



THURBER ENGINEERING LTD.

**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 401 WIDENING, HIGHWAY 16 TO MAITLAND ROAD
CPR OVERHEAD, SITE NO. 16X-0129
GWP 4024-20-00, ASSIGNMENT NO.: 4019-E-0010.2**

SITE NO. 16X-0129/B0

Geocres No.: 31B-113

Report to:

MTO c/o AECOM Canada Ltd.

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TABLE OF CONTENTS

PART 1. FACTUAL INFORMATION

1	INTRODUCTION.....	1
2	BACKGROUND AND SITE DESCRIPTION	1
2.1	General.....	1
2.2	Site Geology.....	2
2.3	Geocres Report 31B00-013	2
3	SITE INVESTIGATIONS AND FIELD TESTING.....	3
4	LABORATORY TESTING	4
5	GENERAL DESCRIPTION OF SUBSURFACE CONDITIONS	4
5.1	Pavement Structure.....	5
5.2	Sand Fill	5
5.3	Sand (SP-SM).....	5
5.4	Silty Clay (CI) to Clayey Silt (CL)	6
5.5	Sandy Gravel (GM) Glacial Till.....	7
5.6	Bedrock	7
5.7	Groundwater.....	8
6	MISCELLANEOUS	9

PART 2. ENGINEERING DISCUSSION AND RECOMMENDATIONS

7	INTRODUCTION.....	10
7.1	Existing Structure	10
7.2	Proposed Structure	11
7.3	Applicable Codes and Design Considerations	11
8	SEISMIC CONSIDERATIONS	12
8.1	Spectral and Peak Acceleration Hazard Values.....	12
8.2	CHBDC Seismic Site Classification.....	12
8.3	Seismic Performance Category.....	13
8.4	Liquefaction Potential	13



9	STRUCTURE FOUNDATION ALTERNATIVES	14
9.1	Foundation Alternatives.....	14
9.2	Recommended Foundation	15
9.3	Construction Methodology.....	15
10	FOUNDATION DESIGN RECOMMENDATIONS	15
10.1	Driven Steel H-Pile Foundations	16
10.1.1	Axial Geotechnical Resistance	16
10.1.2	Downdrag	17
10.1.3	Uplift Resistance.....	17
10.1.4	Lateral Resistance of Piles	17
10.2	Caisson Foundations.....	19
10.2.1	Axial Geotechnical Resistance	19
10.2.2	Downdrag	20
10.2.3	Lateral Resistance of Caissons	20
10.3	Retaining Wall Foundations	20
10.3.1	Shallow Foundations	20
10.3.2	RSS Walls.....	21
10.3.3	Subgrade Preparation, Bedding and Backfilling	22
10.4	Frost Depth.....	22
10.5	Backfill and Lateral Earth Pressures	22
10.5.1	Static Lateral Earth Pressure	23
10.5.2	Combined Static and Seismic Lateral Earth Pressure	24
10.6	Embankment Stability.....	25
10.7	Embankment Settlement	27
11	CONSTRUCTION CONSIDERATIONS.....	28
11.1	Temporary Excavations.....	28
11.2	Surface and Groundwater Control.....	29
12	RECOMMENDED SCOPE FOR DETAIL DESIGN.....	29
13	CLOSURE	31
	REFERENCES	32

STATEMENT OF LIMITATIONS AND CONDITIONS



Appendices

Appendix A.	Borehole Location Plan and Stratigraphic Drawings
Appendix B.	Record of Borehole Sheets
Appendix C.	Laboratory Testing
Appendix D.	Site Photographs
Appendix E.	GSC Seismic Hazard Calculation
Appendix F.	Original General Arrangement Drawing
Appendix G.	Comparison of Foundation Alternatives
Appendix H.	Slope Stability Analysis Figures
Appendix I.	List of Referenced Specifications



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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 401 overhead structure over a former Canadian Pacific Railway (CPR) line, now used as a recreational trail, located approximately 2.0 km east of the town of Prescott in the Township of Augusta 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. 31B00-013).

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 401 overhead structure crosses over a now-abandoned CPR line approximately 2.0 km east of the Edward Street Interchange and the community of Prescott, Ontario. For project orientation purposes, Highway 401 will be described as oriented east-west and the CPR line as oriented north-south.



At the CPR overhead, Highway 401 has two through lanes in each direction. The outside and median shoulders are fully paved. Concrete jersey barriers are present along the median and concrete barrier wall along the outside shoulders. In the area of the structure the Highway 401 median is approximately 9 m wide from edge of lane to edge of lane. Galvanized W-beam guiderails supported on metal posts extend beyond the concrete barrier walls up to about 1.1 km from the abutments. Concrete wingwalls are present at all four quadrants. The embankment side slopes are inclined at approximately 2.1H:1V and are vegetated with grasses and some shrubs. No signs of global instability of the embankment were noted during the field investigation.

The CPR overhead structure is skewed at approximately 30 degrees and has a span of approximately 11 m. The rails have been removed and the ground surface of the former rail line now has a granular surface. CAD drawings provided by AECOM indicate an unpaved path located in the southeast quadrant parallels the eastbound embankment toe. The drawings also indicate two centreline culverts located at approximately 22 m and 93 m from the west and east abutments, respectively.

The site is in a semi-rural setting, and the area directly adjacent to the CPR overhead is undeveloped land with a mix of cleared private properties and densely vegetated areas with deciduous trees and shrubs. A low-lying marsh dominated with grasses, sparse trees, and ponded water is found in the southwest and northwest quadrants of the structure. A tri-chord overhead sign is located in the eastbound lane approximately 68 m east of the structure. The terrain is relatively flat, apart from the existing highway embankments and associated ditches, which are relatively rugged.

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

2.2 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 a slightly undulating sand plain of glaciofluvial origin. The bedrock in both areas is generally limestone, dolostone, and calcareous sandstone.

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

2.3 Geocres Report 31B00-013

The historic foundation report for this site is based on a field investigation completed in 1959 prior to the construction of the existing overhead. The field investigation included a total of six boreholes. Relatively consistent conditions were observed across the site. A sand deposit with



some gravel was observed at ground surface and extended to depths ranging from 1.5 m to 3.3 m. It was underlain by a stiff clay with a thickness ranging from 9.5 m to 11.3 m. The clay was underlain by a thin layer of silty fine sand and a dense sandy silt glacial till deposit which was approximately 2.3 m thick where fully penetrated. Two boreholes cored bedrock which was observed at a depth of 15.5 m and 16.3 m below ground surface and was logged as limestone. A direct conversion of elevation suggests the surface of the bedrock is at 70.8 m.

Consolidation testing indicated that the clay deposit was over-consolidated to the point that addition of the proposed fills would not exceed the pre-consolidation pressure thus soil settlement would be limited to recompression.

Given the lack of identifiable features in the Borehole Location sketch, the information from Geocres Report 31B00-013 has been utilized herein only to establish general context.

3 SITE INVESTIGATIONS AND FIELD TESTING

A site investigation and field-testing program was carried out between December 13 and 15, 2022, and consisted of two on-road boreholes identified as 129-22-01 and 129-22-02 put down near the structure abutments. 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 available MTO benchmarks and existing site features and were cross-referenced with elevations on the original design drawings. 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)
129-22-01	WBL, west of West Abutment	4,955,585.5 (44.735948°)	383,515.3 (-75.506123°)	96.5	28.9
129-22-02	EBL, east of East Abutment	4,955,578.1 (44.735878°)	383,547.5 (-75.505718°)	96.6	28.3

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. Coring was required to advance the boreholes past the existing cobbles and boulders and into bedrock. A standpipe piezometer was installed in



Borehole 129-22-02 to allow for measurements of the groundwater level after drilling. The details of the standpipe piezometer are illustrated on the respective Record of Borehole sheet 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, Borehole 129-22-01 was decommissioned in general accordance MOE requirements (O.Reg. 903, as amended) and capped with cold patch asphalt to reinstate the pavement surface. The standpipe piezometer in Borehole 129-22-02 was decommissioned in accordance with MOE requirements on April 26, 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. Unconfined Compressive Strength (UCS) testing was conducted on two bedrock core samples from Borehole 129-22-02.

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 D2487. 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 sand underlain by silty clay to clayey silt over glacial till. The glacial till was composed of sandy gravel, cobbles, and boulders and was underlain by dolostone bedrock.

5.1 Pavement Structure

A layer of asphaltic concrete approximately 355 mm thick was encountered at both borehole locations. The asphalt was underlain by gravelly sand fill containing trace fines. This gravelly sand was layer was 0.2 m to 0.3 m thick (base elevation 96.0 m to 95.8 m).

The recorded moisture content of a sample of the gravelly sand fill was 2%. The results of a gradation analysis completed on the sample are illustrated on Figure C1 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	28
Sand	65
Silt	7
Clay	

5.2 Sand Fill

Sand fill was encountered beneath the gravelly sand in both boreholes. The sand fill was 9.2 m to 9.3 m thick (base elevation 86.7 m to 86.6 m). SPT N-values in the sand fill ranged from 3 to 64 blows per 0.3 m of penetration, indicating a variable relative density ranging from very loose to very dense.

The recorded moisture content of samples of the sand fill ranged from 4 to 20%. The results of gradation analyses completed on five 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. The results from the gradation analysis carried out on a sample from the lower portion of the east embankment (Borehole 129-22-02) contained some gravel.

Soil Particle	Percentage (%)
Gravel	0 – 10
Sand	78 – 93
Silt	7 – 12
Clay	

A 0.2 m thick layer of sandy silt containing organics was encountered below the sand fill in both boreholes. The recorded moisture content of samples of this layer ranged from 44 to 58%.

5.3 Sand (SP-SM)

A native deposit of sand containing trace fines was encountered below the sandy silty in the boreholes. The layer ranged in thickness from 0.6 m to 1.3 m (base elevation 85.8 m to 85.2 m). SPT N-values in the layer ranged from 8 to 16 blows per 0.3 m of penetration, indicating a loose to compact relative density.

The recorded moisture content of samples of the native sand ranged from 18 to 22%. 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 tests are summarized below and on the Record of Borehole sheet in Appendix B.

Soil Particle	Percentage (%)
Gravel	0
Sand	92
Silt	8
Clay	

5.4 Silty Clay (CI) to Clayey Silt (CL)

A deposit of silty clay to clayey silt was encountered beneath the sand in both boreholes. The thickness of the layer ranged from 11.8 m to 13.7 m (base elev. 73.4 m to 72.1 m). SPT N-values ranged from weight-of-hammer (WH) to 43 blows per 0.3 m of penetration but were generally less than 10 blows. Numerous 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, indicating a very stiff consistency.

Recorded moisture contents ranged from 16 to 41% and generally decreased with depth. Atterberg Limit testing was completed on four 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 to clayey silt generally ranged in plasticity from intermediate (CI) to low (CL) with depth.

Parameter	Value
Liquid Limit	25 – 45
Plastic Limit	16 – 23
Plasticity Index	7 – 22

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 – 3
Sand	0 – 2
Silt	41 – 68
Clay	28 – 59

5.5 Sandy Gravel (GM) Glacial Till

A native deposit of glacial till consisting of sandy gravel containing fines and frequent cobbles and boulders was encountered below the silty clay to clayey silt deposit in both boreholes. The glacial till deposit is 1.5 m to 2.0 m thick (base elev. 71.4 m to 70.6 m). Coring techniques were required to penetrate the layer.

The recorded moisture content of samples of the deposit recovered with a split-spoon sampler ranged from 1% to 8%. The results of a gradation analysis completed on a sample obtained from the upper portion of the deposit in Borehole 129-22-02 are illustrated on Figure C6 of Appendix C. The results of the tests are summarized above and on the Record of Borehole sheet in Appendix B.

Soil Particle	Percentage (%)
Gravel	47
Sand	30
Silt	23
Clay	

5.6 Bedrock

Bedrock was proven by coring in the boreholes. The depth to bedrock from the existing road grade ranged from 25.2 m to 25.9 m (base elevations 71.4 m to 70.6 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), %	100
Solid Core Recovery (SCR), %	71 – 93
Rock Quality Designation (RQD), %	63 – 93
Fracture Index (fractures per 0.3 m) ⁽¹⁾	0 – 7
Unconfined Compressive Strength (UCS), MPa	213 – 224

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

Based on the RQD, the bedrock quality is classified as fair to excellent (CFEM, 2006). The results of unconfined compressive strength testing were 213 MPa and 224 MPa, indicating that the bedrock is very strong (CFEM, 2006).



5.7 Groundwater

A 19 mm diameter standpipe piezometer was installed in Borehole 129-22-02 to allow for subsequent measurements of the groundwater level. The measured groundwater levels are summarized in Table 5-2. Surface water was present on portions of the surrounding lowland areas to the northeast and northwest of the site. Based on historic aerial photographs, surface water is present in these areas for much of the year. The groundwater level provided in the table below may represent a slightly artesian pressure in the silty clay at the site.

Table 5-2: Groundwater Level Observations

Borehole No.	Bottom of Screen Elev. (m)	Screened Unit	Depth (mbgs)¹	Groundwater Elevation (m)	Date of Measurement
129-22-02	76.3	Clayey Silt	8.1	88.5	April 26, 2023

Notes: (1) Indicates depth below top of embankment at Highway 401 grade.

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.

The field work was supervised on a full-time basis by I. Khan, E.I.T., under the direction of K. Walker, P.Eng. Routine geotechnical laboratory testing was completed by Thurber's laboratory in Ottawa, Ontario. Unconfined Compressive Strength Testing of the bedrock was carried out by Stantec's laboratory in Ottawa.

