



THURBER ENGINEERING LTD.

**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 401 WIDENING, HIGHWAY 16 TO MAITLAND ROAD
EDWARD STREET UNDERPASS, SITE NO. 16X-0128/B0
GWP 4024-20-00 / ASSIGNMENT NO. 4019-E-0010.2**

Geocres No.: 31B-111

Report to:

MTO c/o AECOM Canada Ltd.

Latitude: 44.723032°
Longitude: -75.524982°

September 2023
Thurber File No.: 29381



TABLE OF CONTENTS

PART 1. FACTUAL INFORMATION

1	INTRODUCTION.....	1
2	SITE DESCRIPTION	1
3	SITE INVESTIGATION AND FIELD TESTING	2
3.1	Previous Investigations	2
3.2	Current Field Investigation	3
4	LABORATORY TESTING	4
5	DESCRIPTION OF SUBSURFACE CONDITIONS	4
5.1	Fill.....	4
5.2	Sandy Silt to Silt with Sand	6
5.3	Silty Clay to Clayey Silt	6
5.4	Silt	7
5.5	Bedrock	8
5.6	Groundwater.....	8
6	MISCELLANEOUS	9

PART 2. ENGINEERING DISCUSSION AND RECOMMENDATIONS

7	INTRODUCTION.....	11
7.1	Existing Structure	11
7.2	Proposed Structure	12
7.3	Applicable Codes and Design Considerations	12
7.4	Geotechnical Assessment.....	12
8	SEISMIC CONSIDERATIONS	13
8.1	Spectral and Peak Acceleration Hazard Values.....	13
8.2	CHBDC Seismic Site Classification.....	13
8.3	Seismic Performance Category.....	14
8.4	Liquefaction Potential	14
9	STRUCTURE FOUNDATION ALTERNATIVES	15



9.1	Foundation Alternatives.....	15
9.2	Recommended Foundation	16
9.3	Construction Methodology.....	16
10	FOUNDATION DESIGN RECOMMENDATIONS	16
10.1	Driven Steel H-Pile Foundations	16
10.1.1	Axial Resistance	16
10.1.2	Downdrag	17
10.1.3	Lateral Resistance of Piles	18
10.2	Caisson Pier Foundations	19
10.2.1	Axial Resistance	20
10.2.2	Lateral Resistance	20
10.2.3	Caisson Installation.....	21
10.3	Drilled-In Pipe Piles	22
10.3.1	Axial Geotechnical Resistance	22
10.3.2	Downdrag	22
10.3.3	Lateral Geotechnical Resistance and Group Effects	23
10.3.4	Pipe Pile Installation	23
10.3.5	Rock Socket Base Inspection	24
10.4	Backfill and Lateral Earth Pressures	24
10.4.1	Static Lateral Earth Pressure	24
10.4.2	Combined Static and Seismic Lateral Earth Pressure	25
10.5	Frost Depth.....	26
10.6	Embankment Stability.....	26
10.7	Embankment Settlement	28
11	RECOMMENDED SCOPE FOR DETAIL DESIGN.....	29
12	CLOSURE	31
	REFERENCES	32
	STATEMENT OF LIMITATIONS AND CONDITIONS	



APPENDICES

Appendix A.	Borehole Location Plan and Stratigraphic Drawing
Appendix B.	Record of Borehole Sheets
Appendix C.	Laboratory Testing
Appendix D.	Selected Site Photographs
Appendix E.	Existing Structure – General Arrangement and Elevation Drawings Historical GEOCRES Borehole Information
Appendix F.	Foundation Alternative Comparisons
Appendix G.	Slope Stability Analysis Figures
Appendix H.	GSC Seismic Hazard Calculation
Appendix I.	List of Referenced Specifications and NSSPs



**FINAL
PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 401 WIDENING, HIGHWAY 16 TO MAITLAND ROAD
EDWARD STREET UNDERPASS, SITE NO. 16X-0128/B0
GWP 4024-20-00 / ASSIGNMENT NO. 4019-E-0010.2**

Geocres No.: 31B-111

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 Edward Street (Site 16X-0128/B0), located approximately 5.0 km west of Highway 416, in the town of Prescott, Ontario.

This section of the report presents the factual findings obtained from a foundation investigation completed at the site, as well as data from existing subsurface information pertinent to the site, obtained from the MTO's Foundation Library (Geocres No. 31B00-016).

The purpose of this investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan, records of boreholes, 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 during the current investigation.

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

2 SITE DESCRIPTION

Site 16X-0128/B0 is located on Highway 401 approximately 5.0 km west of Highway 416. The location of the structure is shown on the inset Key Plan on Drawing No. 1 in Appendix A.

The current structure carries four lanes of Edward Street traffic over Highway 401. The Ontario Structure Inspection Manual (OSIM) report prepared by MTO on March 7, 2018 indicates that the existing structure is a one-span structure with reinforced cast-in-place concrete box girders and was constructed in 1957. The inspection report indicates that the bridge deck is approximately 38 m long and 17 m wide, with an approximate 16-degree skew to the highway. It is noted that



for project orientation purposes, Highway 401 will be assumed to be oriented east-west and Edward Street to be oriented north-south.

Highway 401 at this location has two through lanes and one acceleration lane in each direction with paved shoulders. The eastbound and westbound lanes are separated by a paved median. There is a concrete barrier wall in the median and steel beam guide rails located along the outer lanes north-east and south-west of the underpass structure.

Within the project limits Edward Street has two lanes in each direction. On the approaches, concrete curb and gutter are present in both directions. Steel beam guiderail systems are present on the approaches. The existing approach embankments are up to approximately 7 m high with slopes that extend down at approximately 2H:1V (Horizontal:Vertical). The embankment slopes are vegetated with long grasses, shrubs, and occasional trees. At the time of the field investigation, the embankments did not show any visible signs of distress or other performance issues.

Based on published geological information in *The Physiography of Southern Ontario* by Chapman and Putnam (1984), the site lies within the physiographic region known as the Edwardsburg Sand Plain. The Edwardsburg Sand Plain is characterized by and is characterized by glaciofluvial (deposited by glacial meltwater) sand deposits overlying bedrock, till or clay. Terrain is relatively flat, with sand ridges and moraines providing some relief. The area is known to be underlain by dolostone and dolomitic sandstone bedrock of the Beekmantown Group.

