



THURBER ENGINEERING LTD.

**FINAL
PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 401 WIDENING, HIGHWAY 16 TO MAITLAND ROAD
HIGHWAY 401 UNDERPASS AT HIGHWAY 16, SITE NO. 16X-0130/B0
GWP 4024-20-00 / ASSIGNMENT NO.: 4019-E-0010.2**

SITE NO. 16X-0130/B0

Geocres No.: 31B-112

Report to:

MTO c/o AECOM Canada Ltd.

Latitude: 44.754353°
Longitude: -75.478514°

November 2023
Thurber File No.: 29381



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PART 1. FACTUAL INFORMATION

1 INTRODUCTION

Thurber Engineering Ltd. (Thurber) has been retained by AECOM Canada Ltd. (AECOM) on behalf of the Ministry of Transportation Ontario (MTO) under Assignment No. 4019-E-0010, Work Item No. 2, to carry out Foundation Investigations to support the Preliminary Design and Environmental Assessment for the widening of Highway 401 from Highway 16 to Maitland Road. The overall scope of work comprises replacement or rehabilitation of 14 existing structures, including 10 bridges and four structural culverts.

This report addresses the Highway 16 structure (Site No. 16X-0130/B0) that crosses over Highway 401 in the Township of Edwardsburgh within Leeds and Grenville County, Ontario.

This section of the report presents the factual findings obtained from a foundation investigation completed at the site and was informed by existing subsurface information pertinent to the site, obtained from the MTO's Foundation Library (Geocres No. 31B00-033).

The purpose of this investigation was to explore the subsurface conditions at the site and, based on the data obtained, provide a borehole location plan, records of boreholes, a stratigraphic profile, laboratory test results, and a written description of the subsurface conditions. A model of the subsurface conditions influencing design and replacement of the structure was developed in the course of the current investigation.

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

2 BACKGROUND AND SITE DESCRIPTION

2.1 General

The Highway 16 Underpass crosses over Highway 401 at an interchange approximately 1.0 km northwest of the community of Johnstown and approximately 5.0 km northeast of the Highway 401 Underpass at Edward Street. For project orientation purposes, Highway 16 will be described as oriented north-south and Highway 401 as oriented east-west.



The existing one-span structure carries Highway 16 (two through lanes in each direction) over Highway 401. Turn tapers associated with interchange on-ramps start just beyond the bridge abutments. Traffic volume on this section of Highway 16 is understood to have been 4,250 AADT in 2016. Concrete parapet walls are placed along the east and west edges of the structure deck, and an approximately 3 m non-traversable paved median is present between the north and south bound lanes. The outside shoulders are of limited width on the bridge and no sidewalks are present. Rectangular concrete wingwalls are present at the four quadrants. Galvanized W-beam guiderails supported on metal posts are present at all four quadrants and extend to as much as approximately 160 m back from the abutments. The embankment side slopes are inclined at about 2H:1V and as steep as approximately 1.6H:1V near the west corner of the south abutment and are vegetated with short trees, shrubs, and grasses. No signs of global instability of the embankment were noted during the field investigation.

Highway 401, at the location of the Highway 16 Interchange, has two through lanes in each direction, a W-N/S off-ramp and N-E and S-E on-ramps in the eastbound direction, and a E-N/S off-ramp and S-W and N-W on-ramps in the westbound direction. The outside and median shoulders are paved, and the east and west bound lanes are separated by a median barrier. Traffic volume on Highway 11 is understood to have been 19,900 AADT in 2016.

The site is in a semi-rural setting, and the area directly adjacent to the Highway 16 Underpass is undeveloped land with a mix of cleared private properties and densely vegetated areas with deciduous trees and shrubs. The CNR Overhead (Site No. 16X-0130) is located about 275 m to the south of the site. Overhead utility lines are not present but light poles are present near the entry/exit of the Highway 401 ramps. The terrain is relatively flat, aside from the existing highway embankments and associated ditches, which are relatively rugged. Storm water drainage in the area includes a median storm sewer system and roadside ditching.

Photographs showing general conditions in the project area at the time of the field investigation are presented in Appendix D.

2.2 Existing Structure Information

The Ontario Structure Inspection Manual (OSIM) report prepared by MTO on June 9, 2016, indicates that the existing structure was built in 1961 and is a one-span rectangular beam structure with reinforced cast-in-place concrete. The inspection report indicates that the bridge deck is approximately 38 m long and 21 m wide, with an approximate 12-degree skew to the highway. There are cast-in-place concrete retaining wing wall walls located at all four quadrants of the bridge to retain the embankment slopes adjacent to Highway 401. The bridge was rehabilitated in 2003.

2.3 Existing Subsurface Information

The following historical foundation investigation report was available for this site within the Online Geocres library:

- Geocres Report No. 31B00-033 (e. m. peto associates ltd., 1959) presents the results of a foundation investigation carried out for the design and construction of the existing bridge structure. The field investigation included five boreholes drilled near the abutments and



existing centerline of Highway 401. In general, the boreholes indicated the presence of topsoil over sand underlain by clay which is, in turn, underlain by silty sand glacial till. The glacial till was typically composed of silty sand containing cobbles and boulders. No bedrock was cored in the investigation. The boreholes were advanced to depths ranging from 4.6 m to 6.9 m below the existing ground surface (base elev. 79.0 m to 76.3 m).

The historical stratigraphy drawings and borehole logs have been included in Appendix A and Appendix B, respectively.

2.4 Site Geology

Based on published geological information in *The Physiography of Southern Ontario* by Chapman and Putnam (1984) and the Ontario Geological Survey maps (MRD228), the site lies on the border of the physiographic regions known as the Glengarry Till Plain and the Edwardsburg Sand Plain. The Glengarry Till Plain is characterized by typically undulating to rolling surface containing well-formed drumlins, intervening clay flats, and stony glacial tills with a high proportion of limestone pieces. The Edwardsburg Sand Plain is characterized by slightly undulating sand plain of glaciofluvial origin. The bedrock in both areas is generally limestone, dolostone, and calcareous sandstone.

According to Crins et al. (2009)ⁱ, the project area is described as Ecoregion 6E (Lake Simcoe-Rideau) within the Ontario Shield Ecozone. According to Wester et al. (2018)ⁱⁱ, the ecoregion is subdivided into Ecodistrict 6E-11 (Smiths Falls Ecodistrict). The area is characterized by discontinuous layer of shallow calcareous morainal material overlying Paleozoic bedrock.

The Ontario Geological Survey maps (MRD126) suggest the site is underlain by dolostone and sandstone. Map P.2722ⁱⁱⁱ indicates that the bedrock in the project area is of Oxford Formation that consists of sub lithographic to fine crystalline dolostone.

3 SITE INVESTIGATIONS AND FIELD TESTING

A site investigation and field-testing program was carried out between November 28 and December 12, 2022, and consisted of two on-road boreholes identified as 130-22-01 and 130-22-02, put down on Highway 16 near the underpass abutments, and one on-road borehole identified as 130-22-03, put down on Highway 401 in the eastbound median shoulder. The boreholes were advanced using a truck mounted CME 55 drill rig equipped with Hollow Stem Augers, NW casing, and NQ coring equipment. Thurber contacted Ontario One Call in advance of the field investigation to obtain utility locates/clearances in the vicinity of the borehole locations. In addition, MTO was contacted to obtain Electrical and Fibre Optic locates for the project limits.

The borehole coordinates, elevations, and termination depths are provided in Table 3-1. The as-drilled elevations of all boreholes were surveyed by Thurber with a Trimble Catalyst DA1 antenna with centimeter accuracy. The elevations were surveyed relative to the benchmark information provided by AECOM and were reviewed with reference to the topographic survey received from AECOM. The horizontal locations were measured by Thurber relative to existing site features. The borehole coordinates and elevation are shown on the Borehole Location and Soil Strata Drawings in Appendix A and on the individual Record of Borehole sheets included in Appendix B. The borehole coordinates are referenced to MTM Zone 9.

Table 3-1: Borehole Summary

Borehole No.	Drilled Location	Northing (Latitude)	Easting (Longitude)	Ground Surface Elevation (m)	Termination Depth (m)
130-22-01	South Abutment (Hwy 16)	4 957 630.0 (44.754103°)	385 688.8 (-75.478361°)	91.6	27.2
130-22-02	North Abutment (Hwy 16)	4 957 680.0 (44.754557°)	385 652.4 (75.478812°)	91.4	18.3
130-22-03	Highway 401 EB median shoulder	4 957 632.6 (44.754129°)	385 653.7 (-75.478803°)	84.3	14.7

Soil samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Tests (SPT) in general accordance with ASTM D 1586. In-situ shear vane testing was carried out within the cohesive strata, where possible, using an MTO 'N' sized vane in general accordance with ASTM D 2573. The boreholes were advanced to depths ranging from 14.7 m to 27.2 m (base elev. 73.1 m to 64.4 m). Coring was required to advance the boreholes past cobbles and boulders and into bedrock. A standpipe piezometer was installed in Borehole 130-22-01 to allow for measurements of the groundwater level after drilling. The details for the standpipe piezometer are illustrated on the respective Record of Borehole sheets provided in Appendix B.

The drilling and sampling operations were supervised on a full-time basis by a member of Thurber's technical staff. The drilling supervisor logged the boreholes and processed the recovered soil and rock samples for transport to the Thurber's Ottawa laboratory for further examination and testing.

Following completion of the field investigation, the boreholes without a standpipe piezometer were decommissioned in general accordance with O.Reg. 903, as amended. Boreholes 130-22-02 and 130-22-03 were capped with cold patch asphalt to reinstate the pavement surface. The standpipe piezometer was decommissioned in general accordance with Ontario MOE Regulation 903 in April 2023.

4 LABORATORY TESTING

Geotechnical laboratory testing carried out as part of the current investigation included natural moisture content determination and visual identification of all retained soil samples. Testing for grain size distribution and Atterberg Limits was also carried out on selected samples to MTO and ASTM standards. All rock cores were photographed and their total core recovery (TCR), solid core recovery (SCR) and rock quality designation (RQD) were measured. One Unconfined Compressive Strength (UCS) Test was conducted on a recovered core sample from Borehole 130-22-01.

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



5 GENERAL DESCRIPTION OF SUBSURFACE CONDITIONS

Details of the encountered soil stratigraphy are presented on the Record of Borehole sheets included in Appendix B and on the Borehole Location and Soil Strata Drawing included in Appendix A. A general description of the stratigraphy, based on the conditions encountered in the boreholes, is given in the following sections. However, the factual data presented on the Record of Borehole sheets takes precedence over this general description for interpretation of the site conditions. It must be recognized that the soil and groundwater conditions will vary between and beyond borehole locations. Soil classification is in accordance with ASTM D 2487. Description of cohesive soils and secondary components are described as outlined in the MTO Guideline for Foundation Engineering Services Manual (April 2022).

In general, the encountered stratigraphy consisted of granular fill over a native deposit of gravelly silty sand underlain by silty clay over glacial till. The glacial till was composed of gravelly silty sand to silty sandy gravel and was underlain by dolostone bedrock.

5.1 Surficial Layers

Asphalt

All the boreholes were advanced from the road surface and encountered a 125 mm to 150 mm thick layer of asphalt.

Base

A base layer was encountered below the asphalt in all the boreholes. The layer was 0.2 m thick and was composed of gravelly silty sand fill.

5.2 Fill Materials

Sand Fill

Sand fill was encountered beneath the gravelly silty sand base layer in Boreholes 130-22-01 and 130-22-02. Trace amounts of fines were noted within the layer. The fill layer was 4.5 m to 5.7 m thick (base elev. 86.5 m to 85.5 m). SPT N-values in the fill materials ranged from 4 to 29, indicating a loose to compact relative density.

The recorded moisture contents ranged from 3 to 8%. The results of gradation analyses completed on two samples of the fill are illustrated on Figure C1 of Appendix C. The results of the tests are summarized below and on the Record of Borehole sheets in Appendix B.

Soil Particle	Percentage (%)
Gravel	0
Sand	91
Silt	9
Clay	

Silty Sand Fill

Silty sand fill was encountered beneath the sand fill in Boreholes 130-22-01 and 130-22-02 and below the gravelly silty sand base layer in Borehole 130-22-03. Some gravel was noted within the layer. The fill layer was 0.6 m to 2.3 m thick (base elev. 84.2 m to 83.2 m). SPT N-values in the fill materials ranged from 21 to 63 blows but were typically fewer than 42 blows, indicating a compact to dense relative density.

The recorded moisture contents ranged from 5 to 10%. The results of gradation analyses completed on two samples of the fill are illustrated on Figure C2 of Appendix C. The results of the tests are summarized below and on the Record of Borehole sheets in Appendix B.

Soil Particle	Percentage (%)
Gravel	19 – 20
Sand	43 – 50
Silt	30 – 38
Clay	

5.3 Gravelly Silty Sand (SM)

A native deposit of gravelly silty sand was encountered below the silty sand fill in Boreholes 130-22-01 and 130-22-02. Varying amounts of organics were noted in the layer. The layer ranged in thickness from 0.3 m to 0.5 m (base elev. 83.9 m to 82.7 m). SPT N-values in the layer were 14 and 23 blows, indicating a compact relative density.

The recorded moisture content of samples of the fill layer ranged from 11 to 26%. The results of a gradation analysis completed on one sample of the layer are illustrated on Figure C3 of Appendix C. The results of the test are summarized below and on the Record of Borehole sheets in Appendix B.

Soil Particle	Percentage (%)
Gravel	34
Sand	41
Silt	25
Clay	

5.4 Silty Clay (CI)

A deposit of silty clay was encountered below the gravelly silty sand in Boreholes 130 22-01 and 130-22-02 and below the silty sand fill in Borehole 130-22-03. The thickness of the layer ranged from 0.6 m to 3.3 m (base elev. 82.8 m to 79.4 m). Where SPTs were conducted within the layer, the N-values ranged from 5 to 15 blows. Blow counts as high as 38 blows were recorded near the base of the layer. Several attempts were made to carry out in-situ undrained shear strength



testing; however, the vane was unable to be turned, and the material is inferred to have undrained shear strengths greater than 102 kPa. The silty clay can be described as very stiff in consistency.

Recorded moisture contents ranged from 25 to 37%. Atterberg Limit testing was completed on three samples of the layer. Results are illustrated in Figure C4 of Appendix C. The results of these tests are summarized below and on the Record of Borehole sheets in Appendix B. The laboratory results indicate that the silty clay exhibits intermediate plastic behaviour (CI).

Parameter	Value
Liquid Limit	39 – 41
Plastic Limit	19 – 24
Plasticity Index	17 – 20

The results of gradation analyses completed on three samples of the layer are illustrated on Figure C5 of Appendix C. The results of the tests are summarized below and on the Record of Borehole sheets in Appendix B.

Soil Particle	Percentage (%)
Gravel	0
Sand	4 – 9
Silt	50 – 54
Clay	41 – 46

5.5 Gravelly Silty Sand (SC-SM) to Silty Sandy Gravel (GC-GM)

A native deposit of glacial till consisting of a mixture of silt, sand, and gravel was encountered below the silty clay in the boreholes. Varying amounts of plastic fines, cobbles, and boulders were also noted in the layer. The layer thickness ranged from 6.1 m to 11.6 m (base elev. 76.6 m to 67.8 m). SPT N-values in the layer ranged from 9 to 67 blows but were typically greater than 21 blows, indicating a compact to very dense relative density. Refusal blows counts were also observed, but this may represent the presence of cobbles and boulders. Coring was required to advance through cobbles and boulders.

The recorded moisture content of the layer ranged from 4 to 15%. The results of Atterberg Limit testing conducted on the fines portion of one tested sample from Borehole 130-22-02 indicate a non-plastic material. The results of gradation analyses completed on six samples of the layer are illustrated on Figure C6 of Appendix C. The results of the tests are summarized below and on the Record of Borehole sheets in Appendix B.

Soil Particle	Percentage (%)	
Gravel	16 – 50	
Sand	23 – 52	
Silt	27 – 47	26
Clay		6

5.6 Bedrock

Bedrock was proven by coring in the boreholes. The depth to bedrock from the existing road grade ranged from 11.5 m to 23.8 m (elev. 76.6 m to 67.8 m). The bedrock encountered consisted of slightly weathered to fresh, fine-grained, grey dolostone interbedded with sandstone. In general, the discontinuities were rough, undulating bedding joints. Bedrock logs are provided in Appendix B, and photographs of the bedrock cores are provided in Appendix C. The rock core quality and strength are summarized in Table 5-1.

Table 5-1: Bedrock Details

Parameter	Range
Total Core Recovery (TCR), %	86 – 100
Solid Core Recovery (SCR), %	33 – 100
Rock Quality Designation (RQD), %	42 – 100
Fracture Index (fractures per 0.3 m) ⁽¹⁾	0 – >10
Unconfined Compressive Strength (UCS) ⁽²⁾ , MPa	196

Notes: (1) Indicated as "FI" on Borehole Logs

(2) Sample tested from Boreholes 130-22-01

Based on the RQD, the bedrock quality is classified as poor to excellent (CFEM, 2006). The result of an unconfined compressive strength testing was 196 MPa, indicating that the bedrock is very strong (CFEM, 2006).

5.7 Groundwater

Observations of unstabilized water levels were completed in the open boreholes during and upon completion of drilling, however, water was used during the drilling operations and therefore this reading may not be representative.

At the completion of drilling, a standpipe piezometer of 19 mm in diameter was installed in Borehole 130-22-01 to allow for measurements of the groundwater level. The measured groundwater levels are summarized in Table 5-2.



Table 5-2: Groundwater Level Observations

Borehole	Groundwater Level		Date of Measurement	Comment
	Depth (mbgs)	Elevation (m)		
130-22-01	7.8	83.8	2022-12-01	Standpipe piezometer with a 1.5-m slotted screen installed with base at a depth of 14.9 m (elev. 76.7 m)
	7.6	84.0	2022-12-18	
	7.6	84.0	2023-04-26	

It should be noted that the values shown above are considered short-term readings and may not reflect groundwater levels at the time of construction, and seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after periods of significant and/or prolonged precipitation events.



6 MISCELLANEOUS

The borehole locations reflect existing site features and access constraints. The as-drilled locations and ground surface elevations were measured by Thurber. George Downing Estate Drilling Ltd. of Hawkesbury, Ontario, supplied and operated the drill rig used to drill, test, sample, install a standpipe piezometer, and decommission the boreholes. Traffic control and water were provided by T.G. Carroll Cartage Ltd. of Carp, Ontario. Traffic control was performed in accordance with Ontario Book 7 for short duration closures. The field work was supervised on a full-time basis by I. Khan, E.I.T., under the direction of K. Walker, P.Eng.

Geotechnical laboratory testing was carried out by Thurber's geotechnical laboratory in Ottawa, Ontario. Unconfined Compressive Strength testing was carried out by Stantec in Ottawa, Ontario.

Interpretation of the data and preparation of this report were carried out by A. de Oliveira, E.I.T., I. Khan, E.I.T., and M. Kennedy, P.Eng. The report was reviewed by Fred Griffiths, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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PART 2. ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 INTRODUCTION

This report presents the interpretation of the factual data obtained from a preliminary foundation investigation and a desktop review of the available subsurface information conducted by Thurber for the replacement of the existing Highway 401 Underpass at Highway 16 in Leeds and Greenville County, Ontario.

