



THURBER ENGINEERING LTD.

**FOUNDATION INVESTIGATION AND DESIGN REPORT
SITE 38S-126 – CENTRE LINE ROAD BRIDGE REPLACEMENT
SAULT STE. MARIE DISTRICT, ALGOMA COUNTY
G.W.P. 5149-13-00
ASSIGNMENT NUMBER: 5014-E-0032**

GEOCRES NUMBER: - 41J-101

**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 Centre Line 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 culvert 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-126 is located on Centre Line Road, 2.2 km south of the Highway 638 Junction with Centre Line Road in Leeburn, Ontario. The location of the structure is shown on the inset Key Plan on Drawing No. 1 in Appendix A.

The existing three-span, Pony truss structure carries a single lane of traffic over the Thessalon River. The bridge is approximately 46 m long, 4.0 m wide with approximate span lengths of 14 m, 18 m, 14 m. It is noted that for project orientation purposes, Centre Line Road, will be assumed to run north-south and the Thessalon River flow is from east to west.

Centre Line Road at this location has one lane in each direction with a rural cross-section and gravel shoulders. The lands surrounding the project limits are typically agricultural with some residential properties. Storm water drainage in the area is to existing ditches and culverts. 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 Borehole Location and Soil Strata Drawing No. 1 provided in Appendix A. The field investigation for this site included advancing four boreholes drilled between February 15, 2016 and February 17, 2016. The locations and elevations of the boreholes are shown Drawing No. 1 and are summarized in Table 3-1.

Table 3-1: Borehole Summary

Borehole	Location	Northing (m)	Easting (m)	Ground Surface Elevation (m)	Depth (m)
201	North approach	5145430.8	319419.1	200.7	6.7
202	North abutment	5145424.3	319418.9	200.4	31.1
203	South abutment	5145372.3	319409.6	200.3	31.1
204	South approach	5145365.7	319407.2	200.1	6.7

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 a CME truck mounted drill rig equipped with hollow stem augers and NW casing. 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 firm to very soft 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.

A 25 mm inside diameter PVC piezometer was installed in Borehole 203 to allow for measurement of the groundwater level at the site. Piezometer construction details are illustrated on the Record of Borehole sheet for Borehole 203, provided in Appendix B.

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 paved areas were capped with 300 mm of cold patch asphalt.

The as-drilled locations of the boreholes and ground surface elevations at the borehole locations were surveyed by Thurber on February 16, 2016. The vertical datum used was a temporary benchmark (TBM) provided by MPCE, located at Station 2+035.2. The TBM has a geodetic elevation of 201.337 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.

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 the Borehole Location and Soil Strata Drawing 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 in the area is characterized by an asphaltic surface, overlying sand with silt and gravel fill, overlying a clay fill material overlying clay, overlying a sandy silt underlain by a granular glacial till. The fill at the south abutment included some woody pieces between a depth of 2.3 and 3.8 m.

More detailed descriptions of the individual strata are presented below.

4.2 Granular Fill

All boreholes were advanced through Centre Line Road.

A granular fill layer consisting predominantly of sand and gravel with varying amounts of silt was encountered below the asphaltic surface. The top of this layer ranges from Elevation 200.6 m to Elevation 200.0 m and has a thickness ranging from 500 mm to 700 mm. At the time of the field investigation frost had penetrated to a depth of up to 2.0 m as such only auger samples could be obtained for the pavement structure.

The moisture content of the samples tested ranged from 3% to 4%. The results of grain size analysis conducted on two samples of this material are summarized in Table 4-1 and are illustrated on Figure 1 in Appendix C.

Table 4-1: Gradation Results for Granular Fill

Soil Particles	%
Gravel	21 to 25
Sand	55 to 62
Silt and Clay	17 to 20

4.3 Clay Fill

A clay fill layer with varying amounts of sand and gravel was encountered beneath the granular fill in all boreholes. The top of this layer ranges from Elevation 200.1 m to Elevation 199.5 m and has a thickness ranging from 900 mm to 3.2 m. The SPT 'N' values ranged from 3 to 32 blows per 0.3 m of penetration indicating a soft to hard consistency. Some of the SPT 'N' values may be affected by the frozen condition of the fill.

The moisture content of the samples tested ranged from 24% to 42%. The results of grain size analysis conducted on samples this material are summarized in Table 4-2 and are illustrated on Figure 2 in Appendix C.

Table 4-2: Gradation Results for Clay Fill

Soil Particles	%
Gravel	0 to 22
Sand	8 to 35
Silt	44 to 50
Clay	21 to 31

The results of Atterberg Limits testing completed on samples of this material are summarized in Table 4-3 and are illustrated on Figures 7 and 8 in Appendix C.

Table 4-3: Atterberg Limits Test Results

Plastic Limit	15 to 23
Liquid Limit	28 to 41
Plasticity Index	13 to 18

4.4 Clay Crust

A brown native clay deposit was encountered beneath the clay fill materials in all boreholes except Borehole 203. The top of this layer ranges from Elevation 199.2 m to Elevation 198.1 m and has a thickness ranging from 1.2 m to 1.5 m. In-situ shear vane test results indicated undrained shear strengths ranging from 64 kPa to 72 kPa indicating a stiff consistency. The moisture content of the samples tested ranged from 29% to 39%. The results of grain size analysis conducted on samples of this material are summarized in Table 4-4 and are illustrated on Figure 3 in Appendix C.

Table 4-4: Gradation Results for Clay

Soil Particles	%
Gravel	0
Sand	0
Silt	47 and 60
Clay	40 and 53

The results of Atterberg Limits testing completed on samples of this material are summarized in Table 4-5 and are illustrated on Figures 7 and 8 in Appendix C.

Table 4-5: Atterberg Limits Test Results

Plastic Limit	20
Liquid Limit	49 and 52
Plasticity Index	29 and 32

4.5 Clay

A grey clay deposit was encountered beneath the clay crust material in Boreholes 201, 202 and 204 and beneath the clay fill in Borehole 203. Boreholes 201 and 204 were terminated in this strata.

The top of this layer ranges from Elevation 197.6 m to Elevation 195.5 m and has a thickness where completely penetrated ranging from 11.5 m to 14.8 m. In-situ shear vane test results indicated undrained shear strengths ranging from 25 kPa to 65 kPa; indicating a firm to stiff

consistency; but typically firm. The moisture content of the samples tested ranged from 21% to 59%. The results of grain size analysis conducted on samples of this material are summarized in Table 4-6 and are illustrated on Figure 4 in Appendix C.

Table 4-6: Gradation Results for Clay

Soil Particles	%
Gravel	0
Sand	0 to 3
Silt	30 to 59
Clay	38 to 70

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

Table 4-7: Atterberg Limits Test Results

Plastic Limit	16 and 20
Liquid Limit	40 and 58
Plasticity Index	12 and 42

The results of oedometer (one-dimensional consolidation) tests carried out on an undisturbed clay sample are summarized in Table 4-8. A copy of the oedometer test results is provided in Appendix C. The results of the tests indicate that the clay is slightly over-consolidated.

Table 4-8: Consolidation Test Results

Parameter	Value
Borehole	202
Sample	ST-1
Depth / Elevation (m) (mid-sample)	4.0 / 196.4
Moisture Content, (%)	53
Unit Weight, (γ) (kN/m ³)	16.5
Specific Gravity (G_s)	2.737
Initial Void Ratio (e_o)	1.491
Pre-consolidation Pressure, (kPa)	120
Compression Index (C_c)	0.803
Recompression Index (C_r)	0.09

4.6 Silt to Sandy Silt

A silt layer with varying amounts of sand was encountered beneath the clay strata in Boreholes 202 and 203. The top of this layer ranges from Elevation 184.1 m to Elevation 182.1 m and has a thickness ranging from 10.7 m to 12.8 m. The SPT 'N' values ranged from 5 to 16 blows per 0.3 m of penetration; indicating a loose to compact condition; but typically loose.

The moisture content of the samples tested ranged from 20% to 34%. The results of grain size analysis conducted on samples of this material are summarized in Table 4-9 and are illustrated on Figure 5 in Appendix C.

Table 4-9: Gradation Results for Silt

Soil Particles	%
Gravel	0
Sand	5 and 31
Silt	66 and 84
Clay	2 and 11

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

4.7 Glacial Till

A stratum of granular glacial till consisting predominantly of sand with silt and gravel and occasional cobbles was encountered in Boreholes 202 and 203. Boreholes 202 and 203 were terminated in this material. The top of this layer ranges from Elevation 171.4 m to Elevation 171.3 m. The SPT 'N' values ranged from 53 to 98 blows per 0.3 m of penetration; indicating a very dense condition.