Overall project management and direction of the field investigation was provided by Matt Kennedy, P.Eng. 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 Highway 401 overhead structure at the former CPR line, now used as a recreational trail, located approximately 2.0 km east of the town of Prescott in the Township of Augusta within the Leeds and Grenville 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 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. Additional foundation investigation and design will be required during detailed design.

The following sections provide preliminary geotechnical recommendations for the design of the foundation elements 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.

7.1 Existing Structure

The existing structure is a one-span reinforced cast-in-place concrete structure that was constructed in 1961 and carries a total of four lanes of Highway 401 over the former CPR right-of-way. It has a span length of 12.8 m (along the Highway 401 alignment) and a width of 32.0 m with an approximate skew angle of 30 degrees from perpendicular to Highway 401. There are cast-in-place concrete wing walls located at all four quadrants of the bridge to retain the embankment slopes adjacent to Highway 401. For project orientation purposes, Highway 401 will be referred to herein as oriented east-west and the former CPR line oriented north-south.



The original General Arrangement drawing (Drawing TWP 27-129-01-A, see Appendix F) indicates that the structure is founded on spread footings approximately 2.7 m wide and 0.8 m thick, founded at Elevation 85.3 m. Structural rehabilitation of the bridge was last carried out in 2011 as part of Contract No. 2010-4008.

7.2 Proposed Structure

The preliminary design of the CPR overhead to accommodate the proposed widening of Highway 401 includes a replacement structure on the same alignment with no significant change in Highway 401 grades. The new structure is to be up to 16 m wider than the existing to accommodate the ultimate 8-lane configuration of Highway 401 (proposed total width of about 48 m). It is assumed that the embankment widening will be constructed with conventional side slopes without retaining walls.

The preferred span length of the new structure is anticipated to be influenced by the foundation type and evaluation of the construction methodology. Maintaining the relatively short existing span length with a rigid-framed bridge structure would require significant excavation (greater than 9 m) through the existing embankment fill to remove the existing structure and construct the new abutment walls.

Alternatively, the excavation depth requirements could be reduced by increasing the span length and supporting the abutments on piles constructed with pile caps “perched” within the embankments. This option could eliminate the requirement to fully remove the existing foundations provided that the new abutment foundations are constructed behind, and do not conflict with, the existing foundations. However, this would require a longer span and either inclined fore slopes in front of the abutments or reinforced earth false abutment walls to maintain the existing grade at the CPR crossing.

Consideration could be given to constructing a closed-bottom box culvert but its feasibility would depend on the future use and span/opening requirements, as a culvert structure would likely necessitate a reduction in span length. Replacement of the existing structure with a box culvert would require significant excavation (greater than 9 m) through the existing embankment fill. Depending on minimum span/opening requirements, it may also be feasible to construct a box culvert within the existing opening to minimize excavation requirements. Adequate frost protection would also be required along the base and walls of the box culvert to minimize potential frost impacts on the structure. However, since the existing span length would likely need to be significantly reduced, replacement with a closed box culvert is not discussed further in this report.

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). 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 0.24 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).

The boreholes encountered up to about 10 m of loose to dense granular embankment fill overlying up to about 1.3 m of native sand which is, in turn, underlain by up to about 13.7 m of very stiff silty clay to clayey silt and 2 m of very dense glacial till. The N_{60} values recorded in the embankment



fill and native sand were 3 to greater than 50 blows per 0.3 m of penetration. The undrained shear strength of the silty clay to clayey silt was measured to be greater than 102 kPa.

The existing overhead is supported on spread footings founded within the silty clay to clay deposit. Depending on the preferred foundation type and elevation, the replacement structure foundations may be within the native silty clay to clayey silt (spread footings) or perched within the embankment fill (driven steel H-piles).

Based on the average N_{60} values in the embankment fill and the measured undrained shear strength of the silty clay to clayey silt, a Seismic Site Class D may be considered for preliminary design for either scenario described above.

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 Seismic Performance Category 3, assuming the bridge has a fundamental period less than 0.5 seconds, as per Section 4.4.4 (Table 4.10) of the CHBDC. If the bridge structure has a fundamental period greater than or equal to 0.5 seconds, it would fall within Seismic Performance Category 2.

8.4 Liquefaction Potential

The susceptibility of the cohesive silty clay to clayey silt deposit at this site to experience liquefaction/cyclic softening during the design earthquake was assessed using the measured undrained shear strength with the Boulanger & Idriss (2007)ⁱⁱ method and is classified as not susceptible to cyclic mobility during a seismic event.

The susceptibility of the cohesionless soils 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)ⁱⁱⁱ. The results indicate that the deposit of native sand that overlies the silty clay to clayey silt is potentially liquefiable under the 2,475 and 975-year design earthquakes. One SPT at the base of the deposit in each of the boreholes indicated the potential for liquefaction.

As per Table 4.1 of the CHBDC, sites with liquefiable soils should be considered a Site Class F and a site-specific evaluation is required. It is recommended that additional foundation investigation be carried out to confirm the potential for liquefaction at the site with a more detailed liquefaction assessment considering the results of a site-specific ground response analysis, if required. There are three possible outcomes upon completion of that more rigorous work:

1. liquefaction is determined to have a low risk of occurrence and does not need to be considered in design,
2. liquefaction is assessed to be an issue and the structure and embankments are designed to accommodate the forces and displacements induced by liquefaction; or
3. liquefaction is assessed to be an issue and ground improvement techniques are employed to densify the soils to reduce the risk of liquefaction to acceptable levels.



The following sections of this report have been prepared based on the assumption that the first or third scenarios will prevail and that design of the structure and embankments will not be influenced by liquefaction. As described above in Section 8.2, it is recommended that the structure be designed considering a Site Class D until the site-specific evaluation has been carried out.

9 STRUCTURE FOUNDATION ALTERNATIVES

9.1 Foundation Alternatives

Based on the soil stratigraphy, details of the existing structure, and anticipated construction challenges and requirements, both shallow and deep foundations have been considered for the replacement of the Highway 401 overhead structure at the CPR crossing. 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

The existing structure is founded on spread footings bearing within the silty clay to clayey silt. Spread footings for the replacement structure could be designed for bearing within the native silty clay to clayey silt, but excavation through the existing Highway 401 embankments would be significant (greater than 11 m deep). Spread footings bearing on the silty clay would result in some differential settlement of the structure. Further, significant groundwater control would be required to maintain excavations below the native ground surface in the dry for footing construction.

Supporting the replacement bridge on spread footings is not recommended and, as such, is not discussed further in this report.

- Driven Steel H-piles

Steel H-piles driven through the existing approach embankments to support “perched” abutments may be considered for support of the structure. Driven H-piles will typically reduce the volumes of excavation required when compared to shallow foundations. However, the bridge span may need to be increased to accommodate “perched” pile caps while maintaining the CPR crossing envelope, as described above in Section 7.2. The use of H-Piles with reinforced tips is the option with the least risk given the cobbles and boulders encountered in the till layer above the bedrock observed at this site.

H-piles allow construction of integral abutments and are recommended for support of the replacement bridge.

- 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 (at or near the existing ground surface of the rail line) will pose additional construction challenges resulting from



potential unbalanced hydraulic pressure heads and caisson base boiling when drilling through the glacial till deposit. This would require the use of temporary liners or synthetic slurry to counterbalance groundwater pressure.

Caissons are considered feasible but are not the preferred option from a foundation design perspective.

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 driven steel H-piles, perched within the approach embankments.

9.3 Construction Methodology

To maintain usage and avoid full closure of Highway 401 during construction, removal of the existing bridge and construction of the replacement structure must be carried out in a staged manner. As such, the foundation recommendations provided herein have been prepared based on the assumption that traffic will be detoured on to a portion of the existing structure to facilitate removal and replacement of the remaining portion. Traffic would then be detoured on to the new, widened portion of the structure to allow the remaining structure removal and replacement. Staging areas to support the removal and replacement construction will need to be established in the areas surrounding the existing bridge and would be accessed from Highway 401.

The existing superstructure and foundation elements will require removal prior to site preparation for, and construction of, the replacement bridge (or staged portions thereof). Based on the original 1959 General Arrangement drawing (see Appendix F), the existing structure is founded on spread footings approximately 2.7 m wide and 0.8 m thick, founded at Elevation 85.3 m. Depending on the location and foundation type of the new abutments, consideration may be given to leaving the existing spread footings in place, provided that they do not conflict with and are outside the zone of influence of the new foundations.

10 FOUNDATION DESIGN RECOMMENDATIONS

Approximate key elevations are as follows:

- | | |
|--|-------------------|
| • Existing Highway 401 grade at structure | 96.5 m to 96.6 m |
| • Underside of perched abutment pile caps, assumed | as high as 94.0 m |
| • Groundwater elevation in silty clay (BH 129-22-02) | 88.5 m |
| • Existing native ground surface | 87.8 m to 88.0 m |
| • Underside of existing spread footings | 85.3 m |
| • Top of Glacial Till | 72.1 m to 73.4 m |
| • Top of Dolostone Bedrock | 70.6 m to 71.4 m |

10.1 Driven Steel H-Pile Foundations

10.1.1 Axial Geotechnical Resistance

The new abutments may be founded on steel H-piles end-bearing on the bedrock. As described above (Section 7.2), the elevation of the pile caps will be influenced by the design span length, to maintain the existing width and height of the CPR crossing. The estimated pile tip elevations based on piles reaching refusal at the bedrock surface are summarized in Table 10-1, below.

It is anticipated that piles will not consistently be able to penetrate the dense glacial till deposit that contains cobbles and boulders to reach bedrock. Accordingly, the pile recommendations provided below are based on driving the piles to refusal in the glacial till on cobbles and boulders; however, it should be noted that some of the piles may get past this layer and reach bedrock.

Table 10-1: Estimated Pile Tip Elevations

Foundation Element	Approximate Underside of Pile Cap Elevation ¹ (m)	Estimated Pile Tip Elevation (Glacial Till) (m)	Estimated Pile Length (m)
West Abutment	94.0	71.5	22.5
East Abutment	94.0	72.5	21.5

Notes: (1) Assumed, based on longer bridge span to minimize required abutment wall heights, maintaining Highway 401 grade, and frost protection requirements

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

The factored geotechnical axial resistances provided include the following factors:

- Consequence factor (Ψ) of 1.0
- Geotechnical resistance factors (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 resistances provided above are applicable for pile spacing greater than 3 pile widths. Driven piles must be installed in accordance with OPSS.PROV 903. The potential for conflict with the existing spread footings supporting the existing structure must be checked.

The pile tips 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 the existing bridge 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.

10.1.2 Downdrag

Downdrag forces (negative skin friction) acting upon driven steel H-piles are expected to be negligible as the anticipated settlement resulting from placement of additional embankment widening fill is expected to be less than 25 mm within the footprint of the new abutments (see Section 10.7).

10.1.3 Uplift Resistance

The native soil and the fill at the site through which the piles are driven will provide uplift resistance. The magnitude of uplift resistance provided by the embankment fill will be determined by the pile cap elevation.

For piles that meet effective refusal to driving in the glacial till deposit, the factored geotechnical tensile resistance for a single HP 310x110 pile under static and seismic conditions may be taken as summarized in Table 10-2. These values provided 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)

Table 10-2: Estimated Geotechnical Tensile (Uplift) Capacity (HP 310x110)

Underside of Pile Cap Elevation (m)	Factored Uplift Resistance	
	Static (kN)	Seismic (kN)
94.0 (perched)	430	1,430
86.6 (deep)	250	830

10.1.4 Lateral Resistance of Piles

The lateral resistance provided by the soils may be determined using P-y curve data, which 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 Embankment Fill	20 (above WT)	6,000	-	3.0
Sand	9 (below WT)	1,300	-	3.0
Silty Clay to Clayey Silt	7 (below WT)	-	100	-
Glacial Till	10 (below WT)	8,000	-	3.7

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



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.

10.2 Caisson Foundations

Support of the abutments 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 existing embankment fill and glacial till layers. A socket would then be drilled into the bedrock, cleaned, and the casing and socket would be filled with concrete in a single pour after installation of the 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.

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.

10.2.1 Axial Geotechnical Resistance

The axial geotechnical capacity at factored ULS for a steel casing filled with concrete and socketed a minimum of 2 caisson diameters into sound bedrock is provided in the table below. The caisson capacities include resistance factors (ϕ_{gu}) of 0.4 and 0.3 for compressive and tensile 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 10-4: Axial Geotechnical Resistance for Caissons

Caisson Diameter (mm)	Factored ULS (Compression) (kN)	Factored SLS (Compression) (kN)	Factored ULS (Tension)	
			Static (kN)	Seismic (kN)
915	11,000	will not govern	8,000	26,500
1220	19,000	will not govern	14,000	46,500

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 10.2.3), moment capacity and seismic analysis to satisfy the structural assessment.



The caissons socketed into the bedrock will also provide tensile (uplift) resistance. The axial geotechnical uplift resistance for a steel casing filled with concrete and socketed a minimum of 2 caisson diameters into sound bedrock is also provided in Table 10-4, above, and 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)

10.2.2 Downdrag

Downdrag forces (negative skin friction) acting upon caissons are expected to be negligible as the anticipated settlement resulting from placement of additional embankment widening fill is expected to be less than 25 mm within the footprint of the new abutments (see Section 10.7).

10.2.3 Lateral Resistance of Caissons

The resistance to lateral loading developed by the soil in front of the caissons may be estimated as outlined in Section 10.1.4.

10.3 Retaining Wall Foundations

It is understood that the preferred structure design includes integral abutments supported on deep foundations. To minimize the required span length for an integral abutment configuration with pile caps perched in the embankment fill, design of false abutment walls in front of the abutments may be considered. The false abutment walls would be uncoupled from the bridge foundations and superstructure and would retain the embankment soil behind the abutments.