This preliminary 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. Contractors must make their own interpretation based on the factual data in Part 1 of the report. Where comments are made on construction, they are provided only in order to highlight those aspects which could affect the design of the project. Contractors must make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods and scheduling.

The following sections provide preliminary geotechnical recommendations for design of the foundations for the new underpass as part of the preliminary structural planning. The discussions and recommendations presented in this report are based on the information provided by AECOM and the MTO, and on the factual data obtained during the course of this investigation.

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

7.1 Existing Structure

The existing bridge carries two lanes in each direction of Highway 16 traffic over Highway 401 and is a one-span rectangular beam structure with reinforced cast-in-place concrete and was constructed in 1961. The bridge deck is approximately 38 m long and 21 m wide, with an approximate 12-degree skew to the highway. It is noted that for project orientation purposes, Highway 401 will be assumed to be oriented east-west and Highway 16 to be oriented north-south.



The original General Arrangement Drawing (Drawing No. TWP# 27-130-1-A, dated November, 1959) for the structure has been reviewed. The available historical drawings do not include a plan view of the foundation layout nor any foundation details of the existing structure. Geocres Report 31B00-033 recommended that the structure be supported either on spread footings on the glacial till or on piles or caissons.

7.2 Proposed Structure

The preliminary design of the Highway 16 Interchange to accommodate the proposed widening of Highway 401 includes realignment of Highway 16 approximately 15 m to the west of the existing structure. It is understood that the replacement overpass structure is to be a two-span structure with the pier constructed in the Highway 401 median. The new bridge will be up to about 15 m wider than the existing structure but will depend on the preferred configuration of and requirements for ramp lengths and active transportation. It is assumed that the embankment widening will be constructed with conventional side slopes without retaining walls. If retaining walls are considered, for preliminary purposes, the foundation types utilized to support the bridge should also be used to support the new retaining walls.

It is understood that the grade of Highway 16 is anticipated to be increased by approximately 0.8 m. The new approach embankments are anticipated to be on the order of 8 to 9 m high.

7.3 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 is in accordance with the Canadian Highway Bridge Design Code, version CSA S6:19, (CHBDC).

In accordance with CHBDC, the analysis and design of the structure 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 of Ontario (MTO).

It is understood that the structure is classified as being part of the “Major Route” importance category.

This project has been assigned Typical Consequence Classification, in accordance with Section 6.5.1 of the CHBDC. Accordingly, a consequence factor (Ψ) of 1.0, as per Table 6.1 of the CHBDC, has been used in assessing the factored geotechnical resistances. If the consequence classification changes, the geotechnical assessment and recommendations provided within this report will need to be reviewed and revised.

The degree of site and prediction model understanding for this site has been assessed to be typical understanding (Section 6.5.3 of CHBDC).

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)^{iv}. Seismic hazard data for this site has been obtained from the GSC's seismic hazard calculator. The data include peak ground acceleration (PGA), peak ground velocity (PGV), and the 5% damped spectral response acceleration values ($S_a(T)$) for the reference ground condition (Site Class C) for a range of periods (T) and for a range of return periods including the 475-year, 975-year and 2475-year events. The GSC seismic hazard calculation data sheet for this site is presented in Appendix E

The site coefficients used to determine the design spectral acceleration and displacement values are a function of the Site Class, the peak ground acceleration (PGA) and $S_a(0.2)$. The PGA for this location for a *reference* Site Class C with a 2% probability of exceedance in 50 years is 0.25 g (1 in 2475 year). This value is to be scaled by the $F(PGA)$ based on the site-specific Site Class as per Section 4.4.3.3 (Table 4.8) of the CHBDC (see Section 8.2).

8.2 CHBDC Seismic Site Classification

In accordance with the CHBDC, the selection of the seismic site classification is based on the soil conditions encountered in the upper 30 m of the stratigraphy below the reference elevation of the foundation element. As outlined in Section 4.4.3.2 of the CHBDC, if the shear wave velocity of the site soil and bedrock is not known, as is the case at this site, the seismic site class may be determined by the harmonic mean of the energy-corrected SPT-N values (N_{60}) and/or the undrained shear strength (s_u) encountered below the foundation element(s). Based on the range of N_{60} values recorded, a Site Class at these foundation elements may be taken as Site Class D.

The site-specific evaluation and classification should be completed during detailed design with measurement of the shear wave velocity in the upper 30 m of the stratigraphy below the reference elevation and should consider the influence of embankment fill thickness and composition at perched foundation elements.

8.3 Seismic Performance Category

In consideration of the Site Class D spectral values for the site and the designated *Major Route* importance category, the bridge structure would fall into either Seismic Performance Category 2, if the bridge has a fundamental period greater than or equal to 0.5 seconds, or Seismic Performance Category 3, if the bridge has a fundamental period less than 0.5 seconds, as per Section 4.4.4 (Table 4.10) of the CHBDC.

8.4 Liquefaction Potential

The susceptibility of the embankment fill and glacial till at the site to experience liquefaction was assessed using the SPT data following the simplified method for cohesionless soil as outlined in



Boulanger and Idriss (2014)^v. The analysis results indicated that the cohesionless soils are generally not susceptible to liquefaction.

The susceptibility of the cohesive soils at this site to experience liquefaction/cyclic softening was assessed following the Bray et al. (2004)^{vi} criteria using index properties. The results of the analysis indicate the cohesive material is not susceptible to cyclic mobility.

9 STRUCTURE FOUNDATION ALTERNATIVES

9.1 Foundation Alternatives

Based on the subsurface conditions at the site, both shallow and deep foundations have been considered for the replacement of the Highway 16 Underpass. The 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 G.

- Spread Footings

Spread footings are considered feasible for support of the replacement structure, provided they are founded on the dense glacial till or on engineered fill placed on the glacial till. The ground water table is anticipated to be perched within the embankment fill and therefore some groundwater control would be required during construction.

Spread footing foundations at the abutments do not allow for construction of integral abutments.

- Caissons

Caisson foundations, particularly when they are socketed into bedrock, offer high geotechnical resistance, however, the high lateral stiffness of caissons is not compatible or suitable for integral abutments. The high groundwater level within the embankment fill will pose additional construction challenges resulting from potential unbalanced hydraulic pressure heads and caisson base boiling when drilling through the deposits comprised of predominantly sand and silt. This would require the use of temporary liners or synthetic slurry to counterbalance groundwater pressure. The presence of cobbles and boulders in the till layer in all boreholes put down in this investigation could also present additional difficulties during caisson installation.

Caissons are considered feasible but are not the preferred option from a foundations perspective to support the new bridge.

- Driven Steel H-piles

Steel H-piles driven through the new approach embankments to support “perched” abutments may also be considered. Driven H-piles will typically reduce the volumes of excavation required when compared to shallow foundations. The use of H-Piles with

reinforced tips is the option with the least risk given the cobbles and boulders in the till layer observed at this site.

H-piles allow construction of integral abutments but minimum pile length to achieve the required lateral flexibility for an integral abutment configuration must be considered.

- Drilled-in Pipe Piles Socketed into Bedrock

Drilled-in steel pipe piles filled with concrete are considered to be a feasible option to support “perched” abutments. Pipe piles, pre-drilled to bedrock would eliminate the potential for piles meeting effective refusal in the cobbles and boulders encountered in the glacial till, and if socketed into bedrock could also be considered if additional support is required due to the seismically-induced load demand.

9.2 Recommended Foundation

Based on an evaluation of the foundation alternatives presented above and in Appendix G, the recommended foundation approach from a geotechnical perspective is to support the new bridge abutments on steel H-piles driven to refusal in the glacial till and the pier on a spread footing on glacial till. Construction staging could limit available construction space in the median of Highway 401; the use of piles or caissons to support the pier could alleviate this constraint.

9.3 Construction Methodology

It is assumed that staging areas for the bridge construction will be set up west of the existing bridge alignment and will be accessible from the existing Highway 401 and Highway 16 to minimize conflict with the Highway 401 and existing Highway 16 traffic. Due to the existing subgrade soils present, embankment construction in advance of construction of the foundation elements (including preload and surcharge) will likely be required for the new bridge approach embankments and abutment locations. It is anticipated that the majority of the construction of the new approach embankments and structure can take place while maintaining traffic flow on existing Highway 16. However, consideration of the staging requirements and impacts from the construction of the new structure and removal of the existing structure should be considered at subsequent design stages.

10 FOUNDATION DESIGN RECOMMENDATIONS

Key elevations (approximate) based on the results of the field investigation as follows:

- | | |
|---|----------------|
| • Highway 16 existing asphalt surface at the South abutment | 91.6 m |
| • Highway 16 existing asphalt surface at the North abutment | 91.4 m |
| • Highway 401 asphalt surface | 84.3 m |
| • Base of Fill | 83.2 to 84.2 m |
| • Top of Silty Clay | 82.7 to 83.9 m |
| • Top of Glacial Till | 79.4 to 82.8 m |



- Top of Dolostone Bedrock 67.8 to 76.6 m

As noted above, it is understood that the grade of Highway 16 is to be increased by approximately 0.8 m which sets the new pavement elevation for Highway 16 at approximately 92.3 m.

10.1 Spread Footings

Shallow spread footings should be founded on the dense glacial till.

At the pier, very dense glacial till was encountered directly beneath the Highway 401 pavement structure at an elevation of approximately 82.8 m. Near the proposed south abutment, SPTs carried out at Borehole 130-22-01 indicated that the glacial till deposit is compact to very dense, but generally very dense. Near the proposed north abutment, SPTs carried out in Borehole 130-22-02 in the glacial till indicate a dense to very dense material. The following table provides the maximum (highest) founding elevations recommended for the preliminary design of spread footings.

Table 10-1: Spread Footing Estimated Maximum Founding Elevations

Foundation Element	Borehole	Founding Stratum	Highest Footing Founding Elev. (m)
North Abutment	130-22-02	Dense to very dense Glacial Till	82.7
Pier	130-22-03	Very dense to compact Glacial Till	81.7
South Abutment	130-22-01	Compact to very dense Glacial Till	79.4

10.1.1 Geotechnical Resistance

Spread footings can be founded directly on the properly prepared glacial till. Spread footings on glacial till should be provided with at least 1.6 m of frost cover or equivalent insulation. Subgrade preparation should be as described in Section 10.1.2 and will include removal of unsuitable materials. Frost protection requirements are provided herein. Dewatering may be required to prepare the subgrade, place the bedding material and construct the foundations in the dry (Section 11.2).

Footings a *minimum* of 2.0 m wide and constructed within the glacial till as outlined above may be designed based on the following factored geotechnical resistances:

- Factored geotechnical resistance at ULS 750 kPa
- Factored geotechnical resistance at SLS 400 kPa

The factored geotechnical resistance at SLS corresponds to total footing settlement of 25 mm.



The bearing resistance values are for vertical, concentric loading. In the case of eccentric or inclined loading, the bearing resistance must be reduced in accordance with CHBDC Clause 6.10.3 and Clause 6.10.4.

The factored geotechnical resistances include the following factors:

- Consequence factor (Ψ) of 1.0 (as per CHBDC Table 6.1)
- Geotechnical resistance factors (as per CHBDC Table 6.2):
 - $\phi_{gu} = 0.5$ ULS (static analysis; typical degree of understanding)
 - $\phi_{gs} = 0.8$ SLS (static analysis; typical degree of understanding)

Resistance to uplift forces may be evaluated considering the weight of overburden/fill above the spread footings at the abutments. The magnitude of uplift resistance will depend on the footing and abutment wall dimensions, as well as the type of backfill material. If additional uplift resistance is needed for footings founded on glacial till, vertical anchors grouted into the underlying bedrock may be considered.

Resistance to lateral forces/sliding resistance between the concrete and the underlying Granular A bedding fill (Section 10.1.2) should be evaluated in accordance with the CHBDC assuming an unfactored coefficient of friction of 0.5. A resistance factor of 0.8 (as per CHBDC Table 6.2) should be used to estimate the sliding resistance between the concrete and Granular A or bedrock.

10.1.2 Subgrade Preparation

All organics, soft or loose deposits, disturbed soils, and deleterious materials must be stripped from the footprint of the foundations to expose competent glacial till subgrade at or below the desired founding elevations.

The glacial till subgrade soils may become disturbed when saturated and should be protected by prompt placement of a geotextile separator (Class II non-woven geotextile with a maximum FOS of 150 μm : OPSS.PROV 1860) and the bedding layer placed immediately after excavation and inspection. Cobbles and boulders should be anticipated in the glacial till; to provide a more uniform foundation subgrade condition for the spread footing, a minimum 300 mm thick layer of bedding material conforming to OPSS.PROV 1010 Granular A requirements should be placed on the undisturbed subgrade and compacted per OPSS.PROV 501.

10.2 Driven Steel H-Pile Foundations

10.2.1 Axial Resistance

The new abutments may be perched within the embankment fill and founded on steel H-piles. Cobbles and boulders were encountered or inferred in the glacial till at all foundation element borehole locations. Difficulty drilling or refusal of the sampler suggested the presence of cobble or boulders in the glacial till and required penetration by coring, or a combination of coring and wash boring techniques was encountered throughout the deposit. It is anticipated that piles will

not consistently be able to penetrate the till layer to reach bedrock. Accordingly, the pile recommendations provided below are based on driving the piles to refusal in till on cobbles and boulders; however, it should be noted that some of the piles may get past this layer and reach bedrock. The estimated pile tip elevations based on piles reaching refusal in glacial till or on the bedrock surface are summarized in Table 10-2.

Table 10-2: Estimated Pile Tip Elevations

Foundation Element	Approximate Underside of Pile Cap Elevation (m)	Estimated Top of Bedrock Elevation (m)	Estimated Pile Tip Elevation (m)	Estimated Pile Length (m)
South Abutment	87.0	67.8	74.0	13.0
Pier	81.7	72.8	75.0	6.7
North Abutment	87.0	76.6	76.6	10.4

The factored design parameters for axial resistance of Grade 350W HP 310x110 steel piles driven to refusal in glacial till can be taken as 1,800 kN at ULS and 1,500 kN at SLS.

The factored geotechnical resistances include the following factors:

- Consequence factor (Ψ) of 1.0 (as per CHBDC Table 6.1)
- Geotechnical resistance factors (as per CHBDC Table 6.2):
 - $\phi_{gu} = 0.4$ ULS (static analysis; typical degree of understanding)
 - $\phi_{gs} = 0.8$ SLS (static analysis; typical degree of understanding)

The structural resistance of the pile must be checked by the structural engineer which may govern the design.

The geotechnical resistance provided above are applicable for pile spacing greater than 3 pile widths. Driven piles must be installed in accordance with OPSS.PROV 903.

As the piles are to be driven into glacial till containing cobbles and boulders, the tips of the new piles at the site should be protected from damage during driving with pile tip protection from an approved manufacturer such as Titus Steel (standard H-Point) or approved equivalent.

If driven piles are employed, the effects of pile driving on nearby structures and underground utilities will need to be assessed during detailed design. The need for vibration monitoring and settlement monitoring of nearby utilities and structures during pile driving should be evaluated on a site-specific basis to limit potential impacts on existing facilities. A preconstruction condition survey of the existing structures and utilities in the vicinity may be required prior to commencement of pile installation.



10.2.2 Downdrag

A deposit of silty clay was identified beneath the native gravelly silty sand at both abutments of the existing bridge. Settlement of the clay is expected to occur if an 8 to 9 m embankment is constructed west of the existing embankment. Based on the preliminary data available this settlement will result in a preliminary unfactored downdrag load of up to 600 kN per pile at the abutments. Downdrag loads on piles supporting the central pier are not anticipated.

This downdrag load should be multiplied by a load factor as per CHBDC Table 3.3 and Commentary Clause C6.11.4.10 to obtain a factored downdrag load. In accordance with Section 6.11.4.10 of the CHBDC and Clause C6.11.4.10 of the Commentary, in the structural design of a pile, the factored downdrag load should be added to the factored permanent loads to assess the effects of downdrag. The factored dead and downdrag load should not exceed the factored structural resistance of a pile at the neutral plane.

The downdrag loading on new foundations could be reduced with construction of a preload prior to the installation of deep foundations, however, the impact of the preload on existing foundations would need to be assessed. Ultimately downdrag loading will be dependant on the grade raise and new embankment requirements and will need to be further assessed at the detailed design stage.

10.2.3 Uplift Resistance

The glacial till and embankment fill at the abutments (if the pile caps are perched within the embankments) will provide uplift resistance to the piles. Shaft friction of the embankment fill and glacial till along the piles were calculated, assuming the piles met effective refusal to driving at the elevations provided in Table 10-2, above.

The factored geotechnical tensile resistance for a single HP 310x110 pile at either abutment may be taken as 500 kN under static conditions and 1,500 kN under seismic conditions. For piles at the pier, the factored geotechnical tensile resistance for a single HP 310x110 pile may be taken as 185 kN under static conditions and 550 kN under seismic conditions. These values include the following factors:

- Consequence factor (Ψ) of 1.0
- Geotechnical resistance factors (CHBDC Table 6.2) of $\phi_{gu} = 0.3$ (static analysis; typical degree of understanding)
- Geotechnical resistance factors (CHBDC Table 6.3) of $\phi_{gu} = 1.0$ (seismic analysis; typical degree of understanding, performance-based design)

10.2.4 Lateral Resistance of Piles

P-Y data can be provided upon request following the receipt of a foundation layout plan. As a preliminary guideline, lateral resistance of the piles can be estimated using a value for the coefficient horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}).

For cohesionless soils:

$$k_s = n_h \cdot z / D \quad (\text{kN/m}^3)$$

$$p_{ult} = 3 \cdot \gamma' \cdot z \cdot K_p \quad (\text{kPa})$$

For cohesive soils:

$$k_s = 67 \cdot c_u / D \quad (\text{kN/m}^3)$$

$$p_{ult} = 0 \quad (\text{kPa}) \text{ at the top of the pile, increasing linearly to}$$

$$= 9 \cdot c_u \quad (\text{kPa}) \text{ at } z \geq 3 \cdot D$$

where:

$$z = \text{depth of embedment along pile (m)}$$

$$D = \text{pile width or diameter (m)}$$

$$n_h = \text{coefficient related to soil density (kN/m}^3\text{)}$$

$$c_u = \text{undrained shear strength (kPa)}$$

$$\gamma' = \text{effective unit weight (kN/m}^3\text{)}$$

$$K_p = \text{coefficient of passive lateral earth pressure}$$

The above equations and recommended parameters in Table 10-3 below may be used to analyse the interaction between a pile and the surrounding soil. The lateral pressures obtained from the analysis must not exceed the ultimate lateral resistance as defined above. A geotechnical resistance factor of 0.5 (ϕ_{gu}) and 0.8 (ϕ_{gs}), as per Table 6.2 of the CHBDC, is to be applied to the calculated ultimate ULS and SLS values, respectively.