The moisture contents of the samples tested were 11% and 17%. The results of grain size analysis tests on samples of this material are summarized in Table 4-10 and are illustrated on Figure 6 in Appendix C.

Table 4-10: Gradation Results for Glacial Till

Soil Particles	%
Gravel	4
Sand	67 and 89
Silt and Clay	7 and 29

4.8 Groundwater Conditions

The groundwater level in the piezometer installed in Borehole 203 was recorded on February 19, 2016, at a depth of 3.2 m; corresponding to Elevation 197.1 m.

The water level in the Thessalon River was indicated on the base plan at Elevation 194.2 m on February 10, 2016. The high water mark was indicated at elevation 195.0 m.

These observations are 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. 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 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.



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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 replacement of the Centre Line 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.

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 of the bridge structure will be carried out on the existing alignment with a full road closure while providing a traffic detour around the site during construction. The recommended bridge replacement is a single lane, single span, prefabricated, modular steel through truss, with a steel deck and an approximate length of 52 m. No grade raise is proposed for this project at either abutment. It is also understood that the single span option with a steel deck would result in abutment loads of approximately 1850 kN at ULS and 1240 kN at SLS.

6.2 Geotechnical Assessment

In general the stratigraphy in the area is characterized by an asphaltic surface, overlying sand with silt and gravel fill, overlying a clay fill material overlying clay, overlying a sandy silt underlain by a granular glacial till.

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 can 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 7-2: 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 (Ψ) 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 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 reference peak ground acceleration (PGA_{ref}).

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 soil stratigraphy was encountered at this site:

- | | | | |
|------------|--------------------|-------------------------|--------------|
| • Layer 1: | Thickness = 1.5 m | Typical S_u = 100 kPa | clay crust |
| • Layer 2: | Thickness = 2 m | Typical S_u = 50 kPa | clay |
| • Layer 3: | Thickness = 7 m | Typical S_u = 35 kPa | clay |
| • Layer 4: | Thickness = 5.5 m | Typical S_u = 45 kPa | clay |
| • Layer 5: | Thickness = 10.5 m | Typical SPT 'N' = 10 | silt |
| • Layer 6: | Thickness = 3.5 m | Typical SPT 'N' = 75 | glacial till |

The seismic site classification for this site is based on the S_u criteria. The harmonic mean of the typical S_u listed is 42 kPa which corresponds to a Seismic Site Class E in accordance with Table 4.1 of the CHBDC.

8.3 Seismic Liquefaction

The potential for liquefaction of the foundation soils has been assessed comparing Cyclic Stress Ratio to Cyclic Resistance Ratio generated from the SPT N-values. Using this method, the results indicate that an adequate Factor of Safety of 1.2 against liquefaction under earthquake loading exists for this site using the site specific PGA value.

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
- Driven H-piles

As indicated above the single span option with a steel deck would result in abutment loads of approximately 1850 kN at ULS and 1240 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 significant depth to a competent foundation stratum and the relatively high river water level, spread footings placed on native soils to support the abutments are not recommended for this site.

9.2 Spread Footing Placed on Engineered Fill Pads

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 May 2016, indicates the existing road grade at Elevation 200.3 m and Elevation 200.0 m at the north abutment and south abutments respectively. Also indicated on the GA drawing is the approximate underside of footing elevation of 198.0 m and the minimum soffit elevation of 199.455 m.

As part of the replacement works, the preliminary GA drawing illustrates new concrete slabs supporting the abutments. Since the superstructure length is to be approximately 7 m longer than the existing bridge the, new footings will likely be located behind the existing abutment foundations. It is noted that the bottom elevation of the existing abutment is not known.

The footings should be constructed on a granular engineered fill pad at least 1.5 m in thickness. The engineered fill should consist of OPSS Granular "A" placed in 150 mm lifts and compacted to 100% of its Standard Proctor Maximum Dry Density (SPMDD) at $\pm 2\%$ of the material's optimum moisture content. 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.

The following values of factored Geotechnical Resistance at ULS and Geotechnical Reaction may be used for design of footings approximately 3.0 m in width and placed at Elevation 198.0 m on a properly compacted granular fill as noted above:

- | | |
|---|---------|
| • Factored Geotechnical Resistance at ULS (kPa) | 100 kPa |
| • Factored Geotechnical Reaction at SLS (kPa) | 55 kPa. |

It should be noted that the SLS value provided above is for up to 50 mm of total settlement. Furthermore, future bridge maintenance and jacking up of the abutments to compensate for the consolidation settlements should be anticipated.

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

- Consequence factor (Ψ) 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 concrete and granular material should be evaluated using an unfactored coefficient of 0.55 for cast-in-place concrete.

9.3 Driven H-Piles

9.3.1 Friction 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.

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 Reaction at SLS (kN) per pile
North and South Abutment	20 / 178	350	275
North and South Abutment	25 / 173	450	350

The actual pile tip elevations may vary during installation and pile length should be controlled in accordance with OPSS 903.

9.3.2 Piles Advanced into the Glacial Till

It may be possible to advance the piles into the very dense glacial till (piles lengths in excess of 30 m would be required). If piles were founded in the very dense till, the factored geotechnical resistance at ULS would be 1,200 kN per pile and the factored geotechnical reaction at SLS would be approximately 1,000 kN per pile.

The actual pile tip elevations may vary during installation and pile length should be controlled in accordance with OPSS 903.

9.3.3 Pile Lateral Resistance

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 (Ψ) 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

Due to the relatively low foundation loading of the proposed structure, the ability of the single span structure to tolerate more than 25 mm of settlement at SLS and the reduced construction costs spread footings on engineered fill are considered a feasible and cost effective option for this project.

10 GENERAL RECOMMENDATIONS

Construction for the replacement of the Centre Line Road Bridge should be carried out in accordance with OPSS 902.

10.1 Excavations

It is anticipated that temporary excavations in the order of 3.0 m to 3.8 m maximum for the removal of the existing fill for the placement of the granular pad. Deeper excavations may be required for the removal the existing concrete abutments.

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 at the site should be classified as Type 3 in accordance with OHSA.

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

10.2 Dewatering

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 and, surface runoff will tend to seep into, and accumulate in proposed excavations.

Dewatering and surface water diversion must remain operational and effective until the temporary excavation is backfilled. Design of an effective dewatering system must be carried out by the Contractor.

10.3 Subgrade Preparation

Subgrade preparation for the abutment foundations should include the removal of the existing fill and any soft or organic materials to a maximum elevation of 196.5 m. The native subgrade within the footprint of the abutment foundation is expected to consist of clay.

The base the excavations should be inspected by a geotechnical inspector (QVE) prior to placing the granular 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 soil within the abutment footprint. Any deleterious materials, organics, or loose/soft or wet conditions observed, should be sub-excavated and removed and the excavations backfilled with OPSS Granular B Type II.

10.4 Abutment Backfill

Backfill behind the abutments should be placed in accordance with OPSS 902. All backfill material should consist of Granular A, or Granular B Type II meeting OPSS.PROV 1010 specifications.

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 should be computed in accordance with the CHBDC but generally are given by the expression:

$$P_h = K^*(\gamma h + q)$$

where:

P_h = horizontal pressure on the wall (kPa)

K = earth pressure coefficient

γ = unit weight of retained soil (kN/m³)

h = 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

Parameter	OPSS Granular A & OPSS Granular B Type II	Native Clay
Soil Unit Weight, kN/m ³ , γ	21	16.5
Angle of Internal Friction, ϕ	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 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 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^3 , γ	21	16.5
Angle of Internal Friction, ϕ	35°	27°
Yielding Wall		
Dynamic Active Earth Pressure Coefficient, K_{AE}	0.29	0.40
Non-Yielding Wall		
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:

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

where:

- σ_h = lateral earth pressure at depth, d (kPa)
- d = depth below the top of the wall (m)
- K_a = static active earth pressure coefficient
- γ = unit weight of the backfill soil (kN/m^3)
- 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

One sample 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 Table 12-1.

Table 12-1: Results of Chemical Analysis

Borehole	Sample	Depth (m)	Sulphate (µg/g)	pH	Resistivity (Ohm-cm)	Chloride (µg/g)
202	SS5	3.3	16	7.7	2400	201

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.
- 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 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 the QVE 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.



