



THURBER ENGINEERING LTD.

**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 401 WIDENING, HIGHWAY 16 TO MAITLAND ROAD
MERWIN LANE UNDERPASS, SITE NO. 16X-0166
GWP 4024-20-00 / ASSIGNMENT NO. 4019-E-0010.2**

SITE NO. 16X-0166/B0

Geocres No.: 31B-110

Report to:

MTO c/o AECOM Canada Ltd.

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Longitude: -75.541920°

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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 Underpass at Merwin Lane (Site 16X-0166/B0), located approximately 1.5 km west of the town of Prescott in the Township of Augusta within the 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. 31B00-002).

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

Merwin Lane crosses over Highway 401 approximately 1.5 km west of the community of Prescott, Ontario. For project orientation purposes, Merwin Lane will be described as oriented north-south and Highway 401 as oriented east-west.



The existing structure carries two through lanes of Merwin Lane traffic over Highway 401. Steel bridge railings integrated into concrete curbs are present along the east and west edges of the structure deck. W-beam guiderails supported on wooden posts are present at all four quadrants and extend up to 34 m behind the abutments. The embankment side slopes are inclined at approximately 2.0H:1V, and the forward slopes beneath the abutments are inclined at approximately 2.0H:1V to 2.2H:1V and are protected with concrete slope paving. All embankment side slopes are vegetated with grasses, shrubs, and small trees. No signs of instability of the embankments were noted during the field investigation.

At the site, Highway 401 consists of two through lanes in each direction. The outside and median shoulders are fully paved and delineated with jersey barriers and W-beam guiderails. In the area of the structure the Highway 401 median is more than 18 m wide from shoulder to shoulder. Beyond the site, approximately 80 m north and south of Merwin Lane, the eastbound and westbound lanes of Highway 401 are separated by a grass median.

The site is in a semi-rural setting, and the area directly adjacent is undeveloped land with a mix of cleared private properties and densely vegetated areas with deciduous trees and shrubs. Overhead utility lines parallel Merwin Lane near the southbound embankment toe, and a utility pole is also present at the southbound embankment approximately 20 m from the south abutment. The terrain along the ditch line is relatively rugged in the vicinity of the site. Storm water drainage in the area is to roadside ditches.

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 Smith's Falls Limestone Plain and the Edwardsburg Sand Plain. The Smith's Falls Limestone Plain is characterized by typically shallow bedrock but includes a few localized deep areas of highly variable soils consisting of clays, sands, and gravels. The Edwardsburg Sand Plain is characterized by slightly undulating sand plain of glaciofluvial origin. The rock strata in both areas are generally composed of limestone, dolostone, and calcareous sandstone.

The Ontario Geological Survey maps (MRD126) suggest the site is underlain by dolostone and sandstone. Map P.2495ⁱ indicates that the bedrock in the project area is from the March Formation of the Beekmantown Group, and consists of interbedded sandstone, dolostone, sandy dolostone, and dolomitic sandstone.

2.3 Geocres Report 31B00-002

The historic foundation report for this site is based on a field investigation completed in 1964 prior to the construction of the existing underpass. The field investigation included a total of ten boreholes and four penetration tests. As bedrock was encountered at very shallow depth, no samples were taken, no laboratory tests were performed and no borehole logs were included in



the report. The stratigraphic plots indicate a compact sand and gravel fill over bedrock. The fill thickness was a maximum of 1.2 m.

Information from Geocres Report 31B00-002 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 November 21 and December 7, 2022, and consisted of three boreholes: one behind each abutment on Merwin Lane (Boreholes 166-22-01 and 166-22-02), and one within the Highway 401 median (Borehole 166-22-03). 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 the location of electrical and fibre optic utilities within the project limits.

The borehole coordinates, elevations, and termination depths are provided below 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 elevations 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)
162-22-01	South Abutment (Existing Alignment)	4 953 390.3 (44.716495°)	380 729.8 (-75.541615°)	100.3	9.5
162-22-02	North Abutment (Existing Alignment)	4 953 452.5 (44.717059°)	380 694.2 (-75.542054°)	101.0	10.8
162-22-03	Pier (Proposed)	4 953 424.4 (44.716803°)	380 726.7 (-75.541649°)	94.3	4.4

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. The boreholes were advanced to depths ranging from 4.4 m to 10.8 m (base elevation 90.8 m to 89.8 m). Coring was required to advance the boreholes through cobbles and boulders in the glacial till and into the underlying bedrock. A standpipe piezometer was installed in Borehole 166-22-02 to allow for measurement of the groundwater level after drilling. The details for the standpipe piezometer are illustrated on the 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, Boreholes 166-22-01 and 166-22-03 were decommissioned in general accordance with MOE requirements (O.Reg. 903, as amended) and capped with cold patch asphalt to reinstate the pavement surface. The standpipe piezometer at Borehole 166-22-02 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. One Unconfined Compressive Strength (UCS) Test was conducted on a recovered core sample from Borehole 166-22-02.

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 glacial till composed of silty sand to sandy silt. The glacial till is, in turn, underlain by dolostone bedrock.

5.1 Surficial Pavement

Boreholes 166-22-01 and 166-22-02 were advanced through the Merwin Lane road surface and encountered 100 mm of asphalt overlying 100 mm to 150 mm of concrete. Borehole 166-22-03 was advanced through the Highway 401 median and encountered 150 mm of asphalt.



5.2 Fill

Upper Sand and Gravel Fill

Sand and gravel fill containing some fines was encountered beneath the pavement structure at all borehole locations. The granular fill layer ranged in thickness from about 0.8 m at the Highway 401 median (Borehole 166-23-03) to between 1.3 m and 1.4 m along the existing Merwin Lane (Boreholes 166-23-01 and 166-23-02). SPT N-values in the sand and gravel ranged from 31 to greater than 100 blows per 0.3 m of penetration, indicating a generally dense relative density.

The recorded moisture content of three samples of the sand and gravel fill ranged from 3 to 11%. The results of gradation analyses completed on two samples of the sand and gravel fill 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	36 – 44
Sand	41 – 48
Silt	15 – 16
Clay	

Embankment Fill

Beneath the sand and gravel fill in Borehole 166-22-02, a layer of sand fill containing some fines and approximately 3.8 m thick was encountered (base elevation 95.5 m). SPT N-values in the sand fill ranged from 20 to 54 blows per 0.3 m of penetration, indicating a compact to very dense relative density.

The recorded moisture content of samples of the sand fill ranged from 5 to 14%. The results of a gradation analysis completed on a sample of the sand fill are illustrated on Figure C2 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	0
Sand	89
Silt	11
Clay	

Beneath the sand and gravel fill in Borehole 166-22-01, a layer of silty sand fill containing some gravel and plastic fines was encountered. The silty sand fill was 2.3 m thick (base elevation 96.5 m). SPT N-values in the silty sand fill ranged from 11 to 36 blows per 0.3 m of penetration,



indicating a compact to dense relative density. The recorded moisture content of samples of the silty sand fill ranged from 8 to 10%.

5.3 Silty Sand (SC-SM) to Sandy Silt (ML) Glacial Till

A native deposit of glacial till was encountered below the embankment fill in Boreholes 166-22-01 and 166-22-02. The glacial till consists of silty sand to sandy silt with varying amounts of gravel and plastic fines. Cobbles and boulders were encountered within the deposit. The layer ranged in thickness from 1.9 m at Borehole 166-22-02 near the north abutment to about 2.1 m at Borehole 166-22-01 near the south abutment (base elevations of 93.6 m and 94.4 m, respectively). SPT N-values measured in the glacial till ranged from 18 to greater than 100 blows per 0.3 m of penetration, indicating a compact to dense relative density. SPTs met refusal in the deposit in Borehole 166-22-01 and could represent a cobble or a boulder within the layer. Coring was required to advance through the deposit at this location.

The recorded moisture content of samples of the glacial till ranged from 8 to 21%. The results of gradation analyses completed on two samples of the glacial till are illustrated on Figure C3 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	9 – 10	
Sand	24 – 45	
Silt	45	54
Clay		13

5.4 Bedrock

Bedrock was proven by coring in all three boreholes. At the boreholes put down through the existing Merwin Lane embankments, the depth to bedrock was 7.4 m and 5.9 m near the north and south abutments (elevations 93.6 m and 94.4 m), respectively. At the pier, the bedrock was encountered at a depth of 1.0 m below the existing Highway 401 median grade (elevation 93.3 m).

Bedrock outcrops near the existing right-of-way limits (likely exposed during the construction of Highway 401 and the existing Merwin Lane underpass) are visible in the vicinity of the proposed replacement structure abutment footprints. Point elevations of the exposed bedrock outcrop surfaces were obtained by Thurber personnel with a Trimble Catalyst DA1 antenna with centimeter accuracy during a site visit in April 2023.

South of Highway 401, the exposed bedrock was observed at elevations as high as 95.0 m, near the existing south abutment, and generally sloped nominally down to the east to elevations in the order of 93.1 m. North of Highway 401, the exposed bedrock was observed at elevations as high as 94.9 m, near the existing north abutment, and generally sloped nominally down to the east.



The natural bedrock surface may have been lowered, where exposed, during construction of the existing Highway 401 corridor.

The bedrock encountered consists of moderately weathered to fresh, fine- to medium-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.

Based on the RQD, the bedrock quality is classified as very poor to excellent (CFEM, 2006). The result of an unconfined compressive strength test (UCS) was 193 MPa, indicating that the tested sample of bedrock is very strong (CFEM, 2006). The results of the UCS testing are included in Appendix C.

Table 5-1: Bedrock Details

Parameter	Range
Total Core Recovery (TCR), %	45 – 100
Solid Core Recovery (SCR), %	21 – 91
Rock Quality Designation (RQD), %	13 – 91
Fracture Index (fractures per 0.3 m) ⁽¹⁾	0 – >10
Unconfined Compressive Strength (UCS) ⁽²⁾ , MPa	193

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

(2) Samples tested from Boreholes 166-22-02

5.5 Groundwater

A standpipe piezometer was installed in Borehole 166-22-02 to allow for measurement of the stabilized groundwater level. The measured groundwater levels are summarized in Table 5-2.

Table 5-2: Groundwater Level Observations

Borehole No.	Bottom of Screen Elev. (m)	Screened Unit	Depth (mbgs) ¹	Groundwater Elevation (m)	Date of Measurement
166-22-02	93.8	Glacial Till	6.2	94.8	December 18, 2022
			7.2	93.8	April 26, 2023

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.



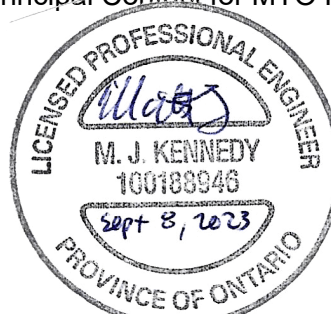
6 MISCELLANEOUS

The borehole locations were selected by Thurber relative to existing site features. The as-drilled locations and ground surface elevations of the boreholes were surveyed by Thurber following completion of the field program. The elevation survey of the boreholes was carried out with reference to geodetic elevation benchmarks provided by the MTO or relative to structure feature elevations provided on as-built drawings. Eastern Ontario Diamond Drilling of Hawkesbury, Ontario supplied and operated the drilling equipment and carried out the drilling, soil sampling, in-situ testing, and borehole decommissioning. Traffic control and water supply were provided by T.G. Carroll Cartage Limited of Carp, Ontario.