The spring constant, K_s , for analysis may be obtained by the expression, $K_s = k_s \cdot L \cdot D$ (kN/m), where L is the length (m) of the pile segment or element used in the analysis. The ultimate lateral resistance, P_{ult} , may be obtained from the expression, $P_{ult} = p_{ult} \cdot L \cdot D$. This represents the ultimate load at which geotechnical failure of the pile occurs and will not support any additional load at greater displacement.

Table 10-3: Soil Parameters for Lateral Pile Design

Soil Type	γ' (kN/m ³)	n_h (kN/m ³)	c_u (kPa)	K_p
Existing/New ¹ Fill	20 (above WT ²)	6,000	N/A	3.0
Native Cohesive Soils	9 (below WT)	N/A	100	2.6
Glacial Till	11 (below WT)	8,000	N/A	3.0

Note: (1) Assuming new embankment fill consists of well-compacted engineered fill
(2) Water Table

The modulus of horizontal subgrade reaction may have to be reduced based on the pile spacing. The reduction factors to be used for a pile group oriented perpendicular or parallel to the direction

of loading are provided in Figures C6.23, C6.24 and C6.25 in the Commentary to the CHBDC 2019.

10.3 Drilled-In Pipe Piles

10.3.1 Axial Geotechnical Resistance

The new abutments may be founded on drilled-in steel pipe piles, end-bearing or socketed in the sound, very strong dolostone bedrock. The factored geotechnical axial resistances for various diameters of pipe piles socketed a minimum of the greater of two pipe diameters or 1.0 m into sound bedrock, with the bottom of the pipe pile seated on the base of the rock socket are provided in Table 10.4 below. A pipe pile wall thickness of 12.7 mm and a steel yield strength of 345 MPa is assumed for all sizes in Table 10.4.

Table 10.4: Factored Axial Geotechnical Resistances of Drilled-In Pipe Piles

Pipe Pile Diameter (mm)	Factored Axial Resistance	
	ULS (kN)	SLS (kN)
324	2,200	Will Not Govern
356	2,500	
406	3,000	

The factored geotechnical resistances include the following factors:

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

The depth of the rock socket will need to satisfy lateral load demands and structural capacities which will vary based on the diameter of the tube pile but should be a minimum of 1.0 m into sound bedrock and be clean and free of drill cuttings or other debris. The tube pile must reach the base of the rock socket and the steel must be seated onto bedrock. After installation, the pipe piles should be filled with 30 MPa tremie concrete.

The structural resistance of the pile must be checked by the structural engineer which may govern the design.

10.3.2 Downdrag

A deposit of silty clay was identified beneath the native gravelly silty sand at both abutments of the existing bridge. Settlement of the clay is expected to occur if an 8 to 9 m embankment is constructed west of the existing embankment. Based on the preliminary data available this settlement will result in a preliminary unfactored downdrag load of up to 600 kN per pile at the abutments. Downdrag loads on piles supporting the central pier are not anticipated.

This downdrag load should be multiplied by a load factor as per CHBDC Table 3.3 and Commentary Clause C6.11.4.10 to obtain a factored downdrag load. In accordance with Section 6.11.4.10 of the CHBDC and Clause C6.11.4.10 of the Commentary, in the structural design of a pile, the factored downdrag load should be added to the factored permanent loads to assess the effects of downdrag. The factored dead and downdrag load should not exceed the factored structural resistance of a pile at the neutral plane.

The downdrag loading on new foundations could be reduced with construction of a preload prior to the installation of deep foundations, however, the impact of the preload on existing foundations would need to be assessed. Ultimately downdrag loading will be dependant on the grade raise and new embankment requirements and will need to be further assessed at the detailed design stage.

10.3.3 Uplift Resistance

The embankment fill below the pile caps at the abutments will provide uplift resistance to the pipe piles. Shaft friction of these deposits along the piles were calculated, assuming the pipe piles are socketed into the sound bedrock as recommended in Section 10.3.1.

The factored geotechnical tensile resistance for individual pipe piles at either abutment is provided in Table 10.5 below.

Table 10.5: Uplift Resistances of Drilled-In Pipe Piles

Pipe Pile Diameter (mm)	Factored Uplift Resistance	
	Static (kN)	Seismic (kN)
324	450	1,350
356	500	1,500
406	550	1,700

These values include the following factors:

- Consequence factor (Ψ) of 1.0
- Geotechnical resistance factors (CHBDC Table 6.2) of $\phi_{gu} = 0.3$ (static analysis; typical degree of understanding)
- Geotechnical resistance factors (CHBDC Table 6.3) of $\phi_{gu} = 1.0$ (seismic analysis; typical degree of understanding, performance-based design)

10.3.4 Lateral Geotechnical Resistance

The lateral resistance provided by the soils may be determined using P-y curve data, which can be provided upon determination of the preferred deep foundation type and receipt of a foundation layout plan at the detailed design phase. As a preliminary guideline, lateral resistance of the soil can be estimated using a value for the coefficient horizontal subgrade reaction (k_s) and ultimate lateral resistance (pult), as described in Section 10.2.4, above.

10.4 Caisson Foundations

Support of the abutments and/or central pier may be provided by caisson foundations. The glacial till at the site contains cobbles and boulders. The caissons would have to be constructed by installing a steel casing into the top of the bedrock using drilling methods that would allow reliable penetration through potential cobbles, boulders and other obstructions that may be encountered in the fill and till layers. A socket would then be drilled below the steel casing and into the bedrock, cleaned, and the casing and socket would be filled with concrete in a single pour after installation of reinforcing steel. Coring equipment must be able to seat the casing into the bedrock and penetrate the bedrock without fracturing the sidewalls. The tension/uplift resistances provided are based on full contact of the caisson concrete with the socket sidewalls. Depth of socket should be measured downward from the lower bedrock elevation for a sloping bedrock scenario. Caissons should be installed in accordance with OPSS.PROV 903. The strength and hardness of bedrock at this site must be considered when selecting equipment to excavate the rock socket.

Given the risk of the till layer sloughing, the caisson construction method should include use of temporary or permanent casings (liners) sealed into the bedrock. The caisson socket would extend into the bedrock below. Using a temporary casing that is extracted during the concrete pour to reduce material costs is feasible, if the caissons are installed in combination with drilling fluid to maintain the stability of the side walls. The use of temporary casing would require careful control of the concrete level. Alternatively, caisson casings may be left in place as permanent liners to reduce the potential for disturbance of the soil-concrete interface that may occur during removal of temporary liners. Permanent liners would assist in maintaining the integrity of the concrete caisson by reducing the risk of infiltration of soil or water prior to concrete curing. Ultimately, the contractor will be responsible for selecting the construction means and methods based on cost and risk considerations.

The Contractor shall use appropriate means to clean and inspect the caisson base. The Contractor shall apply means necessary (such as air lift pump or hydraulic pump, etc.) to clean the base of the caissons. The base cleaning method, inspection method, and any additional measures required to satisfy the acceptance criteria must be selected by the Contractor to ensure direct contact between the concrete and un-weathered bedrock over the entire area of the base.

10.4.1 Axial Resistance

The axial geotechnical capacity at factored ULS for a caisson with a permanent steel casing filled with concrete and socketed a minimum of 2 caisson diameters below the steel casing and into sound bedrock is provided in the table below. The caisson capacities include resistance factors (ϕ_{gu}) of 0.4, 0.3 and 1.0 for compressive, tensile-static and tensile-seismic resistance, respectively, at ULS as per Table 6.2 of the CHBDC (static analysis – typical understanding). The SLS condition will not govern for a caisson socketed into sound bedrock.

Table 10-6: Axial Geotechnical Resistance for Caissons

Caisson Diameter (mm)	Factored ULS Compression (kN)	Factored SLS Compression (kN)	Factored ULS Tension Static (kN)	Factored ULS Tension Seismic (kN)
915	11,000	will not govern	8,000	27,500
1220	15,000	will not govern	11,000	37,500
1525	25,000	will not govern	18,500	62,500
1830	38,000	will not govern	28,500	95,000

The structural resistance of the caissons must be checked by the structural designer. The depth of socket into sound bedrock should be lengthened, if required, based on the required lateral capacity requirements (recommendations provided in Section 10.2.4), moment capacity and seismic analysis to satisfy the structural assessment.

10.4.2 Downdrag

A deposit of silty clay was identified beneath the native gravelly silty sand at both abutments and central pier of the existing bridge. Settlement of the clay is expected to occur if an 8 to 9 m embankment is constructed west of the existing embankment. Based on the preliminary data available this settlement will result in a preliminary unfactored downdrag load of up to 100 kN per pile at the abutments.

This downdrag load should be multiplied by a load factor as per CHBDC Table 3.3 and Commentary Clause C6.11.4.10 to obtain a factored downdrag load. In accordance with Section 6.11.4.10 of the CHBDC and Clause C6.11.4.10 of the Commentary, in the structural design of a pile, the factored downdrag load should be added to the factored permanent loads to assess the effects of downdrag. The factored dead and downdrag load should not exceed the factored structural resistance of a pile at the neutral plane.

The downdrag loading on new foundations could be reduced with construction of a preload prior to the installation of deep foundations, however, the impact of the preload on existing foundations would need to be assessed. Ultimately downdrag loading will be dependant on the grade raise and new embankment requirements and will need to be further assessed at the detailed design stage.

Downdrag forces acting upon pier caissons are expected to be negligible provided the grade of the Highway 401 median is reinstated to match existing.

10.4.3 Lateral Resistance

Lateral bridge loadings can be geotechnically resisted by the caissons through passive pressure developed along the embedded portion of the shaft. The methodology outlined in Section 10.2.4 above for driven piles may be used to estimate the lateral geotechnical resistance of the caissons.



10.5 Frost Depth

The depth of frost penetration at this site is estimated to be 1.6 m (as per OPSD 3090.101). Footings and pile caps should be founded at or below this depth or provided with equivalent insulation. Caisson foundations socketed into bedrock and extending vertically up to the pile cap do not require minimum depths for frost protection.

10.6 Impact on Local Infrastructure

Pile driving and construction activities such as excavations must be planned and carried out in a manner that does not impact or destabilize adjacent infrastructure. A CNR railway line runs below the Highway 16 overpass. CNR must be contacted prior to any construction activities to determine vibration limits and maximum allowable particle velocity during construction activities. It is recommended that a Contract Provision be included alerting the contractor of the vibration and settlement limits.

Vibration and settlement monitoring of the CNR railway is recommended during construction activities, such as excavations and installation of pier foundations.

If vibrations exceed the maximum allowance provided for the local infrastructure through pile driving activities, pre-augering for grouted piles may be an option.

10.7 Backfill and Lateral Earth Pressures

Backfill to the abutments should consist of free-draining granular material conforming to OPSS Granular A or Granular B Type II specifications. The granular material should be placed to the extents shown in OPSD 3101.150. Compaction should be carried out in accordance with OPSS.PROV 206 and OPSS.PROV 501.

Lateral earth pressure parameters provided in Table 10-7 and Table 10-8 in the sections below consider that the wall is vertical and the backfill is fully drained so that there are no unbalanced hydrostatic pressures above the permanent groundwater level. Where back slopes are horizontal, the corresponding coefficients provided in Table 10-7 and Table 10-8 should be used. If other backfill and wall geometries are to be considered, Thurber will need to calculate the appropriate earth pressure coefficients.

10.7.1 Static Lateral Earth Pressure

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

$$\sigma_h = K * (\gamma \cdot h + q)$$

where:

$$\sigma_h = \text{horizontal pressure on the wall at depth } h \text{ (kPa)}$$

K	=	earth pressure coefficient (see Table 10-7) (K_a for yielding walls, K_o for non-yielding walls)
γ	=	unit weight of retained soil (see Table 10-7), use submerged unit weight below groundwater level
h	=	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. Typical lateral earth pressure parameters for use in the design for a horizontal back-slope are provided in Table 10-7.

Table 10-7 Static Earth Pressure Coefficients

Parameter	OPSS Granular A & B Type II	OPSS Granular B Type I	OPSS SSM & Existing Granular Fill
Soil Unit Weight, kN/m^3 , γ	22.8	21.2	20.0
Angle of Internal Friction, ϕ	35°	32°	30°
Coefficient of at Rest Earth Pressure, K_o (Restrained Wall)	0.43	0.47	0.50
Coefficient of Active Earth Pressure, K_a (Unrestrained Wall)	0.27	0.31	0.33
Passive, K_P (Movement towards Soil Mass) in front of wall	3.7	3.3	3.0

The parameters in the table correspond to full mobilization of active and passive earth pressures and require certain relative movements between the wall and adjacent soil to produce these conditions. The movement required can be assessed from Table C6.12 of the Commentary to the CHBDC. Active earth pressures should be used for unrestrained walls. For rigid structures, at-rest horizontal earth pressures would apply for design.

10.7.2 Combined Static and Seismic Lateral Earth Pressure

In accordance with Clause 6.14.7.2 of the CHBDC, retaining structures should be designed using dynamic earth pressure coefficients that incorporate the effects of earthquake loading. The following recommendations are per Section C6.14.7.2 of the Commentary of the CHBDC which states that seismically induced lateral soil pressures may be calculated using the Mononobe-Okabe Method with:

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

The coefficients of horizontal earth pressure for combined static and seismic loading presented in Table 10-8 may be used for a horizontal back-slope. The provided earth pressure coefficients are calculated using a site-adjusted PGA of 0.26 g, based on a Seismic Site Class D, a reference



(Site Class C) PGA with a 2% probability of exceedance in 50 years of 0.248 g (Geological Survey of Canada – Fifth Generation) and a F(PGA) of 1.047 as per Table 4.8 of the CHBDC.

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 soils profile.

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

where:

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

Table 10-8 Combined Static and Seismic Earth Pressure Coefficients

Parameter	OPSS Granular A & B Type II	OPSS Granular B Type I	OPSS SSM & Existing Granular Fill
Soil Unit Weight, kN/m ³ , γ	22.8	21.2	20.0
Angle of Internal Friction, ϕ	35°	32°	30°
Non-Yielding Wall			
Dynamic Active Earth Pressure Coefficient, K_{AE}	0.44	0.49	0.53
Yielding Wall			
Dynamic Active Earth Pressure Coefficient, K_{AE}	0.35	0.39	0.42

10.8 Embankment Stability

Based on the available original structure drawings and observations during the 2022 field investigation, the grade of the current Highway 16 lanes ranges from about 91.4 m to 91.6 m near the Highway 401 underpass with embankments of up to about 8 m in height and sloped at roughly 2H:1V. Near the south abutment, the west side of the south embankment is as steep as about 1.6H:1V.

The embankment to be constructed for the proposed new alignment will shift the centerline of the roadway approximately 15 m to the west and is anticipated to be roughly 9 m in height. For the purpose of preparing preliminary geotechnical design recommendations, a number of assumptions have been made that are consistent with MTO's standard highway design practices:

- Topsoil and other deleterious material will be removed from within the footprint prior to constructing the new embankment.
- Where new fill is placed against an existing embankment slope or on a sloping ground surface steeper than 3H:1V, the existing slope will be benched (OPSD 208.010).
- The embankment will be constructed using granular fill meeting at a minimum the requirements of Select Subgrade Material (SSM) as per OPSS.PROV 1010.
- Conventional (non-reinforced) granular fill embankments will be constructed with side slopes not steeper than 2H:1V.
- Granular fill embankments greater than 9 m in height will be provided with a 2 m wide mid-height berm.
- Permanent drainage and erosion protection will be provided for all granular embankment slopes.

Table 6.2 of the CHBDC for embankment fills with a *typical* degree of site understanding and a *typical* consequence level (a Consequence Factor, Ψ , of 1.0) generates minimum Factors of Safety of 1.5 and 1.3 for static permanent and static temporary conditions respectively.

For seismic analysis, Table 6.3 in Section 6.14.4.1 of the CHBDC indicates a minimum resistance factor of 0.95 ($\phi_{gu, static(temporary)} = 0.75 + 0.2$) for force-based design and 1.0 for performance-based design. Based on these values and Ψ of 1.0, a target Factor of Safety of 1.1 for this temporary condition with a typical degree of understanding is appropriate for the pseudo-static seismic analysis. However, as is stated in Section 6.14.9.1 of the CHBDC, some embankment displacement can occur where the pseudo-static Factor of Safety is less than 1.3; in this case, the bridge foundations must be designed to withstand the permanent deformations and/or slope stabilizing measures shall be incorporated into the design. Where the pseudo-static Factor of Safety is greater than or equal to 1.3, the slope is considered to be seismically stable with deformations of less than 50 mm.

In addition, Sections 6.14.2.1 and 6.14.2.3 of the CHBDC present performance criteria requirements for Major Route geotechnical systems (embankments) inside and outside the bridge interface zone, respectively. Based on Clause 6.14.2.2, the bridge interface zone at this site extends to 20 m behind the abutments (based on fill heights of up to about 8 m). The performance criteria for the Major Route embankments are as follows:

- Within the bridge interface zone (bridge approaches): 100% of the travelled lanes shall be available for use following a ground motion event with a return period of at least 475 years.
- Outside the bridge interface zone (beyond bridge approaches): sites that fall within Seismic Performance Category 2 or 3 (See Section 8.3) shall have at least 50% of travelled lanes, but not less than one, available for use following ground motions with a return period of at least 475 years.



The stability analyses considered design PGA values of 0.26 g and 0.12 g for ground motions with return periods of 2,475 and 475 years, respectively, based on a Seismic Site Class D, a reference (Site Class C) PGA as per Section 4.4.3.2 of the CHBDC.

Slope stability assessments have been carried out for the highest/critical embankment slope, just behind the south abutment. Embankment slope stability was evaluated using GeoStudio 2021 Slope/W software for limit equilibrium analysis. Input parameters for the analyses are based on the findings of the 2022 boreholes and the results of laboratory testing. The following additional parameters were used in the analysis:

- Estimated soil stratigraphy based on the existing ground surface contours and nearest boreholes;
- Embankment maximum fill height of approximately 8.8 m;
- For analysis, a seismic event with a return period 2,475 years site adjusted PGA value of 0.13 g, equal to $\frac{1}{2}$ of the site adjusted PGA value (0.26 g) was used, as per Section 4.4.3.3 of the CHBDC;
- A traffic surcharge of 17 kPa was applied as a temporary load; and,
- The slip surface was forced to edge of traffic lane to eliminate non-consequential surficial failures.

The results of the stability analyses are provided on the figures presented in Appendix H. Each figure shows the slope geometry, groundwater conditions, soil stratigraphy and soil strength parameters utilized in the analysis.

The stability analyses generated the following factor of safety values for the critical embankment slope:

Table 10-9 Slope Stability Analysis Results

Condition	Case	Factor of Safety
Permanent	Long-Term Static (Drained)	1.5 (Fig H1)
Temporary (traffic loading)	Short-Term Static (Undrained)	1.5 (Fig H2)
Temporary (seismic loading)	Pseudo-Static Seismic, 2,475-yr (Undrained)	1.1 (Fig H3)
	Pseudo-Static Seismic, 475-yr (Undrained)	1.3 (Fig H4)

All of the static results presented in Appendix H achieve the target Factors of Safety described above. The pseudo-static result considering the 2,475-year earthquake presented in Figure H3 of Appendix H exceeds the target Factor of Safety of 1.1 for seismic design. However, it is noted that some embankment displacement can occur where the pseudo-static Factor of Safety is less than 1.3 as is the case for the 1 in 2,475 year seismic event (Figure H3). Additional analyses were carried out for that case to determine if performance criteria would be met for the Major Route

geotechnical systems inside and outside the bridge interface zone. Pseudo-static analyses considering the 475-year earthquake event were completed and yielded a factor of safety of 1.3 for the south abutment at that location (Figure H4) indicating that the performance requirements would be met for that scenario.