10.3.1 Shallow Foundations

Conventional concrete shallow foundations for the false abutment walls can be founded directly on the native sand or silty clay to clayey silt with a minimum 500 mm thick Granular A bedding layer as discussed in Section 10.3.3. Shallow footings up to 3.0 m in width and constructed as outlined above may be designed based on the following factored geotechnical resistances:

- Factored geotechnical resistance at ULS 200 kPa
- Factored geotechnical resistance at SLS 125 kPa

The factored geotechnical resistance at SLS corresponds to total footing settlement of 25 mm, and would be in addition to the 20 mm to 25 mm of settlement predicated at the new slope crests, described in Section 10.7.

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):
 - $\phi_{gu} = 0.5$ (static analysis; typical degree of understanding)
 - $\phi_{gs} = 0.8$ (static analysis; typical degree of understanding)

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.

Resistance to lateral forces/sliding resistance between the concrete and the underlying Granular 'A' bedding (Section 10.3) should be evaluated in accordance with the CHBDC assuming an unfactored coefficient of friction of 0.45. A reduction factor of 0.8 (as per CHBDC Table 6.2) should be used to estimate the sliding resistance between the concrete and Granular A. An unfactored coefficient of friction of 0.35 can be assumed for the interface between the Granular 'A' and the native sand or silty clay. A reduction factor of 0.6 (as per CHBDC Table 6.2) should be used to estimate the sliding resistance between the Granular A and the sand or silty clay subgrade.

The 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.

10.3.2 RSS Walls

Retained soil systems (RSS) walls are considered feasible at this site. The design of proprietary RSS walls is the responsibility of the supplier. Typically, such systems do not require full frost protection as they are able to tolerate some movement due to frost heave. The RSS system should be designed in accordance with the MTO RSS Design Guidelines. Once the location and height of the wall is established, recommendations will be provided concerning Performance, Appearance and Acceptance.

RSS walls should have a minimum embedment of 0.8 m. The underside elevation of the RSS wall is anticipated to be at or below approximately 87.2 m. A minimum 1 m thick engineered fill pad constructed on the underlying undisturbed native soils should be provided below the RSS wall as well as under the reinforced retained soil. The engineered fill pads should consist of OPSS Granular A placed and compacted in accordance with OPSS.PROV 501. Engineered fill pads should be constructed with 1H:1V sides slopes with the crest of slope a minimum of 1 m from the edge of footing and reinforced retained soil on all sides. The 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 engineered fill pad placed immediately after excavation and inspection.

The lateral pressure comments provided Section 10.5 may be used in RSS design. Please also refer to Section 10.6 for comments on the global stability of RSS walls.



Preliminary design of RSS walls with a minimum embedment of 0.8 m and bearing on an engineered fill pad as described above may be designed based on the geotechnical resistances provided in Section 10.3.1.

It should be noted that these RSS walls are estimated to be up to about 8.5 m high. Before selecting the RSS wall, discussions should be held with RSS wall suppliers whether RSS walls are feasible based on the relatively low bearing capacities available in the native soils.

10.3.3 Subgrade Preparation, Bedding and Backfilling

Subgrade preparation for the retaining walls should include excavation and removal of the existing bridge foundations, and any associated fill or deleterious material. It is noted that the original general arrangement drawings indicate that the underside of the existing bridge footings are at about Elevation 85.3 m. All organics, soft or loose deposits, disturbed soils, and deleterious materials must be stripped from the footprint of the foundation to expose competent subgrade at or below the desired founding elevations. In areas requiring over excavation for removal of existing bridge foundations or other material, engineered backfill consisting of OPSS.PROV 1010 Granular A may be placed in lifts and compacted up to the required subgrade level.

The exposed final subgrade must be inspected to confirm that the subgrade is suitable and uniformly competent. Any soft or organic materials at the subgrade level should be sub-excavated and backfilled with granular fill consisting of material as soon as practical to protect the subgrade from disturbance during construction. The granular fill should be compacted as per OPSS.PROV 501. The backfill must be in accordance with OPSS 902 and placed to the extents shown on OPSD 803.010.

It is noted that excavation for construction for false abutment walls is expected to extend below the groundwater elevation. Refer to Section 11.2 for additional comments on water control design and groundwater control systems.

10.4 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.

10.5 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 10-5 and Table 10-6 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 10-5 and Table 10-6 should be used. If other backfill and wall geometries are to be considered, Thurber will need to calculate the appropriate earth pressure coefficients.

10.5.1 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:

σ_h	=	horizontal pressure on the wall at depth h (kPa)
K	=	earth pressure coefficient (see Table 10-5) (K_a for yielding walls, K_o for non-yielding walls)
γ	=	unit weight of retained soil (see Table 10-5), 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 10-5.

Table 10-5 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.

10.5.2 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:

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

The coefficients of horizontal earth pressure for combined static and seismic loading presented in Table 10-6 may be used for a horizontal back-slope. The provided earth pressure coefficients are calculated using a site-adjusted PGA of 0.27 g, based on a Seismic Site Class D, a reference (Site Class C) PGA with a 2% probability of exceedance in 50 years of 0.24 g (Geological Survey of Canada – Fifth Generation) and a $F(PGA)$ of 1.12 as per Table 4.8 of the CHBDC.

Table 10-6 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}	0.45	0.50	0.54
Yielding Wall			
Dynamic Active Earth Pressure Coefficient, K_{AE}	0.35	0.39	0.42

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

10.6 Embankment Stability

Based on the available original structure drawings and observations during the field investigation, the grade of the current Highway 401 overhead at the CPR crossing ranges from about 96.5 m to 96.6 m, and the embankment height is about 9 m. The existing slopes are at roughly 2H:1V beyond the wing walls. The existing Highway 401 embankment grades are to be maintained and widened a total of up to about 16 m to accommodate the proposed ultimate 8-lane configuration.

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, widened portions of the 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.
- Retained soil at RSS walls are to consist of OPSS Granular B Type II, up to 6 m in width and supported on a 1 m thick Granular A bedding layer.
- Granular fill sloped embankments greater than 8 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.

In addition and as noted in Section 8.4, it has been assumed that the potential for liquefaction at this site is resolved during the detailed investigation and design stage.

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 target 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 9 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 site-adjusted (Site Class D) design PGA values of 0.27 g and 0.12 g for ground motions with return periods of 2,475 and 475 years, respectively, as per Section 4.4.3.2 of the CHBDC.

Slope stability assessments have been carried out for the southern slope of the east embankment and are considered to be representative of the other embankment slopes due to their symmetry and similar site condition. 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 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 borehole;
- Embankment widening of 16 m (8 m on each side);
- A traffic surcharge of 17 kPa was applied as a temporary load.

Table 10-7 Slope Stability Analysis Results

Condition	Case	Factor of Safety	
		SSM Slope	RSS Wall
Permanent (traffic loading)	Long-Term Static (Drained)	1.5 (Fig H1.1)	1.5 (Fig H1.1)
Temporary (traffic loading)	Short-Term Static (Undrained)	1.5 (Fig H1.2)	1.6 (Fig H1.2)
Temporary (seismic loading)	Pseudo-Static Seismic, 2,475-yr (Undrained)	1.1 (Fig H1.3)	1.3 (Fig H1.3)
	Pseudo-Static Seismic, 475-yr (Undrained)	1.3 (Fig H1.4)	n/a



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 factors of safety values for the critical embankment slope presented in Table 10-7, above.

All of the static results presented in Table 10-7 achieve the target Factors of Safety described above. The pseudo-static result considering the 2,475-year earthquake presented in Table 10-7 meets 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 SSM slope during the 1 in 2,475 year seismic event (Figure H1.3). 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 west abutment at that location (Figure H1.4) indicating that the performance requirements would be met for that scenario.

10.7 Embankment Settlement

At the structure, Highway 401 is to remain on the same alignment, with negligible change to the final grades. Additional fill will be placed to widen the existing approach embankments by up to about 16 m (8 m on each side).

The total settlement resulting from widening of the existing embankment with conventional granular fill by 8 m to each of the north and south was assessed using the multi-layer settlement analysis in Rocscience's Settle3 software. Subsurface stratigraphy was based on the boreholes drilled in the area and historical subsurface information (Geocres No. 31B00-013). Loading was applied to consider the geometry of the existing embankments, and the associated settlement that would have occurred since their construction in the 1960s, and the assumed geometry of the proposed widened embankments using a unit weight for new embankment SSM fill of 20 kN/m³. A conservative approach to settlement has been employed by assuming that the widening will be constructed with side slopes of 2H:1V without retaining walls. The water table was defined based on piezometer readings and observations at the site. It is noted that engineering judgment and experience was used to select the material properties based on the stress range anticipated due to loading. Soil parameters used in the analysis of the proposed new embankment are presented in Table 10-8, below.

Based on the parameters presented in Table 10-8, the total settlement that is expected to have occurred under the loading of the existing embankment (since embankment construction in the 1960s) is about 130 mm at the centreline of the embankments.

Following placement of additional fill on the existing side slopes to widen the embankments an additional 8 m on each side, a *total* of up to about 45 mm of settlement beneath the widened portions is anticipated. Based on the assumed geometry of the widening, the maximum settlement would occur beneath the new embankment side slopes (near the existing embankment

toes). The fill will undergo self settlement, however, given the geometry of the widening, this will contribute little to the settlement under the new lanes. The total settlement at the crest of the new embankment slopes (i.e. the outer edge of the new abutments) is expected to be 20 to 25 mm.

Table 10-8 Properties of Soil Used in Settlement Calculations

Property	Native Sand	Silty Clay to Clayey Silt	Glacial Till
Unit Weight [kN/m ³]	18.0	17.5	21.0
E [kPa]	10,000	-	110,000
e _o	-	0.9	-
P _c ', [kPa]	-	335	-
C _c	-	0.3	-
C _r	-	0.03	-

It is noted that the anticipated loading will not exceed the preconsolidation pressure for the clay soils, thus the settlement is generated by recompression. Although two consolidation tests were included in the investigation described in Geocres Report 31B00-013, the rate of settlement information was not provided. Typically, recompression settlement occurs fairly quickly, thus it is anticipated that 90% of the settlement will occur within six months of embankment widening. This should be confirmed with consolidation testing during the detailed investigation and design stage.

Based on the results presented above, the widened 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 summarized 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

11 CONSTRUCTION CONSIDERATIONS

11.1 Temporary Excavations

Excavations at the abutments will extend into the existing embankment and highway grade fill. Depending on requirements for removal of the existing structure and footings, excavations into the native sand and silty clay beneath the groundwater table may be required. All temporary excavations must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of OHSA, existing embankment and highway fill above the groundwater table may be considered 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. Existing fill



beneath the groundwater table, native sand, and silty clay may be considered Type 4 soil. Unsupported excavations made in Type 4 soils must have side slopes no steeper than 3H:1V from the base of the excavation.

At locations where there are space restrictions, where a slope must be retained, and/or to facilitate staged construction, 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.

11.2 Surface and Groundwater Control

At the site, the natural groundwater level was measured to be within the base of the approach embankments (Elevation 88.5 m in the silty clay), and above ground surface in the low land areas surrounding the site (approximate Elevation 87.5 m). Surface runoff will also tend to seep into and accumulate into the 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 fill must be carried out in a dry and stable excavation.

Based on the anticipated depths of excavation required for construction of pile caps perched within the existing embankments, for temporary excavations it is considered likely that groundwater may be controlled with sump pumps in the bottom of the excavations. For excavations below the groundwater or surface water level at the site, design of groundwater control systems will be required, and should be considered in detail at subsequent stages of design.

12 RECOMMENDED SCOPE FOR DETAIL DESIGN

The recommendations provided above are in support of the preliminary design of the proposed replacement of the Highway 401 & CPR overpass (Site No. 16X-0129/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.
- Additional foundation investigation to clarify the liquefaction potential of the native sand at the site, and site-specific ground response analysis as input to the liquefaction assessment to provide a more detailed evaluation of the liquefaction potential at the proposed foundation locations, as required.
- 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.



- A hydrogeological assessment of the site soils to support development of dewatering recommendations, based on the excavations requirements of the TPA.
- Consolidation testing of the silty clay to clayey silt 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 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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ⁱⁱⁱ 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.