Kenton C. Power, P.Eng.
Geotechnical Engineer

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



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

APPENDIX A
BOREHOLE LOCATIONS AND SOIL STRATA DRAWINGS

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.

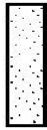


STRATA PLOT:

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

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 (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.

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

METRIC

[illegible]

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 202

1 OF 4

METRIC

GWP# 5149-13-00 LOCATION 38S-126 Centre Line Road ORIGINATED BY NW
 HWY Centre Line Road BOREHOLE TYPE HSA / NW casing COMPILED BY KCP
 DATUM Geodetic DATE 2016.02.16 - 2016.02.17 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 ³	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				
200.4								20	40	60	80	100								
0.0	50 mm SURFACE TREATMENT		AS	SS	-		200							o					21 62 17 (SI+CL)	
199.6	Silty sand with gravel - frost to 2.1 m																			
0.8	Compact Brown FILL		2	SS	24		199							o						
	Silty clay Firm Brown FILL		3	SS	8									o						
198.1																				
2.3	CLAY (CH) - clay crust Stiff Brown		4	SS	7		198							o						
																			0 0 47 53	
196.9			5	SS	4		197							o						
3.5	CLAY (CI) Firm Grey																			
			1	TW	Push		196							o						
			6	SS	WH									o						
							195													
								6.3 +												
								4.4 +												
			7	SS	WH		194							o						
								5.3 +												
								5.3 +												
			8	SS	WH		192							o						
								5.3 +												
								4.6 +												
			9	SS	WH		191							o						

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METRIC

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ONTMT4S 10870 CENTRE LINE RD.GPJ 2012TEMPLATE(MTO).GDT 3/5/16

RECORD OF BOREHOLE No 202

3 OF 4

METRIC

GWP# 5149-13-00 LOCATION 38S-126 Centre Line Road ORIGINATED BY NW
 HWY Centre Line Road BOREHOLE TYPE HSA / NW casing COMPILED BY KCP
 DATUM Geodetic DATE 2016.02.16 - 2016.02.17 CHECKED BY PC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
	Continued From Previous Page		15	SS	10		180							0 23 75 2
	SILT (ML) with sand Loose to compact Grey		16	SS	11		177							
							176							
							175							
				17	SS	8		174						
								173						
171.4	SAND (SP-SM) with silt TILL - occasional cobbles Very dense Grey		18	SS	55		171							4 89 7 (SI+CL)
29.0														

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 202

4 OF 4

METRIC

GWP# 5149-13-00 LOCATION 38S-126 Centre Line Road ORIGINATED BY NW
 HWY Centre Line Road BOREHOLE TYPE HSA / NW casing COMPILED BY KCP
 DATUM Geodetic DATE 2016.02.16 - 2016.02.17 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 ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
							20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					W P W W L 20 40 60					
	Continued From Previous Page																
169.3	SAND (SP-SM) with silt TILL - occasional cobbles Very dense Grey	0 4 0 4	19	SS	98		170										
31.1	Borehole terminated at 31.1 m																

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RECORD OF BOREHOLE No 203

1 OF 4

METRIC

GWP# 5149-13-00 LOCATION 38S-126 Centre Line Road ORIGINATED BY NW
 HWY Centre Line Road BOREHOLE TYPE HSA / NW casing COMPILED BY KCP
 DATUM Geodetic DATE 2016.02.15 - 2016.02.16 CHECKED BY PC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	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				
200.3								20	40	60	80	100								
0.0	50 mm SURFACE TREATMENT																			
199.7	Silty sand with gravel - occasional cobbles - frost to 1.5 m Compact to dense Brown FILL		1	AS	-															
0.6	Clay with gravel, trace sand Brown to grey Firm FILL		2	SS	26															
			3	SS	14									22 8 45 25						
	- wood pieces from 2.3 m to 3.8 m		4	SS	7															
			5	SS	6															
196.5	Sandy clay firm Grey FILL		6	SS	4									0 35 44 21						
3.8																				
195.5	CLAY (CH to CI) - trace rootlets at 4.7 m Firm Grey		7	SS	3															
4.7			8	SS	WH															
			9	SS	WH															
			10	SS	WH									0 1 40 59						
			11	SS	WH															

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Sensitivity

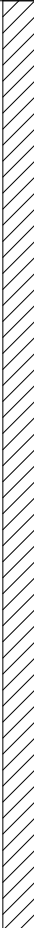


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15
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(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 203

2 OF 4

METRIC

GWP# 5149-13-00 LOCATION 38S-126 Centre Line Road ORIGINATED BY NW
 HWY Centre Line Road BOREHOLE TYPE HSA / NW casing COMPILED BY KCP
 DATUM Geodetic DATE 2016.02.15 - 2016.02.16 CHECKED BY PC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)			
								20 40 60 80 100	w P w w L				
Continued From Previous Page													
184.1	CLAY (CH to CI) Firm Grey - becomes stiff						190	5.6 6.3					
			12	SS	WH		189	5.4 5.1					
			13	SS	WH		188						
							187	3.6 3.6					
			14	SS	WH		186		4.3		+		
							185						
			15	SS	WH								
							184						
							183						
							182						
16.2	SILT (ML), trace sand Loose Grey											0 3 59 38	
16			SS	5									
182.0	Sandy SILT (ML) Loose to compact Grey												
18.3			17	SS	10								

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+³, ×³: Numbers refer to Sensitivity
 20
 15
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 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 203

3 OF 4

METRIC

GWP# 5149-13-00 LOCATION 38S-126 Centre Line Road ORIGINATED BY NW
 HWY Centre Line Road BOREHOLE TYPE HSA / NW casing COMPILED BY KCP
 DATUM Geodetic DATE 2016.02.15 - 2016.02.16 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 ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL	
								20	40	60	80	100	20	40	60						
	Continued From Previous Page		18	SS	14		180											0	31	66	3
							179														
							178														
			19	SS	9		177														
							176														
							175														
			20	SS	16		174														
							173														
							172														
171.3 29.0	Silty SAND (SM) TILL Very dense Grey		21	SS	89		171											4	67	29 (SI+CL)	

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15 10 5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 203

4 OF 4

METRIC

GWP# 5149-13-00 LOCATION 38S-126 Centre Line Road ORIGINATED BY NW
 HWY Centre Line Road BOREHOLE TYPE HSA / NW casing COMPILED BY KCP
 DATUM Geodetic DATE 2016.02.15 - 2016.02.16 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 ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
							20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					W P W W L 20 40 60					
	Continued From Previous Page																
169.2	Silty SAND (SM) TILL Very dense Grey	0 4 0 0 4	22	SS	53		170										
31.1	Borehole terminated at 31.1 m																

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RECORD OF BOREHOLE No 204

1 OF 1

METRIC

GWP# 5149-13-00 LOCATION 38S-126 Centre Line Road ORIGINATED BY NW
 HWY Centre Line Road BOREHOLE TYPE HSA COMPILED BY KCP
 DATUM Geodetic DATE 2016.02.17 - 2016.02.17 CHECKED BY PC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								20 40 60 80 100									
						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT w _p — w — w _L WATER CONTENT (%)							
						20 40 60 80 100				20 40 60							
200.1																	
0.0	50 mm SURFACE TREATMENT						200										
0.1	Silty sand with gravel - frost to 3.0 m		1	AS	-										25 55 20 (SI+CL)		
199.5	Compact Brown FILL																
0.6	Clay with sand Firm Brown FILL		2	SS	32		199								2 17 50 31		
198.6	CLAY (CH) - clay crust Stiff Brown		3	SS	17		198										
1.5			4	SS	15												
197.1	CLAY (CI) Firm to soft Grey		5	SS	10		197										
3.0			6	SS	3		196										
			7	SS	WH		195										
			8	SS	WH		194										
193.4	Borehole terminated at 6.7 m																
6.7																	