10.9 Embankment Settlement

The settlement resulting from a 9 m high embankment constructed with conventional granular fill on a new alignment approximately 15 m to the west of the existing alignment was assessed using the multi-layer settlement analysis in Rocscience's Settle3 software. Subsurface stratigraphy was based on the boreholes drilled in the area. Loading was applied based on the assumed geometry of the new alignment using a unit weight for new embankment SSM fill of 20 kN/m³. The water table was defined based on piezometer readings. It is noted that engineering judgment and experience was used to select the material properties based on the stress range anticipated due to loading. Consolidation testing will be required during detailed foundation investigations. The following discussion and recommendations must be carefully reviewed once site specific compression parameters are available. The soil parameters used in the analysis of the proposed new embankment are presented in Table 10-10 below.

Table 10-10: Properties of Soil Used in Settlement Calculations

Property	Gravelly Silty Sand	Silty Clay	Glacial Till
Unit Weight [kN/m ³]	19	17.5	19
E [kPa]	15,000	-	135,000
e _o	-	1.1	-
P _c ' [kPa]	-	>350	-
C _c	-	0.25	-
C _r	-	0.05	-

The largest contributor to the settlement will be the silty clay layer which varies in thickness from 0.6 m to 3.3 m in the boreholes. Based on the above parameters and loading from the assumed new embankment geometry, the expected total settlement is expected to range from 25 mm to 120 mm at the centerline of the embankment just behind the abutment. As the silty clay layer is heavily over-consolidated and relatively thin, the settlement will occur fairly quickly; the majority of the settlement is expected to be completed within six months of fill placement.

Based on the estimated settlement and time required for that settlement to occur, a preload period of at least 6 months is recommended prior to bridge construction. The embankment preload should be overbuilt to accommodate settlement during the preload period, the settlement should be monitored with monitoring points to confirm when the preload period is complete. Options such as wick drains to expedite the preload time period could be assessed during detailed design.



Provided the new embankment is constructed with a six-month preload period prior to bridge construction, the approach embankments are expected to meet the MTO guidelines for settlement of approach embankments behind bridge abutments for post construction settlement over a period of 20 years after paving as outlined below:

- 25 mm within 20 m of the structure;
- 50 mm from 20 to 50 m from the structure;
- 75 mm from 50 to 75 m from the structure; and
- 100 mm greater than 75 m from the structure

When the preferred new alignment geometry is determined during detailed design, the impact of the new embankment settlement on the existing bridge foundations and lanes of Highway 16 and Highway 401 will need to be assessed. For preliminary purposes, it is estimated that the settlement generated by construction of the new embankment alignment at the western crest of the existing embankment will be in the order to 50 mm near the existing bridge and occur during the preload period.

11 CONSTRUCTION CONSIDERATIONS

11.1 Temporary Excavations

Excavations at the abutments will extend through the existing embankment and highway fill and, in the case of spread footings at the abutments, down to the underlying glacial till. All temporary excavations must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of OHSA, existing fill or glacial till may be classified as Type 3 soil. Unsupported excavations made in Type 3 soils must have side slopes no steeper than 1H:1V from the base of the excavation.

At locations where there are space restrictions or where a slope must be retained, the excavations will need to be carried out within a protection system. Further discussion on temporary protection systems (TPS) should be provided at a subsequent design stage, as required, however it is noted that sheet pile TPS could be problematic at this site due to the cobbles and boulders in the till and the presence of relatively shallow bedrock.

11.2 Surface and Groundwater Control

At the site, the natural groundwater level was measured (in a piezometer installed in Borehole 130-22-01 at the south abutment with screen in the glacial till) to be perched within the embankment fill, up to about 4.6 m above the glacial till surface at Elevation 84.0 m. Surface runoff will also tend to seep into and accumulate in excavations. The Contractor must control groundwater, perched groundwater, and surface water flow at the site to permit foundation construction, subgrade preparation, and placement and compaction of granular bedding must be carried out in a dry and stable excavation.

Based on the anticipated depths of excavation required for spread footings or pile caps, for temporary excavations it is considered likely that groundwater may be controlled with sump pumps in the bottom of the excavations.



Excavations into native soils which do not penetrate to the glacial till should be reviewed during detailed design for potential basal heave due to unbalanced hydrostatic pressures on the base of the silty clay layer.

12 RECOMMENDED SCOPE FOR DETAIL DESIGN

The recommendations provided above are in support of the preliminary design of the proposed replacement of the Highway 16 - Highway 401 Underpass (Site No. 16X-0130/B0) as part of the overall Preliminary Design and Environmental Assessment for the widening of Highway 401 from Highway 416 to Maitland Road. Additional foundation investigation will be required following the selection of the Technically Preferred Alternative (TPA). Additional field investigation should be carried out to provide additional foundation design input to the following:

- Shear wave velocity measurements below the foundation elements to confirm Seismic Site Classification.
- Testing of soil and/or groundwater at the site to determine degree of corrosiveness of the sub-surface environment and potential implications to steel and concrete elements in contact with the soil and groundwater at the site.
- Consolidation testing of the silty clay deposit to confirm settlement magnitude parameters and develop time rate of settlement characteristics.

The required supplementary foundation field investigation scope should be reviewed following the selection of the TPA.

13 CLOSURE

Engineering analysis and preparation of this report was carried out by Mr. Matt Kennedy, P.Eng. The report was reviewed by Paul Carnaffan, P.Eng. and Fred Griffiths, P.Eng., a Designated Principal Contact for MTO Foundation Projects.

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REFERENCES

- ⁱ <https://files.ontario.ca/mnrf-ecosystemspart1-accessible-july2018-en-2020-01-16.pdf>
- ⁱⁱ <https://files.ontario.ca/ecosystems-ontario-part2-03262019.pdf>
- ⁱⁱⁱ <http://www.geologyontario.mndm.gov.on.ca/index.html>
- ^{iv} <https://earthquakescanada.nrcan.gc.ca/hazard-alea/interpolat/calc-en.php>
- ^v Boulanger, R. W., and Idriss, I. M. (2014). CPT and SPT based liquefaction triggering procedures, Report No. UCD/CGM-14/01, Center for Geotechnical Modeling, Department of Civil and Environmental Engineering, University of California, Davis, CA, 134 pp.
- ^{vi} Bray, J. D. et al. (2004b). Liquefaction susceptibility of fine-grained soils. Proc., 11th Int. Conf. on Soil Dynamics and Earthquake Engineering and 3rd Int. Conf. on Earthquake Geotechnical Engineering, D. Doolin et al., eds., Stallion Press, Singapore, 655–662

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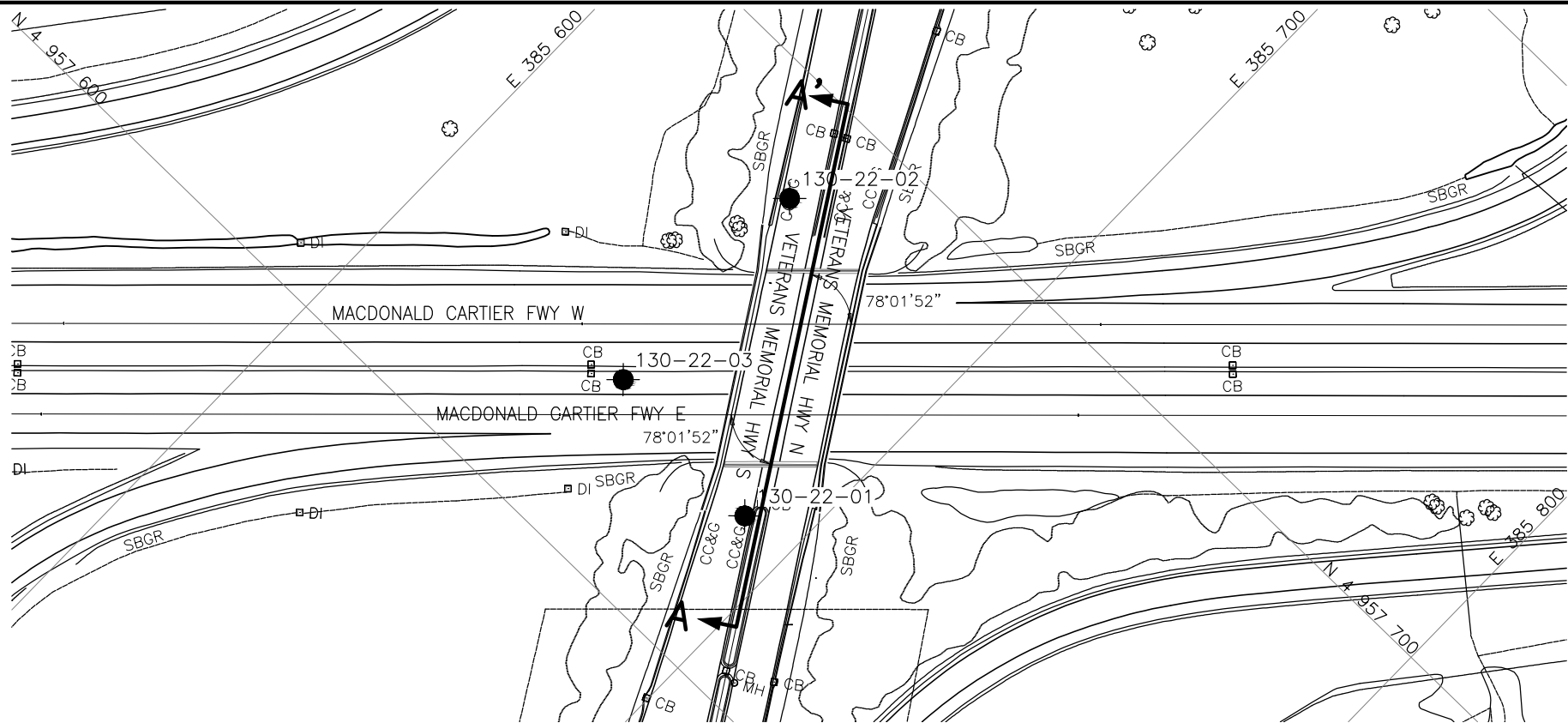
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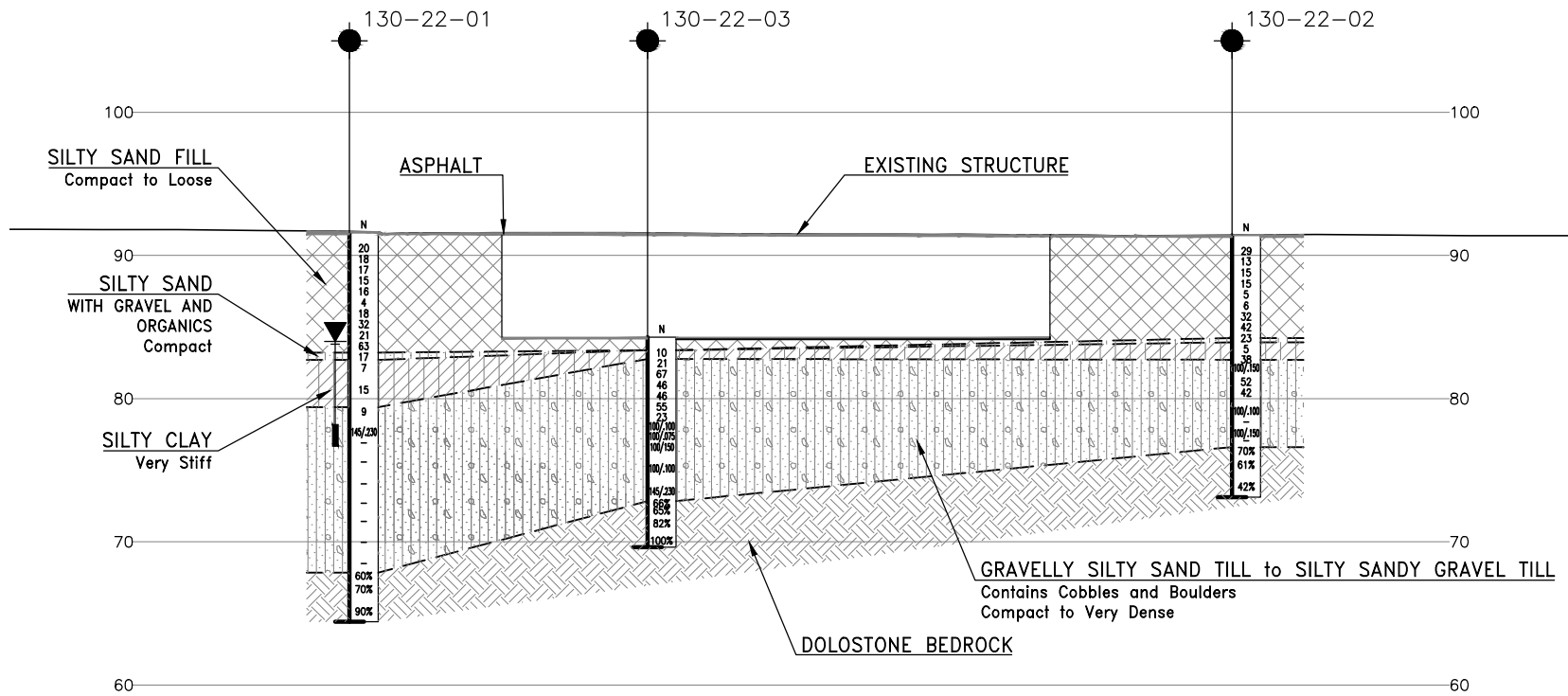


Appendix A.

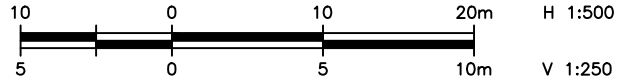
Borehole Location Plan and Stratigraphic Drawing



PLAN



SECTION A-A'



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



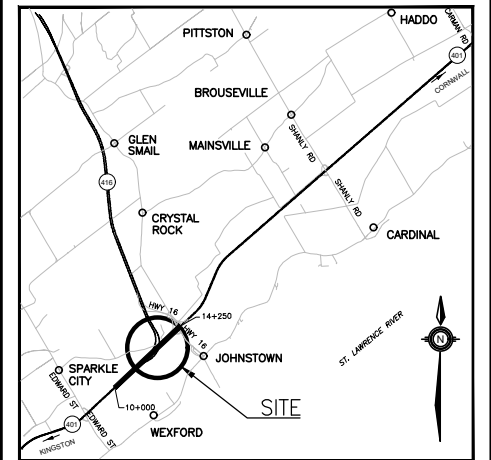
CONT No
GWP No 4024-20-00

HIGHWAY 401
HIGHWAY 16 UNDERPASS
BRIDGE REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA



SHEET

Ontario



KEYPLAN

LEGEND

	Borehole (Current Investigation)
	Borehole (Previous Investigation)
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

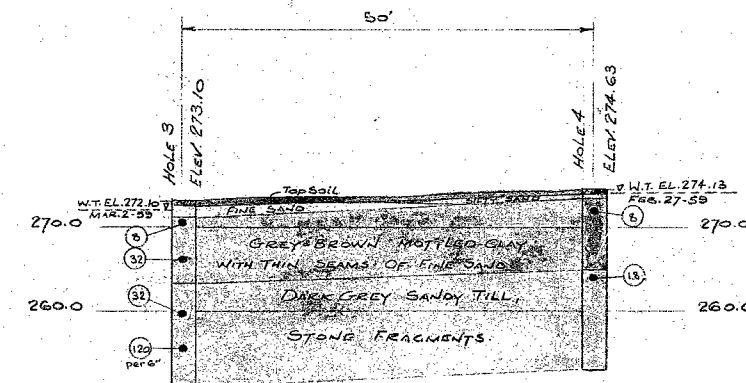
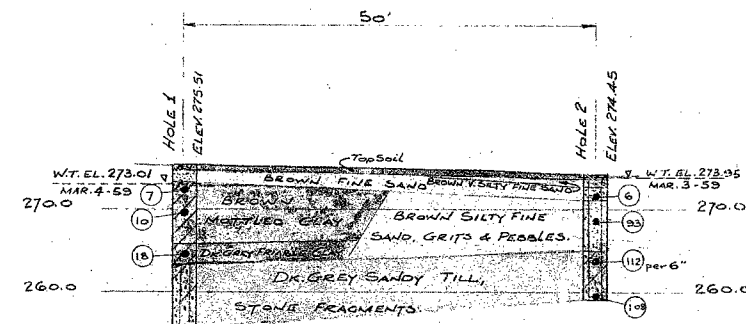
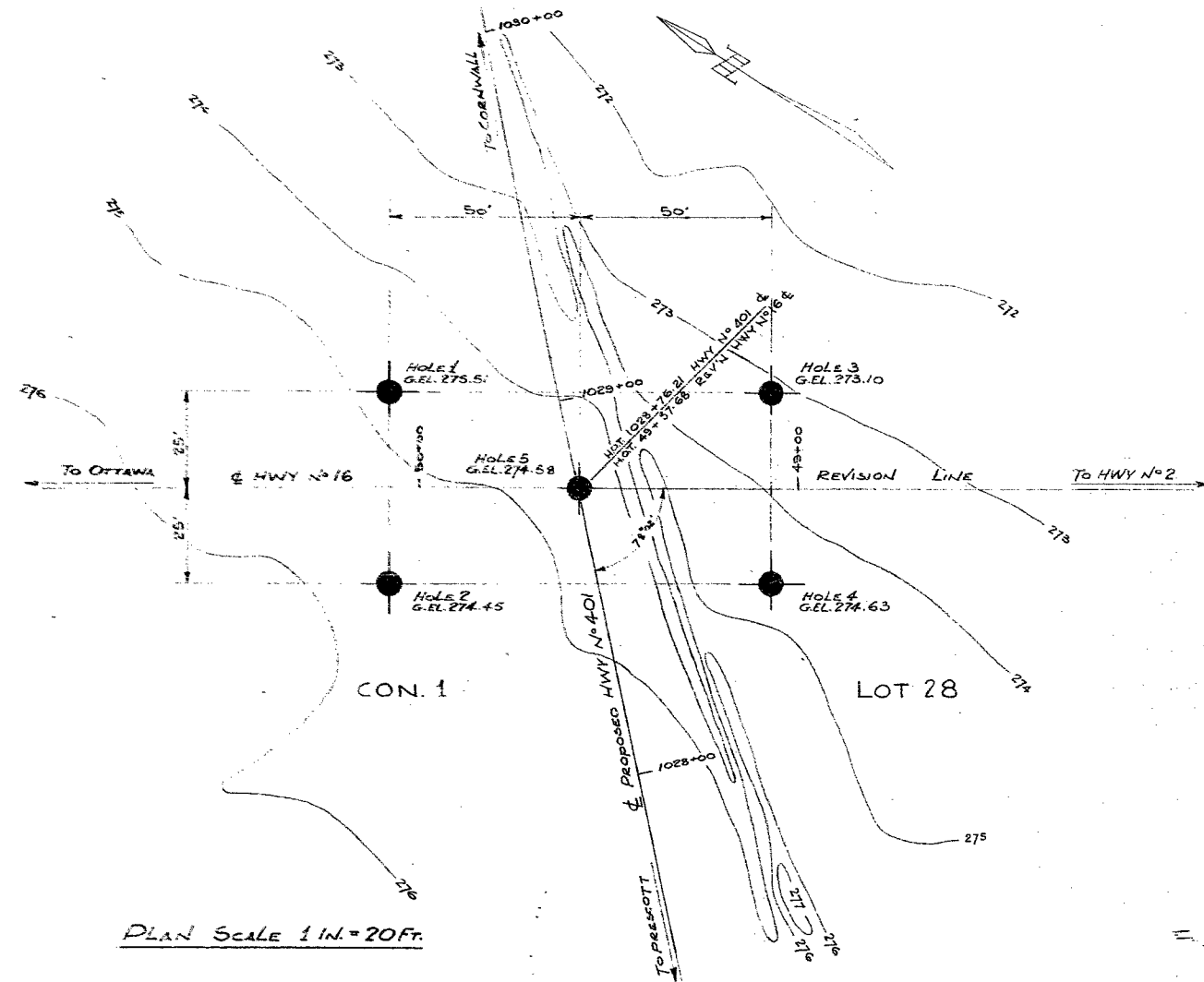
NO	ELEVATION	NORTHING	EASTING
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130-22-02	91.4000	4957679.9700	385652.4100
130-22-03	84.3000	4957632.5500	385653.7000

-NOTES-

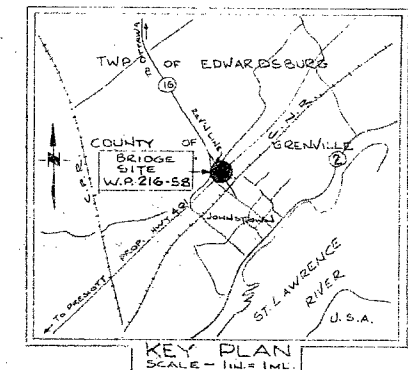
- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- This drawing is for subsurface information only. Surface details and features are for conceptual illustration.
- Coordinate system is MTM NAD 83 Zone 9.