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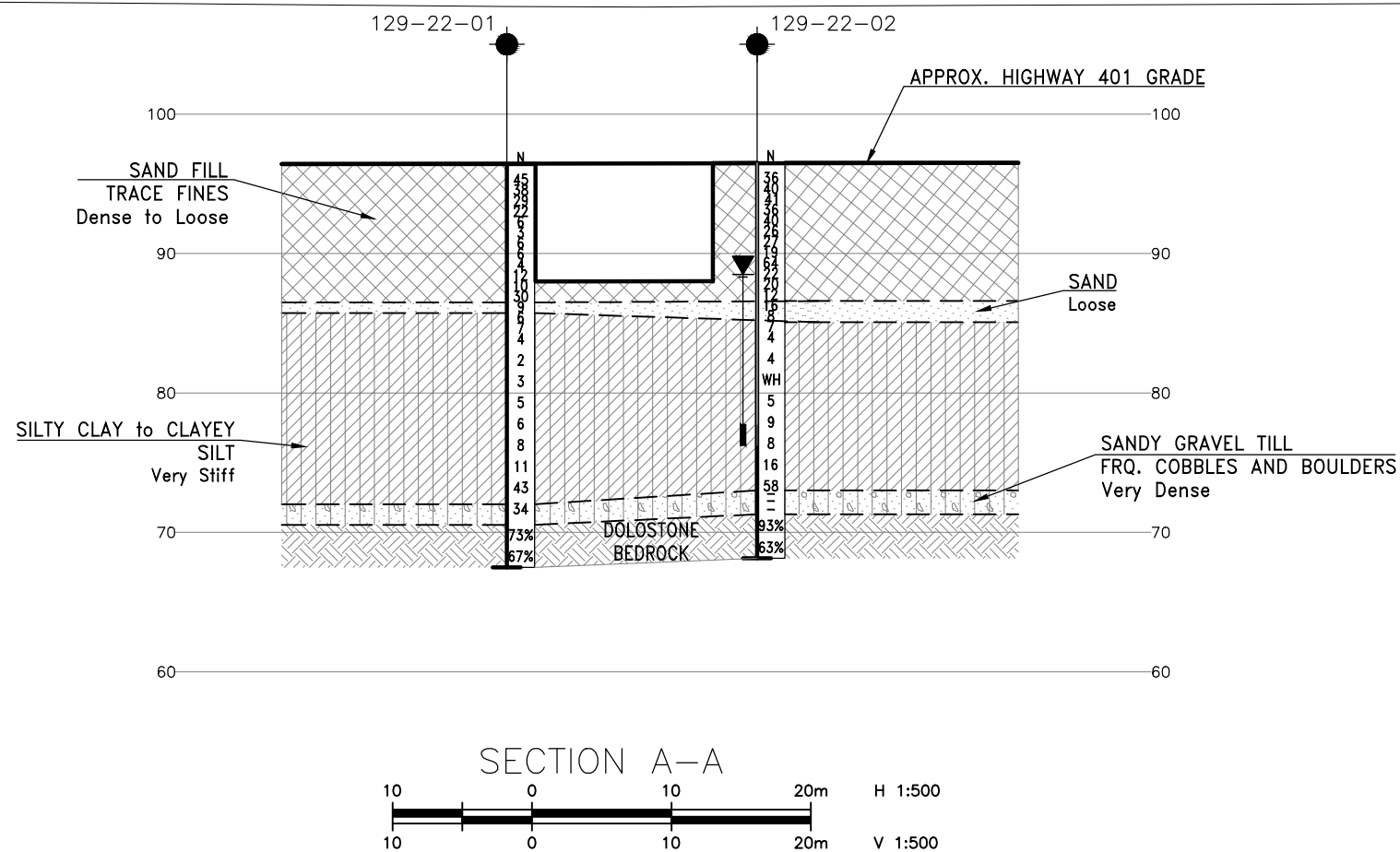
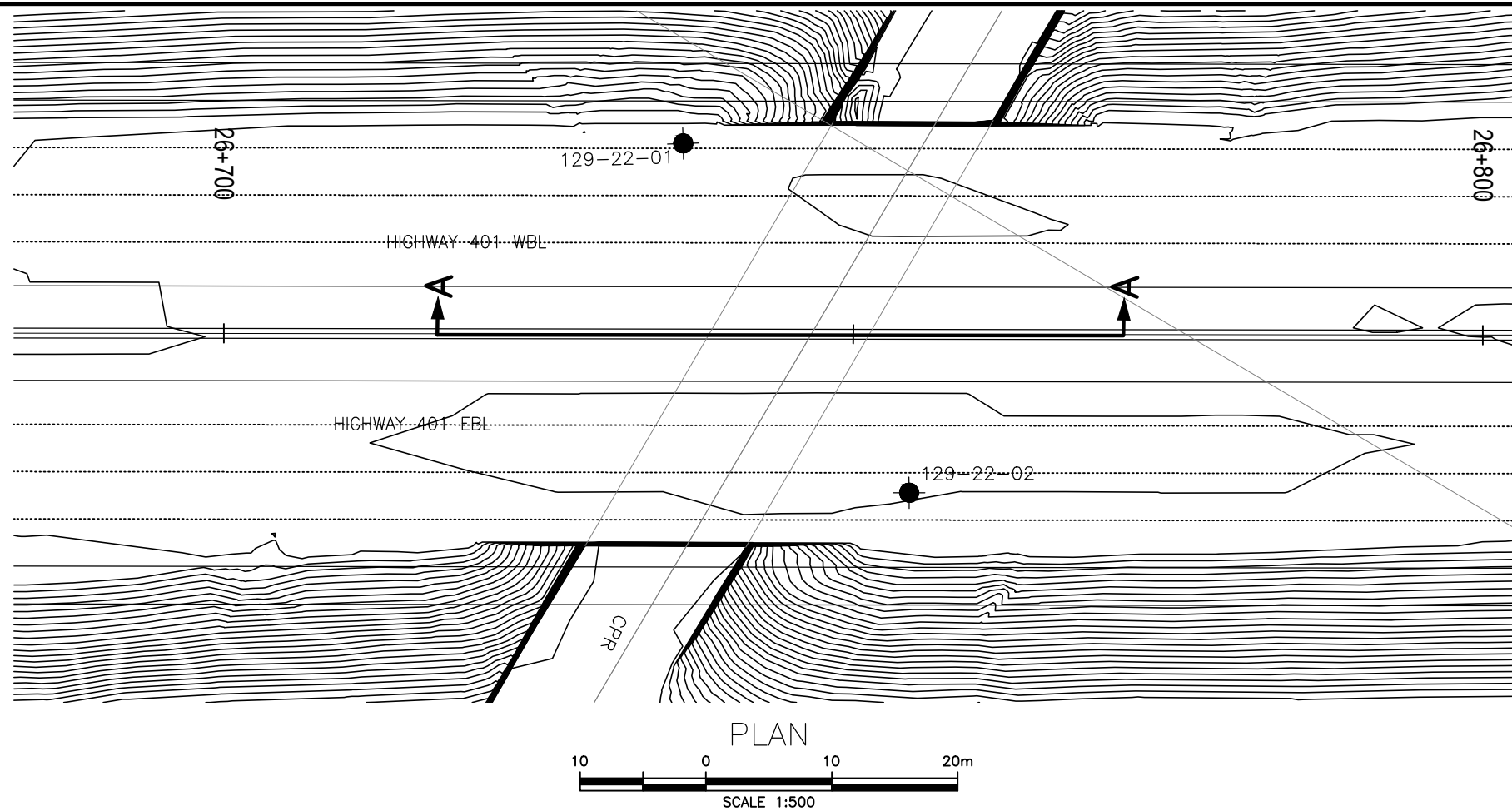
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Appendix A.

Borehole Location Plan and Stratigraphic Drawings

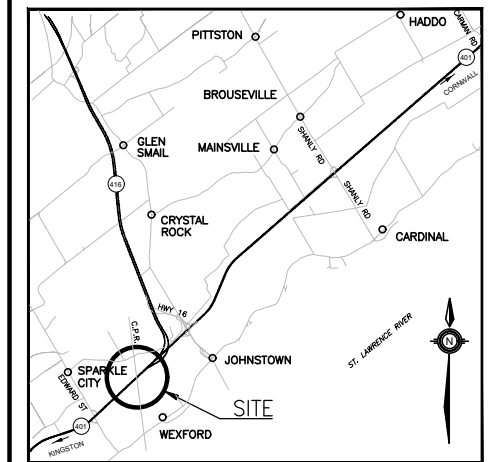


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AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

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GWP No 4024-20-00





HIGHWAY 401
CPR OVERHEAD
BRIDGE REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

Ontario 



KEYPLAN

LEGEND

	Borehole (Current 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

[illegible]

-NOTES-

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

GEOCRES No. 31B-113

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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).

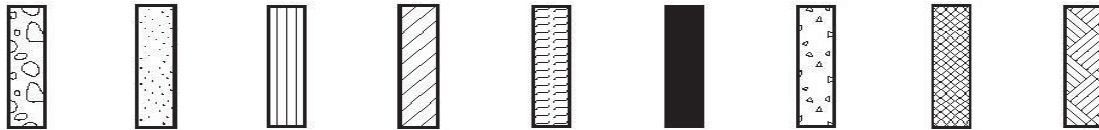
DYNAMIC CONE PENETRATION TEST (DCPT):

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 129-22-01

1 OF 3

METRIC

GWP# 4024-20-00 LOCATION Lat: 44.735948°, Long: -75.506123° Highway 401 & CPR, Edwardsburgh, MTM z9: N 4 955 585.5 E 383 515.3 ORIGINATED BY IK
 HWY 401 BOREHOLE TYPE CME 55 Truck Mount / HSA / NW Casing / NQ Coring COMPILED BY AO
 DATUM Geodetic DATE 2022.12.15 - 2022.12.15 CHECKED BY MJK

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									
96.5	Ground Surface							20	40	60	80	100					
0.0	ASPHALT (355 mm)																
96.1																	
0.4	GRAVELLY SAND, trace fines		1	GS	-		96										28 65 7 (SI+CL)
95.8	Brown																
0.7	FILL (BASE) -----																
	SAND, trace fines		1	SS	45												
	Dense to loose																
	Brown																
	FILL																
			2	SS	38		95										
			3	SS	29		94										
			4	SS	22		93										0 92 8 (SI+CL)
			5	SS	6		92										
		6	SS	3		91											
		7	SS	6		90											
		8	SS	6		89											
		9	SS	4		88										0 93 7 (SI+CL)	
		10	SS	12		87											
		11	SS	10													
87.4																	
9.1	SAND, some fines and gravel		12	SS	30												
	Dense																
	Grey																
	FILL																
86.6																	

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

METRIC

[illegible]

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

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 129-22-02

1 OF 3

METRIC



GWP# 4024-20-00 LOCATION Lat: 44.735878°, Long: -75.505718° Highway 401 & CPR, Edwardsburgh, MTM z9: N 4 955 578.1 E 383 547.5 ORIGINATED BY IK
HWY 401 BOREHOLE TYPE CME 55 Truck Mount / HSA / NW Casing / NQ Coring COMPILED BY AO
DATUM Geodetic DATE 2022.12.13 - 2022.12.14 CHECKED BY MJK

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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE							WATER CONTENT (%) PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT w _p w w _L			
96.6	Ground Surface							20	40	60	80	100		20	40	60		
0.0	ASPHALT (355 mm)																	
96.2																		
0.4	GRAVELLY SAND, trace fines																	
96.0	Brown																	
0.6	FILL (BASE)																	
	SAND, trace fines		1	SS	36													
	Dense to compact																	
	Brown																	
	FILL																	
			2	SS	40													
			3	SS	41													
			4	SS	36													
			5	SS	40													
			6	SS	26													
			7	SS	27													
			8	SS	19													
			9	SS	64													
89.0																		
7.6	SAND, some fines and gravel		10	SS	22													
	Compact																	
	Brown																	
	FILL																	
			11	SS	20													
			12	SS	12													
86.7																		

Continued Next Page

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

METRIC

SOIL PROFILE			SAMPLES			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS
86.6 9.6	Silty SAND with organics		13	SS	16	
10.1	SAND (SP-SM), trace fines Compact to loose Grey		14	SS	8	
85.2 11.4	SILTY CLAY (CI) to CLAYEY SILT (CL) Very stiff Grey		15	SS	7	
			16	SS	4	
			17	SS	4	
			18	SS	WH	
			19	SS	5	
			20	SS	9	

+³, ×³: Numbers refer to Sensitivity

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

METRIC

[illegible]

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

+³, ×³: Numbers refer to Sensitivity



Appendix C.