The field work was supervised on a full-time basis by Ibrahim Khan, E.I.T., under the direction of Katya 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 factual data and preparation of this report was carried out by A. de Oliveira, E.I.T and Matt Kennedy, P.Eng. The report was reviewed by Paul Carnaffan, P.Eng. and Fred Griffiths, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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PART 2. ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 INTRODUCTION

This report presents the interpretation of the factual data obtained from a preliminary foundation investigation and a desktop review of the available subsurface information conducted by Thurber for the replacement of the existing Highway 401 underpass at Merwin Lane, located approximately 1.5 km west of the town of Prescott, Ontario 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

Built in 1965, the existing structure carries two lanes of Merwin Lane traffic over Highway 401. It is a four-span, reinforced cast-in-place concrete structure with a total length of 64 m, a width of 10 m, and an approximate skew from perpendicular to Highway 401 of 10 degrees. It is noted that for project orientation purposes, Highway 401 will be referred to herein as oriented east-west and Merwin Lane oriented north-south.

The original General Arrangement drawing (Drawing TWP 26-166-01-A) indicates that the three piers are supported on pairs of spread footings founded on bedrock at each pier, with undersides



at elevations ranging from about 93.2 m to 93.4 m. The north and south abutments are indicated to each be supported on two rows of H-piles driven to bedrock and pile caps perched within the embankments with undersides at 97.9 m and 97.3 m, respectively. In the area of the structure the Highway 401 median is more than 18 m wide from shoulder to shoulder. Structural rehabilitation of the abutments walls and superstructure was most recently carried out as part of Contract No. 2014-4002.

7.2 Proposed Structure

The preliminary design of the Merwin Street interchange to accommodate the proposed widening of Highway 401 includes realignment of Merwin Lane approximately 12.5 m to the east at the existing structure. It is understood that the replacement overpass structure is to be a two-span structure, roughly the same width as the existing structure, with the centre pier constructed in the Highway 401 median.

The grade of Merwin Lane is anticipated to be increased by up to about 1.1 m, resulting in approach embankments heights anticipated to be on the order of 6 to 7 m.

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 0.22 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 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).

At the boreholes put down through the embankment fill (Boreholes 166-22-01 and 166-22-02), the average N_{60} values in the glacial till below the undersides of the pile caps (Elevations 97.7 m and 97.3 m at the north and south pile caps, respectively) ranged from 18 to greater than 50 blows per 0.3 m of penetration. Based on the average N_{60} values recorded, and the typical ranges of shear wave velocity encountered in similar deposits of glacial till, a Seismic Site Class C may be considered for preliminary design. It should be noted that the thickness, composition, and density of the new embankment fill beneath perched abutment pile caps will influence the design seismic site class.

The bedrock at the site is anticipated to have a shear wave velocity of 760 m/s or greater (Site Class B) but in the absence of confirmatory field measurements, a Site Class C should be assumed for preliminary design.

8.3 Seismic Performance Category

In consideration of the Site Class C spectral values for the site and the designated *Major Route* importance category, the bridge structure would fall into Seismic Performance Category 2, regardless of the fundamental period of the bridge, as per Section 4.4.4 (Table 4.10) of the CHBDC.



8.4 Liquefaction Potential

The susceptibility of the embankment fill and glacial till at the site to experience liquefaction was assessed using the SPT data following the simplified method for cohesionless soil as outlined in Boulanger and Idriss (2014)ⁱⁱ. The cohesionless soil at the site is not considered to be susceptible to liquefaction.

9 STRUCTURE FOUNDATION ALTERNATIVES

9.1 Foundation Alternatives

Based on the subsurface conditions at the site, both shallow and deep foundations have been considered for the replacement of the Merwin Lane underpass. The foundation alternatives are presented below and evaluated from a geotechnical perspective in terms of their respective advantages, disadvantages, risks, and consequences. The evaluation is summarized in the table provided in Appendix G.

- Spread Footings

Spread footings are considered feasible for support of the replacement structure, provided they are founded on the dense glacial till or bedrock. Spread footings perched in the embankment fill would require removal of existing fill and replacement with an engineered fill pad. Spread footings bearing on glacial till at the abutments and bedrock at the pier may result in some differential settlement. The ground water table is anticipated to be within the glacial till, up to several metres above the bedrock surface and therefore some groundwater control would be required during construction.

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

- Driven Steel H-piles

Steel H-piles driven through the new approach embankments to support “perched” abutments may also be considered. Driven H-piles will typically reduce the volumes of excavation required when compared to shallow foundations. The use of H-Piles with reinforced tips is the option with the least risk given the cobbles and boulders in the till layer above the bedrock observed at this site.

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

- 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 groundwater level within the glacial till deposit will pose additional construction challenges resulting from potential unbalanced hydraulic pressure heads and caisson base boiling when drilling through the predominantly sand



and silt 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 foundations perspective to support the new bridge.

9.2 Recommended Foundation

Based on an evaluation of the foundation alternatives presented above and in Appendix G, the recommended foundation approach from a geotechnical perspective is to support the new bridge abutments on steel H-piles driven to bedrock and the pier on spread footings.

9.3 Construction Methodology

It is assumed that staging areas for the bridge construction will be set up east of the existing bridge alignment and will be accessible from the existing Highway 401 and Merwin Lane to minimize conflict with the Highway 401 and existing Merwin Lane traffic. It is anticipated that the majority of the construction of the new approach embankments and structure can take place while maintaining traffic flow on existing Merwin Lane. However, consideration of the staging requirements and impacts from the construction of the new structure and removal of the existing structure should be considered at subsequent design stages.

10 FOUNDATION DESIGN RECOMMENDATIONS

Approximate key elevations are as follows:

- | | |
|--|--------------------|
| • Existing Highway 401 grade | 94.2 m |
| • Existing Merwin Lane grade | 100.3 m to 101.0 m |
| • Proposed Merwin Lane grade | 101.3 m to 102.1 m |
| • Underside of perched abutment pile caps, assumed | 99.4 m |
| • Underside of pier foundation, assumed | 93.0 m |
| • Groundwater elevation | 94.8 m |
| • Top of dolostone bedrock | 93.2 to 94.4 m |

10.1 Spread Footings

Shallow spread footings should be founded on the dense glacial till or directly on the dolostone bedrock, below any existing fill.

At the pier, bedrock was encountered directly beneath the Highway 401 grade fill at about Elevation 93.2 m. Near the proposed north abutment, SPTs carried out at Borehole 166-22-02 indicated that the glacial till deposit is dense below about Elevation 94.8 m. Near the proposed south abutment, SPTs carried out in Borehole 166-22-01 in the glacial indicated a relative density ranging from very dense to compact at the bottom of the deposit. The following table provides the

maximum (highest) founding elevations recommended for the preliminary design of spread footings.

Table 10-1: Spread Footing Estimated Maximum Founding Elevations

Foundation Element	Borehole	Founding Stratum	Highest Footing Founding Elev. (m)
North Abutment	166-22-02	Dense Glacial Till	95.0
Pier	166-22-03	Bedrock ¹	93.2
South Abutment	166-22-01	Compact to Dense Glacial Till	97.2

Note: 1) Frost protection is not required for footings founded on bedrock.

10.1.1 Geotechnical Resistance

Spread footings can be founded directly on the properly prepared glacial till or bedrock. Spread footings on glacial till should be provided with at least 1.6 m of frost cover or equivalent insulation. Subgrade preparation should be as described in Section 10.1.2 and will include removal of unsuitable materials. Frost protection requirements are provided in Section 10.4. Dewatering may be required to prepare the subgrade, place the bedding material and construct the foundations in the dry (Section 11.2).

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

- Factored geotechnical resistance at ULS 750 kPa
- Factored geotechnical resistance at SLS 550 kPa

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

Footings bearing directly on the bedrock surface may be designed with a factored geotechnical resistance of 5,000 kPa at ULS. The SLS condition will not govern design for footings founded on bedrock.

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

The factored geotechnical resistances include the following factors:

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



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

Resistance to lateral forces/sliding resistance between the concrete and the underlying Granular A bedding fill (Section 10.1.2) should be evaluated in accordance with the CHBDC assuming an unfactored coefficient of friction of 0.5. Resistance to lateral forces/sliding resistance between the concrete and the underlying bedrock should be evaluated in accordance with the CHBDC assuming an unfactored coefficient of friction of 0.7. A resistance factor of 0.8 (as per CHBDC Table 6.2) should be used to estimate the sliding resistance between the concrete and Granular A or bedrock.

10.1.2 Subgrade Preparation

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

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

For spread footings on bedrock, the top surface of the bedrock should be stripped of all overburden and be cleaned. Inspection should be carried out to confirm that the bedrock conditions, as exposed at the founding level, are consistent with the design assumption. All shattered and loosened rock fragments should be removed from the footprint of the footing and replaced with mass concrete fill with a structural strength of 30 MPa, where necessary. Where bedrock is lower than anticipated, mass concrete may be placed to raise the subgrade to the design footing level.

10.2 Driven Steel H-Pile Foundations

10.2.1 Axial Geotechnical Resistance

The new abutments may be perched within the embankment fill and founded on steel H-piles end-bearing on the bedrock. The estimated pile tip elevations based on piles reaching refusal at the bedrock surface are summarized in Table 10-2.

Table 10-2: Estimated Pile Tip Elevations

Foundation Element	Approximate Underside of Pile Cap Elevation¹ (m)	Estimated Pile Tip Elevation (top of bedrock) (m)	Estimated Pile Length (m)
South Abutment	99.4	94.4	5.0
North Abutment	99.4	93.6	5.8

Notes: (1) Assumed, based on proposed final grade and frost protection requirements

The factored geotechnical axial resistance at ULS of Grade 350W HP 310x110 steel piles driven to refusal on or in the dolostone bedrock can be taken as 2,500 kN. The SLS condition will not govern for piles founded on bedrock.

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)

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 steel H-piles supporting the existing structure must be checked.

As the piles are to be driven to bedrock. The pile tips of the new piles at the site should be protected from damage during driving with pile tip protection from an approved manufacturer such as Titus Steel (standard H-Point) or approved equivalent. The appropriate pile driving note is "Piles to be driven to bedrock."

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

10.2.2 Downdrag

Downdrag forces (negative skin friction) acting upon driven steel H-piles are expected to be negligible as the anticipated settlement resulting from embankment construction is expected to take place during the construction period, prior to driving of piles (see Section 10.7).



10.2.3 Uplift Resistance

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

The factored geotechnical tensile resistance for a single HP 310x110 pile at either abutment may be taken as 140 kN under static conditions and 465 kN under seismic conditions. These values include the following factors:

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

10.2.4 Lateral Resistance of Piles

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:

$$\begin{aligned} k_s &= n_h \cdot z / D && (\text{kN/m}^3) \\ p_{ult} &= 3 \cdot \gamma' \cdot z \cdot K_p && (\text{kPa}) \end{aligned}$$

For cohesive soils:

$$\begin{aligned} k_s &= 67 \cdot c_u / D && (\text{kN/m}^3) \\ p_{ult} &= 0 && (\text{kPa}) \text{ at the top of the pile, increasing linearly to} \\ &= 9 \cdot c_u && (\text{kPa}) \text{ at } z \geq 3 \cdot D \end{aligned}$$

where:

$$\begin{aligned} z &= \text{depth of embedment along pile (m)} \\ D &= \text{pile width or diameter (m)} \\ n_h &= \text{coefficient related to soil density (kN/m}^3\text{)} \\ c_u &= \text{undrained shear strength (kPa)} \\ \gamma' &= \text{effective unit weight (kN/m}^3\text{)} \\ K_p &= \text{coefficient of passive lateral earth pressure} \end{aligned}$$

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

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

Table 10-3: Soil Parameters for Lateral Pile Design

Soil Type	γ' (kN/m ³)	n_h (kN/m ³)	C_u (kPa)	K_p
Existing/New ¹ Fill	20 (above WT)	6,000	N/A	3.0
Glacial Till	10 (below WT)	8,000	N/A	3.0

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.