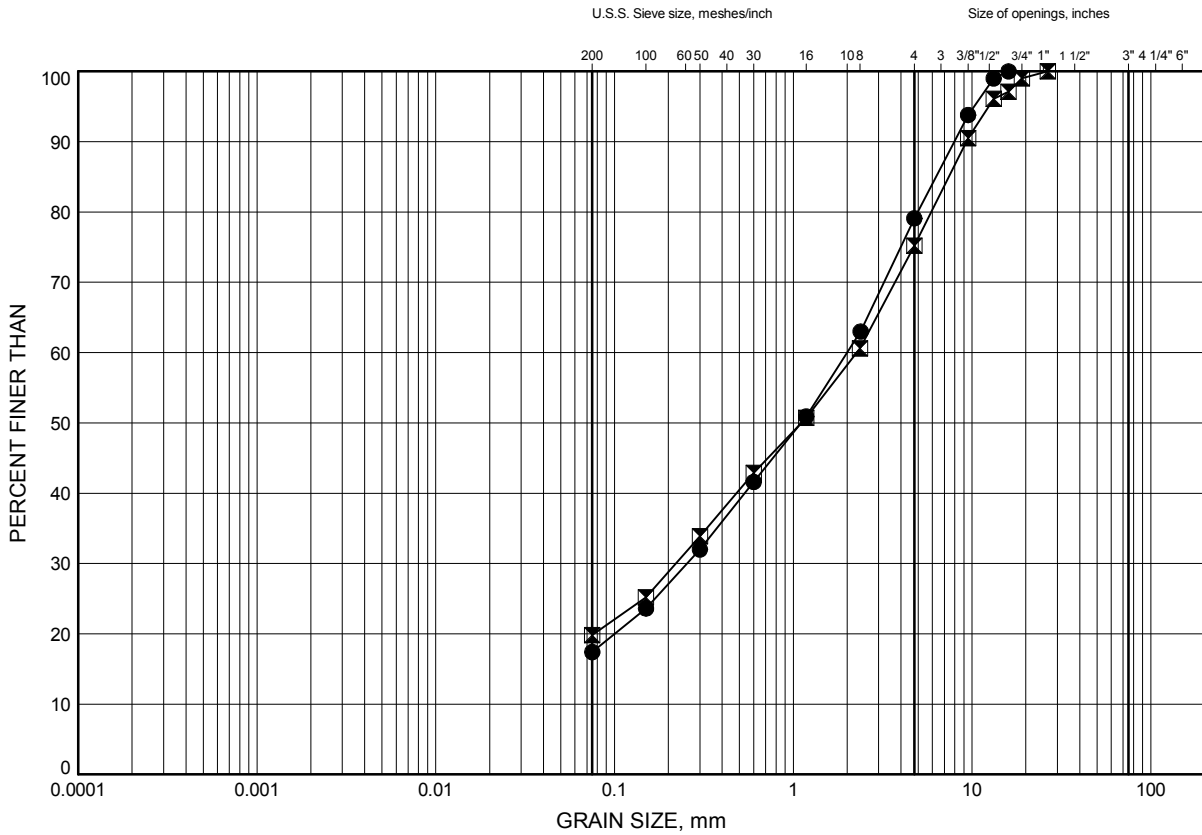
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APPENDIX C
LABORATORY TEST RESULTS

38S-126 Centre Line Road
GRAIN SIZE DISTRIBUTION

FIGURE 1

Granular Fill



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	202	0.30	200.09
⊠	204	0.30	199.83

Date May 2016
 GWP# 5149-13-00

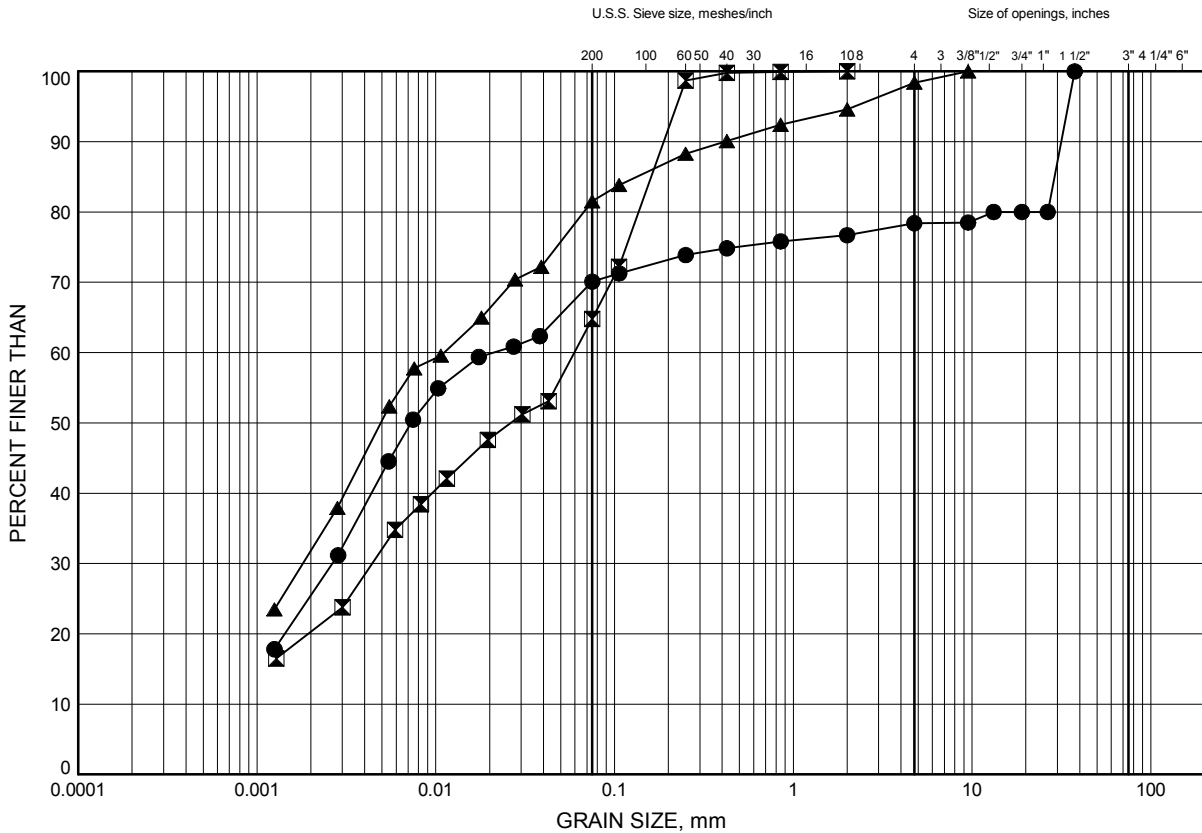


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 Chkd. PC

38S-126 Centre Line Road
GRAIN SIZE DISTRIBUTION

FIGURE 2

Clay Fill



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	203	1.83	198.44
⊠	203	4.11	196.16
▲	204	1.07	199.07

