THURBER ENGINEERING LTD.

FINAL

FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 101 NEMEGOSENDA RIVER BRIDGE
32 KM EAST OF HIGHWAY 129, CHEWETT TOWNSHIP
SITE NO.: 46-215, G.W.P. 5144-10-00

5015-E-0027

Geocres No.: 41O-29

Report to:

McIntosh Perry Consulting Engineers

Latitude: 47.93808
Longitude: -83.06024

October 2018
Thurber File No.: 13624

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PART 1. FACTUAL INFORMATION

1 INTRODUCTION

This section of the report presents the factual findings obtained from a foundation investigation completed for the proposed replacement of the Nemegosenda Lake Bridge (Structure No. 46-215). The structure is located on Highway 101 approximately 32 km east of Highway 129. Thurber Engineering Ltd. (Thurber) carried out the investigation as a subconsultant to McIntosh Perry Consulting Engineers (MPCE) as part of Agreement No. 5015-E-0027.

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. A base plan survey drawing was provided by MPCE for the preparation of this report.

An earlier foundation investigation report that has been obtained from the online Geocres Library in preparation of this report is as follows:

> Foundation Investigation Report, Nemegosenda River and Highway 101 Crossing between Chapleau and Foleyet, W.J. 61-F-21, District #18 (Geocres 41O00-004), dated April 1961.

The position of the boreholes from the historical report relative to the boreholes completed as part of the current investigation are not known, therefore the historic boreholes have been included in Appendix B for information purposes only and have not been included in the description of the subsurface conditions within this report.

2 SITE DESCRIPTION

The existing structure is located on Highway 101 in the township of Chewett (Linear Highway Referencing System Base Points: 40420, Offset: 0.0). The location of the bridge is shown on the inset Key Plan on Drawing No. 1 in Appendix A. The existing bridge is a 25.3 m long single span, rectangular-solid wood beam (glulam) bridge with a laminated timber deck. A 1982 rehabilitation included placement of a concrete topping slab above the timber decking. The bridge deck is approximately 4 m above the river water level. The embankment slopes located adjacent to the abutment are inclined at approximately 2.0H:1V with the surface consisting of granular material near the abutments and vegetation.

Within the project limits, Highway 101 is a two-lane, undivided highway with a rural cross-section. The base plan drawing indicates that the roadway cross-section consists of two, 3.5 m wide lanes, and paved shoulders with a width of 0.5 m and 0.9 m in the east bound and west bound directions respectively. Steel guide rails are present at all four corners of the structure. On the southwest side of the bridge alignment is a gravel access road leading to a water monitoring shed located at the river's bank. The topography adjacent to the bridge site is rolling forested lands with frequent bedrock outcrops. The land in the vicinity of the bridge is uninhabited and undeveloped. Traffic volumes are understood to be less than 1000 AADT (2012)

Select site photographs showing the general conditions in the area of the bridge during the time of the field investigation are presented in Appendix D.

3 SITE INVESTIGATION AND FIELD TESTING

The field investigation for this site included advancing nine boreholes drilled from October 27, 2016 to October 30, 2016. The northing, easting and elevation of the boreholes are shown on the Borehole Location and Soil Strata Drawing No. 1 in Appendix A and are summarized in Table 3-1. In advance of the field investigation, utility locate clearances were obtained at the location of the boreholes.

Table 3-1: Borehole Summary

Borehole No.	Drilled Location	Northing (m)	Easting (m)	Ground Surface Elevation (m)	Termination Depth below Existing Ground Surface (m)
16-05	East Approach – westbound lane	5 311 438	375 025	404.5	9.5
16-06	East Abutment – westbound lane	5 311 444	375 011	404.6	11.7
16-07	East Abutment – eastbound lane	5 311 440	375 009	404.6	8.8
16-08	West Abutment – westbound lane	5 311 458	374 982	404.7	7.1
16-09	West Abutment – eastbound lane	5 311 453	374 980	404.7	3.8
16-10	West Abutment – westbound lane	5 311 457	374 981	404.7	7.8
16-11	West Approach – westbound lane	5 311 459	374 980	404.7	5.3
16-12	West Abutment – eastbound land	5 311 454	374 977	404.7	7.2
16-13	West Approach – eastbound lane	5 311 460	374 964	404.6	5.1

All boreholes were advanced through the roadway embankment with a truck mounted CME 75 drill rig equipped with hollow stem augers and HW/NW casing. The drilling and sampling operations were supervised on a full time basis by a member of Thurber's technical staff. Where possible soil samples were collected at regular depth intervals in the boreholes using a split spoon sampler in conjunction with Standard Penetration Tests (SPT). All soil samples recovered from the boreholes were transported to Thurber's Ottawa geotechnical laboratory for further examination and testing.

A 19 mm inside diameter PVC standpipe piezometer was installed in Borehole 16-06 to allow for measurement of the groundwater level at the east abutment following completion of drilling. The piezometer construction details are illustrated on the Record of Borehole sheet for Borehole 16-06, provided in Appendix B. The piezometer was decommissioned on November 6, 2016 following completion of the field investigation program.

The other boreholes were backfilled with a low-permeability mixture of auger cuttings and bentonite pellets in accordance with Ontario MOE Regulation 903. Boreholes advanced within paved areas were capped with cuttings followed by 150 mm of cold patch asphalt to reinstate the travelling surface.

4 LABORATORY TESTING

Geotechnical laboratory testing consisted of natural moisture content determination and visual identification of all retained soil samples in accordance with the current MTO standards. Grain size distribution analyses testing was also carried out on selected samples to MTO and ASTM standards. Chemical analyses for determination of pH, resistivity, soluble sulphate and chloride concentrations were carried out on two soil samples.

The results of the geotechnical tests are summarized on the Record of Borehole sheets included in Appendix B and all laboratory results are presented on the figures included in Appendix C.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

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 and cross section for the bridge area are presented on Drawing No. 1 and 2 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. It must be recognized that the soil and groundwater conditions may vary between and beyond borehole locations.

The stratigraphy in the boreholes through the embankment is generally characterized by an asphalt pavement structure overlaying an embankment constructed with granular fill overlying native silty sand overlying bedrock.

5.1 Embankment

5.1.1 Asphalt

All boreholes were advanced from the surface of Highway 101 and encountered an asphalt pavement structure. The thickness of the asphalt ranged from 40 mm to 80 mm.

5.1.2 Fill: Sand

Granular fill varying in composition from silty sand with gravel to gravel with sand was encountered below the asphalt in all boreholes. Boulders and cobbles were noted within the fill layers. This fill had a thickness ranging from 3.0 m to 4.3 m (bottom elevation of 400.5 m to 401.7 m). The SPT ‘N’ values ranged from 8 to 79 blows indicating a loose to very dense condition. SPT ‘N’ values greater than 100 blows per 225 mm of penetration were recorded locally in zones containing cobbles.

The moisture content of the samples tested ranged from 2% to 15%. The results of grain size analyses conducted on ten samples of this material are summarized in Table 5-1 and are illustrated on Figures C1 and C2 in Appendix C.

Table 5-1: Gradation Results for Granular Fill

Soil Particle	%	
	Sand Fill	Gravel Fill
Gravel	4 - 39	47 – 55
Sand	48 - 89	37 – 41
Silt and Clay	5 - 13	8 - 12

5.2 Silty Sand to Sand with Silt

Native layers of silty sand to sand with silt with varying amounts of gravel were encountered below the fill materials in Boreholes 16-05, 16-06, 16-07, 16-11 and 16-13. This layer has a thickness ranging from 1.5 m to 6.5 m with an underside elevation of 395.0 to 399.5 m. The SPT ‘N’ values ranged from weight of hammer to 32 blows indicating a very loose to dense condition.

The moisture content for the samples tested typically ranged from was 8% to 19%. The results of grain size analyses conducted on seven samples of this material are summarized in Table 5-2 and are illustrated on Figures C3 and C4 in Appendix C.

Table 5-2: Gradation Results for Silty Sand to Sand with Silt

<table>
<tr><th>Soil Particle</th><th colspan="2">%</th></tr>
<tr><td>Gravel</td><td colspan="2">2 - 26</td></tr>
<tr><td>Sand</td><td colspan="2">49 - 72</td></tr>
<tr><td>Silt</td><td>17 - 23</td><td rowspan="2">10 - 46</td></tr>
<tr><td>Clay</td><td>2 - 3</td></tr>
</table>

5.3 Bedrock

The overburden materials were underlain by granite bedrock. Boreholes 16-06, 16-08, 16-10 and 16-12 were advanced into the bedrock by coring. The bedrock surface elevation ranges from 396.5 to 401.7 m and is summarized in the table below:

Table 5-3 Summary of Bedrock Elevation

Location	Borehole No.	Depth Below Existing Ground Surface (m)	Top of Bedrock or Inferred Bedrock Elevation (m)
East Approach	16-05	9.5	395.0(*)
East Abutment	16-06	8.1	396.5
	16-07	8.8	395.8(*)
West Abutment	16-08	3.0	401.7
	16-09	3.8	400.8(*)
	16-10	4.3	400.5
	16-11	5.3	399.4(*)
	16-12	3.8	400.8
West Approach	16-13	5.1	399.5(*)

Note: (*) inferred by SPT refusal and/or casing advancement refusal

The Total Core Recovery (TCR) ranged from 87 to 100%, the Solid Core Recovery (SCR) ranged from 60 to 100% and the Rock Quality Designation (RQD) ranged from 17 to 93%. Based on the RQD value the bedrock is classified as poor to excellent quality. It is noted that rock quality in Borehole 16-06 near the east abutment was significantly poorer (RQD as low as 17 in the surficial run) than in the other boreholes. Rock core photos have been included in Appendix C.

5.4 Groundwater

Groundwater was observed in Boreholes 16-05 and 16-07 during drilling and was noted to range from elevation 398.2 to 398.6 m. Groundwater was not observed in Boreholes 16-09, 16-11 and 16-13 which were dry following completion of drilling.

The groundwater level was measured in the standpipe piezometer installed in Borehole 16-06 on November 6, 2016 at an approximate depth of 4.1 m; corresponding to an elevation of 400.5 m. The water level in Nemegosenda Lake was measured at the time of Thurber's field investigation at an elevation of 400.3 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 and/or prolonged precipitation. It is expected that the groundwater level will largely be controlled by the water level in Nemegosenda Lake.

5.5 Analytical Results

Two 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 results are summarized in the table below.

Table 5-4: Results of Chemical Analysis

Borehole	Sample	Depth (m)	Sulphate (µg/g)	pH	Resistivity (Ohm-cm)	Chloride (µg/g)
16-6	SS3	1.8	10	7.9	2600	159
16-8	SS4	2.6	31	7.9	1370	346

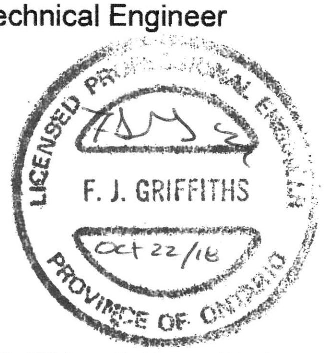
6 MISCELLANEOUS

Borehole locations were selected and positioned relative to existing site features and the proposed foundation locations by Thurber. MPCE surveyed the borehole locations and ground surface elevations.

George Downing Estate Drilling Ltd. of Hawkesbury, Ontario supplied and operated the drilling equipment to carry out the drilling, sampling, in-situ testing, standpipe piezometer installation and borehole decommissioning. The field investigation was supervised on a full-time basis by Mr. Christopher Murray, P.Eng. of Thurber. Overall project management and direction of the field program was provided by Mr. Stephen Peters, P.Eng.

Routine laboratory testing was carried out in Thurber's MTO-approved laboratory in Ottawa. Analytical testing was completed by Paracel Laboratories. Interpretation of the field data and preparation of this report was completed by Dr. Fred Griffiths, P.Eng. and Mr. Stephen Peters, P.Eng. The report was reviewed by and Dr. P.K. Chatterji, P.Eng., the Designated Principal Contact for MTO Foundations Projects.

Stephen Peters, P.Eng.
Geotechnical Engineer



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PART 2. ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 INTRODUCTION

This section of the report presents interpretation of the factual data in Part 1 of this report for the proposed replacement of the Nemegosenda Lake Bridge located on Highway 101, near Chapleau, Ontario. Geotechnical assessment and recommendations are provided to assist the project team in designing a suitable foundation for the proposed replacement bridge.

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 construction or 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 existing 25.3 m long by 9.75 m wide bridge is supported on timber crib abutments filled with rockfill. Settlement has been noted to have occurred at the approaches as documented in the 2015 Ontario Structure Inspection Manual.

The following sections address the foundation aspects of the installation of new bridge foundations. The discussions and recommendations presented in this report are based on the information provided by MPCE including the 30% Contract Drawing dated October 2017 and on the factual data obtained during the course of the investigation.

7.1 Proposed Structure

At the time of preparation of this Foundation Investigation and Design Report, the design of the proposed bridge structure is shown on Sheet 31 of the Contract Drawings to consist of a 13 m wide by 28 m long single span bridge with 5 NU1200 concrete girders. The bridge will be replaced along the same alignment as the existing bridge. The west abutment is indicated to be founded on a footing with an underside elevation of 401.0 m on mass concrete placed directly on bedrock. The east abutment is indicated to be founded on two rows of battered steel H-piles end bearing on bedrock with the underside of the pile cap at elevation 400.5 m.

A full bridge closure and a temporary traffic detour was identified as the preferred approach within the Technically Preferred Alternative (TPA) memorandum for construction staging. A separate field investigation for the temporary detour and modular bridge along the north side of the highway alignment has been undertaken and recommendations are provided within a separate foundation report (Geocres 41O-30).

7.2 Applicable Codes and Design Considerations

The geotechnical assessment presented below has 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.

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	Major Route Bridge	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.

The frost penetration depth and associated recommendations are provided in Section 10.6

8 SEISMIC CONSIDERATIONS

8.1 Spectral and Peak Acceleration Hazard 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 (Sa(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). The PGA value at this site for a *reference* Site Class C with a 2% probability of exceedance in 50 years (2475-year event) is 0.043 g. This value is to be scaled by the site-specific Site Class as discussed below.

8.2 CHBDC Seismic Site Classification

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

Based on the soil and bedrock conditions encountered below the anticipated bridge foundation elevation, the site is classified as a Seismic Site Class D in accordance with Table 4.1 of the CHBDC.

8.3 Seismic Liquefaction

Based on the subsurface conditions encountered at the drilled locations at this site, the foundation soils are considered to be not susceptible to liquefaction during a seismic event taking into consideration the low PGA values.

9 STRUCTURE FOUNDATION ALTERNATIVES

Given the soil and rock stratigraphy encountered during the field investigation, the following options have been considered for the new bridge foundations:

- Spread footings
- Caissons socketed into bedrock (drilled shafts)
- Steel piles (H-piles, pipe piles)

These foundation alternatives are presented below and evaluated from a geotechnical perspective in terms of their respective advantages, disadvantages, risks and consequences. The evaluation is summarized in the table provided in Appendix E.

- Spread Footings

  The existing bridge abutment foundations consist of timber cribbing infilled with rockfill. The east abutment is founded on silty sand fill and native sand with gravel and the west abutment is founded on exposed bedrock. Supporting the new west or east bridge abutment on concrete spread footings constructed behind the existing foundations can be considered feasible at this site.

  The west abutment should be founded directly on the bedrock and the inclination of the bedrock surface will need to be reviewed during design once the location of the footing has been determined.

  Provided the new east abutment footing is adequately protected from scour and erosion the abutment could be founded on an engineered fill pad. Geotechnically, spread footings must be provided with adequate frost protection if not founded on bedrock. The excavation depth and limits for preparation of the footing subgrade should be reviewed to insure it would not destabilize any adjacent temporary or permanent footings. Spread footings not founded on bedrock will have a greater potential for settlement compared to deep foundations alternatives and the bridge structure would need to be designed to tolerate differential settlements.

