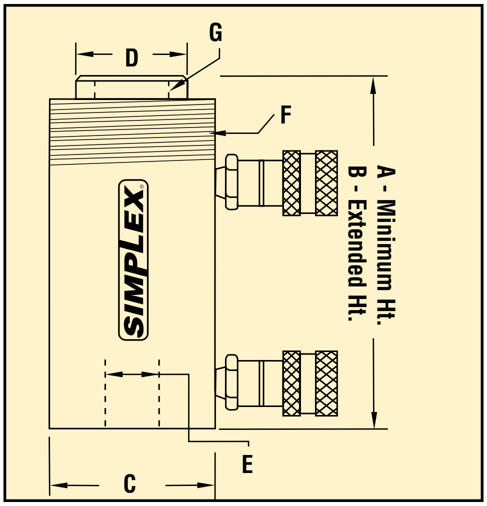
Detailed Results for 22T031 loadcell/load cap

PSI	Cylinder Output	Expected Output	Load Cell Output	Variance	% Variance (+/- 5%)	Pass / Fail
1000	92,320	91,500	91,215	820	0.90%	Pass
2000	182,540	183,000	181,013	-460	-0.25%	Pass
3000	274,980	274,500	274,392	480	0.17%	Pass
4000	365,180	366,000	365,992	-820	-0.22%	Pass
5000	457,900	457,500	457,000	400	0.09%	Pass
6000	549,000	549,000	*549,000*	0	0.00%	Pass
7000	640,500	640,500	*640,500*	0	0.00%	Pass
8000	732,000	732,000	*732,000*	0	0.00%	Pass
9000	823,500	823,500	*823,500*	0	0.00%	Pass
10000	915,000	915,000	*915,000*			

LIGHT BLUE VALUES ARE ASSUMED VALUES DUE TO THE CAPACITY OF OUR TEST BED. VARIANCE AND PASS/FAIL CALCULATED ON THE CYLINDER OUTPUT VS. EXPECTED OUTPUT. LOAD CELL VALUES ADDED TO CHART FROM CAPTURED DATA FOR INFORMATION PURPOSES.

Simplex Center Hole Cylinders Raise 6,000 Ton Bridge Span 170 Ft.

Engineers from around the world watched as Simplex center hole cylinders raised a 6,000 ton bridge section more than 170 ft. over the Willamette River in Portland, OR. The span was constructed up river and barged into position. Using pull rods and couplings, Simplex RCD200 Series cylinders completed the lift in 40 hours.



- 11 standard models.
- HARD-KOR™ Design for longer life.
- Relief valves protect against over pressurizing.
- High flow quality couplers.
- Large diameter center holes.
- Rhino-Rod™ pistons resist scoring & corrosion.
- Stop ring for piston blow-out protection.
- Available in custom strokes.
- Rod wiper protects inner cylinder from dirt.

Base Mounting Holes

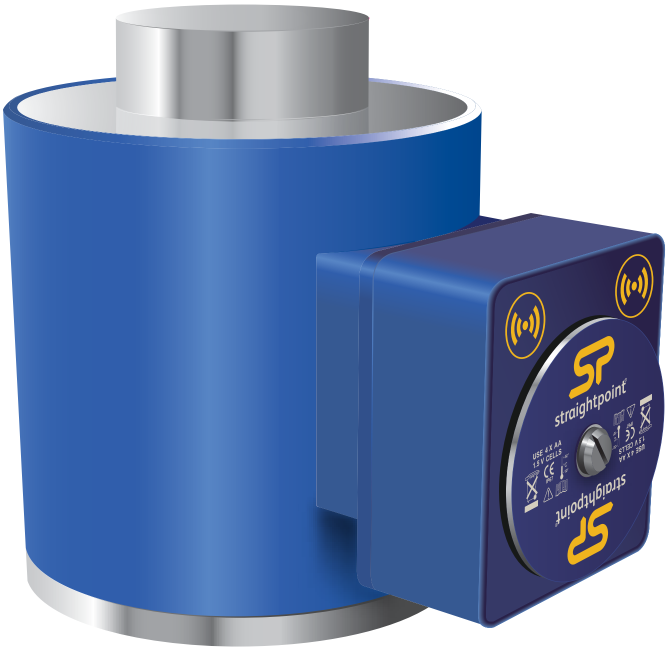
Capacity	Thread Size (in)	Thread Dpth. (in)	Bolt Circle Dia. (in)
30 Ton	3/8-16	3/8	3 5/8
60 Ton	1/2-13	7/16	5 1/8
100 Ton	5/8-11	3/4	7

Double-Acting Center Hole Cylinders 30 Through 500 Ton Capacities

Model Number	Push Cap. (tons)	Pull Cap. (tons)	Stroke (in)	A Min. Ht. (in)	B Ext. Ht. (in)	Effect Area (sq in)	Pres. @ Cap. (psi)	Oil Cap. Req'd (cu in)	C Body O.D. (in)	D Piston O.D. (in)	E Center Hole Dia. (in)	F Collar Thrds. (in)	F Collar Thrd. Lgth. (in)	G Piston Thrds. (in)	G Piston Thrd. Lgth. (in)	Wgt. (lbs)
RCD302	30	30	2	7	9	8.6	6,950	8.6	5	2 1/2	1 5/16	4 1/2 - 12	1 3/4	1 13/16 - 16	7/8	48
RCD307			7	12 1/4	19 1/4			24.75								50
RCD6010	60	45	10	15 1/2	25 1/2	14.5	8,230	55	6 3/4	4	2 1/8	----	----	----	----	95
RCD1001	100	68	1	7	9	20.6	9,700	9.0	8 1/4	5	3 1/8	----	----	----	----	85
RCD1003			3	9	12			21.1				----	----	----	----	100
RCD1006			6	12	18			42.3				----	----	----	----	115
RCD10010			10	16	26			70.5				----	----	----	----	150
RCD1505C	150	----	5	13 1/8	18 1/8	31.60	9,491	78	10	6	3 1/8	----	----	----	----	240
RCD2006C	246	----	6	16 1/4	22 1/4	49.30	10,000	172	12	8	4 1/8	----	----	----	----	385
RCD3006C	434	----		18 3/4	24 3/4	91.50		296	15	9 1/2	4 1/4	----	----	----	----	720
RCD5006C	646	----		21 1/2	27 1/2	129.30		463	17 3/4	12	5 1/4	----	----	----	----	1230

CUSTOM STROKES AND CAPACITIES AVAILABLE
CONTACT FACTORY

Wireless Compression Load Cell



Features and benefits:

- Proprietary 2.4 GHz wireless
- Industry leading wireless range of 700m/2300ft
- Connects to SW-MWLC, WCOGS & SW-PTP software
- Error free data transmission
- Internal antennae
- Environmentally sealed to IP67/NEMA 6
- No cable assemblies required
- Unrivalled resolution
- Unmatched battery life of 1200hrs
- Reduced maintenance cost
- Compact size
- Remote on-off
- Design validated by F.E.A.
- Bluetooth option is available and is supplied with a free HHP App for iOS and Android See page 29

The Straightpoint Wireless Compression Load Cell is taking the heavy lift and structural weighing industry by storm. By adding the Straightpoint wireless system to the already popular compression load cell line we have developed a cost-effective alternative to standard compression load cells.

No longer hindered by troublesome and hard to maintain cables, large scale projects can be completed in a fraction of time previously required. Maintenance costs are all but eliminated due to the absence of cables and connectors, and the products flexibility opens the door to a large number of applications in the heavy lift, energy, defence, rigging, shipping, and general transportation sectors, previously not considered.

Straightpoint's Wireless Compression Load Cells are machined from high grade stainless steel, providing excellent strength and corrosion resistance. The heavy duty, compact load cell utilises Straightpoint's advanced microprocessor based electronics and benefits from unrivalled resolution and accuracy. Data transmission is handled by the Straightpoint wireless systems proprietary transport protocol, is unmatched in performance and capable of a licence free transmission range of up to 700 metres or 2300 feet.

It is not until you add the powerful array of wireless accessories that the full potential of this product is realised. These accessories which include a wireless signal booster and several user friendly Windows-based software packages, provide a level of flexibility not previously known in the load monitoring industry.

When used with Straightpoint's WCOGS software these load cells will calculate centre of gravity and load. Connected to SW-MWLC it will allow the ability to data log and print reports, allowing the simultaneous display and monitoring of up to 100 wireless compression load cells on your PC or tablet. Lastly, coupled with Straightpoint's SW-PTP software the operator can perform load tests at a safe distance and generate real time test certificates on site.

making the lifting industry a safer place

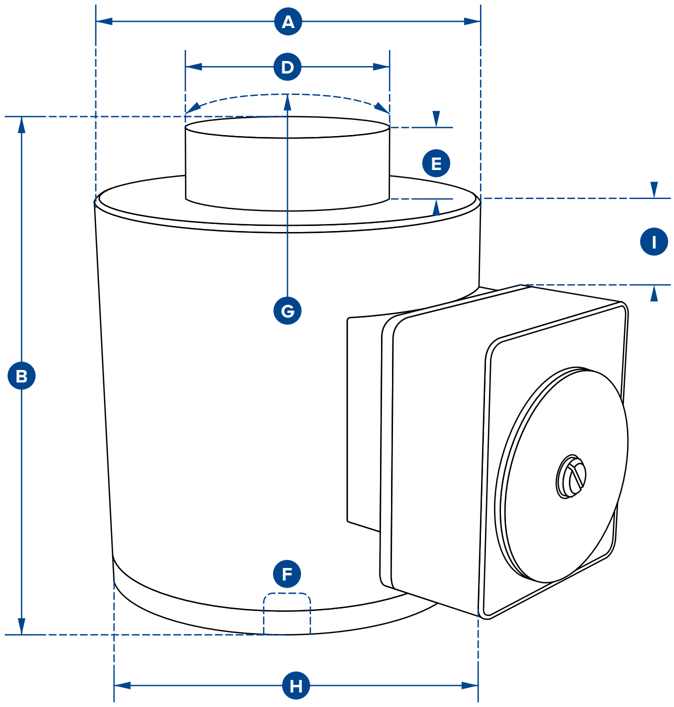
Also available with hazardous area approval

ATEX / IECEx

Ex ia II C T4 Ga

Certification numbers:
WNI ATEX
IECEx SIR 16.0041X / Sira 16ATEX2108X
SW-HHP ATEX
IECEx SIR 15.0072X / Sira 15ATEX2196X

Add 'ATEX' to the part number when ordering ATEX/IECEx products. For example: WNI5TC-ATEX



Part Number	WNI5TC	WNI10TC	WNI25TC	WNI50TC	WNI100TC	WNI150TC	WNI300TC	WNI500TC	WNI1000TC
Capacity	5te	10te	25te	50te	100te	150te	300te	500te	1000te
	11000lb	22000lb	55000lb	110000lb	220000lb	330000	660000lb	1100000lb	2200000lb
Resolution	0.001te	0.002te	0.005te	0.01te	0.05te	0.05te	0.1te	0.2te	0.5te
	2lb	5lb	10lb	20lb	100lb	100lb	200lb	500lb	1000lb
Units	tonne								
	lb								
Weight	6.2kg	6.2kg	6.2kg	6.2kg	15.5kg	15.5kg	65kg	65kg	172kg
	13.64lb	13.64lb	13.64lb	13.64lb	34lb	34lb	143lb	143lb	379lb
Safety Factor	3:1								
Battery Type	Load cell 4 x AA Alkaline								
Battery Life	Load cell 1,200 hours continuous								
Operating Temp	-10°C to +50°C / 14°F to 122°F								
Accuracy	±0.3% of applied load								
Frequency	2.4 GHz								
System Range	700 metres / 2300 feet								
Data Rate	3Hz (configurable to 200Hz)								
Protection	IP67 / NEMA 6								
Dimension ØA	102	102	102	102	152	152	185	185	362
	4.02"	4.02"	4.02"	4.02"	5.98"	5.98"	7.28"	7.28"	14.25"
Dimension B	127	127	127	127	184	184	300	300	310
	5.00"	5.00"	5.00"	5.00"	7.24"	7.24"	11.81"	11.81"	12.20"
Dimension ØD	59	59	59	59	80	80	155	155	270
	2.32"	2.32"	2.32"	2.32"	3.15"	3.15"	6.10"	6.10"	10.63"
Dimension E	16	16	16	16	26	26	27.5	27.5	40
	0.63"	0.63"	0.63"	0.51"	1.02"	1.02"	1.08"	1.08"	1.57"
Dimension F	M18 x 2.5	M18 x 2.5	M18 x 2.5	M20 x 2.5	M20 x 2.5	M20 x 2.5	M20 x 2.5	M20 x 2.5	M30 x 3.5
	M18 x 2.5	M18 x 2.5	M18 x 2.5	M20 x 2.5	M20 x 2.5	M20 x 2.5	M20 x 2.5	M20 x 2.5	M30 x 3.5
Dimension G	152	152	152	152	432	432	432	432	950
	5.98"	5.98"	5.98"	5.98"	17.01"	17.01"	17.01"	17.01"	37.40"
Dimension H	158	158	158	158	208	208	241	241	422
	6.22"	6.22"	6.22"	6.22"	8.19"	8.19"	9.49"	9.49"	16.61"
Dimension I	8	8	8	8	33	33	49	49	102
Loadcell top to SA700 top	0.31"	0.31"	0.31"	0.31"	1.30"	1.30"	1.93"	1.93"	4.02"

CDI CHICAGO

Chicago Dial Indicator Co.
1372 Redeker Road
Des Plaines, IL 60016
ISO Registered Firm

Factory Certificate of Calibration

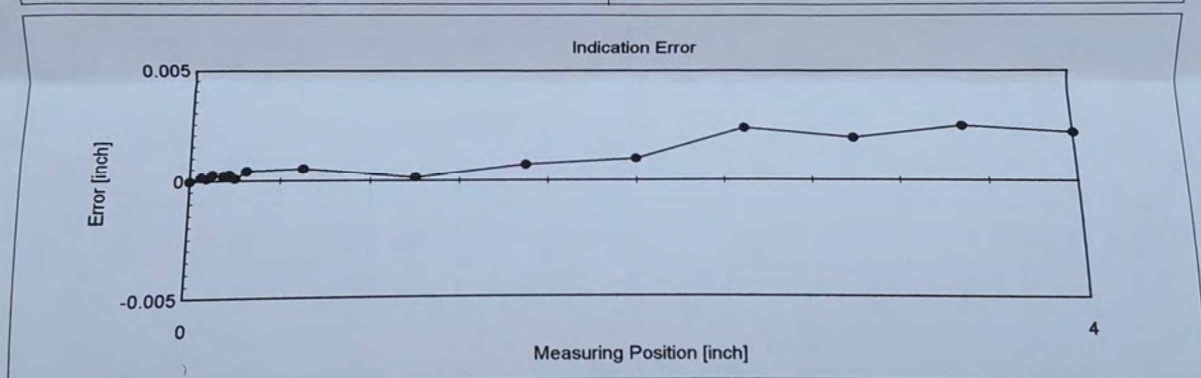
Model No.	26404CJ
Serial No.	173424713
Certificate No.	61924

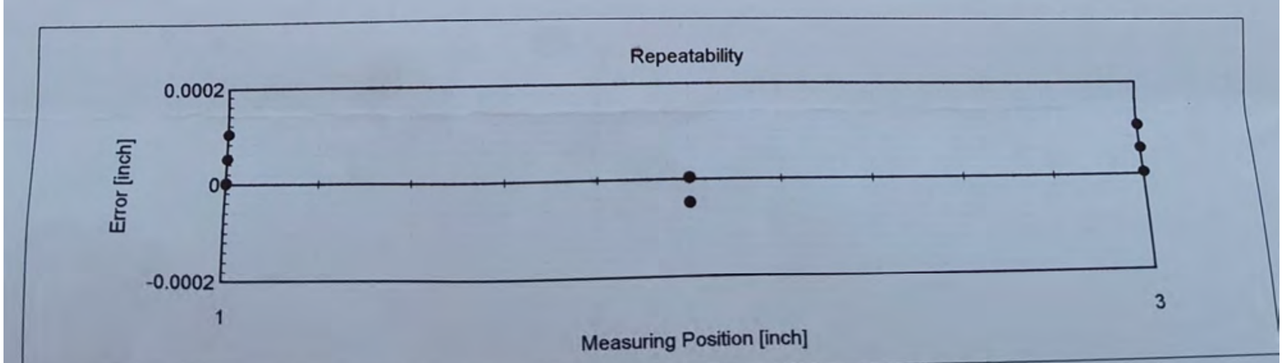
Name of Inspection Standard	CDI STANDARD .001/4.0
Unit	inch
Scale Interval	0.001 inch
Measuring Range	4 inch
Reference Point	0 inch
End Point	4 inch

N.I.S.T. No. 821/268795-03

Inspection Item Name	Result	Permissible Value	Judgment
First 2-1/3 Revolutions	+0.0004056 inch	±0.001 inch	GO
First 10 Revolutions	+0.0005057 inch	±0.002 inch	GO
First 20 Revolutions	+0.0009582 inch	±0.005 inch	GO
Hysteresis	———	———	N/A
Repeatability	+0.0001055 inch	±0.0002 inch	GO

Inspection Item Name	Judgment
Inspection of Function and Appearance	GO





Repeatability is taken at three positions, with five readings at each position.