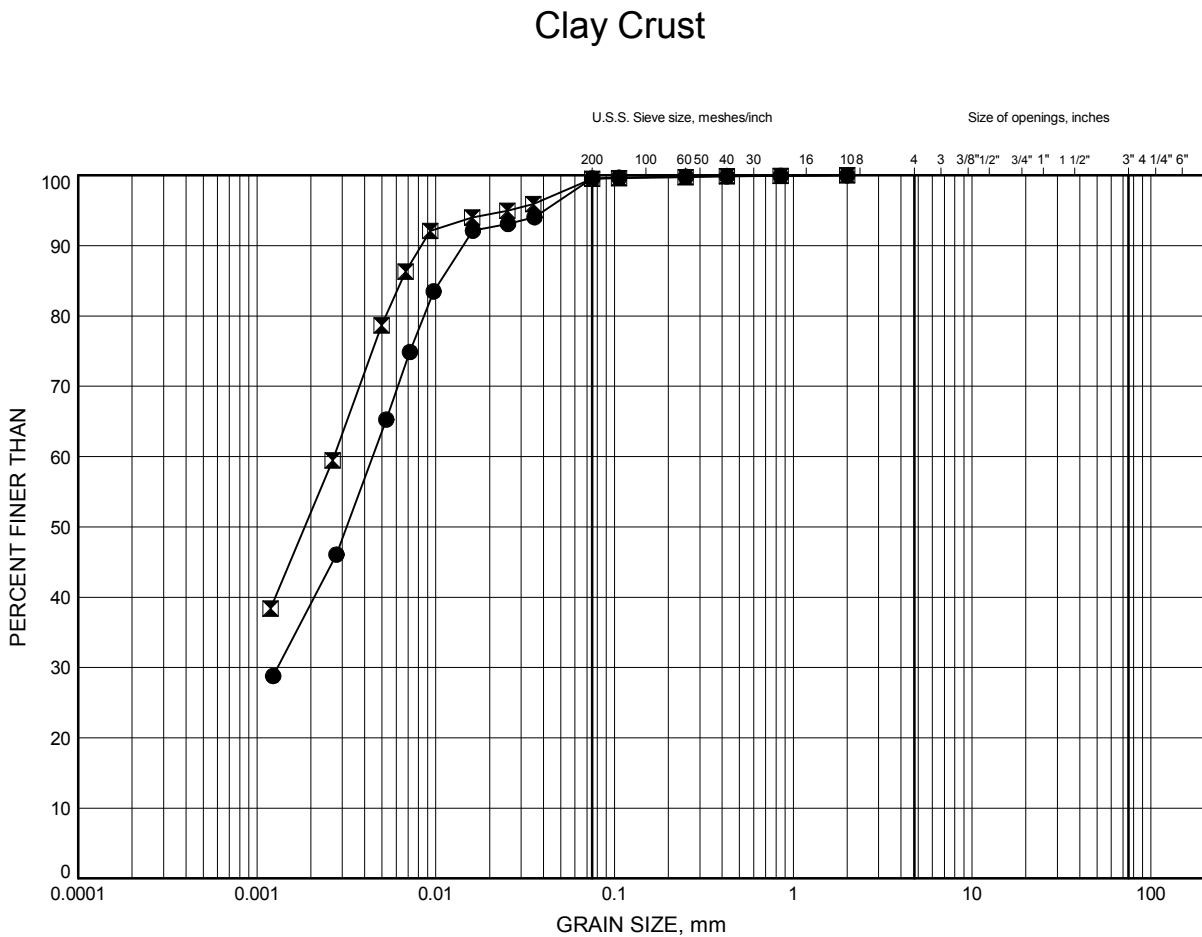
Date May 2016
GWP# 5149-13-00



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Chkd. PC

38S-126 Centre Line Road
GRAIN SIZE DISTRIBUTION

FIGURE 3



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

LEGEND

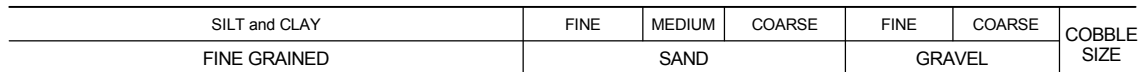
SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	201	1.83	198.87
⊠	202	2.59	197.80

Date May 2016
 GWP# 5149-13-00



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FIGURE 4



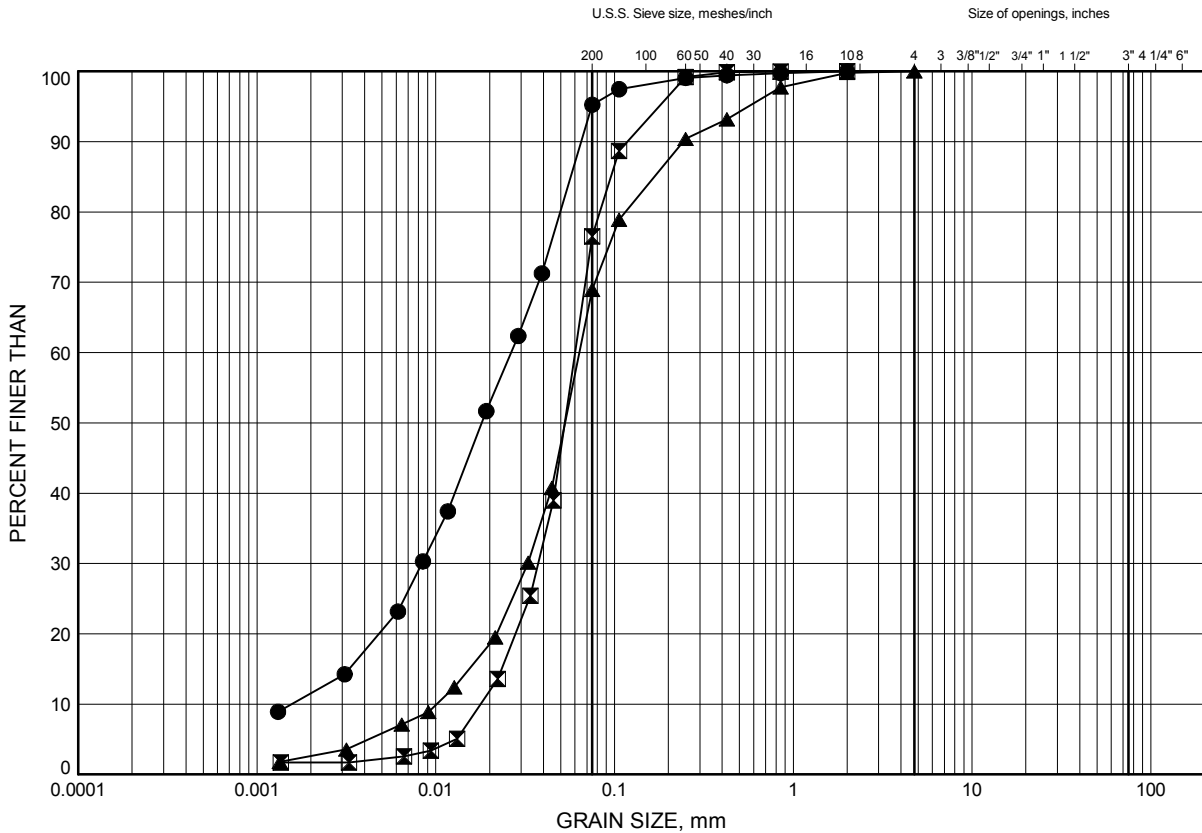
SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	201	6.40	194.30
☒	202	12.50	187.90
▲	203	7.92	192.35
★	203	15.54	184.73

Chkd. PC

38S-126 Centre Line Road
GRAIN SIZE DISTRIBUTION

FIGURE 5

Silt to Sandy Silt



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	202	18.59	181.80
⊠	202	20.12	180.28
▲	203	20.12	180.16

