



**THURBER** ENGINEERING LTD.

**FOUNDATION INVESTIGATION AND DESIGN REPORT  
SITE 38S-154 – McLEOD ROAD BRIDGE REPLACEMENT  
SAULT STE. MARIE DISTRICT, ALGOMA COUNTY  
G.W.P. 5148-13-00  
ASSIGNMENT NUMBER: 5014-E-0032**

**GEOCRES NUMBER: 41J-103**

**SUBMITTED TO  
McINTOSH PERRY CONSULTING ENGINEERS**

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**PART 1: FACTUAL INFORMATION**

**1 INTRODUCTION**

This report presents the factual data obtained from a foundation investigation conducted by Thurber Engineering Ltd. (Thurber) for the replacement of the McLeod Road Bridge crossing of the Thessalon River located within the Sault Ste. Marie district. Thurber carried out the investigation as a subconsultant to McIntosh Perry Consulting Engineers (MPCE) as part of Agreement No. 5014-E-0032.

No previous foundation investigation information for the subject site was available. Base plan mapping and survey data was provided by MPCE for the preparation of this report.

The purpose of this investigation was to explore the subsurface conditions at the site and, based on this data, provide a borehole location plan, record of boreholes, a stratigraphic profile, laboratory test results and a written description of the subsurface conditions.

**2 SITE DESCRIPTION**

Site 38S-154 is located on McLeod Road, 350 m west of the McLeod Road Junction with Centre Line Road near Leeburn, Ontario. The location of the structure is shown on the inset Key Plan on the Borehole Locations and Soil Strata Drawing No. 1 in Appendix A.

The existing four-span, Bailey Bridge structure carries a single lane of traffic over the Thessalon River. The bridge is approximately 37 m long, 4.0 m wide with approximate span lengths of 7.9 m, 10.3 m, 10.3 m, and 7.9 m. It is noted that for project orientation purposes, McLeod Road, will be assumed to run north-south and the Thessalon River flow is from west to east.

McLeod Road at this location has one lane in each direction with a rural cross-section and gravel surface. The lands surrounding the project limits are typically agricultural and forest with some residential properties. Storm water drainage in the area is to existing ditches. Select site photographs are presented in Appendix D.

**3 SITE INVESTIGATION**

**3.1 Field Investigation**

The field investigation plan was finalized after discussion with the MTO Foundations Section. Approximate locations of boreholes are shown on the Drawing No. 1 provided in Appendix A. The field investigation for this site was completed in two stages. The initial field investigation included advancing four boreholes drilled between February 8, 2016 and February 14, 2016 along the existing alignment.

Subsequent to the initial field investigation and after further discussions with the MTO Foundations Section a new alignment was proposed for the replacement bridge. The new alignment is to be located approximately 6.5 m east of the existing center line at the south abutment and 9.2 m at the north abutment. A supplementary investigation was carried out along the new alignment that included advancing three boreholes drilled between July 18 and 21, 2016. The locations and elevations of both the initial and supplementary boreholes are shown Drawing No. 1 and are summarized in Table 3-1.

**Table 3-1: Borehole Summary**

<b>Borehole</b>	<b>Location</b>	<b>Northing (m)</b>	<b>Easting (m)</b>	<b>Ground Surface Elevation (m)</b>	<b>Depth (m)</b>
301	Existing South approach	5144320.6	318552.0	198.7	6.7
302	Existing South abutment	5144323.9	318557.0	198.4	31.1
303	Existing North abutment	5144346.5	318590.8	198.2	31.1
304	Existing North approach	5144349.7	318595.7	198.2	6.7
305	Proposed North abutment	5144342.2	318593.9	197.8	46.3
306	Proposed North abutment	5144336.9	318598.6	197.1	10.5
307	Proposed South abutment	5144318.9	318556.3	197.1	46.3

As a component of our standard procedures and due diligence, Thurber contacted Ontario One Call to provide utility locate clearances for the intended borehole locations.

The boreholes were advanced with either a CME truck or track mounted drill rig equipped with hollow stem augers and NW casing. Marathon Drilling of Greely, Ontario supplied and operated the drilling equipment to carry out the drilling, sampling, and in-situ testing. The subsurface stratigraphy encountered in the boreholes was recorded in the field by Thurber personnel. Split spoon samples were collected at regular depth intervals in the boreholes via the completion of Standard Penetration Tests (SPT), following the methods described in ASTM Standard D1586-11. In-situ shear vane testing was carried out within the cohesive strata. Thin-walled tube samples of soft to firm cohesive deposits were collected at selected locations. All soil samples recovered from the boreholes were placed in moisture-proof containers and the samples were transported to Thurber's Ottawa geotechnical laboratory for further examination and testing.

PVC piezometers with an inside diameter of 25 mm were installed in both Boreholes 303 and 306 to allow for the measurement of the groundwater level at the site. Piezometer construction details are illustrated on the Record of Borehole sheets for Boreholes 303 and 306, provided in Appendix B. The piezometers were decommissioned on July 22, 2016.

The boreholes without piezometer installations were backfilled with a low-permeability combination of auger cuttings, sand and bentonite pellets in general accordance with the intent of Ontario MOE Regulation 903. Boreholes advanced within road areas were capped with granular material.

The as-drilled locations of the boreholes and ground surface elevations at the borehole locations were surveyed by Thurber on February 16, 2016 and July 21, 2016. The vertical datum used was a temporary benchmark (TBM) provided by MPCE, located south of the existing bridge located at Station 1+965.5, 7.0 m right of the road edge. The TBM was a steel spike in the west root of a 350 mm diameter ash tree. The TBM has a geodetic elevation of 200.183 m. The location of the TBM is indicated on Drawing No. 1 in Appendix A.

### **3.2 Laboratory Testing**

Geotechnical laboratory testing consisted of natural moisture content determination and visual identification of all soil samples in accordance with the current MTO standards. Grain size distribution analyses, Atterberg Limits testing and consolidation testing were also carried out on selected samples to MTO and ASTM standards. Also, samples of the native soils were submitted to Paracel Laboratories in Ottawa, Ontario for analysis of pH, water soluble sulphate and chloride concentrations, resistivity and conductivity.

The laboratory test results are presented on the Record of Borehole sheets in Appendix B and are illustrated on the figures in Appendix C.

## **4 DESCRIPTION OF SUBSURFACE CONDITIONS**

### **4.1 Overview / General**

Reference is made to the Record of Borehole sheets in Appendix B for details of the soil stratigraphy encountered in the boreholes. A stratigraphic profile for the site is presented on Drawing No. 1 provided in Appendix A for illustrative purposes. An overall description of the stratigraphy is given in the following paragraphs; however, the factual data presented in the Record of Boreholes governs any interpretation of the site conditions.

In general, the stratigraphy along the proposed alignment is characterized by fill overlying a clay layer, overlying a sandy silt to silty sand deposit. A silty sand or clay with frequent wood pieces layer with a thickness ranging from ranging from 1.6 m to 3.1 m was encountered above the clay strata in the boreholes advanced along the north side of the Thessalon River.

More detailed descriptions of the individual strata are presented below.

### **4.2 Fill**

#### **Sand with Gravel Fill**

A fill layer consisting predominantly of sand and gravel with varying amounts of silt was encountered at surface in all boreholes except Borehole 306.

The top of this layer ranges from Elevation 197.1 m to Elevation 198.7 m and has a thickness ranging from 0.3 m to 2.4 m. The SPT 'N' values in Boreholes 303 and 305 ranged from 11 to 18 blows per 0.3 m of penetration; indicating a compact condition.

The moisture content of the samples tested ranged from 5% to 18%. The results of a grain size analysis test completed on a sample of this material indicated a gravel content of 15%, sand content of 56%, and a fines content (combined silt and clay size particles) of 29%. Grain size analysis results are illustrated on Figure 1 in Appendix C.

#### **Clay Fill**

A clay fill layer with trace amounts of sand and organics was encountered at the surface in Borehole 306. The top of this layer was at Elevation 197.1 m and has a thickness of 2.3 m. The SPT 'N' values ranged from 4 to 8 blows per 0.3 m of penetration indicating a firm consistency. The moisture content of the samples tested ranged from 22% to 36%.

## **Boulders and Cobbles Fill**

A fill layer consisting predominantly of boulders and cobbles was encountered below the granular fill in Borehole 307. The top of this layer was at Elevation 196.8 m and the observed thickness was 2.1 m.

### **4.3 Clay with layers of Silty Sand and Silt with Frequent Wood Pieces**

On the north side of the river, the boreholes (Boreholes 303, 304, 305 and 306) indicate the presence of highly variable soil between the fill and deep clay deposit that included clay (CL), clay with wood pieces, silt (ML) and silty sand (SM) with wood pieces. These layers are discontinuous between borehole locations. The base of this variable deposit ranges from elevation 191.7 m to 192.1 m. The thickness of this variable deposit ranged from 3.0 m to 4.7 m.

The SPT 'N' values ranged from WH (Weight of Hammer) to 7; indicating a very loose to loose or very soft to soft condition.

The moisture content of the samples tested ranged from 20% to 69%. The results of grain size analysis conducted on samples of this material are summarized in Table 4-1 and are illustrated on Figures 9 and 10 in Appendix C.

**Table 4-1: Gradation Results**

<b>Soil Particles</b>	<b>%</b>
Gravel	0 to 4
Sand	28 to 81
Silt and Clay	18 to 72

The results of Atterberg Limits testing completed on samples of this material are summarized in Table 4-2 and are illustrated on Figure 11 in Appendix C.

**Table 4-2: Atterberg Limits Test Results**

Liquid Limit	30 to 46
Plastic Limit	12 to 25
Plasticity Index	18 to 24

Organic content testing was completed on eight samples of this material with results ranging from 3.9% to 14.1% organic content.

### **4.4 Clay Crust (CH)**

A brown native clay crust deposit was encountered beneath the granular fill materials in Boreholes 301 and 302.

The top of this layer ranges from elevation 196.9 m to 198.1 m and has a thickness ranging from 1.5 m to 2.1 m. The SPT 'N' values ranged from 3 to 10 blows per 0.3 m of penetration indicating a soft to stiff consistency.

The moisture content of the samples tested ranged from 30% to 68%. The results of grain size analysis conducted on samples of this material are summarized in Table 4-3 and are illustrated on Figure 2 in Appendix C.

**Table 4-3: Gradation Results for Clay Crust**

Soil Particles	%
Gravel	0
Sand	0
Silt	45 and 31
Clay	55 and 69

The results of Atterberg Limits testing completed on samples of this material are summarized in Table 4-4 and are illustrated on Figure 3 in Appendix C.

**Table 4-4: Atterberg Limits Test Results for Clay Crust**

Liquid Limit	51 and 64
Plastic Limit	21
Plasticity Index	30 and 43

#### 4.5 Clay (CL to CI) to Silty Clay (CL-ML)

A grey clay deposit was encountered beneath the silty sand or clay with frequent wood pieces in the boreholes advanced on the north side of the Thessalon River (Boreholes 303, 304, 305, and 306), below the boulders and cobbles fill in Borehole 307 and below the clay crust material in Boreholes 301 and 302.

The top of this layer ranges from elevation 194.8 to 196.6 m on the south side of the Thessalon River and from 191.7 to 192.1 m on the north side of the Thessalon River. The thickness of the clay unit ranged from 10.7 m to 22.9 m where completely penetrated. Boreholes 301, 304 and 306 were terminated in this strata. In general, the upper portion of the deposit is described as clay (CL to CI) while the lower portion (below approximate elevation 182 m) generally has increased silt content and occasional silt and sandy silt seams and is described as clay (CL to CI) to silty clay (CL-ML).

In-situ shear vane test results indicated undrained shear strengths ranging from 15 kPa to 81 kPa; indicating a soft to stiff consistency. The moisture content of the samples tested ranged from 23% to 61%. The results of grain size analysis conducted on samples of this material are summarized in Table 4-5 and are illustrated on Figures 4 to 6 in Appendix C.

**Table 4-5: Gradation Results for Clay to Silty Clay**

Soil Particles	%
Gravel	0
Sand	0 to 2
Silt	35 to 81
Clay	18 to 64

The results of Atterberg Limits testing completed on samples of this material are summarized in Table 4-6 and are illustrated on Figures 7 to 8 in Appendix C.

**Table 4-6: Atterberg Limits Test Results for Clay to Silty Clay**

Liquid Limit	23 to 46
Plastic Limit	17 to 27
Plasticity Index	6 to 23



The results of oedometer (one-dimensional consolidation) tests carried out on undisturbed clay samples are summarized in Table 4-7. Copies of the oedometer test results are provided in Appendix C. The results of the tests indicate that the clay is slightly over-consolidated.

**Table 4-7: Consolidation Test Results for Clay**

Parameter	Sample		
	302	305	305
Borehole	302	305	305
Sample	ST-2	ST-10	ST-14
Depth / Elevation (m) (mid-sample)	11.0 / 187.4	7.9 / 189.9	14.0 / 183.8
Moisture Content, (%)	46	51	45
Unit Weight, ( $\gamma$ ) (kN/m <sup>3</sup> )	17.1	17.0	17.3
Specific Gravity ( $G_s$ )	2.809	2.730	2.730
Initial Void Ratio ( $e_o$ )	1.342	1.371	1.252
Pre-consolidation Pressure, (kPa)	120	125	135
Compression Index ( $C_c$ )	0.581	0.869	0.576
Recompression Index ( $C_r$ )	0.051	0.057	0.040

#### 4.6 Sandy Silt to Silty Sand

A sandy silt to silty sand layer was encountered beneath the clay strata in all deep boreholes. Boreholes 302, 303, 305 and 307 were terminated in this strata. The top of this layer ranges from elevation 181.4 m to 171.9 m. The SPT 'N' values ranged from 4 to 47; indicating a loose to dense condition; but typically compact.

The moisture content of the samples tested ranged from 20% to 39%. The results of grain size analysis conducted on samples of this material are summarized in Table 4-8 and are illustrated on Figures 12 and 14 in Appendix C.

**Table 4-8: Gradation Results for Sandy Silt to Silty Sand**

Soil Particles	%
Gravel	0 to 8
Sand	1 to 83
Silt	53 to 84
Clay	5 to 15

The results of Atterberg Limits testing completed on samples of this material indicated a non-plastic silt.

#### 4.7 Groundwater Conditions

The groundwater level in the piezometer installed in Borehole 303 was recorded on February 19, 2016, at a depth of 5.5 m; corresponding to elevation 192.7 m.

The groundwater level in the piezometer installed in Borehole 306 was recorded on July 21, 2016, at a depth of 2.8 m; corresponding to elevation 194.6 m.

The water level in the Thessalon River was measured by Thurber on July 20, 2016, at elevation 194.4. The water level in the Thessalon River was indicated on the base plan at

elevation 194.0 m dated February 10, 2016. The high water mark was indicated at elevation 195.0 m.

These observations are considered short-term readings and seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall.

## 5 MISCELLANEOUS

Thurber staked and/or marked the borehole locations in the field and obtained utility clearances prior to drilling. Thurber surveyed the borehole locations and determined the ground surface elevations based on contract drawings provided by McIntosh Perry Consulting Engineers Ltd. Marathon Drilling of Greely, Ontario supplied and operated the drilling equipment to carry out the drilling, sampling, and in-situ testing. The drilling, and sampling operations in the field were supervised on a full time basis by Mr. Nick Weil and Mr. Justin Gray of Thurber. Laboratory testing was carried out by Thurber in its MTO-approved laboratory in Ottawa.

Overall project management and direction of the field program was provided by Paul Carnaffan, P.Eng. Interpretation of the field data and preparation of this report was completed by Kenton Power, P.Eng. The report was reviewed by Paul Carnaffan, P.Eng. and Dr. P.K. Chatterji, P.Eng., the Designated Principal Contact for MTO Foundations Projects.



Paul Carnaffan, M.Eng., P.Eng.  
Principal, Senior Geotechnical Engineer



P.K. Chatterji, Ph.D., P.Eng.  
Review Principal, Designated MTO Contact

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**PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS**

**6 GENERAL**

This report presents the interpretation of the factual data obtained from a foundation investigation conducted by Thurber for the replacement of the McLeod Road Bridge crossing of the Thessalon River along with a geotechnical assessment and geotechnical recommendations for the foundations and approach embankments. The geotechnical assessment and recommendations have been prepared based on the available data regarding the proposed foundations and existing ground conditions and in accordance with the Canadian Highway Bridge Design Code (CHBDC), version CSA S6-14.

This foundation investigation and design report with the interpretation and recommendations are intended for the use of the Ministry of Transportation, and shall not be used or relied upon for any other purposes or by any other parties including the construction or design-build contractor. The design-build contractor must make their own interpretation based on the factual data in Part 1 of the report. Where comments are made on construction, they are provided only in order to highlight those aspects which could affect the design of the project. Contractors must make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods and scheduling.

No previous foundation investigation information for the site was available. Base plan mapping and survey data was provided by MCPE for the preparation of this report.

The frost penetration depth at this site is 2.0 m as per OPSD 3090.100.

The following sections address the foundation aspects of the replacement of the existing bridge structure. The discussions and recommendations presented in this report are based on the information provided by MPCE and on the factual data obtained during the course of this investigation.

**6.1 Proposed Structure**

Based on information provided by MPCE, it is understood that the proposed replacement will be carried out on a new alignment located approximately 6.5 m east of the existing center line at the south abutment and 9.2 m east of the existing center line at the north abutment. It is also understood that the existing bridge is to remain in service until the new bridge is constructed. The recommended bridge replacement is a single lane, single span, prefabricated, modular steel through truss, with a steel deck and an approximate length of 42.6 m. A 0.8 m grade raise is proposed for the north abutment while a 1.4 m grade raise is proposed for the south abutment. It

is also understood that the single span option with a steel deck would result in the requirement to support abutment loads of approximately 1486 kN at ULS and 978 kN at SLS.

## 6.2 Geotechnical Assessment

In general, the stratigraphy along the proposed alignment is characterized by fill overlying a clay, overlying a sandy silt to silty sand. A silty sand or clay with frequent wood pieces layer with a thickness ranging from ranging from 1.5 m to 3.2 m was encountered at or near the top of the clay strata in the boreholes advanced on the north side of the Thessalon River.

The design of the bridge structure foundations will be governed by the presence of a firm compressible clay deposit throughout the site. Based on the results of the field and laboratory investigation and the information provided by MPCE with regards to the proposed project requirements, the geotechnical foundation design considerations include:

- The existing fill is not suitable for support of shallow foundations and should be removed from beneath footings and granular pad
- The firm clay offers a relatively low bearing resistance
- The proposed replacement structure is to be designed to tolerate up to 50 mm of total settlement

Further discussion regarding these design considerations, evaluation of design options and foundation recommendations are provided in the sections that follow.

## 7 STRUCTURE CLASSIFICATION

In accordance with CHBDC CSA S6-14, the analysis and design of structures takes into consideration the importance of the structure and the consequence associated with exceeding limit states. The importance category and consequence classification are defined by the Regulatory Authority, which in this case is the Ministry of Transportation, Ontario (MTO).

It is understood that MTO has designated this structure as follows:

**Table 7-1: Bridge Structure Classification**

Criteria	Classification	CHBDC Section
Importance Category	Other Bridges	4.4.2
Consequence Classification	Typical Consequence	6.5.1

Based on the above, a consequence factor ( $\Psi$ ) of 1.0, as per Table 6.1 of the CHBDC, has been used in assessing factored geotechnical resistances.

If the consequence classification changes, the geotechnical assessment and recommendations provided herein will need to be reviewed and revised.

## 8 SEISMIC CONSIDERATIONS

### 8.1 Seismic Hazard - Spectral and Peak Acceleration Values

The seismic hazard data for the CHBDC is based on the fifth generation seismic model developed by the Geological Survey of Canada (GSC). Seismic hazard data for this site has been obtained from the GSC's seismic hazard calculator. The data includes peak ground acceleration (PGA), peak ground velocity (PGV), and the 5% damped spectral response acceleration values ( $S_a(T)$ ) for the reference ground condition (Site Class C) for a range of periods (T) and for a range of return periods including the 475-year, 975-year and 2475-year events. The GSC seismic hazard calculation data sheet for this site is presented in Appendix F.

The site coefficients used to determine the design spectral acceleration and displacement values are a function of the Site Class and the peak ground acceleration (PGA).

### 8.2 Soil Strength Criteria

In accordance with the CHBDC, the selection of the seismic site classification is based on the soil conditions encountered in the upper 30 m of the stratigraphy.

In the case where the soil stratigraphy consists of both cohesive ( $S_u$  criteria) and non-cohesive (SPT 'N' criteria) strata and the resulting seismic site classification differ, the stratum with the lower site class would govern the site.

The following generalized soil stratigraphy was encountered at this site:

- |            |                  |                        |            |
|------------|------------------|------------------------|------------|
| • Layer 1: | Thickness = 8 m  | Typical $S_u$ = 35 kPa | clay       |
| • Layer 2: | Thickness = 11 m | Typical $S_u$ = 65 kPa | clay       |
| • Layer 3: | Thickness = 9 m  | Typical SPT 'N' = 22   | sandy silt |
| • Layer 4: | Thickness = 2 m  | Typical SPT 'N' = 15   | sandy silt |

The seismic site classification for this site is based on the  $S_u$  criteria. The harmonic mean of the typical  $S_u$  listed is 47 kPa which corresponds to a Seismic Site Class E in accordance with Table 4.1 of the CHBDC.

### 8.3 Seismic Liquefaction

Based on the combination of the grain size distribution, the relative density values of the sandy silt, the highly plastic nature of the native clay, and the relatively low PGA of 0.037g determined for the site, overburden soils at this site are classified as "not susceptible" to liquefaction during the design earthquake event.

## 9 STRUCTURE FOUNDATIONS

Given the soil stratigraphy encountered and the requirements of modular bridge design, the following options could be considered for the new bridge foundations:

- Spread footings placed on native soil
- Spread footing placed on engineered fill pads with preloading at the abutment and approach embankment areas
- Driven H-piles

As indicated above the single span option with a steel deck would result in the requirement to support abutment loads of approximately 1486 kN at ULS and 978 kN at SLS.

Recommendations for design of the feasible foundation alternatives are presented in the following sections together with the corresponding geotechnical design parameters. A preferred foundation alternative from a geotechnical perspective is recommended.

## 9.1 Spread Footings on Native Soils

Given the low strength and highly compressible nature of the native clay encountered immediately below the embankment fill, the presence of a layer containing wood pieces, the significant depth to a competent foundation stratum and the relatively high river water level, spread footings placed directly on native soils to support the abutments are not recommended for this site.

## 9.2 Spread Footing Placed on Engineered Fill Pads

### 9.2.1 Bearing Capacity

Supporting the bridge abutments on concrete foundation placed on granular fill pads can be considered at this site. The preliminary General Arrangement (GA) drawing dated January 2017, indicates the proposed new alignment at elevation 198.7 m at both the north and south abutments. Also indicated on the GA drawing is the approximate top of bearing of 198.048 m at the south abutment and 197.408 m at the north abutment.

The footings should be constructed on an engineered fill pad consisting of OPSS Granular A at least 1.5 m in thickness. A non-woven geotextile should be placed over the subgrade prior to placing the backfill material to prevent the migration of fines into the Granular A engineered pad. The engineered fill should be placed and compacted in accordance with OPSS.PROV 501. The top of the engineered fill pad should be at least 1 m wider than the footprint of the spread footing and should extend downward and outward at 1H:1V, or flatter.

The following values of factored Geotechnical Resistance at ULS and Geotechnical Reaction at SLS may be used for design of footings approximately 8.0 m in length and 2.5 m in width placed at Elevation 196.5 m on a properly compacted granular engineered fill pad as noted above, that has a base Elevation 195.0 m:

- Factored Geotechnical Resistance at ULS (kPa) 150 kPa
- Factored Geotechnical Reaction at SLS (kPa) 75 kPa

Should the above noted values not meet the required loading condition the footing width may need to be increased.

It should be noted that the SLS value provided above corresponds to 70 mm of total settlement at the north abutment and 40 mm of total settlement at the south abutment for the combined footing and grade raise loading. Settlement mitigation measures for the north and south abutments are discussed in Section 9.2.2. Furthermore, future bridge maintenance and jacking up of the abutments to compensate for the time dependant consolidation settlements should be anticipated.

The factored geotechnical resistance and reaction values include the following factors:



- Consequence factor ( $\Psi$ ) of 1.0
- Geotechnical resistance factors (CHBDC Table 6.2):
  - $\phi_{gu} = 0.5$  (static analysis; typical degree of understanding)
  - $\phi_{gs} = 0.8$  (static analysis; typical degree of understanding)

The geotechnical resistances are for vertical concentric loading and will need to be adjusted for the effects of inclined or eccentric loading, if applicable. The geotechnical resistance should be calculated as illustrated in the CHBDC 2014 Clause 6.10.3 and Clause 6.10.4.