- Caissons

  Caisson foundations, particularly when they are socketed into bedrock, offer high geotechnical resistance, however, their high lateral stiffness is not compatible with integral abutments. Cobbles and boulders will be encountered within the boreholes and permanent liners would be required to keep the drill holes open through the granular soils to allow for dewatering for inspection of the base of the caissons. Caissons are not recommended at this site.

- Steel Piles

  Steel piles are not recommended at the west abutment due to the shallow depth of bedrock and the resulting short length of pile.

  At the east abutment, steel H-piles driven to bedrock with a rock point tip are considered feasible and are recommended. Driven piles at the east abutment will reduce the volume of excavation required, limit the interaction with the existing foundations and will be less susceptible to scour and erosion when compared to shallow foundation alternatives. Pre-drilling through cobbles and boulders encountered within the existing soils may be required to install some of the piles. There exists a likelihood for misalignment during pile driving to bedrock.

Based on the proposed structure geometry and the evaluation of foundation alternatives presented above, a spread footing founded on bedrock is considered a feasible and cost effective option and is recommended at the west abutment. It is recommended to found the east abutment on driven piles, however, founding the east abutment on an engineered pad is also considered a feasible option.

10 FOUNDATIONS DESIGN RECOMMENDATIONS

10.1 Geotechnical Resistance for Spread Footings

The geotechnical bearing resistances provided in this report for spread footings include a resistance factor of 0.5 (ϕ_{gu}) and 0.8 (ϕ_{gs}) for the ULS and SLS values, respectively, as per Table 6.2 of the CHBDC (static analysis – typical understanding). The geotechnical resistances presented herein are for vertical concentric loading only on cast-in-place footings and will need to be adjusted for the effects of inclined or eccentric loadings, where applicable, in accordance with CHBDC Clause 6.10.3 and 6.10.4.

10.1.1 Spread Footings on Bedrock at the West Abutment

The depth to bedrock in the boreholes advanced at the west abutment was noted to range from 3 to 5.3 m below the existing road grade. The existing overburden should be excavated and the spread footing should be founded directly on the bedrock. The lowest elevation of bedrock was at elevation 399.4 m which is 0.9 m below the water level noted during the time of the field investigation. Where bedrock is exposed it should be inspected and excavated to create a horizontal surface or alternatively, the founding elevation can be raised with the use of a concrete plug in accordance with OPSS.PROV 904 with the same class of concrete as the footing to reduce the excavation and dewatering efforts.

A spread footing at the west abutment founded on the bedrock can be designed with a factored geotechnical resistance at ULS of 1000 kPa. SLS will not govern design for a footing founded on bedrock. Moving the footing closer towards the river could negatively impact the geotechnical resistance due to slope effects.

The horizontal resistance against sliding between a cast-in-place concrete footings founded on bedrock can be computed using a friction factor of 0.70. Appropriate resistance factors should be applied for the design. Alternatively, anchors or shear pins could be used to provide additional capacity and Thurber can provide values upon request.

10.1.2 Spread Footings on Native Soils at the East Abutment

The existing fill materials at the east abutment are not considered suitable for directly supporting a spread footing for the replacement structure. A spread footing at the east abutment founded on the undisturbed native sand deposit at or below the depth of frost (Section 10.6) can be designed with the geotechnical resistances provided in the table below.

Table 10-1 Bearing Resistances for Spread Footings at the East Abutment

Footing Width (m)	Factored Resistance	
	ULS (kPa)	SLS (kPa)
2	225	170
3	250	160
4	275	150

The geotechnical ULS resistance for footings positioned closer than two equivalent footing widths from the forward slope will need to be reduced and Thurber can provide these values upon request. The geotechnical SLS resistance values given above are based on an estimated total settlement not exceeding 25 mm. This settlement is expected to be substantially completed by the end of construction. Differential settlement is not expected to exceed 15 mm across the width of the structure for subgrades prepared with good workmanship. Differential settlement from the west to east abutments would be equal to the total settlement of the east foundation or 25 mm.

The founding elevation is expected to be above the groundwater and river level observed during the time of the field investigation. If temporary excavation is required to construct the footing extends below the water level, local groundwater control will be required to construct the footings in the dry and to prevent disturbance of the footing base. Excavations and backfilling of the foundation should be carried out in accordance with OPSS 902.

The horizontal resistance against sliding between a cast-in-place concrete footings founded on the undisturbed native soil at the founding elevation can be computed using a friction factor of 0.45. Appropriate resistance factors should be applied for the design.

10.1.3 Spread Footings on Engineered Fill at the East Abutment

An engineered pad consisting of Granular ‘A’ material can be constructed at the east abutment if a bearing resistance greater than those provided in Table 10-1 is required. The founding elevation of the base of the footing should be at or below the depth of frost. The engineered pad can bear on the exposed subgrade provided it is free of any soft or deleterious materials and should be placed on a geotextile (Class II non-woven FOS 50 to 150 μm, OPSS 1860). The top of the Granular ‘A’ pad must extend to 1.0 m beyond the surface of the edge of all sides of the footing and be sloped away from the footing at 1H:1V, or flatter. The following factored geotechnical resistance values are recommended for a 2 m wide cast-in-place footings founded on a 1.0 m thick engineered fill pad at this site:

- Factored Geotechnical Resistance at ULS of 350 kPa
- Factored Geotechnical Resistance at SLS of 225 kPa

The geotechnical ULS resistance for footings positioned closer than two equivalent footing widths from the forward slope will need to be reduced and Thurber can provide these values upon request. The geotechnical SLS resistance values given above are based on an estimated total settlement not exceeding 25 mm. This settlement is expected to be substantially completed by the end of construction. Differential settlement is not expected to exceed 15 mm across the width of the structure for subgrades prepared with good workmanship. Differential settlement from the west to east abutments would be equal to the total settlement of the east foundation or 25 mm.

The horizontal resistance against sliding between a cast-in-place concrete footings founded on engineered fill can be computed using a friction factor of 0.55. Appropriate resistance factors should be applied for the design.

10.2 Geotechnical Resistance for Driven Piles to Bedrock at the East Abutment

For a summary of bedrock elevations at the investigated locations, please refer to Table 5-3. The axial geotechnical capacity at factored ULS for Steel H-Piles (HP 310x110) driven to refusal on bedrock is 2000 kN/pile. This value reflects the poor condition of the bedrock in the initial core run at Borehole 16-06 near the east abutment as reflected in the RQD values. The pile capacity includes a resistance factor of 0.4 (ϕ_{gu}) for ULS as per Table 6.2 of the CHBDC (static analysis – typical understanding). The geotechnical resistance values assume a minimum center-to-center spacing of three pile diameters; the resistance values will need to be reduced for a lesser pile spacing. The SLS condition will not govern for piles driven to bedrock. The structural resistance of the piles must be checked by the structural designer and the lower of the structural and geotechnical capacities should govern. The pile installation should be in accordance with OPSS 903.

10.2.1 Pile Tips

It is expected that pile installation will encounter cobbles and boulders. The Contractor should be prepared to pre-auger or predrill to facility driving pile to bedrock. Care must be exercised while driving to bedrock and the tips of all piles must be protected from damage when driving. Due to the presence of sloping bedrock, the tips of all piles should be fitted with a Titus HD Rock Injector, APF Hard-Bite point or approved equivalent.

10.2.2 Pile Driving

Pile driving must be carried out in accordance with OPSS 903.

It should be recognised that there exists a risk that piles driven into soils containing cobbles and boulders may not meet the specified deviation limits at the top of the piles. If tighter horizontal deviations allowance is required, a driving template or other means may be required.

10.2.3 Downdrag

Downdrag on piles is not considered to be an issue at this site, since the native deposits contain a low clay content.

10.2.4 Pile Lateral Resistance

The geotechnical lateral resistance that can be mobilized in front of an H-Pile may be analysed using a soil-spring model and computed using a value for the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}). The value of k_s varies with depth and may be calculated as follows:

$$k_s = n_h * z / D$$

$$p_{ult} = 3 * \gamma' * z * K_p$$

where:

n_h = coefficient related to soil density, see table below (kN/m^3)
z = depth of embedment of pile (m)
D = pile diameter (m)
γ' = effective unit weight of soil, see table below (kN/m^3)
K_p = passive earth pressure coefficient, see table below (-)

The parameters recommended for the use with the above equations is provided below in Table 10-2.

Table 10-2 Parameters for Lateral Pile Resistance

Location	Elevation (m)	Unit Weight$^{(*)}$ (kN/m^3)	n_h (kPa/M)	K_p (-)	Soil
East Abutment (Borehole 16-06)	401.9 – 401.6	21	5,100	3.3	Fill
	401.6 – 400.3	20	4,000	3.0	Sand
	400.3 – 400.0	11	2,500	3.0	Sand
	400.0 – 396.5	11	2,500	3.0	Silty Sand

Note: (*) submerged unit weights have been provided for calculations below the water table

The above equations and recommended parameters may be used to analyze the interaction between a pile and the surrounding soil. The factored lateral resistance of the piles determined based on the data and methods provided above should incorporate a resistance factor (ϕ_{gu}) of 0.5 as per Table 6.2 of the CHBDC (static analysis – typical understanding). The lateral pressures obtained from the analysis should not exceed the ultimate lateral resistance.

The spring constant, K_s, for analysis may be obtained by the expression, $K_s = k_s * L * D$ (kN/m), where L is the length (m) of the pile segment or element used in the analysis and the remaining parameters are as defined earlier. The ultimate lateral resistance, P_{ult}, on any one segment of pile may be obtained from the expression,

$P_{ult} = p_{ult} * L * D$. This represents the ultimate load at which the pile fails and will not support any additional load at greater displacements. However, it is recommended that the total lateral resistance for one pile be limited to no more than 100 kN at ULS and 35 kN as SLS.

The coefficient of horizontal subgrade reaction may have to be reduced, based on the pile center-to-center spacing less than 4 pile diameters. The reduction factors to be used for a pile group oriented perpendicular or parallel to the direction of loading are provided in Figures C6.11.3(r), C6.11.3(s) and C6.11.3(t) of the CHBDC. Alternatively, horizontal loads may also be resisted by means of battered piles. A frictional horizontal contribution of piles at the bedrock interface should not be included in the lateral stability calculations.

10.3 Wingwalls

If wingwalls are required as part of the bridge design, the footings should be founded on a leveling pad with a minimum thickness of 0.5 m consisting of Granular ‘A’ material with the base of the wingwall at or below the depth of frost (Section 10.6). The engineered pad can bear on the native subgrade or existing fill materials provided that it is undisturbed, uniformly competent and free of any soft and deleterious materials. The top of the Granular ‘A’ pad must extend to 0.5 m beyond the outside edge of all sides of the footing and sloped away from the footing at 1H:1V. The following factored geotechnical resistance values are recommended for wingwalls with a footing width of 1 to 2 m at this site:

- Factored Geotechnical Resistance at ULS of 250 kPa
- Factored Geotechnical Resistance at SLS of 175 kPa

Higher bearing resistance can be obtained, if required, by increasing the thickness of the Granular ‘A’ pad.

Considering the competency of the foundation soils, settlement of the foundation soils under the loading imposed by the wingwalls is expected to be negligible provided additional fill is not placed above the current grades.

10.4 Subgrade Preparation, Bedding and Backfilling

Subgrade preparation for the abutment and wingwall foundations should include the removal of the existing granular fill and any loose, soft or organic materials within the footprint of the proposed foundation.

The base the excavations should be inspected by qualified geotechnical personnel in accordance with SP109S12 prior to placing concrete and/or 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 and wingwall 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 compacted as per OPSS.PROV 501.

10.5 Backfill and Earth Pressure

Structural backfill material should consist of Granular A, or Granular B Type II meeting OPSS.PROV 1010 specifications. The backfill must be in accordance with OPSS 902 and placed to the extents shown on OPSD 3101.150.

The backfill should be compacted and compaction equipment to be used adjacent to the walls should be restricted in accordance with OPSS.PROV 501. The design of the abutments and wingwalls must incorporate a subdrain as shown in OPSD 3101.150. If adequate drainage cannot be confirmed, the potential of hydrostatic pressures should be considered.

10.5.1 Static Lateral Earth Pressure Coefficients

Lateral earth pressures acting on structures should be computed in accordance with the CHBDC but under fully drained conditions is generally given by the expression:

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

where:

σ_h	=	static lateral earth pressure on the wall at depth d (kPa)
K	=	static earth pressure coefficient (see table below)
γ	=	unit weight of retained soil (see table below) use submerged unit weight below water
d	=	depth below top of fill where pressure is computed (m)
q	=	value of any surcharge (kPa)

A lateral earth pressure due to backfill compaction should be added to the calculated lateral earth pressure in accordance with Clause 6.12.3 of the CHBDC. The recommended lateral earth pressure parameters for use in the design for a vertical structure are provided in Table 10-4.

If lateral movement is not permissible and/or the wall is restrained from lateral yielding, the at rest pressure coefficient should be used. If the wall design allows lateral yielding (non-rigid structure or wingwall), the active earth pressure coefficient may be used. Passive earth resistance in front of the structure should be ignored. Where ground surfaces are sloped behind the walls, the corresponding coefficients should be used.

Table 10-3 Static Lateral Earth Pressure Coefficients

Condition	Earth Pressure Coefficient (K)			
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^o$, $\gamma = 22.8$ kN/m^3		Existing Granular Fill or OPSS Granular B Type I $\phi = 32^o$, $\gamma = 21.2$ kN/m^3	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active, K_A (Yielding Wall)	0.27	0.40	0.31	0.48
Active, K_o (Non-Yielding Wall)	0.43	-	0.47	-
Active, K_P (Movement towards soil mass)	3.7	-	3.3	-
Soil Group$^{(*)}$	'medium dense sand'		"loose to medium dense sand"	

Note: (*) for use with Figure C6.16 of the commentary to the CHBDC

The parameters in the table correspond to full mobilization of active and passive earth pressure and require certain relative movements between the wall and adjacent soil to produce these conditions. The values used in design can be assessed from Figure C6.16 of the Commentary to the CHBDC using the soil group designate as outlined in the Table.

10.5.2 Combined Static and Seismic Lateral Earth Pressure Parameters

Retaining structures should be designed using dynamic earth pressure coefficients that incorporate the effects of earthquake loading. 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 = ½ F(PGA)•PGA for structures that 25 mm to 50 mm of movement, and
- k_h = F(PGA)•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 of vertical walls are provided in Table 10-4. The provided earth pressure coefficients are based on a Seismic Site Class D, *reference* PGA with a 2% probability of exceedance in 50years of 0.043g (Geological Survey of Canada – Fifth Generation) and a F(PGA) of 1.29 as per Table 4.8 of the CHBDC (S6-14).

Table 10-4 Dynamic Lateral Earth Pressure Coefficients

Condition	Earth Pressure Coefficient (K)			
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^o$, $\gamma = 22.8$ kN/m^3		OPSS Granular B Type I $\phi = 32^o$, $\gamma = 21.2$ kN/m^3	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active, K_{AE} Yielding Wall	0.28	0.42	0.32	0.51
Active, K_{AE} Non-Yielding Wall	0.30	0.45	0.33	0.54

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:

σ_h = lateral earth pressure on wall at depth d (kPa)
d = depth below the top of the wall where pressure is computed (m)
K = static earth pressure coefficient
(K_a for yielding walls, K_o for non-yielding walls)
γ = unit weight of retained soil
use submerged unit weights below water
K_{AE} = combined static and seismic earth pressure coefficient
H = total height of the wall (m)

10.6 Frost Depth

The frost penetration depth at this site is 2.7 m as per OPSD 3090.100. Footings founded on sound bedrock or founded on mass concrete which is on sound bedrock, do not require frost protection. For all other footings and pile caps, a minimum of 2.7 m of earth cover, or thermal equivalent, must be provided above the base of the footing and pile cap to serve as protection against frost. Thermally equivalent frost protection could be in the form of polystyrene insulation provided it is placed above the highwater level.