The land south of the site generally consists of commercial properties and residential dwellings. A gas station, retail mall, and high school are present to the southeast, with a residential development to the southwest. The land north of the site generally consists of open or treed, undeveloped lands with the exception of a car dealership in the northwest quadrant. The terrain surrounding the site is relatively flat. Storm water drainage in the area is a combination of median storm sewers on Highway 401 and roadside ditches along Edward Street and the outsides of Highway 401.

Photographs showing the existing conditions at the site at the time of the field investigation are included in Appendix D for reference.

3 SITE INVESTIGATION AND FIELD TESTING

3.1 Previous Investigations

A foundation investigation report for the existing Highway 401 Edward Street Underpass (31B00-016, 1956) was obtained from the online Geocres library. The investigation included one unsampled dynamic cone penetration test (DCPT) at the north abutment and two sampled boreholes, one at each abutment. The stratigraphy encountered in the sampled boreholes consisted of a surficial silty sand deposit ranging in thickness from 4.3 m to 5.2 m overlying a deep clay deposit underlain by bedrock. The clay deposit was firm to very stiff in consistency based on vane shear testing. Atterberg Limits testing indicates the clay would be classified to be of intermediate plasticity (CI). Bedrock was cored at the north abutment with a surface elevation



of ~75.3 m and was inferred based on refusal in the south abutment borehole and north abutment DCPT at elevations of ~75.0 m and ~75.5 m, respectively.

3.2 Current Field Investigation

The current field investigation for this site included advancing three boreholes between November 22 and December 8, 2022. The approximate locations in MTM NAD83, Zone 9 coordinates and elevations of the boreholes are shown on Drawing No. 1 provided in Appendix A and are summarized in the table below.

Borehole No.	Location	Northing (m) (Latitude)	Easting (m) (Longitude)	Ground Surface Elevation (m)	Termination Depth (m)
128-22-01	South Abutment	4 954 109.0 (44.722821°)	382 060.9 (-75.524708°)	99.8	28.1
128-22-02	North Abutment	4 954 166.8 (44.723344°)	382 025.2 (-75.525150°)	99.2	27.6
128-22-03	Hwy 401 Median	4 954 148.4 (44.723176°)	382 057.1 (-75.524749°)	92.4	20.9

As a component of our standard procedures and due diligence, Thurber contacted Ontario One Call to obtain utility locates/clearances in advance of drilling.

The boreholes were advanced using a truck-mounted CME 55 drill rig equipped with hollow stem augers, NW casing, and NQ coring equipment. Split spoon samples were collected at regular depth intervals in the boreholes during the completion of Standard Penetration Tests (SPT), following the methods described in ASTM Standard D1586-11. In-situ vane shear testing was conducted in the cohesive deposits following ASTM Standard D2573-18. The drilling and sampling operations were supervised on a full-time basis by a member of Thurber's geotechnical staff. The drilling supervisor logged the boreholes and processed the recovered soil and bedrock samples for transport to Thurber's Ottawa geotechnical laboratory for further examination and testing.

A piezometer consisting of a 19 mm PVC pipe were installed in Borehole 128-22-01 to allow for measurement of the groundwater level at the site. The piezometer construction details are illustrated on the corresponding Record of Borehole sheet provided in Appendix B. The standpipe piezometer at Borehole 128-22-01 was decommissioned in accordance with MOE requirements on April 26, 2023.

Following completion of the field investigation, the boreholes without a piezometer were decommissioned in general accordance with O.Reg. 903, as amended and capped with granular material and cold patch asphalt to reinstate the pavement surface.



The as-drilled borehole elevations were surveyed by Thurber with a surveyor's level with a reported accuracy of ± 1.5 mm, relative to survey benchmarks provided by AECOM. Borehole elevations were reviewed with reference to the topographic survey received from AECOM. Horizontal as-drilled locations were measured relative to several existing site features. The borehole coordinates and elevations are shown on the Borehole Location and Soil Strata Drawing in Appendix A and the individual Record of Borehole sheets in Appendix B.

4 LABORATORY TESTING

Laboratory testing was selected in accordance with the current MTO Guideline for Foundation Engineering Services, Section 5. Geotechnical laboratory testing consisted of natural moisture content determination and visual identification of all retained soil samples. Recovered soil samples were selected for grain size distribution and, where, appropriate, Atterberg Limits testing in accordance with MTO and ASTM standards. The results of these tests are summarized on the Record of Borehole sheets included in Appendix B.

All rock cores were photographed and their total core recovery (TCR), solid core recovery (SCR) and rock quality designation (RQD) were measured. One sample of intact rock core was submitted for Unconfined Compressive Strength (UCS) testing.

All laboratory test results from the investigation are provided in Appendix C.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Details of the encountered soil stratigraphy are presented on the Record of Borehole sheets included in Appendix B and 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 paragraphs. However, the factual data presented on the Borehole Records takes precedence over the Soil Strata Drawing and this general description for interpretation of 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 with cohesive soils described as per current MTO Guidelines for Foundation Engineering Services, Version 3 (April 2022).

In general terms, the encountered stratigraphy consisted of pavement structure and granular embankment fill, overlying a native silty sand to silt with sand deposit, overlying a cohesive deposit ranging in composition from silty clay to clayey silt underlain by a thin discontinuous silt layer overlying dolostone bedrock.

5.1 Fill

Asphalt

Asphalt was encountered at the ground surface in all boreholes and ranged in thickness from 150 mm to 200 mm.



A layer of buried asphalt with a thickness of 150 mm was encountered below the wood fill at a depth of 8.5 m (Elev. 91.3 m) in Borehole 128-22-01.

Silty Sand with Gravel

A fill layer consisting of silty sand with gravel was encountered below asphalt layer in Boreholes 128-22-01 and 128-22-03. The silty sand with gravel fill layer ranged in thickness of from 0.1 to 1.3 m (base Elev. 92.1 to 98.3 m). A single SPT N-Value of 16 blows was obtained within this layer indicating a compact relative density.

The recorded moisture content within this layer was 8%. A gradation analysis completed on one sample of the silty sand with gravel fill indicate the sample contained 18% gravel, 42% sand and 40% silt and clay sized particles. The results are illustrated on Figure C1 of Appendix C and summarized on the corresponding Record of Borehole sheet in Appendix B.