10.3 Caisson Foundations

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

Given the risk of the till layer sloughing, the caisson construction method should include use of temporary or permanent casings (liners) sealed into the bedrock. Using a temporary casing that is extracted during the concrete pour to reduce material costs is feasible, if the caissons are installed in combination with drilling fluid to maintain the stability of the side walls. The use of temporary casing would require careful control of the concrete level. Alternatively, caisson casings may be left in place as permanent liners to reduce the potential for disturbance of the



soil-concrete interface that may occur during removal of temporary liners. Permanent liners would assist in maintaining the integrity of the concrete caisson by reducing the risk of infiltration of soil or water prior to concrete curing. Ultimately, the contractor will be responsible for selecting the construction means and methods based on cost and risk considerations.

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

10.3.1 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 a 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 ULS (Tension) (kN)	Factored SLS (Compression) (kN)
915	15,000	10,000	will not govern
1220	11,000	8,000	will not govern

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

10.3.2 Downdrag

Downdrag forces (negative skin friction) acting upon caissons are expected to be negligible as the anticipated settlement resulting from embankment construction is expected to take place during the construction period, prior to caisson installation (see Section 10.7).

10.3.3 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 10.2.4.



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. It is not necessary to found spread footings directly on bedrock at a depth below frost penetration. Caisson foundations socketed into bedrock and extending vertically up to the pier cap also 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 vertical walls and a horizontal back-slope. The provided earth pressure coefficients are calculated using a PGA of 0.22 g, based on a Seismic Site Class C, and a 2% probability of exceedance in 50 years (Geological Survey of Canada – Fifth Generation).

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

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

where:

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

Table 10-6 Combined Static and Seismic Earth Pressure Coefficients

Parameter	OPSS Granular A & B Type II	OPSS Granular B Type I & Existing Fill	OPSS SSM
Soil Unit Weight, kN/m ³ , γ	22.8	20.0	20.0
Angle of Internal Friction, ϕ	35°	32°	30°
Non-Yielding Wall			
Dynamic Active Earth Pressure Coefficient, K_{AE}	0.41	0.46	0.49
Yielding Wall			
Dynamic Active Earth Pressure Coefficient, K_{AE}	0.33	0.38	0.40

10.6 Embankment Stability

Based on the available original structure drawings and observations during the 2022 field investigation, the grade of the current Merwin Lane lanes ranges from about 100.3 m to 101.0 m in the area of the Highway 401 underpass with embankments of up to about 6 m in height and sloped at roughly 2H:1V.

The embankments to be constructed for the proposed new alignment will shift the centerline of the roadway approximately 12.5 m to the east and are anticipated to be roughly 7 m in height. For preliminary geotechnical design recommendations, a number of assumptions have been made that are consistent with MTO's standard highway design practices:

- Topsoil and other deleterious material will be removed from within the footprint prior to constructing the new embankment.
- Where new fill is placed against an existing embankment slope or on a sloping ground surface steeper than 3H:1V, the existing slope will be benched (OPSD 208.010).
- The embankment will be constructed using granular fill meeting the minimum requirements of Select Subgrade Material (SSM) as per OPSS.PROV 1010.

- Conventional (non-reinforced) granular fill embankments will be constructed with side slopes no steeper than 2H:1V.
- Granular fill embankments greater than 7 m in height will be provided with a 2 m wide mid-height berm.
- Permanent drainage and erosion protection will be provided for all granular embankment slopes.

Table 6.2 of the CHBDC for embankment fills with a *typical* degree of site understanding and a *typical* consequence level (a Consequence Factor, Ψ , of 1.0) generates minimum 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 7.1 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 C) design PGA values of 0.22 g and 0.084 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 highest/critical embankment slope, just behind the north abutment. Embankment slope stability was evaluated using GeoStudio 2021 Slope/W software for limit equilibrium analysis. Input parameters for the analyses are based on

the findings of the 2022 boreholes and the results of laboratory testing. The following additional parameters were used in the analysis:

- Estimated soil stratigraphy based on the existing ground surface contours and nearest boreholes;
- Embankment maximum fill height of 7.1 m;
- For analysis, a seismic event with a return period 2,475 years site adjusted PGA value of 0.11 g, equal to $\frac{1}{2}$ of the site adjusted PGA value (0.22 g) was used, as per Section 4.4.3.3 of the CHBDC; and,
- A traffic surcharge of 17 kPa was applied as a temporary load.

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

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

Table 10-7 Slope Stability Analysis Results

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

All of the static results presented in 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 exceeds the target Factor of Safety of 1.1 for seismic design. However, it is noted that some embankment displacement can occur where the pseudo-static Factor of Safety is less than 1.3 as is the case for the 1 in 2,475 year seismic event (Figure H3). Additional analyses were carried out for that case to determine if performance criteria would be met for the Major Route geotechnical systems inside and outside the bridge interface zone. Pseudo-static analyses considering the 475-year earthquake event were completed and yielded a factor of safety of 1.3 for the west abutment at that location (Figure H4) indicating that the performance requirements would be met for that scenario.

10.7 Embankment Settlement

Based on the results of the current investigation, the existing embankments consist of sand and gravel overlying sand, underlain by up to about 2.5 m of glacial till. Settlement of the existing



embankments has likely occurred since the original embankment construction. The additional load imposed by the new embankment fill up to about 7.1 m high would produce additional elastic settlement in the underlying glacial till of as much as about 20 mm.

Some self-settlement of the new embankment fill will also take place. Embankments constructed of SSM fill will undergo approximately 35 to 70 mm of self settlement, and is expected to occur during embankment construction, provided approximately 95 percent compaction of the embankment fill is achieved.

11 CONSTRUCTION CONSIDERATIONS

11.1 Temporary Excavations

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

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

11.2 Surface and Groundwater Control

At the site, the natural groundwater level was measured to be within the glacial till, up to about 1.2 m above the bedrock surface at Elevations of 93.8 m to 94.8 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 bedding must be carried out in a dry and stable excavation.

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

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 Merwin Street Underpass (Site No. 16X-0166/B0) as part of the overall Preliminary Design and Environmental Assessment for the widening of Highway 401 from Highway 416 to Maitland Road. Additional foundation investigation will be required following the selection of the Technically Preferred Alternative (TPA). Additional field investigation should be carried out to provide additional foundation design input to the following:



- Shear wave velocity measurements below the foundation elements to confirm Seismic Site Classification.
- Testing of soil and groundwater samples from the site to determine degree of corrosiveness of the sub-surface environment and potential implications to steel and concrete elements in contact with the groundwater at the site.
- Additional borehole and laboratory test data designed to increase the degree of site and prediction model understanding to “high”. This would allow the use of more favourable geotechnical resistance factors in the analyses.

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

As noted in Section 10.2.1 above, the effects of construction vibrations on nearby structures and underground utilities will need to be assessed during detailed design. The need for vibration monitoring and settlement monitoring of nearby utilities and structures during construction should be evaluated on a site-specific and activity type 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 construction.



13 CLOSURE

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

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STATEMENT OF LIMITATIONS AND CONDITIONS

1. STANDARD OF CARE

This Report has been prepared in accordance with generally accepted engineering or environmental consulting practices in the applicable jurisdiction. No other warranty, expressed or implied, is intended or made.

2. COMPLETE REPORT

All documents, records, data and files, whether electronic or otherwise, generated as part of this assignment are a part of the Report, which is of a summary nature and is not intended to stand alone without reference to the instructions given to Thurber by the Client, communications between Thurber and the Client, and any other reports, proposals or documents prepared by Thurber for the Client relative to the specific site described herein, all of which together constitute the Report.

IN ORDER TO PROPERLY UNDERSTAND THE SUGGESTIONS, RECOMMENDATIONS AND OPINIONS EXPRESSED HEREIN, REFERENCE MUST BE MADE TO THE WHOLE OF THE REPORT. THURBER IS NOT RESPONSIBLE FOR USE BY ANY PARTY OF PORTIONS OF THE REPORT WITHOUT REFERENCE TO THE WHOLE REPORT.

3. BASIS OF REPORT

The Report has been prepared for the specific site, development, design objectives and purposes that were described to Thurber by the Client. The applicability and reliability of any of the findings, recommendations, suggestions, or opinions expressed in the Report, subject to the limitations provided herein, are only valid to the extent that the Report expressly addresses proposed development, design objectives and purposes, and then only to the extent that there has been no material alteration to or variation from any of the said descriptions provided to Thurber, unless Thurber is specifically requested by the Client to review and revise the Report in light of such alteration or variation.

4. USE OF THE REPORT

The information and opinions expressed in the Report, or any document forming part of the Report, are for the sole benefit of the Client. NO OTHER PARTY MAY USE OR RELY UPON THE REPORT OR ANY PORTION THEREOF WITHOUT THURBER'S WRITTEN CONSENT AND SUCH USE SHALL BE ON SUCH TERMS AND CONDITIONS AS THURBER MAY EXPRESSLY APPROVE. Ownership in and copyright for the contents of the Report belong to Thurber. Any use which a third party makes of the Report, is the sole responsibility of such third party. Thurber accepts no responsibility whatsoever for damages suffered by any third party resulting from use of the Report without Thurber's express written permission.

5. INTERPRETATION OF THE REPORT

- a) Nature and Exactness of Soil and Contaminant Description: Classification and identification of soils, rocks, geological units, contaminant materials and quantities have been based on investigations performed in accordance with the standards set out in Paragraph 1. Classification and identification of these factors are judgmental in nature. Comprehensive sampling and testing programs implemented with the appropriate equipment by experienced personnel may fail to locate some conditions. All investigations utilizing the standards of Paragraph 1 will involve an inherent risk that some conditions will not be detected and all documents or records summarizing such investigations will be based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated and the Client and all other persons making use of such documents or records with our express written consent should be aware of this risk and the Report is delivered subject to the express condition that such risk is accepted by the Client and such other persons. Some conditions are subject to change over time and those making use of the Report should be aware of this possibility and understand that the Report only presents the conditions at the sampled points at the time of sampling. If special concerns exist, or the Client has special considerations or requirements, the Client should disclose them so that additional or special investigations may be undertaken which would not otherwise be within the scope of investigations made for the purposes of the Report.
- b) Reliance on Provided Information: The evaluation and conclusions contained in the Report have been prepared on the basis of conditions in evidence at the time of site inspections and on the basis of information provided to Thurber. Thurber has relied in good faith upon representations, information and instructions provided by the Client and others concerning the site. Accordingly, Thurber does not accept responsibility for any deficiency, misstatement or inaccuracy contained in the Report as a result of misstatements, omissions, misrepresentations, or fraudulent acts of the Client or other persons providing information relied on by Thurber. Thurber is entitled to rely on such representations, information and instructions and is not required to carry out investigations to determine the truth or accuracy of such representations, information and instructions.
- c) Design Services: The Report may form part of design and construction documents for information purposes even though it may have been issued prior to final design being completed. Thurber should be retained to review final design, project plans and related documents prior to construction to confirm that they are consistent with the intent of the Report. Any differences that may exist between the Report's recommendations and the final design detailed in the contract documents should be reported to Thurber immediately so that Thurber can address potential conflicts.
- d) Construction Services: During construction Thurber should be retained to provide field reviews. Field reviews consist of performing sufficient and timely observations of encountered conditions in order to confirm and document that the site conditions do not materially differ from those interpreted conditions considered in the preparation of the report. Adequate field reviews are necessary for Thurber to provide letters of assurance, in accordance with the requirements of many regulatory authorities.