Phone: 847-827-7186
Fax: 847-827-0478
Website: www.dialindicator.com

Date________________
Put into service date, provided by end user

Signature: E. Sw

CDI
CHICAGO

Chicago Dial Indicator Co.
1372 Redeker Road
Des Plaines, IL 60016
ISO Registered Firm

Factory Certificate of Calibration

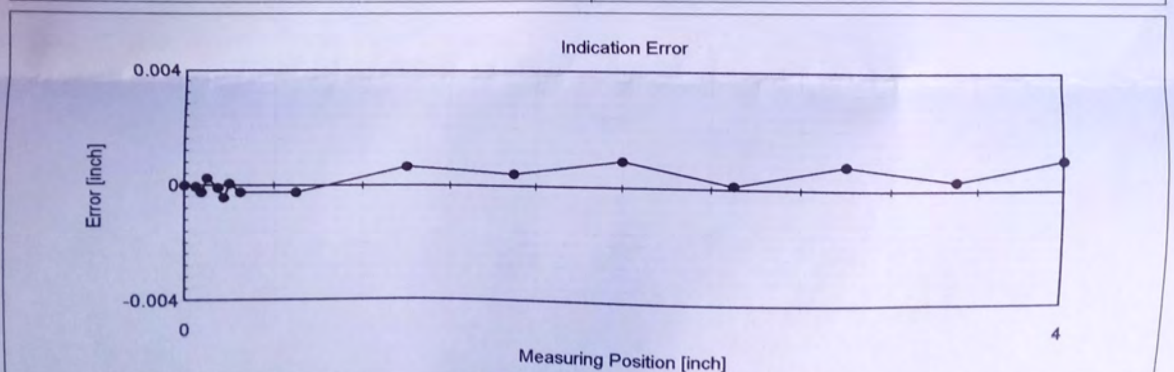
Model No.	26404CJ
Serial No.	172981099
Certificate No.	61109

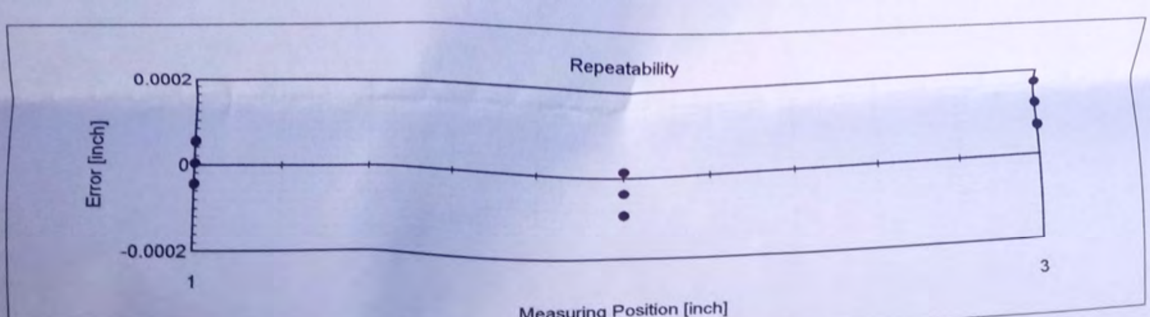
Name of Inspection Standard	CDI STANDARD .001/4.0
Unit	inch
Scale Interval	0.001 inch
Measuring Range	4 inch
Reference Point	0 inch
End Point	4 inch

N.I.S.T. No. 821/268795-03

Inspection Item Name	Result	Permissible Value	Judgment
First 2-1/3 Revolutions	-0.0004462 inch	±0.001 inch	GO
First 10 Revolutions	+0.0006586 inch	±0.002 inch	GO
First 20 Revolutions	+0.0009112 inch	±0.004 inch	GO
Hysteresis	----------	----------	N/A
Repeatability	+0.0001677 inch	±0.0002 inch	GO

Inspection Item Name	Judgment
Inspection of Function and Appearance	GO





Repeatability is taken at three positions, with five readings at each position.

Phone: 847-827-7186
Fax: 847-827-0478
Website: www.dialindicator.com

Date________________
Put into service date, provided by end user

Signature: Ehf.

8-Week Static Load Test Report

Bermingham Construction
600 Ferguson Avenue North
Hamilton, Ontario
L8L 4Z9

Proposed Replacement of Blanche River Bridge Highway 569 in New Liskeard, Ontario Pile Load Test Program ~ Static Load Testing of Pile TP1 (8 Weeks After Initial Driving)

Project Number
BRM- 607254-A0

Prepared By:

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

Date Submitted
2018-11-27

Table of Contents

1 Introduction

The Ministry of Transportation of Ontario (MTO) has commissioned Bermingham Foundation Solutions (Bermingham) to carry out a pre-construction pile load test program for the proposed replacement of the Blanche River Bridge (MTO Structure No. 47-38), located approximately 150 m north of the intersection of Hilliardton Road and Highway 569, in the town of Hilliardton near the district of New Liskeard.

The primary purpose of the program is to examine the geotechnical resistances of steel pile foundations that have been proposed for the planned bridge. The area of the test pile program is the south-east corner of the bridge. The methods for examination include a series of two static load tests of one pile (TP1) and a series of three dynamic load tests on the second, adjacent pile (TP2).

Exp Services Inc. (EXP) was retained by Bermingham to monitor the Static Load Testing of Pile TP1 and to perform dynamic testing of the Pile TP2.

In this report is presented the results of the static load test that was carried out on the Pile TP1 from November 6 to 8, 2018 (approximately 8 weeks after the end of initial driving). This test is the second of two static load tests that were planned. The results of the dynamic tests and first static load test are be published under separate covers.

Pile TP1 is a 310 mm x 110 kg/m steel HP-section and is made up of three 15.2 m (~50 feet) long steel sections and driven to an embedded depth of 44.0 m. The pile was statically load tested in general accordance with ASTM D1143; static load to the pile was imposed by jacking the test pile against a steel reaction frame. The steel frame was designed and constructed by Bermingham and supported in by eight (8) open-ended pipe piles (406 mm dia. x 9.5 mm thick) that are embedded 33 m into the ground.

Subsurface characteristics of the site is described in detail in the project Foundation Investigation Report (report Geocres No. 31M-120); it indicates that the site is underlain by a deposit of soft to firm varved clay that is more than 50 m thick. At the request of the Owner, Bermingham had also commissioned a cone penetrometer test (CPTu) at the site. Full results were published by under a separate cover by others. Selected subsurface information (borehole logs, CPTu plots) are shown on Appendix B.

2 Fieldwork – Pile Installation and Testing

Pile Installation

On September 10, 2018, pile TP1 was driven to an embedded length of approximately 44.0 m. On September 11, it was it was re-tapped and dynamically load tested. On October 4, 2018, pile TP1 was statically load tested at approximately 3 weeks after installation and re-tapping. On November 6, 2018, it was statically tested to examine its geotechnical resistance at approximately 3 weeks after installation and re-tapping.

Pile TP1 is a steel H-pile (HP310x110) and it was driven with a Berminghammer B-32, open-ended diesel impact hammer. The hammer is rated at 110 kJ and has a maximum physical stroke of 3.5 m at the rated energy (35 blows per minute).

The pile installation record indicates that the effort required to drive the pile into the soft to firm varved clay deposit was low (less than 150 blows in total). The penetration resistance at the end of driving was reported to be approx. 0.25 m per 2 blows. It is understood the diesel engine did not combust during installation due to the relatively large pile movement per blow.

The following day after installation, pile TP1 was re-tapped and dynamically tested. Prior to the beginning of the re-tapping, the hammer was warmed up by striking an adjacent pile. The penetration resistance during the beginning of the 1-day re-tap was found to be approx. 7 mm for 5 blows (average rebound of 7 mm); the energy transferred to the pile was approximately 4 kJ, as calculated by the Pile Driving Analyzer.

Load Test Procedure

Pile TP1 was statically load tested in general accordance with ASTM D1143-07 Standard Test Methods for Deep Foundations Under Static Axial Compressive Load. Procedure 'B' of the ASTM D1143, with modifications specified by the Owner.

The procedure for the 8-week test is summarized below, and in Appendix A.

The 8-week static load test comprised a single load–unload cycle where axial compressive load were imposed at 150 kN increments until a load of 1200 kN.

After a load of 1200 kN was reached, load were placed at increments of 100 kN until the imposed load reached 1500 kN. When the load of 1500 kN was reached, loads were to be placed at increments of 50 kN until the Maximum Test Load of

1700 kN or until the pile head had displaced by 15 % of its width (i.e. ~46.5 mm for a HP310x110 pile).

Load increments were maintained on the pile until the pile movement was equal/less than 0.25 mm per hour or for a maximum of 2 hours. At the specific request of the client, the load increment of 1200 kN was maintained for 24 hours.

Pile movement under each load increment were measured after load placement and at intervals of 5, 10 and at every 20 minutes for the first two hours. Where the load was held a load of 1200 kN, readings were to be taken as above for the first 2 hours, at every hour from the 2nd to 12th hour and at every 2 hours from the 12 to 24th hour after load placement.

Removal of the imposed loads was carried out in decrements of 25% of the maximum imposed load and at 1 hour intervals. Readings were taken at zero load until all readings had stabilized (i.e. every 20 mins for the first hour and at 12 hours after fully unloading).

Equipment for Static Load Test and Test Setup

Loads were imposed to the test pile by jacking it against a reaction frame using a hydraulic jack. The reaction frame comprised a steel test beam and anchor piles that were designed and constructed by Bermingham. Eccentric loading was limited through the use of a hemispherical bearing and imposed loads were measured using a load cell in conjunction with a digital pressure gauge. The layout and setup of the test are shown on Appendix A**.**

Pile head movement was measured using two (2) dial gauges mounted at approximately equidistance from the centre and on opposites sides of the pile top. Gauge stems were parallel to the direction of the load application The gauges recorded vertical displacement relative to two (2) self-supporting reference beams. The true movement of the pile was taken to be the average of the two (2) deflections measured on the gauges. The true, imposed load was taken to be the readings from the load cell.

The load cell, hydraulic jack, pressure gauge and dial gauges were calibrated prior to the commencement of the test.

3 Observations and Test Results

Test loads were imposed incrementally at approximately 150 kN increments until an imposed load of 1200 kN was reached. At the request of the Owner, this load was held for 24 hours before load placement resumed at increments of 100 kN.

Once the imposed load had reached 1500 kN, the subsequent load increment was placed at an increment of 50 kN. When the imposed load was increased to 1550 kN, “plunging” failure of the pile was observed. Attempts to maintain the load increment of 1550 kN on the pile resulted in continuous, downward movement of the pile.

The results are graphically illustrated on Drawings 1 & 2. On Drawing 1, the results of the previous load test were also plotted. Several key observations are noted below:

a. Immediately after the load of 1200 kN was placed, the pile head movement was approximately 9.14 mm (0.36 inches).

b. Two hours after the load of 1200 kN was placed, the pile head movement was 9.82 mm (0.39 inches). The rate of movement was 0.076 mm / hour (0.003 inches / hour).

c. 24 hours after the load of 1200 kN was placed, the pile head movement was observed to be 10.62 mm (0.418 inches).

d. At a load of approximately 1300 kN, the load increment prior to an obvious increase in the load-movement gradient, the pile head movement was approximately 11.38 mm

e. The incremental load of 1550 kN could not be maintained on the pile and resulted in “plunging” failure of the pile.

When the total (gross) downward movement of the pile head reached ~52 mm, the pile was unloaded in decrements of approximately 25% of its maximum imposed load.

Upon fully unloading, the pile head movement had recovered; the displacement was ~38.7 mm from its original position. At 12 hours after full unloading, the pile head movement had further recovered; its displacement was found to be ~37.8 mm from its original position.

4 Closure

As part of a preconstruction test pile program for the proposed replacement of Blanche River Bridge on Highway 569, New Liskeard, a static load test was carried out on Test Pile TP1 at about 8 weeks after it was installed. The 8-week static test is the second of a series of two planned static load tests. The first test is took place in October 2018.

Pile TP1 was originally driven to an embedment depth of 44 m below grade on September 10, 2018 and then re-tapped and dynamically at the beginning of its 1-day restrike (BOR_1) with equipment from the PDA. The ultimate geotechnical resistance at the BOR_1 was evaluated to be 560 kN.

On October 2, 2018, Pile TP1 was statically load tested at approximately 3 weeks after installation. The results of the test indicated that the pile was not able to maintain the incremental load of 1350 kN, which resulted in its plunging failure.

On November 6, 2018, Pile TP1 was statically tested at approximately 8 weeks after installation. The results of the test indicate that the pile was able to maintain the imposed load of 1200 kN for 24 hours, but was not able to maintain the incremental load of 1550 kN, which resulted in its plunging failure.

The results of the static load test also confirm that the ultimate geotechnical resistance of Pile TP1 had increased since it was installed on September 10, and since the static testing on November 6, 2018.

The behaviour and evaluated geotechnical resistance of TP1 reflects the pile resistance at the time of testing i.e. *at 8 week after installation*; it is possible that the resistance may continue to increase further with time.

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.