Date May 2016

GWP# 5149-13-00

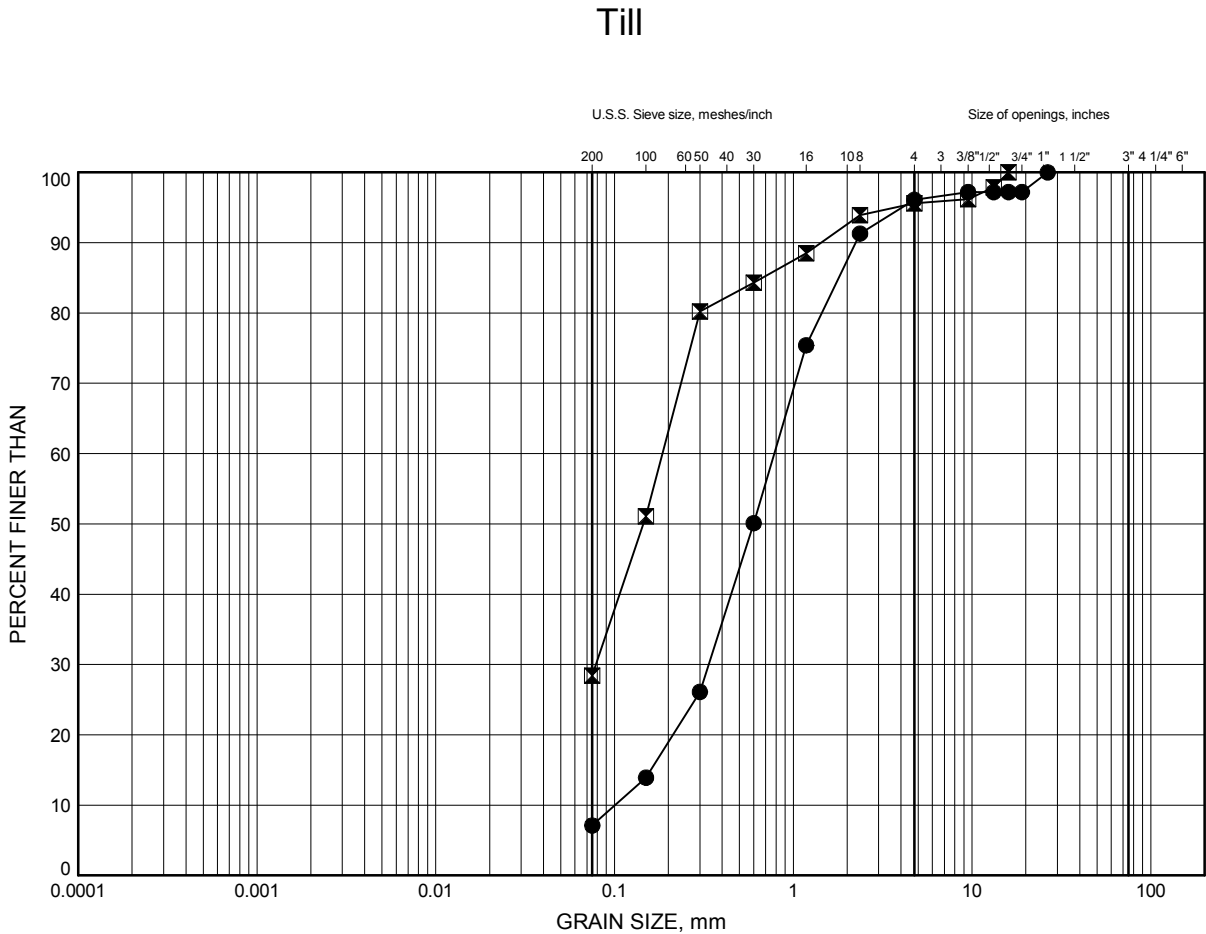


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38S-126 Centre Line Road
GRAIN SIZE DISTRIBUTION

FIGURE 6



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	202	29.26	171.13
⊠	203	29.26	171.01

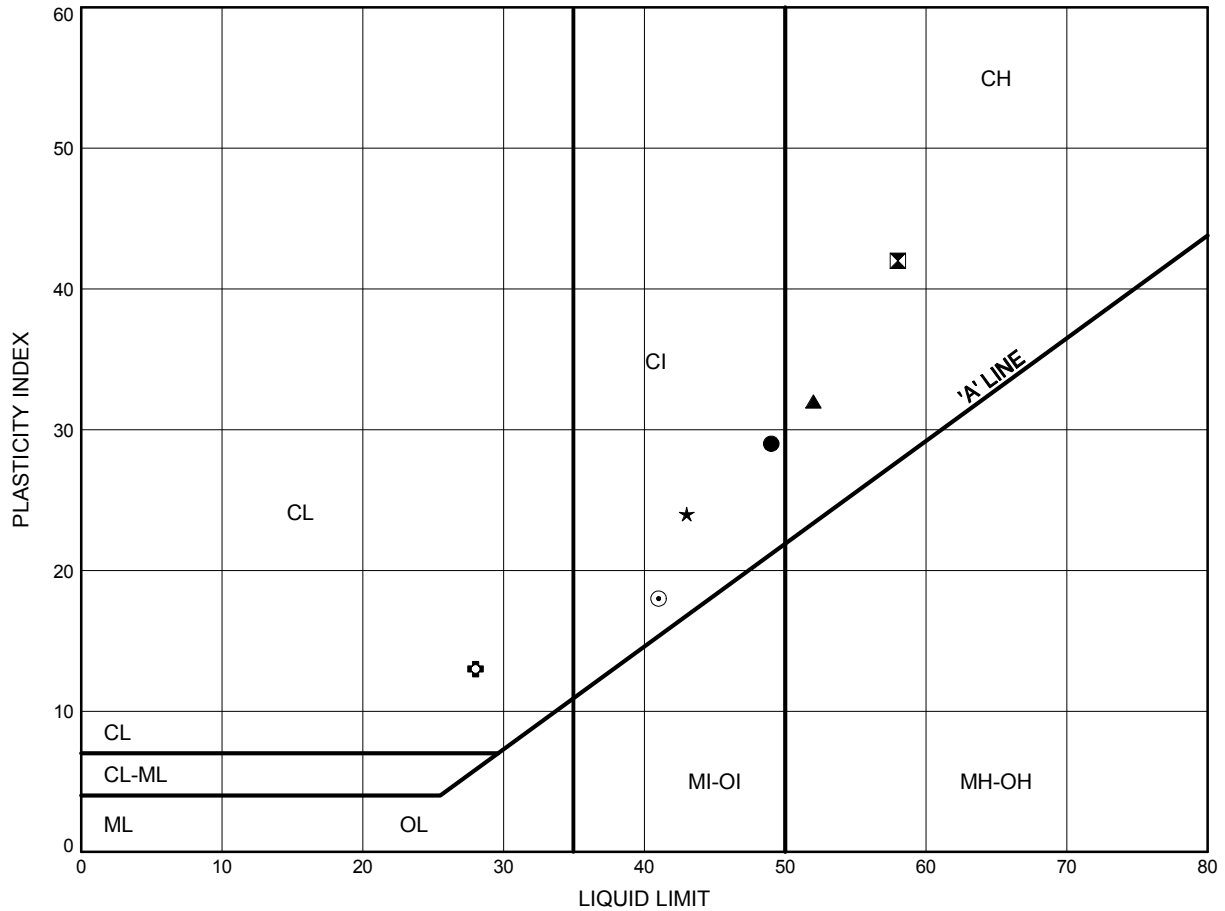
Date May 2016
GWP# 5149-13-00



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38S-126 Centre Line Road
ATTERBERG LIMITS TEST RESULTS

FIGURE 7



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	201	1.83	198.87
⊠	201	6.40	194.30
▲	202	2.59	197.80
★	202	12.50	187.90
⊙	203	1.83	198.44
⊕	203	4.11	196.16

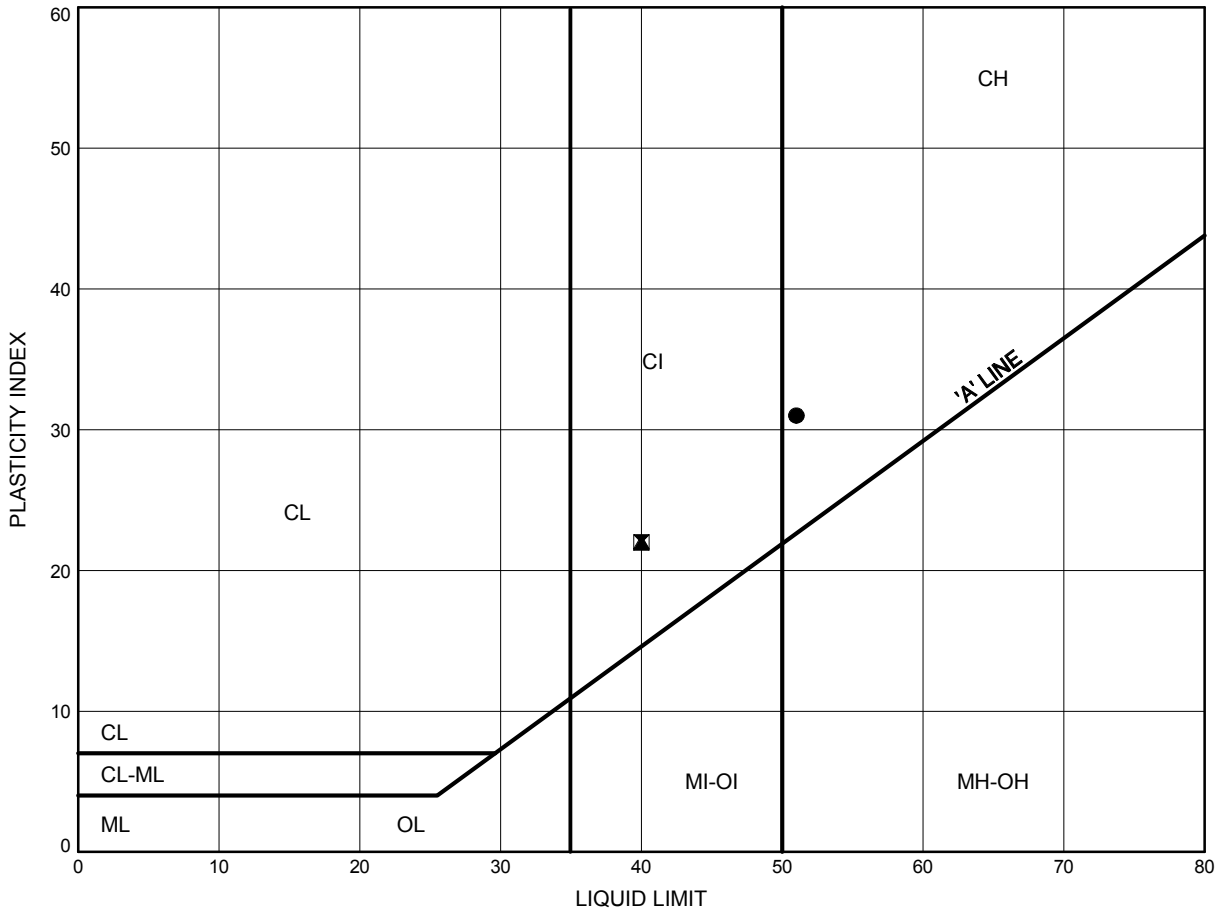
Date May 2016
 GWP# 5149-13-00



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38S-126 Centre Line Road
ATTERBERG LIMITS TEST RESULTS