6. RELEASE OF POLLUTANTS OR HAZARDOUS SUBSTANCES

Geotechnical engineering and environmental consulting projects often have the potential to encounter pollutants or hazardous substances and the potential to cause the escape, release or dispersal of those substances. Thurber shall have no liability to the Client under any circumstances, for the escape, release or dispersal of pollutants or hazardous substances, unless such pollutants or hazardous substances have been specifically and accurately identified to Thurber by the Client prior to the commencement of Thurber's professional services.

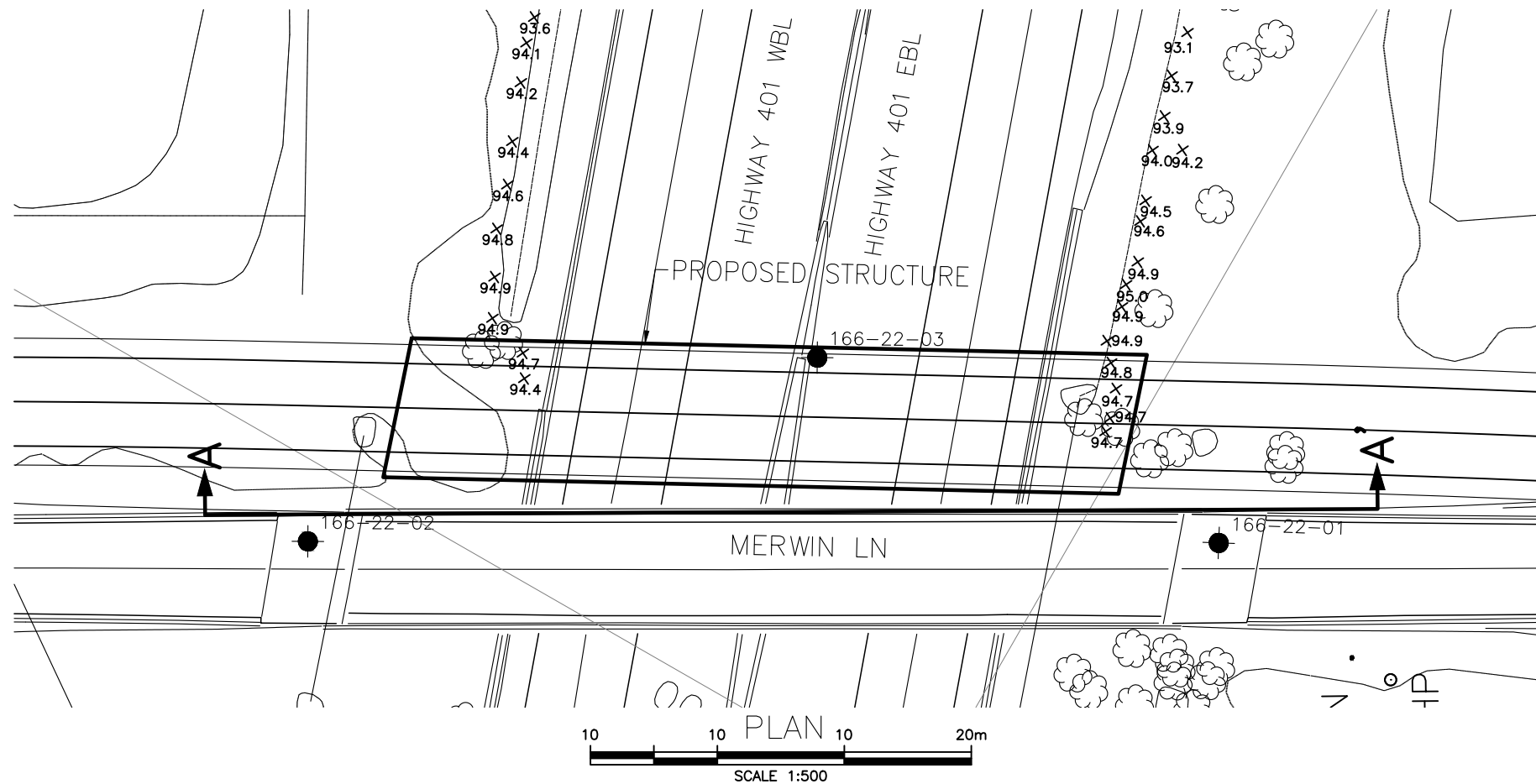
7. INDEPENDENT JUDGEMENTS OF CLIENT

The information, interpretations and conclusions in the Report are based on Thurber's interpretation of conditions revealed through limited investigation conducted within a defined scope of services. Thurber does not accept responsibility for independent conclusions, interpretations, interpolations and/or decisions of the Client, or others who may come into possession of the Report, or any part thereof, which may be based on information contained in the Report. This restriction of liability includes but is not limited to decisions made to develop, purchase or sell land.



Appendix A.

Borehole Location Plan and Stratigraphic Drawing



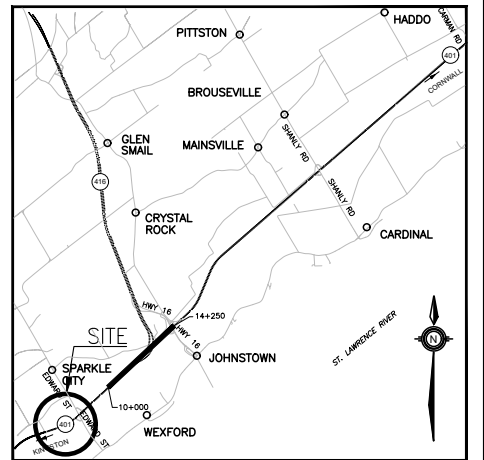
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AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



CONT No
GWP No 4024-20-00

HIGHWAY 401
MERWIN LANE UNDERPASS
BRIDGE REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

Ontario



KEYPLAN

LEGEND

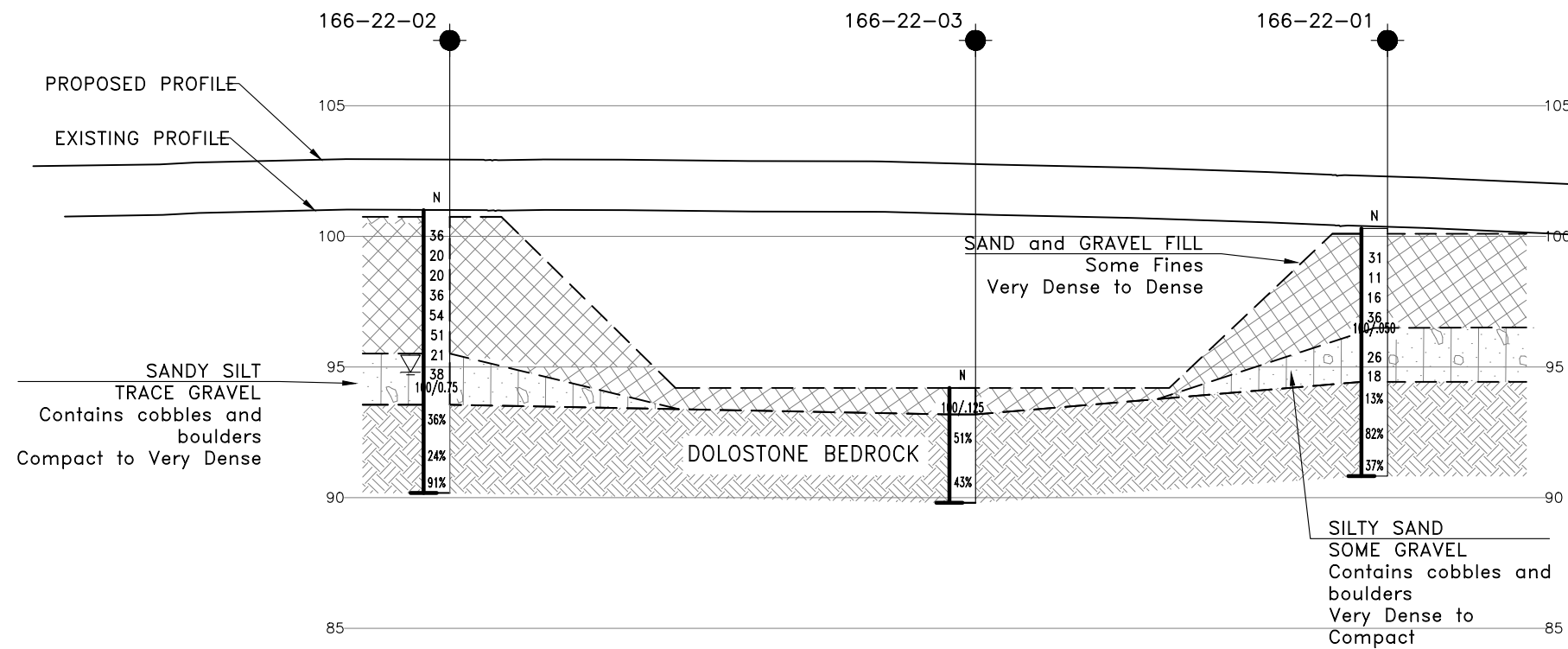
- Borehole (Current Investigation)
- Borehole (Previous Investigation)
- N Blows /0.3m (Std Pen Test, 475J/blow)
- CONE Blows /0.3m (60° Cone, 475J/blow)
- PH Pressure, Hydraulic
- Water Level
- Head Artesian Water
- Piezometer
- 90% Rock Quality Designation (RQD)
- X Exposed Bedrock Surface Point Elevation

NO	ELEVATION	NORTHING	EASTING
166-22-01	100.3	4 953 390.3	380 729.8
166-22-02	101.0	4 953 452.5	380 694.2
166-22-03	94.2	4 953 424.4	380 726.7
X	94.7	4 953 403.2	380 735.4
X	94.7	4 953 402.9	380 736.7
X	94.7	4 953 403.0	380 738.9
X	94.8	4 953 403.4	380 740.6
X	94.9	4 953 404.2	380 742.5
X	94.9	4 953 404.9	380 745.1
X	95.0	4 953 405.5	380 746.7
X	94.9	4 953 406.4	380 749.1

NOTES

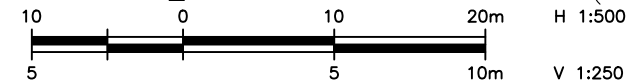
- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- This drawing is for subsurface information only. Surface details and features are for conceptual illustration.
- Coordinate system is MTM NAD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

GEOCRES No. 31B-110



NO	ELEVATION	NORTHING	EASTING
X	94.6	4 953 407.6	380 751.9
X	94.5	4 953 408.0	380 754.1
X	94.2	4 953 409.1	380 757.2
X	94.0	4 953 408.7	380 759.5
X	93.9	4 953 409.6	380 762.7
X	93.7	4 953 411.3	380 766.5
X	93.1	4 953 413.7	380 772.1
X	94.4	4 953 445.9	380 714.7
X	94.7	4 953 447.8	380 715.5
X	94.9	4 953 449.3	380 716.9
X	94.9	4 953 450.5	380 718.1
X	94.8	4 953 451.7	380 721.2
X	94.6	4 953 452.4	380 723.7
X	94.4	4 953 453.5	380 726.9
X	94.2	4 953 454.5	380 728.7
X	94.1	4 953 455.6	380 731.6
X	93.6	4 953 456.2	380 734.1

PROFILE ALONG C OF MERWIN LANE (A-A)



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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).