Resistance to lateral forces and sliding resistance between cast-in-place concrete and the compacted granular engineered fill pad should be evaluated using an unfactored coefficient of 0.55.

### 9.2.2 Settlement Mitigation at the Abutments

Based on the information provided by MPCE, up to a 0.8 m grade raise is proposed for the north abutment while up to a 1.4 m grade is proposed for the south abutment. It is also understood that the single span replacement option, with a steel deck would result in abutment loads of approximately 1486 kN at ULS and 978 kN at SLS.

Embankment settlement mitigation measures will be required to accommodate the predicted settlement of the founding soils at both abutments, as noted above, to address post-construction embankment settlement that could impact the new McLeod Road Bridge. Provided that there is sufficient time in the construction schedule, the simplest and most economical mitigation measure would be preloading the abutment and approach embankment areas. It is anticipated that 90% of the settlement due to the grade raise to construct the embankment will be complete within approximately six months following construction of the embankment to the final design grades. The foundations must not be constructed until the settlement under the preloading is complete.

The construction of the footing and the applied bridge loading will result in further settlement in the area of the north abutment. The settlement due to the footing loading would be in the order of a further 40 mm of settlement. The majority of the settlement due to the footing load is anticipated to be completed within approximately twelve months following the bridge construction.

Geotechnical instrumentation monitoring will be required at this site to confirm the waiting period after fill placement and the magnitude and time-rate of settlement. An NSSP is provide in Appendix G. The actual preloading time will be governed by results from the instrumentation monitoring program and there is a risk that it may be longer than provided herein.

It is not considered cost effective or practical to excavate and remove the silty sand and clay containing frequent wood pieces which extends to elevation 191.7 m, about 3.3 m below the anticipated base of the proposed excavation. Excavation to fully remove this deposit would extend to a depth of up to 6.3 m below existing grade; this would require additional quantities for removal and new granular fill as well as having impacts on construction efforts for dewatering and protection systems for the existing road and structure.

### 9.2.3 Embankment Construction

Embankment construction at this site is fairly limited as most of the new alignment is either in a shallow cut or near existing grades. Regrading and fill placement around the abutments will be required to tie into the existing grades but will generally be no more than 1.4 m in height.

Embankment construction should be carried out in accordance with OPSS.PROV 206. Embankment fill shall consist of OPSS Granular A in compliance with OPSS.PROV 1010.

Granular fill should be placed and compacted in accordance with OPSS.PROV 501. Where new embankment fill is placed against existing embankment slopes or on a sloping ground surface steeper than 3H:1V, the existing earth or fill slope must be benched in accordance with OPSD 208.010.

The embankment slopes should be graded at 2H:1V or flatter. Embankment slopes up to 1.4 m in height, constructed with Granular A and graded at 2H:1V or flatter are considered stable.

Granular fill embankment slopes must be provided with erosion protection in accordance with OPSS 804.

### 9.3 Driven H-Piles

If a piled foundation is considered, driven steel H-piles, designed as friction piles can be used at the abutments. In order to develop an adequate resistance, the steel H-piles will have to be driven to significant depths. The geotechnical resistances recommended for HP 310x110 piles are presented in Table 9-1. A preliminary GA drawing dated May 2016 indicates that the top of pile would be at approximate elevation 197.0 m at both abutments.

**Table 9-1: Geotechnical Resistance and Reaction for Driven HP310x110 Friction Piles**

Foundation Element	Pile Tip Depth / Elevation (m)	Factored Geotechnical Resistance at ULS (kN) per pile	Factored Geotechnical Resistance at SLS (kN) per pile
North and South Abutment	20 / 177	350	275
	25 / 172	450	350
	30 / 167	700	525

The actual pile tip elevations may vary during installation and pile length should be controlled in accordance with OPSS 903. Pile tip protection is not required for friction piles.

The factored geotechnical resistance and reaction values include the following factors:

- Consequence factor ( $\Psi$ ) of 1.0
- Geotechnical resistance factors (CHBDC Table 6.2):
  - $\phi_{gu} = 0.4$  (static analysis; typical degree of understanding)
  - $\phi_{gs} = 0.8$  (static analysis; typical degree of understanding)

A soil-structure interaction analysis to assess the response of a pile under lateral loading was carried out using Ensoft Inc.'s LPILE software. A copy of the results in the form of load-deflection curves (p-y curves) and lateral load vs maximum bending moment are illustrated on Figures 5 and 6 provided in Appendix F.

The resistance to lateral deflection should include the following factors:

- Consequence factor ( $\Psi$ ) of 1.0
- Geotechnical resistance factors (CHBDC Table 6.2):
  - $\phi_{gs} = 0.8$ ; (typical degree of understanding)



Pile spacing and group effects will need to be considered in assessing the overall lateral resistance of the piles at each foundation unit. The group efficiency factors should be in accordance with Figures C6.11.3(r), C6.11.3(s), and C6.11.3(t) in Section C6.11.3.4 of the Commentary to the CHBDC.

#### **9.4 Recommended Foundation**

A comparison of the foundation alternatives is presented in Appendix E. Taking into account the soil stratigraphy and the requirements of modular bridge design, the recommended foundation is a concrete slab foundation on a 1.5 m (minimum) thick engineered fill pad. The engineered pad and bridge foundations should be constructed following preloading of the abutment and approach embankment areas to reduce the post construction settlement.

### **10 GENERAL RECOMMENDATIONS**

Excavation and backfilling for the replacement of the McLeod Road Bridge should be carried out in accordance with OPSS 902.

The concrete slab foundation can be placed on the 1.5 m thick engineered granular fill pad with no further requirements for frost protection measures.

#### **10.1 Excavations**

It is anticipated that temporary excavations as deep as 3 m at the north abutment and 4.4 m at the south abutment will be required for placement of the granular pad beneath the abutment foundations.

All excavations must be conducted in accordance with the requirements of the Occupational Health & Safety Act & Regulations (OHSA) for Construction Projects. The fills and native clay above the water table at the site should be classified as Type 3 in accordance with OHSA. It is anticipated that temporary protection systems will be required between the excavations and the adjacent existing road alignment. Temporary protection systems shall be in accordance with OPSS.PROV 539, performance level 2.

Selection of the equipment and methodology to excavate and prepare the founding surface is the responsibility of the Contractor.

#### **10.2 Dewatering**

The water level in the Thessalon River was measured by Thurber on July 20, 2016, at elevation 194.4. The anticipated base of excavation will be at approximately elevation 195.0 m; above the observed water level in the river. Subgrade preparation and placement of granular pads and abutment footings must be carried out in the dry.

The Contractor must be prepared to control the groundwater and surface water flow at the site to permit construction in a dry and stable excavation. It is recommended that the replacement of the bridge be conducted during a drier season such as after the spring freshet or prior to the fall season.

Water from either surface flow and/or groundwater must be diverted away from the excavation at all times. Groundwater perched within the embankment fill, from within cohesionless soils below

the water table and, surface runoff will tend to seep into, and accumulate in proposed excavations. Since the excavations are in the clay (base elevation of approximately 195.0 m), dewatering with sumps and pumps should be sufficient.

Dewatering and surface water diversion must remain operational and effective until the temporary excavation is backfilled. Design of an effective dewatering system is the responsibility of the Contractor. The Contract Documents must alert the Contractor to this responsibility and to design the system in accordance with SP FOUN003 with amends OPSS 902. A preconstruction survey is not required, thus Design Fill-In \*\* in SP FOUN003 should be "N/A".

In accordance with SP FOUN0003, the dewatering system is to be designed in accordance with OPSS.PROV 517 and SP517F01. If the excavation is not extended below the river level, it is not necessary for the design Engineer and design-checking Engineer to have a minimum of 5 years of experiences in design systems of similar nature and scope to the required work, thus Designer Fill-In \*\*\*\*\* in SP 517F01 should be "No". A preconstruction survey is not required, thus Designer Fill-In \*\*\*\*\* in this SP should be "N/A".

The groundwater level will fluctuate. The minimum groundwater elevation for the site at the time of the proposed works should be taken as the level provided by the design storm defined in SP 517F01 and SP FOUN0003.

The need for a Permit to take Water (PTTW) should be carried out by specialists experienced in this field.

### **10.3 Subgrade Preparation**

Subgrade preparation for the new abutment foundations should include the removal of the existing fill. The native subgrade within the footprint of the abutment foundations is expected to consist of clay. Excavation at the subgrade elevation should be performed with a smooth bucket. The use of a toothed bucket would disturb the subgrade.

The base the excavations should be inspected prior to placing the geotextile and engineered fill pad in order to confirm that the founding conditions are consistent with the recommendations described herein, and to ensure that there is no disturbance of the underlying soil.

The compaction of the engineered fill pad directly above the subgrade may result in disturbance of the material, with pumping of fines into the granular pad and difficulty achieving the specified degree of compaction. Protection of the subgrade should include installation of Class II non-woven geotextile with a maximum FOS of 150 µm (OPSS 1860) installed beneath the engineered pad. The geotextile should be placed as soon as possible after reaching the subgrade level and following receipt of written notice to proceed in accordance with SP 109S12.

### **10.4 Structure Backfill**

Backfill of the structure foundations should be placed in accordance with OPSS 902. All backfill material should consist of Granular A, meeting the specifications of OPSS.PROV 1010.

Compaction equipment to be used adjacent to the walls should be restricted in accordance with OPSS.PROV 501.

## 11 EARTH RETAINING STRUCTURES

### 11.1 Static Lateral Earth Pressure Coefficients

Lateral earth pressures acting on structures for a fully drained backfill condition should be computed in accordance with the CHBDC but generally are given by the expression:

$$\sigma_h = K^*(\gamma d + q)$$

where:

$\sigma_h$  = lateral earth pressure on the wall at depth d (kPa)

K = earth pressure coefficient

$\gamma$  = unit weight of retained soil (kN/m<sup>3</sup>),

use submerged unit weights below the groundwater depth

d = depth below top of fill where pressure is computed (m)

q = value of any surcharge (kPa)

The recommended lateral earth pressure parameters for use in the design for a horizontal back-slope are provided in Table 11-1.

**Table 11-1: Static Lateral Earth Pressure Coefficients for Horizontal Ground Surface**

Parameter	OPSS Granular A & OPSS Granular B Type II	Native Clay
Soil Unit Weight, kN/m <sup>3</sup> , $\gamma$	21	17
Angle of Internal Friction, $\phi$	35°	27°
Coefficient of at Rest Earth Pressure, $K_o$ (Restrained Wall)	0.43	0.55
Coefficient of Active Earth Pressure, $K_a$ (Unrestrained Wall)	0.27	0.38

For rigid structures, it is recommended that at-rest horizontal lateral earth pressures be used for design. Active pressures should be used for the design of unrestrained walls. The parameters in the table correspond to full mobilization of earth pressures and require a certain relative movement between the wall and adjacent soil to produce these conditions. The ratio of wall movement to wall height required to mobilize the active condition would be approximately 0.002.

For static analysis, passive earth resistance in front of the abutments should be ignored, and therefore has not been provided. A lateral pressure due to backfill compaction should be added to the calculated lateral earth pressure in accordance with Section 6.12.3 of the CHBDC.

### 11.2 Combined Static and Seismic Lateral Earth Pressure Parameters

The following recommendations are per Section C4.6.5 of the Commentary of the CHBDC which states that seismically induced lateral soil pressures may be calculated using the Mononobe-Okabe Method with:

- $k_h = \frac{1}{2} F(PGA) \cdot PGA$  for structures that allow 25 mm to 50 mm of movement, and
- $k_h = F(PGA) \cdot PGA$  for non-yielding walls

The ratio of wall movement to wall height required to mobilize the active condition would be approximately 0.002 for a yielding structure with respect to the assessment of seismically induced lateral earth pressures.

The recommended seismic lateral earth pressure parameters for use in the design that are provided in Table 11-2 assume the following:

- Horizontal back-slope behind the wall
- Seismic Site Class of E, and a PGA with a 2% probability of exceedance in 50 years of 0.037g; as outlined in Section 8.2

**Table 11-2: Lateral Earth Pressure (Under Seismic Loads)**

Parameter	OPSS Granular A & OPSS Granular B Type II	Clay
Soil Unit Weight, kN/m <sup>3</sup> , $\gamma$	21	17
Angle of Internal Friction, $\phi$	35°	27°
<b>Yielding Wall</b>		
Dynamic Active Earth Pressure Coefficient, $K_{AE}$	0.29	0.40
<b>Non-Yielding Wall</b>		
Dynamic Active Earth Pressure Coefficient, $K_{AE}$	0.31	0.42

The total pressure due to combined static and seismic loads acting at a specific depth below the top of the wall may be determined using the following equation that includes consideration of material properties and the soil profile:

$$\sigma_h = K\gamma d + (K_{AE} - K_a) \gamma (H - d)$$

where:

- $\sigma_h$  = lateral earth pressure at depth, d (kPa)
- d = depth below the top of the wall (m)
- K = static active earth pressure coefficient  
( $K_a$  for yielding walls,  $K_o$  for non-yielding walls)
- $\gamma$  = unit weight of the backfill soil (kN/m<sup>3</sup>)  
use submerged unit weights below the groundwater depth
- $K_{AE}$  = combined static and seismic earth pressure coefficient
- H = total height of the wall (m)

### 11.3 Abutment Wall Backfill Drainage

The parameters provided in Table 11-1 and 11-2 are based on the assumption that the backfill is fully drained so that there are no unbalanced hydrostatic pressures. If adequate drainage cannot be confirmed, the potential for buildup of hydrostatic pressures should be considered in the design.

## 12 CEMENT TYPE AND CORROSION POTENTIAL

Three samples of the native soils were submitted to Paracel Laboratories in Ottawa, Ontario for analysis of pH, water soluble sulphate and chloride concentrations, resistivity and conductivity. The analysis was completed to determine the potential for degradation of the concrete in the presence of soluble sulphates and the potential for corrosion of exposed steel used in foundations

and buried infrastructure. The analysis results are summarized in the Tables 12-1. A copy of the test results have also been provided in Appendix C.

**Table 12-1: Results of Chemical Analysis**

Borehole	Sample	Depth (m)	Sulphate (µg/g)	pH	Resistivity (Ohm-cm)	Chloride (µg/g)
302	SS3	1.8	31	7.7	3950	69
305	SS5	2.6	37	7.7	5000	16
307	SS4	2.8	14	5.1	28100	9

The concentration of soluble sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with soil and groundwater at the site. Soluble sulphate concentrations less than 1000 µg/g generally indicate that a low degree of sulphate attack is expected for concrete in contact with soil and groundwater. Type GU Portland Cement should therefore be suitable for use in concrete at this site.

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. The test results provided in the Table 12-1 may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects.

### 13 CONSTRUCTION CONSIDERATIONS

#### 13.1 Erosion Protection

Slope protection and drainage measures will be required to ensure the long-term surficial stability of the embankment slopes. Normal slope vegetation should be established as soon as possible after completion of the embankment fills in order to control surficial erosion in general accordance with OPSS 804. The contractor should provide silt fences and erosion control blankets, as required, throughout the duration of the construction to prevent silt/sediments from running off the site as per OPSS 805.

#### 13.2 Construction Concerns

Potential construction concerns include, but are not necessarily limited to, the following:

- Confirmation that the granular backfill is adequately placed and compacted to specifications.
- Sufficient time is available to allow for settlement mitigation to occur
- A settlement monitoring program is properly carried out to monitor the preload settlement prior to the construction of the north abutment.
- The Contractor's selection of construction equipment and methodology should include assessment of the capability of the subgrade soils to support the proposed construction equipment and any temporary structures or fill (i.e. as a pad for crane support). Site conditions may limit the type of equipment suitable for use. The design and safety of any temporary works is the responsibility of the Contractor. Recommended wording for an NSSP (Geotechnical Assessment) addressing this issue is provided in Appendix G
- Seasonal fluctuations of the groundwater and river level are to be expected. In particular, the water level may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall, which may impact the construction.

The successful performance of the embankments will depend largely upon good workmanship and quality control during construction. Observation of the excavation and backfilling operations by a qualified geotechnical personal in accordance with SP 109S12 will be required during construction to confirm that the foundation recommendations are correctly implemented and material specifications are met.

#### 14 CLOSURE

Overall project management and direction of the field program was provided by Paul Carnaffan, P.Eng. Interpretation of the field data and preparation of this report was completed by Kenton Power, P.Eng. The report was reviewed by Paul Carnaffan, P.Eng. and Dr. P.K. Chatterji, P.Eng., the Designated Principal Contact for MTO Foundations Projects.



Paul Carnaffan, P.Eng.  
Principal, Senior Geotechnical Engineer

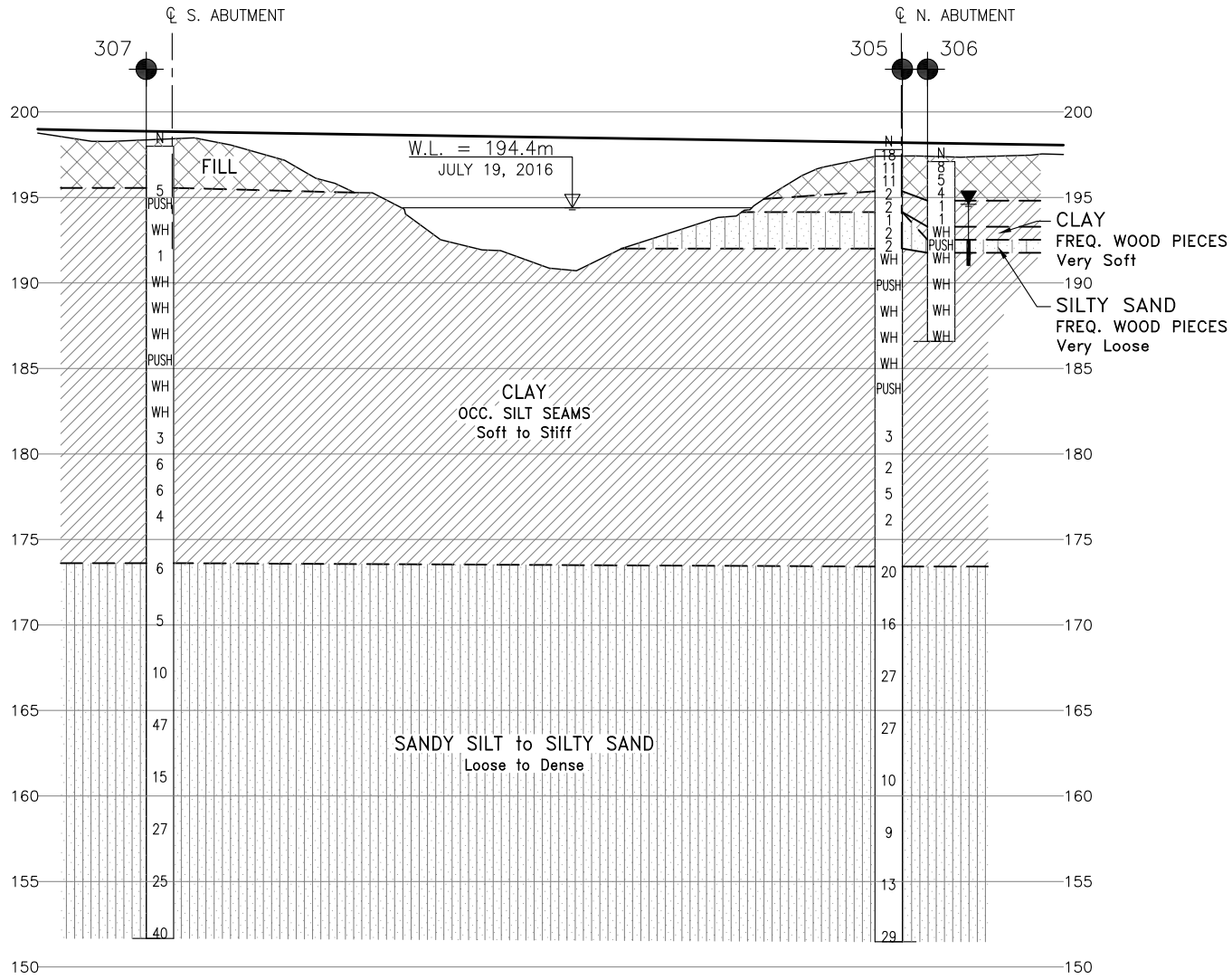
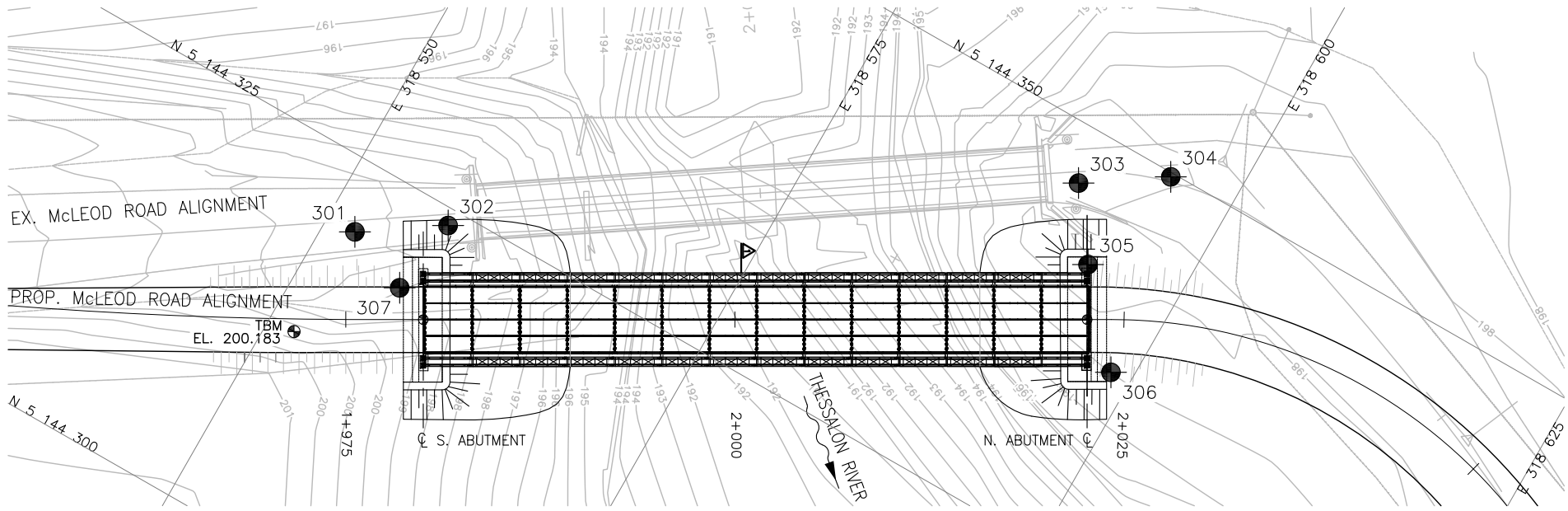


P.K. Chatterji, P.Eng.  
Review Principal, Designated MTO Contact

**APPENDIX A**

**BOREHOLE LOCATIONS AND SOIL STRATA DRAWINGS**





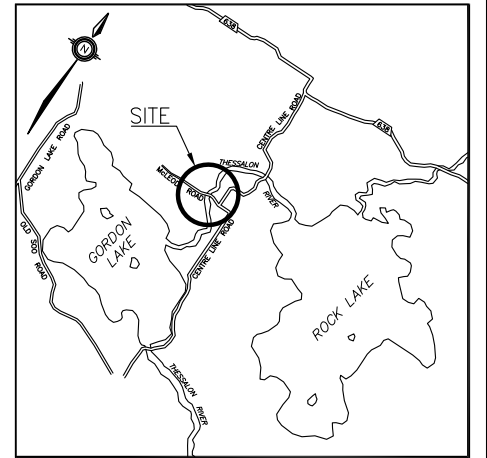
METRIC  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN



CONT No  
GWP No 5148-13-00

McLEOD ROAD  
BRIDGE REPLACEMENT  
BOREHOLE LOCATIONS AND SOIL STRATA

McINTOSH PERRY



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	EASTING
301	198.7	5 144 320.6	318 552.0
302	198.4	5 144 323.9	318 557.0
303	198.2	5 144 346.5	318 590.8
304	198.2	5 144 349.7	318 595.7
305	197.8	5 144 342.2	318 593.9
306	197.1	5 144 336.9	318 598.6
307	198.0	5 144 318.9	318 556.3

-NOTES-

- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- This drawing is for subsurface information only. Surface details and features are for conceptual illustration.
- Borehole locations are shown in MTM Zone 13 coordinates.