FIGURE 8



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	203	7.92	192.35
⊠	203	15.54	184.73
▲	204	1.07	199.07

Date May 2016
 GWP# 5149-13-00



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 Chkd. PC

APPENDIX D

SELECT PHOTOGRAPHS OF THE BRIDGE LOCATION



Figure 1: Looking south across Centre Line Road Bridge



Figure 2: Existing Centre Line Bridge elevation looking west



Figure 3: Looking upstream from Centre Line Road Bridge



Figure 4: Looking downstream from Centre Line Road Bridge

APPENDIX E

TABLE E-1: COMPARISON OF FOUNDATION OPTIONS

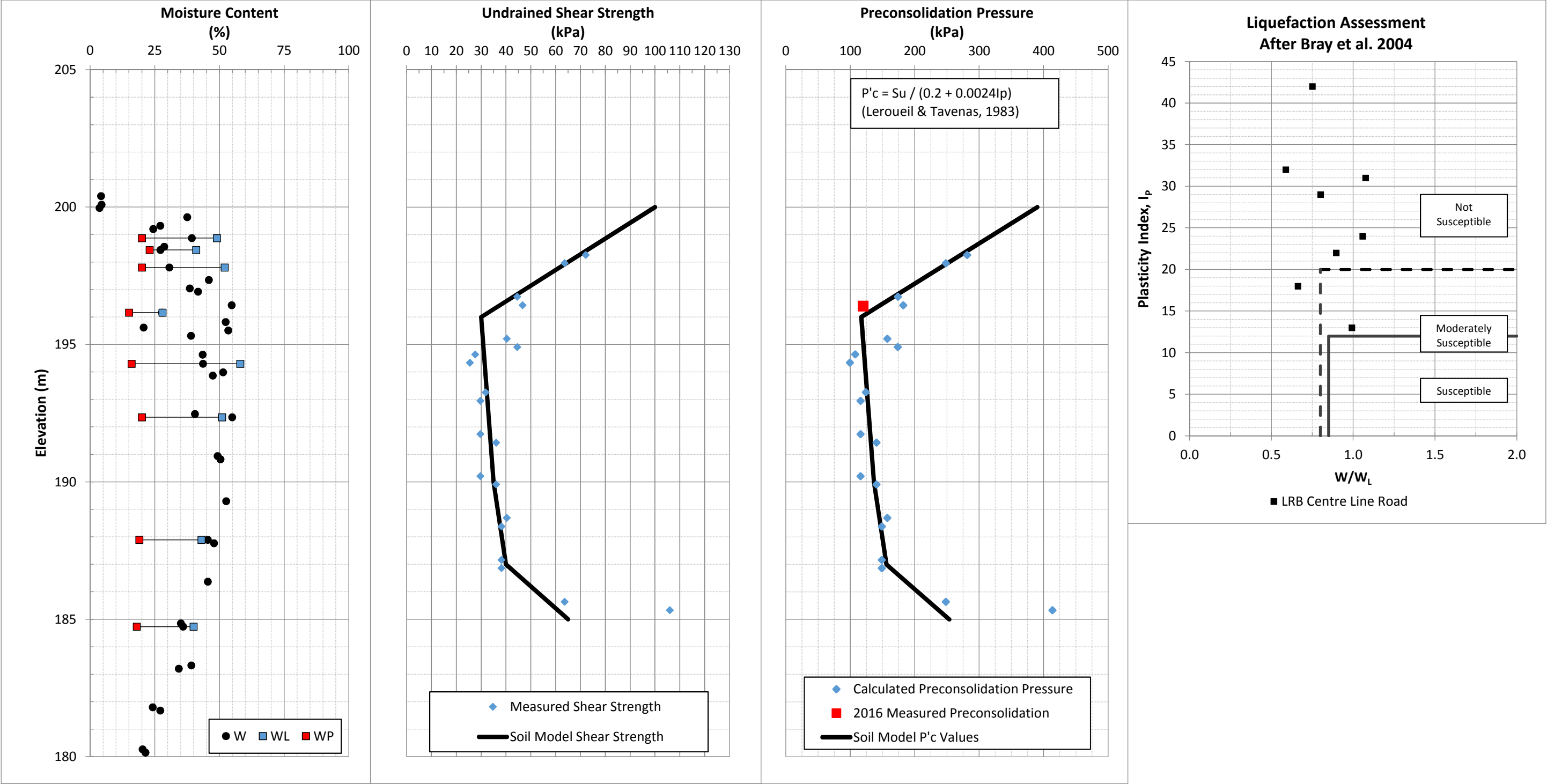
E- 1: Comparison of Foundation Alternatives

Comment	Friction Steel H-Piles	Spread Footings on Native Soils	Spread Footing Placed on Engineered Fill Pads
Advantages:	Quick installation procedure	N/A	Quick installation and construction
Disadvantages:	High cost for installation Downdrag loads will due to the settlement of the clay layer will require additional piles to be installed	Low strength and highly compressible nature native clay would require abutment foundations to be designed at a comparatively low geotechnical resistance	Future bridge maintenance and jacking up of the abutments to compensate for the consolidation settlements
Relative Cost	High	Moderate	Moderate
	Feasible	NOT RECOMMENDED	Recommended

APPENDIX F

GEOTECHNICAL CLAY PROPERTIES GSC SEISMIC HAZARD CALCULATION L-PILE ANALYSIS FOR HP 310X110 STEEL PILES

Clay Properites for Site LRB Centre Line Road



Clay Properties
Site 38S-126
Centreline Road Overpass

G.W.P. 5149-13-00

Project No.: 10870

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

May 03, 2016

Site: 46.448 N, 83.8099 W User File Reference: Centre Line 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)	Sa(0.2)	Sa(0.3)	Sa(0.5)	Sa(1.0)	Sa(2.0)	Sa(5.0)	Sa(10.0)	PGA (g)	PGV (m/s)
0.047	0.066	0.064	0.055	0.047	0.029	0.015	0.0036	0.0016	0.037	0.036

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 m/s^2). Peak ground velocity is given in m/s . Values are for "firm ground" (NBCC 2015 Site Class C, average shear wave velocity 450 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

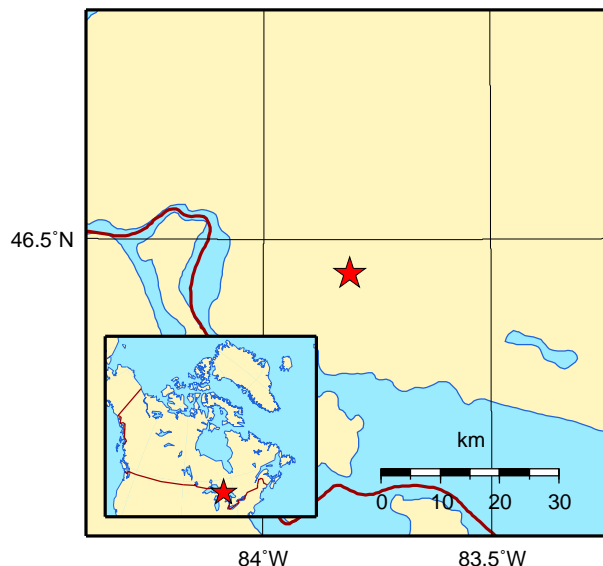
National Building Code of Canada 2015 NRCC no. 56190;
Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

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 and www.nationalcodes.ca for more information