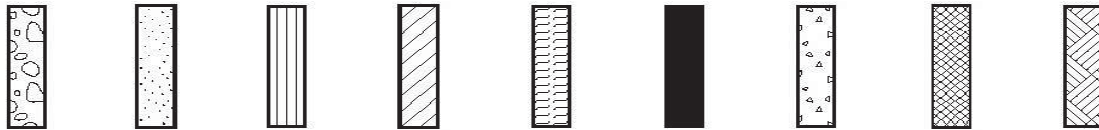
DYNAMIC CONE PENETRATION TEST (DCPT):

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 166-22-01

1 OF 2

METRIC

GWP# 4024-20-00 LOCATION Lat: 44.716495°, Long: -75.541615° Merwin Lane, Augusta, MTM z9: N 4 953 390.3 E 380 729.8 ORIGINATED BY IK
 HWY 401 BOREHOLE TYPE CME 55 Truck Mount / NW Casing / NQ Coring COMPILED BY AO
 DATUM Geodetic DATE 2022.11.21 - 2022.11.21 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
100.3	Ground Surface												
0.0	ASPHALT (100 mm)		1	NQ	-								
0.1	CONCRETE (100 mm)												
0.2	SAND and GRAVEL, some fines Dense Brown FILL		1	SS	31								
98.8													
1.5	SILTY SAND, some gravel Contains plastic fines Compact to dense Brown FILL		2	SS	11								
			3	SS	16								
			4	SS	36								
96.5			5	SS	100/50mm								
3.8	SILTY SAND (SC-SM), some gravel to gravelly Contains cobbles and boulders Contains plastic fines Very dense to compact Brown GLACIAL TILL		2	NQ	-								
			6	SS	26								
			7	SS	18								
94.4													
5.9	DOLOSTONE BEDROCK Interbedded with Sandstone Contains quartz inclusions Slightly weathered to fresh Fine to medium grained Grey Medium bedded Very strong		1	RUN	-								
			2	RUN	-								
			3	RUN	-								
90.8													
9.5	End of Borehole												

Continued Next Page

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

DOUBLE LINE 29381 BOREHOLE LOGS REPLACEMENT SITES.GPJ 2012TEMPLATE(MTO).GDT 9-5-23

METRIC

[illegible]

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 166-22-02

1 OF 2

METRIC

GWP# 4024-20-00 LOCATION Lat: 44.717059°, Long: -75.542054°
Merwin Lane, Augusta, MTM z9: N 4 953 452.5 E 380 694.2 ORIGINATED BY IK
HWY 401 BOREHOLE TYPE CME 55 Truck Mount / NW Casing / NQ Coring COMPILED BY AO
DATUM Geodetic DATE 2022.11.22 - 2022.11.22 CHECKED BY

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										
101.0	Ground Surface							20	40	60	80	100						
0.0	ASPHALT (100 mm)		1	NQ	-			20	40	60	80	100						
100.7	CONCRETE (150 mm)							20	40	60	80	100						
0.3	SAND and GRAVEL, some fines Dense to compact Brown FILL		1	SS	36			20	40	60	80	100						
99.3								20	40	60	80	100						
1.7	SAND, some fines Compact to very dense Brown FILL		2	SS	20			20	40	60	80	100						
			3	SS	20			20	40	60	80	100						
			4	SS	36			20	40	60	80	100						
			5	SS	54			20	40	60	80	100						
			6	SS	51			20	40	60	80	100						
	- Gravelly below a depth of 5.3 m							20	40	60	80	100						
95.5			7	SS	21			20	40	60	80	100						
5.5	SANDY SILT (ML), trace gravel Contains plastic fines Inferred cobbles and boulders Compact to very dense Grey GLACIAL TILL		8	SS	38			20	40	60	80	100						
			9	SS	100/ 75mm			20	40	60	80	100						
93.6								20	40	60	80	100						
7.4	DOLOSTONE BEDROCK Interbedded with Sandstone Moderately weathered to fresh Fine to medium grained Grey Medium bedded Very strong		1	RUN	-			20	40	60	80	100						
			2	RUN	-			20	40	60	80	100						

Continued Next Page

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

DOUBLE LINE 29381 BOREHOLE LOGS REPLACEMENT SITES.GPJ 2012TEMPLATE(MTO).GDT 9-5-23

METRIC

[illegible]

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 166-22-03

1 OF 1

METRIC

GWP# 4024-20-00 LOCATION Lat: 44.716803°, Long: -75.541649° Merwin Lane, Augusta, MTM z9: N 4 953 424.4 E 380 726.7 ORIGINATED BY IK
 HWY 401 BOREHOLE TYPE CME 55 Truck Mount / HSA/ NW Casing / NQ Coring COMPILED BY AO
 DATUM Geodetic DATE 2022.12.07 - 2022.12.07 CHECKED BY

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			20	40	60	80	100		
94.3	Ground Surface													
0.0	ASPHALT (150 mm)													
0.2	GRAVELLY SAND, some fines													
93.9	Brown													
0.4	FILL (BASE)													
93.3	SAND and GRAVEL, some fines		1	SS	100/									
1.0	Very dense				125mm									
	Brown													
	FILL													
	DOLOSTONE BEDROCK													
	Interbedded with Sandstone													
	Moderately weathered to fresh													
	Fine to medium grained													
	Grey													
	Medium bedded													
	Very strong													
			1	RUN	-									
			2	RUN	-									
89.9														
4.4	End of Borehole													
	A representative open-hole groundwater level measurement was not obtained due to the introduction of water during drilling.													

DOUBLE LINE 29381 BOREHOLE LOGS REPLACEMENT SITES.GPJ 2012TEMPLATE(MTO).GDT 9-5-23



Appendix C.

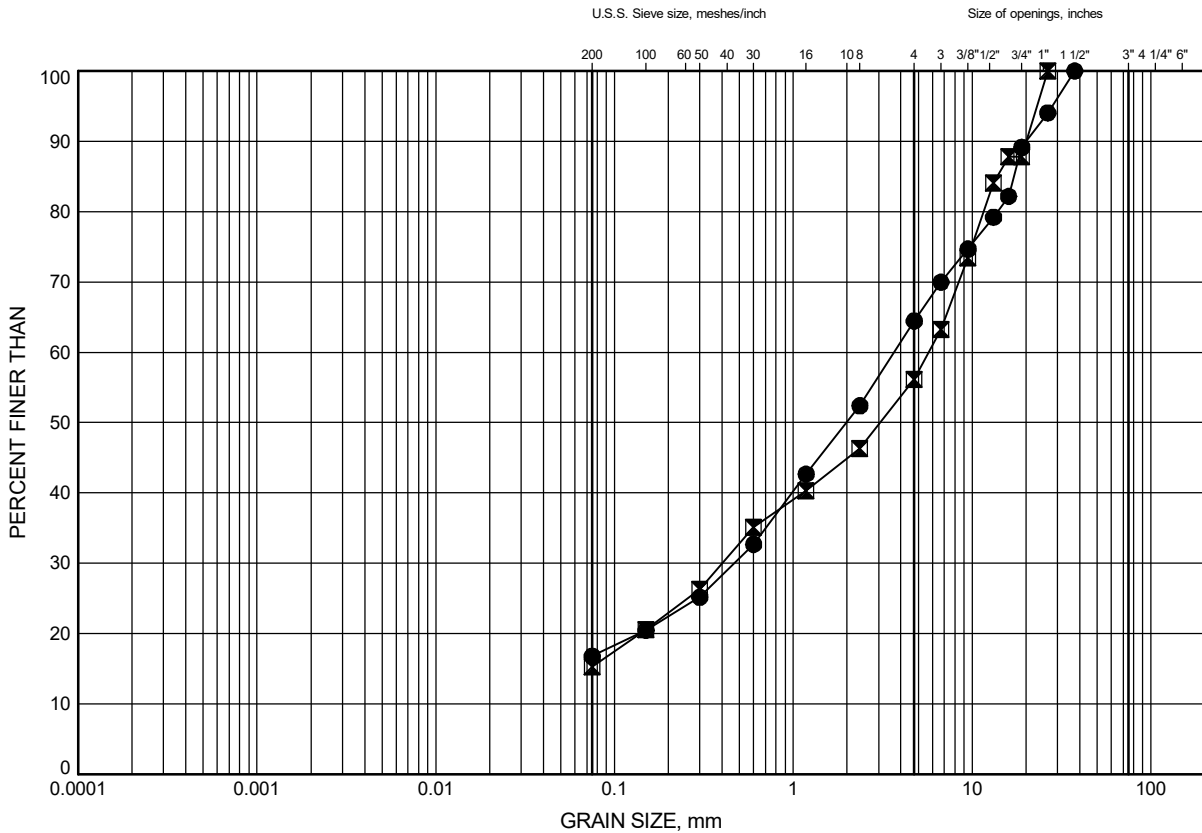
Laboratory Testing



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

GRAIN SIZE DISTRIBUTION

FILL: Sand and Gravel



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	166-22-01	1.1	99.2
⊠	166-22-03	0.9	93.3