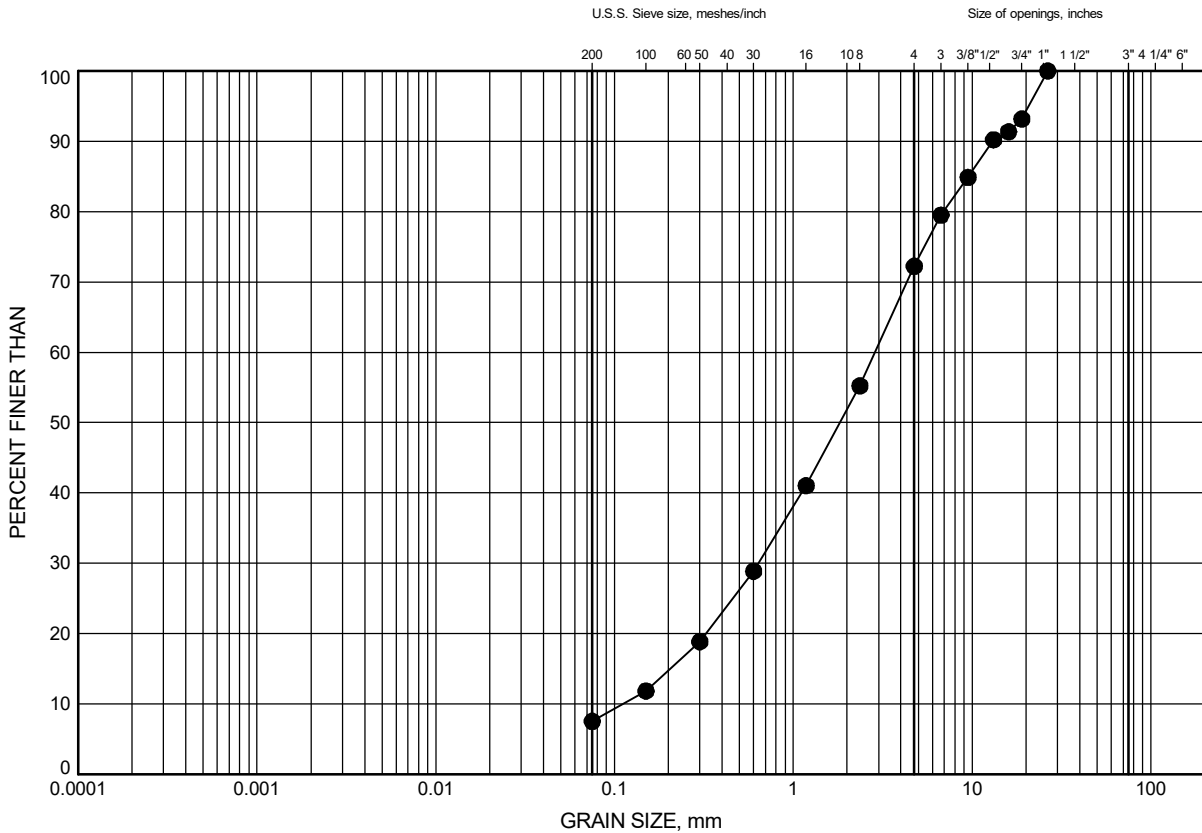
Laboratory Testing



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

GRAIN SIZE DISTRIBUTION

FILL: Gravelly Sand



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	129-22-01	0.5	95.9

Date March 2023

GWP# 4024-20-00

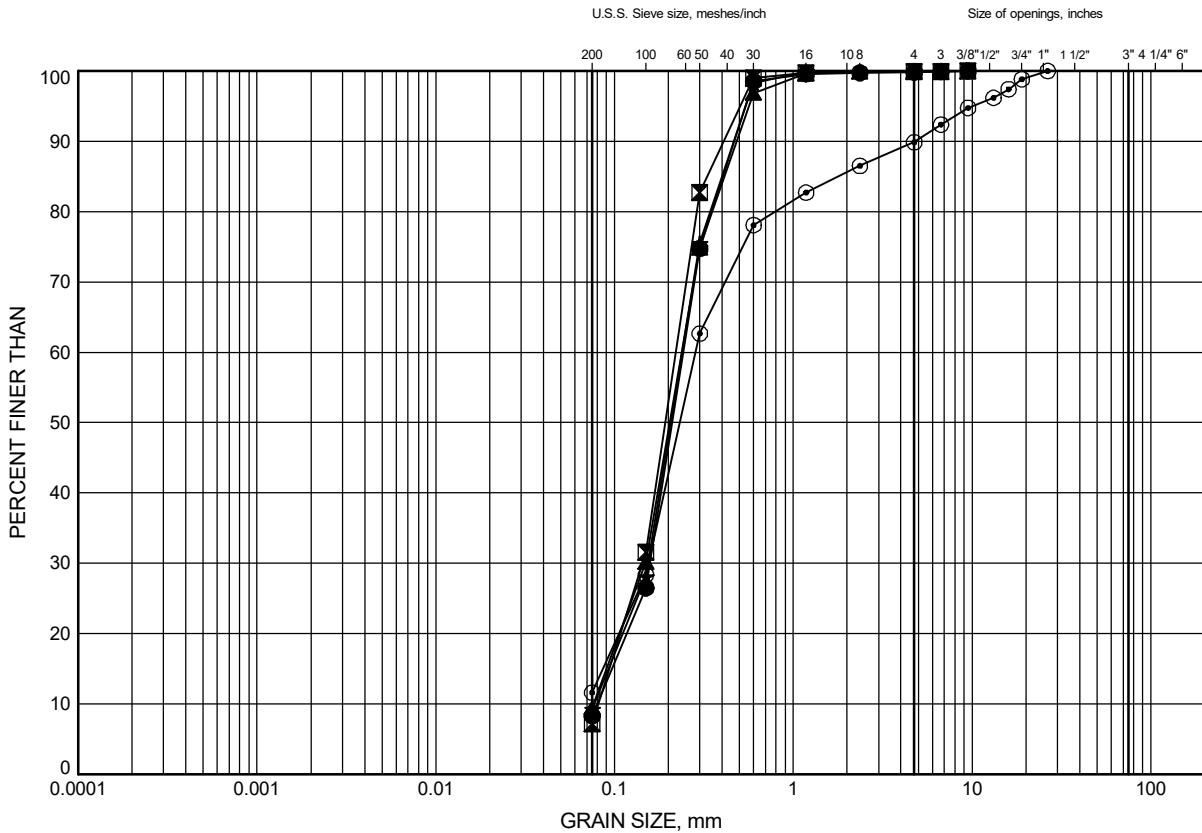


Prep'd RH

Chkd. AO

GRAIN SIZE DISTRIBUTION

FILL: Sand



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

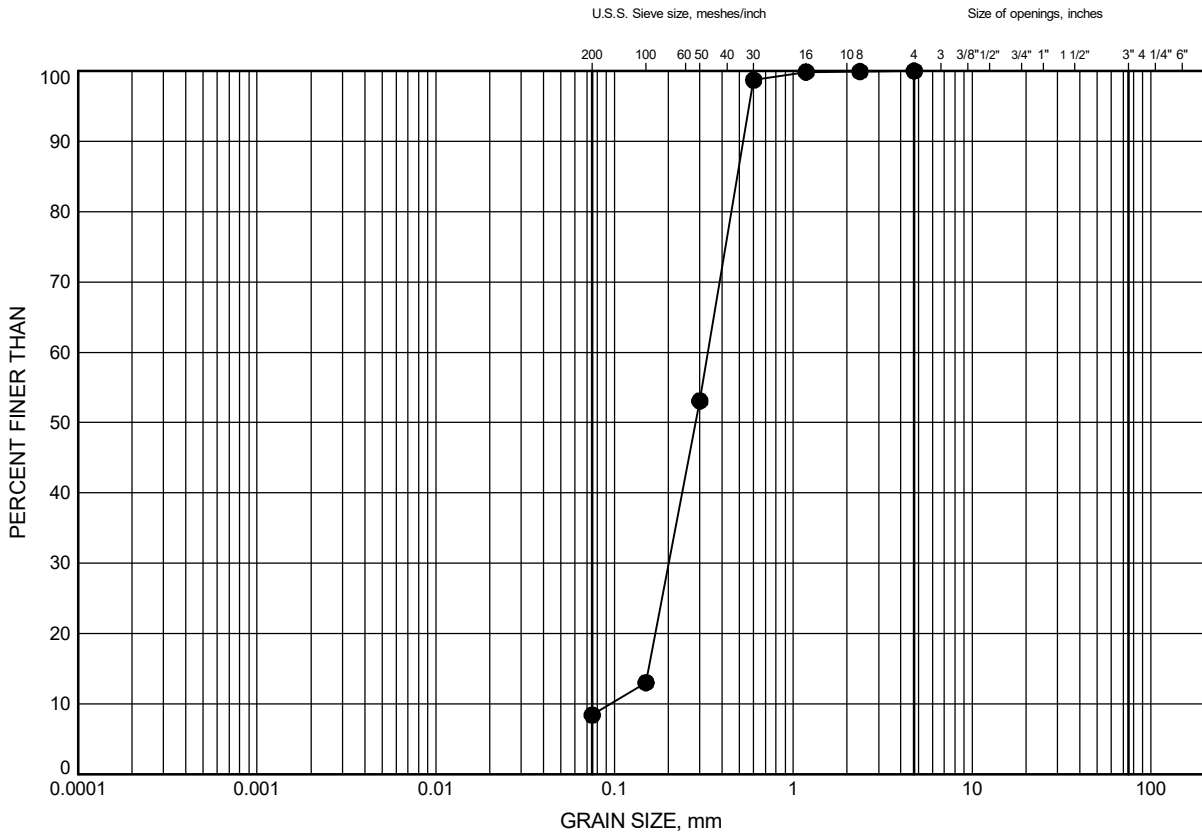
LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	129-22-01	3.4	93.0
⊠	129-22-01	7.2	89.2
▲	129-22-02	1.8	94.7
★	129-22-02	4.9	91.6
⊙	129-22-02	8.7	87.8

Date March 2023GWP# 4024-20-00Prep'd RHChkd. AO

GRAIN SIZE DISTRIBUTION

Sand (SP-SM)



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	129-22-02	11.0	85.5

Date March 2023

GWP# 4024-20-00

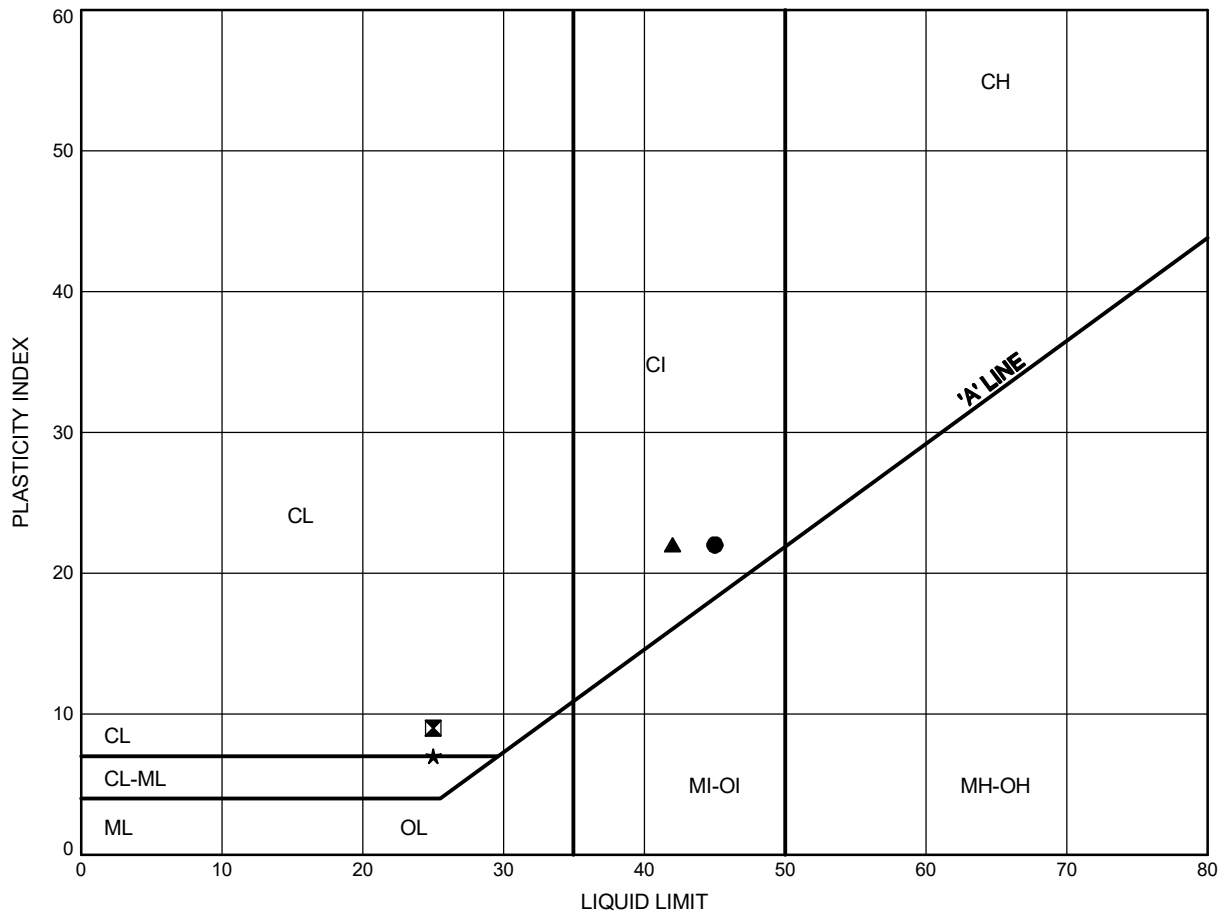


Prep'd RH

Chkd. AO

GRAIN SIZE DISTRIBUTION

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



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	129-22-01	17.1	79.3
⊠	129-22-01	21.6	74.8
▲	129-22-02	12.5	84.0
★	129-22-02	20.1	76.4