GEOCRES No. 41J-103

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	KP	CHK -	CODE
DRAWN	MFA	CHK KP	SITE 38S-154 STRUCT
			LOAD
			DATE
			APR 2019
			DWG 1



**APPENDIX B**  
**RECORD OF BOREHOLE SHEETS**



## SYMBOLS, ABBREVIATIONS AND TERMS USED ON TEST HOLE RECORDS

### TERMINOLOGY DESCRIBING COMMON SOIL GENESIS

Topsoil	mixture of soil and humus capable of supporting vegetative growth
Peat	mixture of fragments of decayed organic matter
Till	unstratified glacial deposit which may include particles ranging in sizes from clay to boulder
Fill	material below the surface identified as placed by humans (excluding buried services)

### TERMINOLOGY DESCRIBING SOIL STRUCTURE:

Desiccated	having visible signs of weathering by oxidization of clay materials, shrinkage cracks, etc.
Fissured	having cracks, and hence a blocky structure
Varved	composed of alternating layers of silt and clay
Stratified	composed of alternating successions of different soil types, e.g. silt and sand
Layer	> 75 mm in thickness
Seam	2 mm to 75 mm in thickness
Parting	< 2 mm in thickness

### RECOVERY:

For soil samples, the recovery is recorded as the length of the soil sample recovered.

### N-VALUE:

Numbers in this column are the field results of the Standard Penetration Test: the number of blows of a 63.5 kg hammer falling 0.76 m, required to drive a 50 mm O.D. split spoon sampler 0.3 m into undisturbed soil. For samples where insufficient penetration was achieved and N-value cannot be presented, the number of blows are reported over the sampler penetration in millimetres (e.g. 50/75).

### DYNAMIC CONE PENETRATION TEST (DCPT):

Dynamic cone penetration tests are performed using a standard 60 degree apex cone connected to an "A" size drill rods with the same standard fall height and weight as the Standard Penetration Test. The DCPT value is the number of blows of the hammer required to drive the cone 0.3 m into the soil. The DCPT is used as a probe to assess soil variability.



### STRATA PLOT:

Strata plots symbolize the soil and bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.



Boulders  
Cobbles  
Gravel      Sand      Silt      Clay      Organics      Asphalt      Concrete      Fill      Bedrock

### TEXTURING CLASSIFICATION OF SOILS

Classification	Particle Size
Boulders	Greater than 200 mm
Cobbles	75 – 200 mm
Gravel	4.75 – 75 mm
Sand	0.075 – 4.75 mm
Silt	0.002 – 0.075 mm
Clay	Less than 0.002 mm

### TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

Descriptive Term	Undrained Shear Strength (kPa)
Very Soft	12 or less
Soft	12 – 25
Firm	25 – 50
Stiff	50 – 100
Very Stiff	100 – 200
Hard	Greater than 200

NOTE: Clay sensitivity is defined as the ratio of the undisturbed strength over the remolded strength.

### SAMPLE TYPES

SS	Split spoon samples
ST	Shelby tube or thin wall tube
DP	Direct push sample
PS	Piston sample
BS	Bulk sample
WS	Wash sample
HQ, NQ, BQ etc.	Rock core sample obtained with the use of standard size diamond coring equipment

### TERMS DESCRIBING CONSISTENCY (COHESIONLESS SOILS ONLY)

Descriptive Term	SPT "N" Value
Very Loose	Less than 4
Loose	4 – 10
Compact	10 – 30
Dense	30 – 50
Very Dense	Greater than 50

### MODIFIED UNIFIED SOIL CLASSIFICATION

Major Divisions		Group Symbol	Typical Description
COARSE GRAINED SOIL	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS	SW	Well-graded sands or gravelly sands, little or no fines.
		SP	Poorly-graded sands or gravelly sands, little or no fines.
		SM	Silty sands, sand-silt mixtures.
		SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILT AND CLAY SOILS $W_L < 35\%$	ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
		OL	Organic silts and organic silty-clays of low plasticity.
	SILT AND CLAY SOILS $35\% < W_L < 50\%$	MI	Inorganic compressible fine sandy silt with clay of medium plasticity, clayey silts.
		CI	Inorganic clays of medium plasticity, silty clays.
		OI	Organic silty clays of medium plasticity.
	SILT AND CLAY SOILS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy of silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of high plasticity, organic silts.
HIGHLY ORGANIC SOILS		Pt	Peat and other organic soils.

Note -  $W_L$  = Liquid Limit



## EXPLANATION OF ROCK LOGGING TERMS

### ROCK WEATHERING CLASSIFICATION

Fresh (FR)	No visible signs of weathering.
Fresh Jointed (FJ)	Weathering limited to surface of major discontinuities.
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock materials.
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structures are preserved.

### TERMS

Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.
Solid Core Recovery: (SCR)	Percent ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1 m in length or larger, as a percentage of total core length
Unconfined Compressive Strength: (UCS)	Axial stress required to break the specimen.
Fracture Index: (FI)	Frequency of natural fractures per 0.3 m of core run.

### DISCONTINUITY SPACING

Bedding	Bedding Plane Spacing
Very thickly bedded	Greater than 2 m
Thickly bedded	0.6 to 2 m
Medium bedded	0.2 to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 to 60 mm
Laminated	6 to 20 mm
Thinly laminated	Less than 6 mm

### STRENGTH CLASSIFICATION

Rock Strength	Approximate Uniaxial Compressive Strength (MPa)
Extremely Strong	Greater than 250
Very Strong	100 – 250
Strong	50 – 100
Medium Strong	25 – 50
Weak	5 – 25
Very Weak	1 – 5
Extremely Weak	0.25 – 1

# RECORD OF BOREHOLE No 301

1 OF 1

METRIC

GWP# 5148-13-00 LOCATION Lat: 46.438427°, Long: -83.821034° 38S-154 McLeod Rd Bridge, MTM Z13: N 5 144 320.6 E 318 552.0 ORIGINATED BY NW  
HWY McLeod Road BOREHOLE TYPE HSA COMPILED BY KCP  
DATUM Geodetic DATE 2016.02.11 - 2016.02.11 CHECKED BY PC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT  W <sub>p</sub>	NATURAL MOISTURE CONTENT  W	LIQUID LIMIT  W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE								
								20 40 60 80 100								
198.7																
0.0	Silty Sand with Gravel - occasional cobbles - frost to 1.4 m FILL		1	AS	-											15 56 29 (SI+CL)
198.1																
0.6	CLAY (CH) - clay crust Stiff Brown		2	SS	10		198									0 0 45 55
			3	SS	8		197									
196.6																
2.1	CLAY (CI) Firm Grey		4	SS	1		196	5.6 +								
								2.8 +								
			5	SS	WH		195	9.3 +								
								4.7 +								
			6	SS	WH		194									0 0 48 52
								12.0 +								
							193	4.7 +								
192.0																
6.7	Borehole terminated at 6.7 m						192									

DOUBLE LINE 10870 MCLEOD RD.GPJ 2012TEMPLATE(MTO).GDT 11/4/19

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 302

1 OF 4

METRIC

GWP# 5148-13-00 LOCATION Lat: 46.438456°, Long: -83.820969° 38S-154 McLeod Rd Bridge, MTM Z13: N 5 144 323.9 E 318 557.0 ORIGINATED BY NW  
HWY McLeod Road BOREHOLE TYPE HSA, and NW casing COMPILED BY KCP  
DATUM Geodetic DATE 2016.02.10 - 2016.02.14 CHECKED BY PC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT  $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
								WATER CONTENT (%)					
198.4													
0.0	Silty Sand with Gravel - occasional cobbles - frost to 1.4 m FILL		1	AS	-								
			2	AS	-								
196.9													
1.5	CLAY (CH) - Clay crust Soft to firm Brown		3	SS	5								
			4	SS	3								
			1	ST	Push								
194.8													
3.6	CLAY (CI) Soft to firm Grey												
			5	SS	WH								
			6	SS	WH								
			7	SS	WH								
			8	SS	WH								

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity  
20  
15  
10  
(%) STRAIN AT FAILURE

DOUBLE LINE 10870 MCLEOD RD.GPJ 2012TEMPLATE(MTO).GDT 11/4/19

## METRIC

[illegible]

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity



## METRIC

[illegible]

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity

DOUBLE LINE 10870 MCLEOD RD.GPJ 2012TEMPLATE(MTO).GDT 11/4/19

# RECORD OF BOREHOLE No 302

4 OF 4

METRIC

GWP# 5148-13-00 LOCATION Lat: 46.438456°, Long: -83.820969° 38S-154 McLeod Rd Bridge, MTM Z13: N 5 144 323.9 E 318 557.0 ORIGINATED BY NW  
 HWY McLeod Road BOREHOLE TYPE HSA, and NW casing COMPILED BY KCP  
 DATUM Geodetic DATE 2016.02.10 - 2016.02.14 CHECKED BY PC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									
	Continued From Previous Page																
167.3	SANDY SILT (ML) Loose Grey		18	SS	6		168									0 43 57 (SI+CL)	
31.1	Borehole terminated at 31.1 m																

DOUBLE LINE 10870 MCLEOD RD.GPJ 2012TEMPLATE(MTO).GDT 11/4/19

# RECORD OF BOREHOLE No 303

1 OF 4

METRIC

GWP# 5148-13-00 LOCATION Lat: 46.438659°, Long: -83.820529° 38S-154 McLeod Rd Bridge, MTM Z13: N 5 144 346.5 E 318 590.8 ORIGINATED BY NW  
HWY McLeod Road BOREHOLE TYPE HSA, and NW casing COMPILED BY KCP  
DATUM Geodetic DATE 2016.02.08 - 2016.02.09 CHECKED BY PC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT  $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
198.2								20 40 60 80 100						
0.0	Sandy gravel trace silt Brown <b>FILL</b> - frost from 0 m to 1.4 m		1	GS	-		198	○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
			2	GS	-		197	20 40 60 80 100						
196.7								PLASTIC LIMIT W P						
1.5	Sand with silt some clay Compact Brown <b>FILL</b>		3	SS	11		196	NATURAL MOISTURE CONTENT W						
195.9								LIQUID LIMIT W L						
2.3	<b>CLAY (CL)</b> Soft to firm Brown to grey		4	SS	2		195	WATER CONTENT (%)						
			5	SS	2		195							
194.4														
3.8	<b>SILTY SAND (SM)</b> with organics - frequent wood pieces Very loose Brown		6	SS	3		194						2 63 35 (SH+CL) 14.1% organic content	
			7	SS	2		193						4 70 26 (SH+CL) 13.6% organic content	
			8	SS	2		193						1 81 18 (SH+CL)	
192.1														
6.1	<b>CLAY (Cl)</b> Soft to stiff Grey		9	SS	1		192							
							191							
			10	SS	WH		190							
							190	3.0 +						
								6.0 +						
			11	SS	WH		189							

Continued Next Page

+ 3, x 3: Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 303

2 OF 4

METRIC

GWP# 5148-13-00 LOCATION Lat: 46.438659°, Long: -83.820529° 38S-154 McLeod Rd Bridge, MTM Z13: N 5 144 346.5 E 318 590.8 ORIGINATED BY NW  
 HWY McLeod Road BOREHOLE TYPE HSA, and NW casing COMPILED BY KCP  
 DATUM Geodetic DATE 2016.02.08 - 2016.02.09 CHECKED BY PC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT		NATURAL MOISTURE CONTENT		LIQUID LIMIT		UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W P		W		W L			GR SA SI CL					
								20 40 60 80 100														
	Continued From Previous Page							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE														
	CLAY (CI) Soft to stiff Grey    - varved from 13.5 m to 15.9 m    - occasional silt seams						188	6.0 +														
		12	SS	WH			187	8.8 + 11.7 +														
							186															
		13	SS	WH																		
							185	6.2 + 6.2 +														
		14	SS	WH			184															
								9.0 + 7.2 +														
		15	SS	WH			183															
181.4	SANDY SILT (ML) Loose to compact Grey						182											0	1	84	15	
16.8		16	SS	WH			181															
								3.5 + 3.9 +														
							180															
							179															

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity  
 20  
 15  
 10  
 (%) STRAIN AT FAILURE

DOUBLE LINE 10870 MCLEOD RD.GPJ 2012TEMPLATE(MTO).GDT 11/4/19

## METRIC

[illegible]

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity

DOUBLE LINE 10870 MCLEOD RD.GPJ 2012TEMPLATE(MTO).GDT 11/4/19

# RECORD OF BOREHOLE No 303

4 OF 4

METRIC

GWP# 5148-13-00 LOCATION Lat: 46.438659°, Long: -83.820529° 38S-154 McLeod Rd Bridge, MTM Z13: N 5 144 346.5 E 318 590.8 ORIGINATED BY NW  
 HWY McLeod Road BOREHOLE TYPE HSA, and NW casing COMPILED BY KCP  
 DATUM Geodetic DATE 2016.02.08 - 2016.02.09 CHECKED BY PC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
	Continued From Previous Page							SHEAR STRENGTH kPa						
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
								WATER CONTENT (%)						
								20	40	60	80	100		
167.1	SILTY SAND (ML) loose to compact Grey		21	SS	25		168						0	0 83 17 (SI+CL)
31.1	Borehole terminated at 31.1 m Groundwater level was measured at 5.5 m BGS on February 2, 2016													



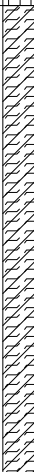

DOUBLE LINE 10870 MCLEOD RD.GPJ 2012TEMPLATE(MTO).GDT 11/4/19

# RECORD OF BOREHOLE No 304

1 OF 1

METRIC

GWP# 5148-13-00 LOCATION Lat: 46.438688°, Long: -83.820465° 38S-154 McLeod Rd Bridge, MTM Z13: N 5 144 349.7 E 318 595.7 ORIGINATED BY NW  
 HWY McLeod Road BOREHOLE TYPE HSA COMPILED BY KCP  
 DATUM Geodetic DATE 2016.02.09 - 2016.02.09 CHECKED BY PC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT  $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL							
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa													
								20 40 60 80 100													
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE													
WATER CONTENT (%)					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT																
20 40 60					W P W L																
198.2	Sandy gravel trace silt Brown <b>FILL</b> - frost from 0 m to 1.5 m		1	AS	-		198														
0.0																					
			2	AS	-		197														
196.6																					
1.5	<b>SILT (ML)</b> some clay Loose Brown		3	SS	5		196														
			4	SS	7																
195.1																					
3.0	<b>CLAY (Cl)</b> , sandy with organics - frequent wood pieces Soft to stiff		5	SS	5		195														
					6		SS	3	194												
			7	SS	5																
			8	SS	4		193														
192.0																					
6.2	<b>CLAY (Cl)</b> Soft to stiff Grey		9	SS	WH		192														
191.5																					
6.7	Borehole terminated at 6.7 m																				

DOUBLE LINE 10870 MCLEOD RD.GPJ 2012TEMPLATE(MTO).GDT 11/4/19

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity  
 20  
 15  
 10  
 (%) STRAIN AT FAILURE

## METRIC

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity



## METRIC

[illegible]

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity

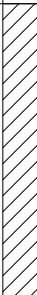

DOUBLE LINE 10870 MCLEOD RD.GPJ 2012TEMPLATE(MTO).GDT 11/4/19

# RECORD OF BOREHOLE No 305

3 OF 5

METRIC

GWP# 5148-13-00 LOCATION Lat: 46.438620°, Long: -83.820488° 38S-154 McLeod Rd Bridge, MTM Z13: N 5 144 342.2 E 318 593.9 ORIGINATED BY JAG  
 HWY McLeod Road BOREHOLE TYPE HSA, and NW casing COMPILED BY JM  
 DATUM Geodetic DATE 2016.07.18 - 2016.07.19 CHECKED BY PC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT			UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR   SA   SI   CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa			WATER CONTENT (%)								
								○ UNCONFINED      + FIELD VANE			W P      W      W L								
								● QUICK TRIAXIAL      × LAB VANE											
	Continued From Previous Page		17	SS	5		177												
	CLAY (CL to CI) Firm to stiff Grey																		
			18	SS	2		176												
173.4																			
24.4	SANDY SILT (ML) to SILTY SAND (SM) Loose to compact Grey		19	SS	20		173												
							172												
							171												
			20	SS	16		170												
							169												
							168												

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity  
 20  
 15  
 10  
 (%) STRAIN AT FAILURE

DOUBLE LINE 10870 MCLEOD RD.GPJ 2012TEMPLATE(MTO).GDT 11/4/19

# RECORD OF BOREHOLE No 305

4 OF 5

METRIC

GWP# 5148-13-00 LOCATION Lat: 46.438620°, Long: -83.820488° 38S-154 McLeod Rd Bridge, MTM Z13: N 5 144 342.2 E 318 593.9 ORIGINATED BY JAG  
 HWY McLeod Road BOREHOLE TYPE HSA, and NW casing COMPILED BY JM  
 DATUM Geodetic DATE 2016.07.18 - 2016.07.19 CHECKED BY PC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)					
								○ UNCONFINED      + FIELD VANE ● QUICK TRIAXIAL    × LAB VANE														
	Continued From Previous Page							20	40	60	80	100					GR	SA	SI	CL		
	SANDY SILT (ML) to SILTY SAND (SM) Loose to compact Grey		21	SS	27		167								○			0	9	83	8	
							166															
							165															
							164									○			8	51	41 (SI+CL)	
							163															
							162															
							161									○			0	14	75	11
							160															
							159															
			24	SS	9		158							○								

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity  
 20  
 15  
 10  
 (%) STRAIN AT FAILURE

## METRIC

[illegible]

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity

## METRIC

[illegible]

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity

DOUBLE LINE 10870 MCLEOD RD.GPJ 2012TEMPLATE(MTO).GDT 11/4/19

# RECORD OF BOREHOLE No 306

2 OF 2

METRIC

GWP# 5148-13-00 LOCATION Lat: 46.438573°, Long: -83.820427° 38S-154 McLeod Rd Bridge, MTM Z13: N 5 144 336.9 E 318 598.6 ORIGINATED BY JAG  
 HWY McLeod Road BOREHOLE TYPE HSA COMPILED BY JM  
 DATUM Geodetic DATE 2016.07.20 - 2016.07.20 CHECKED BY PC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
							20	40	60	80	100						
	Continued From Previous Page																
186.5	CLAY (Cl) Soft to firm Grey		11	SS	WH												
10.5	Borehole terminated at 10.5 m Groundwater level was measured at 2.8 m BGS on July 21, 2016																

DOUBLE LINE 10870 MCLEOD RD.GPJ 2012TEMPLATE(MTO).GDT 11/4/19

# RECORD OF BOREHOLE No 307

1 OF 5

METRIC

GWP# 5148-13-00 LOCATION Lat: 46.438411°, Long: -83.820978° 38S-154 McLeod Rd Bridge, MTM Z13: N 5 144 318.9 E 318 556.3 ORIGINATED BY JAG  
 HWY McLeod Road BOREHOLE TYPE HSA, and NW casing COMPILED BY JM  
 DATUM Geodetic DATE 2016.07.20 - 2016.07.21 CHECKED BY PC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT			UNIT WEIGHT  $\gamma$  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR   SA   SI   CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				W P      W      W L							
								○ UNCONFINED      + FIELD VANE				WATER CONTENT (%)							
								● QUICK TRIAXIAL      × LAB VANE											
197.1							20	40	60	80	100								
0.0																			
196.8																			
0.3																			
194.7																			
2.4																			

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity  
 20  
 15  
 10  
 (%) STRAIN AT FAILURE

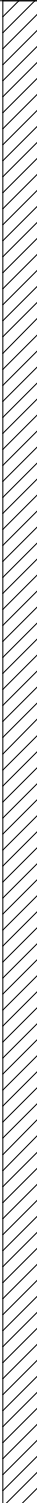
DOUBLE LINE 10870 MCLEOD RD.GPJ 2012TEMPLATE(MTO).GDT 11/4/19

# RECORD OF BOREHOLE No 307

2 OF 5

METRIC

GWP# 5148-13-00 LOCATION Lat: 46.438411°, Long: -83.820978° 38S-154 McLeod Rd Bridge, MTM Z13: N 5 144 318.9 E 318 556.3 ORIGINATED BY JAG  
 HWY McLeod Road BOREHOLE TYPE HSA, and NW casing COMPILED BY JM  
 DATUM Geodetic DATE 2016.07.20 - 2016.07.21 CHECKED BY PC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)
								○ UNCONFINED	+ FIELD VANE							
								● QUICK TRIAXIAL	× LAB VANE							
	Continued From Previous Page							20 40 60 80 100								
	CLAY (CL to Cl) Firm to stiff Grey						187	5.0 +								
			7	SS	WH		186	4.9 +							0 2 54 44	
			8	ST	PUSH		185	10.8 +								
			9	SS	WH		183	4.9 +								
			10	SS	WH		182	3.5 +								
			11	SS	3		181	4.9 +						0 1 81 18		
			12	SS	6		180	2.5 +								

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity  
 20  
15  
10  
5  
0 (%) STRAIN AT FAILURE

DOUBLE LINE 10870 MCLEOD RD.GPJ 2012TEMPLATE(MTO).GDT 11/4/19



## METRIC

[illegible]

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity

DOUBLE LINE 10870 MCLEOD RD.GPJ 2012TEMPLATE(MTO).GDT 11/4/19

## METRIC

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity

## METRIC

[illegible]

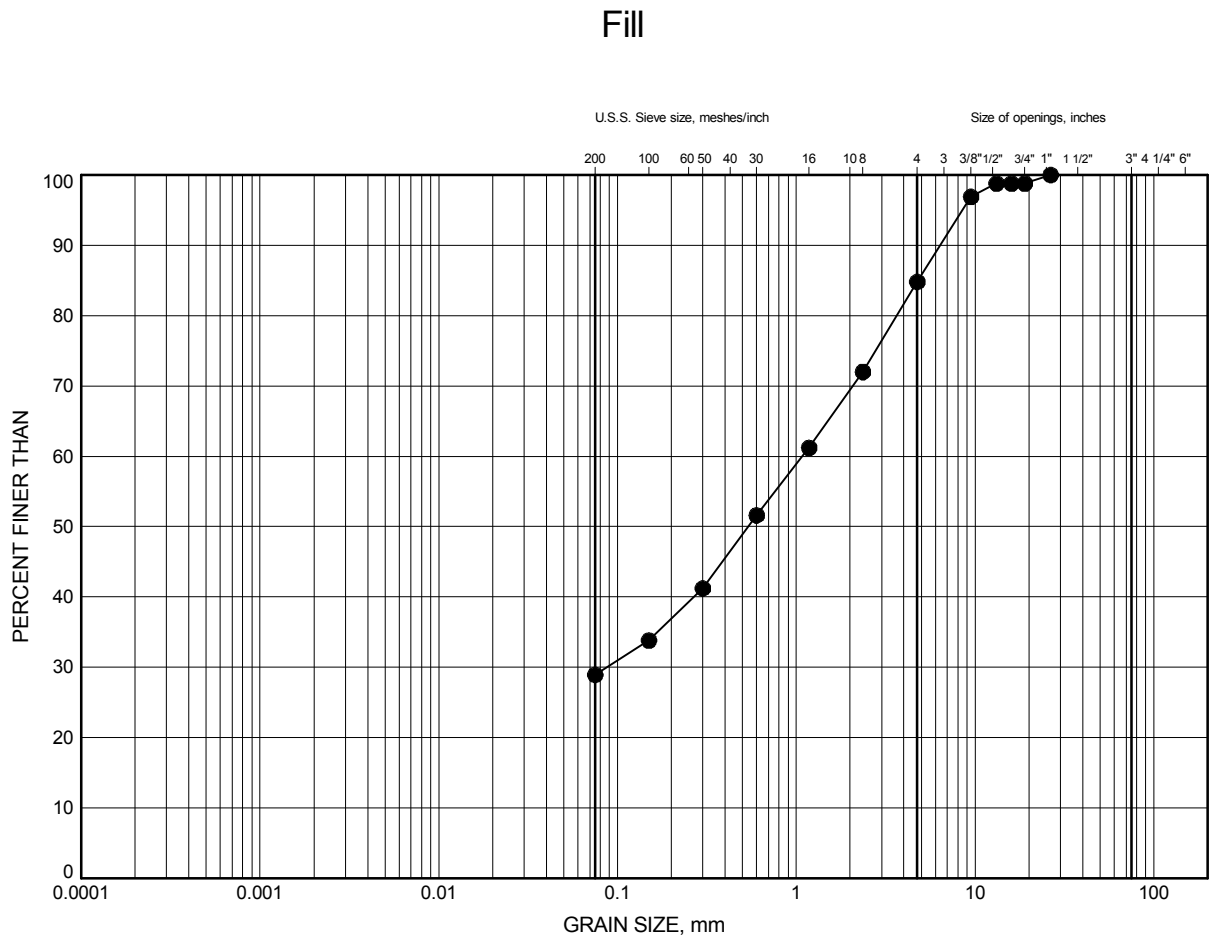
+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity

**APPENDIX C**  
**LABORATORY TEST RESULTS**

# 38S-154 McLeod Rd Bridge

## GRAIN SIZE DISTRIBUTION

FIGURE 1



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	301	0.30	198.39

Date April 2019  
GWP# 5148-13-00

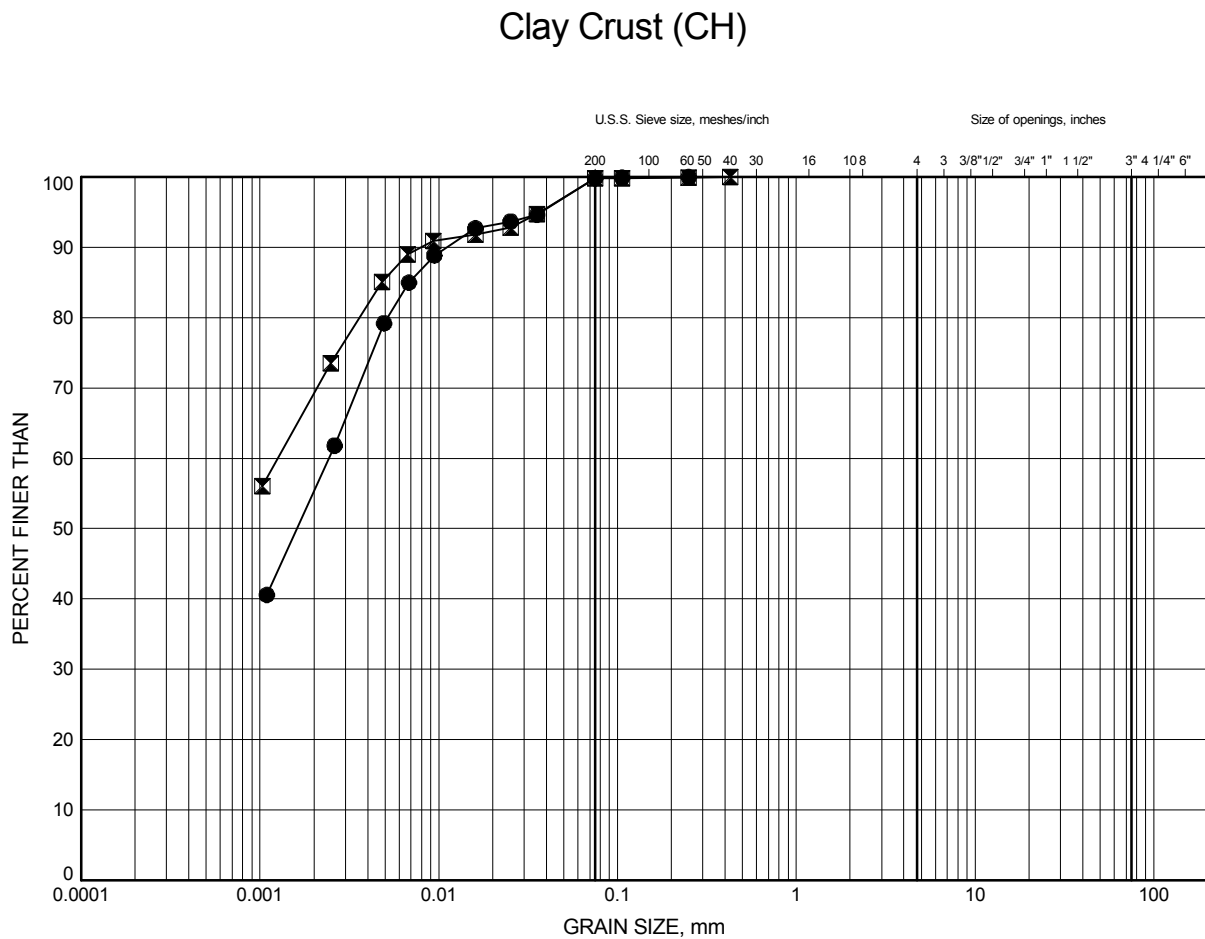


Prep'd DJP  
Chkd. PC

# 38S-154 McLeod Rd Bridge

## GRAIN SIZE DISTRIBUTION

FIGURE 2



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	301	1.07	197.63
⊠	302	2.59	195.80

Date April 2019  
GWP# 5148-13-00

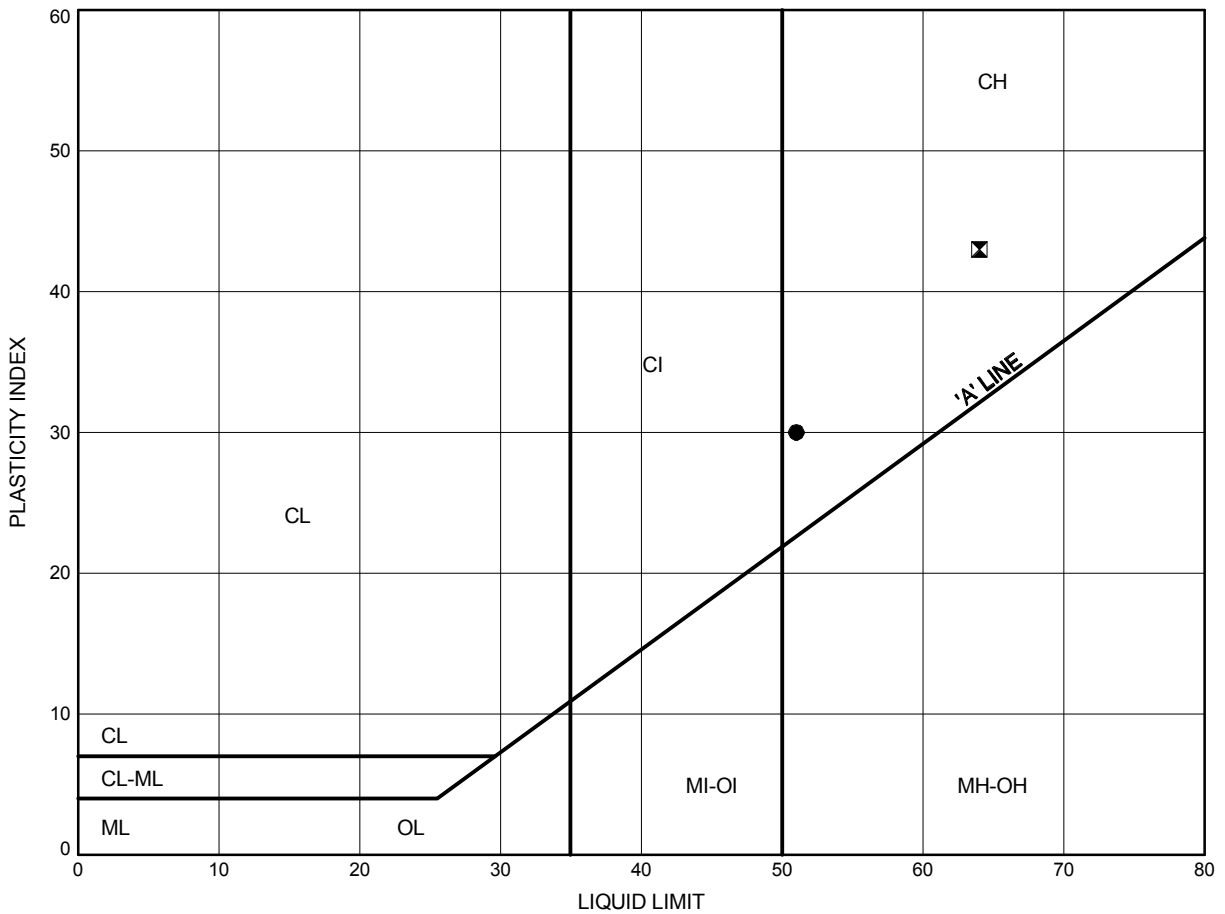


Prep'd DJP  
Chkd. PC

38S-154 McLeod Rd Bridge  
**ATTERBERG LIMITS TEST RESULTS**

FIGURE 3

Clay Crust (CH)



**LEGEND**

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	301	1.07	197.63
⊠	302	2.59	195.80

Date ..April 2019.....  
 GWP# ..5148-13-00.....



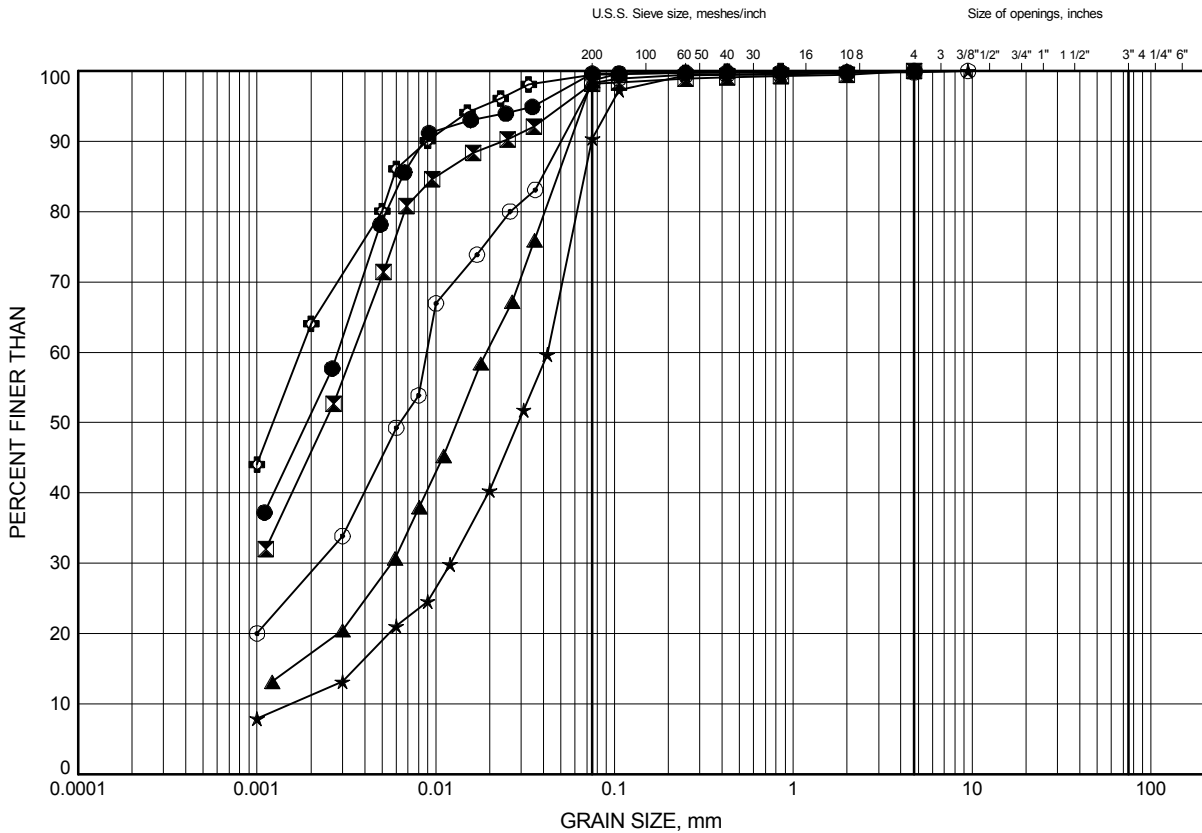
Prep'd .....DJP.....  
 Chkd. ....PC.....

# 38S-154 McLeod Rd Bridge

## GRAIN SIZE DISTRIBUTION

FIGURE 4

Clay (CL to CI) to Silty Clay (CL-ML)



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	301	4.88	193.82
⊠	302	9.45	188.94
▲	302	17.07	181.32
★	302	20.12	178.28
⊙	302	23.16	175.23
⊕	305	6.40	191.38

Date April 2019

GWP# 5148-13-00



Prep'd DJP

Chkd. PC

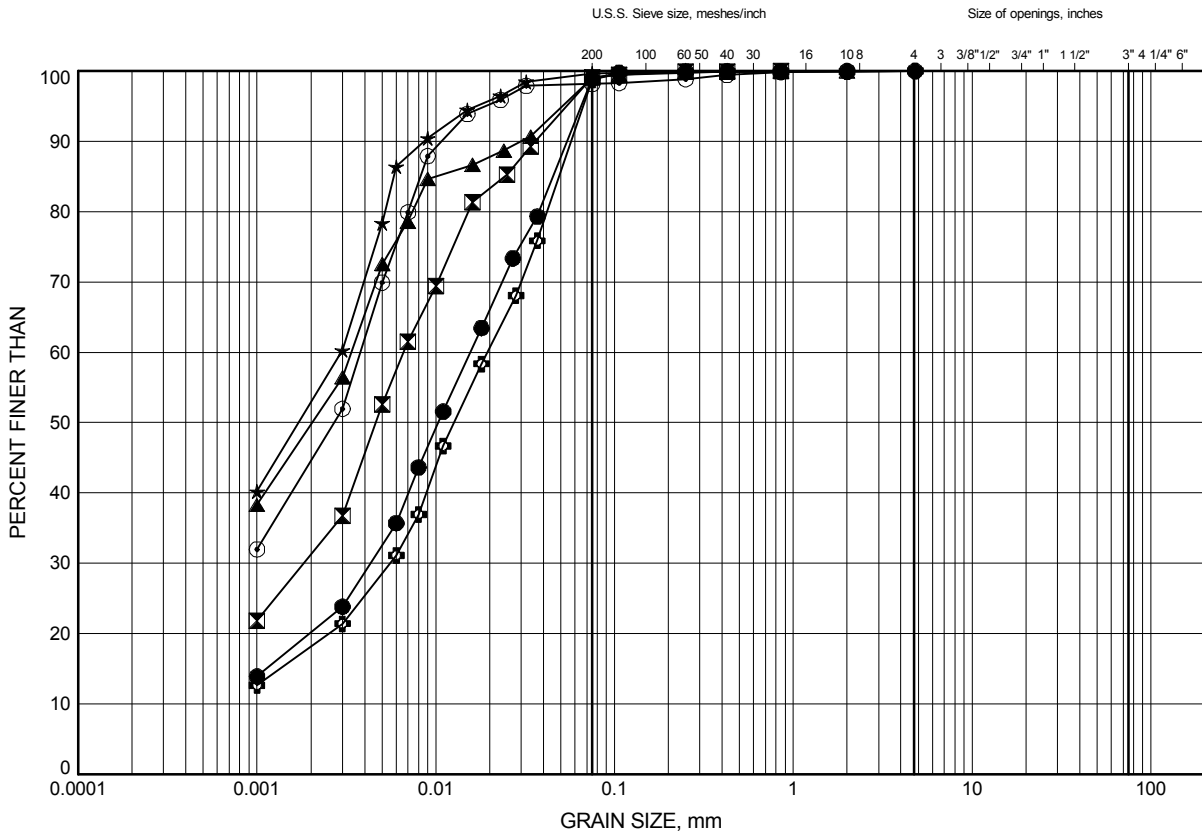


# 38S-154 McLeod Rd Bridge

## GRAIN SIZE DISTRIBUTION

FIGURE 5

Clay (CL to CI) to Silty Clay (CL-ML)



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	305	16.76	181.02
⊠	305	21.64	176.14
▲	306	7.16	189.90
★	307	4.88	192.24
⊙	307	10.97	186.15
⊕	307	17.07	180.05

Date April 2019  
GWP# 5148-13-00



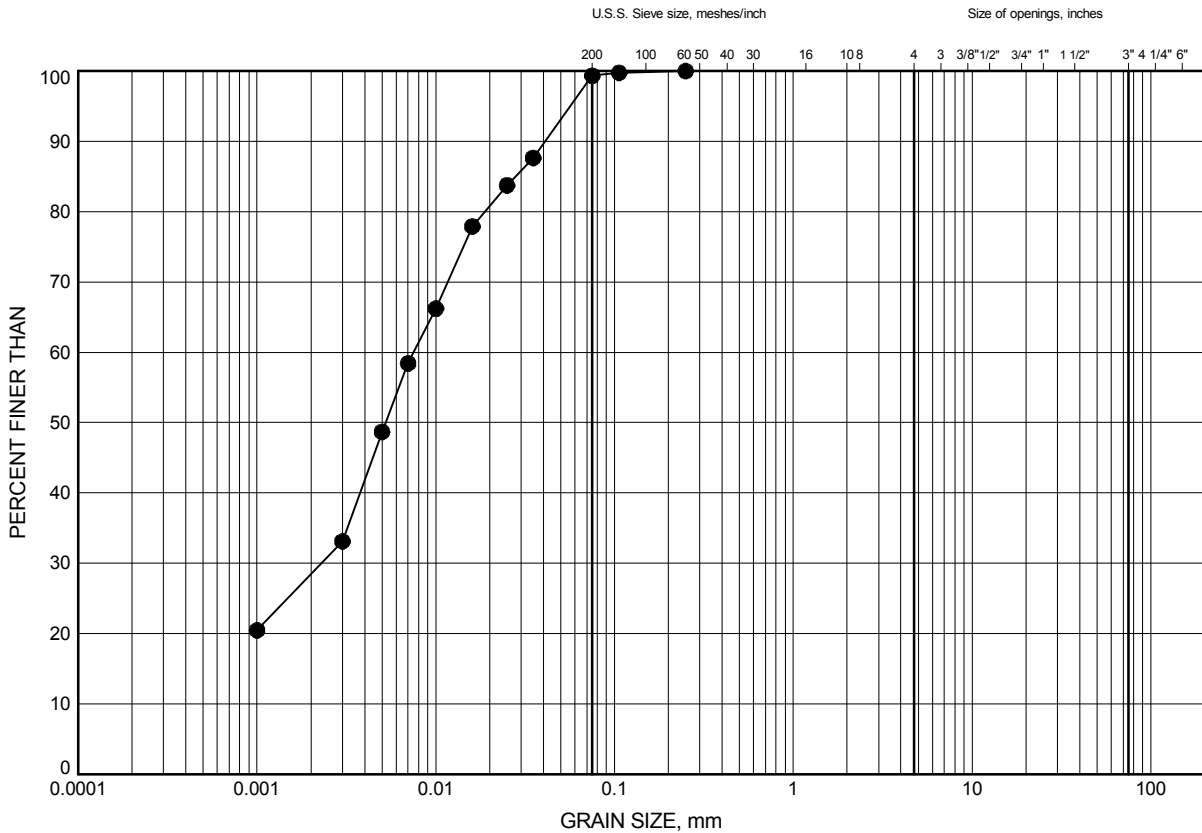
Prep'd DJP  
Chkd. PC

# 38S-154 McLeod Rd Bridge

## GRAIN SIZE DISTRIBUTION

FIGURE 6

Clay (CL to CI) to Silty Clay (CL-ML)



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	307	21.64	175.48

Date April 2019  
GWP# 5148-13-00



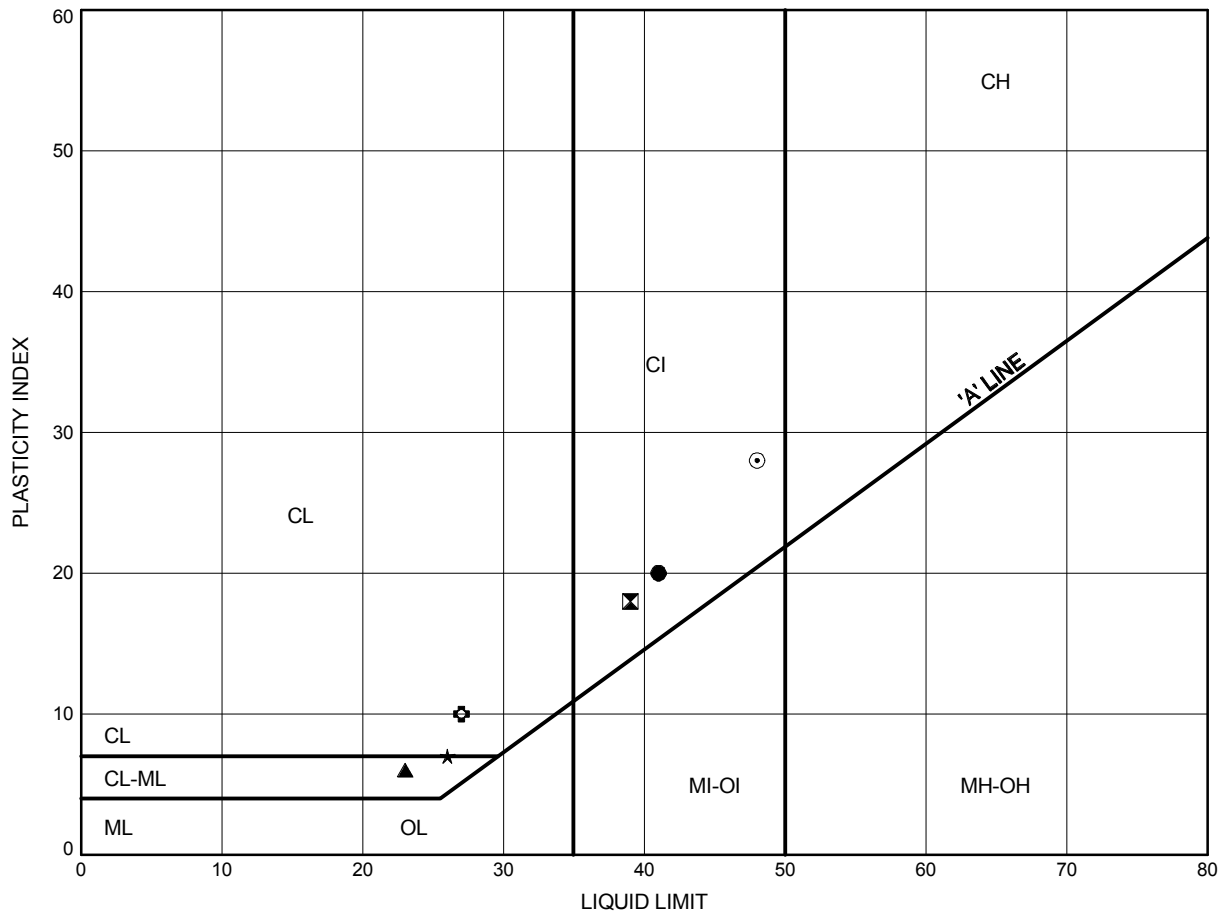
Prep'd DJP  
Chkd. PC

38S-154 McLeod Rd Bridge

# ATTERBERG LIMITS TEST RESULTS

FIGURE 7

Clay (CI to CI) to Silty Clay (CL-ML)



## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	301	4.88	193.82
⊠	302	9.45	188.94
▲	302	17.07	181.32
★	302	23.16	175.23
⊙	305	6.40	191.38
⊕	305	16.76	181.02

Date April 2019  
GWP# 5148-13-00

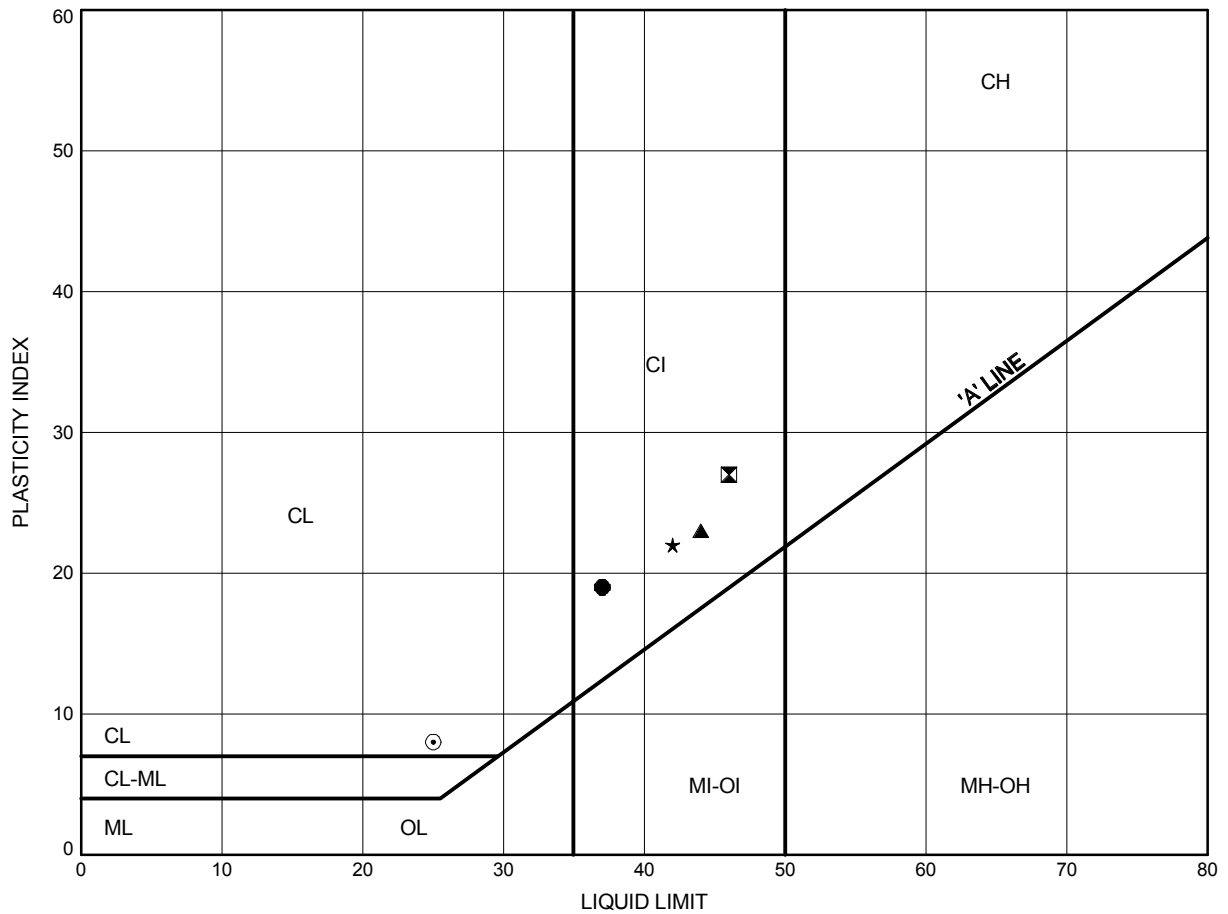


Prep'd DJP  
Chkd. PC

38S-154 McLeod Rd Bridge  
**ATTERBERG LIMITS TEST RESULTS**

FIGURE 8

Clay (CI to CI) to Silty Clay (CL-ML)



**LEGEND**

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	305	21.64	176.14
⊠	306	7.16	189.90
▲	307	4.88	192.24
★	307	10.97	186.15
⊙	307	17.07	180.05
⊕	307	21.64	175.48

Date ..April 2019.....  
 GWP# ..5148-13-00.....