Date March 2023

GWP# 4024-20-00

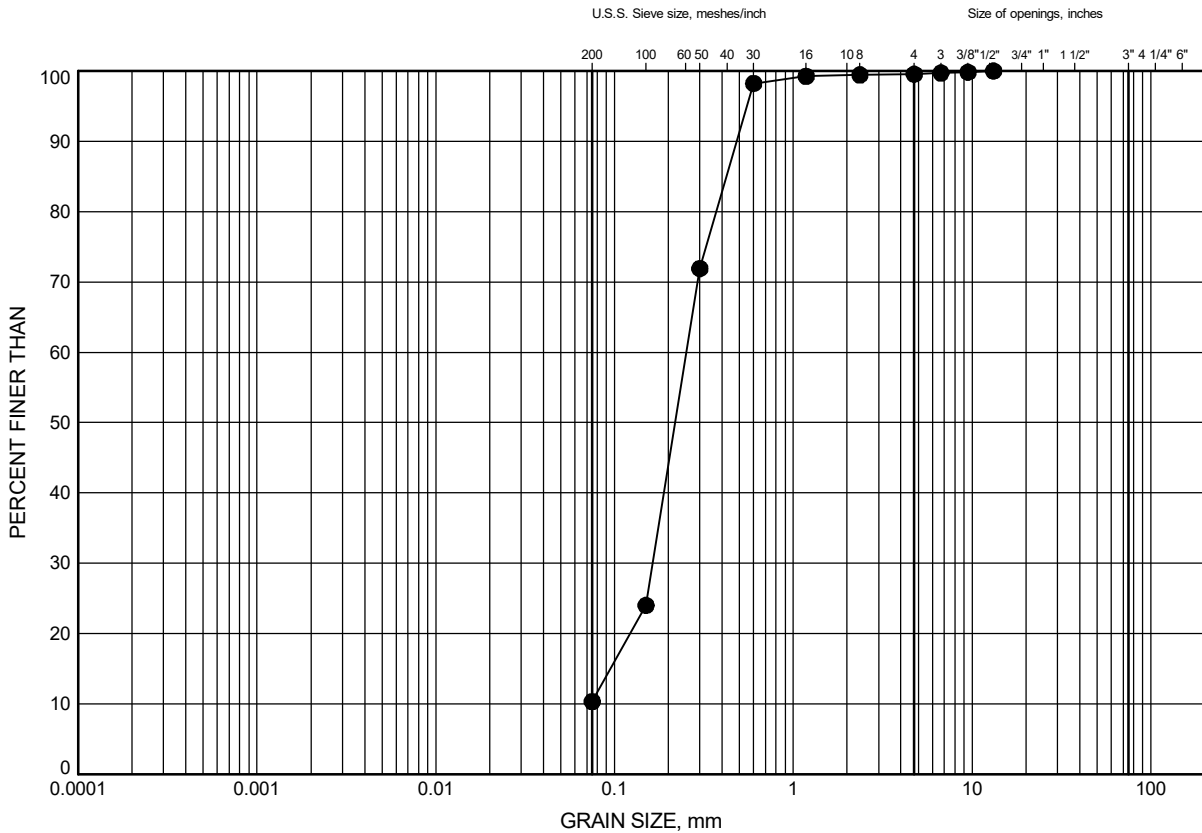


Prep'd RH

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)
●	166-22-02	4.1	96.9

Date March 2023

GWP# 4024-20-00

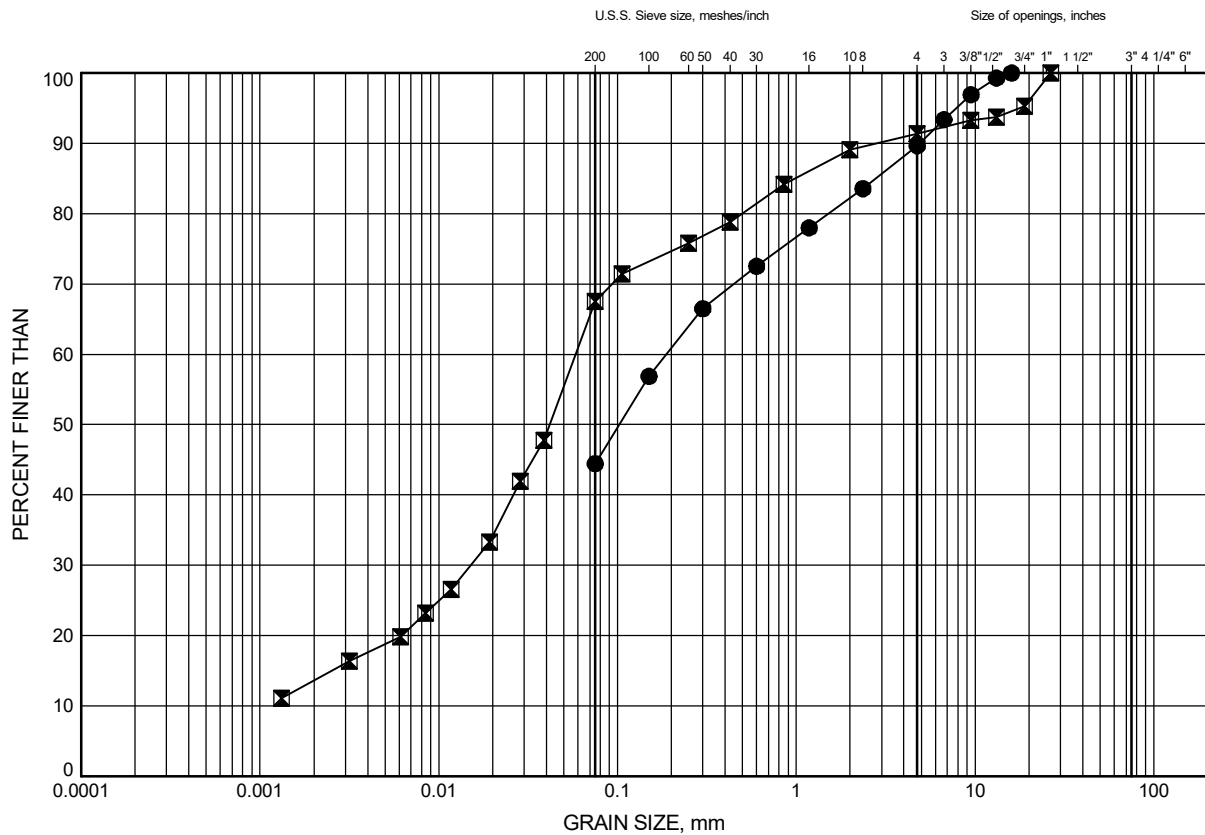


Prep'd RH

Chkd. AO

GRAIN SIZE DISTRIBUTION

GLACIAL TILL: Silty Sand (SC-SM) to Sandy Silt (ML)



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	166-22-01	4.9	95.4
⊠	166-22-02	6.5	94.5

Date March 2023

GWP# 4024-20-00



Prep'd RH

Chkd. AO



Appendix C.2

UCS Test Results



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

May 2, 2023
File: 122410864

Client: Thurber Engineering, File #29381

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

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

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

Sincerely,

Stantec Consulting Ltd.

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



Appendix C.3

Bedrock Core Photographs

Borehole 166-22-01

RUNS 1 and 2

Depth 5.9 m to 8.6 m

Elevation 94.4 m to 91.7 m

Dry Sample

Run 1 Start
elev. 94.4 m

Run 1 End
elev. 93.3 m



Run 2 Start
elev. 93.3 m



Run 2 End
elev. 91.7 m



THURBER ENGINEERING LTD.

Highway 401 Underpass at Merwin Lane
(Site No. 16X-0166)
Assignment No. 4019-E-0010.2, GWP 4024-20-00

BH 166-22-01
Project No.: 29381

Borehole 166-22-01

RUNS 1 and 2

Depth 5.9 m to 8.6 m

Elevation 94.4 m to 91.7 m

Wet Sample

Run 1 Start
elev. 94.4 m

Run 1 End
elev. 93.3 m



Run 2 Start
elev. 93.3 m



Run 2 End
elev. 91.7 m

Borehole 166-22-01

RUN 3

Depth 8.6 m to 9.5 m

Elevation 91.7 m to 90.8 m

Dry Sample

Run 3 Start
elev. 91.7 m



Run 3 End
elev. 90.8 m



THURBER ENGINEERING LTD.

Highway 401 Underpass at Merwin Lane
(Site No. 16X-0166)
Assignment No. 4019-E-0010.2, GWP 4024-20-00

BH 166-22-01
Project No.: 29381

Borehole 166-22-01

RUN 3

Depth 8.6 m to 9.5 m

Elevation 91.7 m to 90.8 m

Dry Sample

Run 3 Start
elev. 91.7 m



Run 3 End
elev. 90.8 m

Borehole 166-22-02

RUN 1

Depth 7.4 m to 8.8 m

Elevation 93.6 m to 92.2 m

Dry Sample

Run 1 Start
elev. 93.6 m



Run 1 End
elev. 92.2 m

Borehole 166-22-02

RUN 1

Depth 7.4 m to 8.8 m

Elevation 93.6 m to 92.2 m

Wet Sample

Run 1 Start
elev. 93.6 m



Run 1 End
elev. 92.2 m



THURBER ENGINEERING LTD.

Highway 401 Underpass at Merwin Lane
(Site No. 16X-0166)
Assignment No. 4019-E-0010.2, GWP 4024-20-00

BH 166-22-02
Project No.: 29381

Borehole 166-22-02

RUNS 2 and 3

Depth 8.8 m to 10.8 m

Elevation 92.2 m to 90.2 m

Dry Sample

Run 1 Start
elev. 92.2 m



Run 1 End
elev. 90.8 m



Run 2 Start
elev. 90.8 m

Run 2 End
elev. 90.2 m

Borehole 166-22-02

RUNS 2 and 3

Depth 8.8 m to 10.8 m

Elevation 92.2 m to 90.2 m

Dry Sample

Run 1 Start
elev. 92.2 m



Run 1 End
elev. 90.8 m



Run 2 Start
elev. 90.8 m

Run 2 End
elev. 90.2 m

Borehole 166-22-03

RUN 1

Depth 1.0 m to 2.8 m

Elevation 93.3 m to 91.5 m

Dry Sample

Run 1 Start
elev. 93.3 m



Run 1 End
elev. 91.5 m



THURBER ENGINEERING LTD.

Highway 401 Underpass at Merwin Lane
(Site No. 16X-0166)
Assignment No. 4019-E-0010.2, GWP 4024-20-00

BH 166-22-03
Project No.: 29381

Borehole 166-22-03

RUN 1

Depth 1.0 m to 2.8 m

Elevation 93.3 m to 91.5 m

Wet Sample

Run 1 Start
elev. 93.3 m



Run 1 End
elev. 91.5 m



THURBER ENGINEERING LTD.

Highway 401 Underpass at Merwin Lane
(Site No. 16X-0166)
Assignment No. 4019-E-0010.2, GWP 4024-20-00

BH 166-22-03
Project No.: 29381

Borehole 166-22-03

RUN 2

Depth 2.8 m to 4.4 m

Elevation 91.5 m to 89.9 m

Dry Sample

Run 1 Start
elev. 91.5 m



Run 1 End
elev. 89.9 m

Borehole 166-22-03

RUN 2

Depth 2.8 m to 4.4 m

Elevation 91.5 m to 89.9 m

Wet Sample

Run 1 Start
elev. 91.5 m



Run 1 End
elev. 89.9 m



Appendix D.