Sand with Silt to Silty Sand

A fill layer ranging in composition from sand with silt to silty sand was encountered below the silty sand with gravel fill layer in Boreholes 128-22-01 and 128-22-03 and below the asphalt in Borehole 128-22-02. The sand with silt to silty sand fill layer had a thickness ranging from 2.0 to 7.9 meters (base Elev. 90.1 to 91.6 m). The SPT N-values obtained typically ranged from 2 to 35 blows for 0.3 m of penetration indicating a very loose to dense relative density. A single SPT N-value of 100 blows for 100 mm of penetration was obtained near the base of the layer in Borehole 128-22-02 but is attributed to the buried concrete encountered below the layer rather than the sand with silt to silty sand fill layer.

The recorded moisture contents within this layer ranged from 4 to 22%. The results of gradation analyses completed on five samples of the sand with silt to silty sand fill are illustrated on Figure C2 of Appendix C. The results of the tests are summarized below and on the Record of Boreholes sheets in Appendix B.

Soil Particle	Percentage (%)
Gravel	0 to 2
Sand	82 to 95
Silt	4 to 17
Clay	

Concrete

A layer of concrete with a thickness of 0.38 m was encountered below the sand with silt fill in Borehole 128-22-02 at a depth of 8.1 m (Elev. 91.1 m). Auger refusal was encountered at the surface of this layer and wash boring and coring techniques were required to advance the borehole.



Wood

A fill layer consisting primarily of wood with silty sand with gravel infills was encountered below the sand with silt fill in Borehole 128-22-01 and below the concrete in Borehole 128-22-02. The wood fill layer had a thickness ranging from 0.3 to 1.4 meters (base Elev. 89.3 to 91.3 m). The SPT N-values obtained in the wood fill layer ranged from 42 to 81 blows for 0.3 m of penetration but are not considered to be indicative of the relative density of the layer due to the wood. The recorded moisture contents within the wood fill layer ranged from 11 to 200%.

Silty Sand

A thin layer of silty sand fill was encountered below buried asphalt layer in Borehole 128-22-01. This layer had a thickness of 0.3 m (base Elev. 90.8 m) and a recorded moisture content of 18%.

5.2 Sandy Silt to Silt with Sand

A native layer ranging in composition from sandy silt to silt with sand was encountered below the fill in all boreholes. The thickness of this layer ranged from 2.3 to 3.6 meters (base Elev. 87.0 to 87.2 m). The SPT N-values in sandy silt ranged from 2 to 24 blows for 0.3m of penetration indicating a very loose to compact relative density; but typically, loose to compact.

The recorded moisture contents typically ranged from 20 to 26%. A single moisture content of 83% was recorded near the base of the sandy silt to silt with sand layer in Borehole 128-22-03. The results of gradation analyses completed on three samples of the sandy silt to sand with silt are illustrated on Figure C3 of Appendix C. The results of the tests are summarized below and on the Record of Boreholes sheets in Appendix B.

Soil Particle	Percentage (%)
Gravel	0
Sand	17 to 39
Silt	53 to 74
Clay	6 to 9

The results of Atterberg limit testing completed on three samples of the sandy silt to silt with sand found the material to be non-plastic.

5.3 Silty Clay to Clayey Silt

A layer of silty clay was encountered beneath the sandy silt to silt with sand layer in all boreholes. The thickness of this layer ranged from 10.9 to 11.7 meters (base Elev. 75.3 to 76.3 m). SPT N-values in the silty clay ranged from weight of hammer to 8 blows but are not considered an accurate indicator of undrained shear strength. A single SPT N-value of 100 blows for 125 mm of penetration was obtained at the base of the silty clay layer in Borehole 128-22-02 but is attributed to the underlying bedrock. Field vane tests performed within this layer ranged from 86 kPa to greater than 102 kPa in indicating a stiff to very stiff consistency. Sensitivity values recorded in

the silty clay to clayey silt range from 6.3 to 32 indicating the class of sensitivity to range from sensitive to quick clay based on the Canadian Foundation Engineering Manual, 4th Edition.

Recorded moisture contents ranged from 21 to 39%. The results of gradation analyses completed on five samples of the silty clay to clayey silt are illustrated on Figure C4 of Appendix C. The results of the tests are summarized below and on the Record of Boreholes sheets in Appendix B.

Soil Particle	Percentage (%)
Gravel	0
Sand	0 to 2
Silt	40 to 55
Clay	44 to 60

The results of Atterberg limit tests completed on five samples of the silty clay to clayey silt are illustrated on Figure C5 of Appendix C. The results of the tests are summarized below and on the Record of Boreholes sheets in Appendix B. The laboratory results indicate that the silty clay to clayey silt exhibits low to intermediate plasticity (CL to CI).

Parameter	Value (%)
Liquid Limit	34 to 43
Plastic Limit	17 to 24
Plasticity Index	14 to 19
Liquidity Index	0.4 to 1.1

5.4 Silt

A thin layer of silt was encountered beneath the silty clay to clayey silt deposit in Boreholes 128-22-01 and 128-22-03. The thickness of this layer ranged from 0.8 to 1.6 m (base Elev. 74.7 m to 74.8 m). SPT N-values in this layer ranged from 9 to 10 blows indicating a loose to compact relative density. A single SPT N-value of 100 blows for 50 mm of penetration was obtained at the base of the layer in Borehole 128-22-03 but is attributed to the underlying bedrock.

The moisture contents ranged from 5% to 20%. The results of gradation analyses completed on two samples of the silt are illustrated on Figure C6 of Appendix C. The results of these tests are summarized below and on the Record of Boreholes sheet in Appendix B.

Soil Particle	Percentage (%)
Gravel	0
Sand	0 to 1
Silt	85 to 87
Clay	13 to 14

The results of Atterberg limit testing completed on two samples of the silt found the material to be non plastic.

5.5 Bedrock

The overburden materials were underlain by a grey dolostone bedrock. Bedrock was cored in all boreholes; the bedrock surface ranges from elevation 74.7 m to 75.3 m. Photographs of the bedrock core are provided in Appendix C. The table below summarizes the depths and elevations of the bedrock surface.