Michael W.K. Choy, P. Eng
Senior Geotechnical Engineer

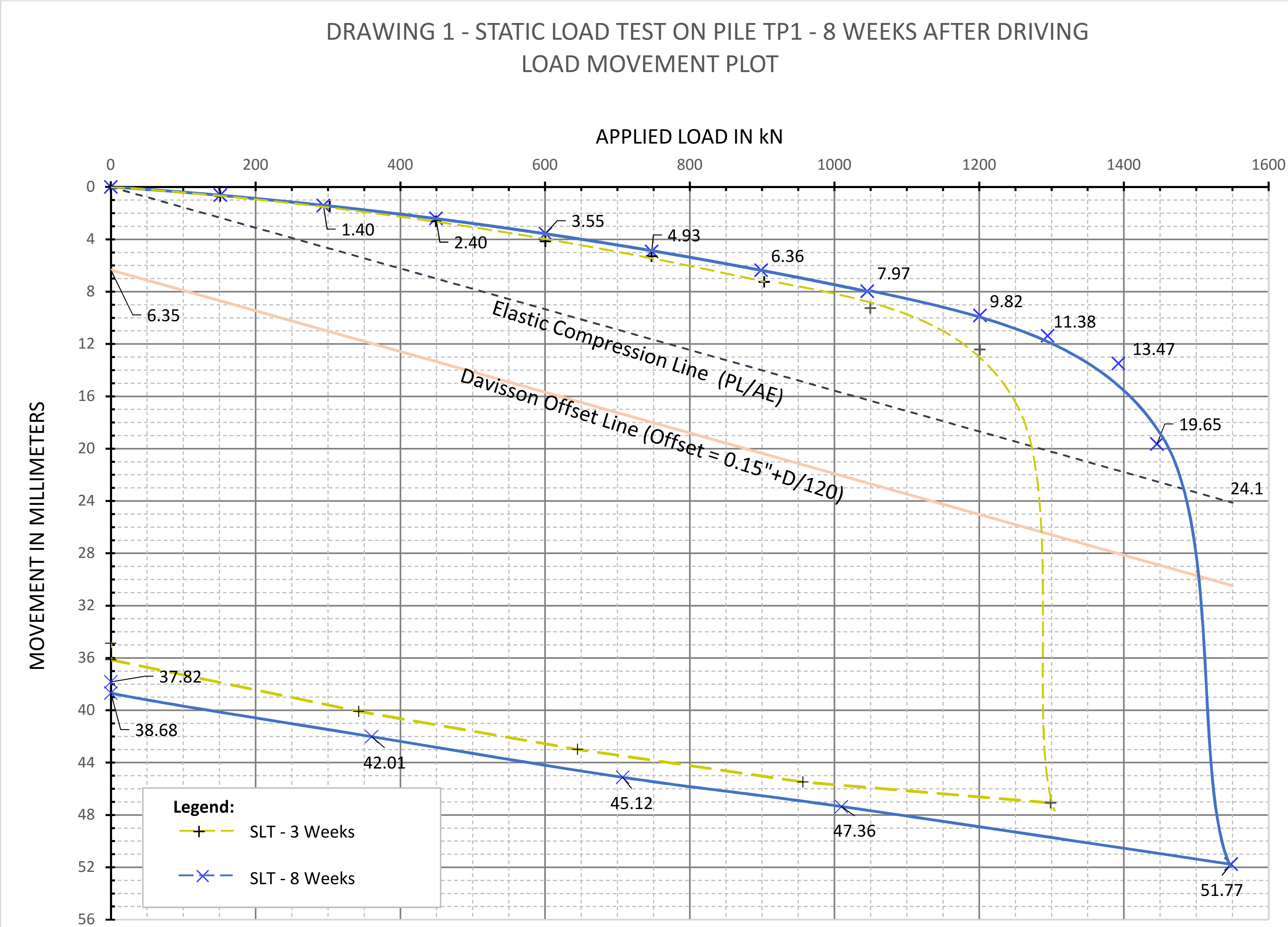
Stephen S.M. Cheng, P. Eng
Discipline Manager,
Geotechnical Division

MCHOY/I:\2003-Brampton\Projects\Geotechnical Engineering\0600000\607000\607200\607254-A0 Blanche River\Scope 3 - SLT\SLT Report 8 Wks -rev. 0.doc

exp.

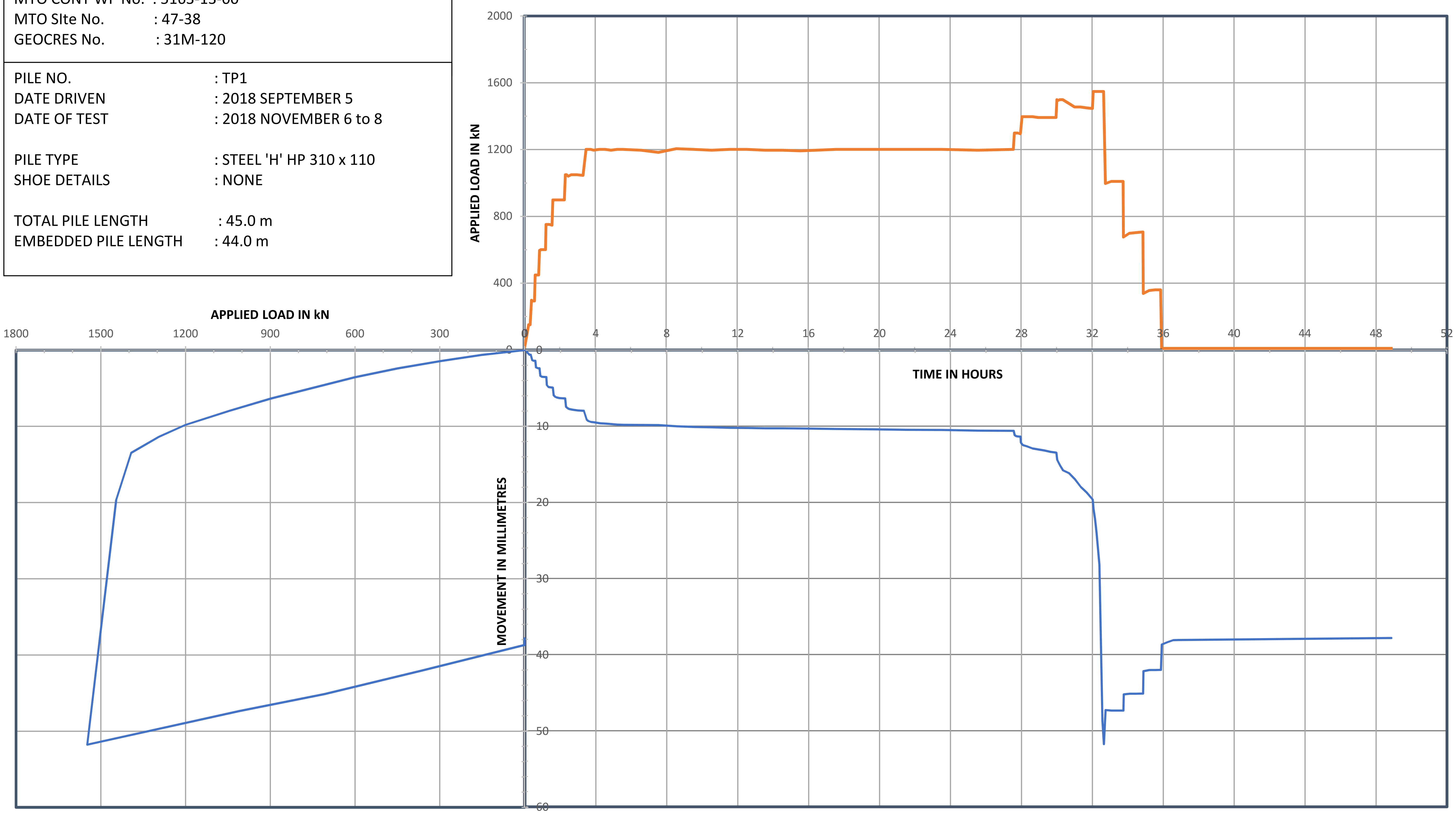
Appendix A

Drawings and Tables


DRAWING 1 - STATIC LOAD TEST ON PILE TP1 - 8 WEEKS AFTER DRIVING
LOAD MOVEMENT PLOT
APPLIED LOAD IN kN
0
200
400
600
800
1000
1200
1400
1600
MOVEMENT IN MILLIMETERS
0
4
8
12
16
20
24
28
32
36
40
44
48
52
56
1.40
2.40
3.55
4.93
6.36
7.97
9.82
11.38
13.47
19.65
24.1
6.35
Elastic Compression Line (PL/AE)
Davisson Offset Line (Offset = 0.15"+D/120)
37.82
38.68
42.01
45.12
47.36
51.77
Legend:
SLT - 3 Weeks
SLT - 8 Weeks

LOCATION	: Blanche River Bridge Hwy 569 & Blanche River 10 km North of Temiskaming Shores (New Liskeard)
MTO CONT WP No.	: 5163-13-00
MTO SIte No.	: 47-38
GEOCRES No.	: 31M-120

PILE NO.	: TP1
DATE DRIVEN	: 2018 SEPTEMBER 5
DATE OF TEST	: 2018 NOVEMBER 6 to 8
PILE TYPE	: STEEL 'H' HP 310 x 110
SHOE DETAILS	: NONE
TOTAL PILE LENGTH	: 45.0 m
EMBEDDED PILE LENGTH	: 44.0 m



Exp Services Inc.

Blanche River Trial Pile Program - Load Placement and Holding Table

Static Load Test at 8 Weeks after Pile Installation

Test Stage	Test Load (kN)	Load Holding Period	Frequency of Readings
LOADING	0	-	baseline reading
	150	2 hours see Note 3	5 mins., 10 mins. & 20 mins. after load placement, every 20 mins. thereafter for the first 2 hours after load placement, every 60 mins. thereafter from 2nd to 12th hour after load placement, and every 120 mins. thereafter
	300		
	450		
	600		
	750		
	900		
	1050		
	1200	24 hours	
	1300	2 hours see Note 3	
	1400		
	1500		
	1550		
UNLOADING	75 % of MTL	1 hour	immediately after unloading, every 20 mins. thereafter, and immediately before next unloading
	50% of MTL		
	25 % of MTL		
	0	12 hours	immediately after unloading, every 20 mins for first hour, and 12 hours after unloading.

Notes:

1. Pile is a steel H-Pile, HP 310x110. MTL is Maximum Test Load.

2. Apply loads incrementally up to MTL or until total pile head movement reaches 46.5mm

3. Maintain load for prescribed period or until movement rate is < 0.25 mm / hr., whichever earlier.

THURBER

7-Month Static Load Test Report

Bermingham Construction
600 Ferguson Avenue North
Hamilton, Ontario
L8L 4Z9

Proposed Replacement of Blanche River Bridge
Highway 569 in New Liskeard, Ontario

Pile Load Test Program
~ Static Load Testing of Pile TP1
(37 Weeks After Initial Driving)

Project Number
BRM- 607254-A0

Prepared By:

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

Date Submitted
2019-05-08

Table of Contents

1 Introduction

The Ministry of Transportation of Ontario (MTO) has commissioned Bermingham Foundation Solutions (Bermingham) to carry out a pre-construction pile load test program for the proposed replacement of the Blanche River Bridge (MTO Structure No. 47-38), located approximately 150 m north of the intersection of Hilliardton Road and Highway 569, in the town of Hilliardton near the district of New Liskeard.

The primary purpose of the program is to examine the geotechnical resistances of steel pile foundations that have been proposed for the planned bridge. The area of the test pile program is the south-east corner of the existing bridge. The methods for examination include a series of three static load tests on one pile (TP1) and a series of three dynamic load tests a second, adjacent pile (TP2).

Exp Services Inc. (EXP) was retained by Bermingham to monitor the Static Load Testing of Pile TP1 and to perform dynamic testing of the Pile TP2.

In this report is presented the results of the third static load test that was carried out on the Pile TP1 from April 30 to May 1, 2019 (approximately 37 weeks after the end of initial driving). The results of earlier dynamic tests and static load tests were published under separate covers.

Pile TP1 is a 310 mm x 110 kg/m steel HP-section and is made up of three 15.2 m (~50 feet) long steel sections and driven to an embedded depth of ~44.0 m. The pile was statically load tested in general accordance with ASTM D1143; static load to the pile was imposed by jacking the test pile against a steel reaction frame. The steel frame was designed and constructed by Bermingham and supported in by eight (8) open-ended pipe piles (406 mm dia. x 9.5 mm thick) that are embedded 33 m into the ground.

Subsurface characteristics of the site are described in detail in the project Foundation Investigation Report (report Geocres No. 31M-120); it indicates that the site is underlain by a deposit of soft to firm varved clay that is more than 50 m thick. At the request of the Owner, Bermingham had also commissioned a cone penetrometer test (CPTu) at the site. Full results were published by under a separate cover by others. Selected subsurface information (borehole logs, CPTu plots) are shown on Appendix B.

2 Fieldwork – Pile Installation and Testing

Pile Installation

On September 10, 2018, pile TP1 was driven to an embedded length of approximately 44.0 m.

Pile TP1 is a steel H-pile (HP310x110) and it was driven with a Berminghammer B-32, open-ended diesel impact hammer. The hammer is rated at 110 kJ and has a maximum physical stroke of 3.5 m at the rated energy (~35 blows per minute).

The pile installation record indicates that the effort required to drive the pile into the soft to firm varved clay deposit was low (less than 150 blows in total). The penetration resistance at the end of driving was reported to be approx. 0.25 m per 2 blows. It is understood the diesel engine did not combust during installation due to the relatively large pile movement per blow.

Pile Testing

The following day after installation, pile TP1 was re-tapped and dynamically tested. Prior to the beginning of the re-tapping, the hammer was warmed up by striking an adjacent pile. The penetration resistance at the beginning of the 1-day re-tap was found to be approx. 7 mm for 5 blows (average rebound of 7 mm); the energy transferred to the pile was approximately 4 kJ, as calculated by the Pile Driving Analyzer.

On October 4 and November 6, 2018, pile TP1 was statically load tested at approximately 3 and 8 weeks, respectively, after installation. The results were published under separate covers.

On April 30, 2019, pile TP1 was statically load tested to examine its geotechnical resistance at approximately 37 weeks after installation. The test procedure and results are set out in the following sections.

Procedure For Static Load Test At 37 Weeks After Pile Installation

Pile TP1 was statically load tested in general accordance with ASTM D1143-07 Standard Test Methods for Deep Foundations Under Static Axial Compressive Load. Procedure 'B' of the ASTM D1143, with modifications specified by the Owner.

The 37-week static load test comprised a single load–unload cycle where axial compressive load were imposed at ~200 kN increments until a load of ~1400 kN, after which loads were placed at increments of ~100 kN until "failure", which was defined by the Owner as axial displacement of pile head by 15 % of its width (i.e. ~46.5 mm for a HP310x110 pile). Load increments were maintained on the pile until the pile movement was less than 0.25 mm (0.01") per hour or for a maximum of 2 hours.

Pile movement under each load increment were measured after load placement and at intervals of 5, 10 and at every 20 minutes for the first two hours.

Removal of the imposed loads was carried out in decrements of ~25% of the maximum imposed load and at 1 hour intervals. Readings were taken at 20 minute intervals. Upon removal of all of the imposed load, readings were taken at 20 minute intervals for the first hour, and at 12 hours after fully unloading.

Equipment for Static Load Test and Test Setup

Loads were imposed to the test pile by jacking it against a reaction frame using a hydraulic jack. The reaction frame comprised a steel test beam and anchor piles that were designed and constructed by Bermingham. Eccentric loading was limited through the use of a hemispherical bearing and imposed loads were measured using a load cell in conjunction with a digital pressure gauge. The layout and setup of the test are shown on Appendix A**.**

Pile head movement was measured using two (2) dial gauges mounted at approximately equidistance from the centre and on opposites sides of the pile top. Gauge stems were parallel to the direction of the load application The gauges recorded vertical displacement relative to two (2) self-supporting reference beams. The true movement of the pile was taken to be the average of the two (2) deflections measured on the gauges. The true, imposed load was taken to be the readings from the load cell.

3 Observations and Test Results

Test loads were imposed at ~200 kN increments until a load of ~1400 kN, after which loads were placed at increments of ~100 kN until "failure", which was which was defined by the Owner as an axial pile head displacement of ~46.5 mm.

When the imposed load was increased from 1600 to 1700 kN, "plunging" failure of the pile was observed. Attempts to maintain the load increment of ~1640 kN on the pile resulted in continuous, downward movement of the pile.

The results are graphically illustrated on Drawing 1. The results of the previous load tests are also plotted on Drawing 1. Several key observations are noted below:

a. At the imposed load of ~590 kN (as compared to the design SLS resistance of 600 kN), the pile head movement was ~3.8 mm.

b. At the imposed load of ~970 kN (as compared to the design ULS resistance of 900 kN), the pile head movement was ~7.3 mm.

c. At the imposed load of *~1380* kN, the load increment prior to an obvious increase in the load-movement gradient, the pile head movement was approximately ~11.8 mm.

d. The incremental load of ~1640 kN could not be maintained on the pile and resulted in "plunging" failure of the pile.

When the total (gross) downward movement of the pile head reached ~47 mm, the pile was unloaded in decrements of ~25% of its maximum imposed load.

Upon fully unloading, some pile head movement was recovered. At 12 hours after full unloading, the pile head movement had further recovered. The pile was found to have displaced ~35.8 mm from its original position.

4 Closure

As part of a preconstruction test pile program for the proposed replacement of Blanche River Bridge on Highway 569, New Liskeard, a static load test was carried out on Test Pile TP1 at ~37 weeks after it was installed. The 37-week test is the third in a series of three static load tests.

Pile TP1 was originally driven to an embedment depth of 44 m below grade on September 10, 2018 and then re-tapped and dynamically at the beginning of its 1-day restrike (BOR_1) with equipment from the PDA. The ultimate geotechnical resistance at the BOR_1 was evaluated to be 560 kN.