Date March 2023

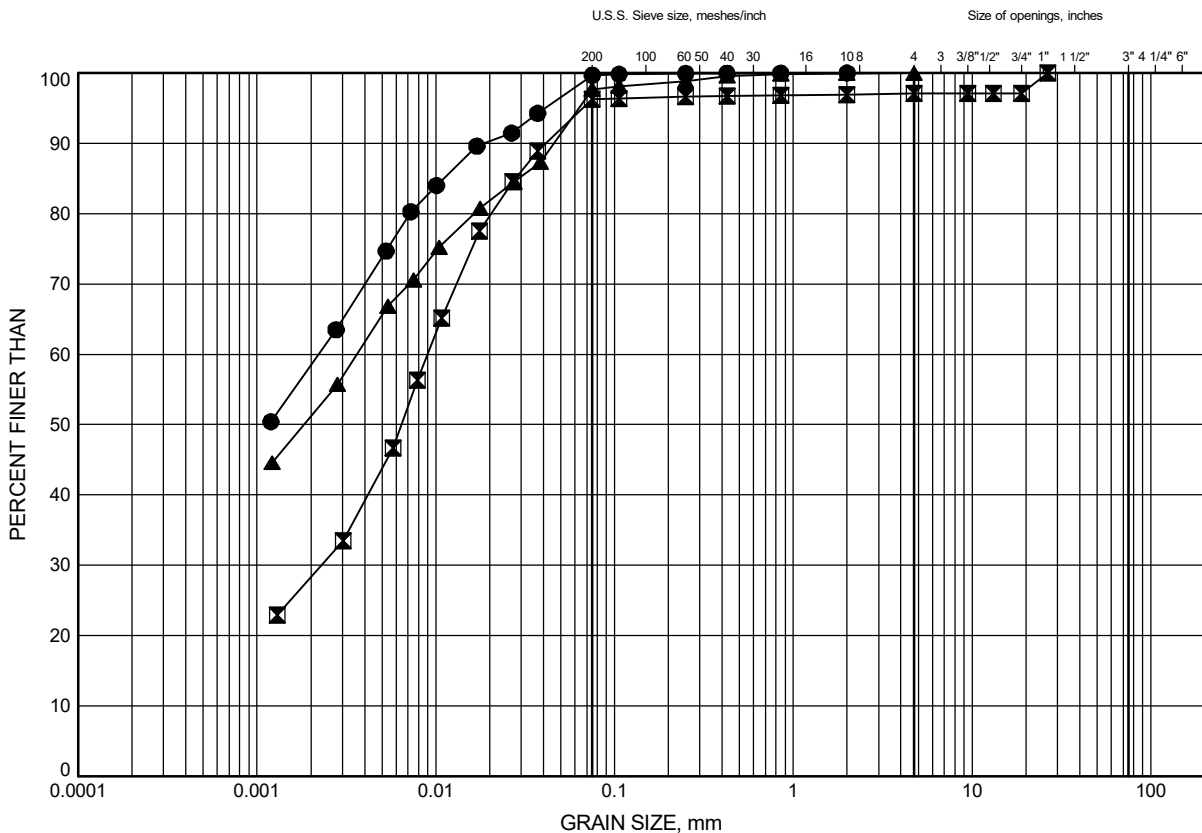
GWP# 4024-20-00



Prep'd RH

Chkd. AO

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



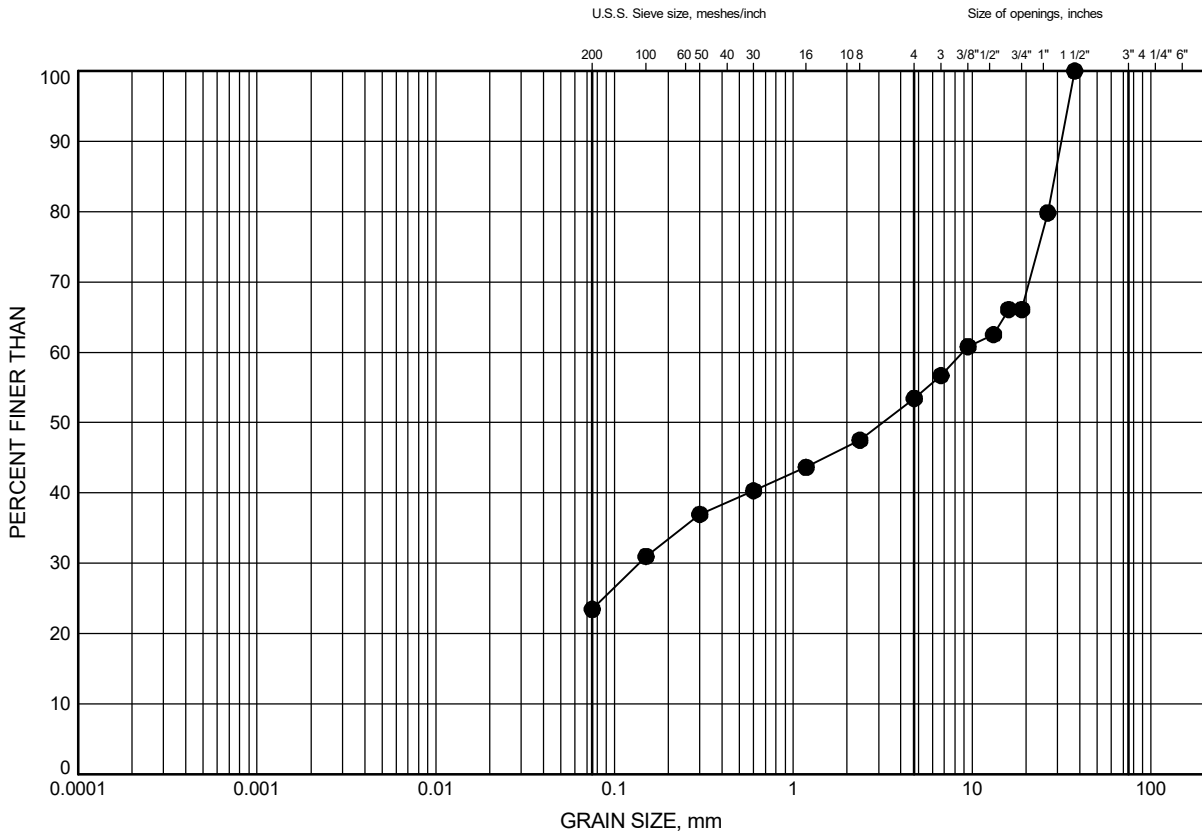
SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	129-22-01	17.1	79.3
⊠	129-22-01	21.6	74.8
▲	129-22-02	12.5	84.0

GRAIN SIZE DISTRIBUTION

GLACIAL TILL: Sandy Gravel (GM)



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	129-22-02	23.3	73.2

Date March 2023

GWP# 4024-20-00



Prep'd RH

Chkd. AO



Appendix C.3

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.4

Bedrock Core Photographs

Borehole 129-22-01

RUN 1

Depth 25.9 m to 27.3 m

Elevation 70.5 m to 69.1 m

Dry Sample

NQ1 – Cobbles and Boulders
elev. 71.2 m to 70.5 m

Run 1 Start
elev. 70.5 m



Run 1 End
elev. 69.1 m

Borehole 129-22-01

RUN 1

Depth 25.9 m to 27.3 m

Elevation 70.5 m to 69.1 m

Wet Sample

NQ1 – Cobbles and Boulders
elev. 71.2 m to 70.5 m

Run 1 Start
elev. 70.5 m



Run 1 End
elev. 69.1 m

Borehole 129-22-01

RUN 2

Depth 27.3 m to 28.9 m

Elevation 69.1 m to 67.5 m

Dry Sample

Run 2 Start
elev. 69.1 m



Run 2 End
elev. 67.5 m

Borehole 129-22-01

RUN 2

Depth 27.3 m to 28.9 m

Elevation 69.1 m to 67.5 m

Wet Sample

Run 2 Start
elev. 69.1 m



Run 2 End
elev. 67.5 m

Borehole 129-22-02
NQ1, NQ2, and NQ3 – Cobbles and Boulders
Depth 23.5 m to 25.2 m
Elevation 73.0 m to 71.3 m
Dry Sample

NQ1, NQ2, and NQ3 – Cobbles and Boulders Start
elev. 73.0 m



NQ1, NQ2, and NQ3 – Cobbles and Boulders End
elev. 71.3 m

Borehole 129-22-02
NQ1, NQ2, and NQ3 – Cobbles and Boulders
Depth 23.5 m to 25.2 m
Elevation 73.0 m to 71.3 m
Wet Sample

NQ1, NQ2, and NQ3 – Cobbles and Boulders Start
elev. 73.0 m



NQ1, NQ2, and NQ3 – Cobbles and Boulders End
elev. 71.3 m

Borehole 129-22-02

RUN 1

Depth 25.2 m to 26.8 m
Elevation 71.3 m to 69.7 m
Dry Sample

Run 1 Start
elev. 71.3 m



Run 1 End
elev. 69.7 m

Borehole 129-22-02

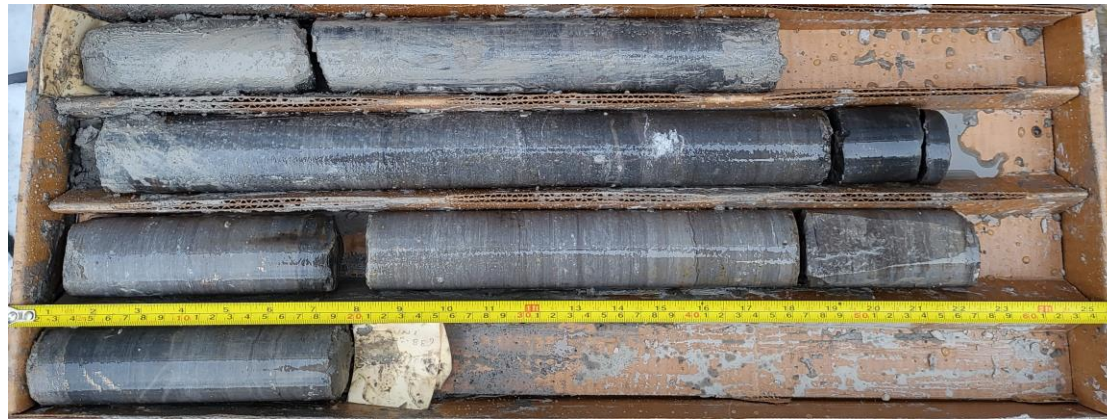
RUN 1

Depth 25.2 m to 26.8 m

Elevation 71.3 m to 69.7 m

Wet Sample

Run 1 Start
elev. 71.3 m



Run 1 End
elev. 69.7 m

Borehole 129-22-02

RUN 2

Depth 26.8 m to 28.3 m

Elevation 69.7 m to 68.2 m

Dry Sample

Run 2 Start
elev. 69.7 m



Run 2 End
elev. 68.2 m

Borehole 129-22-02

RUN 2

Depth 26.8 m to 28.3 m

Elevation 69.7 m to 68.2 m

Wet Sample

Run 2 Start
elev. 69.7 m



Run 2 End
elev. 68.2 m



Appendix D.

Site Photographs



Photograph 1: Looking east at the east embankment, north slope.
[taken on December 18, 2022]



Photograph 2: Looking north at the east embankment, south slope.
[taken on December 18, 2022]



Photograph 3: Looking east at the east embankment, south slope.
[taken on December 13, 2022]



Photograph 4: Looking west at the west embankment, south slope.
[taken on December 13, 2022]



Photograph 5: Looking north at the Highway 401 overhead at CPR
[taken on December 18, 2022]



Photograph 6: Looking east across the bridge deck, eastbound Highway 401 lanes.
[taken on December 13, 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: 44.736N 75.506W

User File Reference: Highway 401 CPR Overhead

2023-02-02 15:13 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)	0.374	0.212	0.127	0.036
Sa (0.1)	0.443	0.260	0.163	0.051
Sa (0.2)	0.374	0.224	0.143	0.048
Sa (0.3)	0.286	0.173	0.111	0.039
Sa (0.5)	0.205	0.124	0.080	0.028
Sa (1.0)	0.104	0.064	0.041	0.014
Sa (2.0)	0.050	0.030	0.019	0.006
Sa (5.0)	0.013	0.007	0.004	0.001
Sa (10.0)	0.005	0.003	0.002	0.001
PGA (g)	0.240	0.142	0.089	0.028
PGV (m/s)	0.170	0.099	0.061	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



Natural Resources
Canada

Ressources naturelles
Canada

Canada



Appendix F.

Original General Arrangement Drawing



Appendix G.

Comparison of Foundation Alternatives







COMPARISON OF FOUNDATION ALTERNATIVES

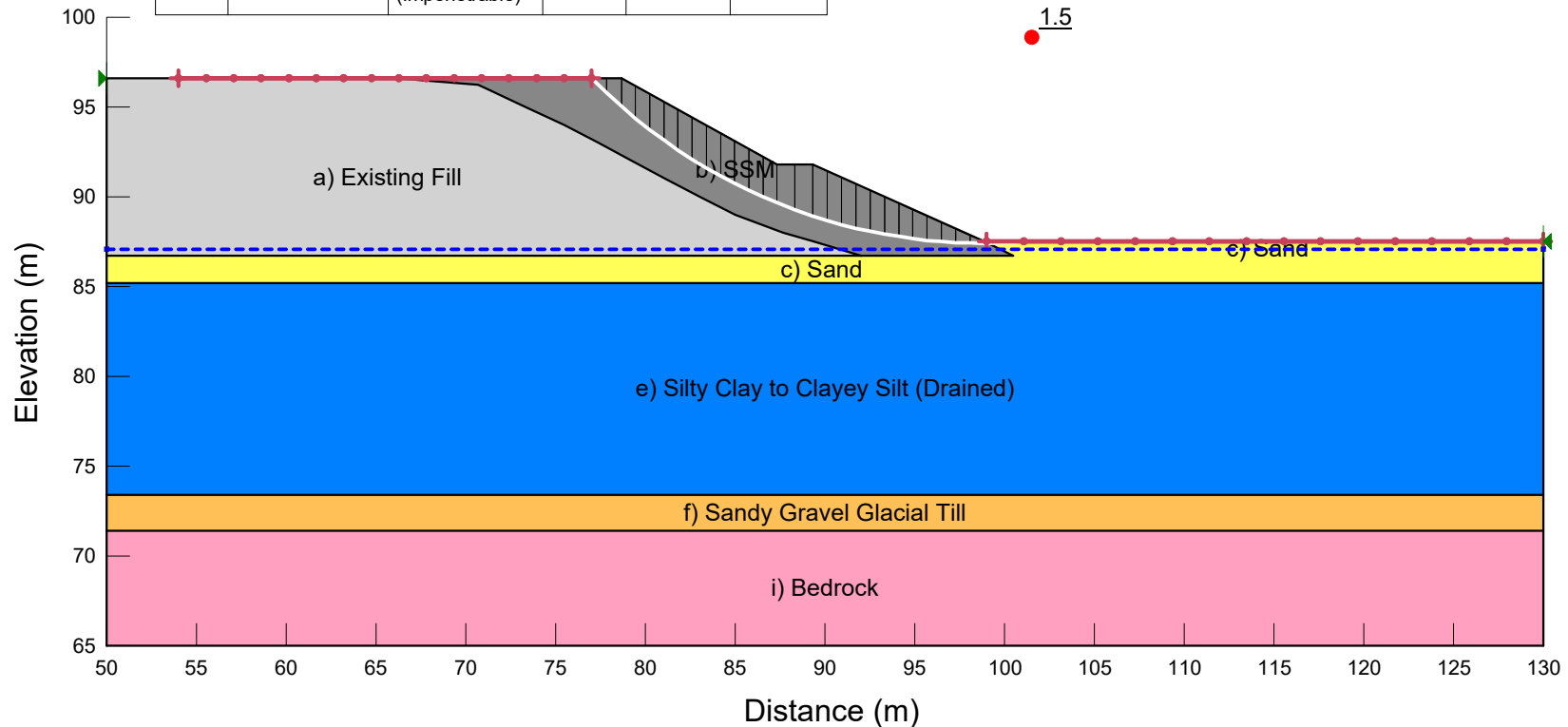
	<i>Spread Footings</i>	<i>Driven Steel H-Piles</i>	<i>Concrete Caissons</i>
Description	<ul style="list-style-type: none"> Foundation elements founded on traditional shallow spread footings on engineering fill, native stiff silty clay. 	<ul style="list-style-type: none"> The abutments would be supported by a single row of steel H-piles driven to refusal on bedrock. 	<ul style="list-style-type: none"> A reinforced concrete column installed within an augered/cored hole in the ground/rock that derives axial resistance from end bearing
Advantages	<ul style="list-style-type: none"> Existing structure supported on shallow footings at abutments and appears has performed reasonably. 	<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. Pile caps can be perched within the embankments to reduce excavations. 	<ul style="list-style-type: none"> Moderate to high axial geotechnical resistance. Can handle oversized obstructions. Suitable for semi-integral abutment design approach. Requires less excavation than spread footings.
Disadvantages	<ul style="list-style-type: none"> Significant excavations required, beneath existing high embankments and will likely require temporary protection systems. Excavations to bearing strata expected to be below groundwater level and require significant dewatering operations. Potential for differential settlement of the structure. 	<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.
Risks / Consequences	<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.
Relative Cost	<ul style="list-style-type: none"> Low/Moderate 	<ul style="list-style-type: none"> Moderate 	<ul style="list-style-type: none"> High
Conclusion	<ul style="list-style-type: none"> Not recommended 	<ul style="list-style-type: none"> Recommended 	<ul style="list-style-type: none"> Feasible, but not recommended