10.7 Cement Type and Corrosion Potential

Analytical analyses were 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 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. The class of concrete selected should consider the effects of road de-icing salts.

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 5-4 may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects. The effects of road de-icing salts should also be considered.

10.8 Embankment Design and Reinstatement

10.8.1 Embankment Reconstruction

Embankment reconstruction should be carried out in accordance with OPSS.PROV 206. The embankment should be reinstated with side slopes of 2H:1V (or flatter) if constructed using Select Subgrade Material (SSM) or Granular B Type I.

Where new embankment fill is placed against existing embankment slopes or on a sloping ground surface steeper than 3H:1V, benching of the existing slope should be carried out in accordance with OPSD 208.010.

10.8.2 Embankment Settlement and Stability

It is understood that no grade raise or embankment widening is required and provided that proper construction methods are used, no long term or global stability issues are anticipated for embankments built at this site. Material stockpiling above the existing grades is a temporary construction measure and the stability implications should be reviewed by the contractor. In addition, the Contractor's selection and placement of construction equipment (such as heavy cranes) must be included in that stability assessment.

Since only a minor grade raise is anticipated along the alignment of Highway 11, negligible settlement of the soils beneath the embankment is expected to occur.

The magnitude of the embankment compression for embankments constructed with granular materials is in the order of 0.5% of the embankment height and is expected to occur following fill placement. Placement of the final lift of asphalt should be delayed for at least one month to improve performance.

10.8.3 Temporary Detour Structure

The foundation conditions and design recommendations for a temporary detour alignment along the north side of the highway alignment has been provided in Geocres 41O-30.

11 CONSTRUCTION CONSIDERATIONS

11.1 Excavation

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 soils above the water table at the site should be classified as Type 3 and Type 4 below the water table in accordance with OHSA. All excavations must not encroach within 1H:1V from the base of the excavation to the existing bridge foundation or to a temporary detour bridge support.

Excavation for the structure replacement must be carried out in accordance with OPSS 902. The sides of temporary excavations must be sloped in accordance with the requirements of OHSA. Selection of the equipment and methodology to excavate and prepare the founding surface is the responsibility of the Contractor. Stockpiling or surface surcharge should not be allowed within a horizontal distance encompassed within a 1H:1V inclination from the perimeter of the base of the excavation.

At locations where there is space restrictions or where a slope has to be retained, the excavations will need to be carried out within a protection system. Design of the temporary protection system is the responsibility of the Contractor.

11.2 Temporary Protection Systems

It is understood that a full road closure will be utilized during construction and therefore a temporary protection system (TPS) is not anticipated. However, if a TPS is required as part of construction activities, the design of the TPS is the responsibility of the Contractor and all TPS's should be designed by a licensed Professional Engineer experienced in such design and retained by the Contractor. Temporary protection systems should be provided in accordance with OPSS.PROV 539 and designed for Performance Level 2 (maximum 25 mm horizontal deflection). The actual pressure distribution acting on the shoring systems is a function of the construction sequence and relative flexibility of the wall and these factors must be considered when design the shoring system. Thurber can provide geotechnical parameters upon request. The Contractor should be made aware that cobbles and boulders were encountered within the boreholes.

11.3 Dewatering

Subgrade preparation and placement of granular or mass concrete pads and abutments 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.

Based on the high-water level and the proposed footing and pile cap elevations, excavation to construct the abutments will not extend below the river level. Nonetheless, 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. Water from surface flow and/or groundwater flow 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.

Pumping with sump pumps will be required in order to maintain a dry excavation. The groundwater level should be lowered at least 0.5 m below the excavation elevation at the east abutment. Dewatering and surface water diversion must remain operational and effective until the temporary excavation is backfilled. Dewatering systems must be designed by a dewatering specialist and should be designed, operated and removed in accordance with OPSS.PROV 517 and Special Provision No. 517F01 with the following inputs for Table A: Note 1 = Yes and ****** = 100 m. The assessment for the need for a Permit to take Water (PTTW) should be carried out by a specialist experienced in this field.

11.4 Scour Protection and Erosion Control

Slope protection and drainage measures will be required to ensure the long-term surficial stability of the embankment slopes. The embankment material primarily consisting of sand and gravel, the native sand and granite bedrock are all considered to have a low erosion potential. The native silty sand is considered to have a low to moderate erosion potential. Slope vegetation should be established as soon as possible after completion of the earth embankment fills in order to control surficial erosion in general accordance with OPSS.PROV 804. The contractor should provide silt fences and erosion control blankets, as required, throughout the duration of the construction to prevent silt/sediment from running off the site as per OPSS 805.

Scour and erosion protection should be provided along the banks in the area of the new bridge. Consideration should be given to leaving the existing rockfilled timber cribbing in place to reduce the disturbance to the existing conditions and provide protection to the new abutments. Design of the scour and erosion protection measures must consider hydrologic and hydraulic concerns and should be carried out by specialists experienced in the field. Typically, rock protection should be provided over all earth surfaces subjected to flowing water in accordance with OPSS 511.

12 CONSTRUCTION CONCERNS

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

- Buried obstructions may be encountered during construction in the existing approach embankments.
- Seasonal fluctuations of the groundwater and river level are to be expected which may impact the construction and dewater scheme.
- Interference with the existing timber cribbing and disturbance to the existing slopes.
- 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).

The successful performance of the bridge will depend largely upon good workmanship and quality control during construction. Subgrade examination and field density testing should be carried out by qualified geotechnical personnel during construction in accordance with SP109S12 to confirm that foundation recommendations are correctly implemented, and material specifications are met.

13 CLOSURE

Engineering analysis and preparation of this report were carried out by Dr. Fred Griffiths, P.Eng and Mr. Stephen Peters, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng a Designated Principal Contact for MTO Foundation Projects.

Thurber Engineering Ltd.
Report Prepared By:

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Geotechnical Engineer

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Senior Associate
Senior Geotechnical Engineer

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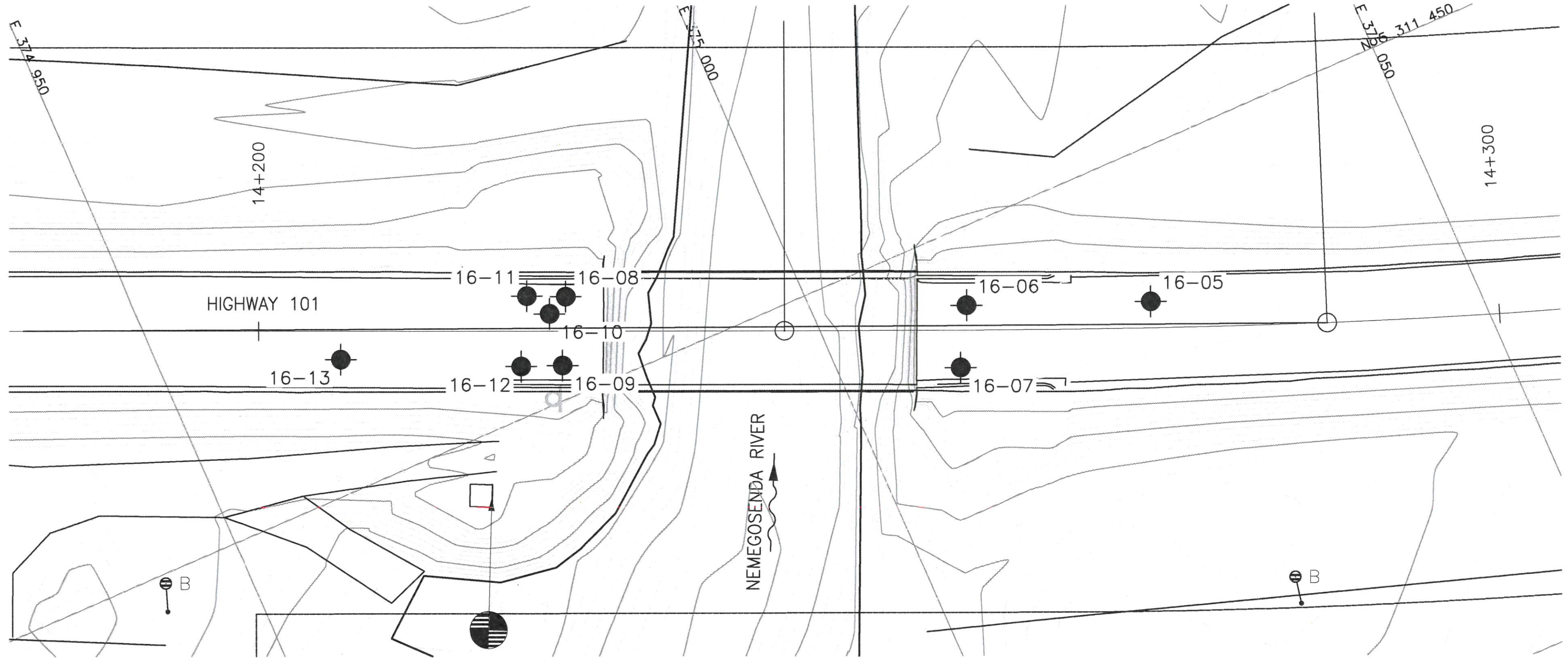
Review Principal,
Senior Geotechnical Engineer

Appendix A.

Borehole Location Plan and Stratigraphic Drawings



PLAN

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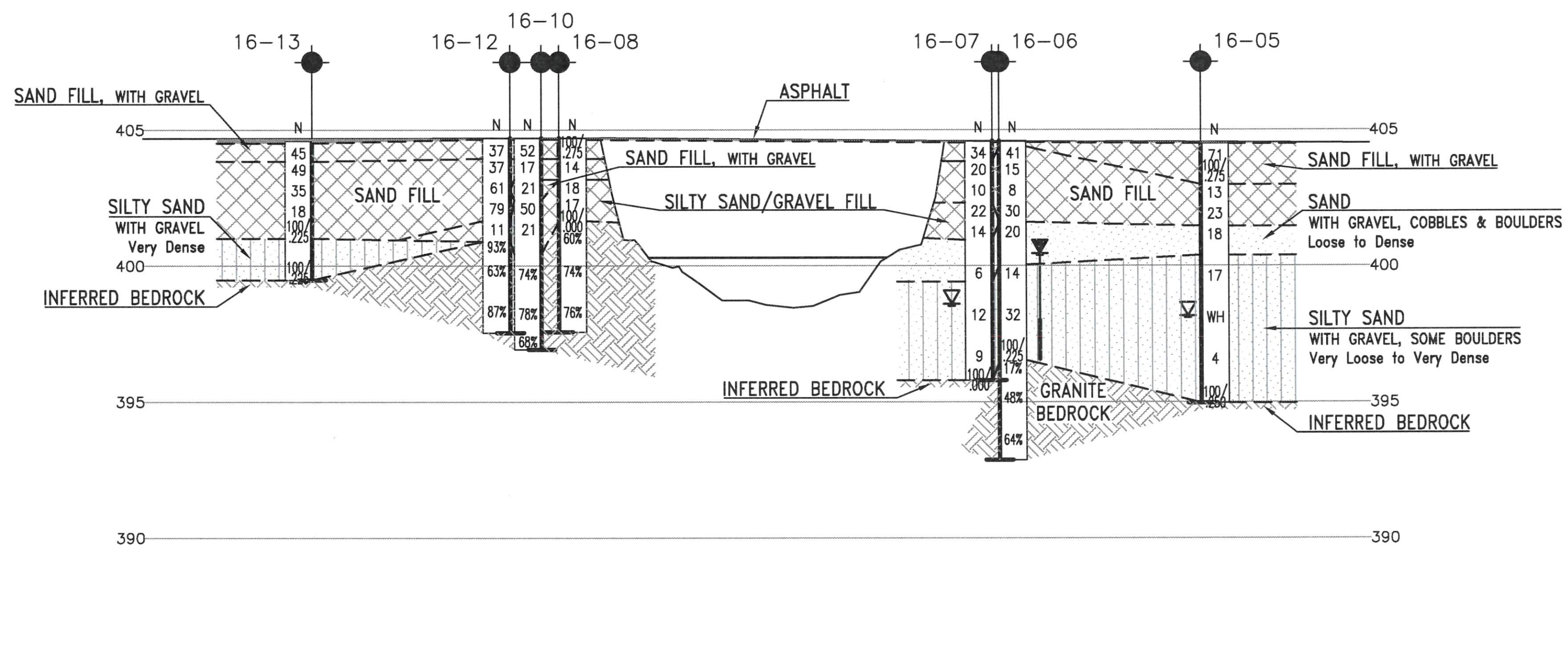

PROFILE

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V 1:250

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

LICENSED PROFESSIONAL ENGINEER P. K. CHATTERJI Oct 19/18 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER F. J. GRIFFITHS Oct 25/18 PROVINCE OF ONTARIO

CONT No

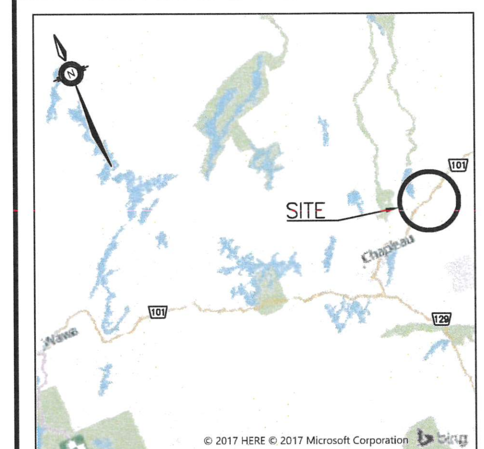
GWP No 5144-10-00

HIGHWAY 101
NEMEGOSENDA RIVER
BRIDGE REHABILITATION
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

McINTOSH PERRY

THURBER ENGINEERING LTD.



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
16-05	404.5	5 311 438.4	375 025.3
16-06	404.6	5 311 444.1	375 011.6
16-07	404.6	5 311 439.7	375 009.2
16-08	404.7	5 311 457.6	374 982.4
16-09	404.7	5 311 452.6	374 979.9
16-10	404.7	5 311 456.9	374 980.6
16-11	404.7	5 311 458.9	374 979.5
16-12	404.7	5 311 453.9	374 976.8
16-13	404.6	5 311 460.2	374 963.7

-NOTES-

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

GEOCRES No. 41O-29

REVISIONS	DATE	BY	DESCRIPTION

DESIGN JG	CHK –	CODE	LOAD	DATE OCT 2018	
DRAWN MFA	CHK JG	SITE 46-215	STRUCT	DWG 1	

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PLOTDATE: 10/25/2018 11:55 AM

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

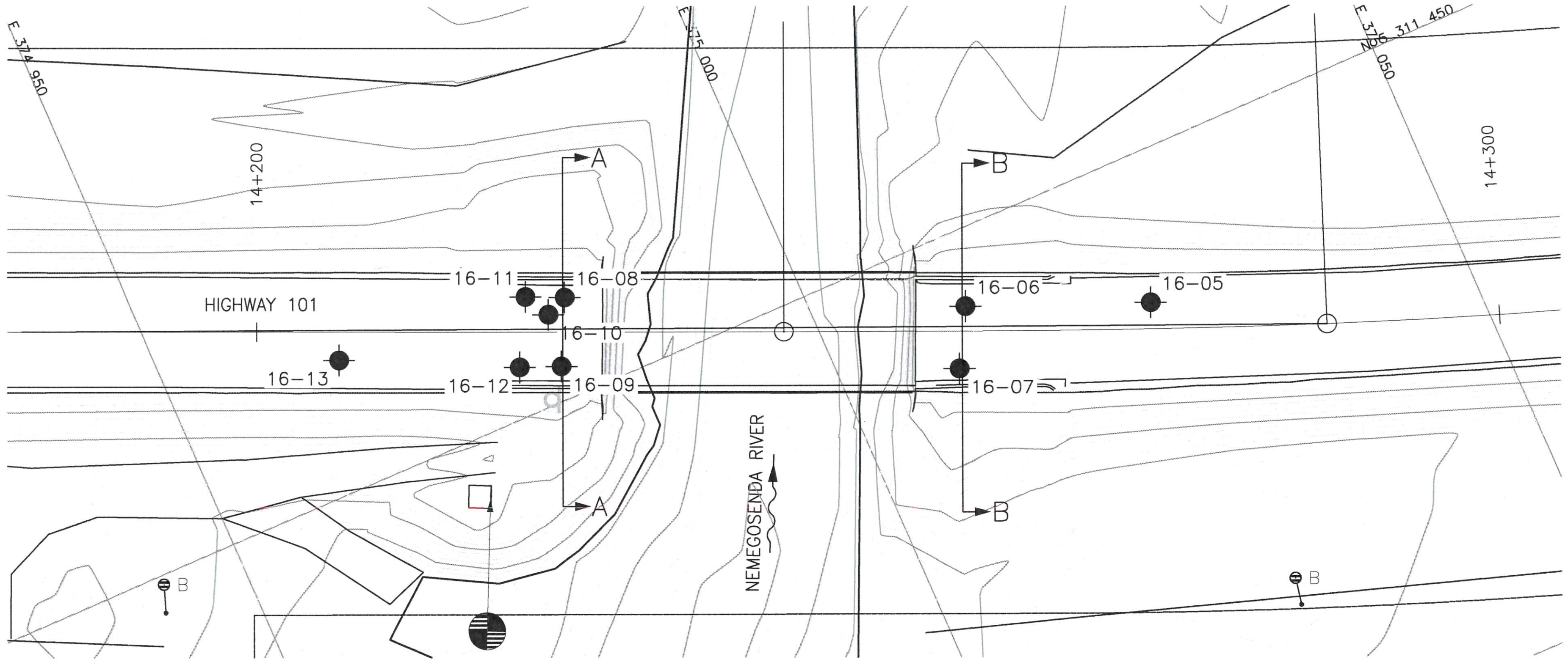
CONT No
GWP No 5144-10-00

HIGHWAY 101
NEMEGOSENDA RIVER
BRIDGE REHABILITATION
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

McINTOSH PERRY

THURBER ENGINEERING LTD.