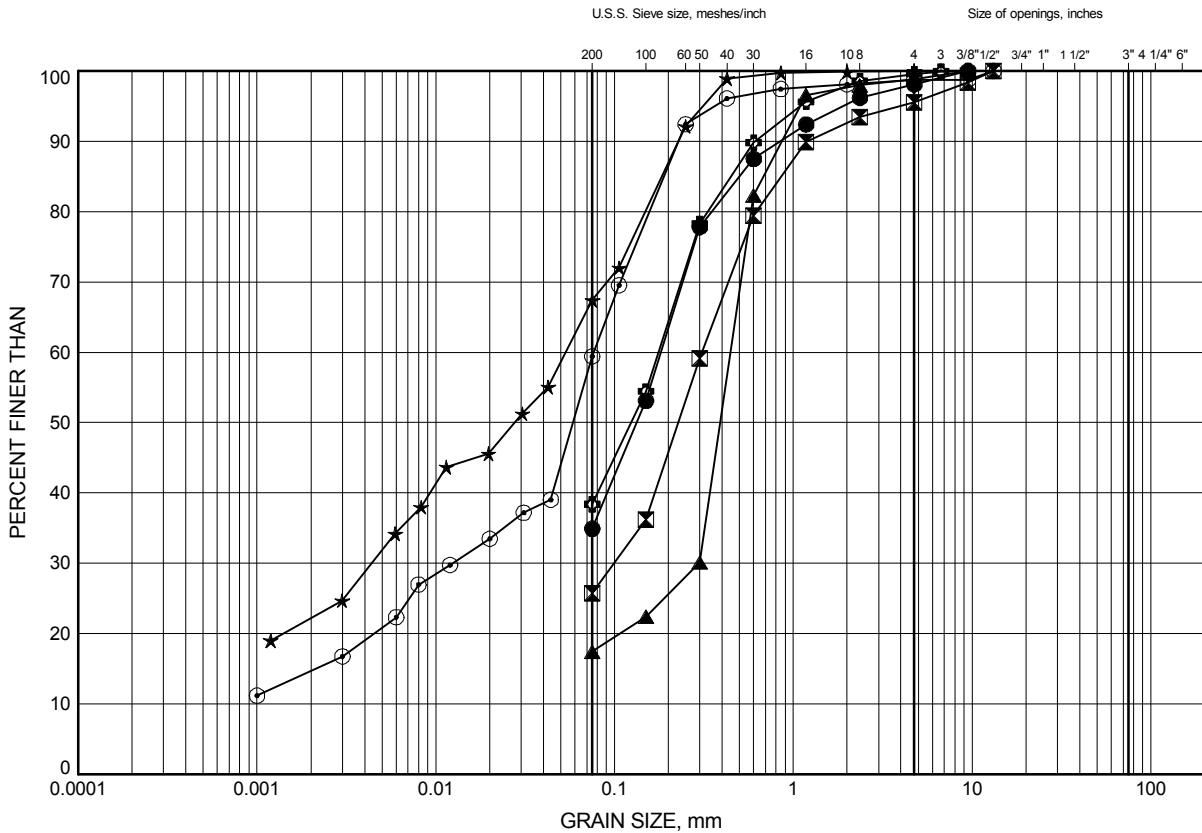
Prep'd .....DJP.....  
 Chkd. ....PC.....

# 38S-154 McLeod Rd Bridge

## GRAIN SIZE DISTRIBUTION

FIGURE 9

Clay with layers of Silty Sand and Silt with Frequent Wood Pieces



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	303	4.11	194.09
⊠	303	4.88	193.33
▲	303	5.64	192.57
★	304	4.11	194.05
⊙	305	3.35	194.43
⊕	305	4.11	193.67

Date April 2019

GWP# 5148-13-00



Prep'd DJP

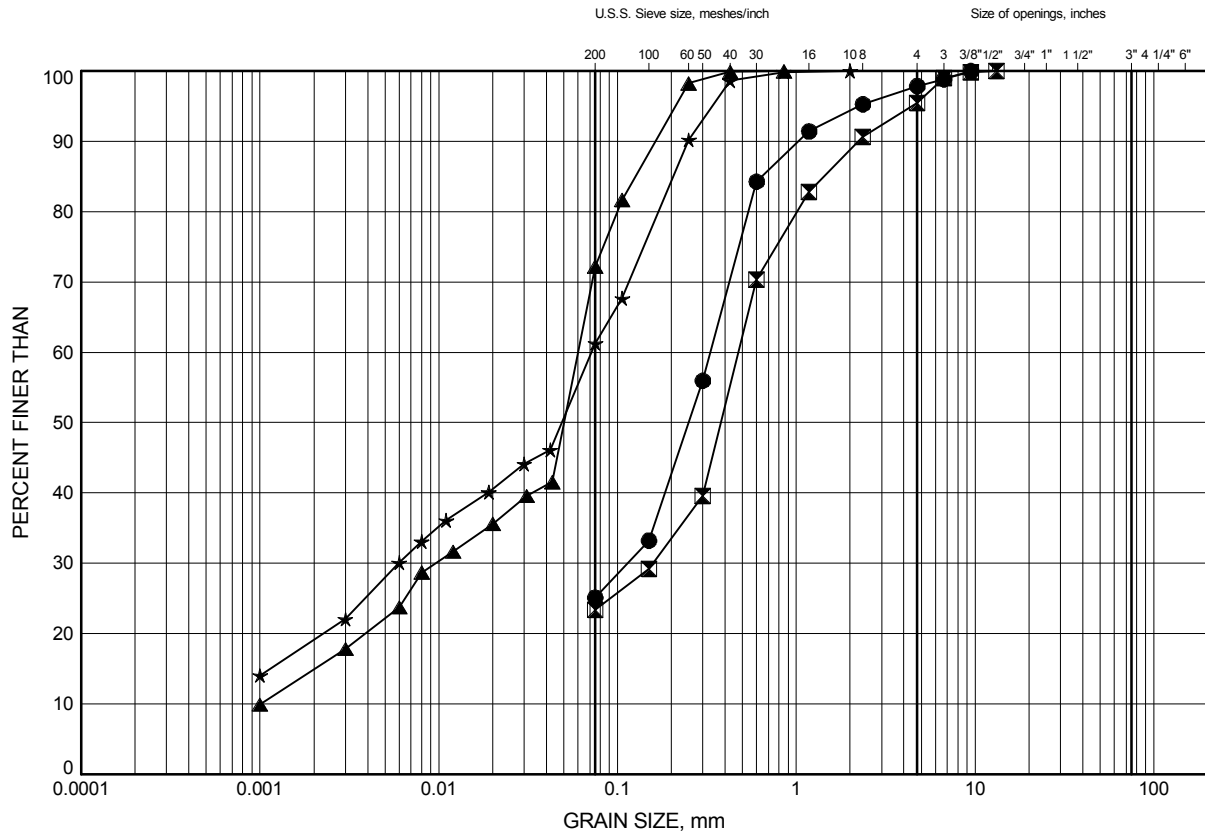
Chkd. PC

# 38S-154 McLeod Rd Bridge

## GRAIN SIZE DISTRIBUTION

FIGURE 10

Clay with layers of Silty Sand and Silt with Frequent Wood Pieces



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	305	4.88	192.91
⊠	305	5.56	192.22
▲	306	2.59	194.47
★	306	4.11	192.95

Date April 2019  
GWP# 5148-13-00

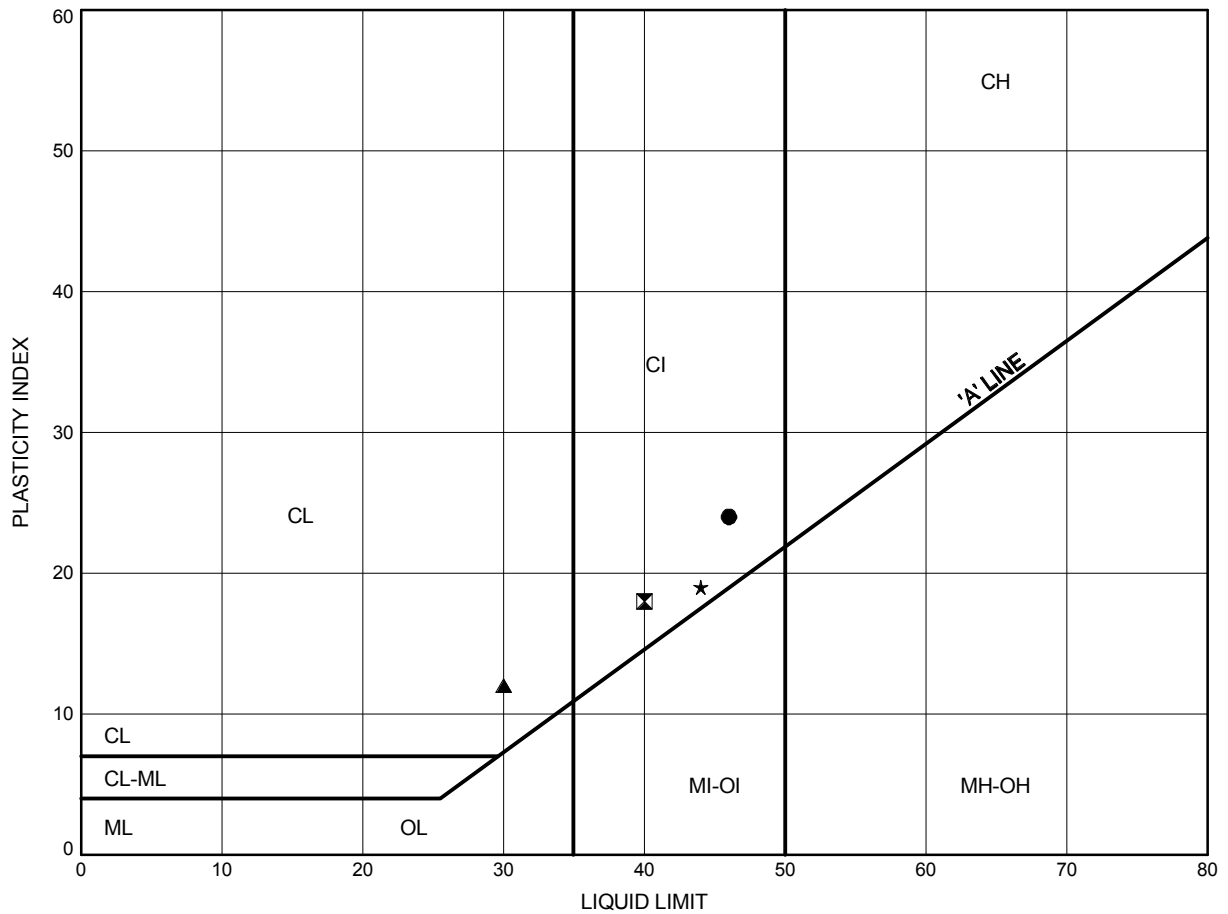


Prep'd DJP  
Chkd. PC

38S-154 McLeod Rd Bridge  
**ATTERBERG LIMITS TEST RESULTS**

FIGURE 11

Clay with layers of Silty Sand and Silt with Frequent Wood Pieces



**LEGEND**

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	304	4.11	194.05
⊠	305	3.35	194.43
▲	306	2.59	194.47
★	306	4.11	192.95

Date April 2019  
 GWP# 5148-13-00



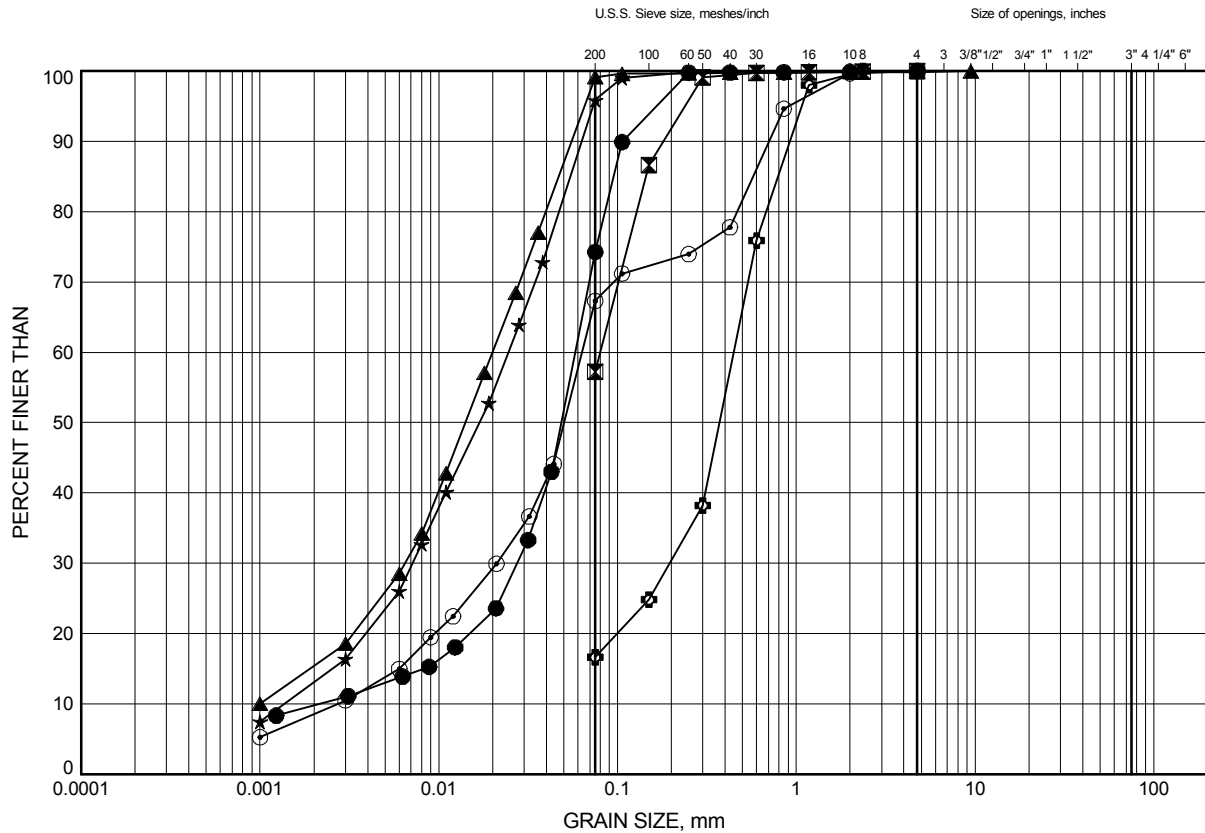
Prep'd DJP  
 Chkd. PC

# 38S-154 McLeod Rd Bridge

## GRAIN SIZE DISTRIBUTION

FIGURE 12

### Sandy Silt to Silty Sand



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	302	29.26	169.13
⊠	302	30.78	167.61
▲	303	17.07	181.14
★	303	20.12	178.09
⊙	303	26.21	172.00
⊕	303	30.78	167.42

Date April 2019

GWP# 5148-13-00



Prep'd DJP

Chkd. PC

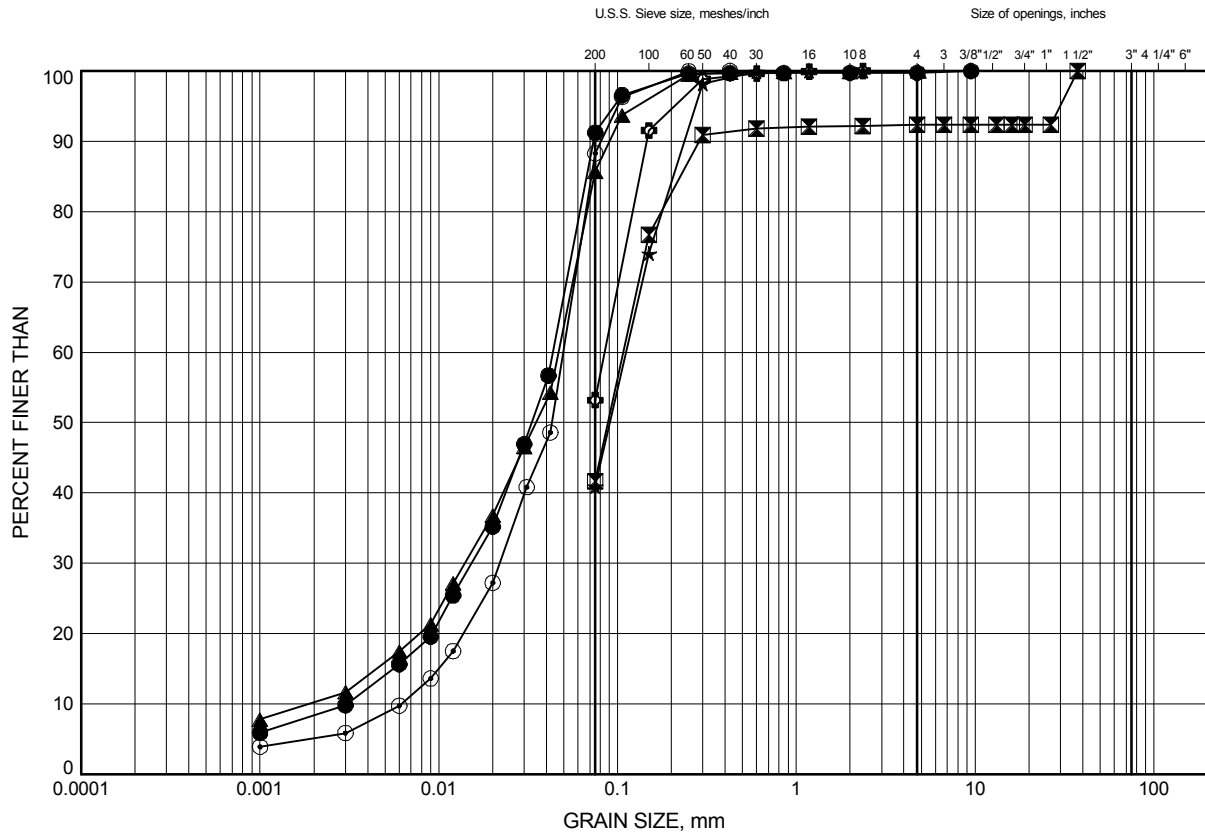


# 38S-154 McLeod Rd Bridge

## GRAIN SIZE DISTRIBUTION

FIGURE 13

### Sandy Silt to Silty Sand



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	305	30.78	167.00
⊠	305	33.83	163.95
▲	305	36.88	160.90
★	305	42.98	154.81
⊙	307	24.69	172.43
⊕	307	33.83	163.29

Date April 2019  
GWP# 5148-13-00



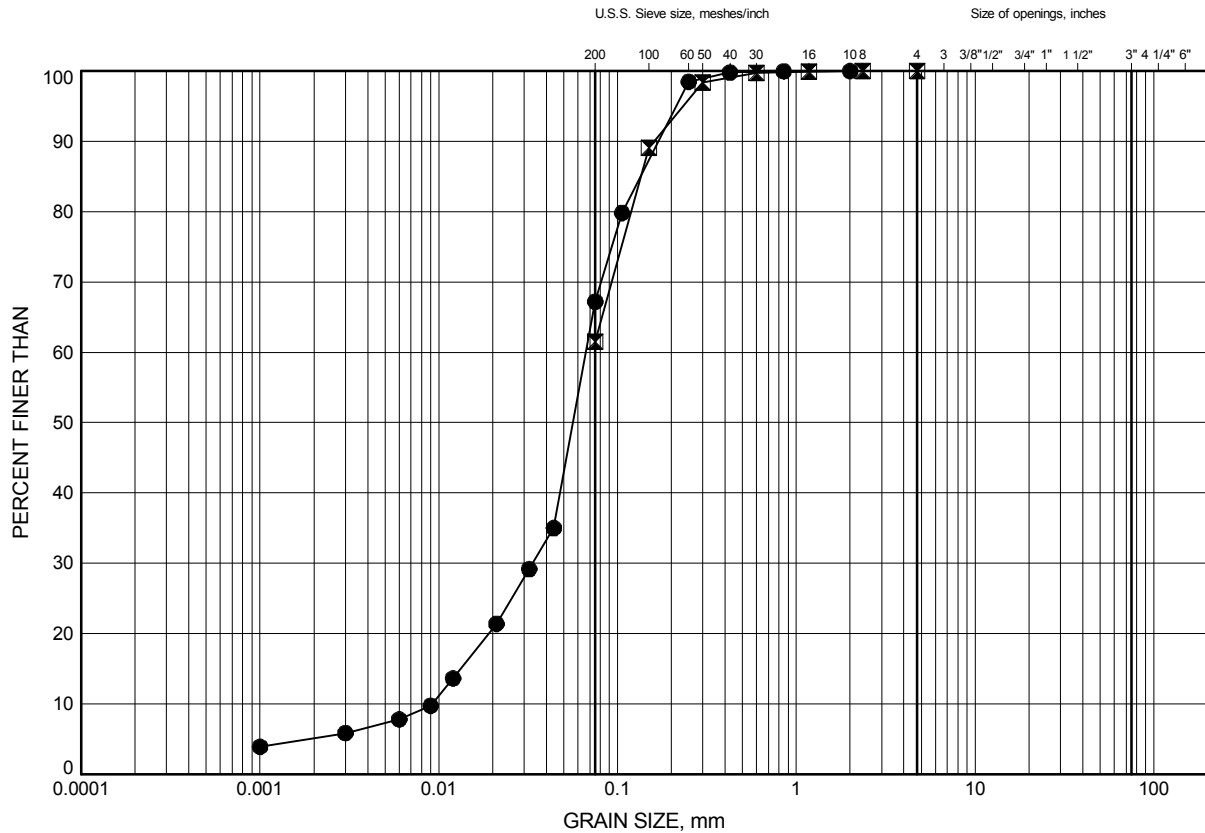
Prep'd DJP  
Chkd. PC

# 38S-154 McLeod Rd Bridge

## GRAIN SIZE DISTRIBUTION

FIGURE 14

### Sandy Silt to Silty Sand



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	307	36.88	160.24
⊠	307	42.98	154.14

Date April 2019  
GWP# 5148-13-00



Prep'd DJP  
Chkd. PC



**Stantec**

**Stantec Consulting Ltd**  
100B – 2781 Lancaster Rd  
Ottawa, ON K1B 1A7  
Tel: (613) 738-6075  
Fax: (613) 738-6067

March 30, 2016  
File: 122410864

**Attention: Kenton Power, Thurber Engineering File #10870**

**Reference: ASTM D2974 Organic Matter of Peat & Other Soils**

The table below summarizes four (4) test results for Organic Matter of Peat and Other Soils.