On October 2, 2018, Pile TP1 was statically load tested at approximately 3 weeks after installation. The results of the test indicated that the pile was not able to maintain the incremental load of ~1350 kN, which resulted in its plunging failure.

On November 6, 2018, Pile TP1 was statically tested at approximately 8 weeks after installation. The results of the test indicate that the pile was able to maintain the imposed load of 1200 kN for 24 hours, but was not able to maintain the incremental load of ~1550 kN, which resulted in its plunging failure.

On April 30, 2019, Pile TP1 was statically tested at approximately 37 weeks after installation. The results of the test indicate that the pile not able to maintain the incremental load of ~1640 kN, which resulted in its plunging failure.

Finally, the behaviour of TP1 reflects the pile resistance at the time of testing (during spring thaw at *~37 weeks after installation*). The results suggest that further increase in geotechnical resistance with time is likely to be marginal unless there is an increase in the soil strength of the soil.

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.

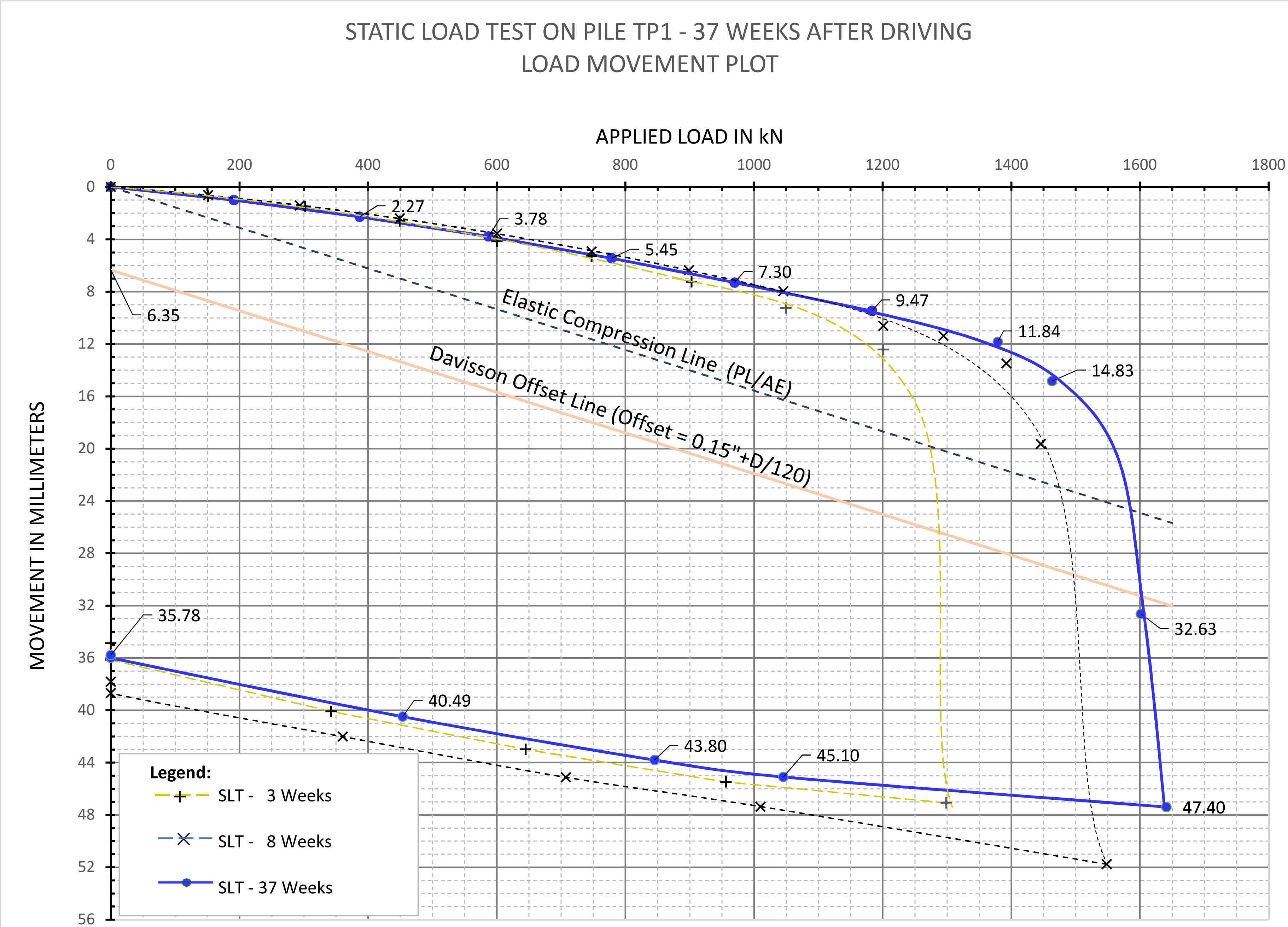
Michael W.K. Choy, P. Eng
Senior Geotechnical Engineer

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

MCHOY/I:\2003-Brampton\Projects\Geotechnical Engineering\0600000\607000\607200\607254-A0 Blanche River\Scope 3 - SLT\37 weeks\SLT Report 37 Wks -rev. 0.doc

Appendix A

Drawings and Tables


STATIC LOAD TEST ON PILE TP1 - 37 WEEKS AFTER DRIVING
LOAD MOVEMENT PLOT
APPLIED LOAD IN kN
0
200
400
600
800
1000
1200
1400
1600
1800
MOVEMENT IN MILLIMETERS
0
4
8
12
16
20
24
28
32
36
40
44
48
52
56
2.27
3.78
5.45
7.30
9.47
11.84
14.83
32.63
47.40
45.10
43.80
40.49
35.78
6.35
Elastic Compression Line (PL/AE)
Davisson Offset Line (Offset = 0.15"+D/120)
Legend:
SLT - 3 Weeks
SLT - 8 Weeks
SLT - 37 Weeks

Blanche River Trial Pile Program
Load Placement and Holding Table
Static Load Test at 37 Weeks after Pile Installation

Test Stage	Load (kN)	Load Holding Period	Frequency of Readings
LOADING	0	-	baseline reading
	200	2 hours (see note 3)	5 mins., 10 mins. & 20 mins. after load placement and at every 20 mins. thereafter
	400		
	600		
	800		
	1000		
	1200		
	1400		
	1500		
	1600		
	1640		
UNLOADING	75% of MTL	1 hour	immediately after unloading, and at every 20 mins. thereafter.
	50% of MTL		
	20% of MTL		
	0	12 hours	every 20 mins for first hour, and 12 hours after unloading.

Notes:
1. Pile is a steel pile; HP 310x110.
2. Apply and maintain loads incrementally up to Maximum Test Load (MTL) or until total pile head reaches 46.5 mm.
3. Maintain load for prescribed period or until movement rate is < 0.25 mm / hour (0.01 in. / hour), whicever occurs first.

THURBER

Appendix D

Pore Pressure and Settlement Monitoring Data

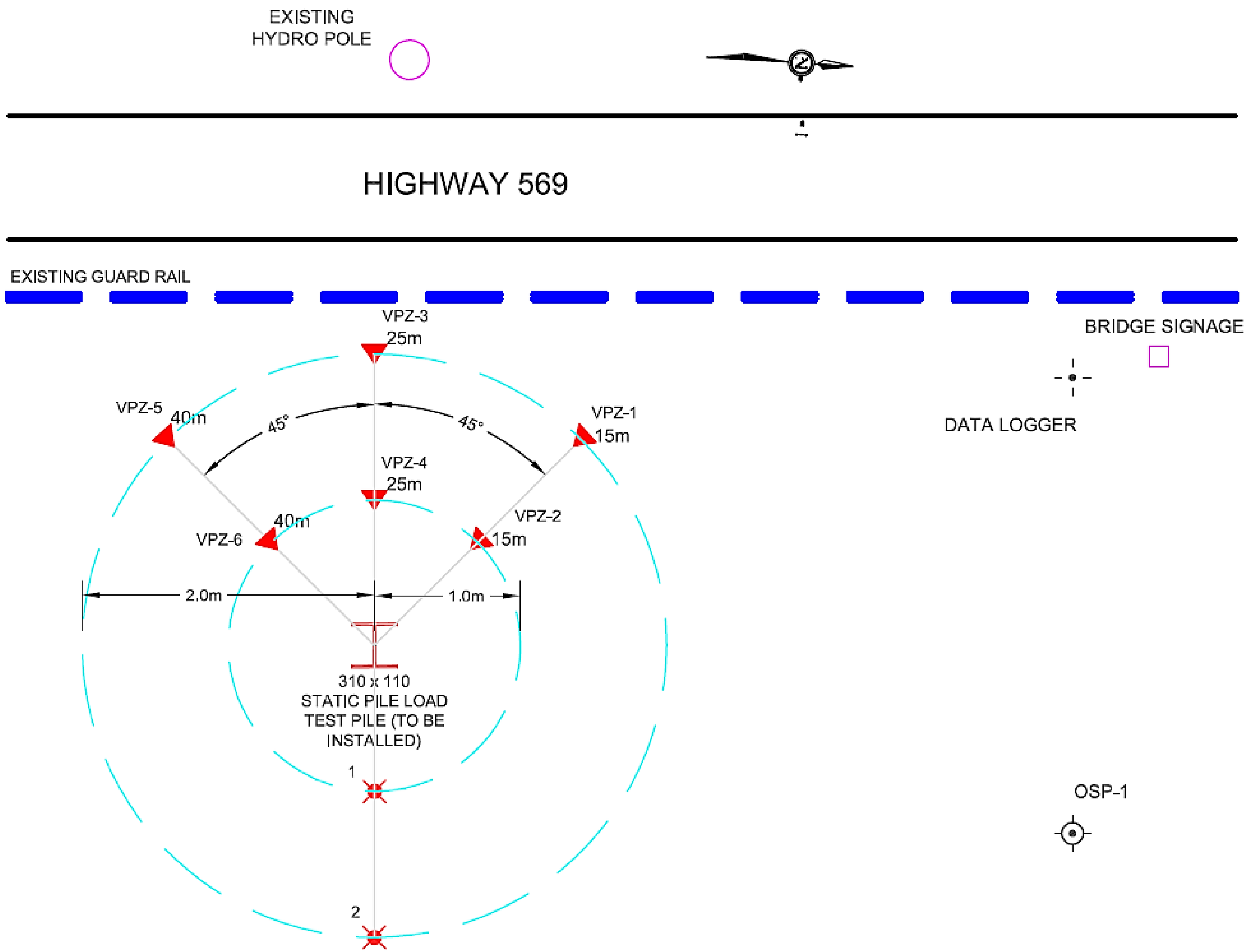


FIGURE 1 : GENERAL LAYOUT OF INSTRUMENTS

NOTES:

1. INSTRUMENTATION LOCATIONS SHOWN ON THE FIGURE ARE INDICATIVE; ACTUAL LOCATIONS TO BE RE-SURVEYED (BY OTHERS) AFTER PILE INSTALLATION TO OBTAIN AN AS-BUILT.
2. THIS DOCUMENT SHALL BE READ IN CONJUNCTION WITH "C18-19 LOAD TEST INSTRUMENTATION DETAILS" BY BERMINGHAM. LOCATION/COORDINATES OF SOIL INSTRUMENTS SHOWN ON THIS DOCUMENT SUPERSEDE LOCATIONS SHOWN ON BERMINGHAM DRAWING (REV. P5).

TABLE OF INSTRUMENTATION COORDINATES

	Northing (m)	Easting (m)
Test Pile	5288442.461	402611.049
SMP 1	5288442.522	402612.047
SMP 2	5288442.583	402613.045
VWP-1	5288443.786	402609.551
VWP-2	5288443.123	402610.300
VWP-3	5288442.339	402609.053
VWP-4	5288442.400	402610.051
VWP-5	5288440.963	402609.724
VWP-6	5288441.712	402610.387
OSP-1	5288438.539	402608.873

LEGEND

SETTLEMENT MONITORING POINT (SMP)

VIBRATING WIRE PIEZOMETER (VWP)

STANDPIPE PIEZOMETER (SSP)

TITLE : GENERAL LAYOUT PLAN OF SOIL INSTRUMENTATION

DATE : 2018 SEPTEMBER 5

PREPARED BY : M.CHOY

SCALE : NTS

REV. A

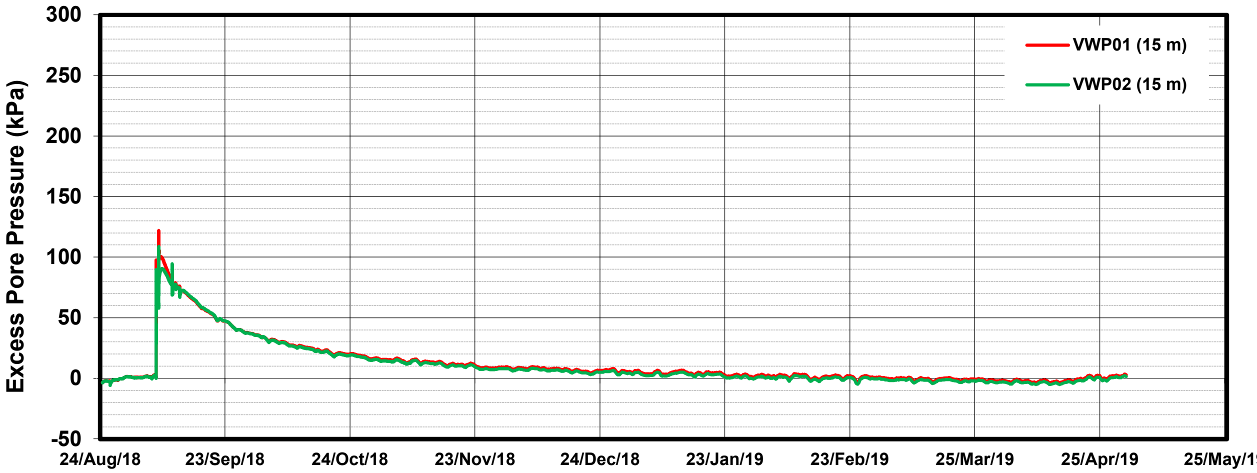
Vibrating Wire Piezometer (VWP) Readings