PLAN

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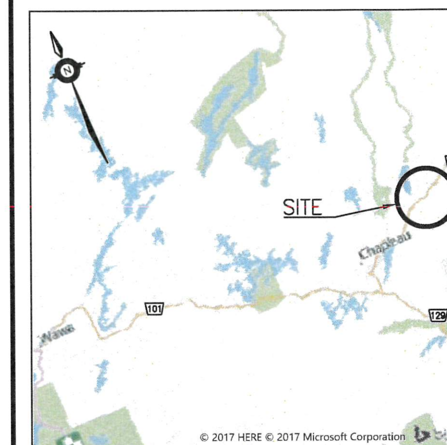
SCALE 1:500

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
Oct 19/18
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
F. J. GRIFFITHS
Oct 25/18
PROVINCE OF ONTARIO



KEYPLAN

LEGEND

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

NO	ELEVATION	NORTHING	EASTING
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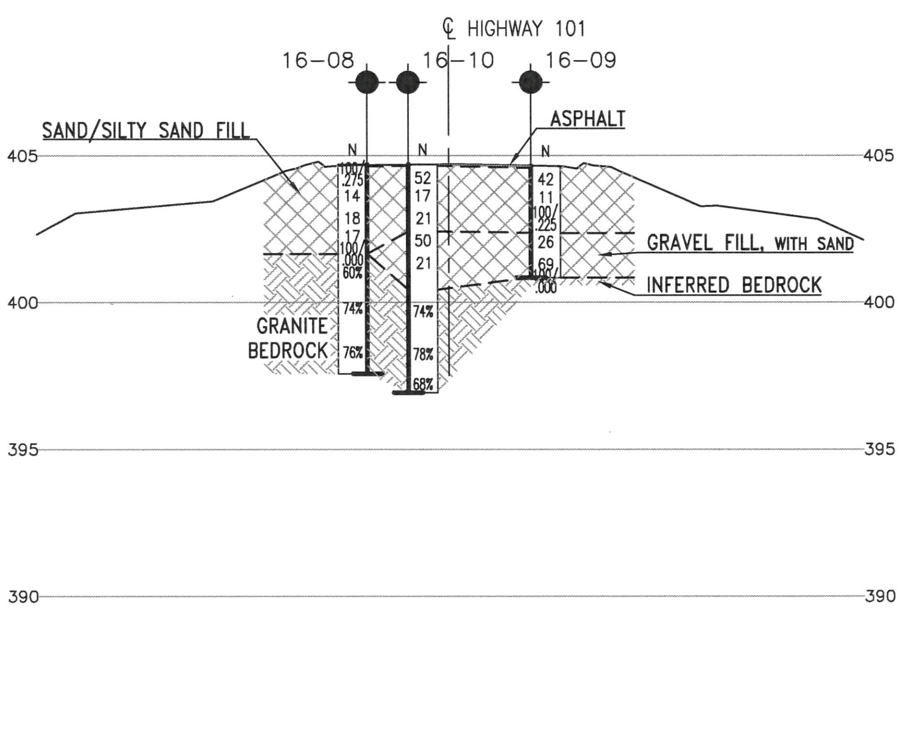
16-07	404.6	5 311 439.7	375 009.2
16-08	404.7	5 311 457.6	374 982.4
16-09	404.7	5 311 452.6	374 979.9
16-10	404.7	5 311 456.9	374 980.6
16-11	404.7	5 311 458.9	374 979.5
16-12	404.7	5 311 453.9	374 976.8
16-13	404.6	5 311 460.2	374 963.7



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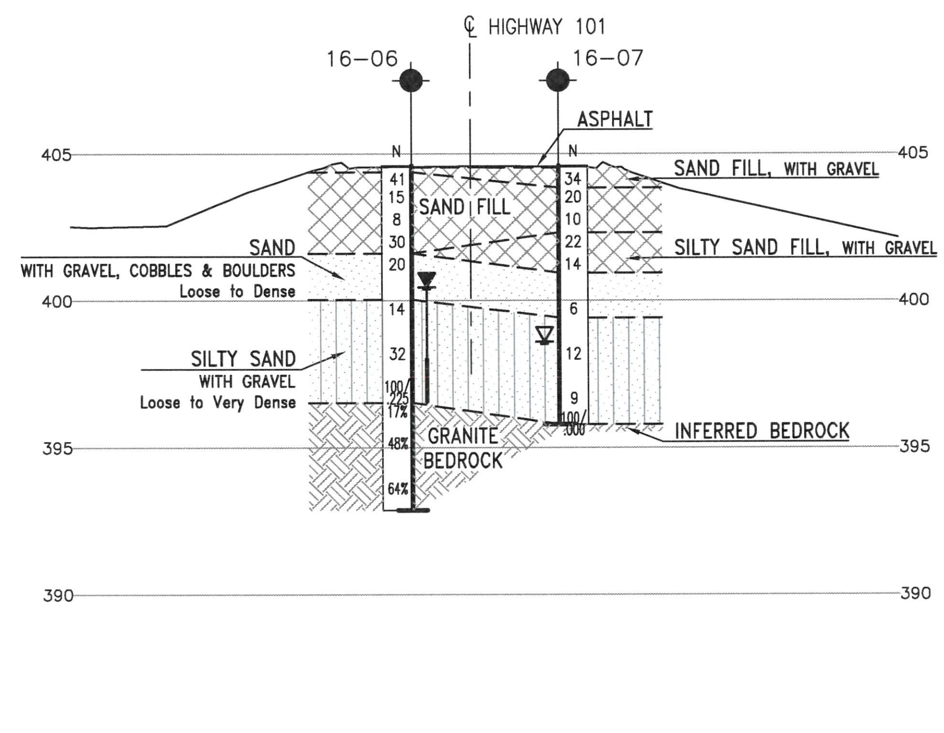

SECTION B-B

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-NOTES-

1) The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
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GEOCRES No. 41O-29

REVISIONS	DATE	BY	DESCRIPTION		
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MINISTRY OF TRANSPORTATION, ONTARIO

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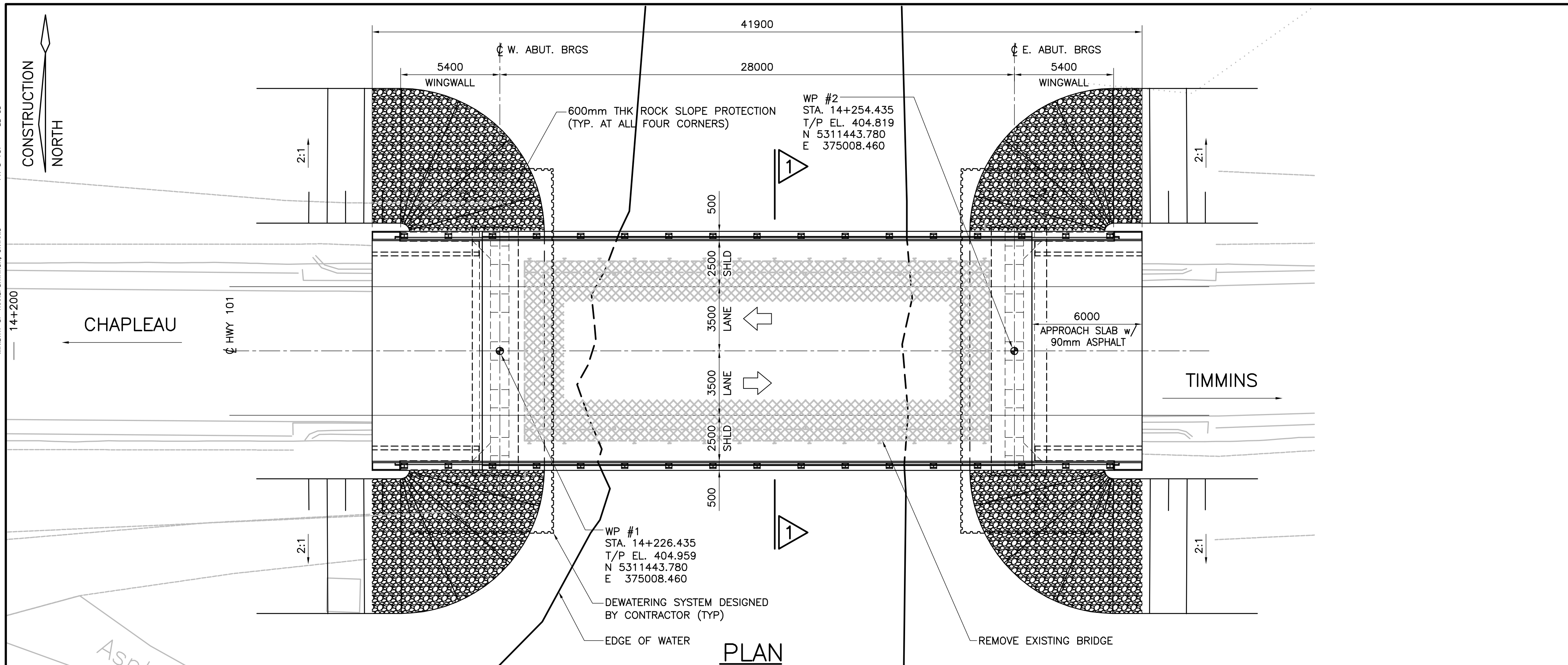
	McINTOSH PERRY	



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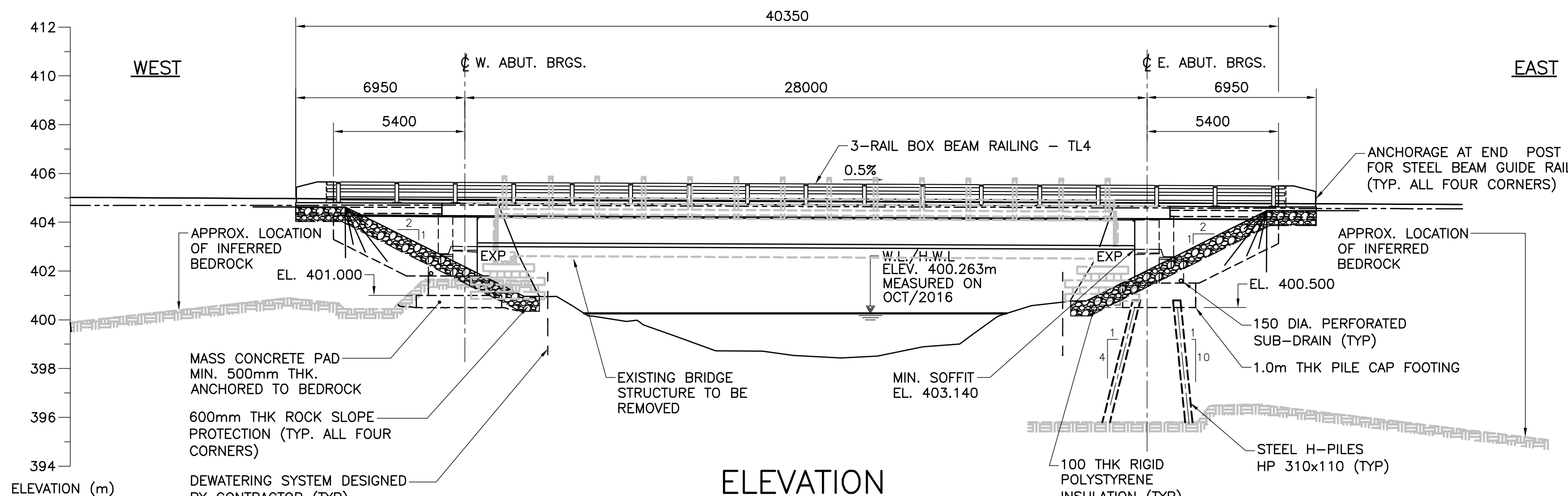

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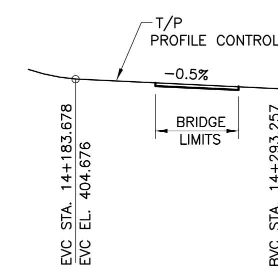
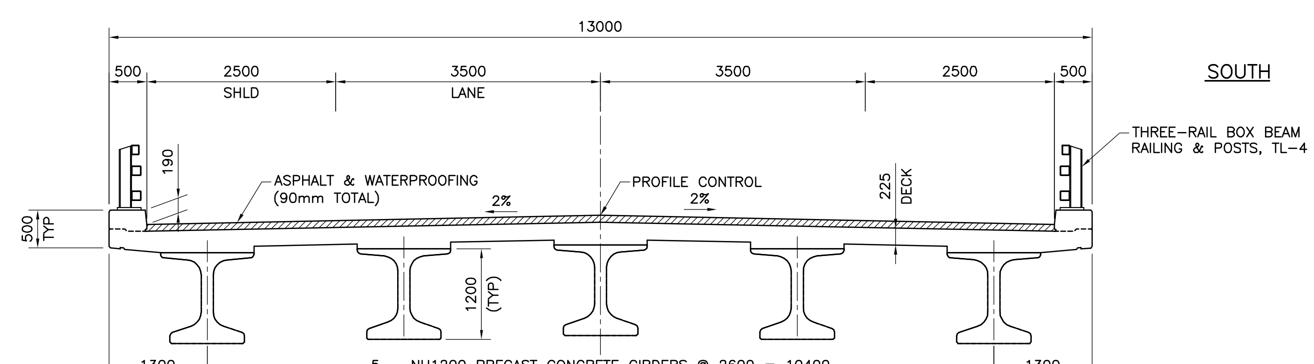


PROFILE OF HWY. 101
N. T. S.



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GENERAL NOTES:

1. CLASS OF CONCRETE

30 MPa

CLASS OF CONCRETE FOR PRECAST GIRDERS ARE GIVEN ON PRESTRESSED DRAWINGS.