Source	Location	Depth	% Organic Content
BH-303 SS-6	LRB McLeod Road	12'6"-14'6"	14.07
BH-303 SS-7	LRB McLeod Road	15'-17'	13.55
BH-304 SS-6	LRB McLeod Road	12'6"-14'6"	6.15
BH-304 SS-5	LRB McLeod Road	10'-12'	3.90

Sincerely,

**Stantec Consulting Ltd.**

*Brian Prevost*

Brian Prevost  
Laboratory Supervisor  
Tel: 613-738-6075  
Fax: 613-738-6067  
[brian.prevost@stantec.com](mailto:brian.prevost@stantec.com)



**Stantec**

**Stantec Consulting Ltd**  
100B – 2781 Lancaster Rd  
Ottawa, ON K1B 1A7  
Tel: (613) 738-6075  
Fax: (613) 738-6067

August 16, 2016  
File: 122410864

**Attention:      Kenton Power, Thurber Engineering File #10870**

**Reference:      ASTM D2974 Organic Matter of Peat & Other Soils**

The table below summarizes four test results for Organic Matter of Peat and Other Soils.

Source	Location	Depth	Organic Content
<i>BH305 SS-6</i>	McLeod Road	12'6"-14'6"	8.3%
<i>BH305 SS-7</i>	McLeod Road	15'-17'	11.8%
<i>BH305 SS-8A</i>	McLeod Road	17'6"-19'6"	9.8%
<i>BH306 SS-6</i>	McLeod Road	12'6"-14'6"	8.2%

Sincerely,

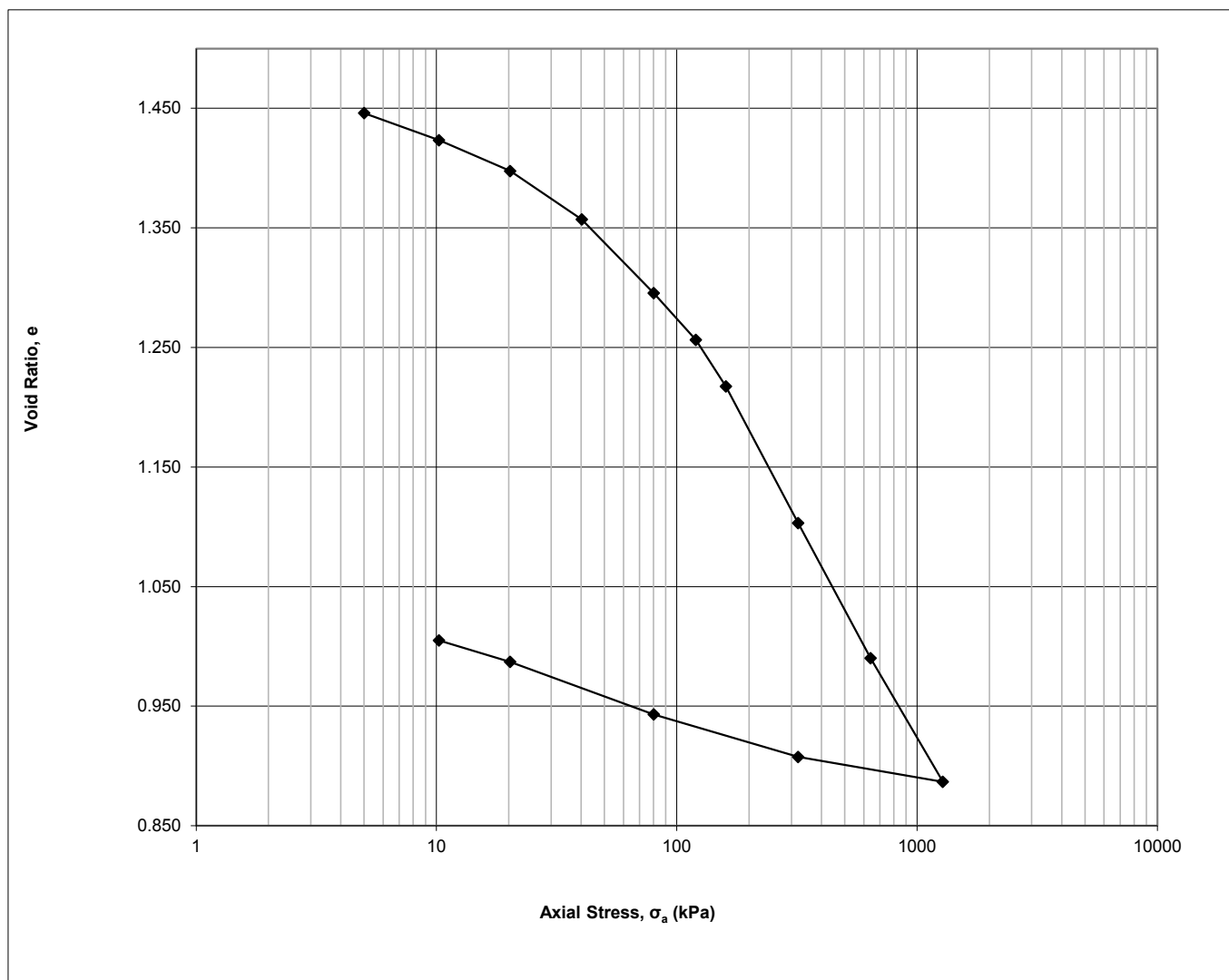
**Stantec Consulting Ltd.**

*Brian Prevost*

Brian Prevost  
Laboratory Supervisor  
Tel: 613-738-6075  
Fax: 613-738-6067  
[brian.prevost@stantec.com](mailto:brian.prevost@stantec.com)

**Project**  
**Project No.**  
**Borehole No.**  
**Sample No.**  
**Sample Depth**

**Thurber Consulting Ltd., LRB McLeod Road, Project # 10870**  
**122410864**  
**BH 302**  
**ST-1**  
**10-12 ft**



**One-Dimensional Consolidation Test using Incremental Loading**  
**ASTM D2435/D2435M - 11**

2016-04-22  
2016-04-22

Date:  
Date:

Checked by: MO  
Approved by: RH

**Specimen Details**

Project Name	Thurber Consulting Ltd., LRB McLeod Road, Project # 10870
Project Location	Sault Ste. Marie
Borehole	BH 302
Sample No.	ST-1
Depth	10-12 ft
Sample Date	09/02/2016
Test Number	One
Technician Name	Daniel Boateng

**Soil Description & Classification**

Silty Clay	
Specific Gravity of Solids	2.699
Average water content of trimmings %	64
<b>Additional Notes (information source, occurrence and size of large isolated particles etc.)</b>	
Sample was sensitive and severely desiccated	

**Initial Specimen Conditions**

Height	mm	20.00
Diameter	mm	50.00
Area	mm <sup>2</sup>	1963
Volume	mm <sup>3</sup>	39270
Mass	g	71.26
Dry Mass	g	43.33
Density	Mg/m <sup>3</sup>	1.815
Dry Density	Mg/m <sup>3</sup>	1.103
Water Content	%	64.46
Degree of Saturation	%	120.3
Height of Solids	mm	8.18
Initial Void Ratio		1.446

**Final Specimen Conditions**

Water Content	%	49.43
Final Void Ratio		1.005

## One-Dimensional Consolidation Test using Incremental Loading

### ASTM D2435/D2435M - 11

**Specimen Details**

Project Name	Thurber Consulting Ltd., LRB McLeod Road, Project # 10870
Project Location	Sault Ste. Marie
Borehole	BH 302
Sample No.	ST-1
Depth	10-12 ft
Sample Date	09/02/2016
Test Number	One
Technician Name	Daniel Boateng

**Test Procedure**

Date Started	25/02/2016
Date Finished	09/03/2016
Machine Number	C
Cell Number	C
Ring Number	C
Trimming Procedure	Turntable
Moisture Condition	Inundated
Axial Stress at Inundation	5 kPa
Water Used	Distilled
Test Method	A
Interpretation Procedure for $c_v$	2

**All Departures from Outlined ASTM D2435/D2435M-11 Procedure**
**Calculations**

Load Increment	Increment Duration	Axial Stress $\sigma_a$ kPa	Corrected Deformation $\Delta H$ mm	Specimen Height H mm	Axial Strain $\epsilon_a$ %	Void Ratio e
Seating	1440.0	5	0.0000	20.0000	0.00	1.446
1	1440.0	10	0.1855	19.8145	0.93	1.423
2	1440.0	20	0.3967	19.6033	1.98	1.398
3	1440.0	40	0.7287	19.2713	3.64	1.357
4	1440.0	80	1.2311	18.7689	6.16	1.296
5	1440.0	120	1.5524	18.4476	7.76	1.256
6	1440.0	160	1.8692	18.1308	9.35	1.217
7	1440.0	320	2.8050	17.1950	14.03	1.103
8	1440.0	640	3.7277	16.2723	18.64	0.990
9	1440.0	1280	4.5738	15.4262	22.87	0.887
10	1440.0	320	4.4033	15.5967	22.02	0.908
11	1440.0	80	4.1130	15.8870	20.57	0.943
12	1440.0	20	3.7536	16.2464	18.77	0.987
13	1440.0	10	3.6063	16.3937	18.03	1.005
14						
15						
16						
17						
18						
19						

## One-Dimensional Consolidation Test using Incremental Loading

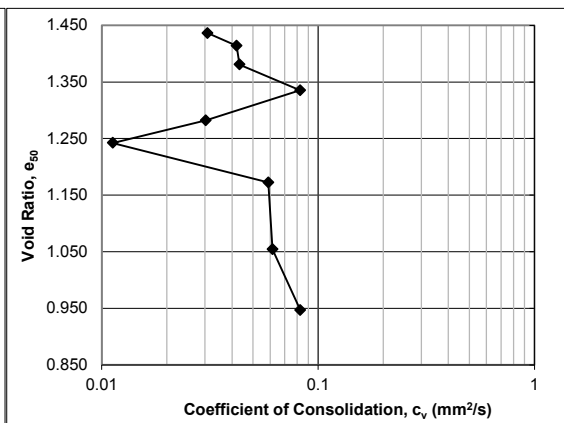
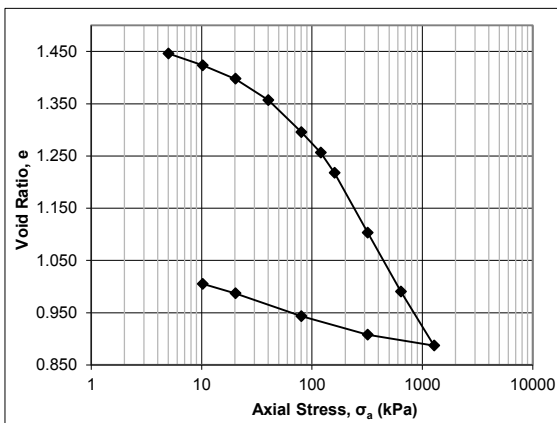
### ASTM D2435/D2435M - 11

**Specimen Details**

Job Ref.	Thurber Consulting Ltd., LRB McLeod Road, Project # 10870
Job Location	Sault Ste. Marie
Borehole	BH 302
Sample No.	ST-1
Depth	10-12 ft
Sample Date	09/02/2016
Test Number	One
Technician Name	Daniel Boateng

**Calculations**

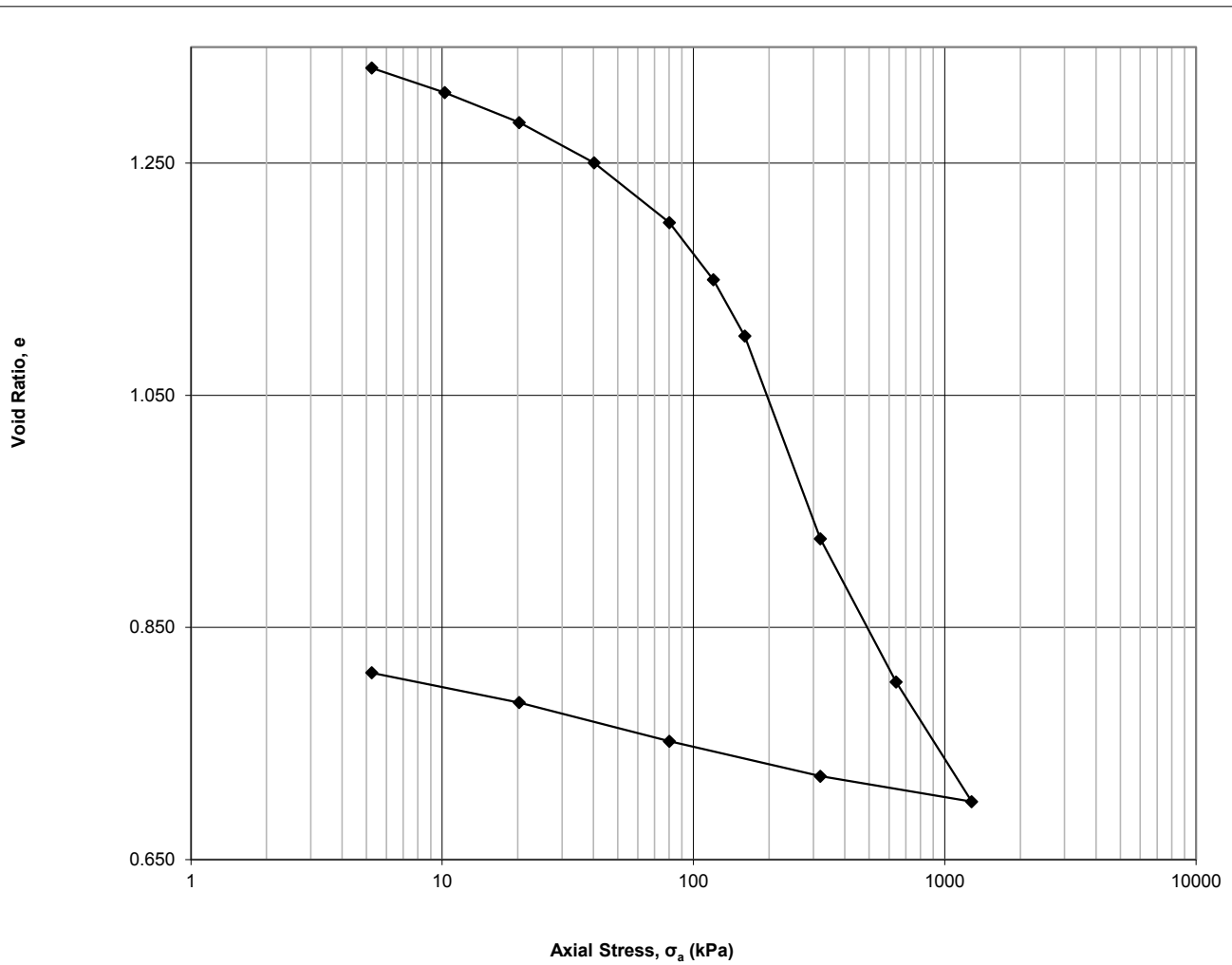
Load Increment	Axial Stress $\sigma_a$ , average kPa	Calculated using Interpretation Procedure 2				Interpretation Procedure 1		Interpretation Procedure 2	
		Corrected Deformation $\Delta H_{50}$ mm	Specimen Height $H_{50}$ mm	Axial Strain $\epsilon_{a,50}$ %	Void Ratio $e_{50}$	Time $t_{50}$ sec	Coeff. Consol. $c_v$ mm <sup>2</sup> /s	Time $t_{90}$ sec	Coeff. Consol. $c_v$ mm <sup>2</sup> /s
Seating	5								
1	10	0.0797	19.9203	0.40	1.436	558	3.50E-02	2731	3.08E-02
2	15	0.2635	19.7365	1.32	1.414	614	3.12E-02	1965	4.20E-02
3	30	0.5345	19.4655	2.67	1.381	515	3.62E-02	1853	4.34E-02
4	60	0.9056	19.0944	4.53	1.335	448	3.98E-02	936	8.26E-02
5	100	1.3403	18.6597	6.70	1.282	881	1.94E-02	2440	3.02E-02
6	140	1.6668	18.3332	8.33	1.242	485	3.43E-02	6335	1.12E-02
7	240	2.2386	17.7614	11.19	1.172	267	5.82E-02	1136	5.88E-02
8	480	3.2022	16.7978	16.01	1.054	242	5.75E-02	971	6.16E-02
9	960	4.0822	15.9178	20.41	0.947	107	1.17E-01	650	8.27E-02
10	800	4.4343	15.5657	22.17	0.904				
11	200	4.2115	15.7885	21.06	0.931				
12	50	3.9020	16.0980	19.51	0.969				
13	15	3.7467	16.2533	18.73	0.988				
14									
15									
16									
17									
18									
19									





**Project**  
**Project No.**  
**Borehole No.**  
**Sample No.**  
**Sample Depth**

**Thurber Consulting Ltd., LRB McLeod Road, Project # 10870**  
**122410864**  
**BH 302**  
**ST-2**  
**35-37 ft**



**One-Dimensional Consolidation Test using Incremental Loading**  
**ASTM D2435/D2435M - 11**

2016-04-22  
2016-04-22

Date:  
Date:

Checked by: MO  
Approved by: RH

**Specimen Details**

Project Name	Thurber Consulting Ltd., LRB McLeod Road, Project # 10870
Project Location	Sault Ste. Marie
Borehole	BH 302
Sample No.	ST-2
Depth	35-37 ft
Sample Date	09/02/2016
Test Number	Two
Technician Name	Daniel Boateng

**Soil Description & Classification**

Clay	
Specific Gravity of Solids	2.809
Average water content of trimmings %	46
<b>Additional Notes (information source, occurrence and size of large isolated particles etc.)</b>	
Significant amounts of sand seams present in sample	

**Initial Specimen Conditions**

Height	mm	20.00
Diameter	mm	50.00
Area	mm <sup>2</sup>	1963
Volume	mm <sup>3</sup>	39270
Mass	g	68.56
Dry Mass	g	47.10
Density	Mg/m <sup>3</sup>	1.746
Dry Density	Mg/m <sup>3</sup>	1.199
Water Content	%	45.56
Degree of Saturation	%	95.4
Height of Solids	mm	8.54
Initial Void Ratio		1.342

**Final Specimen Conditions**

Water Content	%	28.13
Final Void Ratio		0.811

## One-Dimensional Consolidation Test using Incremental Loading

### ASTM D2435/D2435M - 11

**Specimen Details**

Project Name	Thurber Consulting Ltd., LRB McLeod Road, Project # 10870
Project Location	Sault Ste. Marie
Borehole	BH 302
Sample No.	ST-2
Depth	35-37 ft
Sample Date	09/02/2016
Test Number	Two
Technician Name	Daniel Boateng

**Test Procedure**

Date Started	25/02/2016
Date Finished	09/03/2016
Machine Number	D
Cell Number	D
Ring Number	D
Trimming Procedure	Turntable
Moisture Condition	Inundated
Axial Stress at Inundation	3 kPa
Water Used	Distilled
Test Method	A
Interpretation Procedure for $c_v$	2

**All Departures from Outlined ASTM D2435/D2435M-11 Procedure**
**Calculations**

Load Increment	Increment Duration	Axial Stress $\sigma_a$ kPa	Corrected Deformation $\Delta H$ mm	Specimen Height $H$ mm	Axial Strain $\epsilon_a$ %	Void Ratio $e$
Seating	1440.0	3	0.0000	20.0000	0.00	1.342
1	1440.0	5	0.0868	19.9132	0.43	1.332
2	1440.0	10	0.2672	19.7328	1.34	1.311
3	1440.0	20	0.4877	19.5123	2.44	1.285
4	1440.0	40	0.7836	19.2164	3.92	1.250
5	1440.0	80	1.2239	18.7761	6.12	1.199
6	1440.0	120	1.6452	18.3548	8.23	1.149
7	1440.0	160	2.0587	17.9413	10.29	1.101
8	1440.0	320	3.5504	16.4496	17.75	0.926
9	1440.0	640	4.6028	15.3972	23.01	0.803
10	1439.0	1280	5.4832	14.5168	27.42	0.700
11	1439.0	320	5.2962	14.7038	26.48	0.722
12	1440.0	80	5.0389	14.9611	25.19	0.752
13	1440.0	20	4.7539	15.2461	23.77	0.785
14	1440.0	5	4.5353	15.4647	22.68	0.811
15						
16						
17						
18						
19						

## One-Dimensional Consolidation Test using Incremental Loading

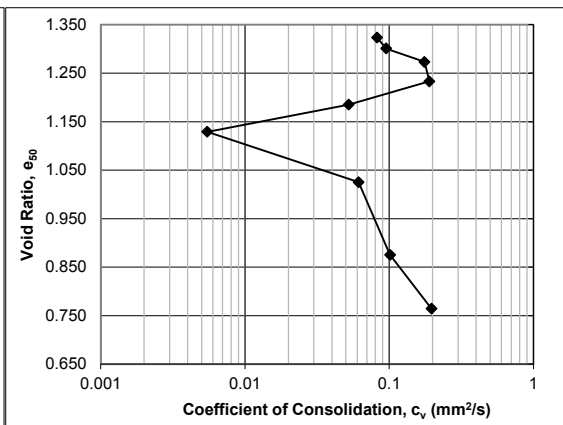
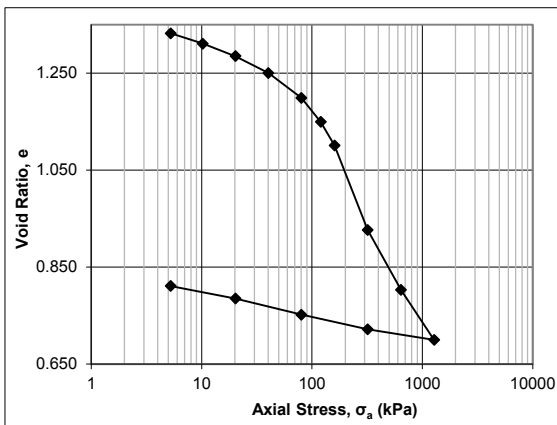
### ASTM D2435/D2435M - 11

**Specimen Details**

Job Ref.	Thurber Consulting Ltd., LRB McLeod Road, Project # 10870
Job Location	Sault Ste. Marie
Borehole	BH 302
Sample No.	ST-2
Depth	35-37 ft
Sample Date	09/02/2016
Test Number	Two
Technician Name	Daniel Boateng