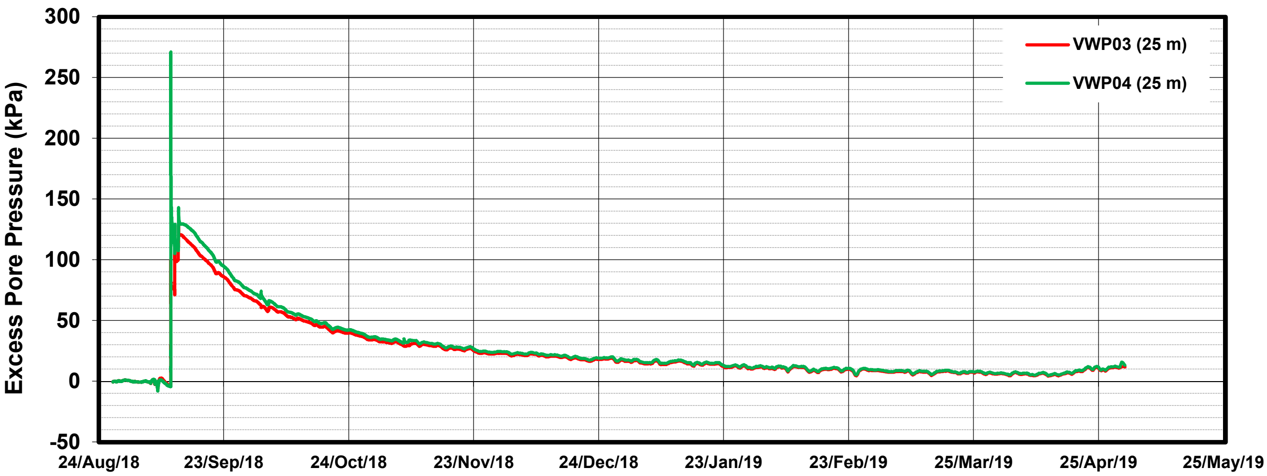
HWY 569 Blanche River Bridge

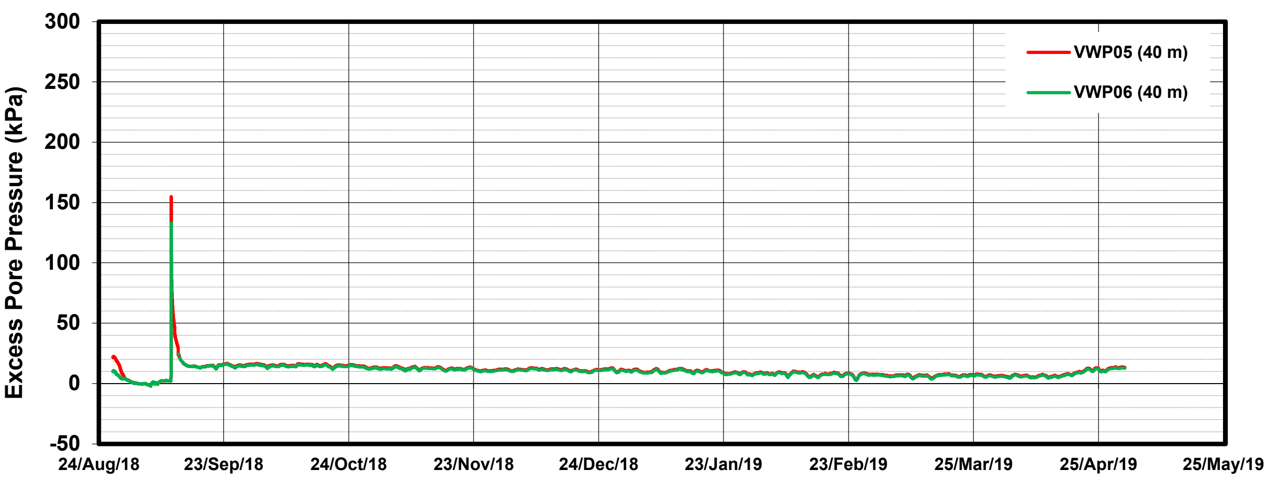
Vibrating Wire Piezometer (VWP) Reading Summary

VWP ID	Location	Ground Elevation (m)	Tip Elevation (m)	Piezometric Elevation (m)	Last Reading		Maximum Reading		Remaining Excess Pore Pressure
					Time	EPP (kPa)	Time	EPP (kPa)	
VWP01 (15 m)	2 m Radius	186.0	170.8	183.3	1/May/2019	3	7/Sep/2018	122	**2.5%**
VWP02 (15 m)	1 m Radius	186.0	171.0	184.2	1/May/2019	2	7/Sep/2018	109	**1.5%**
VWP03 (25 m)	2 m Radius	186.0	161.0	185.3	1/May/2019	12	12/Sep/2018	128	**9.4%**
VWP04 (25 m)	1 m Radius	186.0	161.0	185.3	1/May/2019	13	10/Sep/2018	271	**4.9%**
VWP05 (40 m)	2 m Radius	186.0	146.0	186.1	1/May/2019	13	10/Sep/2018	155	**8.6%**
VWP06 (40 m)	1 m Radius	186.0	146.0	187.3	1/May/2019	13	10/Sep/2018	133	**9.5%**

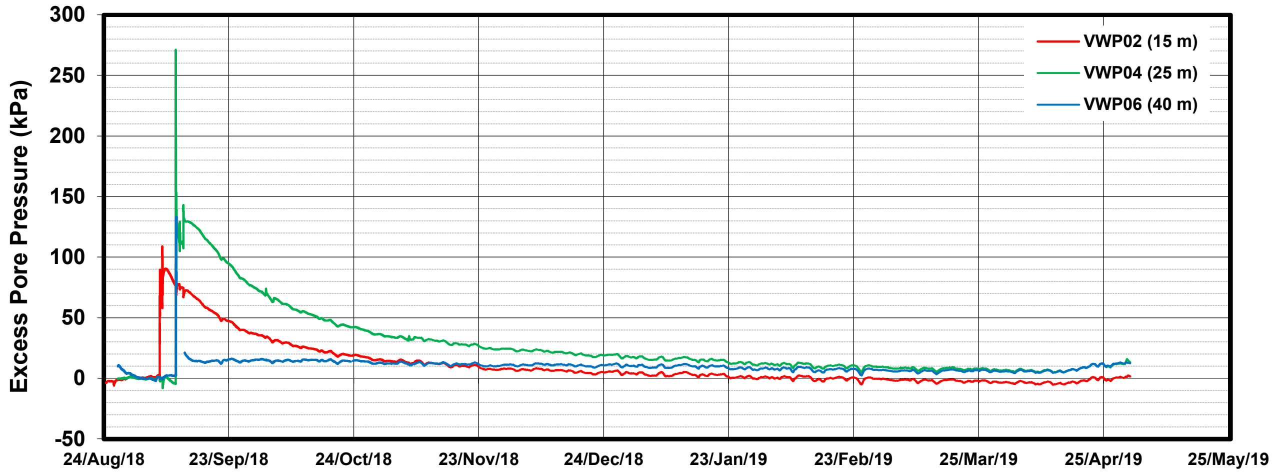
EXCESS PORE PRESSURE Vs. TIME
May 1, 2019
HWY 569 Blanche River Bridge Replacement - Pile Load Test Program
Vibrating Wire Piezometers



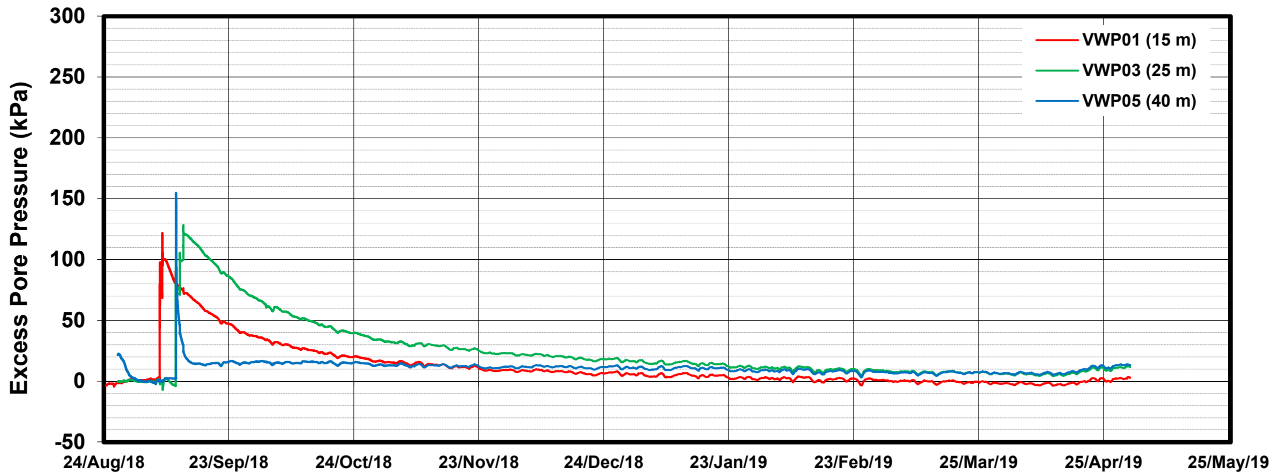




EXCESS PORE PRESSURE Vs. TIME
May 1, 2019
HWY 569 Blanche River Bridge Replacement - Pile Load Test Program
Vibrating Wire Piezometers



Excess Pore Pressure Responses at 1 m Radius



Excess Pore Pressure Responses at 2 m Radius

THURBER

Surface Settlement Monitoring Readings

NWL-01801046 - SETTLEMENT MONITORING HWY 569 - Blanche River Bridge

BASELINE READING - September 6, 2018					
LOCATION	SEPTEMBER 6,2018 BASELINE ELEV. (m)	SEPTEMBER 28,2018 Settlement Monitoring #1 ELEV. (m)	DIFFERENCE BETWEEN SM#1-Baseline (m)	November 9, 2018 Settlement Monitoring #2 ELEV. (m)	DIFFERENCE BETWEEN SM#2-Baseline (m)
BM 314	189.52300				
SMP 1	185.94000	185.93198	-0.00802	185.9306	-0.00940
SMP2	185.95000	185.93648	-0.01352	185.9373	-0.01270
TBM SE ABUTMENT	185.72300	185.72300	0.00000	185.723	0.00000
TBM SW ABUTMENT	185.715	185.715	0.00000	185.715	0.00000

Appendix E

CPTu Test Report

PRESENTATION OF SITE INVESTIGATION RESULTS

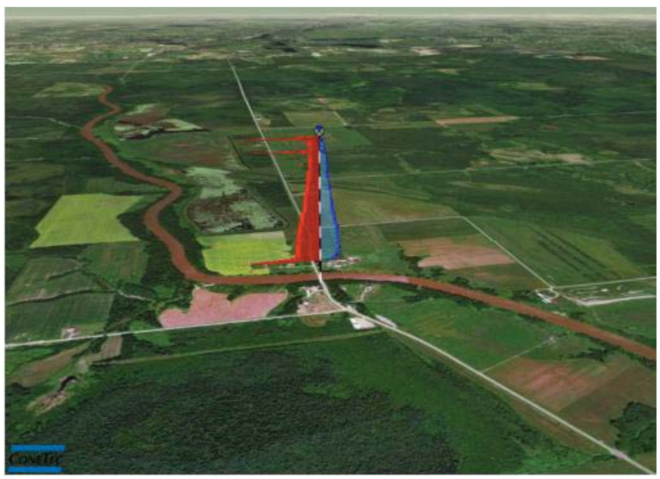
Blanche River Test Pile Program

Prepared for:

Bermingham Foundation Solutions

ConeTec Job No: 18-05053

Project Start Date: 04-Sep-2018
Project End Date: 04-Sep-2018
Report Date: 10-Sep-2018



Prepared by:

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www.conetecdataservices.com

Introduction

The enclosed report presents the results of the site investigation program conducted by ConeTec Investigations Ltd. for Bermingham Foundation Solutions at Blanche River and Highway 569, north of Temiskaming Shores, ON. The program consisted of one cone penetration test (CPT).

Project Information

Project	
Client	Bermingham Foundation Solutions
Project	Blanche River Test Pile Program
ConeTec project number	18-05053

An aerial overview from Google Earth including the CPT location is presented below.

Rig Description	Deployment System	Test Type
CPT truck (C3)	30 ton rig cylinder	CPT

Coordinates		
Test Type	Collection Method	EPSG Number
CPT	Consumer grade GPS	32617

Cone Penetration Test (CPT)	
Depth reference	Depths are referenced to the existing ground surface at the time of each test.
Tip and sleeve data offset	0.1 meter This has been accounted for in the CPT data files.
Additional plots	Advanced CPT plots with Ic, Su, OCR, and N160 (Ic RW1998), and Soil Behavior Type (SBT) scatter plots have been included in the data release package.
Additional Comments	Depths were corrected for measured inclination deviation from vertical.

Cone Penetrometers Used for this Project						
Cone Description	Cone Number	Cross Sectional Area (cm^2)	Sleeve Area (cm^2)	Tip Capacity (bar)	Sleeve Capacity (bar)	Pore Pressure Capacity (psi)
428:T1000F10U500	428	10	150	1000	10	500

Calculated Geotechnical Parameter Tables	
Additional information	The Normalized Soil Behaviour Type Chart based on Q_{tn} (SBT Q_{tn}) (Robertson, 2009) was used to classify the soil for this project. A detailed set of calculated CPT parameters have been generated and are provided in Excel format files in the release folder. The CPT parameter calculations are based on values of corrected tip resistance (q_t) sleeve friction (f_s) and pore pressure (u_2). Effective stresses were calculated based on unit weights that have been assigned to the individual soil behaviour type zones and the assumed equilibrium pore pressure profile. Soils were classified as either drained or undrained based on the Q_{tn} Normalized Soil Behaviour Type Chart (Robertson, 2009). Both drained and undrained parameters were calculated for soils that classified as silt mixtures (zone 4).

Limitations

This report has been prepared for the exclusive use of Bermingham Foundation Solutions (Client) for the project titled “Blanche River Test Pile Program”. The report’s contents may not be relied upon by any other party without the express written permission of ConeTec Investigations Ltd. (ConeTec). ConeTec has provided site investigation services, prepared the factual data reporting and provided geotechnical parameter calculations consistent with current best practices. No other warranty, expressed or implied, is made.

The information presented in the report document and the accompanying data set pertain to the specific project, site conditions and objectives described to ConeTec by the Client. In order to properly understand the factual data, assumptions and calculations, reference must be made to the documents provided and their accompanying data sets, in their entirety.

The cone penetration tests (CPTu) are conducted using an integrated electronic piezocone penetrometer and data acquisition system manufactured by Adara Systems Ltd. of Richmond, British Columbia, Canada.

ConeTec's piezocone penetrometers are compression type designs in which the tip and friction sleeve load cells are independent and have separate load capacities. The piezocones use strain gauged load cells for tip and sleeve friction and a strain gauged diaphragm type transducer for recording pore pressure. The piezocones also have a platinum resistive temperature device (RTD) for monitoring the temperature of the sensors, an accelerometer type dual axis inclinometer and a geophone sensor for recording seismic signals. All signals are amplified down hole within the cone body and the analog signals are sent to the surface through a shielded cable.

ConeTec penetrometers are manufactured with various tip, friction and pore pressure capacities in both 10 cm^2 and 15 cm^2 tip base area configurations in order to maximize signal resolution for various soil conditions. The specific piezocone used for each test is described in the CPT summary table presented in the first Appendix. The 15 cm^2 penetrometers do not require friction reducers as they have a diameter larger than the deployment rods. The 10 cm^2 piezocones use a friction reducer consisting of a rod adapter extension behind the main cone body with an enlarged cross sectional area (typically 44 mm diameter over a length of 32 mm with tapered leading and trailing edges) located at a distance of 585 mm above the cone tip.

The penetrometers are designed with equal end area friction sleeves, a net end area ratio of 0.8 and cone tips with a 60 degree apex angle.

All ConeTec piezocones can record pore pressure at various locations. Unless otherwise noted, the pore pressure filter is located directly behind the cone tip in the "u_2" position (ASTM Type 2). The filter is 6 mm thick, made of porous plastic (polyethylene) having an average pore size of 125 microns (90-160 microns). The function of the filter is to allow rapid movements of extremely small volumes of water needed to activate the pressure transducer while preventing soil ingress or blockage.

The piezocone penetrometers are manufactured with dimensions, tolerances and sensor characteristics that are in general accordance with the current ASTM D5778 standard. ConeTec's calibration criteria also meets or exceeds those of the current ASTM D5778 standard. An illustration of the piezocone penetrometer is presented in Figure CPTu.