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	30
	b) SSM	Mohr-Coulomb	20	0	30
	c) Sand	Mohr-Coulomb	19	0	31
	e) Silty Clay to Clayey Silt (Drained)	Mohr-Coulomb	17.5	7	29
	f) Sandy Gravel Glacial Till	Mohr-Coulomb	19	0	35
	i) Bedrock	Bedrock (Impenetrable)			



Project
Highway 401 O/H at CPR Crossing - SE Embankment

Analysis
1.1) Permanent - Long Term

Seismic Coefficient
H: g, V: g

Last Run
2023/08/16, 09:13:40 AM

Scale
1:400

Additional Details

Name: 1 East Embankment - SSM Widening

Comments:







Method: Morgenstern-Price, Half-Sine

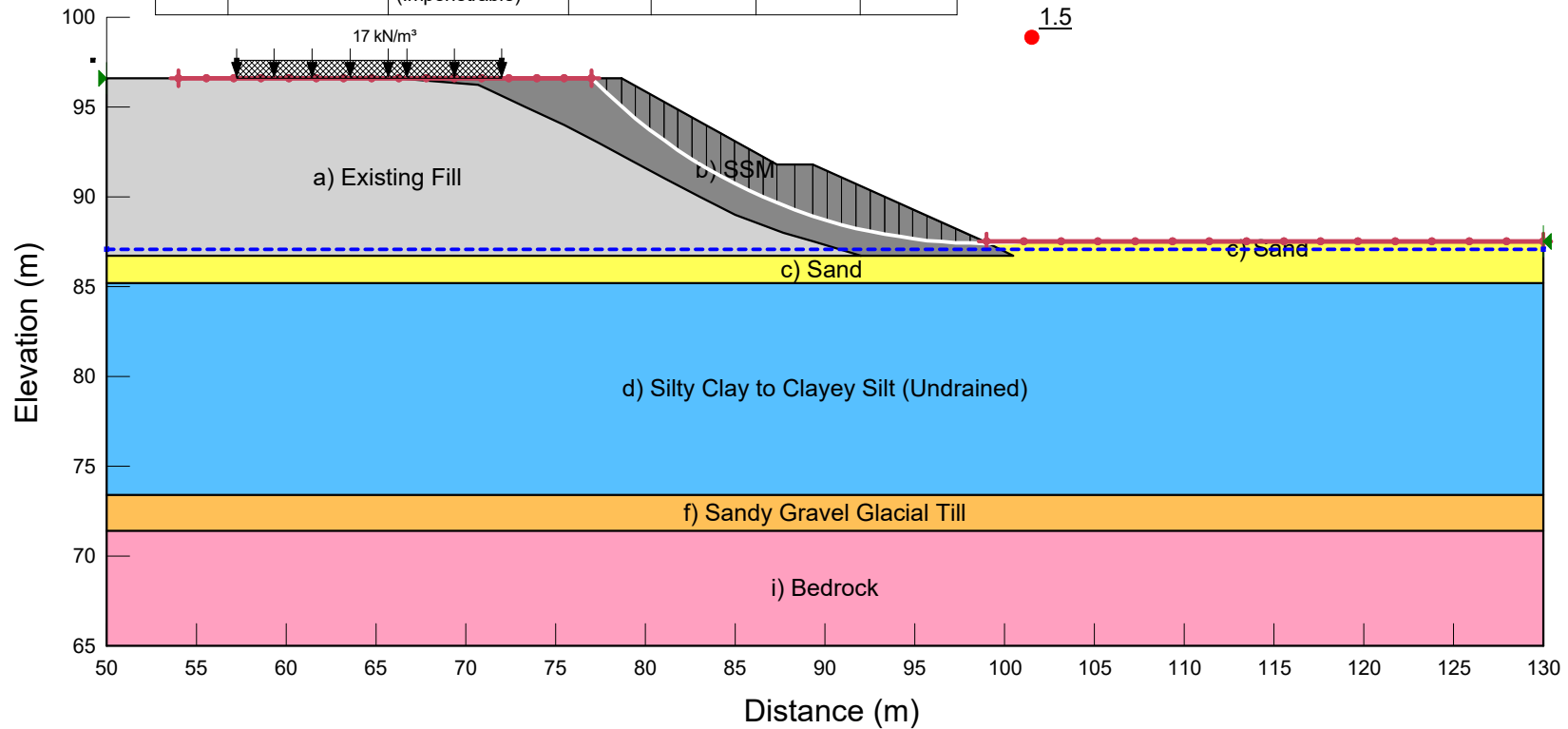
Minimum Slip Surface Depth: 1.52 m

Entry: (77, 96.6) m, Exit: (101.06667, 87.5) m

Center: (98.819092, 117.93029) m, Radius: 30.513176 m

Figure H1.1

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	30
	b) SSM	Mohr-Coulomb	20		0	30
	c) Sand	Mohr-Coulomb	19		0	31
	d) Silty Clay to Clayey Silt (Undrained)	Undrained (Phi=0)	17.5	100		
	f) Sandy Gravel Glacial Till	Mohr-Coulomb	19		0	35
	i) Bedrock	Bedrock (Impenetrable)				



Project
Highway 401 O/H at CPR Crossing - SE Embankment

Analysis
1.2) Temporary (traffic) - Short Term

Seismic Coefficient
H: g, V: g







Last Run
2023/08/16, 09:13:41 AM

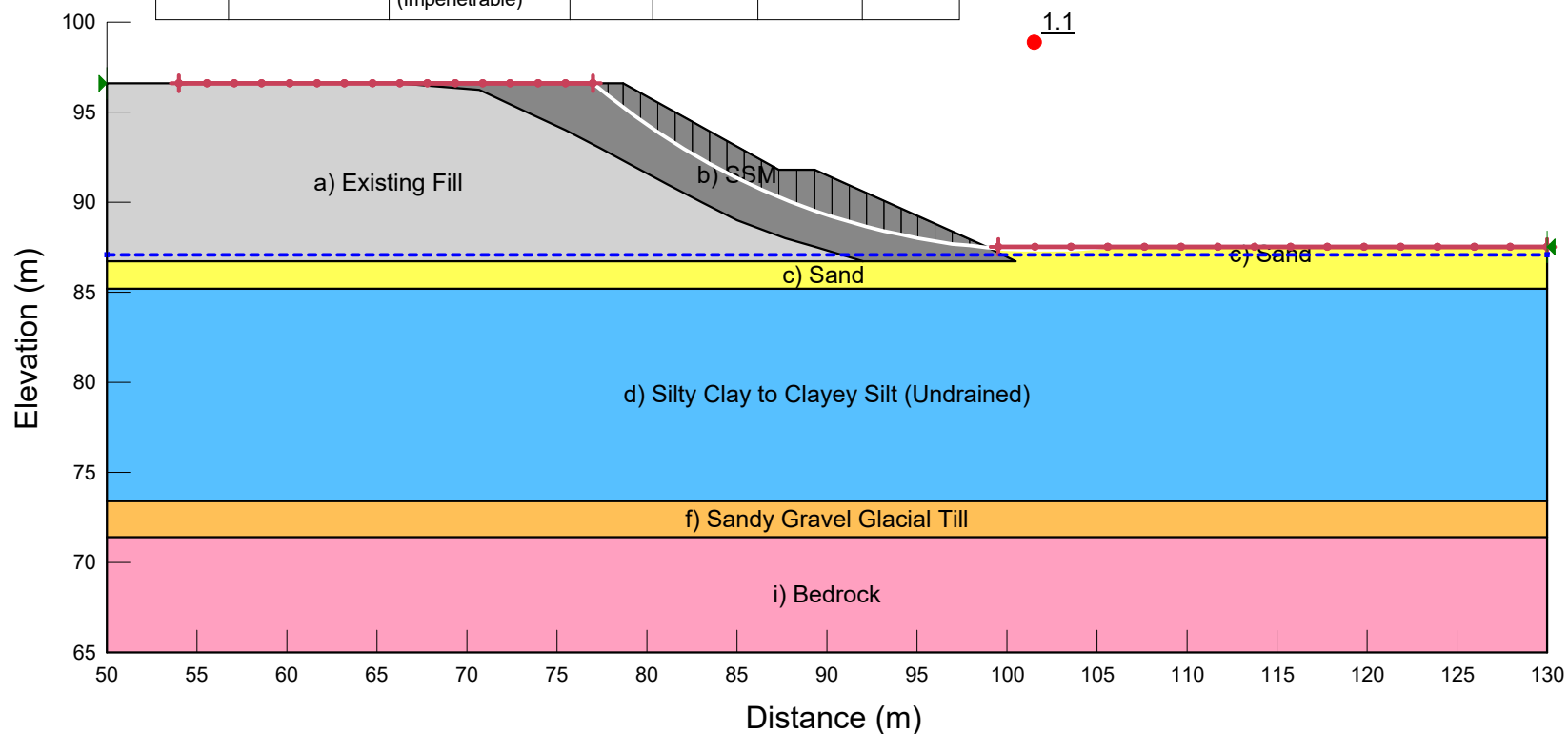
Scale
1:400

Additional Details

Name: 1 East Embankment - SSM Widening
Comments:
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
Entry: (77, 96.6) m, Exit: (101.06667, 87.5) m
Center: (98.819092, 117.93029) m, Radius: 30.513176 m

Figure H1.2







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	30
	b) SSM	Mohr-Coulomb	20		0	30
	c) Sand	Mohr-Coulomb	19		0	31
	d) Silty Clay to Clayey Silt (Undrained)	Undrained (Phi=0)	17.5	100		
	f) Sandy Gravel Glacial Till	Mohr-Coulomb	19		0	35
	i) Bedrock	Bedrock (Impenetrable)				