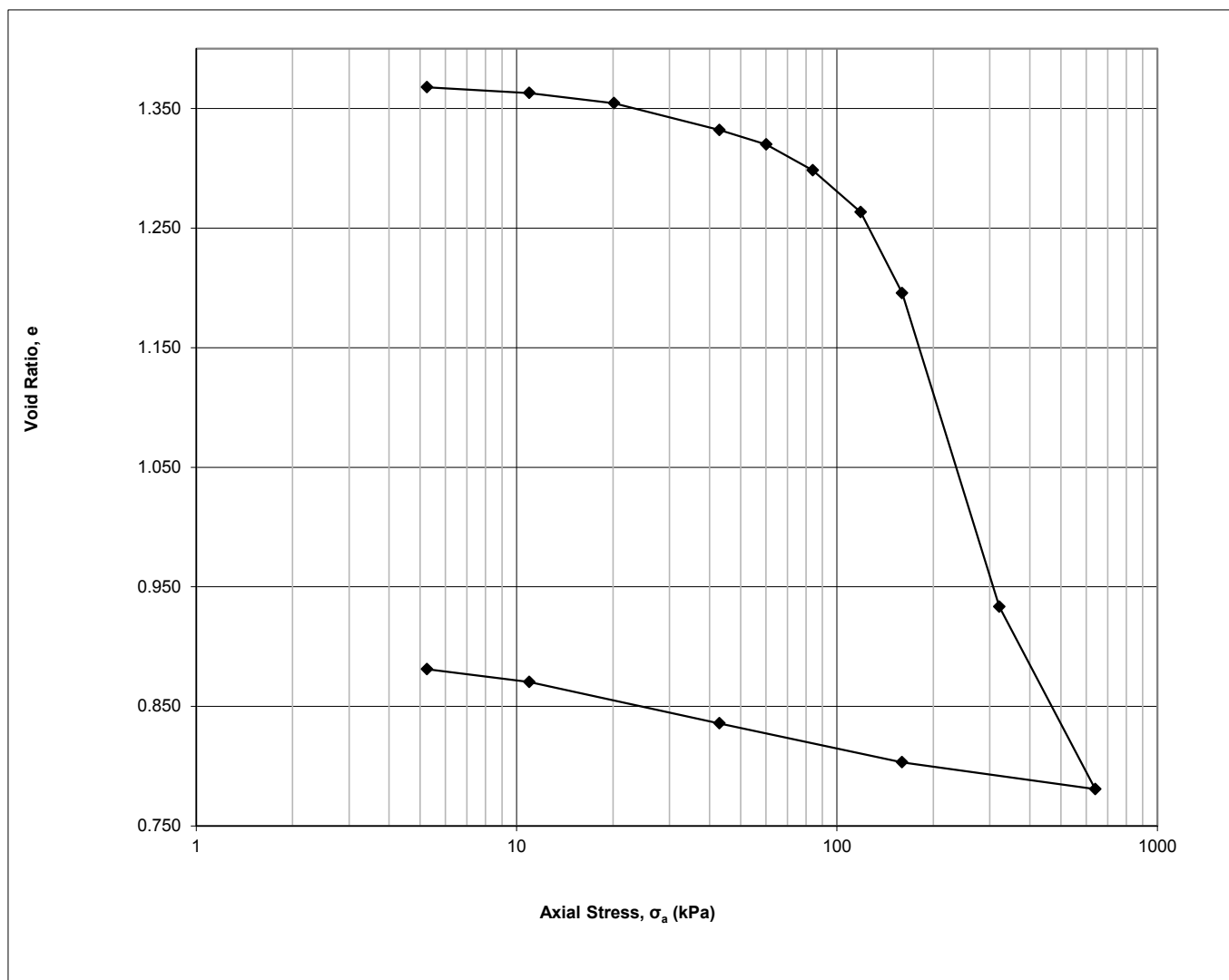
**Calculations**

Load Increment	Axial Stress $\sigma_{a, average}$ kPa	Calculated using Interpretation Procedure 2				Interpretation Procedure 1		Interpretation Procedure 2	
		Corrected Deformation $\Delta H_{50}$ mm	Specimen Height $H_{50}$ mm	Axial Strain $\epsilon_{a, 50}$ %	Void Ratio $e_{50}$	Time $t_{50}$ sec	Coeff. Consol. $c_v$ mm <sup>2</sup> /s	Time $t_{90}$ sec	Coeff. Consol. $c_v$ mm <sup>2</sup> /s
Seating	3								
1	5	0.0204	19.9796	0.10	1.340	363	5.41E-02	273	3.10E-01
2	8	0.1601	19.8399	0.80	1.323	250	7.76E-02	1011	8.25E-02
3	15	0.3524	19.6476	1.76	1.301	303	6.26E-02	859	9.53E-02
4	30	0.5881	19.4119	2.94	1.273	283	6.53E-02	455	1.76E-01
5	60	0.9326	19.0674	4.66	1.233	296	6.01E-02	406	1.90E-01
6	100	1.3409	18.6591	6.70	1.185	77	2.24E-01	1408	5.24E-02
7	140	1.8181	18.1819	9.09	1.129	334	4.93E-02	12787	5.48E-03
8	240	2.7063	17.2937	13.53	1.025	146	1.02E-01	1034	6.13E-02
9	480	3.9853	16.0147	19.93	0.875	105	1.21E-01	536	1.01E-01
10	960	4.9333	15.0667	24.67	0.764	92	1.20E-01	245	1.97E-01
11	800	5.3142	14.6858	26.57	0.720				
12	200	5.1025	14.8975	25.51	0.745				
13	50	4.8579	15.1421	24.29	0.773				
14	13	4.7321	15.2679	23.66	0.788				
15									
16									
17									
18									
19									



**Project**  
**Project No.**  
**Borehole No.**  
**Sample No.**  
**Sample Depth**

**Thurber Engineering, File# 10870**  
**122410864**  
**BH 305**  
**ST 10**  
**25-27 ft**



**One-Dimensional Consolidation Test using Incremental Loading**  
**ASTM D2435/D2435M - 11**

16-Aug-16  
16-Aug-16

Date: Date:

D. Boateng  
R. Hache

Checked by:  
Approved by:

**Specimen Details**

Project Name	Thurber Engineering, File# 10870
Project Location	ON
Borehole	BH 305
Sample No.	ST 10
Depth	25-27 ft
Sample Date	July 18, 2016
Test Number	One
Technician Name	Daniel Boateng

**Soil Description & Classification**

Silty Clay	
Specific Gravity of Solids	2.730
Average water content of trimmings %	51
<b>Additional Notes (information source, occurrence and size of large isolated particles etc.)</b>	

**Initial Specimen Conditions**

Height	mm	19.01
Diameter	mm	50.02
Area	mm <sup>2</sup>	1965
Volume	mm <sup>3</sup>	37356
Mass	g	64.79
Dry Mass	g	43.01
Density	Mg/m <sup>3</sup>	1.734
Dry Density	Mg/m <sup>3</sup>	1.151
Water Content	%	50.64
Degree of Saturation	%	100.0
Height of Solids	mm	8.02
Initial Void Ratio		1.371

**Final Specimen Conditions**

Water Content	%	37.31
Final Void Ratio		0.881

## One-Dimensional Consolidation Test using Incremental Loading

### ASTM D2435/D2435M - 11

**Specimen Details**

Project Name	Thurber Engineering, File# 10870
Project Location	ON
Borehole	BH 305
Sample No.	ST 10
Depth	25-27 ft
Sample Date	July 18, 2016
Test Number	One
Technician Name	Daniel Boateng

**Test Procedure**

Date Started	July 26, 2016
Date Finished	August 11, 2016
Machine Number	Manual Frame A
Cell Number	A
Ring Number	A
Trimming Procedure	Turntable
Moisture Condition	Inundated
Axial Stress at Inundation kPa	5
Water Used	Distilled
Test Method	A
Interpretation Procedure for $c_v$	2

**All Departures from Outlined ASTM D2435/D2435M-11 Procedure**

--

**Calculations**

Load Increment	Increment Duration min	Axial Stress $\sigma_a$ kPa	Corrected Deformation $\Delta H$ mm	Specimen Height H mm	Axial Strain $\epsilon_a$ %	Void Ratio e
Seating	0.0	5	0.0000	19.0100	0.00	1.371
1	1440.0	5	0.0260	18.9840	0.14	1.368
2	1440.0	11	0.0650	18.9450	0.34	1.363
3	1440.0	20	0.1330	18.8770	0.70	1.355
4	1440.0	43	0.3130	18.6970	1.65	1.332
5	1440.0	60	0.4100	18.6000	2.16	1.320
6	1440.0	84	0.5830	18.4270	3.07	1.298
7	1440.0	119	0.8630	18.1470	4.54	1.263
8	1440.0	160	1.4070	17.6030	7.40	1.196
9	1440.0	321	3.5100	15.5000	18.46	0.933
10	1440.0	640	4.7330	14.2770	24.90	0.781
11	1440.0	160	4.5530	14.4570	23.95	0.803
12	1440.0	43	4.2920	14.7180	22.58	0.836
13	1440.0	11	4.0150	14.9950	21.12	0.870
14	1440.0	5	3.9280	15.0820	20.66	0.881

## One-Dimensional Consolidation Test using Incremental Loading

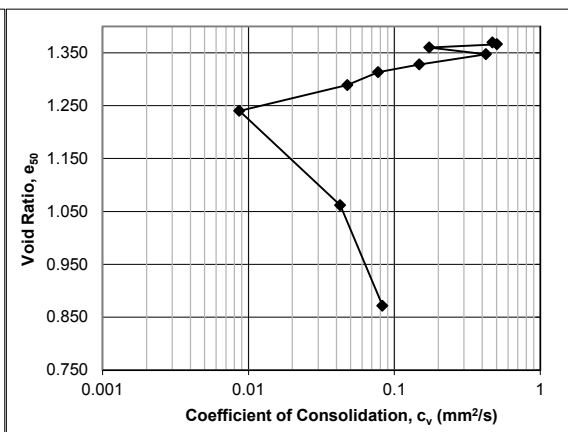
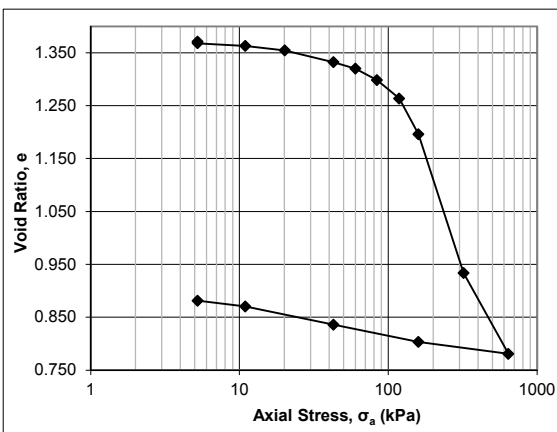
### ASTM D2435/D2435M - 11

**Specimen Details**

Project Name	Thurber Engineering, File# 10870
Project Location	ON
Borehole	BH 305
Sample No.	ST 10
Depth	25-27 ft
Sample Date	July 18, 2016
Test Number	One
Technician Name	Daniel Boateng

**Calculations**

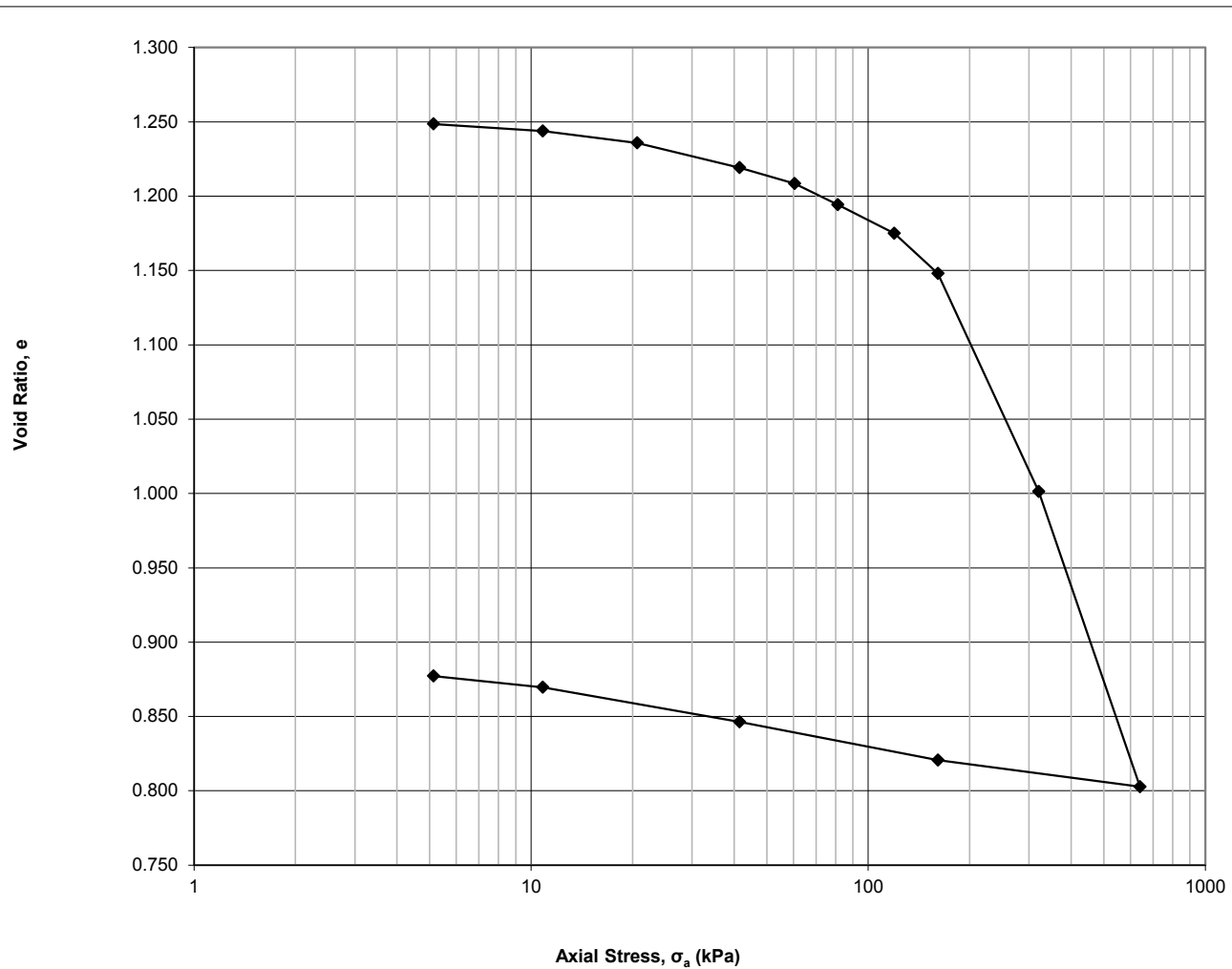
Load Increment	Axial Stress $\sigma_a$ , average kPa	Calculated using Interpretation Procedure 2				Interpretation Procedure 1		Interpretation Procedure 2	
		Corrected Deformation $\Delta H_{50}$ mm	Specimen Height $H_{50}$ mm	Axial Strain $\epsilon_{a,50}$ %	Void Ratio $e_{50}$	Time $t_{50}$ sec	Coeff. Consol. $c_v$ mm <sup>2</sup> /s	Time $t_{90}$ sec	Coeff. Consol. $c_v$ mm <sup>2</sup> /s
Seating	3								
1	5	0.0129	18.9971	0.07	1.370			163	4.71E-01
2	8	0.0412	18.9688	0.22	1.366			151	5.05E-01
3	16	0.0904	18.9196	0.48	1.360			437	1.74E-01
4	32	0.1900	18.8200	1.00	1.347			177	4.25E-01
5	52	0.3456	18.6644	1.82	1.328			497	1.49E-01
6	72	0.4651	18.5449	2.45	1.313			943	7.73E-02
7	101	0.6586	18.3514	3.46	1.289			1498	4.77E-02
8	139	1.0497	17.9603	5.52	1.240			7901	8.66E-03
9	240	2.4805	16.5295	13.05	1.062			1363	4.25E-02
10	481	4.0048	15.0052	21.07	0.872			577	8.28E-02
11	400	4.6021	14.4079	24.21	0.797				
12	101	4.4126	14.5974	23.21	0.821				
13	27	4.1654	14.8446	21.91	0.852				
14	8	3.9931	15.0169	21.01	0.873				





**Project**  
**Project No.**  
**Borehole No.**  
**Sample No.**  
**Sample Depth**

**Thurber Engineering, File# 10870**  
**122410864**  
**BH 305**  
**ST 14**  
**45-47 ft**



**One-Dimensional Consolidation Test using Incremental Loading**  
**ASTM D2435/D2435M - 11**

16-Aug-16  
16-Aug-16

Date: Date:

D. Boateng  
R. Hache

Checked by:  
Approved by:

**Specimen Details**

Project Name	Thurber Engineering, File# 10870
Project Location	ON
Borehole	BH 305
Sample No.	ST 14
Depth	45-47 ft
Sample Date	July 18, 2016
Test Number	Two
Technician Name	Daniel Boateng

**Soil Description & Classification**

Silty Clay	
Specific Gravity of Solids	2.730
Average water content of trimmings %	45
<b>Additional Notes (information source, occurrence and size of large isolated particles etc.)</b>	

**Initial Specimen Conditions**

Height	mm	19.03
Diameter	mm	50.86
Area	mm <sup>2</sup>	2032
Volume	mm <sup>3</sup>	38662
Mass	g	68.09
Dry Mass	g	46.87
Density	Mg/m <sup>3</sup>	1.761
Dry Density	Mg/m <sup>3</sup>	1.212
Water Content	%	45.27
Degree of Saturation	%	98.7
Height of Solids	mm	8.45
Initial Void Ratio		1.252

**Final Specimen Conditions**

Water Content	%	33.38
Final Void Ratio		0.877

## One-Dimensional Consolidation Test using Incremental Loading

### ASTM D2435/D2435M - 11

**Specimen Details**

Project Name	Thurber Engineering, File# 10870
Project Location	ON
Borehole	BH 305
Sample No.	ST 14
Depth	45-47 ft
Sample Date	July 18, 2016
Test Number	Two
Technician Name	Daniel Boateng

**Test Procedure**

Date Started	July 26, 2016
Date Finished	August 26, 2016
Machine Number	Manual Frame B
Cell Number	B
Ring Number	B
Trimming Procedure	Turntable
Moisture Condition	Inundated
Axial Stress at Inundation kPa	5
Water Used	Distilled
Test Method	B
Interpretation Procedure for $c_v$	2

**All Departures from Outlined ASTM D2435/D2435M-11 Procedure**

--

**Calculations**

Load Increment	Increment Duration min	Axial Stress $\sigma_a$ kPa	Corrected Deformation $\Delta H$ mm	Specimen Height H mm	Axial Strain $\epsilon_a$ %	Void Ratio e
Seating	0.0	5	0.0000	19.0300	0.00	1.252
1	1440.0	5	0.0280	19.0020	0.15	1.249
2	1440.0	11	0.0690	18.9610	0.36	1.244
3	1440.0	21	0.1360	18.8940	0.71	1.236
4	1440.0	42	0.2770	18.7530	1.46	1.219
5	1440.0	61	0.3670	18.6630	1.93	1.208
6	1440.0	81	0.4880	18.5420	2.56	1.194
7	1440.0	120	0.6500	18.3800	3.42	1.175
8	1440.0	161	0.8780	18.1520	4.61	1.148
9	1440.0	320	2.1180	16.9120	11.13	1.001
10	1440.0	640	3.7960	15.2340	19.95	0.803
11	1440.0	161	3.6450	15.3850	19.15	0.821
12	1440.0	42	3.4280	15.6020	18.01	0.846
13	1440.0	11	3.2310	15.7990	16.98	0.870
14	1440.0	5	3.1670	15.8630	16.64	0.877

## One-Dimensional Consolidation Test using Incremental Loading

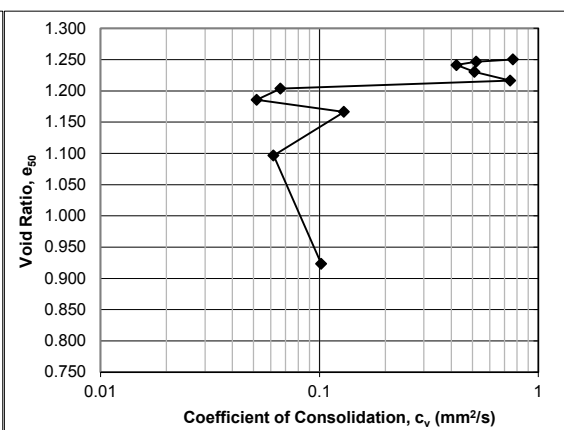
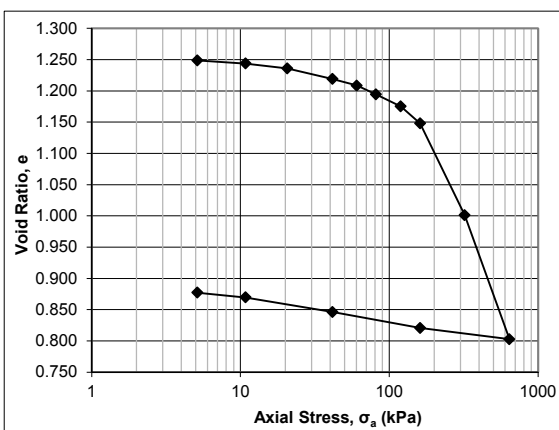
### ASTM D2435/D2435M - 11

**Specimen Details**

Project Name	Thurber Engineering, File# 10870
Project Location	ON
Borehole	BH 305
Sample No.	ST 14
Depth	45-47 ft
Sample Date	July 18, 2016
Test Number	Two
Technician Name	Daniel Boateng

**Calculations**

Load Increment	Axial Stress $\sigma_a$ , average kPa	Calculated using Interpretation Procedure 2				Interpretation Procedure 1		Interpretation Procedure 2	
		Corrected Deformation $\Delta H_{50}$ mm	Specimen Height $H_{50}$ mm	Axial Strain $\epsilon_{a,50}$ %	Void Ratio $e_{50}$	Time $t_{50}$ sec	Coeff. Consol. $c_v$ mm <sup>2</sup> /s	Time $t_{90}$ sec	Coeff. Consol. $c_v$ mm <sup>2</sup> /s
Seating	3								
1	5	0.0130	19.0170	0.07	1.250			100	7.67E-01
2	8	0.0432	18.9868	0.23	1.247			147	5.21E-01
3	16	0.0903	18.9397	0.47	1.241			180	4.23E-01
4	31	0.1820	18.8480	0.96	1.230			147	5.11E-01
5	51	0.3007	18.7293	1.58	1.216			100	7.45E-01
6	71	0.4074	18.6226	2.14	1.204			1107	6.64E-02
7	100	0.5615	18.4685	2.95	1.185			1399	5.17E-02
8	140	0.7223	18.3077	3.80	1.166			549	1.29E-01
9	241	1.3133	17.7167	6.90	1.096			1077	6.18E-02
10	480	2.7768	16.2532	14.59	0.923			549	1.02E-01
11	401	3.6843	15.3457	19.36	0.816				
12	101	3.5241	15.5059	18.52	0.835				
13	26	3.3406	15.6894	17.55	0.857				
14	8	3.2148	15.8152	16.89	0.871				



**Certificate of Analysis**
**Client: Thurber Engineering Ltd.**
**Client PO:**
**Report Date: 01-Mar-2016**
**Order Date: 25-Feb-2016**
**Project Description: 10870**

<b>Client ID:</b>	McLeod Rd BH302	Centreline Rd	-	-
	SS3 5-7	BH202 SS5 10-12	-	-
<b>Sample Date:</b>	14-Feb-16	17-Feb-16	-	-
<b>Sample ID:</b>	1609281-01	1609281-02	-	-
<b>MDL/Units</b>	Soil	Soil	-	-

**Physical Characteristics**

% Solids	0.1 % by Wt.	73.8	72.3	-	-
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**General Inorganics**

pH	0.05 pH Units	7.68	7.65	-	-
Resistivity	0.10 Ohm.m	39.5	24.1	-	-

**Anions**

Chloride	5 ug/g dry	69	201	-	-
Sulphate	5 ug/g dry	31	16	-	-

Certificate of Analysis  
**Client:** Thurber Engineering Ltd.  
**Client PO:**

Report Date: 16-Aug-2016

Order Date: 10-Aug-2016

**Project Description:**

<b>Client ID:</b>	307 SS1	305 SS4	-	-
<b>Sample Date:</b>	10-Aug-16	10-Aug-16	-	-
<b>Sample ID:</b>	1633200-01	1633200-02	-	-
<b>MDL/Units</b>	Soil	Soil	-	-

**Physical Characteristics**

% Solids	0.1 % by Wt.	64.6	80.1	-	-
----------	--------------	------	------	---	---

**General Inorganics**

pH	0.05 pH Units	7.71	5.08	-	-
Resistivity	0.10 Ohm.m	50.3	281	-	-

**Anions**

Chloride	5 ug/g dry	16	9	-	-
Sulphate	5 ug/g dry	37	14	-	-

## **APPENDIX D**

### **SELECT PHOTOGRAPHS OF THE BRIDGE LOCATION**





**Figure 1: Looking southwest towards the existing McLeod Road Bridge**



**Figure 2: Looking north along the proposed alignment from south abutment**





**Figure 3: Looking south along the proposed alignment from north abutment**



**Figure 4: Looking west across the proposed location of the north abutment towards the existing McLeod Road Bridge**





**Figure 5: Looking south towards location of Borehole 307 (south abutment)**



**Figure 6: Looking upstream from the McLeod Road Bridge**



**Figure 7: Looking downstream from Borehole 306**

**APPENDIX E**  
**COMPARISON OF FOUNDATION OPTIONS**

### Comparison of Foundation Alternatives

Comment	Friction Steel H-Piles	Spread Footings on Native Soils	Spread Footing Placed on Engineered Fill Pads and Preloading
<b>Advantages:</b>	<p>Quick installation procedure</p> <p>Requires less excavation than spread footings on native soil</p>	N/A	<p>Quick installation and construction</p> <p>Does not require specialized piling equipment</p>
<b>Disadvantages:</b>	<p>High cost for installation</p> <p>Downdrag loads due to the settlement of the clay layer will require additional piles to be installed</p> <p>May be difficult to get appropriate construction equipment across existing bridge</p>	<p>Low strength and highly compressible nature of native clay would require abutment foundations to be designed at a comparatively low geotechnical resistance</p> <p>Requires large excavation and footings</p>	<p>Preloading of the north abutment would require additional construction time and cost</p> <p>Future ongoing bridge maintenance and jacking up of the abutments to compensate for the consolidation settlements</p>
<b>Relative Cost</b>	<b>High</b>	<b>Moderate</b>	<b>Moderate</b>
	<b>Feasible</b>	<b>NOT RECOMMENDED</b>	<b>Recommended</b>

## **APPENDIX F**

### **GSC SEISMIC HAZARD CALCULATION L-PILE ANALYSIS FOR HP 310X110 STEEL PILES**



# 2015 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836  
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

September 19, 2016

Site: 46.4385 N, 83.8207 W User File Reference: McLeod Road Bridge

Requested by: , Thurber Engineering Ltd.