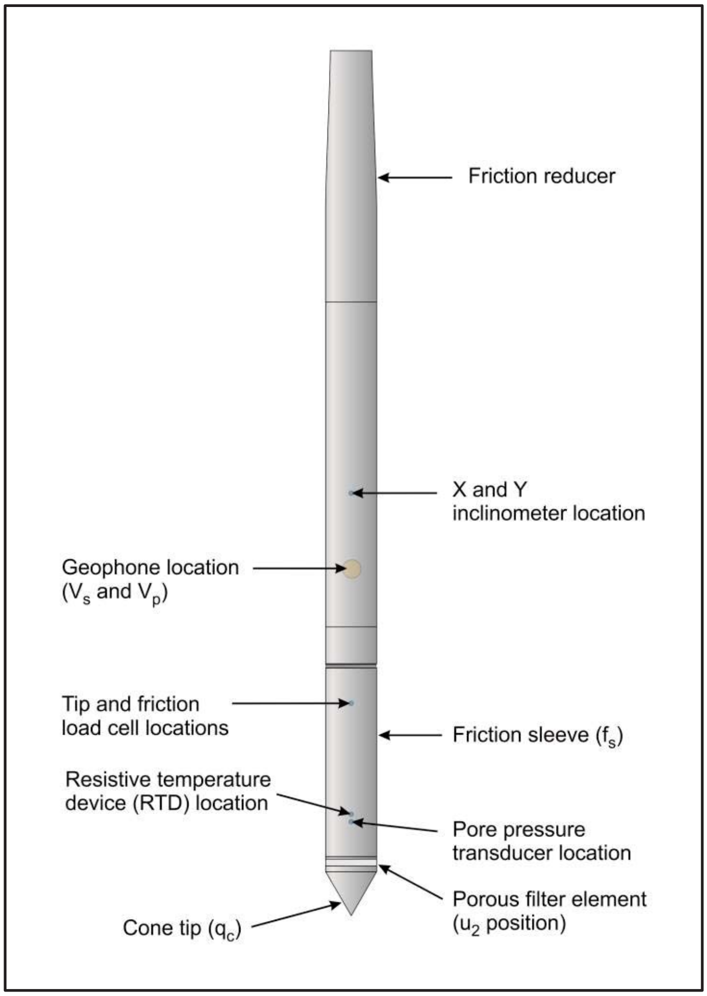

Figure CPTu. Piezocone Penetrometer (15 cm^2)

The ConeTec data acquisition systems consist of a Windows based computer and a signal conditioner and power supply interface box with a 16 bit (or greater) analog to digital (A/D) converter. The data is recorded at fixed depth increments using a depth wheel attached to the push cylinders or by using a spring loaded rubber depth wheel that is held against the cone rods. The typical recording intervals are either 2.5 cm or 5.0 cm depending on project requirements; custom recording intervals are possible. The system displays the CPTu data in real time and records the following parameters to a storage media during penetration:

- Depth
- Uncorrected tip resistance (q_c)
- Sleeve friction (f_s)
- Dynamic pore pressure (u)
- Additional sensors such as resistivity, passive gamma, ultra violet induced fluorescence, if applicable

All testing is performed in accordance to ConeTec's CPT operating procedures which are in general accordance with the current ASTM D5778 standard.

Prior to the start of a CPTu sounding a suitable cone is selected, the cone and data acquisition system are powered on, the pore pressure system is saturated with either glycerine or silicone oil and the baseline readings are recorded with the cone hanging freely in a vertical position.

The CPTu is conducted at a steady rate of 2 cm/s, within acceptable tolerances. Typically one meter length rods with an outer diameter of 1.5 inches are added to advance the cone to the sounding termination depth. After cone retraction final baselines are recorded.

Additional information pertaining to ConeTec's cone penetration testing procedures:

- Each filter is saturated in silicone oil or glycerine under vacuum pressure prior to use
- Recorded baselines are checked with an independent multi-meter
- Baseline readings are compared to previous readings
- Soundings are terminated at the client's target depth or at a depth where an obstruction is encountered, excessive rod flex occurs, excessive inclination occurs, equipment damage is likely to take place, or a dangerous working environment arises
- Differences between initial and final baselines are calculated to ensure zero load offsets have not occurred and to ensure compliance with ASTM standards

The interpretation of piezocone data for this report is based on the corrected tip resistance (q_t), sleeve friction (f_s) and pore water pressure (u). The interpretation of soil type is based on the correlations developed by Robertson (1990) and Robertson (2009). It should be noted that it is not always possible to accurately identify a soil type based on these parameters. In these situations, experience, judgment and an assessment of other parameters may be used to infer soil behaviour type.

The recorded tip resistance (q_c) is the total force acting on the piezocone tip divided by its base area. The tip resistance is corrected for pore pressure effects and termed corrected tip resistance (q_t) according to the following expression presented in Robertson et al, 1986:

$$q_t = q_c + (1\text{-}a) \bullet u_2$$

where: q_t is the corrected tip resistance
q_c is the recorded tip resistance
u_2 is the recorded dynamic pore pressure behind the tip (u_2 position)
a is the Net Area Ratio for the piezocone (0.8 for ConeTec probes)

The sleeve friction (f_s) is the frictional force on the sleeve divided by its surface area. As all ConeTec piezocones have equal end area friction sleeves, pore pressure corrections to the sleeve data are not required.

The dynamic pore pressure (u) is a measure of the pore pressures generated during cone penetration. To record equilibrium pore pressure, the penetration must be stopped to allow the dynamic pore pressures to stabilize. The rate at which this occurs is predominantly a function of the permeability of the soil and the diameter of the cone.

The friction ratio (Rf) is a calculated parameter. It is defined as the ratio of sleeve friction to the tip resistance expressed as a percentage. Generally, saturated cohesive soils have low tip resistance, high friction ratios and generate large excess pore water pressures. Cohesionless soils have higher tip resistances, lower friction ratios and do not generate significant excess pore water pressure.

A summary of the CPTu soundings along with test details and individual plots are provided in the appendices. A set of interpretation files were generated for each sounding based on published correlations and are provided in Excel format in the data release folder. Information regarding the interpretation methods used is also included in the data release folder.

For additional information on CPTu interpretations, refer to Robertson et al. (1986), Lunne et al. (1997), Robertson (2009), Mayne (2013, 2014) and Mayne and Peuchen (2012).

The cone penetration test is halted at specific depths to carry out pore pressure dissipation (PPD) tests, shown in Figure PPD-1. For each dissipation test the cone and rods are decoupled from the rig and the data acquisition system measures and records the variation of the pore pressure (u) with time (t).

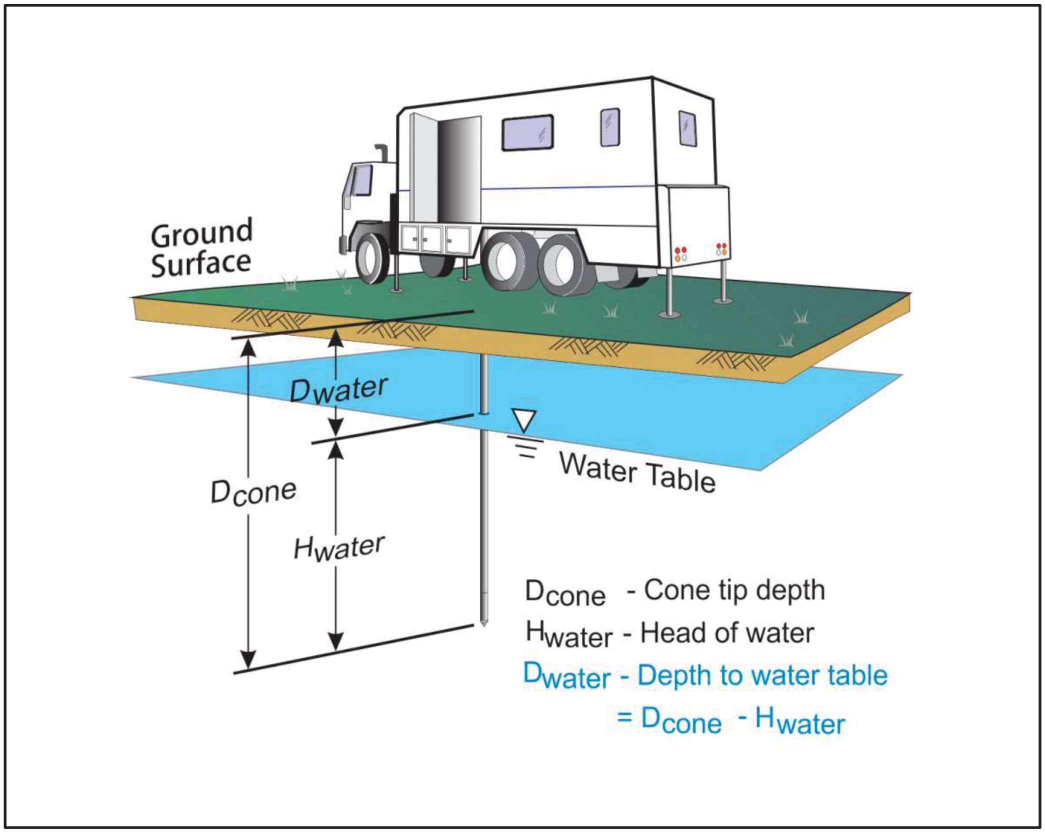

Figure PPD-1. Pore pressure dissipation test setup

Pore pressure dissipation data can be interpreted to provide estimates of ground water conditions, permeability, consolidation characteristics and soil behaviour.

The typical shapes of dissipation curves shown in Figure PPD-2 are very useful in assessing soil type, drainage, in situ pore pressure and soil properties. A flat curve that stabilizes quickly is typical of a freely draining sand. Undrained soils such as clays will typically show positive excess pore pressure and have long dissipation times. Dilative soils will often exhibit dynamic pore pressures below equilibrium that then rise over time. Overconsolidated fine-grained soils will often exhibit an initial dilatory response where there is an initial rise in pore pressure before reaching a peak and dissipating.

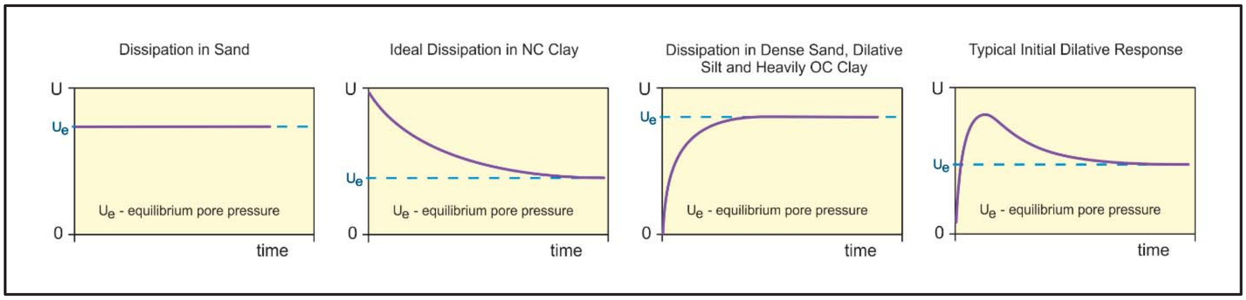

Figure PPD-2. Pore pressure dissipation curve examples

In order to interpret the equilibrium pore pressure (u_{eq}) and the apparent phreatic surface, the pore pressure should be monitored until such time as there is no variation in pore pressure with time as shown for each curve of Figure PPD-2.

In fine grained deposits the point at which 100% of the excess pore pressure has dissipated is known as t_{100}. In some cases this can take an excessive amount of time and it may be impractical to take the dissipation to t_{100}. A theoretical analysis of pore pressure dissipations by Teh and Houlsby (1991) showed that a single curve relating degree of dissipation versus theoretical time factor (T*) may be used to calculate the coefficient of consolidation (c_h) at various degrees of dissipation resulting in the expression for c_h shown below.

$$c_h = \frac{T^* \cdot a^2 \cdot \sqrt{I_r}}{t}$$

Where:

T*	is the dimensionless time factor (Table Time Factor)
a	is the radius of the cone
I_r	is the rigidity index
t	is the time at the degree of consolidation

Table Time Factor. T* versus degree of dissipation (Teh and Houlsby, 1991)

Degree of Dissipation (%)	20	30	40	50	60	70	80
T* (u_2)	0.038	0.078	0.142	0.245	0.439	0.804	1.60

The coefficient of consolidation is typically analyzed using the time (t_{50}) corresponding to a degree of dissipation of 50% (u_{50}). In order to determine t_{50}, dissipation tests must be taken to a pressure less than u_{50}. The u_{50} value is half way between the initial maximum pore pressure and the equilibrium pore pressure value, known as u_{100}. To estimate u_{50}, both the initial maximum pore pressure and u_{100} must be known or estimated. Other degrees of dissipations may be considered, particularly for extremely long dissipations.

At any specific degree of dissipation the equilibrium pore pressure (u at t_{100}) must be estimated at the depth of interest. The equilibrium value may be determined from one or more sources such as measuring the value directly (u_{100}), estimating it from other dissipations in the same profile, estimating the phreatic surface and assuming hydrostatic conditions, from nearby soundings, from client provided information, from site observations and/or past experience, or from other site instrumentation.

For calculations of c_h (Teh and Houlsby, 1991), t_{50} values are estimated from the corresponding pore pressure dissipation curve and a rigidity index (I_r) is assumed. For curves having an initial dilatory response in which an initial rise in pore pressure occurs before reaching a peak, the relative time from the peak value is used in determining t_{50}. In cases where the time to peak is excessive, t_{50} values are not calculated.

Due to possible inherent uncertainties in estimating I_r, the equilibrium pore pressure and the effect of an initial dilatory response on calculating t_{50}, other methods should be applied to confirm the results for c_h.

Additional published methods for estimating the coefficient of consolidation from a piezocone test are described in Burns and Mayne (1998, 2002), Jones and Van Zyl (1981), Robertson et al. (1992) and Sully et al. (1999).

A summary of the pore pressure dissipation tests and dissipation plots are presented in the relevant appendix.

ASTM D5778-12, 2012, "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils", ASTM, West Conshohocken, US.

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The appendices listed below are included in the report:

- Cone Penetration Test Summary and Standard Cone Penetration Test Plots
- Advanced Cone Penetration Test Plots
- Soil Behavior Type (SBT) Scatter Plots
- Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

Cone Penetration Test Summary and Standard Cone Penetration Test Plots

ConeTec

Job No:	18-05053
Client:	Bermingham Foundation Solutions
Project:	Blanche River Test Pile Program
Start Date:	04-Sep-2018
End Date:	04-Sep-2018

CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Cone	Assumed Phreatic Surface[1] (m)	Final Depth (m)	Northing[2] (m)	Easting (m)
CPT18-01	18-05053_CP01	04-Sep-18	428:T1000F10U500	0.5	49.65	5286865	597774

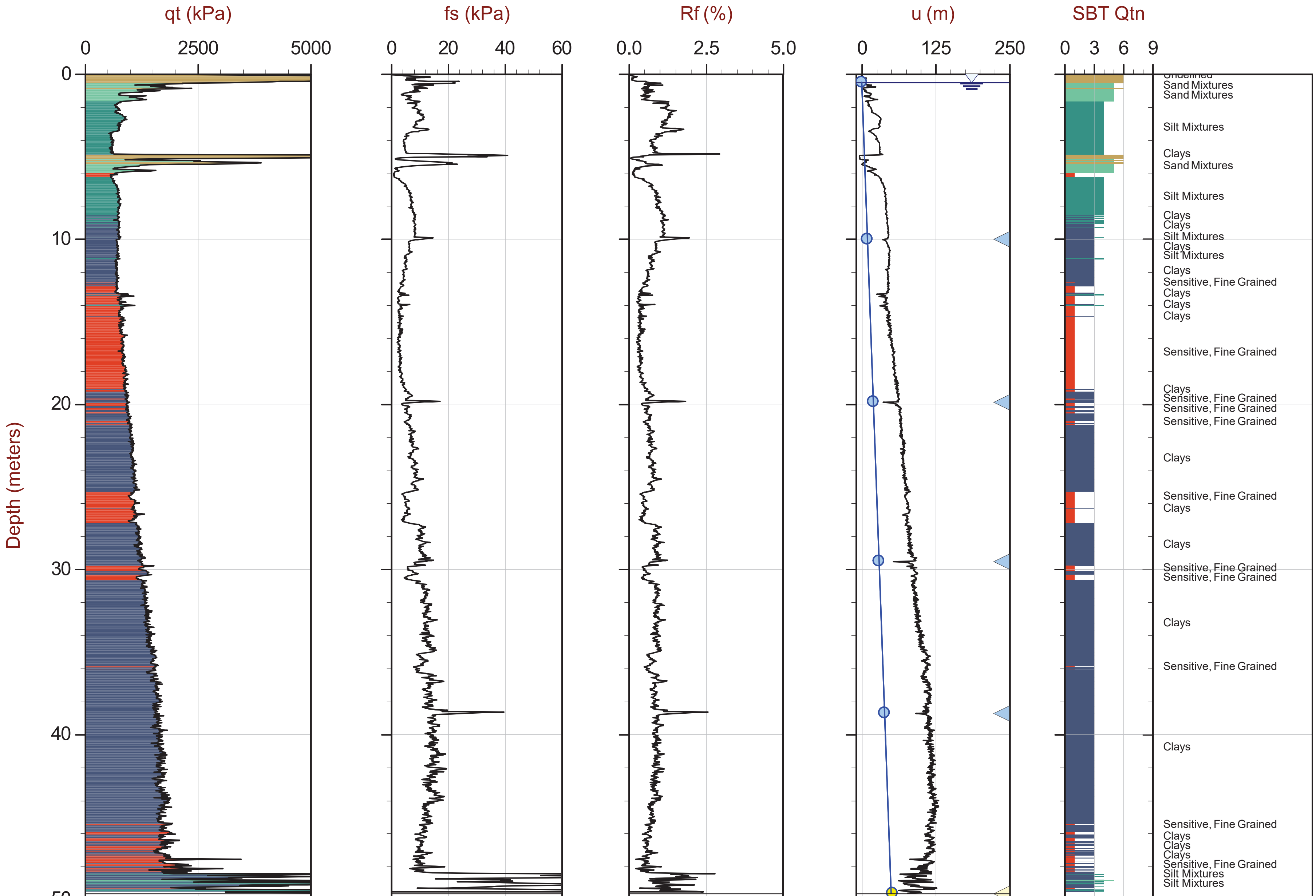
1. The assumed phreatic surface was based on the dynamic pore pressure response. Hydrostatic conditions were assumed for the calculated parameters.
2. Coordinates were acquired using consumer grade GPS equipment in datum WGS1984/UTM Zone 17 North.