Location	Borehole	Ground Surface Elevation (m)	Depth Below Existing Grade (m)	Top of Bedrock Elevation (m)
South Abutment	128-22-01	99.8	25.1	74.7
North Abutment	128-22-02	99.2	23.9	75.3
Hwy 401 Median	128-22-03	92.4	17.6	74.8

All boreholes were advanced into the bedrock with NQ-size coring equipment.

The bedrock had a total core recovery ranging from 96% to 100%, a solid core recovery ranging from 71% to 100% and a Rock Quality Designation (RQD) ranging from 81% to 100%. Based on the RQD value the bedrock is classified as good to excellent quality.

Unconfined compressive strength (UCS) testing was carried out on a sample of the dolostone bedrock from Borehole 128-22-01. The results of this test indicated a UCS strength of 174 MPa indicating a very strong bedrock. The results of the UCS testing are included in Appendix C.

5.6 Groundwater

A standpipe piezometer was installed in Borehole 128-22-01 to monitor the groundwater level after completion of drilling. The measured groundwater levels are summarized in the table below.



Borehole No.	Bottom of Screen Elev. (m)	Screened Unit	Depth (mbgs)	Groundwater Elevation (m)	Date of Measurement
128-22-01	78.6	Silty Clay	8.3	91.5	December 18, 2022
			8.2	91.6	April 26, 2023

A water level depth of 8.7 m (Elev. 90.5 m) was also obtained at Borehole 128-22-02 within the open borehole drilling casing which was left in the ground over the weekend during the drilling investigation.

These observations are considered short term and it should be noted that the groundwater level at the time of construction may be different. Seasonal fluctuations are to be expected. In particular, the water level may be at a higher elevation after periods of significant and/or prolonged precipitation and spring snow melts.

6 MISCELLANEOUS

Thurber staked and/or marked the borehole locations in the field and obtained utility clearances prior to drilling. The as-drilled locations and ground surface elevation were measured by Thurber following completion of the field program.

Downing Drilling of Hawkesbury, Ontario supplied and operated the truck mounted CME 55 drill rig to carry out the drilling, sampling, in-situ testing, standpipe piezometer installation and borehole decommissioning. Water for wash boring was transported and provided by T.G. Carroll Cartage of Carp, Ontario. Traffic control was performed in accordance with Ontario Book 7 for short duration closures; all signs, barrels, cones and traffic control personnel were provided by T.G. Carroll Cartage of Carp, Ontario. The field investigation was supervised on a full-time basis by Ibrahim Khan, EIT.

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, Ontario. Overall project management and direction of the field investigation was provided by Matt Kennedy, P.Eng. Management of the field investigation was carried out by Katya Walker, P.Eng. Interpretation of the factual data and preparation of this report was completed out by Ibrahim Khan, EIT and Christopher Murray, P.Eng. The report was reviewed by Matt Kennedy, P.Eng. and Fred Griffiths, P.Eng., a Designated Principal Contact for MTO Foundations Projects.



Christopher Murray, M.A.Sc., P.Eng.
Geotechnical Engineer



Matt Kennedy, M.Sc.(Eng.), P.Eng.
Senior Geotechnical Engineer



Paul Carnaffan, M.Eng., P.Eng.
Principal | Branch Manager
Senior Geotechnical Engineer



Fred Griffiths, Ph.D., P.Eng.
MTO Review Principal,
Senior Geotechnical Engineer



**FINAL
PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 401 WIDENING, HIGHWAY 16 TO MAITLAND ROAD
EDWARD STREET UNDERPASS, SITE NO. 16X-0128/B0
GWP 4024-20-00 / ASSIGNMENT NO. 4019-E-0010.2**

Geocres No.: 31B-111

PART 2. ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 INTRODUCTION

This report presents the interpretation of the factual data obtained from a preliminary foundation investigation conducted by Thurber for the replacement of the existing Highway 401 Underpass structure at Edward Street in the town of Prescott, Ontario.

This preliminary foundation investigation and design report with the interpretation and recommendations are intended for the use of the Ministry of Transportation and shall not be used or relied upon for any other purposes or by any other parties including the construction or design-build contractor. Contractors must make their own interpretation based on the factual data in Part 1 of the report. Where comments are made on construction, they are provided only in order to highlight those aspects which could affect the design of the project. Contractors must make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods and scheduling.

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

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

7.1 Existing Structure

The existing bridge carries four lanes of Edward Street traffic over Highway 401 and is a one-span structure with reinforced cast-in-place concrete box girders and was constructed in 1957. The bridge deck is approximately 38 m long and 17 m wide, with an approximate 16-degree skew to the highway. It is noted that for project orientation purposes, Highway 401 will be assumed to be oriented east-west and Edward Street to be oriented north-south.

Based on the original construction Elevation & Profile drawing (No. TWP# 26-128-1-A, dated October 5th, 1956) the abutments and wingwalls are supported on 12.2 m long, Class B (12" diameter) untreated timber piles. The historical drawings available do not include a plan view of the foundation layout, but the Elevation & Profile drawing suggests the abutments are



supported by three rows of piles; the back row of piles is vertical, and the front two rows are battered towards Highway 401 at approximately 1H:4.25V. The Elevation & Profile drawing also indicates that the wingwalls are supported on 12.2 m long, Class B untreated timber piles but it is not indicated if there are multiple rows of piles or if the piles are battered.

The undersides of the existing pile caps are indicated to be at about Elevation 90.2 m, with all piles driven to a tip elevation in the silty clay to clayey silt of approximately 78.7 m.

7.2 Proposed Structure

The preliminary design of the Edward Street interchange to accommodate the proposed widening of Highway 401 includes realignment of Edward Street approximately 20 m to the east of the existing structure. It is understood that the replacement overpass structure is to be a two-span structure with centre pier constructed in the Highway 401 median and roughly the same width as the existing structure.

The new approach embankments are anticipated to be on the order of 8 to 9 m high.

7.3 Applicable Codes and Design Considerations

The geotechnical assessment presented below has been prepared based on the available data regarding the proposed foundations and existing ground conditions and is in accordance with the Canadian Highway Bridge Design Code, version CSA S6:19, (CHBDC).

In accordance with CHBDC, the analysis and design of the structure takes into consideration the importance of the structure and the consequence associated with exceeding limit states. The importance category and consequence classification are defined by the Regulatory Authority, which in this case is the Ministry of Transportation of Ontario (MTO).