2. CLEAR COVER TO REINFORCING STEEL

FOOTING	100 ± 25
DECK TOP	70 ± 20
BOTTOM	40 ± 10
PIER CAP	70 ± 10
REMAINDER UNLESS NOTED OTHERWISE	70 ± 20

3. REINFORCING STEEL

1. REINFORCING STEEL SHALL BE GRADE 400W.
2. UNLESS SHOWN OTHERWISE, TENSION LAPS SPLICES FOR REINFORCING STEEL BARS SHALL BE CLASS B.
3. STAINLESS STEEL REINFORCING SHALL BE TYPE 316LN OR DUPLEX 2205 AND HAVE A MINIMUM YIELD STRENGTH OF 500 MPa, UNLESS OTHERWISE SPECIFIED.
4. BAR MARK WITH PREFIX 'S' DENOTE STAINLESS STEEL BARS.
5. BAR HOOKS SHALL HAVE STANDARD HOOK DIMENSIONS USING MINIMUM BEND DIAMETERS, WHILE STIRRUPS AND TIES SHALL HAVE MINIMUM HOOK DIMENSIONS. ALL HOOKS SHALL BE IN ACCORDANCE WITH THE STRUCTURAL STEEL DRAWING SS12.1 UNLESS INDICATED OTHERWISE.

4. CONSTRUCTION NOTES

1. CONTRACTOR SHALL OBTAIN LOCATES PRIOR TO PROCEEDING WITH CONSTRUCTION OPERATIONS.
2. THE CONTRACTOR SHALL ESTABLISH THE BEARING SEAT ELEVATIONS BY DEDUCTING THE ACTUAL BEARING THICKNESS FROM THE TOP OF BEARING ELEVATIONS. IF THE ACTUAL BEARING THICKNESS ARE DIFFERENT FROM THOSE GIVEN WITH THE BEARING DESIGN DATA, THE CONTRACTOR SHALL MAKE ADJUSTMENTS TO SUIT.
3. THE ROADWAY WILL BE CLOSED FOR THE FULL DURATION OF THE BRIDGE CONSTRUCTION.
4. INFORMATION OF EXISTING STRUCTURE SHOWN WAS TAKEN FROM THE ORIGINAL DESIGN DRAWINGS. THE CONTRACTOR SHALL VERIFY ALL RELEVANT DIMENSIONS, ELEVATIONS, STATIONS AND DETAILS ON SITE AND REPORT ANY DISCREPANCIES TO THE DESIGN ENGINEER PRIOR TO PROCEEDING WITH THE CONSTRUCTION OF THE NEW BRIDGE.
5. BACKFILL SHALL NOT BE PLACED BEHIND THE ABUTMENTS UNTIL THE DECK SLAB IS IN PLACE AND HAS REACHED 70% OF ITS DESIGN STRENGTH. BACKFILL SHALL BE PLACED SIMULTANEOUSLY BEHIND BOTH ABUTMENTS KEEPING THE HEIGHT OF THE BACKFILL APPROXIMATELY THE SAME. AT NO TIME SHALL THE DIFFERENCE IN ELEVATION BE GREATER THAN 500mm.
6. THE CONTRACTOR IS RESPONSIBLE FOR DEVELOPING AND PROVIDING TEMPORARY SUPPORT SYSTEMS AND DEWATERING SYSTEMS FOR THE SAFE REMOVAL OF THE EXISTING STRUCTURE AND THE CONSTRUCTION OF THE NEW ABUTMENTS AND THE NEW SUPERSTRUCTURE AS SHOWN ON THE DRAWINGS.
7. THE CONTRACTOR SHALL PROVIDE DEBRIS PLATFORM AND NECESSARY CONTAINMENT MEASURES TO COLLECT FALLING CONCRETE AND CONSTRUCTION DEBRIS SUCH THAT NO DEBRIS OR MATERIALS RESULTING FROM THE STRUCTURE REMOVAL AND RECONSTRUCTION WORK FALLS ON THE WATERWAY BELOW OR OTHER AREAS ON THE BRIDGE SITE.

DRAWING LIST:

1. GENERAL ARRANGEMENT
2A. BOREHOLE SOIL STRATA I
2B. BOREHOLE SOIL STRATA II
3. REMOVALS
4. FOUNDATION LAYOUT AND DETAILS
5. ABUTMENTS AND WINGWALL I
6. ABUTMENTS AND WINGWALLS II
7. BEARING DETAILS
8. PRESTRESSED NU GIRDER 1 (SSD)
9. PRESTRESSED NU GIRDER 2 (SSD)
10. DECK CONSTRUCTION
11. DECK REINFORCEMENT
12. BARRIER WALL (SSD)
13. TWO TUBE RAILING ON CURB – TL4 (SSD)
14. 6000mm APPROACH SLABS (SSD)
15. STANDARD DETAILS

APPLICABLE DRAWING STANDARDS:

OPSD 3101.150	WALLS, ABUTMENT, BACKFILL, MINIMUM GRANULAR REQUIREMENT
OPSD 3102.100	WALLS, ABUTMENT, BACKFILL DRAIN
OPSD 3370.100	DECK, WATERPROOFING HOT APPLIED ASPHALT MEMBRANE WITH PROTECTION BOARD
OPSD 3370.101	DECK, WATERPROOFING HOT APPLIED ASPHALT MEMBRANE AT ACTIVE CRACKS GREATER THAN 2mm WIDE AND CONSTRUCTION JOINTS
OPSD 3419.100	BARRIERS AND RAILINGS, STEEL BEAM, GUIDE RAIL AND CHANNEL ANCHORAGE
OPSD 3941.200	FIGURES IN CONCRETE, SITE NUMBER AND DATE LAYOUT

LIST OF ABBREVIATIONS

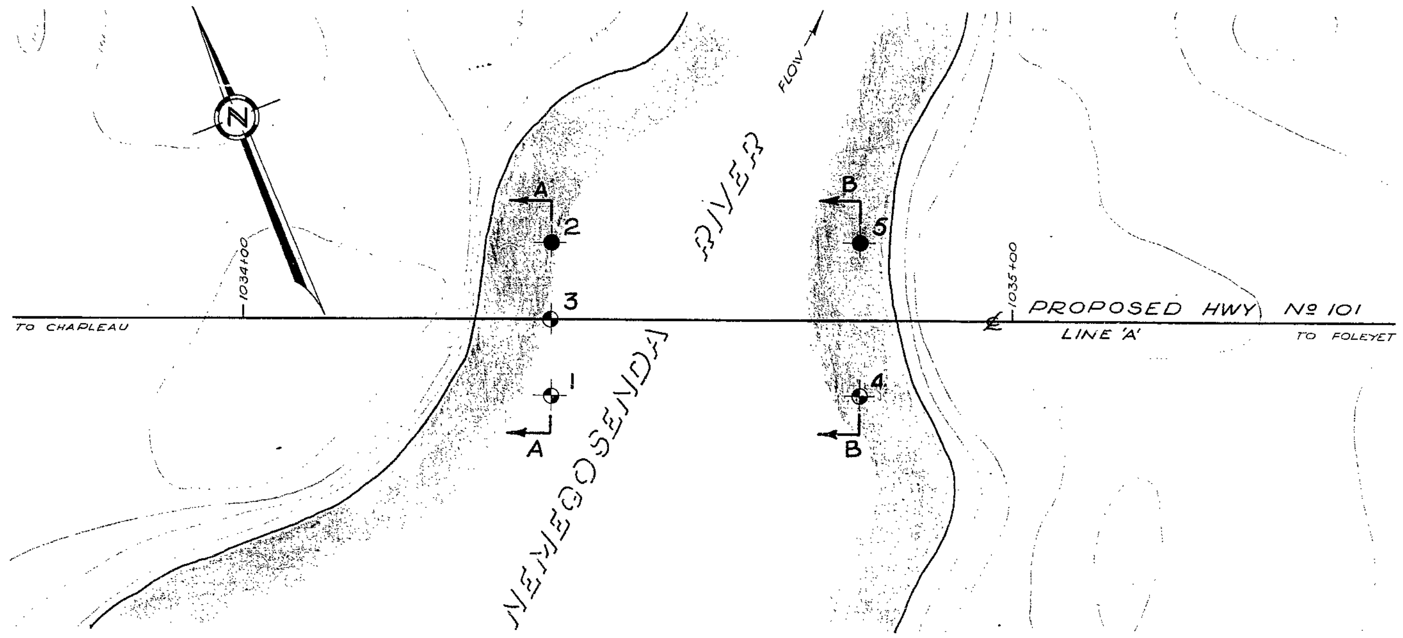
WP	DENOTES WORKING POINT
T/P	DENOTES TOP OF PAVEMENT
T/	DENOTES TOP OF
ABUT	DENOTES ABUTMENT
BRGS	DENOTES BEARINGS
SHLD	DENOTES SHOULDER
U/S	DENOTES UNDERSIDE
CSP	DENOTES CORRUGATED STEEL PIPE
TUL	DENOTES TOP UPPER LAYER
TLL	DENOTES TOP LOWER LAYER
BOT	DENOTES BOTTOM
EF	DENOTES EACH FACE
ES	DENOTES EACH SIDE
EQ SPCD	DENOTES EQUALLY SPACED
CJ	DENOTES CONSTRUCTION JOINT
EE	DENOTES EACH END
EW	DENOTES EACH WAY
OF	DENOTES OUTSIDE FACE
IF	DENOTES INSIDE FACE
STIRR	DENOTES STIRRUPS
(TYP)	DENOTES TYPICAL

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

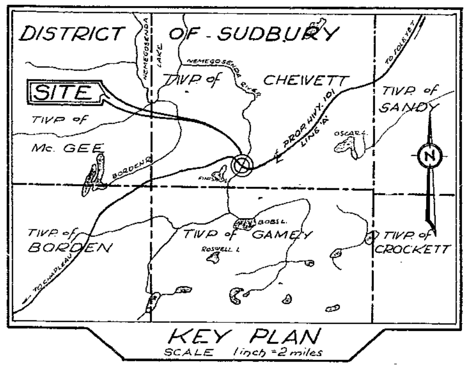
402
BM 403.499
CC ON BOLT AT NE CORNER
WEATHER STATION

REVISIONS			
DATE	BY	REV	DESCRIPTION

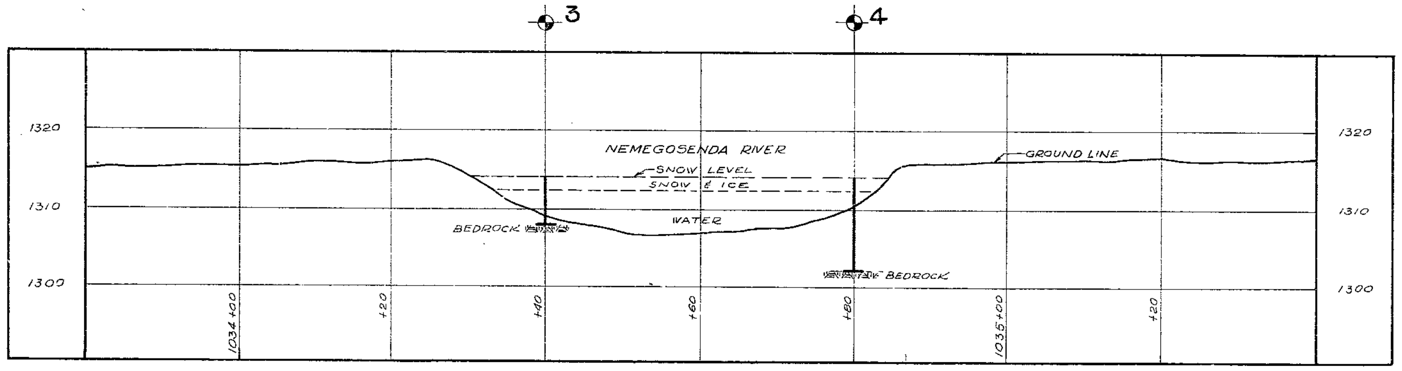
DESIGN AS	CHK TT	CODE CHBDC-14	LOAD CL-625-ONT	DATE OCT/17
DRAWN KGP	CHK AS	SITE 46-215/C	STRUCT SCHEME	DWG 1



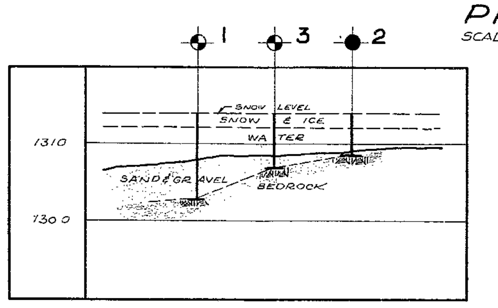
PLAN
SCALE 1inch = 10feet



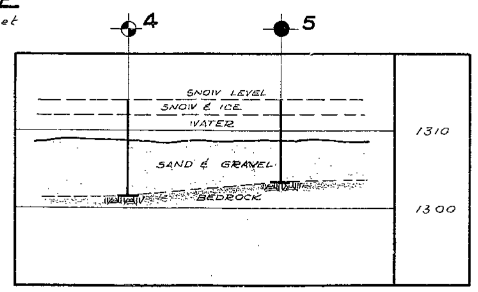
KEY PLAN
SCALE 1inch = 2 miles



PROFILE
SCALE 1inch = 10 feet



A – A



B – B

LEGEND

BORE HOLE

BORE AND PENETRATION HOLE

HOLE	ELEVATION	STATION	DISTANCE FROM ℄
1	1314·00	1034 + 40	10 'RT.
2	1314·00	1034 + 40	10 'LT.
3	1314·00	1034+40	℄
4	1314·00	1034 +80	10 'RT.
5	1314·00	1034 +80	10 'LT.

REFERENCE PLAN E-3630-1

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH SECTION

NEMEGOSENDA RIVER AND PROPOSED HIGHWAY Nº 101

LINE 'A'

ORIGINATED VART KORLU	DISTRICT NO. 17	DATE 27 APR. 1961
DRAWN	W.P. NO	JOB NO. 61-F-21
CHECKED VAK	SCALE 1inch = 10feet	DRAWING NO. 61-F-21 A
APPROVED		

Appendix B.

Record of Borehole Sheets

THURBER

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).

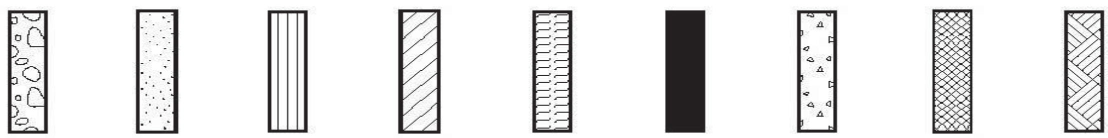
DYNAMIC CONE PENETRATION TEST (DCPT):

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.