GEORES No. 31B-112

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	MJK	CHK -	CODE
DRAWN	MC	CHK MJK	SITE 16-250/C/STRUCT
			LOAD
			DATE JUNE 2023
			DWG 1



NOTE: - PLEASE SEE BOREHOLE LOGS FOR COMPLETE SOIL DETAILS.



PROFILES.
SCALES: HOR 1" = 10'
VERT.

LEGEND.
● TEST HOLE.
○ BLOWS/FOOT.
○ STD. PENETRATION TEST.
W.T. - GROUND WATER TABLE IN SOIL.

NOTE: -
THE BOUNDARIES BETWEEN SOIL STRATA HAVE BEEN ESTABLISHED ONLY AT BOREHOLE LOCATIONS. A LINEAR VARIATION IN SOIL STRATIGRAPHY HAS BEEN ASSUMED BETWEEN BOREHOLES, AND THIS MAY ACTUALLY DIFFER FROM THAT SHOWN.



e.m. peto & associates ltd.
SOIL SITE INVESTIGATION
AT
CROSSING HWY 401-HWY 16
EDWARDSBURG TOWNSHIP BRIDGE NO. 4
FOR
DEPARTMENT OF HIGHWAYS OF ONTARIO
OUR JOB No. 5819 DATE: MAR. 13-59
CLIENTS PLAN No. E 3556-1 PER: G.T.



Appendix B.

Record of Borehole Sheets



SYMBOLS, ABBREVIATIONS AND TERMS USED ON TEST HOLE RECORDS

TERMINOLOGY DESCRIBING COMMON SOIL GENESIS

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

TERMINOLOGY DESCRIBING SOIL STRUCTURE:

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

RECOVERY:

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

N-VALUE:

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

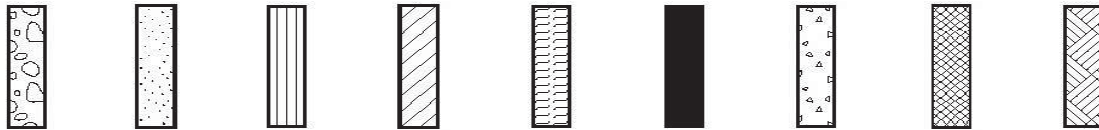
DYNAMIC CONE PENETRATION TEST (DCPT):

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



STRATA PLOT:

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



Boulders
Cobbles
Gravel Sand Silt Clay Organics Asphalt Concrete Fill Bedrock

TEXTURING CLASSIFICATION OF SOILS

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

TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

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

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

SAMPLE TYPES

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

TERMS DESCRIBING CONSISTENCY (COHESIONLESS SOILS ONLY)

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

MODIFIED UNIFIED SOIL CLASSIFICATION

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

Note - W_L = Liquid Limit



EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION

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

TERMS

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

DISCONTINUITY SPACING

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

STRENGTH CLASSIFICATION

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

RECORD OF BOREHOLE No 130-22-01

1 OF 3

METRIC

GWP# 4024-20-00 LOCATION Lat: 44.754103°, Long: -75.478361° Highway 16 & 401, Edwardsburgh, MTM z9: N 4 957 630.0 E 385 688.8 ORIGINATED BY IK
 HWY 401 BOREHOLE TYPE CME 55 Truck Mount / HSA / NW Casing / NQ Coring COMPILED BY AO
 DATUM Geodetic DATE 2022.11.29 - 2022.12.01 CHECKED BY KW

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
91.6	Ground Surface												
0.0	ASPHALT (150 mm)												
0.2	GRAVELLY SILTY SAND												
0.4	Brown FILL (BASE)												
	SAND, trace fines Compact to loose Brown FILL		1	SS	20								
			2	SS	18								
			3	SS	17								
			4	SS	15								
			5	SS	16								
			6	SS	4								
			7	SS	18								
85.5													
6.1	SILTY SAND, some gravel Compact to very dense Brown FILL		8	SS	32								
			9	SS	21								
			10	SS	63								
83.2													
8.4	Gravelly SILTY SAND (SM) Contains organics Compact Grey		11	SS	17								
82.7													
8.9	SILTY CLAY (CI) Very stiff Brown to grey		12	SS	7								

Continued Next Page

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

DOUBLE LINE 29381 BOREHOLE LOGS REPLACEMENT SITES.GPJ 2012TEMPLATE(MTO).GDT 10-30-23

METRIC

[illegible]

+³, ×³: Numbers refer to Sensitivity

DOUBLE LINE 29381 BOREHOLE LOGS REPLACEMENT SITES.GPJ 2012TEMPLATE(MTO).GDT 10-30-23

RECORD OF BOREHOLE No 130-22-01

3 OF 3

METRIC

GWP# 4024-20-00 LOCATION Lat: 44.754103°, Long: -75.478361° Highway 16 & 401, Edwardsburgh, MTM z9: N 4 957 630.0 E 385 688.8 ORIGINATED BY IK
 HWY 401 BOREHOLE TYPE CME 55 Truck Mount / HSA / NW Casing / NQ Coring COMPILED BY AO
 DATUM Geodetic DATE 2022.11.29 - 2022.12.01 CHECKED BY KW

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE												
	Continued From Previous Page						20	40	60	80	100	20	40	60						
	Gravelly SILTY SAND (SC-SM) Contains cobbles and boulders Contains plastic fines Compact to very dense Grey GLACIAL TILL - 150 mm cobble at a depth of 20.3 m		5	NQ	-															
			6	NQ	-											19 34 47 (SI+CL)				
			7	NQ	-															
67.8	- 210 mm boulder at a depth of 23.0 m																			
23.8	DOLOSTONE BEDROCK Interbedded with Sandstone Contains quartz inclusions Fresh Fine grained Grey Medium bedded Very strong		1	RUN	-										FI >10	RUN #1 TCR=86% SCR=60% RQD=60%				
			2	RUN	-										5 1 4	RUN #2 TCR=100% SCR=70% RQD=70%				
			3	RUN	-										4 6 2 1	RUN #3 TCR=100% SCR=70% RQD=90% UCS=196MPa				
64.4																				
27.2	End of Borehole Flushmount standpipe piezometer consists of a 19 mm diameter Schedule 40 PVC pipe with a 1.5-m slotted screen. Water level readings: DATE DEPTH (m) ELEV. (m) 2022.12.01 7.8 83.8 2022.12.18 7.6 84.0 2023.04.26 7.6 84.0																			

RECORD OF BOREHOLE No 130-22-02

1 OF 2

METRIC

GWP# 4024-20-00 LOCATION Lat: 44.754557°, Long: -75.478812° Highway 16 & 401, Edwardsburgh, MTM z9: N 4 957 680.0 E 385 652.4 ORIGINATED BY IK
 HWY 401 BOREHOLE TYPE CME 55 Truck Mount / HSA / NW Casing / NQ Coring COMPILED BY AO
 DATUM Geodetic DATE 2022.11.28 - 2022.11.29 CHECKED BY KW

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa			WATER CONTENT (%)					
								○ UNCONFINED + FIELD VANE			— P — W — L					
								● QUICK TRIAXIAL × LAB VANE								
91.4	Ground Surface						20	40	60	80	100	20	40	60		
0.0	ASPHALT (150 mm)															
0.4	GRAVELLY SILTY SAND Brown FILL (BASE)															
	SAND, trace fines Compact to loose Brown FILL		1	SS	29											
			2	SS	13											
			3	SS	15											
			4	SS	15											
			5	SS	5											
86.5			6	SS	6											
4.9	SILTY SAND, some gravel Loose to dense Brown FILL		7	SS	32											
			8	SS	42											
84.2			9	SS	23											
7.2	Gravelly SILTY SAND (SM)															
83.9	Contains organics															
7.5	Dense Grey SILTY CLAY (CI) Very stiff Grey		10	SS	5											
82.7			11	SS	38											
8.7	Gravelly SILTY SAND (SC-SM) to Silty SANDY GRAVEL (GC-GM) Contains cobbles and boulders Contains plastic fines Dense to very dense Grey GLACIAL TILL		12	SS	100/ 150mm											

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

DOUBLE LINE 29381 BOREHOLE LOGS REPLACEMENT SITES.GPJ 2012TEMPLATE(MTO).GDT 10-30-23

METRIC

[illegible]

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 130-22-03

1 OF 2

METRIC

GWP# 4024-20-00 LOCATION Lat: 44.754129°, Long: -75.478803° Highway 16 & 401, Edwardsburgh, MTM z9: N 4 957 632.6 E 385 653.7 ORIGINATED BY IK
 HWY 401 BOREHOLE TYPE CME 55 Truck Mount / HSA / NW Casing / NQ Coring COMPILED BY AO
 DATUM Geodetic DATE 2022.12.09 - 2022.12.12 CHECKED BY KW

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
84.3	Ground Surface												
84.0	ASPHALT (125 mm)												
83.7	GRAVELLY SILTY SAND Brown FILL (BASE)												
83.4	SILTY SAND, some gravel Compact Brown FILL		1	SS	10								
82.8	SILTY CLAY (CI) Very stiff Brown		2	SS	21								
82.5	Silty SANDY GRAVEL (GC-GM) to Gravelly SILTY SAND (SC-SM) Contains cobbles and boulders Contains plastic fines Very dense to compact Brown to grey GLACIAL TILL		3	SS	67								
82.2			4	SS	46								
81.9			5	SS	46								
81.6			6	SS	55								
81.3			7	SS	23								
81.0			8	SS	100/ 100mm								
80.7			9	SS	100/ 75mm								
80.4			10	SS	100/ 150mm								
80.1			11	SS	100/ 100mm								

DOUBLE LINE 29381 BOREHOLE LOGS REPLACEMENT SITES.GPJ 2012TEMPLATE(MTO).GDT 10-30-23

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

2 OF 2

GWP#	4024-20-00	LOCATION	Lat: 44.754129°, Long: -75.478803° Highway 16 & 401, Edwardsburgh, MTM z9: N 4 957 632.6 E 385 653.7	ORIGINATED BY	IK
HWY	401	BOREHOLE TYPE	CME 55 Truck Mount / HSA / NW Casing / NQ Coring	COMPILED BY	AO
DATUM	Geodetic	DATE	2022.12.09 - 2022.12.12	CHECKED BY	KW

[illegible]

+³, ×³: Numbers refer to Sensitivity

BOREHOLE LOG

Checked By E.M.P.

ABBREVIATIONS

W. T. GROUND WATER TABLE IN SOIL

R. C. ROCK CORE

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
			0'0"					
ORGANIC TOPSOIL, ROOTS	BLACK		275.5'		1	FROM CASING		SATURATED WATER SEEPAGE AT 0'8"
FINE SAND	BROWN		277.84 2'0"					
MOTTLED FRIABLE CLAY	BROWN & GREY	FIRM	276" 273.5'		2	S.S.	7	W.T. MARCH 4, 1950. WETTER THAN PLASTIC LIMIT
AS ABOVE	AS ABOVE	STIFF			3	S.S.	10	M.C. = 31.1% WETTER THAN PLASTIC LIMIT.
			9'6" 266.0'					
FRIABLE CLAY WITH GRITS & PEBBLES	DK. GREY	VERY STIFF	12'0" 263.5'		4	S.S.	18	M.C. = 20.0% W.T.P.L. AT 12' REFUSAL ON BOULDER
SANDY TILL WITH BOULDERS	GREY					W.S.		
AS ABOVE	AS ABOVE		19' 1/2" 256.47			W.S.		"AX" ROD! 50 BLOWS FOR 1/2 IN.
			HOLE TERMINATED					NOTE: USING WASH WATER FROM G'

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



SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

BOREHOLE LOG

Bridge # 4 Hwy. # 401 - Hwy.
Job Name # 16. Re-alignment, Johnstown Job No. 5919
Client Dep't. of Highways of Ontario Casing BX (2 1/2" Dia.)
Datum Geodetic. Compiled By K.P.

Borehole No. 2
Boring Date March 2nd. & 3rd. 1959.
Checked By E.M.P.

SAMPLE CONDITION

-  UNDISTURBED
-  FAIR
-  DISTURBED
-  LOST

SAMPLE TYPE

- S.S. 2" STANDARD SPLIT TUBE SAMPLE
- S.L. SPLIT BARREL WITH LINERS
- S.T. THIN-WALLED SHELBY TUBE SAMPLE
- W.S. WASH SAMPLE
- R.C. ROCK CORE

ABBREVIATIONS

- V.T. IN SITU VANE SHEAR TEST
- Q/u UNCONFINED COMPRESSIVE STRENGTH
- W.L. WATER LEVEL IN CASING
- W.T. GROUND WATER TABLE IN SOIL





SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft	WATER LEVELS, SOIL MOISTURE & REMARKS
GROUND SURFACE			0' 0"					
SILTY SAND LOAM TOP SOIL VERY SILTY FINE SAND PEBBLES, STONE FRAGMENTS	DK. BROWN BROWN	8' 6" 273'95	274'45 273'20	1' 3"	0' 8"	275'78		W.L. MARCH 3, 1959. SATURATED
SILTY SAND, PEBBLES STONE FRAGMENTS	BROWN	LOOSE	3' 0"	1	S.S.	6		M.C.=10.6% SATURATED
SILTY FINE SAND WITH STONE FRAGMENTS GRITS, PEBBLES	BROWN	VERY DENSE	5' 0"	2	S.S.	93		M.C.=8.7% SATURATED
SANDY TILL BOULDERS!	DK. GREY	VERY DENSE	9' 0"	3	S.S.	112/6"		L.L.=14.5%, P.L.=12.5% M.C.=8.8% SATURATED REFUSAL ON BOULDER AT 10' 6"
AS ABOVE, STONE FRAGMENTS	AS ABOVE		15' 3"		S.S.	108		M.C.=7.0%
HOLE TERMINATED								NOTE: USING WASH WATER FROM 8'.

e. m. peto associates ltd.
SOIL ENGINEERING SERVICE - TORONTO, ONTARIO
BOREHOLE LOG

Bridge # 4 Hwy. # 401 - Hwy. # 16
Job Name Re-alignment, Johnstown. Job No. 5919
Client Dep't. of Highways of Ontario Casing 4" Pipe
Datum Geodetic. Compiled By K.A.P.

Borehole No. 3
Boring Date Feb. 28th. & March 2nd. 1959
Checked By E.M.A.P.

SAMPLE CONDITION






















-  UNDISTURBED
-  FAIR
-  DISTURBED
-  LOST

SAMPLE TYPE

- S.S. 2" STANDARD SPLIT TUBE SAMPLE
- S.L. SPLIT BARREL WITH LINERS
- S.T. THIN-WALLED SHELBY TUBE SAMPLE
- W.S. WASH SAMPLE
- P.C. ROCK CORE

ABBREVIATIONS

- V.T. IN SITU VANE SHEAR TEST
- Q/u UNCONFINED COMPRESSIVE STRENGTH
- W.L. WATER LEVEL IN CASING
- W.T. GROUND WATER TABLE IN SOIL

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft	WATER LEVELS, SOIL MOISTURE & REMARKS
GROUND SURFACE			0' 0"					
SILTY SAND LOAM TOP SOIL ROOTS, ORG. MATTER, PEBBLES	BLACK	110%	0' 8" 273' 10"		1		FROM CASING	W.T. SATURATED MARCH 2, 1959
FINE SAND, FEW ROOTS	BROWN	272' 10" 272' 43"	2' 0"		2		FROM CASING	SATURATED
MOTTLED FRIABLE CLAY WITH THIN SEAMS OF FINE SAND,	BROWN & GREY	FIRM TO STIFF	271' 10"		3		S.S.	8 M.C. = 35.1% WETTER THAN PLASTIC LIMIT
GRITS, PEBBLES & STONE FRAGMENTS			5' 0"		4		S.L. TAPPED	M.C. = 41.0% L.L. = 56.8% P.L. = 27.8%
AS ABOVE, MORE SAND	AS ABOVE	DENSE			5		S.S.	32 $\gamma_w = 114.2 \text{ lb/cuft}$ $C = 112.1 \text{ lb/s.f.}$ SATURATED
GRITS, PEBBLES & STONE FRAGMENTS			10' 0" 263' 10"					
					6		W.S.	
SANDY TILL, PEBBLES, STONE FRAGMENTS	DK. GREY	DENSE	15' 0"		7		S.S.	32 M.C. = 6.9%
					8		W.S.	
AS ABOVE	AS ABOVE	VERY DENSE			9		S.S.	120/6" M.C. = 6.0%
			20' 0"					NOTE: "AX" ROD; 100 BLOWS FOR 1/2 IN.
			22' 9"					NOTE: USING WASH WATER FROM 7' 6"
			250' 35"					REFUSAL ON BOULDER

BOREHOLE LOG

Borehole No. 4
Boring Date Feb. 27th. & 28th. 1959.
Checked By E. M. P.