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

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

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

7.4 Geotechnical Assessment

Based on the results of the field and laboratory investigation, the review of historical information and the information provided by AECOM with regards to the proposed project requirements, the geotechnical foundation design considerations include:

- The existing bridge abutments are supported on timber friction piles bearing within the silty clay to clayey silt and have performed well. Deep foundations such as driven steel H-piles will likely be required to support the new bridge abutments and pier.
- The native soils include a thick deposit of sensitive stiff to very stiff silty clay to clayey silt. Any grade raise or new embankment will result in long term settlement and depending on the proximity to the existing structure could cause settlement of the existing bridge which is supported by friction piles rather than end bearing piles.
- Due to the sensitive nature of the existing silty clay to clayey silt soil supporting the existing bridge piles, the foundation installation methodology will need to be considered in design since excessive vibrations could lead to settlement of the existing bridge structure.

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)ⁱ. 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 0.23 g (1 in 2475 year). This value is to be scaled by the $F(PGA)$ based on the site-specific Site Class as per Table 4.8 of Section 4.4.3.3 of the CHBDC (see Section 8.2).

8.2 CHBDC Seismic Site Classification

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

The native soils at the site generally consist of very loose to compact sandy silt to silt with sand overlying 10.9 to 11.7 m of sensitive to quick silty clay to clayey silt, underlain by a thin layer of silt overlying dolostone bedrock.

It is noted that Table 4.1 of the CHBDC indicates sites with liquifiable soils or quick/highly sensitive clays must be classified as Site Class F requiring site specific evaluation. Since this site contains



both liquifiable soils (see additional discussion in Section 8.4 below), and quick or highly sensitive clays, a site-specific evaluation will be required. As this is beyond the scope of this preliminary design assignment, it has been assumed that the site specific evaluation will result in the site being treated as a Site Class E.

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

8.3 Seismic Performance Category

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

8.4 Liquefaction Potential

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

The susceptibility of the cohesionless soils at the site to experience liquefaction was assessed using the SPT data following the simplified method for cohesionless soil as outlined in Boulanger and Idriss (2014)ⁱⁱⁱ. The results indicate that both the sandy silt to silt with sand deposit ranging in thickness from 2.3 to 3.6 meters (base Elev. 87.0 to 87.2 m) and the silt layer ranging in thickness from 0.8 to 1.6 m (base Elev. 74.7 m to 74.8 m) are considered susceptible to liquefaction under the design earthquake with return periods of 2,475 and 975 years but is not considered susceptible to liquefaction under the 475 year return period design earthquake. It is noted that the 1955 preliminary investigation indicated that the native sand/silt layer was in a quick condition when vibrated slightly.

As per Table 4.1 of the CHBDC, sites with liquifiable soils should be considered a Site Class F and a site-specific evaluation is required. It is recommended that additional foundation investigation and design, including a more detailed liquefaction assessment considering the results of a site-specific ground response analysis, be carried out during a subsequent project stage. There are three possible outcomes upon completion of that more rigorous work:

1. liquefaction is determined to have a low risk of occurrence and does not need to be considered in design,
2. liquefaction is assessed to be an issue and the structure and embankments are designed to accommodate the forces and displacements induced by liquefaction; or

3. liquefaction is assessed to be an issue and ground improvement techniques are employed to densify the soils to reduce the risk of liquefaction to acceptable levels.

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

9 STRUCTURE FOUNDATION ALTERNATIVES

9.1 Foundation Alternatives

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

- | | |
|---|----------------|
| • Existing Highway 401 grade near structure (128-22-03) | 92.4 m |
| • Edward Street asphalt surface at the South abutment (128-22-01) | 99.8 m |
| • Edward Street asphalt surface at the North abutment (128-22-02) | 99.2 m |
| • Base of Sandy Silt to Silt with Sand | 87.0 to 87.2 m |
| • Base of Silty Clay to Clayey Silt | 75.3 to 76.3 m |
| • Top of Dolostone Bedrock | 74.7 to 75.3 m |

Based on the soil stratigraphy and anticipated loading the replacement bridge abutments and pier will require deep foundations; shallow foundations are not considered feasible at this site due to the low bearing capacity available from the underlying soils. Given the requirements of the proposed structure proposed by AECOM, the following deep foundation alternatives were considered for the new bridge abutments and pier:

- Caissons (drilled shafts),
- Drilled-in Pipe Piles (down-the-hole hammer), and
- Steel piles (H-piles).

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 F.

- Caissons

Caisson foundations, particularly when they are socketed into bedrock, offer high geotechnical resistance, however the high lateral stiffness of caissons is not compatible or suitable for integral abutments. The high groundwater level (within a few meters of the existing ground surface) will pose additional construction challenges resulting from potential unbalanced hydraulic pressure heads and caisson base boiling when drilling through the sandy silt to silt and lower silt deposits. 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.

- Drilled-in Pipe Piles Socketed into Bedrock

Based on the foundation soils encountered, drilled-in steel pipe piles filled with concrete are considered to be a feasible option for the support of the new abutments and pier. Pipe piles are compatible with integral abutment bridges and have successfully been used to support integral abutment bridge abutments previously. Pipe piles would be required to be socketed into bedrock and could be considered if additional support is required due to the seismic induced load demand.

- Driven Steel H-piles

Driven steel H-piles are also considered feasible for the support of the new abutments and pier. The use of H-Piles with reinforced tips is recommended as wood and concrete obstructions were encountered within the fill material at this site.

9.2 Recommended Foundation

Based on an evaluation of the foundation alternatives presented above and in Appendix F, the recommended foundation approach from a geotechnical perspective is to support the new bridge abutments and pier on driven steel H-piles. Caisson foundations could be considered for the pier foundation.

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 Edward Street to minimize conflict with the Highway 401 and Edward Street traffic. Due to the existing subgrade soils present, embankment construction in advance of construction of the foundation elements (including preload and surcharge) will likely be required for the new bridge approach embankments and abutment locations.

10 FOUNDATION DESIGN RECOMMENDATIONS

10.1 Driven Steel H-Pile Foundations

10.1.1 Axial Resistance

The new abutments and pier may be 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 10-1.