THURBER

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

THURBER

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

THURBER

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

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 16-05

METRIC

GWP# 5144-10-00
LOCATION Hwy 101 - Negemosenda River Bridge N 5 311 438.4 E 375 025.3
ORIGINATED BY CM

HWY 101
BOREHOLE TYPE HSA / CME 75 Truck Mount
COMPILED BY JM

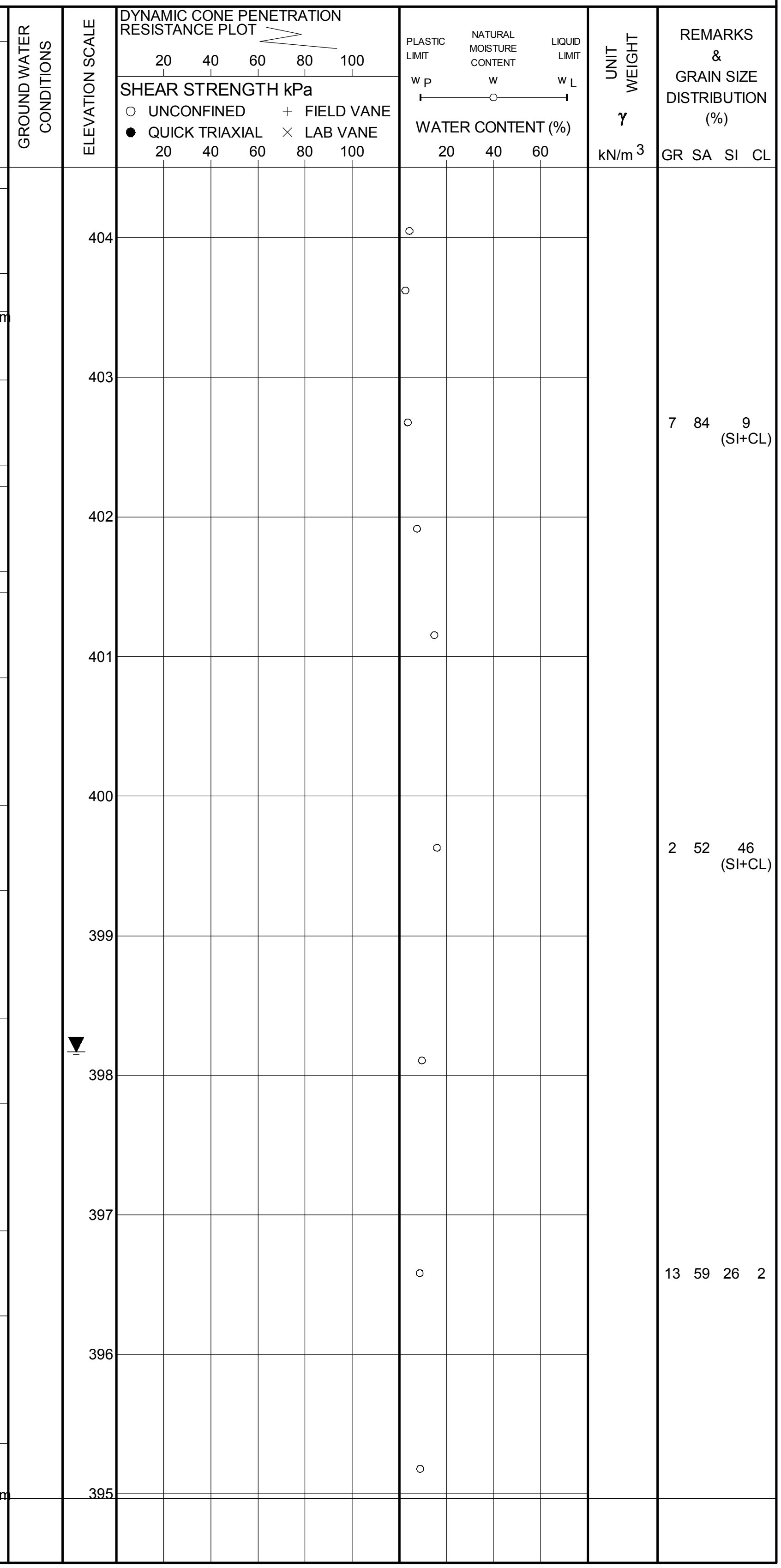
DATUM Geodetic
DATE 2016.10.28 - 2016.10.28
CHECKED BY SP

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
404.5 / 0.0	**40 mm ASPHALT**
	SAND with Gravel, Brown, Very Dense, **FILL**; -Frequent boulders 0.9 m to 1.5 m
403.0 / 1.5	SAND, Brown, Compact, **FILL**
401.5 / 3.0	**SAND (SP)**, trace Wood, Grey, Compact
400.4 / 4.1	**Silty SAND (SM)**, Grey, Very Loose to Very Dense; - Some gravel; - With gravel
395.0 / 9.5	End of borehole (Inferred Bedrock); Groundwater at 6.34 m BGS (Elev. 398.2 m) on completion of drilling

SAMPLES

NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
1	SS	71				
2	SS	100/ 275mm				
3	SS	13	7	84	9 (SI+CL)	
4	SS	23				
5	SS	18				
6	SS	17	2	52	46 (SI+CL)	
7	SS	WH				
8	SS	4	13	59	26	2
9	SS	100/ 250mm				



+3, ×3: Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No 16-06

METRIC

GWP# 5144-10-00
LOCATION Hwy 101 - Negemosenda River Bridge N 5 311 444.1 E 375 011.6
ORIGINATED BY CM

HWY 101
BOREHOLE TYPE HSA / NW Casing / NQ Coring
COMPILED BY JM

DATUM Geodetic
DATE 2016.10.27 - 2016.10.27
CHECKED BY SP

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
404.6						
0.0 / 0.1	**60 mm ASPHALT**					
0.2	SAND with Gravel, Brown, Dense, **FILL**	1	SS	41	404	
	SAND, Brown, Dense to Loose, **FILL**	2	SS	15		6 86 8 (SI+CL)
		3	SS	8	403	
		4	SS	30	402	
401.6 / 3.0	**SAND (SW)** with Gravel, Brown, Dense to Compact, - Frequent cobbles/boulders 3.0 m to 4.6 m	5	SS	20	401	
400.0 / 4.6	**Silty SAND (SM)** with Gravel, Brown, Compact to Very Dense	6	SS	14	400, 399	
		7	SS	32	398	21 60 17 2
396.5		8	SS	100/ 225mm	397	
8.1	**Bedrock**, Granite, Occassional Quartz seams, Grey, Fresh, Moderately Bedded	1	RUN		396	RUN #1 TCR=100% SCR=65% RQD=17%
		2	RUN		395	RUN #2 TCR=95% SCR=90% RQD=48%

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

Continued Next Page

+3, ×3: Numbers refer to Sensitivity 20 15 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 16-06

METRIC

GWP# 5144-10-00	LOCATION Hwy 101 - Negemosenda River Bridge N 5 311 444.1 E 375 011.6	ORIGINATED BY CM
HWY 101	BOREHOLE TYPE HSA / NW Casing / NQ Coring	COMPILED BY JM
DATUM Geodetic	DATE 2016.10.27 - 2016.10.27	CHECKED BY SP

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE) (20 40 60 80 100)	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L / WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page										
							394				
			3	RUN							RUN #3 TCR=100% SCR=60% RQD=64%
							393				
392.8 11.7	End of Borehole Groundwater level in piezometer at 4.14 m BGS (Elev. 400.5 m) on 2016/11/06										

ONTMT4S 13624 - 101 AND 129 - NEMEGOSENDA.GPJ 2012TEMPLATE(MTO).GDT 23/10/18

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

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 16-07

METRIC

GWP#	5144-10-00	LOCATION	Hwy 101 - Negemosenda River Bridge N 5 311 439.7 E 375 009.2	ORIGINATED BY	CM
HWY	101	BOREHOLE TYPE	HSA / CME 75 Truck Mount	COMPILED BY	JM
DATUM	Geodetic	DATE	2016.10.30 - 2016.10.30	CHECKED BY	SP

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
404.6							
0.0	**75 mm ASPHALT**						
0.1	SAND with Gravel, Brown, Dense, **FILL**	1	SS	34		404	
403.8 / 0.8	SAND, Brown, Compact, **FILL**	2	SS	20			
		3	SS	10		403	
402.3 / 2.3	Silty SAND with Gravel, Brown, Compact, **FILL**	4	SS	22		402	39 48 13 (SI+CL)
		5	SS	14			
400.9 / 3.7	**SAND (SW)** with Gravel, Frequent Boulders, Brown, Loose - Difficulty augering below 3.7 m					401	
		6	SS	6		400	26 64 10 (SI+CL)
399.4 / 5.2	**Silty SAND (SM)**, Grey, Compact to Loose					399	
					▼		
		7	SS	12		398	
		8	SS	9		397	6 72 20 2
						396	
395.8 / 8.8	End of Borehole (auger refusal on inferred bedrock)	9	SS	100/ 0mm			

Groundwater level was measured at 4.58 m BGS (Elev. 400.0 m) on 2016/10/30

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

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

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

RECORD OF BOREHOLE No 16-08

METRIC

GWP# 5144-10-00 | LOCATION Hwy 101 - Negemosenda River Bridge N 5 311 457.6 E 374 982.4 | ORIGINATED BY CM

HWY 101 | BOREHOLE TYPE HSA / NW Casing / NQ Coring | COMPILED BY JM

DATUM Geodetic | DATE 2016.10.28 - 2016.10.28 | CHECKED BY SP

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
404.7									
0.0 / 0.1	**50 mm ASPHALT**								
	SAND with Gravel, frequent Cobbles, Brown, Very Dense, **FILL**	1	SS	100/ 275mm		404	○		
403.9 / 0.8	SAND, Brown, Compact, **FILL**	2	SS	14			○		6 89 5 (SI+CL)
403.2 / 1.5	Silty SAND with Gravel, frequent Cobbles, Brown, Compact, **FILL**	3	SS	18		403	○		
		4	SS	17		402	○		
401.7 / 3.0		5	SS	100/ 0mm					
	Bedrock Granite, Grey, Fresh, Moderately Bedded, Occassional mud seams from 3.1 m to 5.5 m	1	RUN			401			RUN #1 TCR=90% SCR=86% RQD=60%
		2	RUN			400, 399			RUN #2 TCR=87% SCR=85% RQD=74%
		3	RUN			398			RUN #3 TCR=100% SCR=100% RQD=76%
397.6 / 7.1	End of borehole Borehole dry prior to coring								

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

+3, ×3: Numbers refer to Sensitivity

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

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 16-09

METRIC

GWP# 5144-10-00 LOCATION Hwy 101 - Negemosenda River Bridge N 5 311 452.6 E 374 979.9 ORIGINATED BY CM

HWY 101 BOREHOLE TYPE HSA / CME 75 Truck Mount COMPILED BY JM

DATUM Geodetic DATE 2016.10.29 - 2016.10.29 CHECKED BY SP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
404.7										
0.0 / 0.1	**75 mm ASPHALT**									
	Silty SAND some Gravel, Brown, Dense, **FILL**	1	SS	42	404	○				
403.9 / 0.8	SAND, frequent Cobbles, Brown, Compact, **FILL**	2	SS	11		○				
403.1 / 1.5	Silty SAND, frequent Cobbles, Brown, Loose, **FILL**	3	SS	100/ 225mm	403	○				
402.4 / 2.3	GRAVEL, Silty with Sand, Brown, Compact, **FILL**	4	SS	26	402	○	47	41	12 (SI+CL)	
	- Frequent Cobbles/Boulders below 3.1 m	5	SS	69	401					
400.8 / 3.8	End of Borehole (inferred bedrock), Borehole dry on completion	6	SS	100/ 0mm						

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

+ ^{3}, × ^{3}: Numbers refer to Sensitivity

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 16-10

METRIC

GWP# 5144-10-00	LOCATION Hwy 101 - Negemosenda River Bridge N 5 311 456.9 E 374 980.6	ORIGINATED BY CM
HWY 101	BOREHOLE TYPE HSA / NW Casing / NQ Coring	COMPILED BY JM
DATUM Geodetic	DATE 2016.10.28 - 2016.10.28	CHECKED BY SP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
404.7									
0.0	**40 mm ASPHALT**								
	SAND with Gravel Brown Very Dense **FILL**	1	SS	52			○		
404.0 0.8	SAND Brown Compact **FILL**	2	SS	17		404	○		
403.2 1.5	SAND with Gravel, frequent Cobbles Brown Compact **FILL**	3	SS	21		403	○		27 65 8 (SI+CL)
402.5 2.3	GRAVEL with Sand Grey Dense to Compact **FILL**	4	SS	50		402	○		55 37 8 (SI+CL)
		5	SS	21		401	○		
400.5	- Auger refusal at 4.3 m								
4.3	**Bedrock** Granite Grey Fresh Moderately Bedded	1	RUN			400			RUN #1 TCR=98% SCR=95% RQD=74%
		2	RUN			399, 398			RUN #2 TCR=98% SCR=97% RQD=78%
		3	RUN						RUN #3 TCR=95% SCR=95% RQD=68%
397.0 7.8	End of borehole Borehole dry prior to coring					397			

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

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

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

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 16-11

METRIC

GWP# 5144-10-00	LOCATION Hwy 101 - Negemosenda River Bridge N 5 311 458.9 E 374 979.5	ORIGINATED BY CM
HWY 101	BOREHOLE TYPE HSA / CME 75 Truck Mount	COMPILED BY JM
DATUM Geodetic	DATE 2016.10.28 - 2016.10.28	CHECKED BY SP

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
404.7 / 0.0	**65 mm ASPHALT**					
0.1	SAND with Gravel, Brown, Dense to Compact, **FILL**	1	SS	47	404	
403.9 / 0.8	SAND, Brown, Compact, **FILL**	2	SS	10		
		3	SS	14	403	
		4	SS	14	402	
401.6 / 3.0	**Silty SAND (SM)** with Gravel, Brown, Compact	5	SS	28	401	
		6	SS	100/ 250mm	400	21 49 30 (SI+CL)
399.4 / 5.3	End of Borehole (inferred bedrock) Borehole dry on completion	7	SS	100/ 0mm		

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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

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

UNIT WEIGHT γ kN/m³

+ 3, × 3: Numbers refer to Sensitivity

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

ONTMT4S 13624 - 101 AND 129 - NEMEGOSENDA.GPJ 2012TEMPLATE(MTO).GDT 23/10/18

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 16-12

METRIC

GWP# 5144-10-00 | LOCATION Hwy 101 - Negemosenda River Bridge N 5 311 453.9 E 374 976.8 | ORIGINATED BY CM

HWY 101 | BOREHOLE TYPE HSA / NW Casing / NQ Coring | COMPILED BY JM

DATUM Geodetic | DATE 2016.10.29 - 2016.10.29 | CHECKED BY SP

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
404.7									
0.0	**40 mm ASPHALT**								
	Sand with Gravel, Brown, Dense, **FILL**	1	SS	37		404	○		18 73 9 (SI+CL)
403.9 / 0.8	SAND, Brown, Dense to Very Dense, **FILL**	2	SS	37			○		
		3	SS	61		403	○		
	- Frequent Cobbles/Boulders below 2.2 m	4	SS	79		402	○		
401.6 / 3.0	GRAVEL, Silty with Sand, Frequent Cobbles and Boulders, Brown, Compact, **FILL**	5	SS	11		401	○		47 41 12 (SI+CL)
400.8 / 3.8	- Auger refusal at 3.8 m **Bedrock**, Granite, Grey, Fresh, Moderately Bedded	1	RUN						RUN #1 TCR=93% SCR=93% RQD=93%
		2	RUN			400			RUN #2 TCR=100% SCR=91% RQD=63%
		3	RUN			399 / 398			RUN #3 TCR=100% SCR=100% RQD=87%
397.5 / 7.2	End of borehole Borehole dry prior to coring								

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

+³, ×³: Numbers refer to Sensitivity

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

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 16-13

METRIC

GWP# 5144-10-00	LOCATION Hwy 101 - Negemosenda River Bridge N 5 311 460.2 E 374 963.7	ORIGINATED BY CM
HWY 101	BOREHOLE TYPE HSA / CME 75 Truck Mount	COMPILED BY JM
DATUM Geodetic	DATE 2016.10.29 - 2016.10.29	CHECKED BY SP

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
404.6										
0.0	**80 mm ASPHALT**									
0.1	SAND with Gravel, Brown, Dense, **FILL**		1	SS	45		404			
403.9 / 0.8	SAND, Brown, Dense, **FILL**		2	SS	49					
			3	SS	35		403			
			4	SS	18		402			4 86 10 (SI+CL)
	- Wood log at 3.4 m		5	SS	100/ 225mm					
401.0 / 3.6	**Silty SAND (SM)** with Gravel, Grey, Very Dense						401			
			6	SS	100/ 225mm		400			20 54 23 3
399.5 / 5.1	End of Borehole (inferred bedrock) Borehole dry on completion									

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

+ 3, × 3: Numbers refer to Sensitivity

15 20 5 10 (%) STRAIN AT FAILURE

FORM OB-ML-126 (REV. 1959.)—58-5691

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS AND RESEARCH SECTION

W.P. -

JOB 61-F-21

DATUM 1313.0'

BORING DATE Mar. 25/61

BORE HOLE NO. 1

STATION 1034+40 (10' Lt.)