ABBREVIATIONS

V.T. IN SITU VANE SHEAR TEST
Q/u UNCONFINED COMPRESSIVE STRENGTH
W.L. WATER LEVEL IN CASING
W.T. GROUND WATER TABLE IN SOIL

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft	WATER LEVELS, SOIL MOISTURE & REMARKS
GROUND SURFACE			0'0"					FEBRUARY 27, 1959.
TOPSOIL SILTY SAND	BLACK BROWN	1'0" 273'63	0'7 1/2" 273'98		0'6" 274'13	W.T.		ROOTS, ORG. MATTER -- SAT. WATER SEEPAGE AT 0'6"
MOTTLED FRIABLE CLAY WITH THIN SEAMS OF VERY FINE SAND	GREY & BROWN	FIRM TO STIFF			1		8	M.C.=37.2% WETTER THAN PLASTIC LIMIT
SAME AS ABOVE	AS ABOVE				2	S.L. TAPPED		M.C.=30.4% W.T.P.L. $\gamma_w = 122.9 \text{ lb./cu ft}$ $e = .922$ $C = 1311 \text{ lb.S.F.}$
			9'8"					
SANDY TILL, PEBBLES, STONE FRAGMENTS AS ABOVE	DK. GREY AS ABOVE	COMPACT	264'96		3	S.S. W.S.	18	L.L.=14.1%, P.L.=12.1% SATURATED
BOULDERS !								
AS ABOVE	AS ABOVE					W.S.		
			21'9"					
			252'88					NOTE: USING WASH WATER FROM 11'9"
						HOLE TERMINATED		

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

BOREHOLE LOG





Bridge # 4 Hwy. # 401 - Hwy. # 16
 Job Name Re-alignment Johnstown. Job No. 5919
 Client Dep't. of Highways of Ontario. Casing BX (2 1/2" Dia.)
 Datum Geodetic. Compiled By K.P.

Borehole No. 5.
 Boring Date March 5th. & 6th. 1959.
 Checked By E.M.P.

SAMPLE CONDITION











SAMPLE TYPE

ABBREVIATIONS

 UNDISTURBED
 FAIR
 DISTURBED
 LOST

S.S. 2" STANDARD SPLIT TUBE SAMPLE
 S.L. SPLIT BARREL WITH LINERS
 S.T. THIN-WALLED SHELBY TUBE SAMPLE
 W.S. WASH SAMPLE
 R.C. ROCK CORE

V.T. IN SITU VANE SHEAR TEST
 Q/u UNCONFINED COMPRESSIVE STRENGTH
 W.L. WATER LEVEL IN CASING
 W.T. GROUND WATER TABLE IN SOIL

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
GROUND SURFACE			0'0"					
SILTY SAND ORGANIC TOPSOIL	BLACK		1'0" 274.58					
SILTY SAND	BROWN		273.58 1'7"					
FRIABLE CLAY	BROWN & GREY	FIRM TO STIFF	272.95		1	S.S.	8	M.C. = 39.3% WETTER THAN PLASTIC LIMIT
AS ABOVE	AS ABOVE	STIFF			2	S.S.	9	M.C. = 38.5% W.T. P.L.
AS ABOVE	AS ABOVE				3	S.L.	TAPPED	
			9'5" 265.16					
FRIABLE CLAY	DK. GREY	VERY STIFF	11'3" 263.33		4	S.S.	18	M.C. = 18.1% ABOUT P.L.
SANDY TILL	DK. GREY				5	W.S.		
			15'0"					
AS ABOVE	AS ABOVE	VERY DENSE			6	S.S.	130	M.C. = 7.6%
			17'4" 257.25		7	S.S.	100/4"	NOTE: USING WASH WATER FROM 11'.
	"AX" ROD CANNOT BE DRIVEN.							



Appendix C.

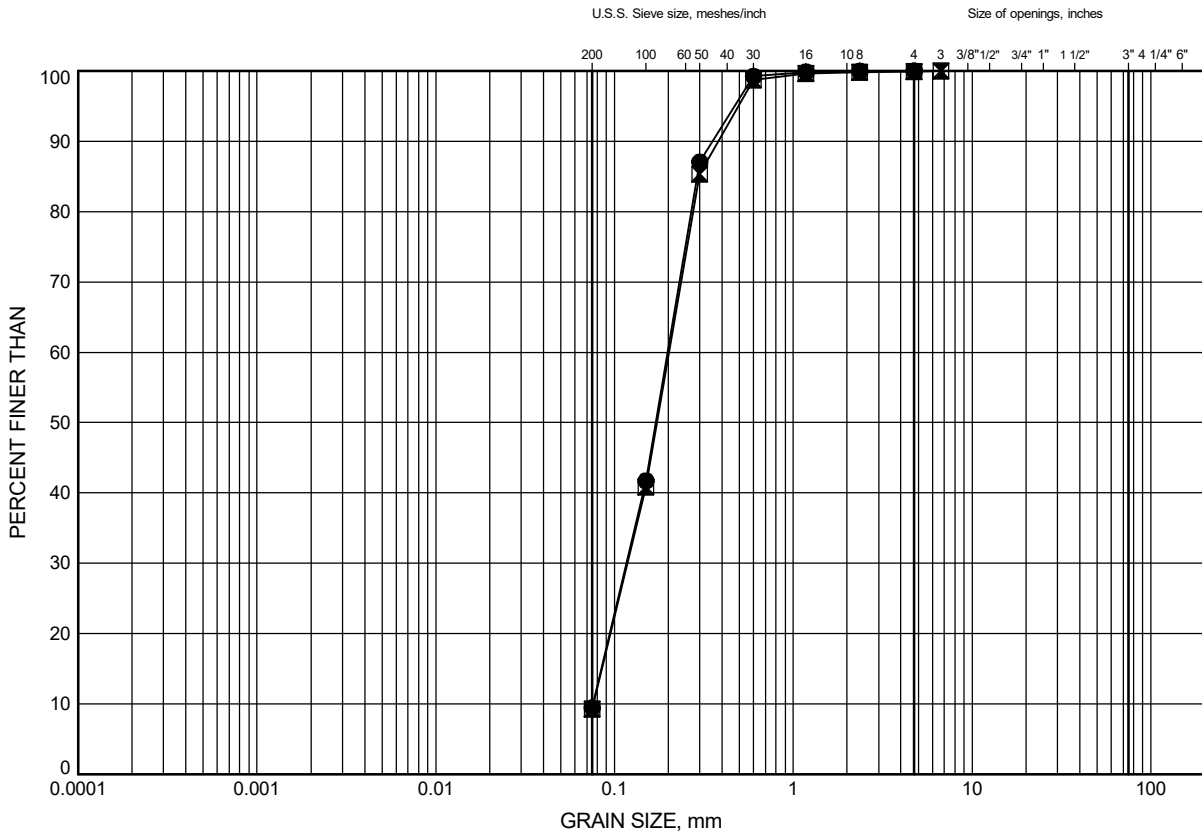
Laboratory Testing



Appendix C.1
Particle Size Analysis Figures
Atterberg Limit Test Results

GRAIN SIZE DISTRIBUTION

FILL: Sand



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	130-22-01	2.6	89.0
⊠	130-22-02	3.4	88.0

Date March 2023

GWP# 4024-20-00

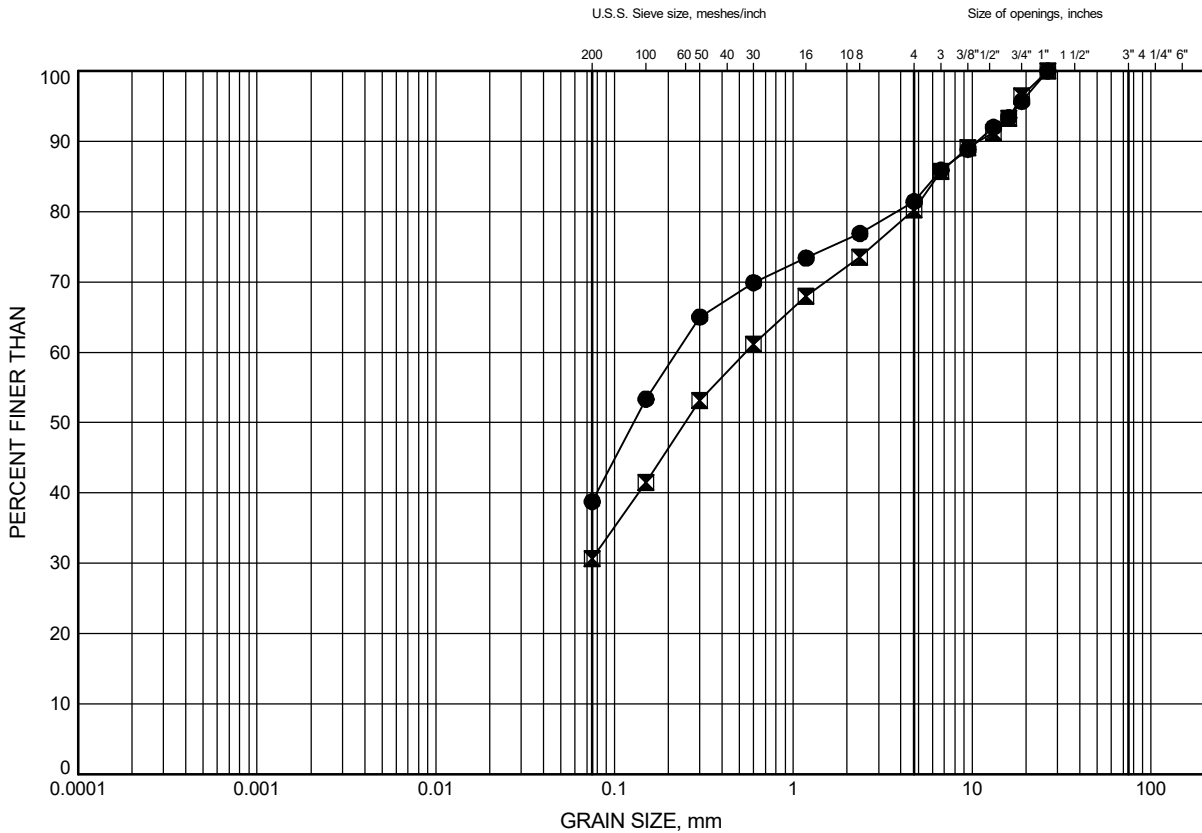


Prep'd RH

Chkd. AO

GRAIN SIZE DISTRIBUTION

FILL: Silty Sand



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	130-22-01	7.2	84.4
⊠	130-22-02	5.6	85.8

Date March 2023

GWP# 4024-20-00

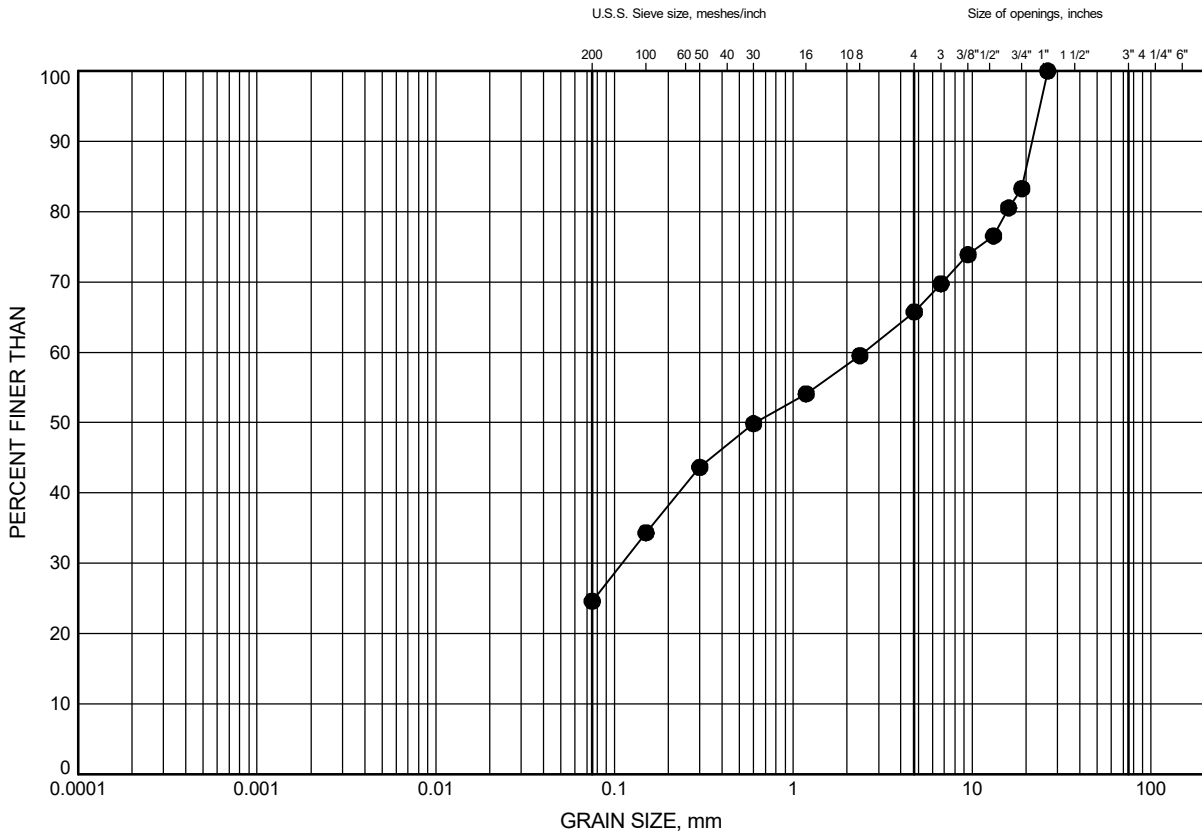


Prep'd RH

Chkd. AO

GRAIN SIZE DISTRIBUTION

Gravelly Silty Sand (SM)



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	130-22-02	7.3	84.1

Date March 2023

GWP# 4024-20-00



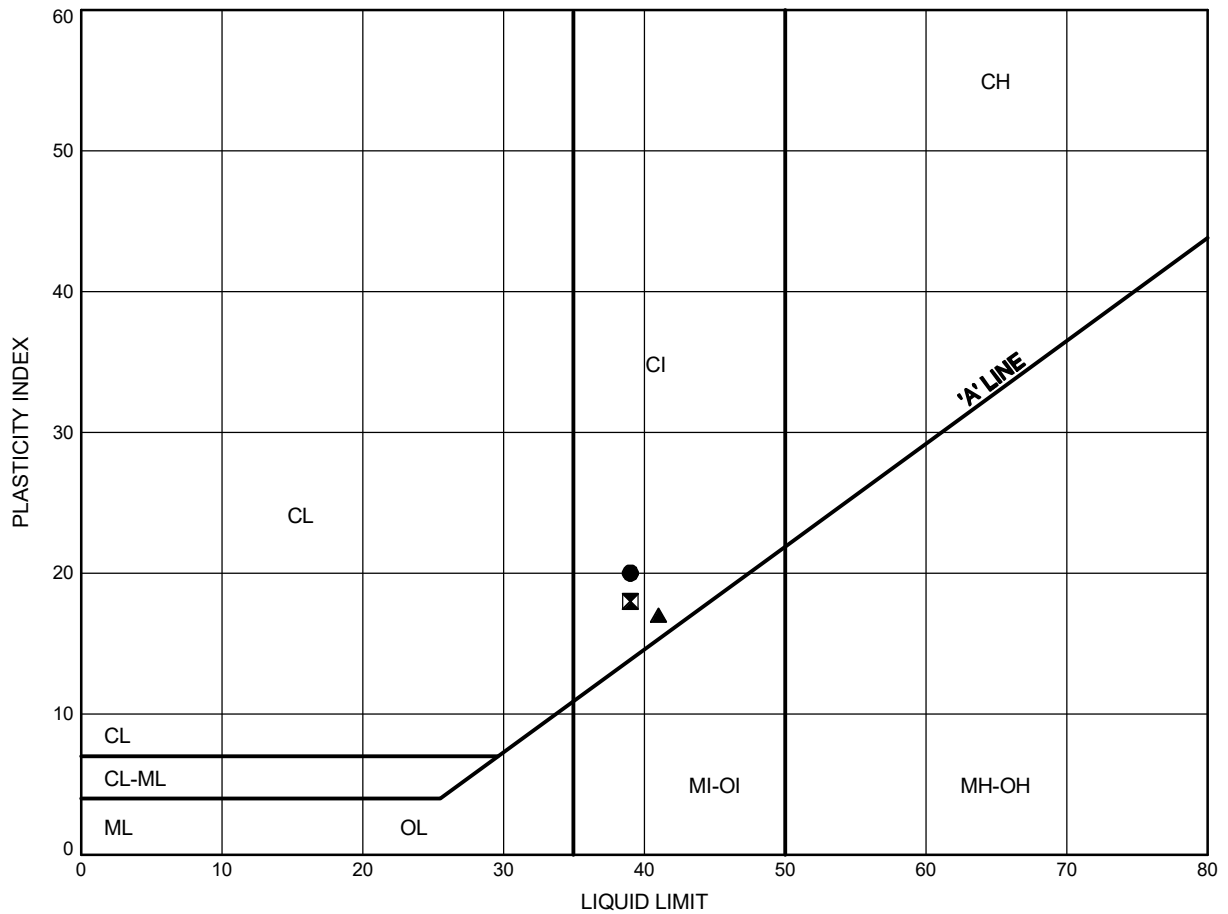
Prep'd RH

Chkd. AO

Highway 401 Underpass at Highway 16 (Site No. 16X-0130)
ATTERBERG LIMITS TEST RESULTS

FIGURE C4

Silty Clay (CI)



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	130-22-01	11.0	80.6
⊠	130-22-02	7.9	83.5
▲	130-22-03	1.1	83.2