Table 10-1: Estimated Pile Tip Elevations

Foundation Element	Approximate Underside of Pile Cap Elevation (m)	Estimated Pile Tip Elevation (top of bedrock) (m)	Estimated Pile Length (m)
South Abutment	98.8	74.7	24.1
Pier	90.9	74.8	16.1
North Abutment	98.2	75.3	22.9

The design parameters for axial resistance of Grade 350W HP 310x110 steel piles driven to refusal on or in the dolostone bedrock can be taken as:

Factored geotechnical axial resistance at ULS 2,150 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 resistance 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 timber piles 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". The Contractor's installation method must be capable of dislodging, removing or penetrating obstructions such as asphalt, wood and/or concrete.

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.

10.1.2 Downdrag

A thick deposit of silty clay to clayey silt was identified beneath the embankment fill and sandy silt to silt with sand layer at both abutments of the existing bridge. Settlement of the clay is expected to occur if an 8 to 9 m embankment is constructed east of the existing embankment. Based on



the preliminary data available this settlement will result in a preliminary unfactored downdrag load of up to 500 kN per pile at the abutments. Downdrag is not expected to occur at the pier as the Highway 401 grade is to remain the same.

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

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

10.1.3 Uplift Resistance

The native soils at the abutments will provide uplift resistance to the piles. Shaft friction of the native soil along the piles were calculated, assuming the piles met effective refusal to driving at the elevations provided in Table 10-1, above.

The factored geotechnical tensile resistance for a single HP 310x110 pile at either abutment may be taken as 210 kN under static conditions and 700 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)

10.1.4 Lateral Resistance of Piles

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

For cohesionless soils:

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

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 10-2 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 10-2: Soil Parameters for Lateral Pile Design

Soil Type	γ' (kN/m ³)	n_h (kN/m ³)	c_u (kPa)	K_p
Existing Fill	11	3,000	N/A	3.0
Native Cohesionless Soils	9	2,000	N/A	3.0
Native Cohesive Soils	7.5	N/A	100	2.6

The modulus of horizontal subgrade reaction may have to be reduced based on the pile spacing. The reduction factors to be used for a pile group oriented perpendicular or parallel to the direction of loading are provided in Figures C6.23, C6.24 and C6.25 in the Commentary to the CHBDC 2019.

10.2 Caisson Pier Foundations

Augered caissons are considered a feasible foundation option for the centre pier, but it is noted that the installation would involve advancing the caisson holes through very loose to loose sandy



silt and silty clay to clayey silt with the excavation extending well below the groundwater table. Based on this, the augered caisson installation would require the use of temporary steel liners, see discussion in Section 10.2.4 below.

10.2.1 Axial Resistance

Caissons extending through overburden soils and founded within the very strong dolostone bedrock may be considered to support the bridge pier. Table 10-3 presents the geotechnical resistances of typical caisson diameters for planning and design purposes.

Table 10-3: Axial Geotechnical Resistances for Caisson Design at Pier

Caisson Diameter (m)	Socket Length in Sound Rock (m)	Factored Geotechnical Resistance (Axial Compression (kN))	
		ULS	SLS
		Static ($\phi_{gu}=0.4$)	Static ($\phi_{gs}=0.8$)
1.2	3.0	7,400	N/A ⁽¹⁾
1.5	3.0	9,250	N/A ⁽¹⁾
3.0	3.0	18,500	N/A ⁽¹⁾

Notes: (1) The SLS condition will not govern for caissons bearing in the bedrock

The factored geotechnical resistances include the following factors:

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

Caissons must be installed in accordance with OPSS.PROV 903.

The resistance values provided in Table 10-3 above are based on rock socket shaft friction assuming that the walls and base of each caisson are free of loose, soft or otherwise disturbed material prior to placement of concrete.

Provided the grade of the Highway 401 median is reinstated to match existing, downdrag forces on the pier caissons need not be applied.

10.2.2 Lateral Resistance

Lateral bridge loadings can be geotechnically resisted by the caissons through passive pressure developed along the embedded portion of the shaft. The methodology outlined in Section 10.1.4 above for driven piles may be used to estimate the lateral geotechnical resistance of the caissons.



10.2.3 Uplift Resistance

The caisson rock sockets will provide uplift resistance. Shaft friction of the rock sockets were calculated, assuming the caissons are socketed a minimum of 3 m into sound bedrock as recommended in Section 10.2.1, aTable 10-1bove.

The preliminary factored geotechnical uplift resistance for single caissons at the pier are provided in Table 10-4 below.

Table 10-4 Uplift Resistances of Caissons with 3 m long Rock Socket

Caisson Diameter (m)	Factored Uplift Resistance	
	Static (kN)	Seismic (kN)
1.2	5,550	18,500
1.5	6,937	23,125
3.0	13,875	46,250

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 Caisson Installation

Caisson installation must be carried out in accordance with OPSS.PROV 903 where applicable.

The caisson installation equipment should be able to dislodge and remove any obstructions such as concrete, wood, cobbles and boulders within the fill and to penetrate into the very strong dolostone bedrock. Selection of the methods and equipment employed to install the caissons is the responsibility of the Contractor.

The caissons will extend below the groundwater table and will require temporary steel liners to stabilize the sidewall during drilling. High volumes of seepage from the water-bearing sands and silts should be anticipated into the caisson excavations. Placement of concrete using pumped tremie methods will be required and should be initiated within 6 hours of the completion of drilling and inspection of the base.

Caisson integrity testing such as cross hole sonic logging (CSL) testing or thermal integrity profiling (TIP) should also be considered during detailed design and should be selected based in part on caisson diameter.

10.3 Drilled-In Pipe Piles

10.3.1 Axial Geotechnical Resistance

The new abutments and pier may be founded on steel drilled-in pipe piles end-bearing within the very strong dolomite bedrock. The geotechnical axial resistances for various diameters of pipe piles socketed into sound bedrock with the bottom of the pipe pile seated on the base of the rock socket are provided in Table 10-5 below.

Table 10-5 Axial Geotechnical Resistances of Drilled-In Pipe Piles

Pipe Pile Diameter (mm)	Factored Axial Resistance	
	ULS (kN)	SLS (kN)
305	3,000	Will Not Govern
324	3,300	
356	3,800	

The factored geotechnical resistances include the following factors:

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

The depth of the rock socket will need to satisfy lateral load demands and structural capacities which will vary based on the diameter of the tube pile but should be a minimum of 1.0 m into sound bedrock. After installation, the pipe piles should be filled with tremie concrete.