Aussi disponible en français



Natural Resources
Canada

Ressources naturelles
Canada



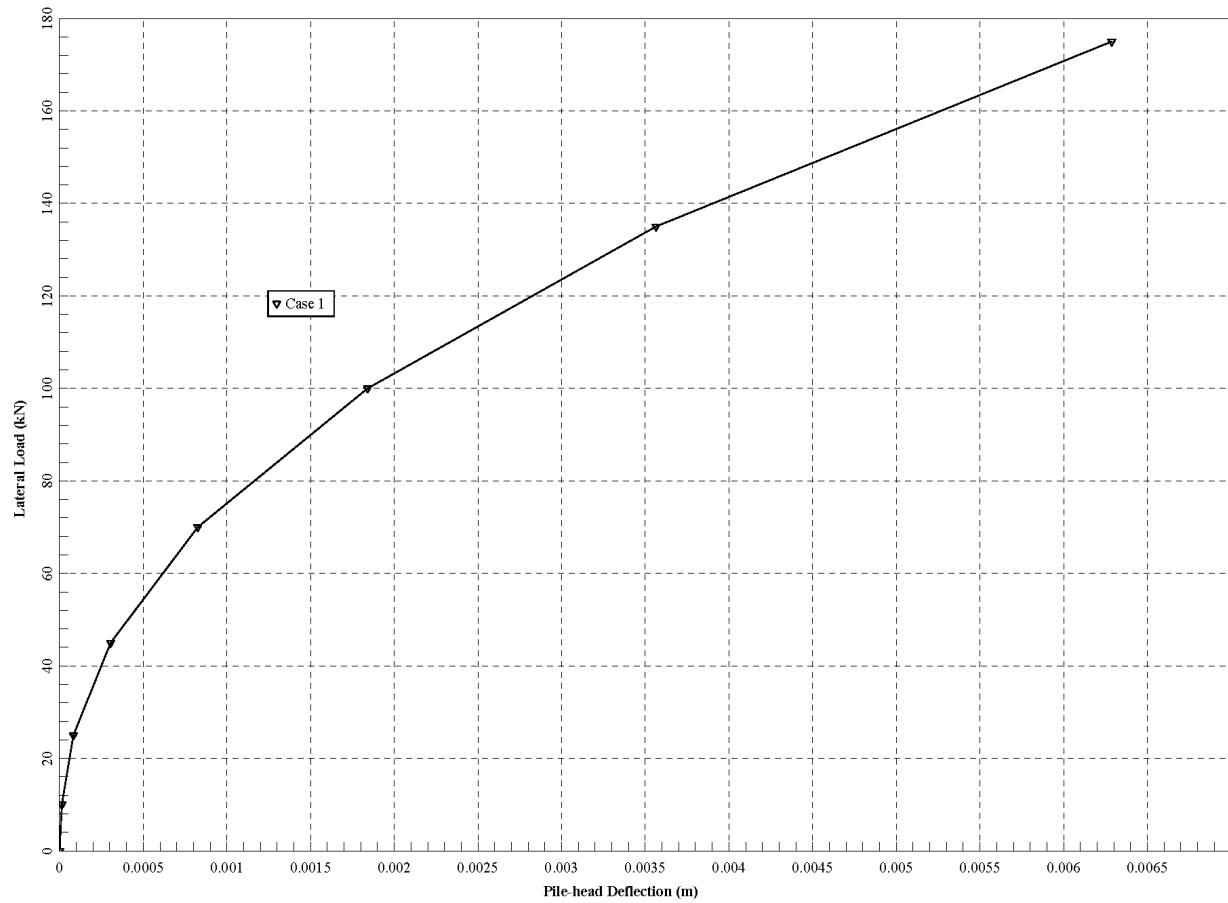


Figure 5: Lateral Load vs. Pile-head Deflection for the North Embankment (Borehole 202)

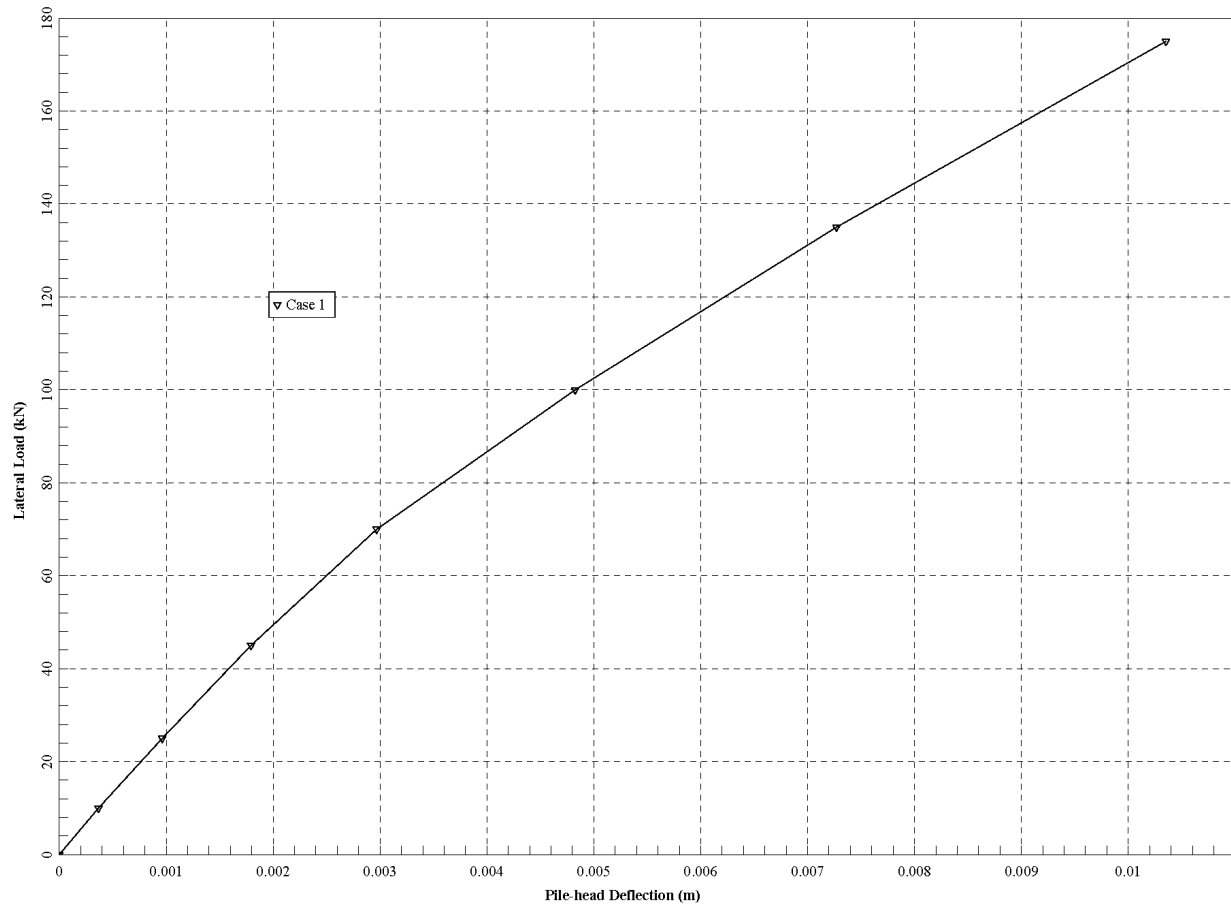


Figure 6: Lateral Load vs. Pile-head Deflection for the South Embankment (Borehole 203)

APPENDIX G

LIST OF REFERENCED SPECIFICATIONS NON-STANDARD SPECIAL PROVISIONS - USE OF HEAVY CONSTRUCTION EQUIPMENT

LIST OF REFERENCED SPECIFICATIONS

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

RECOMMENDED WORDING FOR "NSSP- USE OF HEAVY CONSTRUCTION EQUIPMENT"

The use of heavy construction equipment and in particular heavy lift cranes may be required during removal of the existing and erection of the new bridge. The impact of the heavy equipment loads on the existing embankment, the native soft to firm soils clay underlying the embankment and the existing bridge foundations must be considered during selection of the methodology and equipment employed for construction.

Prior to commencement of construction, the Contractor shall retain a Geotechnical Consultant to assess the impact of the proposed equipment loads and methodology, and determine requirements and/or restrictions necessary to safely support the loads. 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:

- Determining appropriate setbacks for heavy equipment from the bridge abutments and existing foundations;
- Determining the permissible ground pressure that may be applied to the foundation soils by the equipment; and
- Providing recommendations for crane pad design to distribute the crane loads without causing foundation failure.

The Contractor shall submit the findings of the geotechnical assessment and details of the proposed equipment and construction methodology to the Contract Administrator for information purposes a minimum of two weeks prior to the start of construction.