Site Photographs



Photograph 1: Looking north at south abutment along Merwin Lane
[taken on December 08, 2022]



Photograph 2: Looking south from south abutment along Merwin Lane
[taken on November 22, 2022]



Photograph 3: Looking west along Highway 401
[taken on December 07, 2022]



Photograph 4: Looking south underside of south abutment
[taken on December 08, 2022]



Photograph 5: Looking north at the southern approach
[taken on December 08, 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: 44.717N 75.542W

User File Reference: Highway 401 Merwin Lane Underpass

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)	0.340	0.196	0.120	0.035
Sa (0.1)	0.406	0.243	0.154	0.050
Sa (0.2)	0.346	0.211	0.136	0.047
Sa (0.3)	0.267	0.164	0.107	0.038
Sa (0.5)	0.194	0.119	0.078	0.028
Sa (1.0)	0.100	0.062	0.040	0.014
Sa (2.0)	0.048	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)	0.222	0.133	0.084	0.027
PGV (m/s)	0.161	0.095	0.059	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

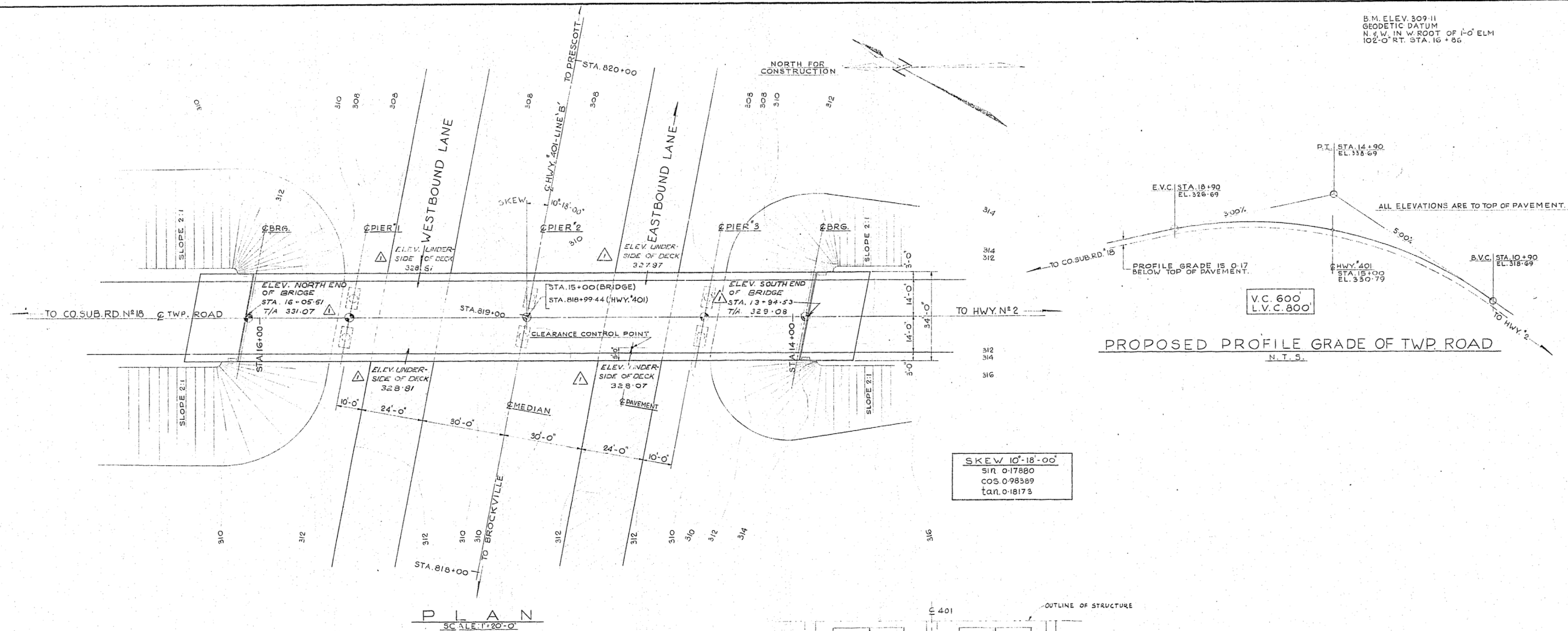
Canada



Appendix F.

Original Structural Design Drawings

B.M. ELEV. 309.11
GEODETIC DATUM
N.W. IN W. ROOT OF 1'-0" ELM
102'-0" RT. STA. 16+66



PLAN
SCALE: 1"=20'-0"

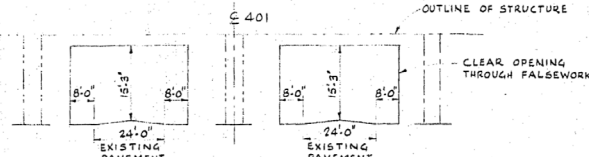
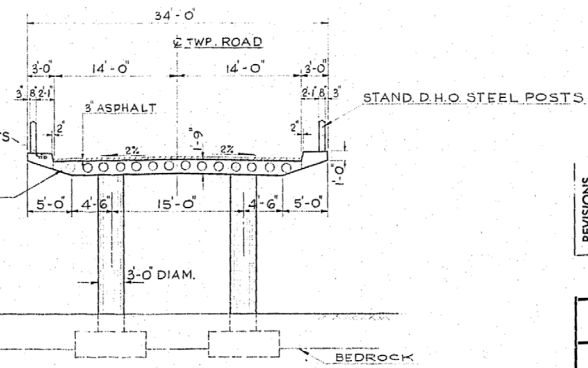
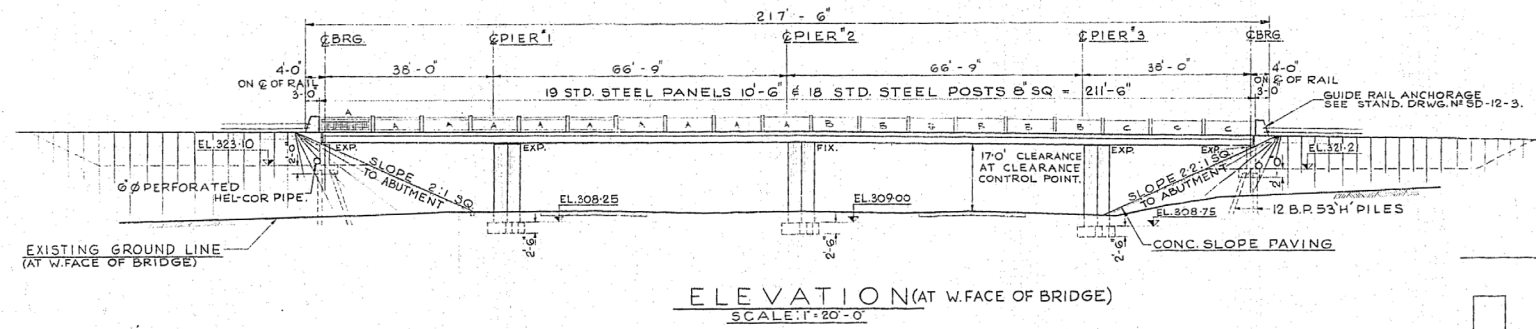


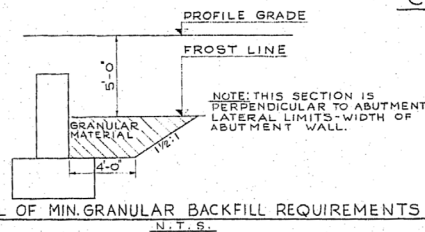
DIAGRAM OF MINIMUM CLEARANCES
TO BE PROVIDED DURING CONSTRUCTION
SEE SPECIAL PROVISIONS FOR OTHER REQUIREMENTS



CROSS SECTION AT PIER
SCALE: 1"=10'-0"



ELEVATION (AT W. FACE OF BRIDGE)
SCALE: 1"=20'-0"



DETAIL OF MIN. GRANULAR BACKFILL REQUIREMENTS
N.T.S.

REFERENCE PLANS
F 3417-4
F 3417-5
E-4265-1
BA.1898



NOTES:
TO CONTRACTOR
STRUCTURE TO BE BUILT IN ACCORDANCE WITH FORM
N° 9 AND THE SPECIAL PROVISIONS, EXTRA COPIES
OF WHICH MAY BE OBTAINED FROM THE ENGINEER.
CONCRETE MIX

	MIN. STRENGTH AT 28 DAYS
DECK & PIERS	5,000 P.S.I.
FOOTINGS	3,000 P.S.I.
ELSWHERE	3,000 P.S.I.

CLEAR COVER ON REINFORCING STEEL (UNLESS OTHERWISE
STATED) FOOTINGS 3", ABUTMENTS 2", PIERS 2", DECK 1 1/2",
ENDPOSTS 1 1/2". APPROACH SLABS 1 1/2".
CONSTRUCTION NOTES
ALL EXPOSED EDGES TO BE CHAMFERED 1" x 1" EXCEPT
AS NOTED. ALL CONSTRUCTION JOINTS MUST BE
APPROVED BY THE ENGINEER.

PILE LIST		
N° OF PILES	LENGTH	LOCATION
5	16'-0"	N. ABUTM.
5	13'-0"	S. ABUTM.
DESIGN LOAD 90T/PILE		

- DRAWING LIST
- 1 - GENERAL PLAN
 - 2 - BOREHOLE LOCATION & SOIL STRATA
 - 3 - ABUTMENT & PIERS
 - 4 - DECK & APPROACH SLAB
 - 5 - MISCELLANEOUS DETAILS
 - 6 - HANDRAIL DETAIL
 - 7 - SLOPE DETAILS
 - 8 - REINFORCING STEEL SCHEDULE
 - 9 - "
 - 10 - "

REVISIONS	
DATE	DESCRIPTION
30/6/67 A.E.S.	ADD STA. ELEV. OF T/A & UNDS. OF DECK - PLAN - REV. AS CONST.

DEPARTMENT OF HIGHWAYS ONTARIO
BRIDGE DIVISION

TWP. ROAD U'PASS (MERVIN LANE)
0.8 MI. WEST OF EDWARD STREET (PRESCOTT)

KING'S HIGHWAY No. 401 DIST. No. 8
CO. OF GRENVILLE
TWP. OF AUGUSTA LOT 6 & 7 CON. 1

GENERAL PLAN

APPROVED	<i>A. A. WITECKI</i>	SITE No.	16-106	W.P. No.	195-59
DESIGN	A.A.	CHECK	F.S.R.	CONTRACT	65-161
DRAWING	B.M.S.	CHECK	F.S.R.	DRAWING	65-161
DATE	OCT. '64	LOADING	H20/SIG	DRAWING	65-161

TWP# 26-166-1-A

16-166

1-65-10



Appendix G.

Comparison of Foundation Alternatives





COMPARISON OF FOUNDATION ALTERNATIVES

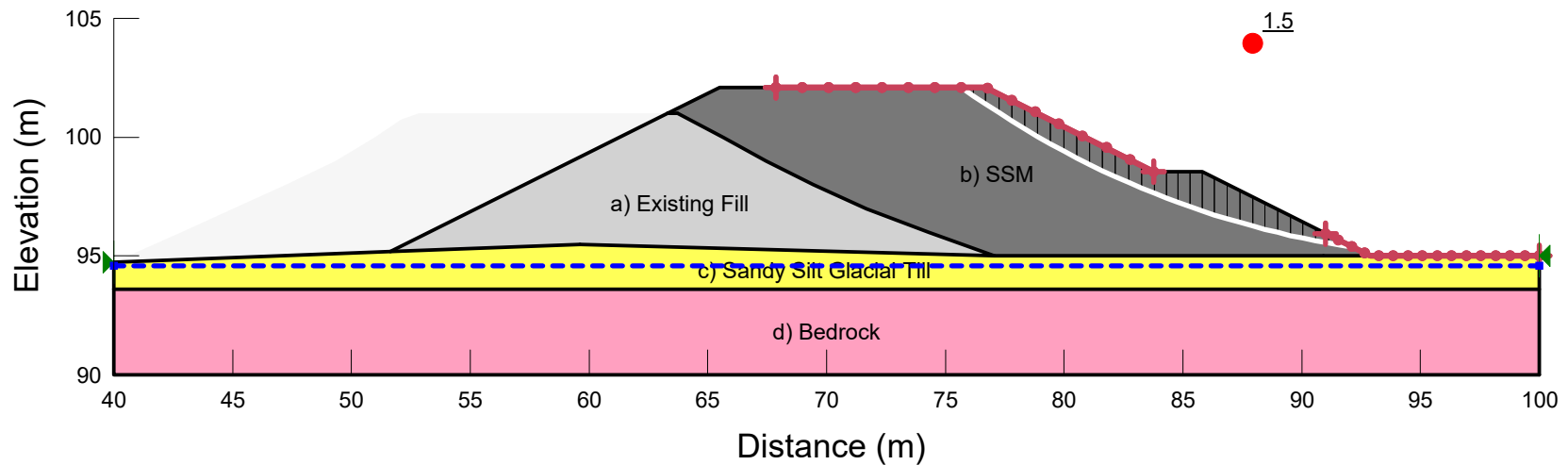
	<i>Spread Footings</i>	<i>Driven Steel H-Piles</i>	<i>Concrete Caissons</i>
Description	<ul style="list-style-type: none"> Foundation element founded on traditional shallow spread footings on engineering fill, dense glacial till or bedrock. 	<ul style="list-style-type: none"> The abutments would be supported 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 hole in the ground that derives axial resistance from end bearing
Advantages	<ul style="list-style-type: none"> Existing structure supported on shallow footings at pier, and 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. 	<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, particularly at abutments beneath portions of existing embankments, adjacent to existing structure. Excavations adjacent to existing structure and at the pier in the existing Highway 401 median may require temporary protection systems. 	<ul style="list-style-type: none"> Has potential to encounter obstructions in the fill and glacial till. Lower geotechnical resistance than caissons. 	<ul style="list-style-type: none"> Not compatible with integral abutment design approach. Likely requires concrete to be placed using tremie techniques. Temporary steel casing required to keep hole open during drilling. The base of the caisson would need to be inspected to ensure end bearing capacity.
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 	<ul style="list-style-type: none"> Moderate 	<ul style="list-style-type: none"> High
Conclusion	<ul style="list-style-type: none"> Recommended for pier 	<ul style="list-style-type: none"> Recommended for abutments 	<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) Sandy Silt Glacial Till	Mohr-Coulomb	19	0	35
	d) Bedrock	Bedrock (Impenetrable)			