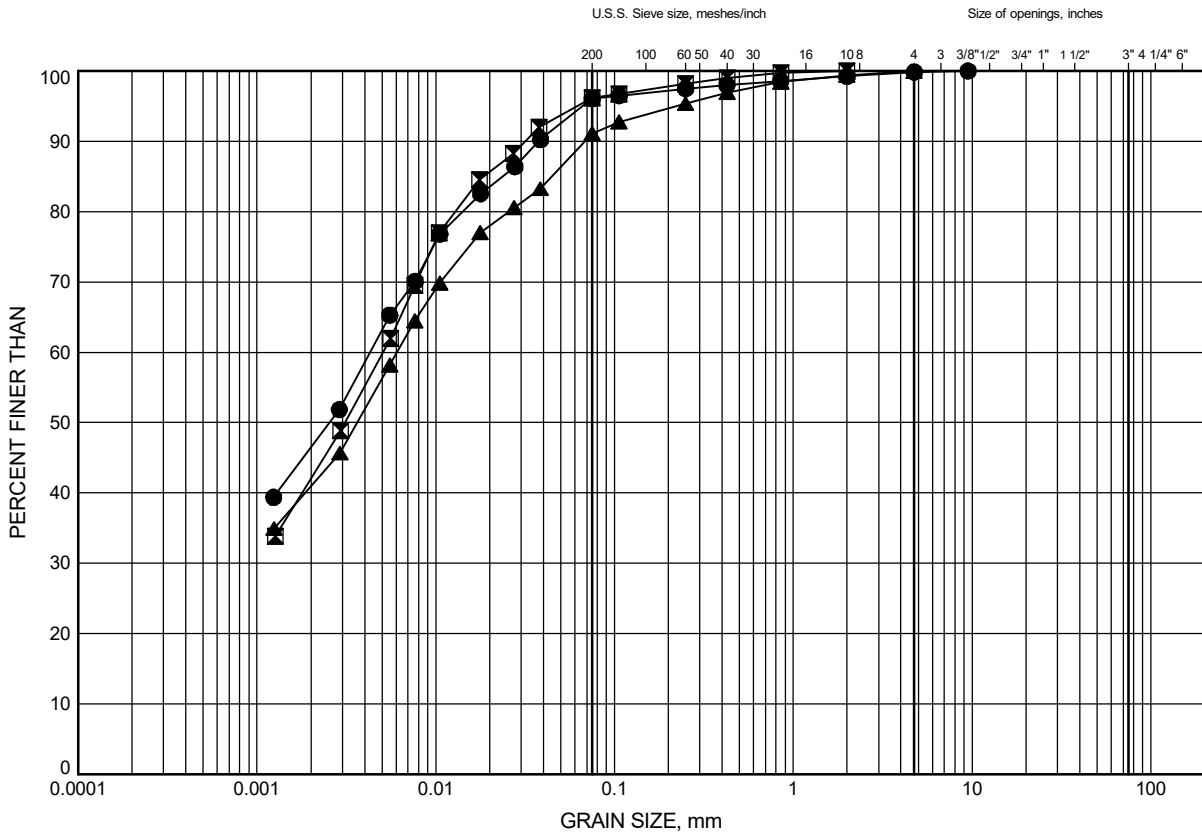
Date March 2023
 GWP# 4024-20-00



Prep'd RH
 Chkd. AO

GRAIN SIZE DISTRIBUTION

Silty Clay (CI)



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	130-22-01	11.0	80.6
⊠	130-22-02	7.9	83.5
▲	130-22-03	1.1	83.2

Date March 2023

GWP# 4024-20-00

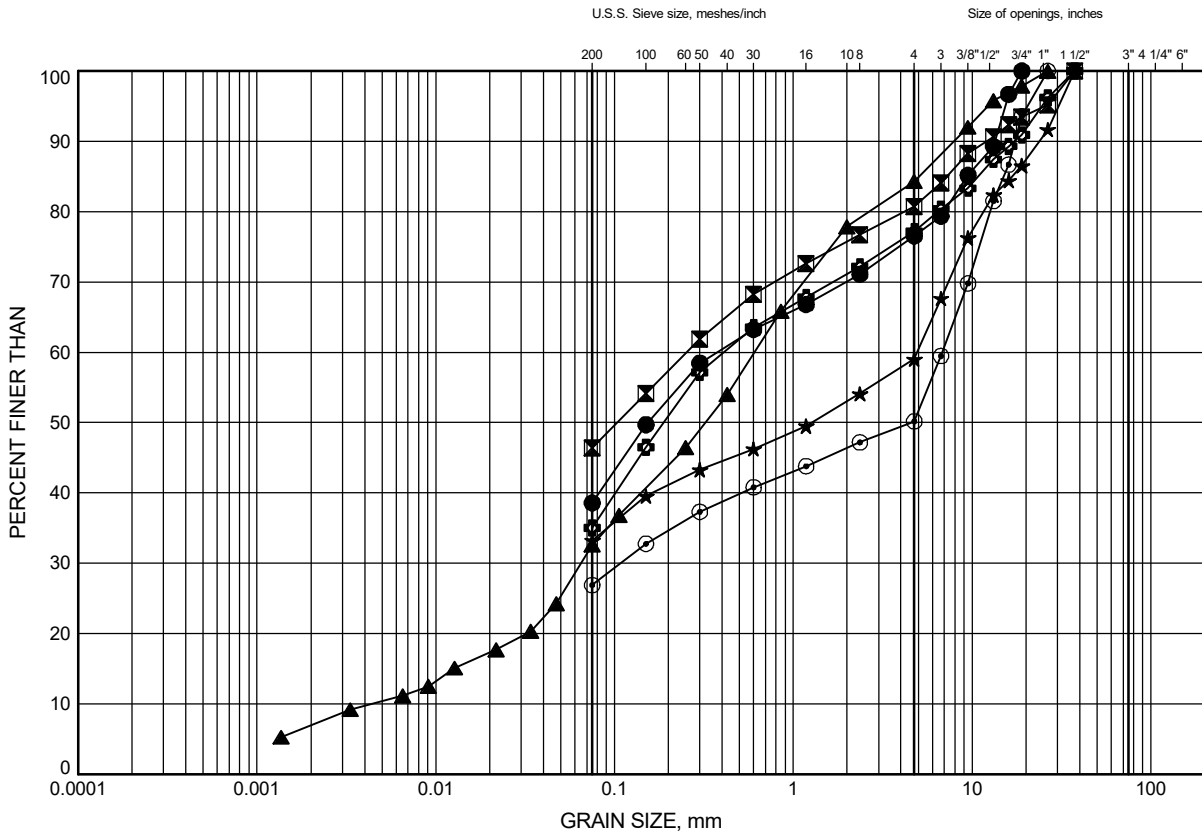


Prep'd RH

Chkd. AO

GRAIN SIZE DISTRIBUTION

GLACIAL TILL: Gravelly Silty Sand (SC-SM) to Silty Sandy Gravel (GC-GM)



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	130-22-01	14.0	77.6
⊠	130-22-01	21.6	70.0
▲	130-22-02	11.0	80.4
★	130-22-02	14.3	77.1
⊙	130-22-03	3.4	80.9
⊕	130-22-03	4.9	79.4

Date .. March 2023

GWP# .. 4024-20-00



Prep'd .. RH

Chkd. AO



Appendix C.2

UCS Test Results



Stantec Consulting Ltd.
2781 Lancaster Rd, Suite 100 A&B, Ottawa ON K1B 1A7

May 2, 2023
File: 122410864

Client: Thurber Engineering, File #29381

Reference: ASTM D7012, Method C, Unconfined Compressive Strength of Intact Rock Core

The following table summarizes unconfined compressive strength results for five intact rock cores.

Location	Sample Depth	Compressive Strength (MPa)	Description of Break
BH129-22-1 Run-1	88'7"-89'3"	212.5	Well-formed cones at both ends
BH129-22-2 Run-1	85'9"-86'3"	224.3	Well-formed cones at both ends.
BH130-22-1 Run-3	86'6"-87'4"	196.3	Vertical cracking throughout, no cones formed
BH131-22-2 Run-2	77'1"-77'10"	237.2	Vertical cracking throughout, no cones formed
BH166-22-2 Run-3	33'8"-34'2"	192.6	Well-formed cones at both ends

Sincerely,

Stantec Consulting Ltd.

Brian Prevost
Laboratory Supervisor
Tel: 613-738-6075
Fax: 613-722-2799
brian.prevost@stantec.com



Appendix C.3

Bedrock Core Photographs

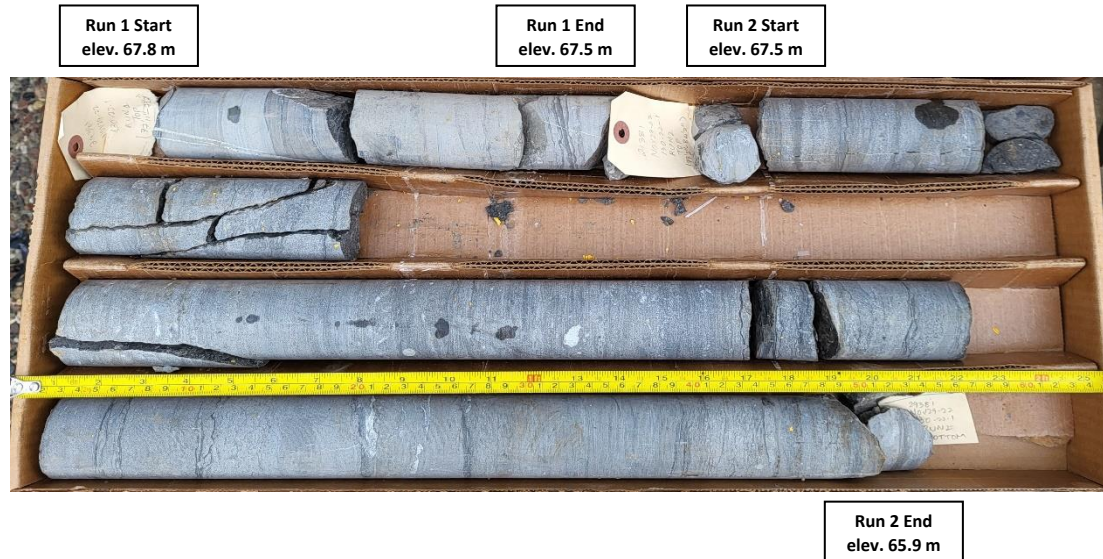
Borehole 130-22-01

RUNS 1 and 2

Depth 23.8 m to 25.7 m

Elevation 67.8 m to 65.9 m

Dry Sample



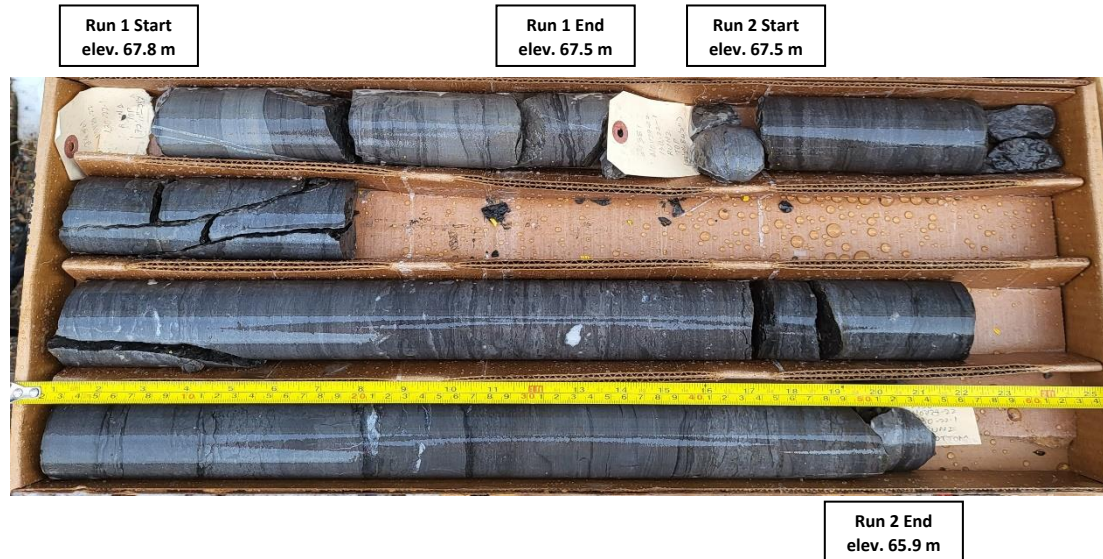
Borehole 130-22-01

RUNS 1 and 2

Depth 23.8 m to 25.7 m

Elevation 67.8 m to 65.9 m

Wet Sample



THURBER ENGINEERING LTD.

Geotechnical Investigation
Highway 401 Underpass at Highway 16
Edwardsburgh Township

BH 130-22-01
Project No.: 29381

Borehole 130-22-01

RUN 3

Depth 25.7 m to 27.2 m

Elevation 65.9 m to 64.4 m

Dry Sample

Run 3 Start
elev. 65.9 m



Run 3 End
elev. 64.4 m

Borehole 130-22-01

RUN 3

Depth 25.7 m to 27.2 m

Elevation 65.9 m to 64.4 m

Wet Sample

Run 3 Start
elev. 65.9 m



Run 3 End
elev. 64.4 m

Borehole 130-22-02

RUNS 1 and 2

Depth 14.8 m to 16.8 m

Elevation 76.6 m to 74.6 m

Dry Sample

Run 1 Start
elev. 76.6 m

Run 1 End
elev. 76.1 m



Run 2 Start
elev. 76.1 m



Run 2 End
elev. 74.6 m

Borehole 130-22-02

RUNS 1 and 2

Depth 14.8 m to 16.8 m

Elevation 76.6 m to 74.6 m

Wet Sample

Run 1 Start
elev. 76.6 m

Run 1 End
elev. 76.1 m



Run 2 Start
elev. 76.1 m



Run 2 End
elev. 74.6 m

Borehole 130-22-02

RUN 3

Depth 16.8 m to 18.3 m

Elevation 74.6 m to 73.1 m

Dry Sample

Run 3 Start
elev. 74.6 m



Run 3 End
elev. 73.1 m

Borehole 130-22-02

RUN 3

Depth 16.8 m to 18.3 m

Elevation 74.6 m to 73.1 m

Wet Sample

Run 3 Start
elev. 74.6 m



Run 3 End
elev. 73.1 m



THURBER ENGINEERING LTD.

Geotechnical Investigation
Highway 401 Underpass at Highway 16
Edwardsburgh Township

BH 130-22-02
Project No.: 29381

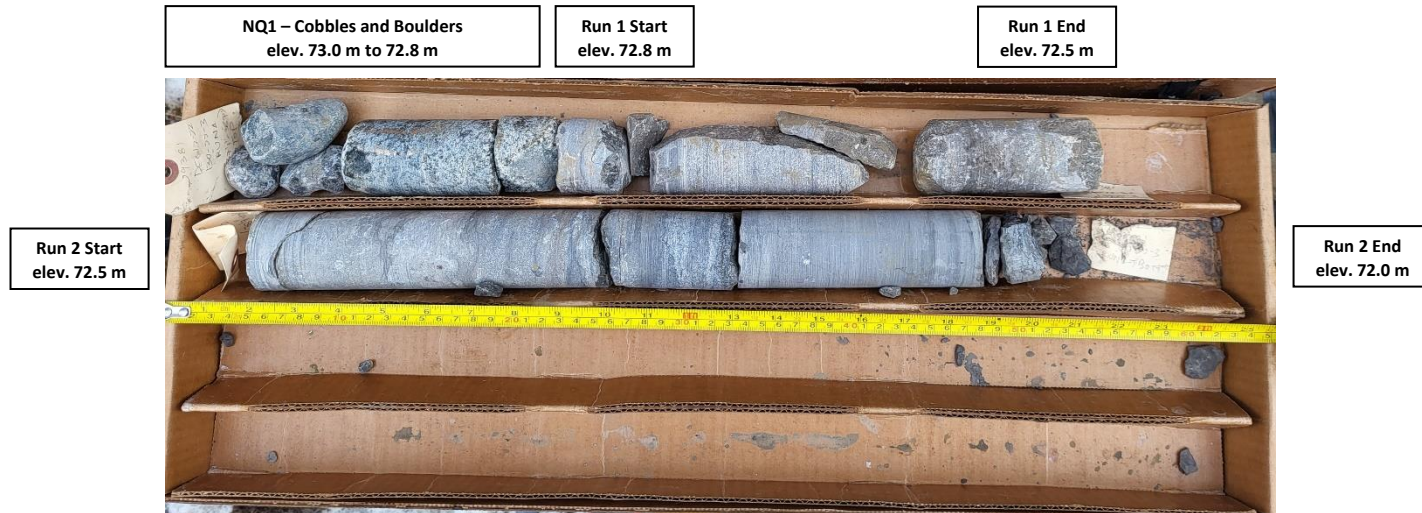
Borehole 130-22-03

RUNS 1 and 2

Depth 11.5 m to 12.3 m

Elevation 72.8 m to 72.0 m

Dry Sample



Borehole 130-22-03

RUNS 1 and 2

Depth 11.5 m to 12.3 m

Elevation 72.8 m to 72.0 m

Wet Sample

NQ1 – Cobbles and Boulders
elev. 73.0 m to 72.8 m

Run 1 Start
elev. 72.8 m

Run 1 End
elev. 72.5 m

Run 2 Start
elev. 72.5 m

Run 2 End
elev. 72.0 m



Borehole 130-22-03

RUN 3

Depth 12.3 m to 13.7 m

Elevation 72.0 m to 70.6 m

Dry Sample

Run 3 Start
elev. 72.0 m



Run 3 End
elev. 70.6 m

Borehole 130-22-03

RUN 3

Depth 12.3 m to 13.7 m

Elevation 72.0 m to 70.6 m

Wet Sample

Run 3 Start
elev. 72.0 m



Run 3 End
elev. 70.6 m

Borehole 130-22-03

RUN 4

Depth 13.7 m to 14.7 m

Elevation 70.6 m to 69.6 m

Dry Sample

Run 4 Start
elev. 70.6 m



Run 4 End
elev. 69.6 m

Borehole 130-22-03

RUN 4

Depth 13.7 m to 14.7 m
Elevation 70.6 m to 69.6 m
Wet Sample

Run 4 Start
elev. 70.6 m



Run 4 End
elev. 69.6 m



Appendix D.

Site Photographs



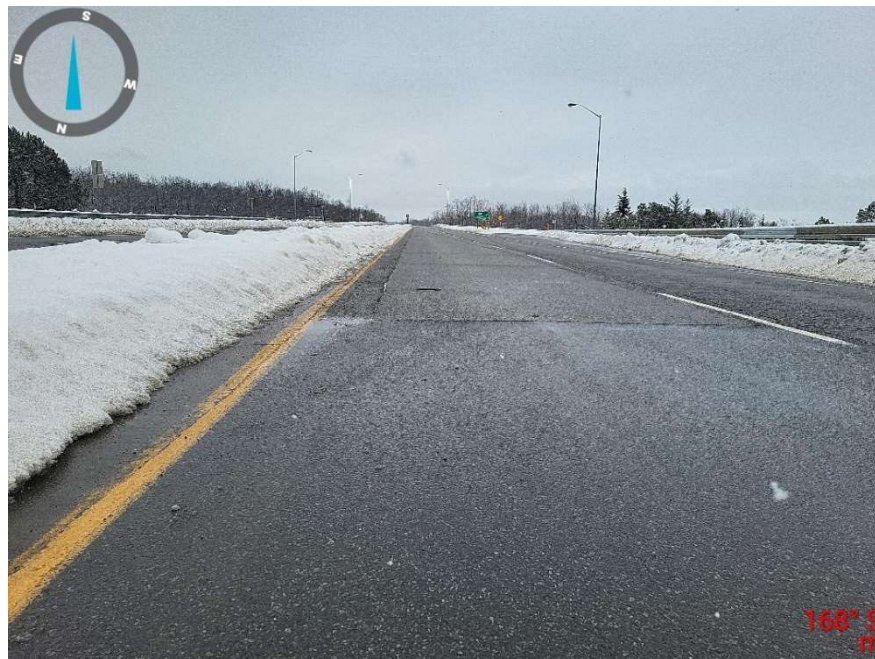
Photograph 1: Looking northwest at the southbound embankment
[taken on December 12, 2022]



Photograph 2: Looking southwest at the southbound embankment
[taken on December 12, 2022]



Photograph 4: Looking north at the southbound embankment
[taken on December 19, 2022]



Photograph 5: Looking south of from Highway 16 Underpass
[taken on December 19, 2022]



Photograph 6: Looking north of from Highway 16 Underpass
[taken on December 19, 2022]



Photograph 6: Looking east at the Highway 16 Underpass from the Highway 401
[taken on December 09, 2022]



Appendix E.