COMPILED BY B.K.

CHECKED BY V.K.

2" DIA. SPLIT TUBE

2" SHELBY TUBE

2" SPLIT TUBE

2" DIA. CONE

2" SHELBY

CASING

LEGEND

1/2 UNCONFINED COMPRESSION (Qu) — O

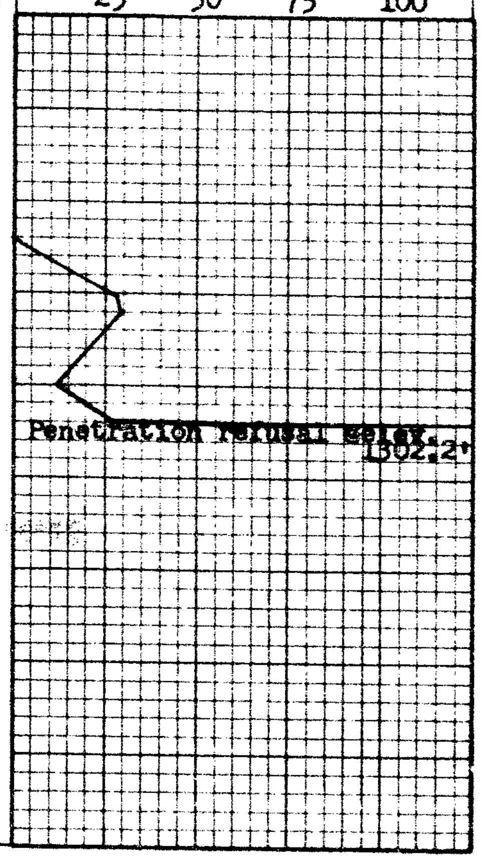
VANE TEST (C) AND SENSITIVITY (S) — +S

NATURAL MOISTURE AND LIQUIDITY INDEX — LI X

LIQUID LIMIT

PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE P.S.F. / BLOWS/FT. 25 50 75 100	CONSISTENCY / MOIST. CONTENT - % DRY WT.	SAMPLE	NATURAL UNIT WT. P.C.F.
	Snow Level	1313.0	0				
	Snow & ice	1310.5					
	Water	1307.0	5				
	Coarse sand & gravel	1302.0	10	Penetration refusal below. 1302.2'			
	Bedrock	1297.5	15			RC1	-
	End of borehole Penetration resistance profile shown; obtained by driving a 2" dia. cone from groundlevel to depth noted with an energy of 350 ft. lb. per blow.						



FORM OB-ML-126 (REV. 1959.)—59-5008

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS AND RESEARCH SECTION

W.P. -
JOB 61-F-21
DATUM 1313.0'
BORING DATE Apr. 25/61

BORE HOLE NO. 2
STATION 1034+40 , 10' Lt.)
COMPILED BY B.K.
CHECKED BY V.K.

2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
2" SHELBY
CASING

LEGEND

1/2 UNCONFINED COMPRESSION (Qu)
VANE TEST (C) AND SENSITIVITY (S)
NATURAL MOISTURE AND LIQUIDITY INDEX
LIQUID LIMIT
PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE P.S.F. / BLOWS/FT	CONSISTENCY MOIST CONTENT-% DRY WT.	SAMPLE	NATURAL UNIT WT. P.C.F.
	Snow Level	1313.0	0				
	Snow & ice	1310.5					
	Water	1308.3	5				
	Bedrock?						
			10				
			15				

FORM OB-ML-126 (REV 1959.) 59-5008

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS AND RESEARCH SECTION

W.P. -

BORE HOLE NO. 3

JOB 61-F-21

STATION 1034+40 ℄

DATUM 1313.0'

COMPILED BY B.K.

BORING DATE Apr. 25/61

CHECKED BY V.K.

2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
2" SHELBY
CASING

LEGEND

1/2 UNCONFINED COMPRESSION (Qu)
VANE TEST (C) AND SENSITIVITY (S)
NATURAL MOISTURE AND LIQUIDITY INDEX
LIQUID LIMIT
PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE P.S.F. / BLOWS/FT 25 50 75 100	CONSISTENCY / MOIST. CONTENT % DRY WT.	SAMPLE	NATURAL UNIT WT. P.C.F.
	Snow Level	1313.0	0				
	Snow & ice	1310.5					
	Water	1308.0	5				
	Sand & gravel	1306.0		Penetration refusal at elev. 1306.0'			
	Bedrock?		10				
			15				
			20				

Penetration resistance profile shown; obtained by driving a 2" dia. cone from groundlevel to depth noted with an energy of 350 ft. lb. per blow.

FORM OB-ML-126 (REV. 1959) 59-5006

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS AND RESEARCH SECTION

W.P. -	BORE HOLE NO. 4		
JOB 61-F-21	STATION 1034+80 (10' Rt.)		
DATUM 1313.0'	COMPILED BY B.K.		
BORING DATE Apr. 25/61	CHECKED BY V.K.		

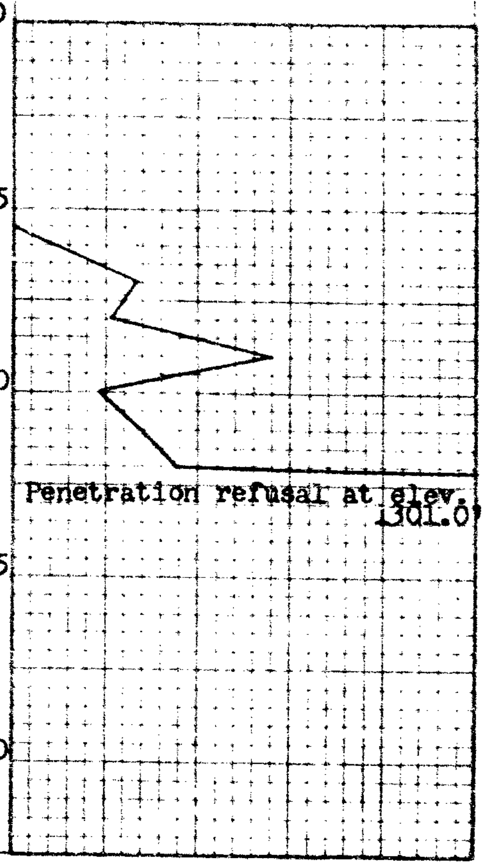
LEGEND

2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
2" SHELBY
CASING

1/2 UNCONFINED COMPRESSION (Qu)
VANE TEST (C) AND SENSITIVITY (S)
NATURAL MOISTURE AND LIQUIDITY INDEX
LIQUID LIMIT
PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
	↓ Snow Level	1313.0	0
	Snow & ice	1311.8	
	Water	1307.5	5
		1301.0	10
	End of cone penetration Bedrock?		15
			20

STRENGTH AND PENETRATION RESISTANCE — P.S.F. — BLOWS/FT.



CONSISTENCY — MOIST. CONTENT % DRY WT.

NATURAL UNIT WT. P.C.F.

SAMPLE

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS AND RESEARCH SECTION

W P -

BORE HOLE NO 5

JOB 61-F-21

STATION 1034+80 (10' Lt.)

DATUM 1313.0'

COMPILED BY B.K.

BORING DATE Apr. 25/61

CHECKED BY V.K.

LEGEND

2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
2" SHELBY
CASING

1/2 UNCONFINED COMPRESSION (Qu) ○
VANE TEST (C) AND SENSITIVITY (S) +S
NATURAL MOISTURE AND LIQUIDITY INDEX X LI
LIQUID LIMIT
PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE (P.S.F. / BLOWS/FT. 25 50 75 100)	CONSISTENCY (MOIST. CONTENT - % DRY WT. 10 20 30)	SAMPLE	NATURAL UNIT WT. P.C.F.
	↓ Snow Level	1313.0	0				
	Snow & ice	1312.0					
	Water	1308.0	5			S1	-
	Sand and gravel	1302.5	10		X	S2	-
	Bedrock	1297.5	15			RC3	-
	End of borehole		20				

Appendix C.

Laboratory Testing

Appendix C.1

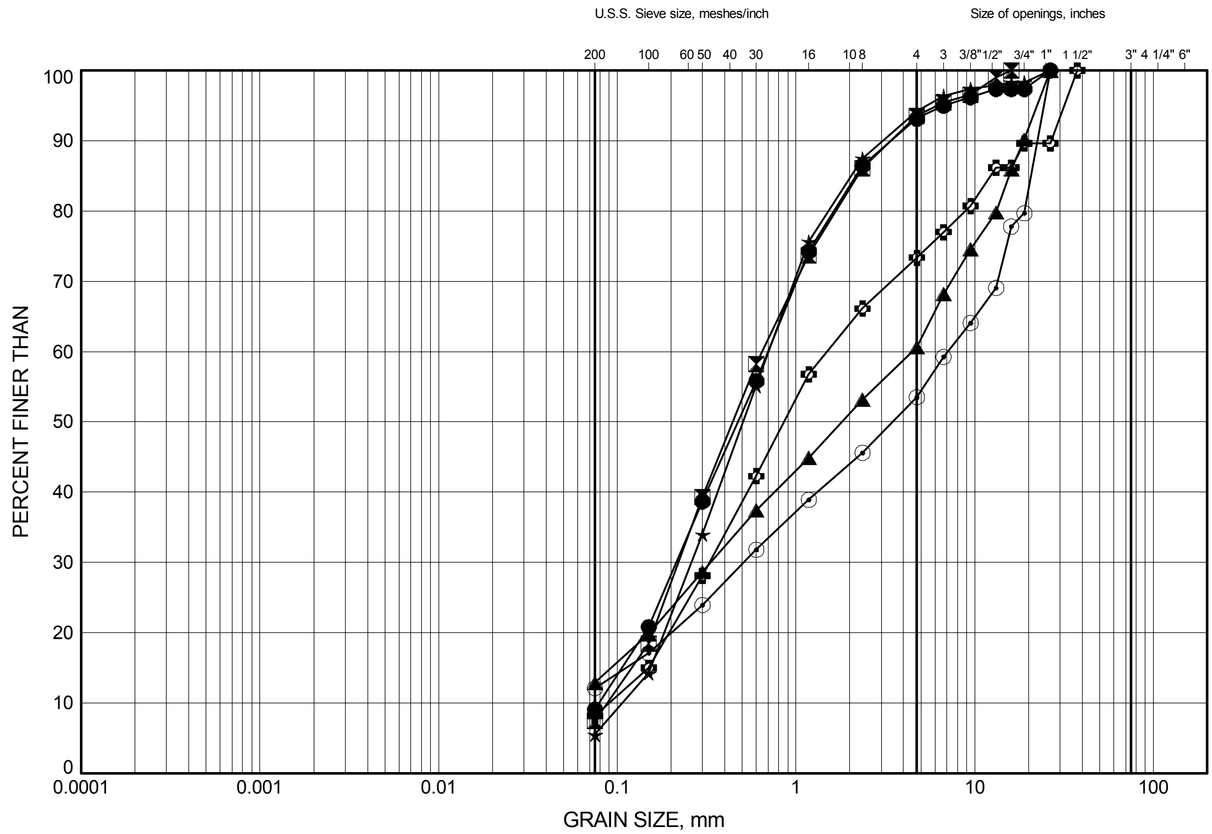
Particle Size Analysis Figures

Nemegosenda River Bridge

GRAIN SIZE DISTRIBUTION

FIGURE C1

Embankment FILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	16-05	1.83	402.68
⧗	16-06	1.07	403.49
▲	16-07	2.59	401.99
★	16-08	1.07	403.64
⊙	16-09	2.59	402.06
✣	16-10	1.83	402.91

Date December 2016

GWP# 5144-10-00

THURBER

Prep'd JM

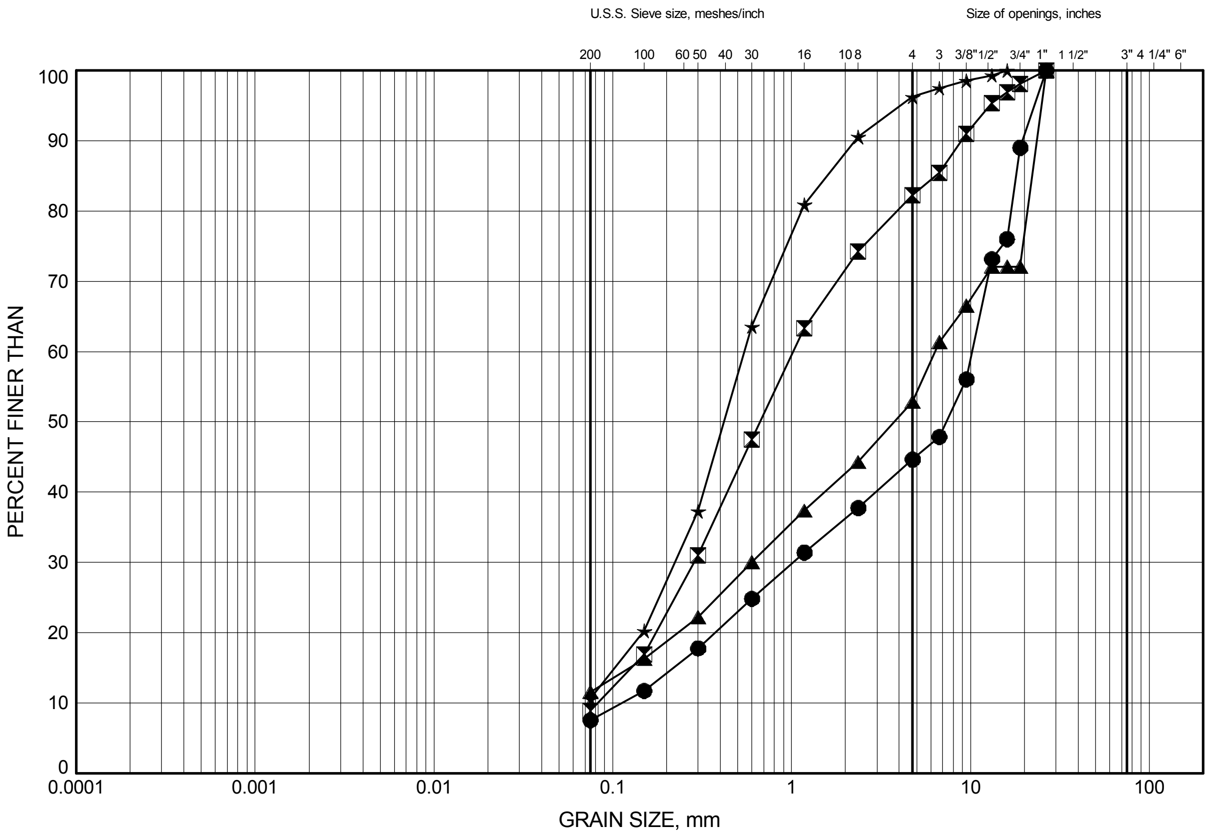
Chkd. FJG

Nemegosenda River Bridge

GRAIN SIZE DISTRIBUTION

FIGURE C2

Embankment FILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	16-10	2.59	402.15
⧗	16-12	0.46	404.20
▲	16-12	3.35	401.30
★	16-13	2.59	402.04