Project
Highway 401 Merwin Lane Underpass

Analysis
01) Permanent – Long Term

Seismic Coefficient
H: g, V: g

Last Run
2023/04/27, 03:40:05 PM

Scale
1:300

Additional Details

Name: North Embankment

Comments:





Method: Morgenstern-Price, Half-Sine

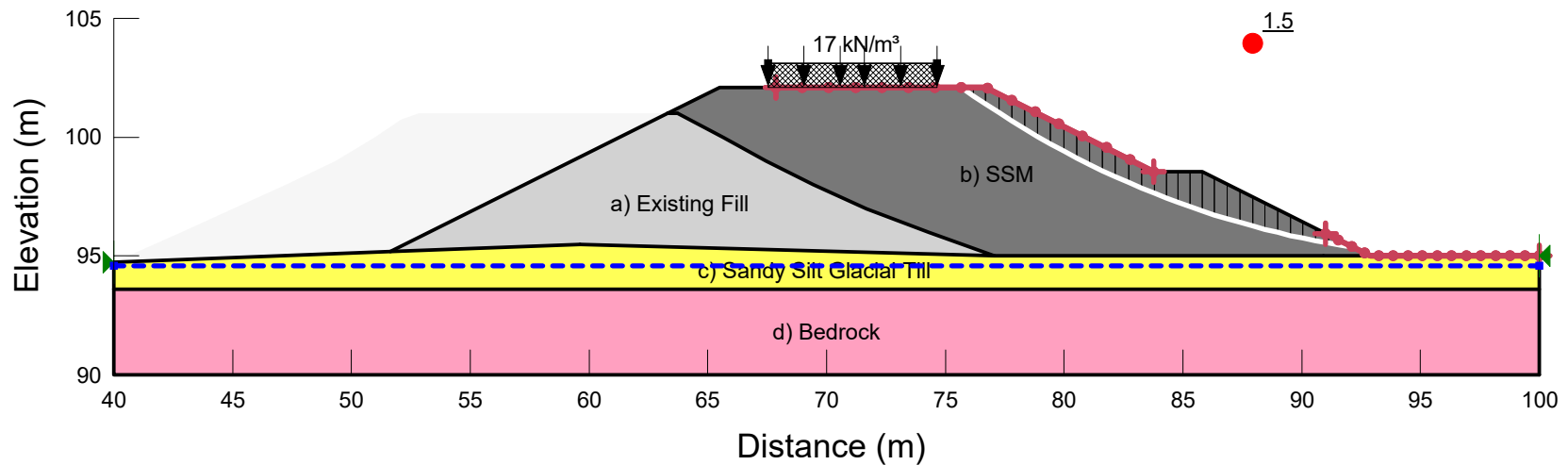
Minimum Slip Surface Depth: 1.52 m

Entry: (75.682064, 102.1) m, Exit: (92.17524, 95.36238) m

Center: (97.827741, 132.75509) m, Radius: 37.817528 m

Figure H1

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) Sandy Silt Glacial Till	Mohr-Coulomb	19	0	35
	d) Bedrock	Bedrock (Impenetrable)			



Project
Highway 401 Merwin Lane Underpass

Analysis
02) Temporary (traffic) – Short Term

Seismic Coefficient
H: g, V: g

Last Run
2023/04/27, 03:40:06 PM

Scale
1:300

Additional Details

Name: North Embankment

Comments:





Method: Morgenstern-Price, Half-Sine

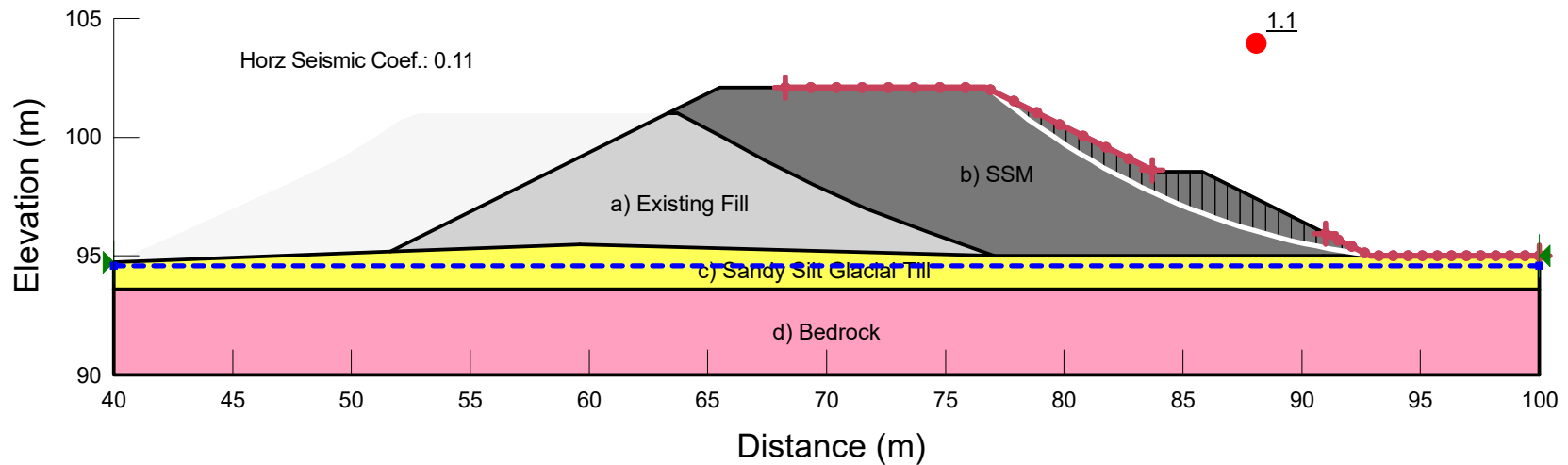
Minimum Slip Surface Depth: 1.52 m

Entry: (75.682064, 102.1) m, Exit: (92.17524, 95.36238) m

Center: (97.827741, 132.75509) m, Radius: 37.817528 m

Figure H2

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) Sandy Silt Glacial Till	Mohr-Coulomb	19	0	35
	d) Bedrock	Bedrock (Impenetrable)			



Project
Highway 401 Merwin Lane Underpass

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

Seismic Coefficient
H: 0.11g, V: g

Last Run
2023/04/27, 03:40:07 PM

Scale
1:300

Additional Details

Name: North Embankment

Comments:





Method: Morgenstern-Price, Half-Sine

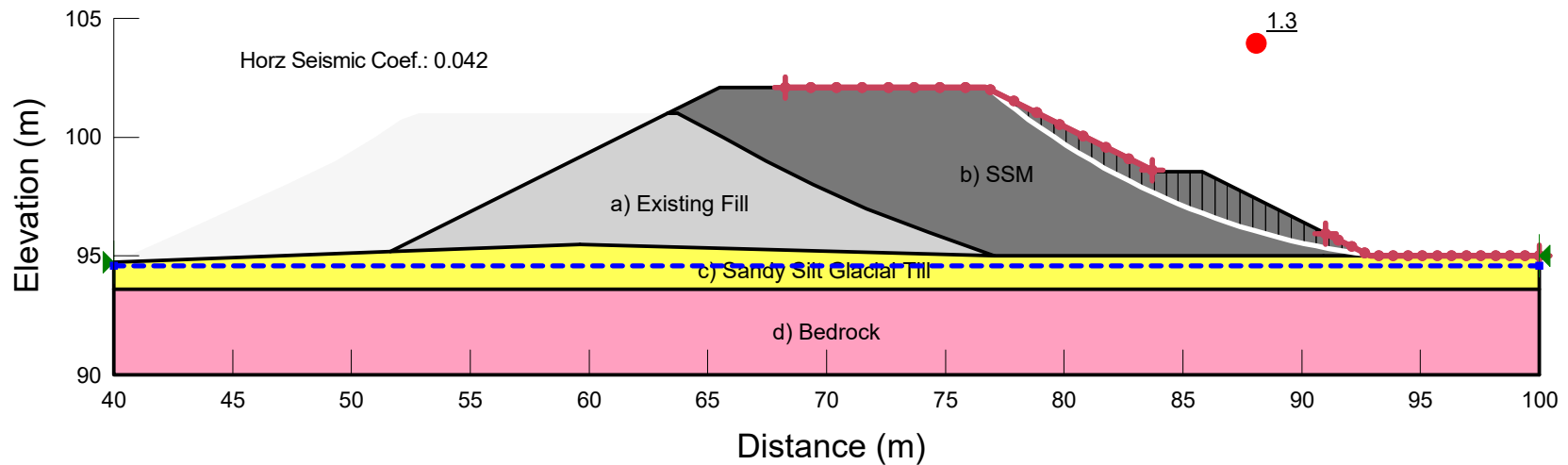
Minimum Slip Surface Depth: 1.52 m

Entry: (76.910561, 101.99472) m, Exit: (92.764185, 95.067908) m

Center: (96.669055, 125.61088) m, Radius: 30.791572 m

Figure H3

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) Sandy Silt Glacial Till	Mohr-Coulomb	19	0	35
	d) Bedrock	Bedrock (Impenetrable)			



Project
Highway 401 Merwin Lane Underpass

Analysis
04 Temporary (475 yr EQ) - Short Term

Seismic Coefficient
H: 0.042g, V: g

Last Run
2023/04/27, 03:40:07 PM

Scale
1:300

Additional Details

Name: North Embankment

Comments:

Method: Morgenstern-Price, Half-Sine

Minimum Slip Surface Depth: 1.52 m

Entry: (76.910561, 101.99472) m, Exit: (92.764185, 95.067908) m

Center: (96.669055, 125.61088) m, Radius: 30.791572 m

Figure H3



Appendix I.

List of Referenced Specifications



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

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