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

Site: 44.754N 75.479W

User File Reference: Highway 401 Highway 16 Underpass

2023-02-02 15:14 UT

Requested by: Thurber Engineering Ltd.

Probability of exceedance per annum	0.000404	0.001	0.0021	0.01
Probability of exceedance in 50 years	2 %	5 %	10 %	40 %
Sa (0.05)	0.389	0.218	0.130	0.037
Sa (0.1)	0.459	0.268	0.166	0.052
Sa (0.2)	0.386	0.229	0.145	0.048
Sa (0.3)	0.295	0.177	0.113	0.039
Sa (0.5)	0.210	0.126	0.081	0.029
Sa (1.0)	0.106	0.064	0.042	0.014
Sa (2.0)	0.051	0.031	0.019	0.006
Sa (5.0)	0.014	0.008	0.005	0.001
Sa (10.0)	0.005	0.003	0.002	0.001
PGA (g)	0.248	0.146	0.091	0.028
PGV (m/s)	0.174	0.101	0.062	0.019

Notes: Spectral ($S_a(T)$, where T is the period in seconds) and peak ground acceleration (PGA) values are given in units of g (9.81 m/s^2). Peak ground velocity is given in m/s . Values are for "firm ground" (NBCC2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are highlighted in yellow. 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.**

References

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

Structural Commentaries (User's Guide - NBC 2015: Part 4 of Division B)
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



Appendix F.

General Plan Drawing (1959)



Appendix G.

Comparison of Foundation Alternatives







COMPARISON OF FOUNDATION ALTERNATIVES

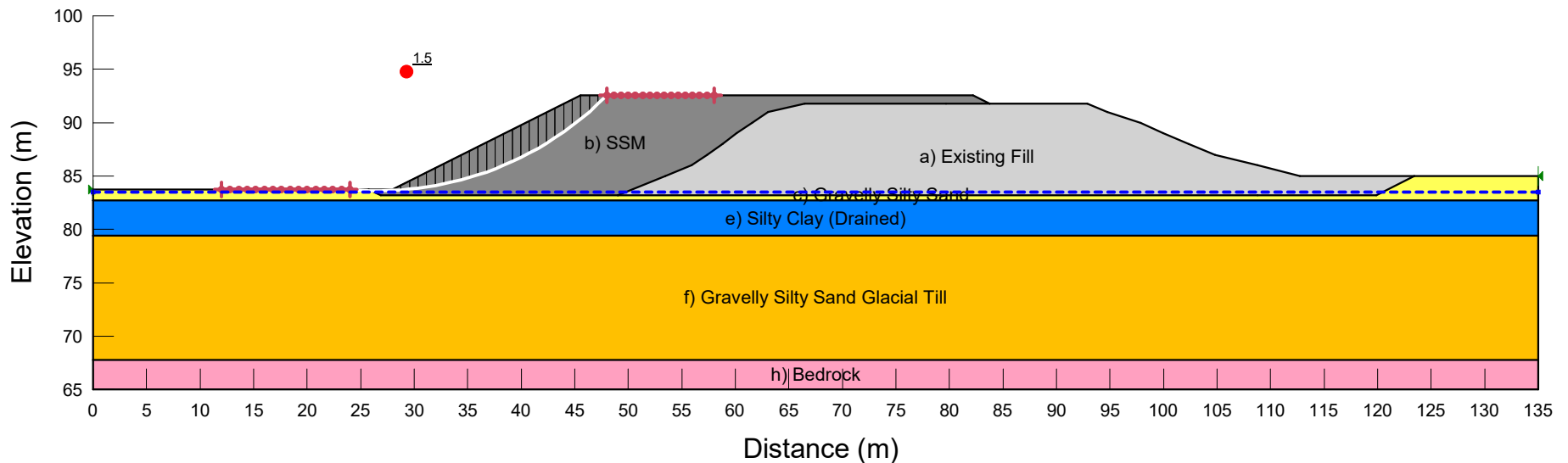
Spread Footings	Driven Steel H-Piles	Concrete Caissons	Drilled in Pipe Piles
Description			
Foundation element founded on traditional shallow spread footings on glacial till.	The abutments would be supported by a single row of steel H-piles driven to refusal on bedrock.	A reinforced concrete column installed within an augered hole in the ground that derives axial resistance from end bearing	Steel casing is advanced using a down-the-hole hammer with the cuttings/tailings flushed back to surface inside the drill string. Steel casing keeps hole open during drilling and prevents adjacent soil from collapsing into the hole. The casing is left in place as the bearing structure and would be filled with concrete (no rebar)
Advantages			
<ul style="list-style-type: none"> • Lower construction costs 	<ul style="list-style-type: none"> • Steel H-piles are well suited for use in integral abutment design. • Requires less excavation than spread footings. • Requires less concrete than caissons. • Alleviate constraints related to limited construction space in Highway 401 median. 	<ul style="list-style-type: none"> • Moderate to high axial geotechnical resistance. • Can handle obstructions. • Suitable for semi-integral abutment design approach. • Requires less excavation than spread footings. • Alleviate constraints related to limited construction space in Highway 401 median. 	<ul style="list-style-type: none"> • Moderate geotechnical resistance due to end-bearing only capacity in bedrock • Drilling system is well suited for penetrating through most obstructions, like cobbles and boulders encountered in the till • High drilling production rates • Better lateral resistance than H-Piles due to infilled concrete
Disadvantages			
<ul style="list-style-type: none"> • Significant excavations required, particularly at abutments beneath portions of existing embankments, adjacent to existing structure. • Excavations adjacent to existing structure and in the existing Highway 401 median may require temporary protection systems. 	<ul style="list-style-type: none"> • Has potential to encounter obstructions in the fill and glacial till. • Lower geotechnical resistance than caissons. 	<ul style="list-style-type: none"> • Not compatible with integral abutment design approach. • Likely requires concrete to be placed using tremie techniques. • Temporary steel casing required to keep hole open during drilling. • The base of the caisson would need to be inspected to ensure end bearing capacity. 	<ul style="list-style-type: none"> • Smaller number of contractors with suitable equipment • Not commonly used for integral abutments
Risks			
<ul style="list-style-type: none"> • Risk of instability of existing embankment slopes without appropriate temporary protection measures during excavations at abutments. 	<ul style="list-style-type: none"> • Difficult advancing through obstructions; may get “hung-up” in glacial till. 	<ul style="list-style-type: none"> • Difficulty penetrating through obstructions such as concrete and wood can cause construction delays. • Increased concrete volume may be required if additional soil is pulled in from sidewall while advancing through obstructions. • Position and alignment could be affected by obstructions. 	<ul style="list-style-type: none"> • Lateral response of pile difficult to predict since some equipment uses a drill head with greater diameter than pile and the width of this annular space may vary
Recommendation			
Recommended for pier	Recommended for abutments	Feasible, but not recommended	Feasible for Abutment



Appendix H.

Slope Stability Analysis Figures







Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	a) Existing Fill	Mohr-Coulomb	20	0	32
	b) SSM	Mohr-Coulomb	20	0	32
	c) Gravelly Silty Sand	Mohr-Coulomb	19	0	32
	e) Silty Clay (Drained)	Mohr-Coulomb	17.5	7	29
	f) Gravelly Silty Sand Glacial Till	Mohr-Coulomb	19	0	35
	h) Bedrock	Bedrock (Impenetrable)			

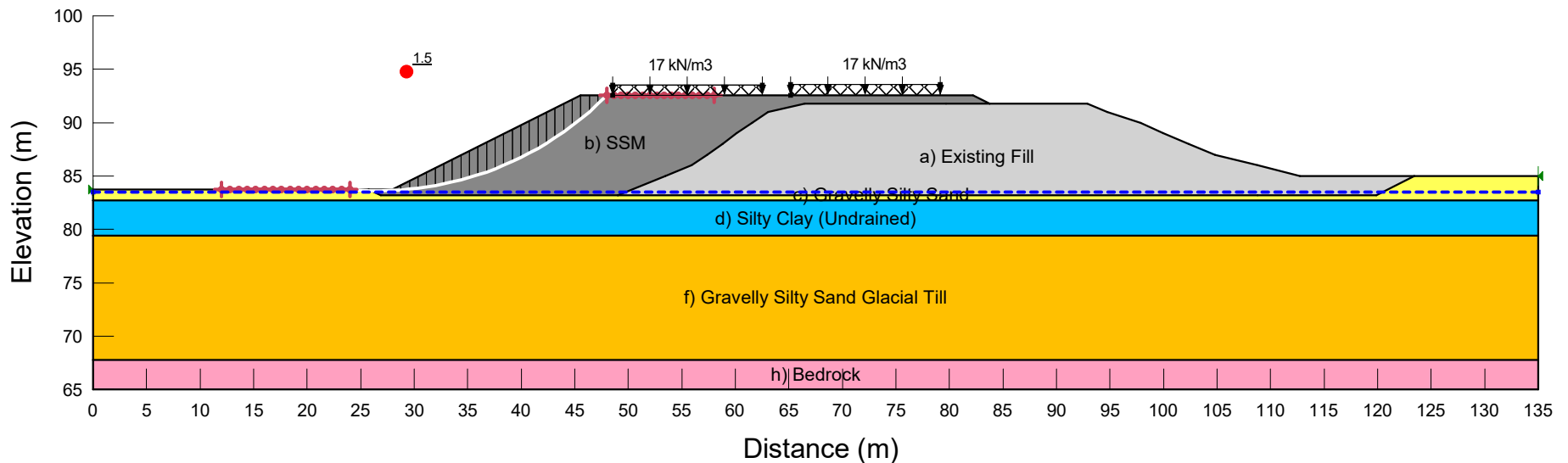


Project		
Hwy 401 Underpass at Hwy 36		
Analysis		
2.1) Permanent - Long Term		
Seismic Coefficient	Last Run	Scale
H: g, V: g	2023/07/10, 01:06:18 PM	1:600

Additional Details
Name: 2. South Embankment - SSM Widening
Comments:
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
Entry: (48, 92.55) m, Exit: (24, 83.75) m
Center: (26.648283, 113.65468) m, Radius: 30.021717 m

Figure H1

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Total Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	a) Existing Fill	Mohr-Coulomb	20		0	32
	b) SSM	Mohr-Coulomb	20		0	32
	c) Gravelly Silty Sand	Mohr-Coulomb	19		0	32
	d) Silty Clay (Undrained)	Undrained (Phi=0)	17.5	100		
	f) Gravelly Silty Sand Glacial Till	Mohr-Coulomb	19		0	35
	h) Bedrock	Bedrock (Impenetrable)				



Project
Hwy 401 Underpass at Hwy 16

Analysis
2.2) Temporary (traffic)

Seismic Coefficient
H: g, V: g

Last Run
2023/07/10, 01:06:19 PM

Scale
1:600

Additional Details

Name: 2. South Embankment - SSM Widening

Comments:







Method: Morgenstern-Price, Half-Sine

Minimum Slip Surface Depth: 1.52 m

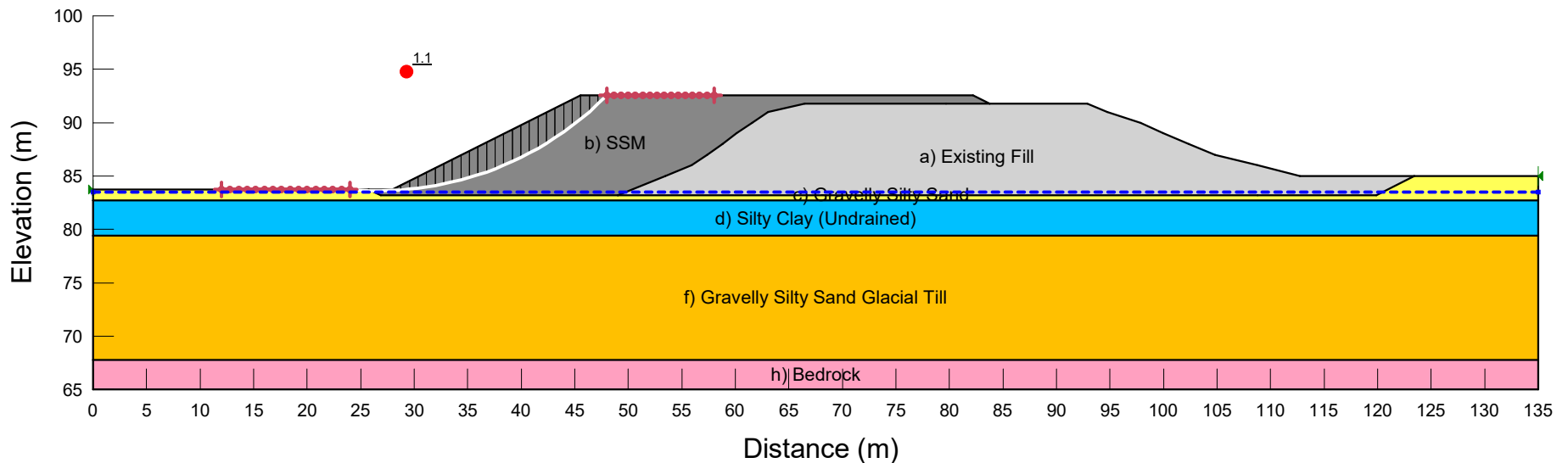
Entry: (48, 92.55) m, Exit: (24, 83.75) m

Center: (26.648283, 113.65468) m, Radius: 30.021717 m

Figure H2

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Total Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	a) Existing Fill	Mohr-Coulomb	20		0	32
	b) SSM	Mohr-Coulomb	20		0	32
	c) Gravelly Silty Sand	Mohr-Coulomb	19		0	32
	d) Silty Clay (Undrained)	Undrained (Phi=0)	17.5	100		
	f) Gravelly Silty Sand Glacial Till	Mohr-Coulomb	19		0	35
	h) Bedrock	Bedrock (Impenetrable)				

Horz Seismic Coef.: 0.13



Project
Hwy 401 Underpass at Hwy 16

Analysis
2.3) Pseud-Static (2,475 yr EQ)

Seismic Coefficient
H: 0.13g, V: g

Last Run
2023/07/10, 01:06:20 PM

Scale
1:600

Additional Details

Name: 2. South Embankment - SSM Widening

Comments:







Method: Morgenstern-Price, Half-Sine

Minimum Slip Surface Depth: 1.52 m

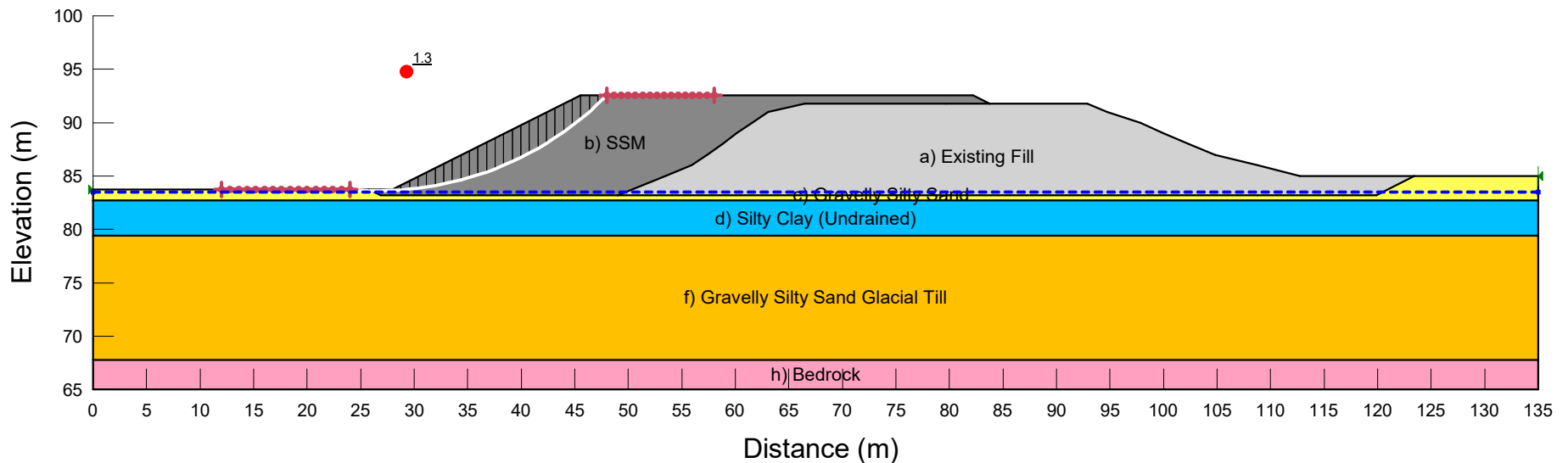
Entry: (48, 92.55) m, Exit: (24, 83.75) m

Center: (26.648283, 113.65468) m, Radius: 30.021717 m

Figure H3

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Total Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	a) Existing Fill	Mohr-Coulomb	20		0	32
	b) SSM	Mohr-Coulomb	20		0	32
	c) Gravelly Silty Sand	Mohr-Coulomb	19		0	32
	d) Silty Clay (Undrained)	Undrained (Phi=0)	17.5	100		
	f) Gravelly Silty Sand Glacial Till	Mohr-Coulomb	19		0	35
	h) Bedrock	Bedrock (Impenetrable)				

Horz Seismic Coef.: 0.059



Project
Hwy 401 Underpass at Hwy 16

Analysis
2.4) Pseudo-Static (475 yr EQ)

Seismic Coefficient

H: 0.059g, V: g

Last Run

2023/07/10, 01:06:20 PM

Scale

1:600

Additional Details

Name: 2. South Embankment - SSM Widening

Comments:

Method: Morgenstern-Price, Half-Sine

Minimum Slip Surface Depth: 1.52 m

Entry: (48, 92.55) m, Exit: (24, 83.75) m

Center: (26.648283, 113.65468) m, Radius: 30.021717 m

Figure H4



Appendix I.

List of Referenced Specifications



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

OPSD 208.010	Benching of Earth Slopes
OPSD 3090.101	Foundation Frost Depths for Southern Ontario
OPSD 3101.150	Walls Abutment, Backfill Minimum Granular Requirement
OPSS.PROV 1010	Material Specification for Aggregates Base, Subbase, Select Subgrade, and Backfill Material
OPSS.PROV 1860	Material Specification for Geotextiles
OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 903	Construction Specification for Deep Foundations

2. Contract Provision – Vibration and Settlement Limits for Local Infrastructure

The Contractor is advised that a CNR railway line runs below the Highway 16 Overpass. CNR must be contacted prior to any construction activities to determine vibration limits and maximum allowable particle velocity and acceptable settlement limits during construction activities.