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

10.3.2 Downdrag

Based on the preliminary data available, settlement at the abutments will result in a preliminary unfactored downdrag load of up to 500 kN per pipe pile; downdrag is not expected to occur at the pier as the Highway 401 grade is to remain the same. Please refer to Section 10.1.2 for recommendations on the application of load factors.

10.3.3 Uplift Resistance

The native soils at the abutments will provide uplift resistance to the pipe piles. Shaft friction of the native soil along the piles were calculated, assuming the pipe piles are socketed a minimum of 1 m into sound bedrock as recommended in Section 10.3.1, aTable 10-1bove.

The preliminary factored geotechnical uplift resistance for single pipe piles at both the abutments and pier are provided in Table 10-6 below.

Table 10-6 Uplift Resistances of Drilled-In Pipe Piles

Pipe Pile Diameter (mm)	Factored Uplift Resistance	
	Static (kN)	Seismic (kN)
305	160	535
324	171	570
356	187	625

These values include the following factors:

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

10.3.4 Lateral Geotechnical Resistance and Group Effects

Lateral bridge loadings can be geotechnically resisted by the drilled-in pipe piles through passive pressure developed along the embedded portion of the shaft. The methodology outlined in Section 10.1.4 above for driven piles may be used to estimate the lateral geotechnical resistance of the drilled-in pipe piles.

10.3.5 Pipe Pile Installation

Pipe pile installation must be in accordance with the relevant “Pipe Pile” sections and, where appropriate, the “Caisson Pile” sections of OPSS.PROV 903.

Drilled-in pipe piles must be installed in rock sockets that are clean and free of drill cuttings or other debris. The tube pile must reach the base of the rock socket and the steel must be seated onto bedrock. The method of installation of the pipe piles, in order to achieve a clean socket, is the responsibility of the Contractor. All drilled-in pipe piles shall be installed using a down the hole hammer with casing advancer type pile installation equipment such as Atlas Copco Symmetrix System or approved equivalent. The Contractor’s drilling method must be capable of dislodging, removing or penetrating obstructions such as asphalt, wood, and concrete in the embankment fill. Care must be exercised while drilling into the bedrock. The drilling methodology used must be capable of advancing the pile without disturbing or fracturing the bedrock at the base or sides of the pile socket. Depth of the socket should be measured from the lower bedrock elevation for a sloping bedrock condition at the individual tube pile locations.

The pipe pile must advance to the bottom of the rock socket. A flush rock cutting bit should be used for the pile sockets. Otherwise, the rock cutting bit at the tip of the pipe pile will be slightly larger in diameter than the outside diameter of the tube pile and there will be a gap between the rock socket wall and the tube pile that would need to be adequately grouted.



Temporary liners should not be permitted for drilled-in pipe pile installation.

10.3.6 Rock Socket Base Inspection

Given the smaller diameter of pipe pile, it is likely that visual observation of the base of the rock socket and/or inspection of the socket base using a Miniature Shaft Inspection Device (Mini-SID) may be difficult. Therefore, effective cleaning by the Contractor and confirmation of cleanliness with sounding of the base of the rock socket with a weighted tape will be required. Additional cleaning or re-cleaning of the base of the rock may be required by the Contractor prior to the approval of the rock socket.

10.4 Backfill and Lateral Earth Pressures

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

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

10.4.1 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 10-7) (K_a for yielding walls, K_o for non-yielding walls)
γ	=	unit weight of retained soil (see Table 10-7), use submerged unit weight below groundwater level
h	=	depth below top of fill where pressure is computed (m)
q	=	value of any surcharge (kPa)

A lateral earth pressure due to backfill compaction should be added to the calculated lateral earth pressure in accordance with Clause 6.12.3 of the CHBDC. Typical lateral earth pressure parameters for use in the design for a horizontal back-slope are provided in Table 10-7.

Table 10-7 Static Earth Pressure Coefficients

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

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

10.4.2 Combined Static and Seismic Lateral Earth Pressure

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

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

The coefficients of horizontal earth pressure for combined static and seismic loading presented in Table 10-8 may be used for a horizontal back-slope. The provided earth pressure coefficients are calculated using a site-adjusted PGA of 0.264 g, based on a Seismic Site Class E, a reference (Site Class C) PGA with a 2% probability of exceedance in 50 years of 0.227 g (Geological Survey of Canada – Fifth Generation) and a $F(\text{PGA})$ of 1.163 as per Table 4.8 of the CHBDC.

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

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

where:

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

Table 10-8 Combined Static and Seismic Earth Pressure Coefficients

Parameter	OPSS Granular A & B Type II	OPSS Granular B Type I	OPSS SSM & Existing Granular Fill
Soil Unit Weight, kN/m ³ , γ	22.8	21.2	20.0
Angle of Internal Friction, ϕ	35°	32°	30°
Non-Yielding Wall			
Dynamic Active Earth Pressure Coefficient, K_{AE}	0.45	0.50	0.53
Yielding Wall			
Dynamic Active Earth Pressure Coefficient, K_{AE}	0.35	0.39	0.42

10.5 Frost Depth

The depth of frost penetration at this site is estimated to be 1.6 m (as per OPSD 3090.101). Footings and pile caps for the abutments, pier and retaining walls should be founded at or below this depth or provided with equivalent insulation.

10.6 Embankment Stability

Based on the available original structure drawings and observations during the 2022 field investigation, the grade of the current Edward Street lanes ranges from about 99.2 m to 99.8 m in the area of the Highway 401 underpass with embankments of up to about 7 m in height and sloped at roughly 2H:1V.

The embankment to be constructed for the proposed new alignment will shift the centerline of the roadway approximately 20 m to the east and is anticipated to be roughly 9 m in height. For the purpose of preparing preliminary geotechnical design recommendations, a number of assumptions have been made that are consistent with MTO's standard highway design practices:

- Topsoil and other deleterious material will be removed from within the footprint prior to constructing the new embankment.