ConeTec

Bermingham Foundation Solutions

Job No: 18-05053
Date: 2018-09-04 09:19
Site: Hwy 569 and Hilliardton Rd, New Liskeard, ON

Sounding: CPT18-01
Cone: 428:T1000F10U500



Max Depth: 49.650 m / 162.89 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 18-05053_CP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 5286865m E: 597774m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Hydrostatic Line Dissipation, Ueq assumed

Advanced Cone Penetration Test Plots

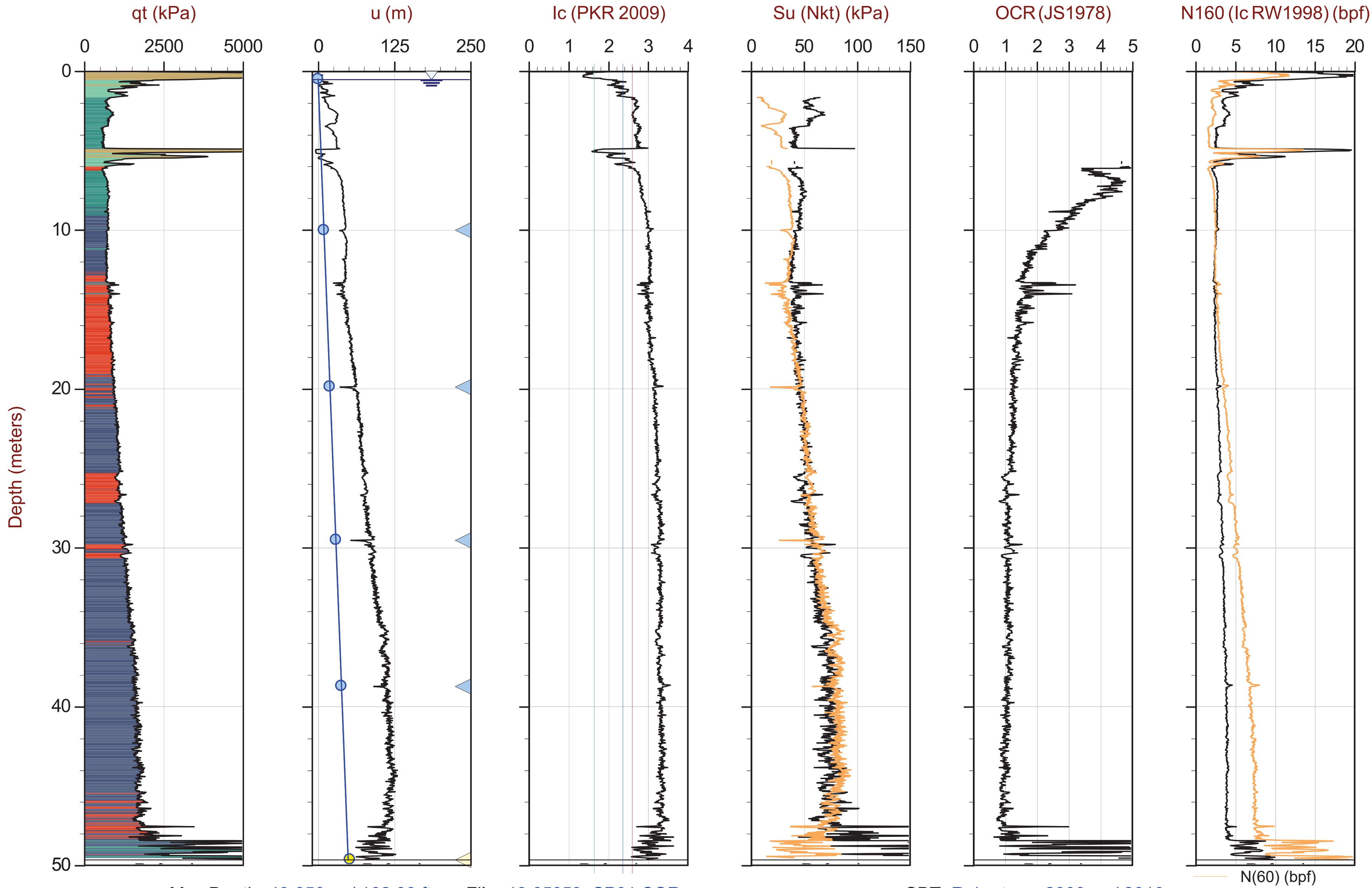
CONETEC

CONETEC

Bermingham Foundation Solutions

Job No: 18-05053
Date: 2018-09-04 09:19
Site: Hwy 569 and Hilliardton Rd, New Liskeard, ON

Sounding: CPT18-01
Cone: 428:T1000F10U500



Max Depth: 49.650 m / 162.89 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 18-05053_CP01.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 12.5 / 9.0

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 5286865m E: 597774m
Sheet No: 1 of 1

Overplot Item: Ueq | Assumed Ueq | Dissipation, Ueq achieved | Hydrostatic Line | Dissipation, Ueq assumed

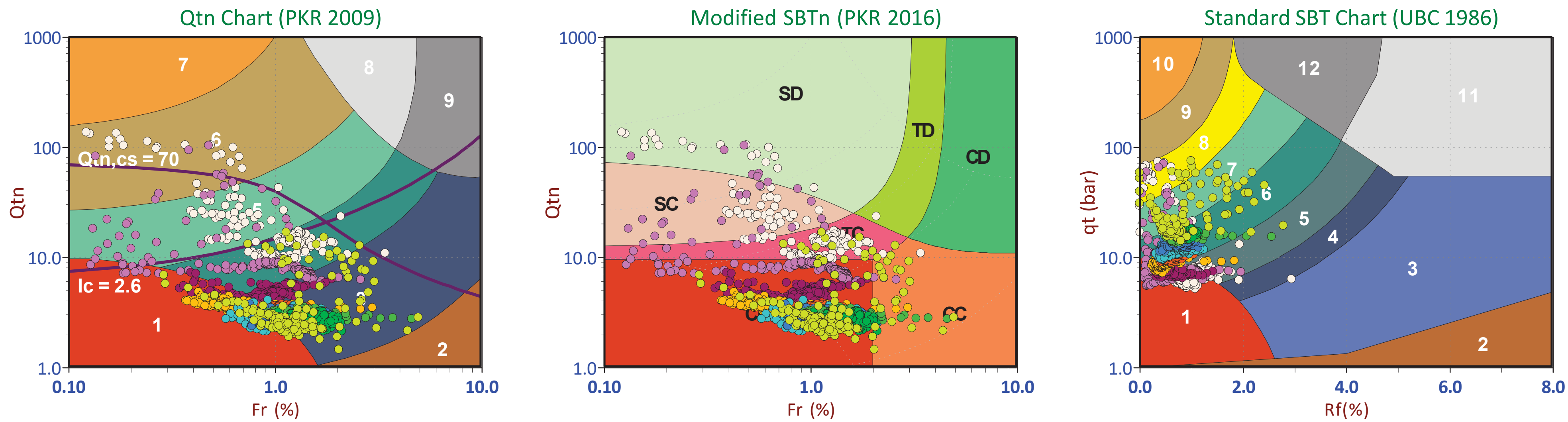
Soil Behaviour Type (SBT) Scatter Plots

ConeTec

CONETEC

Bermingham Foundation Solutions

Job No: 18-05053
Date: 2018-09-04 09:19
Site: Hwy 569 and Hilliardton Rd, New Liskeard, ON
Sounding: CPT18-01
Cone: 428:T1000F10U500



Depth Ranges
- >0.0 to 5.0 m
- >5.0 to 10.0 m
- >10.0 to 15.0 m
- >15.0 to 20.0 m
- >20.0 to 25.0 m
- >25.0 to 30.0 m
- >30.0 to 35.0 m
- >35.0 to 40.0 m
- >40.0 to 45.0 m
- >45.0 to 50.0 m
- >50.0 m

Legend
- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

Legend
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

Pore Pressure Dissipation Summary and
Pore Pressure Dissipation Plots

Job No: 18-05053
Client: Bermingham Foundation Solutions
Project: Blanche River Test Pile Program
Start Date: 04-Sep-2018
End Date: 04-Sep-2018

CPTu PORE PRESSURE DISSIPATION SUMMARY

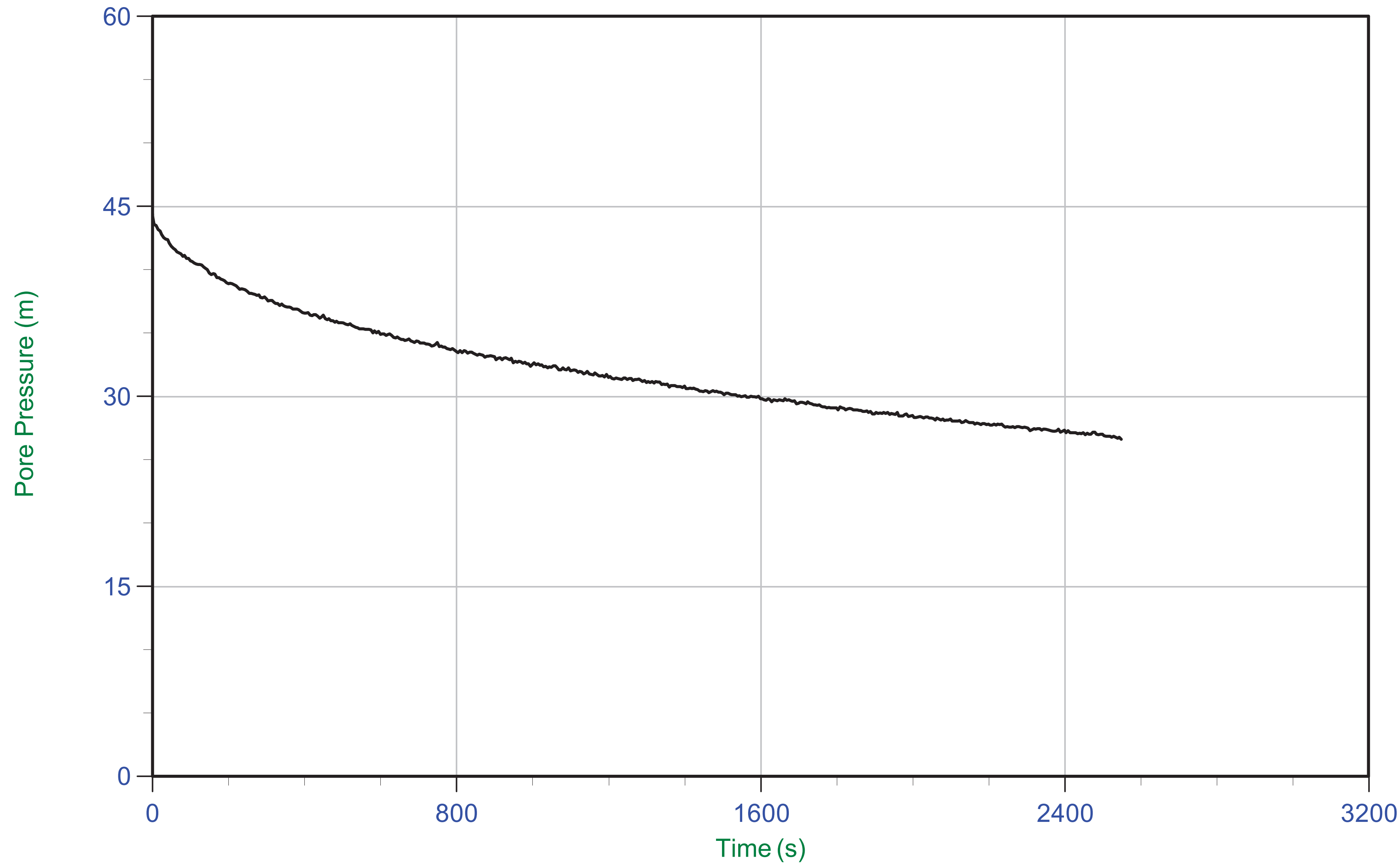
Sounding ID	File Name	Cone Area (cm^2)	Duration (s)	Test Depth (m)	Estimated Equilibrium Pore Pressure U_{eq} (m)	Calculated Phreatic Surface (m)	Estimated Phreatic Surface (m)	t_{50}[a] (s)	Assumed Rigidity Index (I_r)	c_h[b] (cm^2/min)
CPT18-01	18-05053_CP01	10	2550	10.000	9.5		0.5	2503	100	0.19
CPT18-01	18-05053_CP01	10	1500	19.875	19.4		0.5	533	100	0.88
CPT18-01	18-05053_CP01	10	2100	29.525	29.0		0.5	627	100	0.75
CPT18-01	18-05053_CP01	10	2200	38.725	38.2		0.5	718	100	0.66
CPT18-01	18-05053_CP01	10	500	49.650	52.0	-2.3				

a. Time is relative to where umax occurred
b. Houlsby and Teh, 1991

Bermingham Foundation Solutions

Job No: 18-05053
Date: 09/04/2018 09:19
Site: Hwy 569 and Hilliardton Rd, New Liskeard, ON

Sounding: CPT18-01
Cone: 428:T1000F10U500 Area=10 cm²



Trace Summary:

Filename: 18-05053_CP01.PPF
Depth: 10.000 m / 32.808 ft
Duration: 2550.0 s

U Min: 26.6 m
U Max: 44.4 m

WT: 0.500 m / 1.640 ft
Ueq: 9.5 m
U(50): 26.96 m

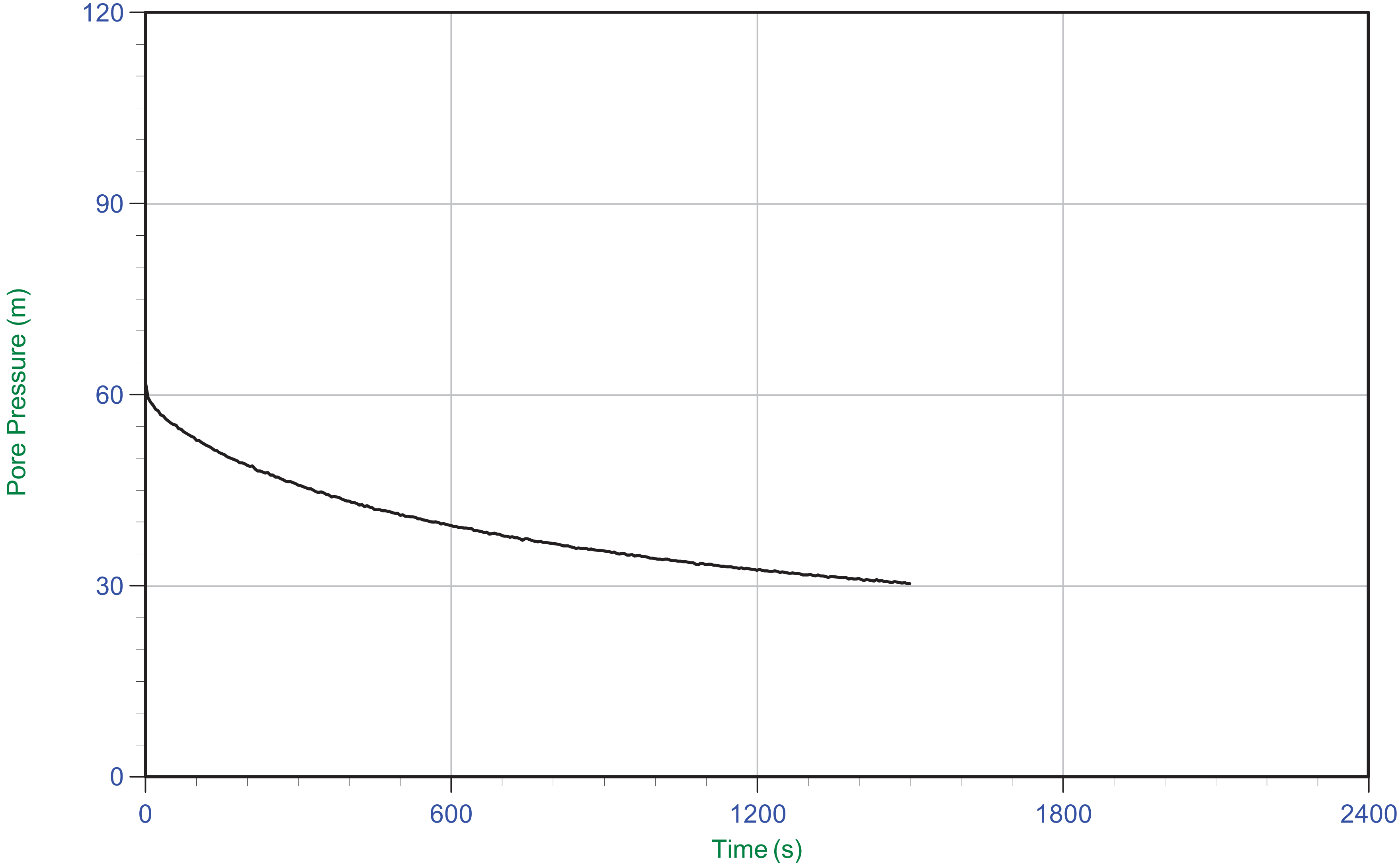
T(50): 2502.6 s
Ir: 100
Ch: 0.2 cm²/min

ConeTec

Bermingham Foundation Solutions

Job No: 18-05053
Date: 09/04/2018 09:19
Site: Hwy 569 and Hilliardton Rd, New Liskeard, ON

Sounding: CPT18-01
Cone: 428:T1000F10U500 Area=10 cm²



Trace Summary:

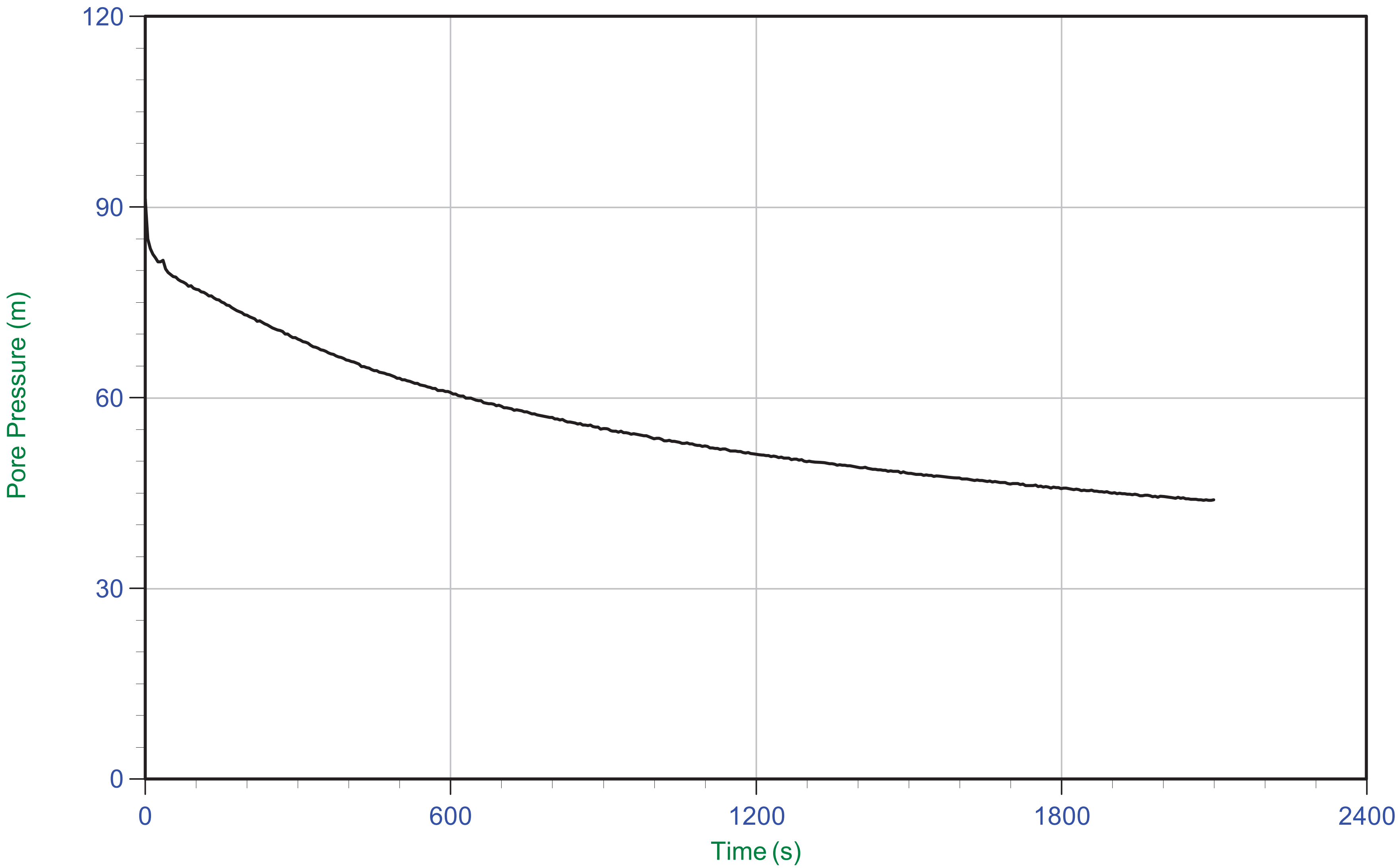
Filename: 18-05053_CP01.PPF
Depth: 19.875 m / 65.206 ft
Duration: 1500.0 s

U Min: 30.3 m
U Max: 62.0 m

WT: 0.500 m / 1.640 ft
Ueq: 19.4 m
U(50): 40.67 m

T(50): 532.6 s
Ir: 100
Ch: 0.9 cm²/min

Bermingham Foundation Solutions	Job No: 18-05053 Date: 09/04/2018 09:19 Site: Hwy 569 and Hilliardton Rd, New Liskeard, ON	Sounding: CPT18-01 Cone: 428:T1000F10U500 Area=10 cm²

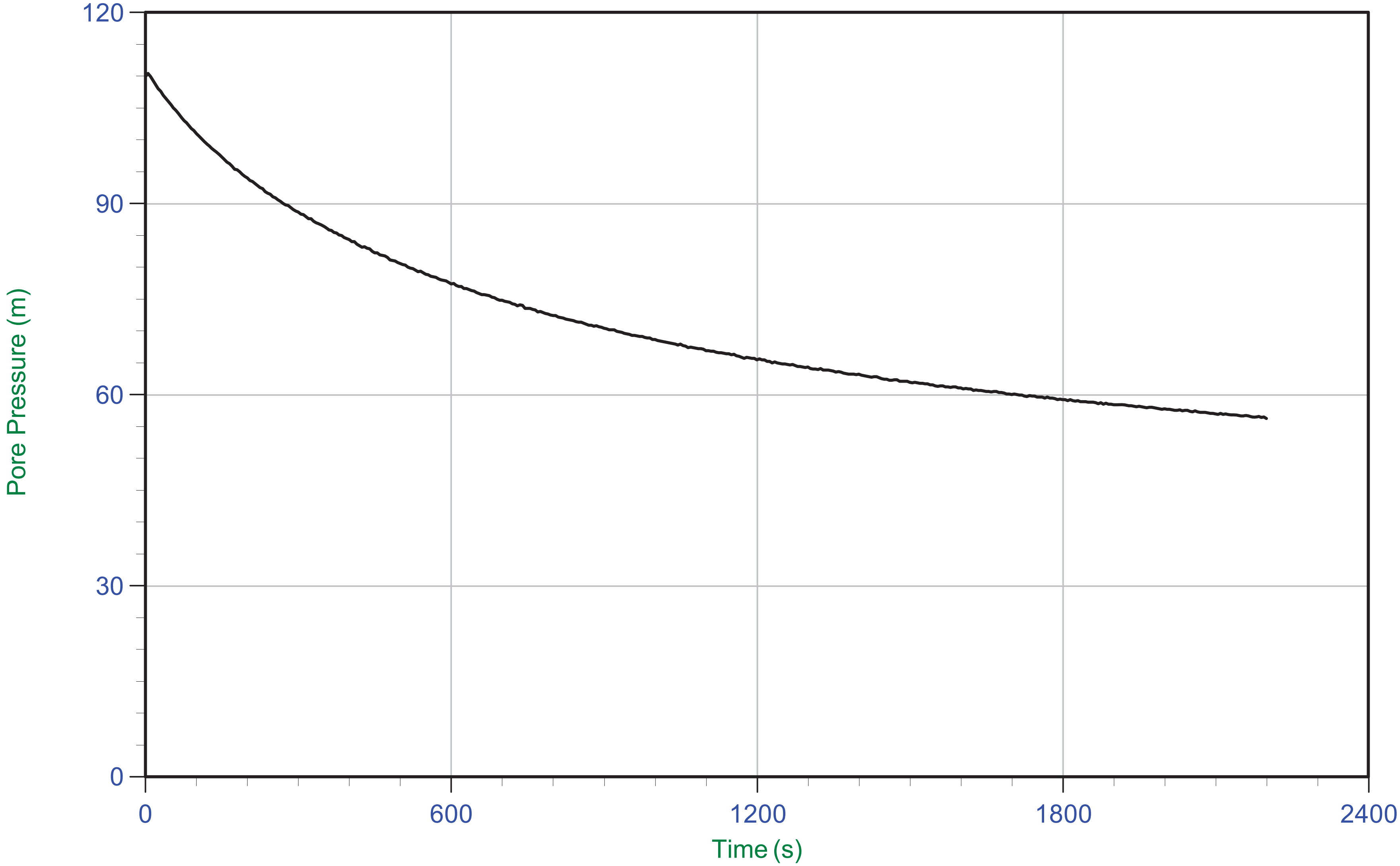


Trace Summary:				
	Filename: 18-05053_CP01.PPF	U Min: 43.9 m	WT: 0.500 m / 1.640 ft	T(50): 626.6 s
	Depth: 29.525 m / 96.866 ft	U Max: 91.3 m	Ueq: 29.0 m	Ir: 100
	Duration: 2100.0 s		U(50): 60.18 m	Ch: 0.7 cm²/min

Bermingham Foundation Solutions

Job No: 18-05053
Date: 09/04/2018 09:19
Site: Hwy 569 and Hilliardton Rd, New Liskeard, ON

Sounding: CPT18-01
Cone: 428:T1000F10U500 Area=10 cm²



Trace Summary:

Filename: 18-05053_CP01.PPF
Depth: 38.725 m / 127.049 ft
Duration: 2200.0 s

U Min: 56.3 m
U Max: 110.4 m

WT: 0.500 m / 1.640 ft
Ueq: 38.2 m
U(50): 74.33 m

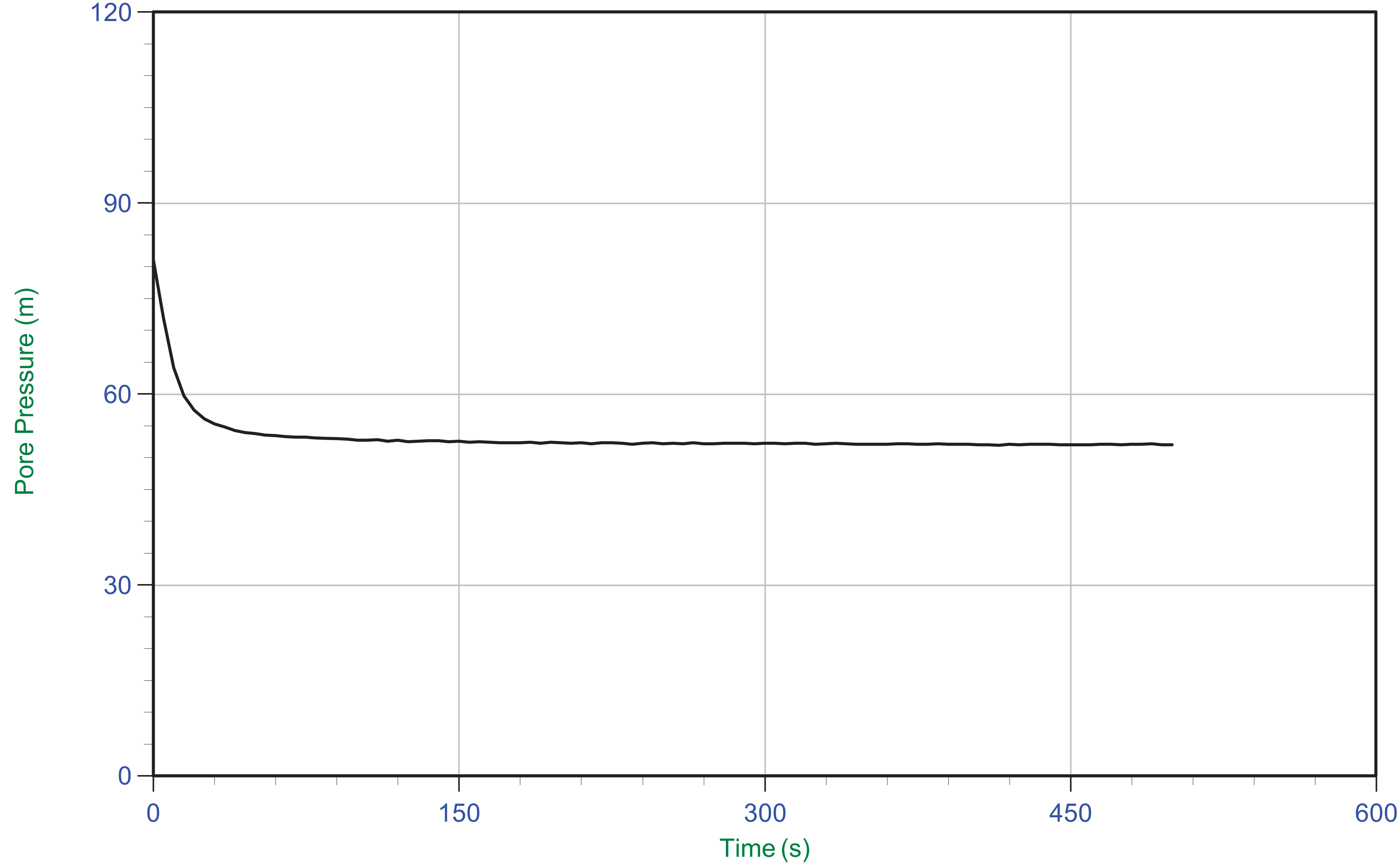
T(50): 713.4 s
Ir: 100
Ch: 0.7 cm²/min

ConeTec

Bermingham Foundation Solutions

Job No: 18-05053
Date: 09/04/2018 09:19
Site: Hwy 569 and Hilliardton Rd, New Liskeard, ON

Sounding: CPT18-01
Cone: 428:T1000F10U500 Area=10 cm²



Trace Summary:

Filename: 18-05053_CP01.PPF
Depth: 49.650 m / 162.892 ft
Duration: 500.0 s

U Min: 52.0 m
U Max: 81.1 m

WT: -2.341 m / -7.680 ft
Ueq: 52.0 m