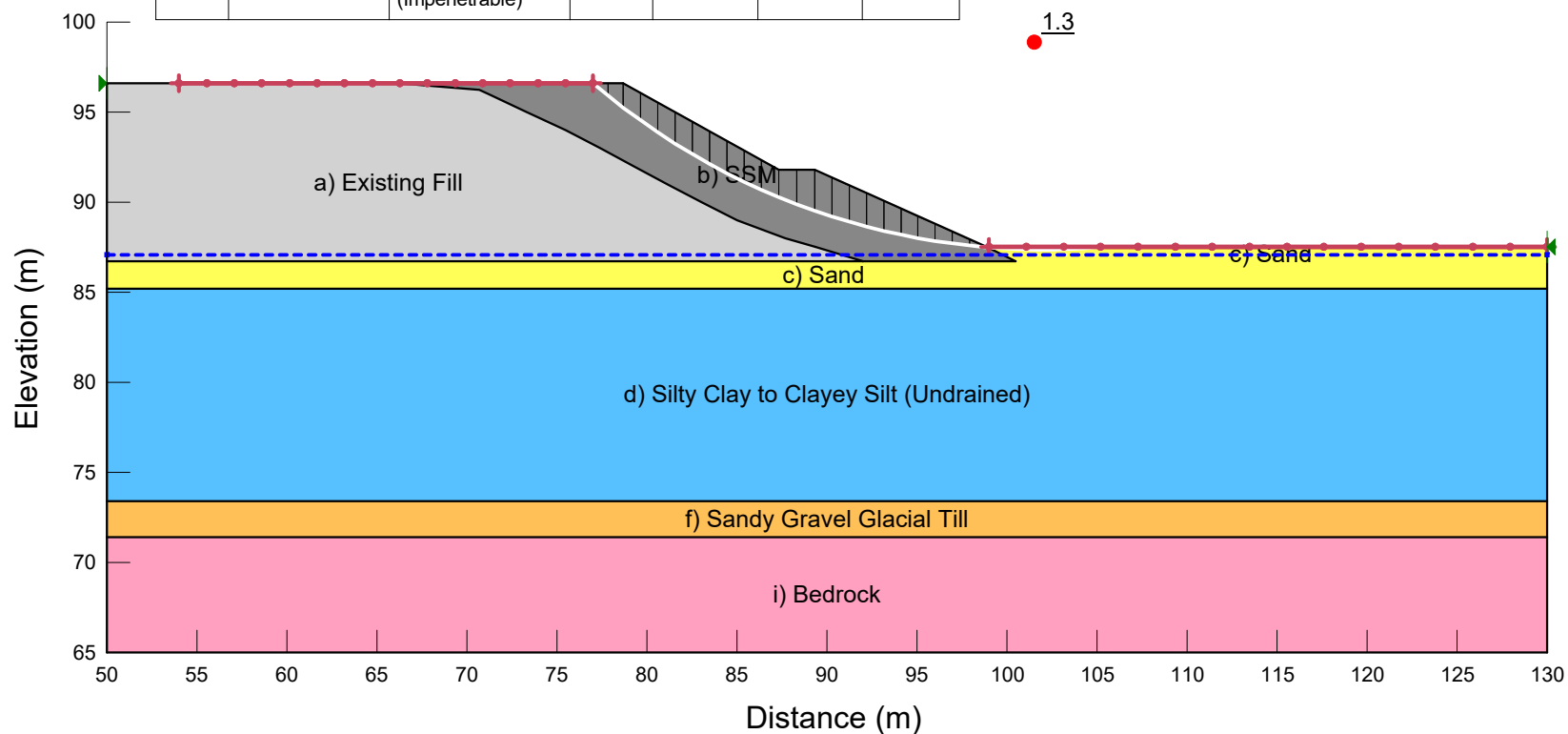


Project Highway 401 O/H at CPR Crossing - SE Embankment		
Analysis 1.3) Temporary (2,475 yr EQ) - Short Term		
Seismic Coefficient H: 0.134g, V: g	Last Run 2023/08/16, 09:13:42 AM	Scale 1:400

Additional Details
 Name: 1 East Embankment - SSM Widening
 Comments:
 Method: Morgenstern-Price, Half-Sine
 Minimum Slip Surface Depth: 1.52 m
 Entry: (77, 96.6) m, Exit: (105.6, 87.5) m
 Center: (102.15616, 126.16936) m, Radius: 38.822411 m

Figure H1.3

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	30
	b) SSM	Mohr-Coulomb	20		0	30
	c) Sand	Mohr-Coulomb	19		0	31
	d) Silty Clay to Clayey Silt (Undrained)	Undrained (Phi=0)	17.5	100		
	f) Sandy Gravel Glacial Till	Mohr-Coulomb	19		0	35
	i) Bedrock	Bedrock (Impenetrable)				



Project
Highway 401 O/H at CPR Crossing - SE Embankment

Analysis
1.4) Temporary (475 yr EQ) - Short Term

Seismic Coefficient
H: 0.057g, V: g

Last Run
2023/08/16, 09:13:42 AM

Scale
1:400

Additional Details

Name: 1 East Embankment - SSM Widening

Comments:








Method: Morgenstern-Price, Half-Sine

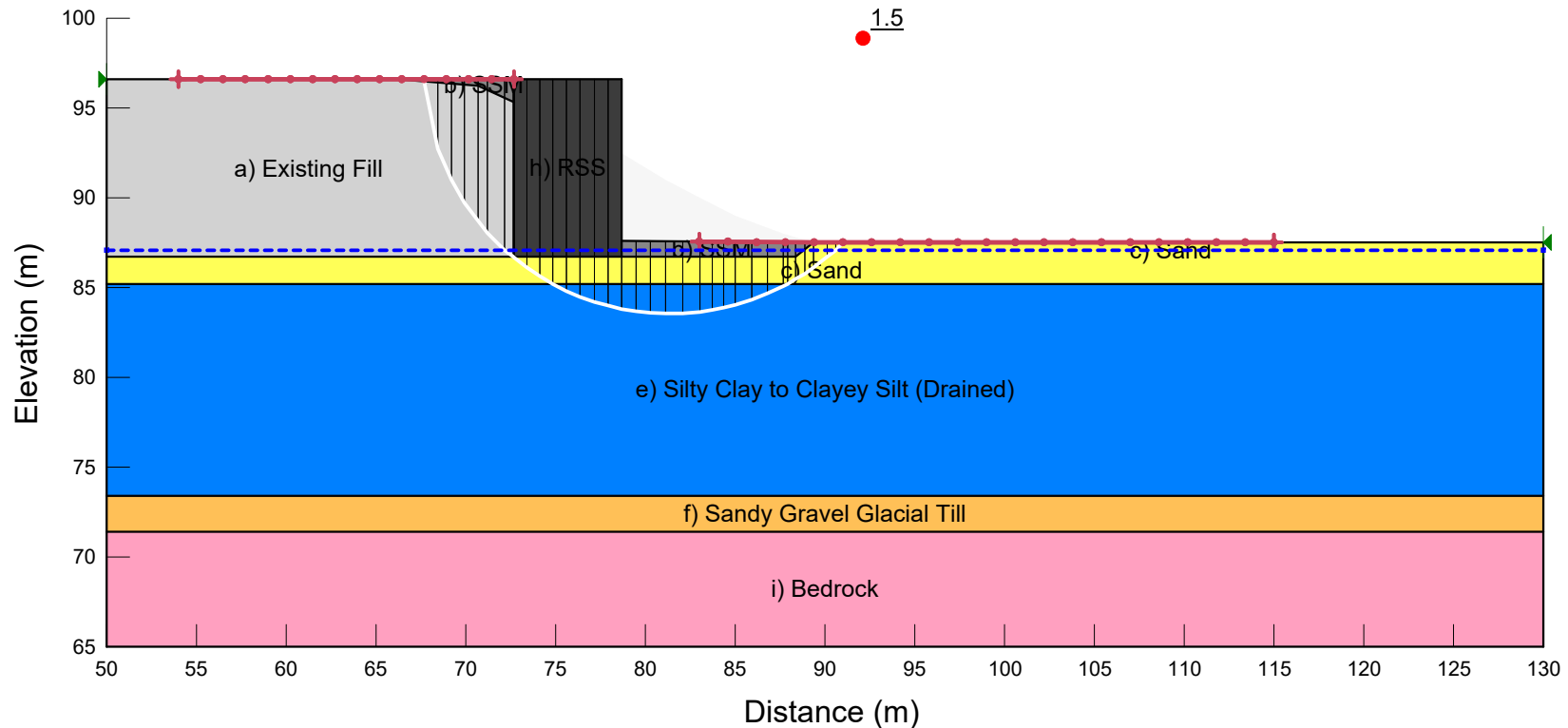
Minimum Slip Surface Depth: 1.52 m

Entry: (77, 96.6) m, Exit: (105.2, 87.5) m

Center: (102.00685, 125.84926) m, Radius: 38.481966 m

Figure H1.4

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	a) Existing Fill	Mohr-Coulomb	20	0	30
	b) SSM	Mohr-Coulomb	20	0	30
	c) Sand	Mohr-Coulomb	19	0	31
	e) Silty Clay to Clayey Silt (Drained)	Mohr-Coulomb	17.5	7	29
	f) Sandy Gravel Glacial Till	Mohr-Coulomb	19	0	35
	h) RSS	Mohr-Coulomb	22.8	250	35 </td
	i) Bedrock	Bedrock (Impenetrable)			



Project
Highway 401 O/H at CPR Crossing - SE Embankment

Analysis
2.1) Permanent - Long Term

Seismic Coefficient
H: g, V: g

Last Run
2023/08/16, 09:13:43 AM

Scale
1:400

Additional Details

Name: 2 RSS Wall

Comments:

Method: Morgenstern-Price, Half-Sine

Minimum Slip Surface Depth: 1.52 m

Entry: (67.684, 96.6) m, Exit: (90.999792, 87.5) m

Center: (81.370708, 97.248171) m, Radius: 13.702047 m

Figure H2.1



Project	Highway 401 O/H at CPR Crossing - SE Embankment
---------	---

Analysis

2.2) Temporary (traffic) - Short Term

Seismic Coefficient
H: g, V: g

Last Run
2023/08/16, 09:13:45 AM

Scale
1:400

Additional Details

Name: 2 RSS Wall

Comments:








Method: Morgenstern-Price, Half-Sine

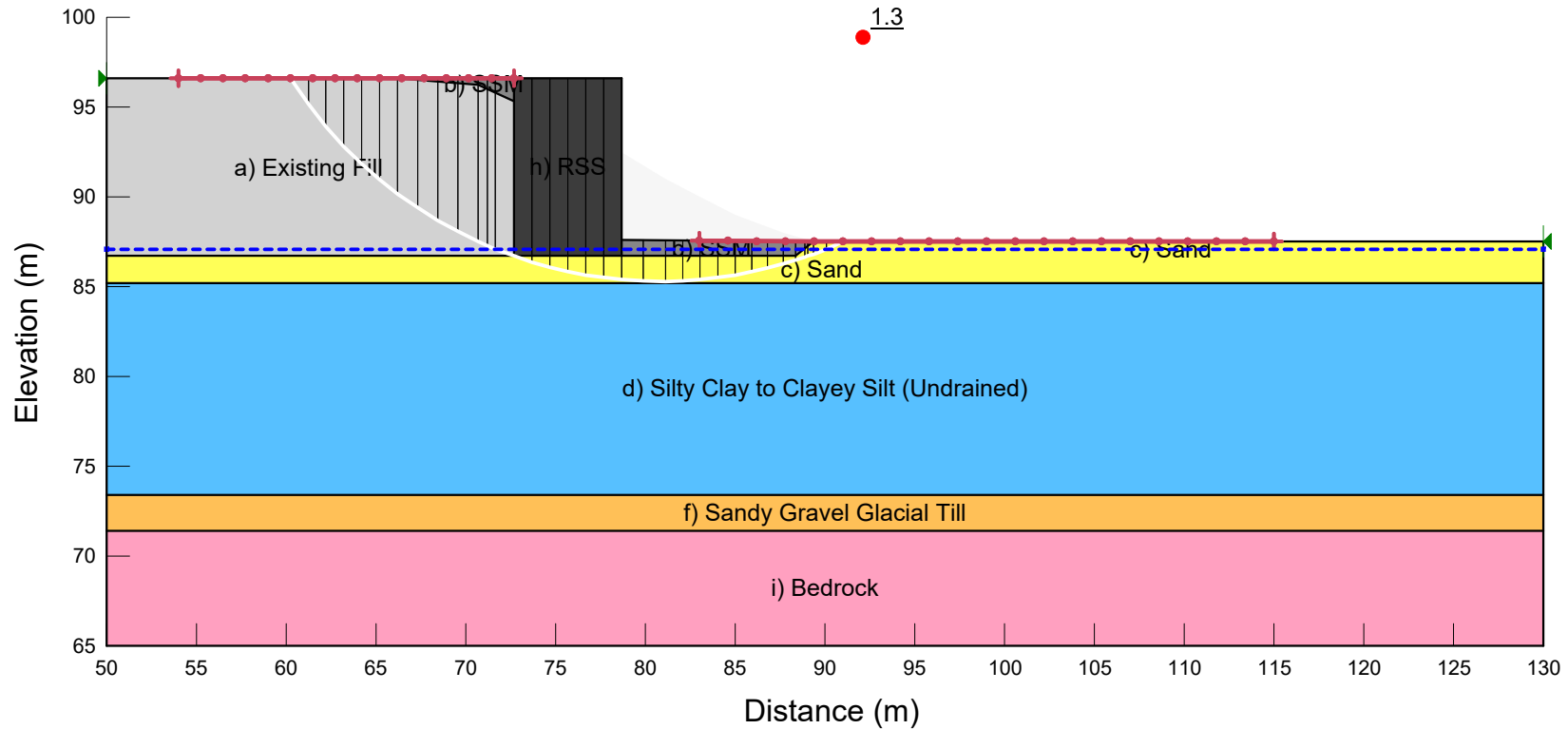
Minimum Slip Surface Depth: 1.52 m

Entry: (65.196, 96.6) m, Exit: (84.599944, 87.544383) m

Center: (77.270238, 97.155371) m, Radius: 12.087004 m

Figure H2.2

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	30
	b) SSM	Mohr-Coulomb	20		0	30
	c) Sand	Mohr-Coulomb	19		0	31
	d) Silty Clay to Clayey Silt (Undrained)	Undrained (Phi=0)	17.5	100		
	f) Sandy Gravel Glacial Till	Mohr-Coulomb	19		0	35
	h) RSS	Mohr-Coulomb	22.8		250	35
	i) Bedrock	Bedrock (Impenetrable)				



Project
Highway 401 O/H at CPR Crossing - SE Embankment

Analysis
2.3) Temporary (2,475 yr EQ) - Short Term

Seismic Coefficient
H: 0.134g, V: g

Last Run
2023/08/16, 09:13:45 AM

Scale
1:400

Additional Details

Name: 2 RSS Wall

Comments:

Method: Morgenstern-Price, Half-Sine

Minimum Slip Surface Depth: 1.52 m

Entry: (60.22, 96.6) m, Exit: (90.999792, 87.5) m

Center: (80.849913, 109.77381) m, Radius: 24.477387 m

Figure H2.3



Appendix I.

List of Referenced Specifications



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 3121.150	Walls, Retaining Backfill, Minimum Granular Requirement
OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS.PROV 1010	Material Specification for Aggregates Base, Subbase, Select Subgrade, and Backfill Material
OPSD 208.010	Benching of Earth Slopes