**National Building Code ground motions: 2% probability of exceedance in 50 years (0.000404 per annum)**

Sa(0.05)	Sa(0.1)	<b>Sa(0.2)</b>	Sa(0.3)	<b>Sa(0.5)</b>	<b>Sa(1.0)</b>	<b>Sa(2.0)</b>	<b>Sa(5.0)</b>	<b>Sa(10.0)</b>	<b>PGA (g)</b>	<b>PGV (m/s)</b>
0.047	0.066	<b>0.064</b>	0.055	<b>0.047</b>	<b>0.029</b>	<b>0.015</b>	<b>0.0036</b>	<b>0.0016</b>	<b>0.037</b>	<b>0.036</b>

**Notes.** Spectral ( $S_a(T)$ , where  $T$  is the period in seconds) and peak ground acceleration (PGA) values are given in units of  $g$  ( $9.81 \text{ m/s}^2$ ). Peak ground velocity is given in  $\text{m/s}$ . Values are for "firm ground" (NBCC 2015 Site Class C, average shear wave velocity  $450 \text{ m/s}$ ). NBCC2015 and CSAS6-14 values are specified in **bold font**. Three additional periods are provided - their use is discussed in the NBCC2015 Commentary. Only 2 significant figures are to be used. *These values have been interpolated from a 10-km-spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the directly calculated values.*

Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.05)	0.0043	0.015	0.025
Sa(0.1)	0.0072	0.023	0.037
Sa(0.2)	0.0089	0.026	0.039
Sa(0.3)	0.0083	0.024	0.036
Sa(0.5)	0.0065	0.021	0.031
Sa(1.0)	0.0034	0.012	0.019
Sa(2.0)	0.0013	0.0054	0.0093
Sa(5.0)	0.0004	0.0012	0.0020
Sa(10.0)	0.0003	0.0007	0.0010
PGA	0.0041	0.013	0.021
PGV	0.0037	0.014	0.022

## References

**National Building Code of Canada 2015 NRCC no. 58190;**  
**Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada**

**User's Guide - NBC 2015, Structural Commentaries NRCC no. xxxxxx (in preparation)**  
**Commentary J: Design for Seismic Effects**

**Geological Survey of Canada Open File 7893** Fifth Generation Seismic Hazard Model for Canada: Grid values of mean hazard to be used with the 2015 National Building Code of Canada

See the websites [www.EarthquakesCanada.ca](http://www.EarthquakesCanada.ca) and [www.nationalbuildingcode.ca](http://www.nationalbuildingcode.ca) for more information

*Aussi disponible en français*



Natural Resources  
Canada

Ressources naturelles  
Canada

Canada

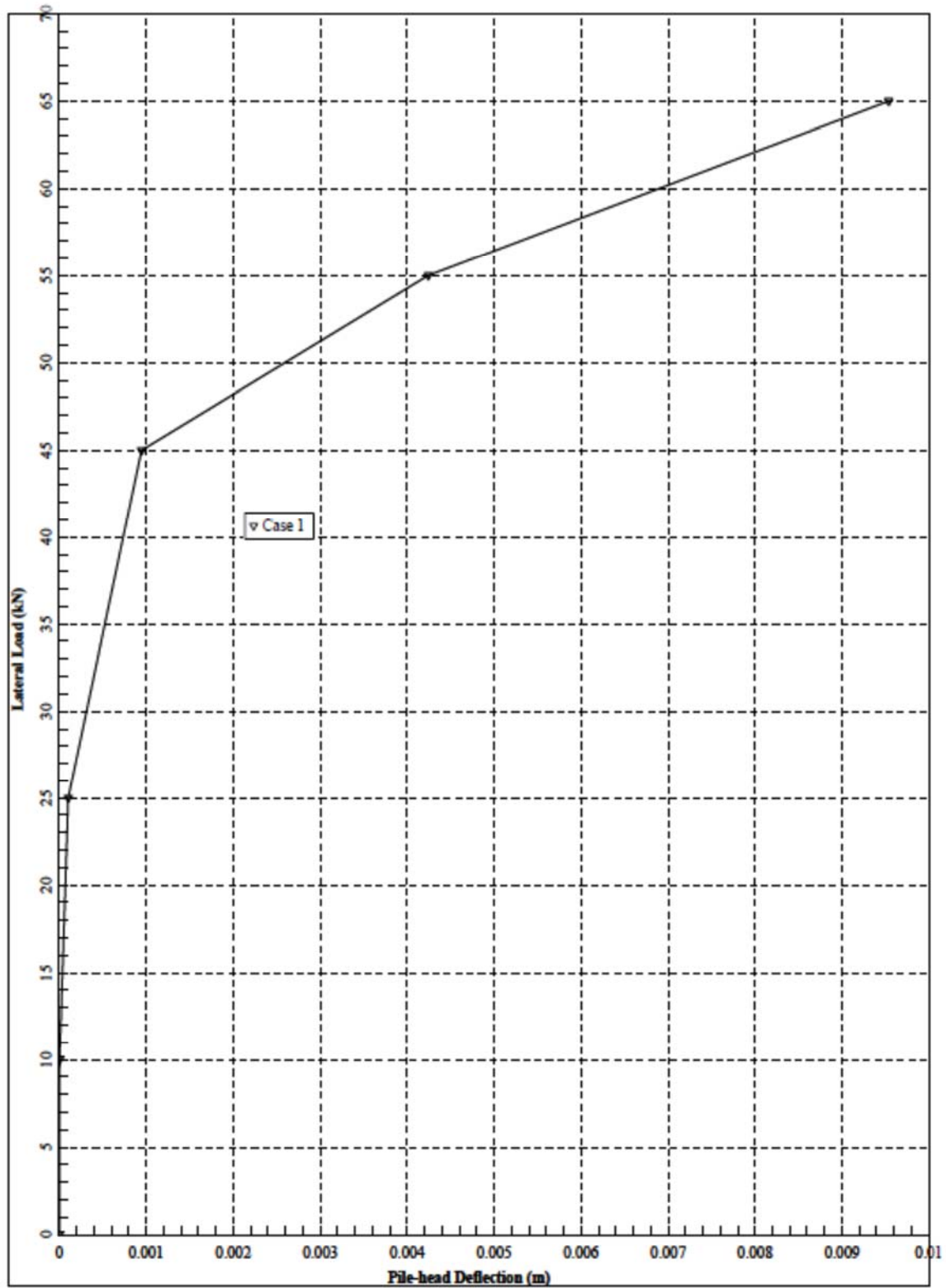


Figure 8: Lateral Load vs. Pile-head Deflection for the North Embankment (Borehole 305)



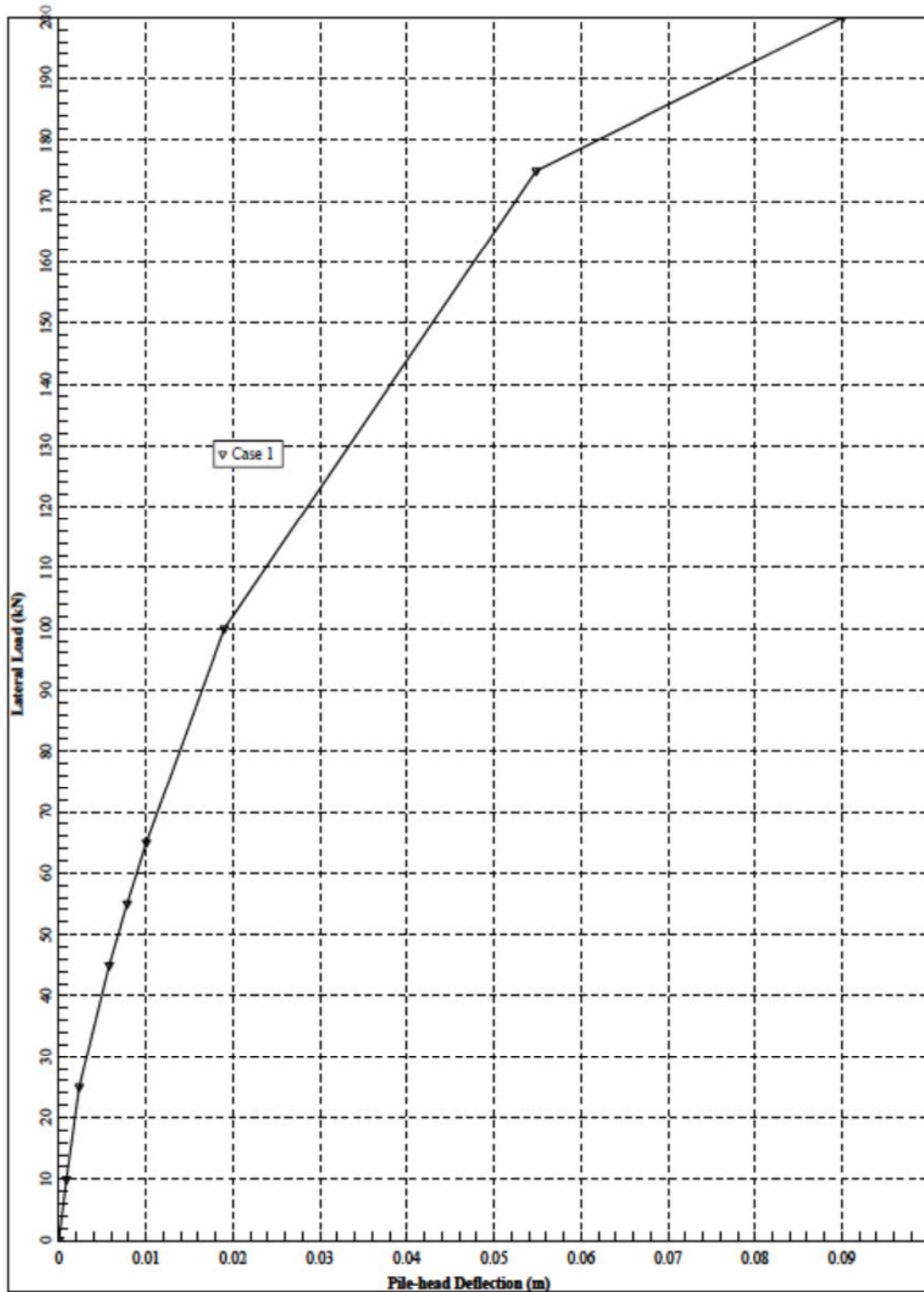


Figure 9: Lateral Load vs. Pile-head Deflection for the South Embankment (Borehole 307)

## **APPENDIX G**

**LIST OF REFERENCED SPECIFICATIONS**

**NOTICE TO CONTRACTOR**

**OPERATIONAL CONSTRAINTS**

**SPECIAL PROVISION – GEOTECHNICAL ASSESSMENT**

**NON-STANDARD SPECIAL PROVISION – SETTLEMENT MONITORING**

## **LIST OF REFERENCED SPECIFICATIONS**

OPSD 208.010	Benching of Earth Slopes
OPSD 3090.100	Foundation Frost Penetration Depths for Northern Ontario
OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 501	Construction Specification for Compacting
OPSS 804	Construction Specification for Seed and Cover
OPSS 805	Construction Specification for Temporary Erosion and Sediment Control Measures
OPSS 902	Construction Specification for Excavating and Backfilling-Structures
OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS.PROV 1010	Material Specification for Aggregates - Base, Subbase, Select Subgrade, and Backfill Material
OPSS.PROV 1860	Material Specifications for Geotextile
NSSP FOUN003	Dewater Structure Excavations
SP 517F01	Design Storm Return Period and Preconstruction Survey
SP 109S12	QVE, Backfilling, Compaction and Certificate of Conformance

## **RECOMMENDED WORDING FOR NOTICE TO CONTRACTOR - BOULDERS**

The Contractor is hereby notified that the soils at this site include rockfill and granular fill that may contain cobbles and boulders, which could affect excavations and the installation of temporary protection systems. Consideration of the presence of these obstructions shall be made in selection of appropriate equipment and procedures for excavation and installation of the foundations and temporary protection systems.

## **RECOMMENDED WORDING FOR OPERATIONAL CONSTRAINT - PRELOADING**

The Contractor is notified that the contract includes a requirement for preloading of the ground in the area of the north and south abutments and approaches, and for settlement monitoring of the north abutment and approach area.

The Contractor shall construct the engineered fill pads beneath the abutment footings as specified elsewhere in the contract and then place OPSS Granular A material up to the proposed final grades prior to installation of the settlement monitoring pins and start of the preload period.

A preload period of six months is anticipated, however, the end of the preload period will be determined by the Contract Administrator based on the results of the settlement monitoring program. Construction of the bridge foundations shall not proceed until the Contract Administrator provided written notification that the preload period has been completed.

## **GEOTECHNICAL ASSESSMENT - Item No.**

---

Special Provision

---

### **TABLE OF CONTENTS**

- 1.0 SCOPE**
- 2.0 REFERENCES**
- 3.0 DEFINITIONS - Not Used**
- 4.0 DESIGN AND SUBMISSION REQUIREMENTS**
- 5.0 MATERIALS - Not Used**
- 6.0 EQUIPMENT - Not Used**
- 7.0 CONSTRUCTION - Not Used**
- 8.0 QUALITY ASSURANCE - Not Used**
- 9.0 MEASUREMENT FOR PAYMENT - Not Used**
- 10.0 BASIS OF PAYMENT**

#### **1.0 SCOPE**

The use of heavy construction equipment and in particular heavy lifting cranes may be required during removal of the existing bridge and erection of the new bridge. The impact of the heavy equipment loads on the underlying soils, river valley slopes and existing bridge foundations must be considered during selection of the methodology and equipment employed for construction.

#### **2.0 REFERENCES**

Foundation Investigation Report, Site 38S-154, McLeod Road Bridge Replacement, Sault Ste Marie District, Algoma County, Ontario, G.W.P. 5148-13-00, Geocres No. 41J-103), by Thurber Engineering Ltd., dated April 2019.

#### **4.0 DESIGN AND SUBMISSION REQUIREMENTS**

##### **4.1 Design Requirements**

Prior to commencement of construction, the Contractor shall retain a Geotechnical Consultant to assess the impact of the proposed equipment loads and construction methodology and determine requirements and/or restrictions necessary to safely support the loads without a foundation or slope failure. All Foundation Engineering services required for this project shall be performed by consultant(s) listed as accepted under the MTO's RAQS for providing services under the specialty of Geotechnical (Structures and Embankments) – Medium Complexity.

The assessment shall include, but not be limited to, the following:

- Review of available geotechnical information and supplementing with additional subsurface information as required in the equipment pad/access road areas;
- Determining appropriate setback distances for heavy equipment from the existing and new bridge abutments/piers and their foundations, and from the crests of the river valley slopes and existing/new embankment side slopes;
- Determining permissible ground pressure that may be applied to the foundation soils by the equipment, such as through a combination of crane pad design and sub-excavation;
- Providing recommendations for distribution of equipment loads to limit the lateral deflections of foundation piles of the existing and new bridges;
- If use of a crane pad and/or sub-excavation is not feasible, an alternative pile-supported platform system may be considered. The Contractor shall provide recommendations for crane pad design to transfer the crane loads for lifting girders to the ground during construction of the new bridge or demolition of the existing bridge through the alternative pile-supported platform system, if necessary.

#### **4.2 Submission Requirements**

At least two (2) weeks prior to mobilization of heavy construction equipment to the site, the Contractor shall submit a report detailing the findings of the geotechnical assessment to the Contract Administrator. The report shall be signed and sealed by the Geotechnical Consultant and provide the following, as a minimum:

- Appropriate setback distances for heavy equipment from existing/new structures and river valley slopes;
- Permissible ground pressures which may be applied to the foundation soils by heavy equipment;
- Recommendations for distribution of equipment loads to limit lateral deflections of existing and new foundation piles;
- Recommendations for pile-supported platform systems to support heavy equipment, if required.

#### **10.0 BASIS OF PAYMENT**

Payment at the Contract price for the above tender items shall be full compensation for all labour to do the work.

Payment for costs associated with heavy construction equipment necessary to complete the work, such as design and construction of temporary works, supply, mobilization/de-mobilization, and operation shall be made under the associated items.

## **SUPPLY AND INSTALLATION OF SETTLEMENT MONITORING EQUIPMENT**

Item No.

---

Special Provision

---

### **1.0 SCOPE**

This special provision describes requirements to supply and install eight (8) settlement pins for monitoring the settlement of the preload fill at the north abutment and approach of the proposed McLeod Road Bridge structure.

### **2.0 REFERENCES – Not Used**

### **3.0 DEFINITIONS – Not Used**

### **4.0 DESIGN AND SUBMISSION REQUIREMENTS**

The Contractor shall submit details of the proposed installation methods for the instruments, including the installation schedule and details of the survey monuments/benchmarks to be used, to the CA, a minimum of ten (10) working days before the start of instrument installation.

### **5.0 MATERIALS**

#### **5.1 Concrete**

The Contractor shall supply concrete (OPSS 1350) with strength and set time sufficient to secure the settlement pin within two (2) days of placing.

#### **5.1 Steel Pin**

The Contractor shall supply a 25.4 mm minimum diameter reinforcing steel bar (OPSS 905) cut to a length of 0.4 m.

The top of the reinforcing steel bar shall be angled or rounded in such a way that a single survey point can be clearly identified and repeated.

### **6.0 EQUIPMENT – Not Used**

### **7.0 CONSTRUCTION**

#### **7.1 Installation**

Settlement pins are to be installed immediately after the granular pad at each abutment has been constructed and the embankment has been constructed to the preload grade with OPSS Granular A material prior to start of the waiting period.

The Contractor shall install the settlement pins at the locations specified in the Contract Drawings.

## **7.2 Protection of Instruments**

All instruments shall be adequately protected by the Contractor such that they are not damaged during construction or during the winter wait period. Any instrument damaged by the Contractor's work shall be immediately replaced at the Contractor's cost.

## **7.2 Coordination with Monitoring**

The Contractor shall notify the Contract Administrator no later than two days after the settlement pins have been installed. At this time, the Contractor shall also supply the following information:

- Settlement pin location, easting, northing;
- Elevation of top of pin;
- Dates of installation and datum readings; and
- Installation notes, sketches and photographs.

Monitoring of the settlement pins shall be carried out by others.

## **7.3 Decommissioning of Instruments**

The instrumentation shall not be decommissioned until instructed by the Contract Administrator after discussion with and concurrence from MTO.

The Contractor shall decommission the settlement pins as directed by the Contract Administrator.

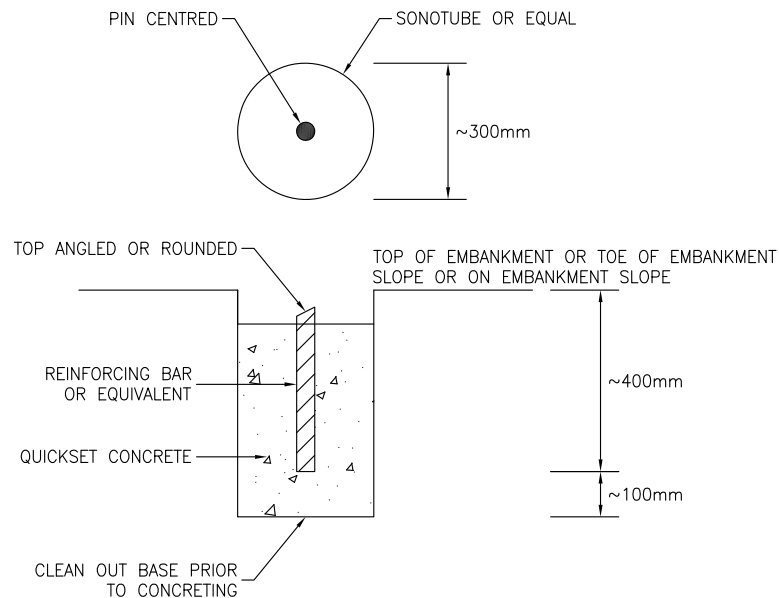
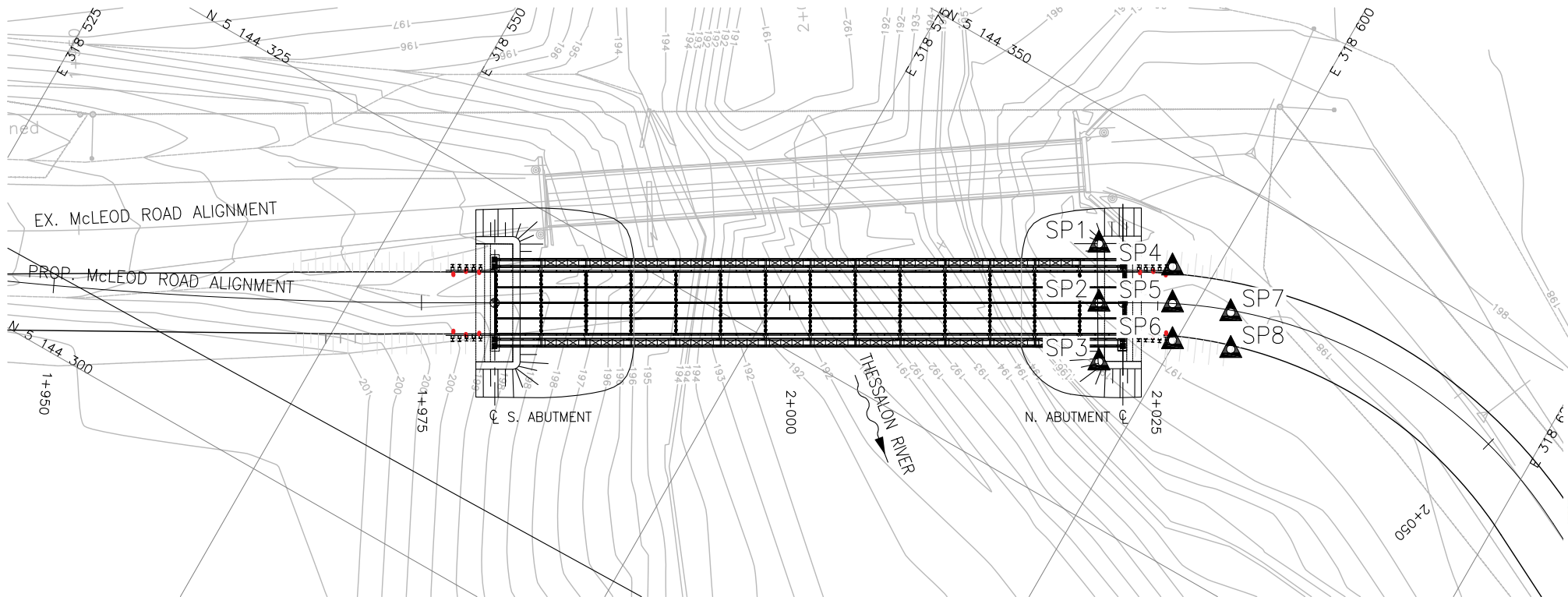
## **8.0 QUALITY ASSURANCE – Not Used**

## **9.0 MEASUREMENT FOR PAYMENT – Not Used**

## **10.0 BASIS OF PAYMENT**

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.





SETTLEMENT PIN (SP) DETAIL  
N.T.S.

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



CONT No  
GWP No 5148-13-00

McLEOD ROAD  
BRIDGE REPLACEMENT  
SETTLEMENT PIN LOCATION PLAN

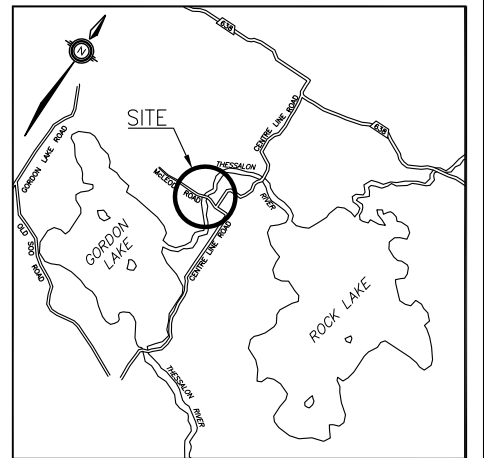


SHEET

McINTOSH PERRY



THURBER ENGINEERING LTD.



KEYPLAN

LEGEND

▲ Settlement Pin

NO	NORTHING	EASTING
SP1	5 144 341.8	318 592.2
SP2	5 144 338.3	318 594.2
SP3	5 144 334.8	318 596.2
SP4	5 144 342.9	318 597.3
SP5	5 144 340.8	318 598.6
SP6	5 144 338.6	318 599.8
SP7	5 144 342.2	318 602.3
SP8	5 144 340.0	318 603.5

-NOTE-

Settlement Pin locations are shown in MTM Zone 13 coordinates.

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	KP	CHK PKC	CODE
DRAWN	MFA	CHK KP	SITE 38S-154 STRUCT DWG 1