Date December 2016

GWP# 5144-10-00

THURBER

Prep'd JM

Chkd. FJG

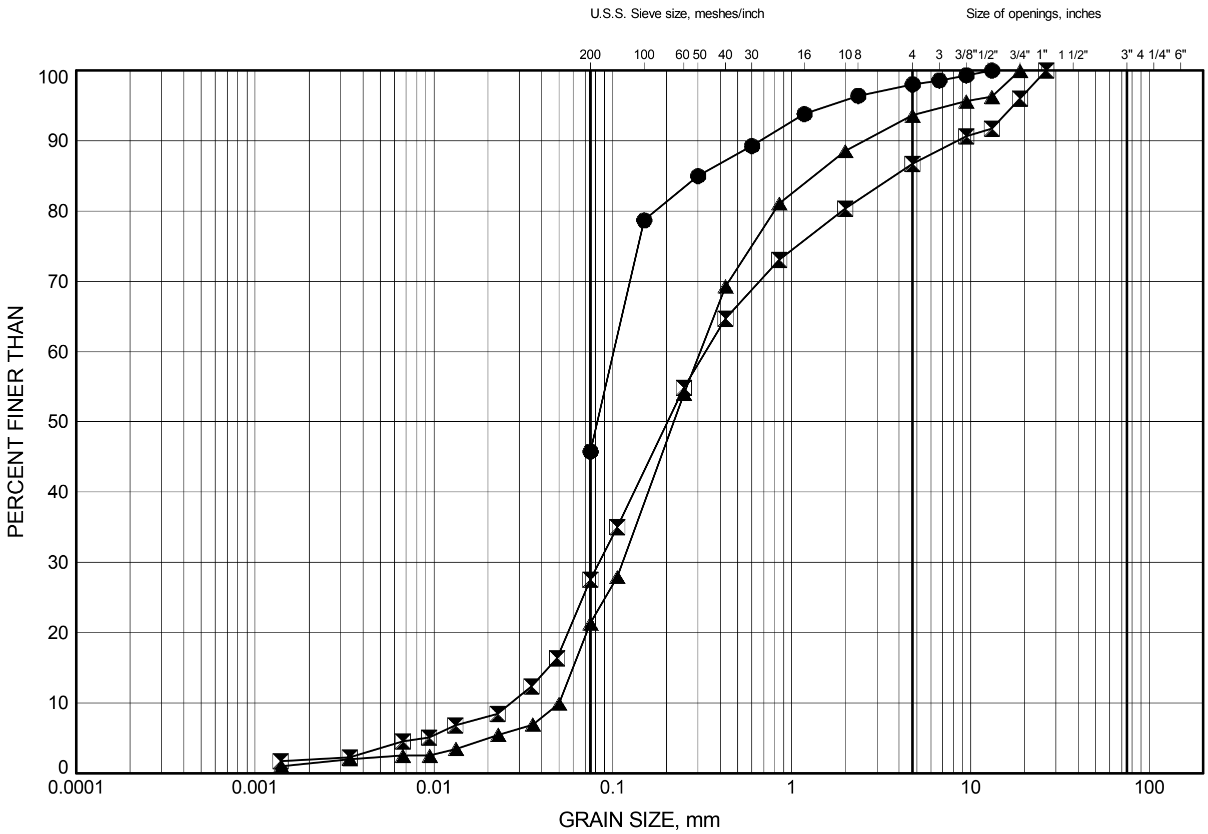
GRAIN SIZE DISTRIBUTION - THURBER 13624 - 101 AND 129 - NEMEGOSENDA.GPJ 21/12/16

Nemegosenda River Bridge

GRAIN SIZE DISTRIBUTION

FIGURE C3

Silty SAND to Silty SAND with Gravel



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	16-05	4.88	399.63
⧗	16-05	7.92	396.58
▲	16-07	7.92	396.65

Date December 2016

GWP# 5144-10-00

THURBER

Prep'd JM

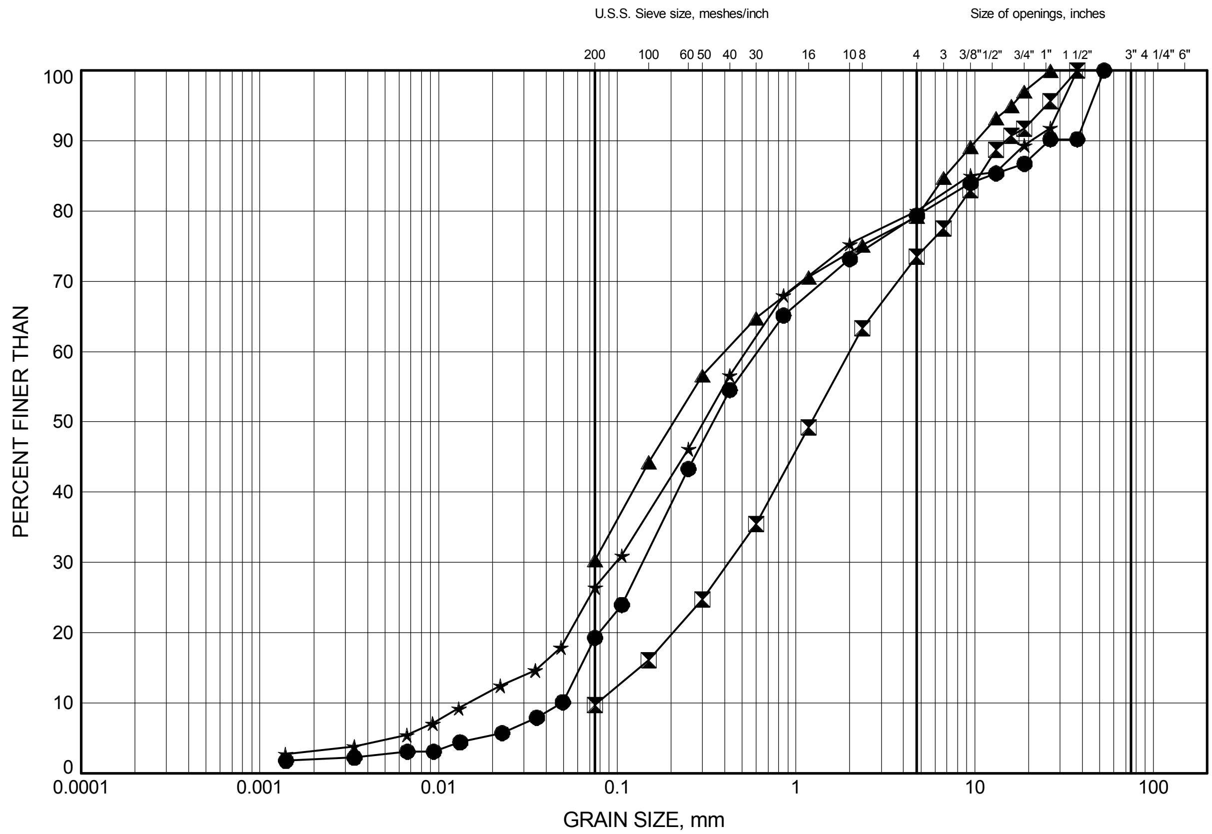
Chkd. FJG

Nemegosenda River Bridge

GRAIN SIZE DISTRIBUTION

FIGURE C4

Silty SAND to Silty SAND with Gravel



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	16-06	6.40	398.15
⧗	16-07	4.88	399.70
▲	16-11	4.85	399.85
★	16-13	4.85	399.78

GRAIN SIZE DISTRIBUTION - THURBER 13624 - 101 AND 129 - NEMEGOSENDA.GPJ 21/12/16

Date December 2016

GWP# 5144-10-00

THURBER

Prep'd JM

Chkd. FJG

Appendix C.2

Analytical Testing Results

PARACEL LABORATORIES LTD.

Order #: 1646369

Certificate of Analysis

Client: Thurber Engineering Ltd.

Client PO:

Report Date: 17-Nov-2016

Order Date: 11-Nov-2016

Project Description: 13624

	Client ID:	16-1 SS2 (2'-4')	16-4 (1-4)	16-6 SS3 (5'-7')	16-8 SS4 (7'6-9'6)
	Sample Date:	21-Oct-16	23-Oct-16	27-Oct-16	28-Oct-16
	Sample ID:	1646369-01	1646369-02	1646369-03	1646369-04
	MDL/Units	Soil	Soil	Soil	Soil
Physical Characteristics					
% Solids	0.1 % by Wt.	81.8	85.3	96.7	92.0
General Inorganics					
Conductivity	5 uS/cm	109	109	385	728
pH	0.05 pH Units	7.41	6.41	7.89	7.89
Resistivity	0.10 Ohm.m	91.5	91.7	26.0	13.7
Anions					
Chloride	5 ug/g dry	16	15	159	346
Sulphate	5 ug/g dry	19	14	10	31

	Client ID:	16-15 SS6 (40-41-4)	16-18 SS6 (15-17)	-	-
	Sample Date:	31-Oct-16	03-Nov-16	-	-
	Sample ID:	1646369-05	1646369-06	-	-
	MDL/Units	Soil	Soil	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	89.1	84.1	-	-
General Inorganics					
Conductivity	5 uS/cm	171	351	-	-
pH	0.05 pH Units	7.78	6.84	-	-
Resistivity	0.10 Ohm.m	58.4	28.5	-	-
Anions					
Chloride	5 ug/g dry	24	171	-	-
Sulphate	5 ug/g dry	54	18	-	-

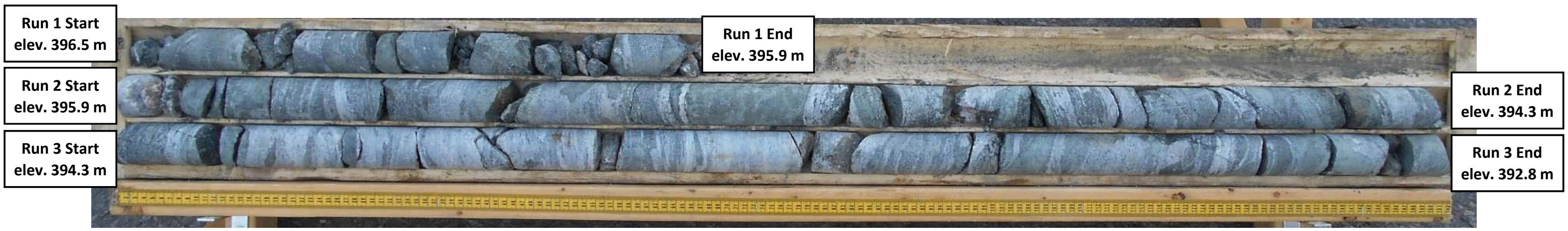
Appendix C.3

Rock Core Photographs

Borehole 16-6

Run 1 to 3 (of 3)

Elevation 396.5 m to 392.8 m



THURBER ENGINEERING LTD.

Foundation Investigation
Highway 101 – Nemegosenda River Bridge
Site 46-215

GWP: 5144-10-00

Project No.: 13624

Borehole 16-8

Run 1 to 3 (of 3)

Elevation 401.7 m to 397.6 m

THURBER ENGINEERING LTD.

Foundation Investigation
Highway 101 – Nemegosenda River Bridge
Site 46-215

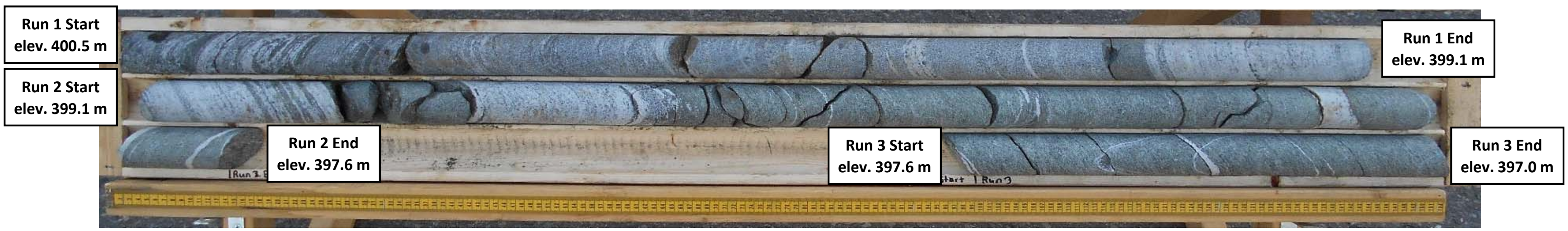
GWP: 5144-10-00

Project No.: 13624

Borehole 16-10

Run 1 to 3 (of 3)

Elevation 400.5 m to 397.0 m



THURBER ENGINEERING LTD.	Foundation Investigation Highway 101 – Nemegosenda River Bridge Site 46-215	GWP: 5144-10-00 Project No.: 13624

Borehole 16-12

Run 1 to 3 (of 3)

Elevation 400.8 m to 397.5 m

THURBER ENGINEERING LTD.	**Foundation Investigation Highway 101 – Nemegosenda River Bridge Site 46-215**	**GWP: 5144-10-00** **Project No.: 13624**

Appendix D.

Selected Site Photographs

Photo 1. Looking southwest at the Bridge from near the east abutment. [taken October 2016]

Photo 2. Looking northeast at the bridge from near the west abutment. [taken October 2016]

Photo 3. Looking at the west abutment. [taken October 2016]

Photo 4. Looking at the east abutment. [taken October 2016]

Photo 5. Looking at the gravel access road south of the highway alignment [taken October 2016]

Appendix E.

Foundation Alternative Comparisons

COMPARISON OF ALTERNATIVE FOUNDATION TYPES

Comment	Spread Footings	**Caissons** (Socketed into Bedrock)	**Steel Piles** (H-Piles, Pipe Piles)
Advantages	- Generally less costly construction then deep foundations - Accommodates abutments perched within approach fills - Requires less specialized construction equipment	- Higher geotechnical capacity then spread footings or H-Piles - Construction can continue in winter weather conditions - Reduces magnitude of excavations and limits dewatering requirements	- Higher geotechnical capacity then spread footings - Construction can continue in winter weather conditions - Likely requires less concrete than spread footings or caissons - Can provide frost protection by insulation
Disadvantages	- Requires larger excavation - Requires deeper excavation to construct footing below the frost penetration depth - Lower geotechnical resistance then deep foundations - Ineffective for resistance to uplift or overturning - Requires local availability of concrete if cast-in-place footings are used	- Higher unit cost than spread footings - Requires local availability of concrete - Specialized installation measures such as equipment, liners and drilling mud will be required - Potential difficulty in cleaning and inspecting base drilled into bedrock	- Higher unit cost than spread footings - Has potential to encounter obstructions in the native soils
Risks / Consequences	Large excavation	Difficulty in advancing through obstructions and bedrock	Difficulty advancing through obstructions
Relative Cost	Moderate	High	Moderate to High
	Recommended at west abutment	Not Recommended	Recommended at east abutment

Appendix F.

GSC Seismic Hazard Calculation

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

December 19, 2016

Site: 47.9381 N, 83.0604 W User File Reference: Nemegosenda

Requested by: Chris Murray, Thurber Engineering

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)**
0.055	0.077	**0.074**	0.063	**0.052**	**0.032**	**0.016**	**0.0038**	**0.0017**	**0.043**	**0.041**

Notes. Spectral (Sa(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)	0.0052	0.018	0.030
Sa(0.1)	0.0088	0.028	0.044
Sa(0.2)	0.011	0.030	0.046
Sa(0.3)	0.0095	0.027	0.041
Sa(0.5)	0.0072	0.023	0.034
Sa(1.0)	0.0036	0.013	0.021
Sa(2.0)	0.0014	0.0058	0.010
Sa(5.0)	0.0004	0.0013	0.0022
Sa(10.0)	0.0003	0.0007	0.0010
PGA	0.0048	0.016	0.025
PGV	0.0040	0.015	0.025

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

Canada

Appendix G.

List of Special Provisions and OPSS Documents Referenced in this Report

1. The following Special Provisions and OPSS Documents are referenced in this report:

OPSD 208.010	Benching of Earth Slopes
OPSD 3090.100	Foundation, Frost Penetration Depths for Northern Ontario
OPSD 3101.150	Walls, Abutment, Backfill Minimum Granular Requirements
OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 501	Construction Specification for Compacting
OPSS 511	Construction Specification for Rip-Rap, Rock Protection, and Granular Sheeting
OPSS.PROV 517	Construction Specification for Dewatering
OPSS.PROV 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 903	Construction Specification for Deep Foundations
OPSS.PROV 1010	Material Specification for Aggregates-Base, Subbase, Select Subgrade, and Backfill Material
OPSS 1860	Material Specification for Geotextiles
SP 109S12	
SP 517F01	