- Where new fill is placed against an existing embankment slope or on a sloping ground surface steeper than 3H:1V, the existing slope will be benched (OPSD 208.010).
- The embankment will be constructed using granular fill meeting at a minimum the requirements of Select Subgrade Material (SSM) as per OPSS.PROV 1010.
- Conventional (non-reinforced) granular fill embankments will be constructed with side slopes not steeper than 2H:1V.
- Granular fill embankments greater than 8 m in height will be provided with a 2 m wide mid-height berm.
- Permanent drainage and erosion protection will be provided for all granular embankment slopes.

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

For seismic analysis, Table 6.3 in Section 6.14.4.1 of the CHBDC indicates a minimum resistance factor of 0.95 ($\phi_{gu, static(temporary)} = 0.75 + 0.2$) for force-based design and 1.0 for performance-based design. Based on these values and Ψ of 1.0, a target Factor of Safety of 1.1 for this temporary condition with a typical degree of understanding is appropriate for the pseudo-static seismic analysis.

Slope stability assessments have been carried out for the highest/critical embankment slope, just behind the 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 9 m;
- For analysis a seismic event with a return period 475 years site adjusted PGA value of 0.078 g, equal to $\frac{1}{2}$ of the site adjusted PGA value (0.156 g) was used, as per Section 4.4.3.3 of the CHBDC; and,
- A traffic surcharge of 17 kPa applied as a temporary load.

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

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

Table 10-9 Slope Stability Analysis Results

Condition	Case	Factor of Safety
Permanent (traffic loading)	Long-Term Static (Drained)	1.5 (Fig G1)
Temporary (traffic loading)	Short-Term Static (Undrained)	1.5 (Fig G2)
Temporary (seismic)	Pseudo-Static Seismic, 475-yr (Undrained)	1.2 (Fig G3)

All of the static results presented in Table 10-9 achieve the target Factors of Safety described above. The pseudo-static result presented in Table 10-9 above exceeds the target Factor of Safety 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 475 year seismic event (Figure G3). It is also noted that liquefaction was flagged as a concern for the 1 in 2475 year seismic event (see Section 8.4) and will need to be considered in the slope stability modeling during detailed design once a site specific seismic assessment is carried out.

10.7 Embankment Settlement

The settlement resulting from a 9 m high embankment constructed with conventional granular fill on a new alignment approximately 20 m to the east of the existing alignment was assessed using the multi-layer settlement analysis in Rocscience's Settle3 software. Subsurface stratigraphy was based on the boreholes drilled in the area. Loading was applied based on the assumed geometry of the new alignment using a unit weight for new embankment SSM fill of 20 kN/m³. The water table was defined based on piezometer readings. It is noted that engineering judgment and experience was used to select the material properties based on the stress range anticipated due to loading. Soil parameters used in the analysis of the proposed new embankment are presented in Table 10-10 below.

Based on the parameters in Table 10-10 and loading from the assumed new embankment geometry, the expected total settlement is expected to be in the order of 200 mm at the centerline of the embankment just behind the abutment. The majority of the settlement is expected to be completed within two years of fill placement.

Based on the estimated settlement and time required for that settlement to occur, a preload period of at least 2 years is recommended prior to bridge construction. The embankment preload should be overbuilt to accommodate settlement during the preload period, the settlement should be monitored with monitoring points to confirm when the preload period is complete. Options such as wick drains to expedite the preload time period could be assessed during detailed design.

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

Property	Silt with Sand	Silty Clay	Silt
Unit Weight [kN/m ³]	19	17.5	19
E [kPa]	10,000	-	25,000
e _o	-	1.1	-
OCR	-	4.5	-
C _c	-	0.75	-
C _r	-	0.1	-
c _v [m ² /d]	-	8.6 x 10 ⁻⁵	-
c _{vr} [m ² /d]	-	8.6 x 10 ⁻⁵	-
C _a /C _c	-	0.03	-
B-bar	-	0.8	-

Provided the new embankment is constructed with a 2-year preload period prior to bridge construction, the approach embankments are expected to meet the MTO guidelines for post construction settlement of approach embankments behind bridge abutments over a period of 20 years after paving, as outlined below:

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

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

11 RECOMMENDED SCOPE FOR DETAIL DESIGN

The recommendations provided above are in support of the preliminary design of the proposed replacement of the Highway 401 Edward Street Underpass (Site No. 16X-0128/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. Seismic CPTu holes with periodic dissipation testing are recommended due to the sensitive nature of the silty clay to clayey silt layer and the requirement for a site-specific seismic assessment.
- A site-specific ground response analysis as input to the liquefaction assessment to provide a more detailed evaluation of the liquefaction potential at the proposed foundation locations.
- Testing of soil and/or groundwater at the site to determine degree of corrosiveness of the sub-surface environment and potential implications to steel and concrete elements in contact with the soil and groundwater at the site.
- Additional field and laboratory testing to acquire compressibility characteristics of the cohesive soils near the abutments and pier including collection of undisturbed Shelby Tube samples to allow consolidation and/or triaxial testing which will lead to more rigorous estimation of settlement and downdrag loads on abutment piles.

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

12 CLOSURE

Engineering analysis and preparation of this report was carried out by Christopher Murray, P.Eng 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 Foundation Projects.

Thurber Engineering Ltd.
Report Prepared By:



Christopher Murray, M.A.Sc., P.Eng.
Geotechnical Engineer



Matt Kennedy, M.Sc.(Eng.), P.Eng.
Senior Geotechnical Engineer



Paul Carnaffan, M.Eng., P.Eng.
Principal | Branch Manager
Senior Geotechnical Engineer



Fred Griffiths, Ph.D., P.Eng.
MTO Review Principal
Senior Geotechnical Engineer



REFERENCES

ⁱ <https://earthquakescanada.nrcan.gc.ca/hazard-alea/interpolat/calc-en.php>

ⁱⁱ Boulanger, R. W., & Idriss, I. M. (2007). Evaluation of cyclic softening in silts and clays. *Journal of geotechnical and geoenvironmental engineering*, 133(6), 641-652.

ⁱⁱⁱ Boulanger, R. W., and Idriss, I. M. (2014). CPT and SPT based liquefaction triggering procedures, Report No. UCD/CGM-14/01, Center for Geotechnical Modeling, Department of Civil and Environmental Engineering, University of California, Davis, CA, 134 pp.

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.

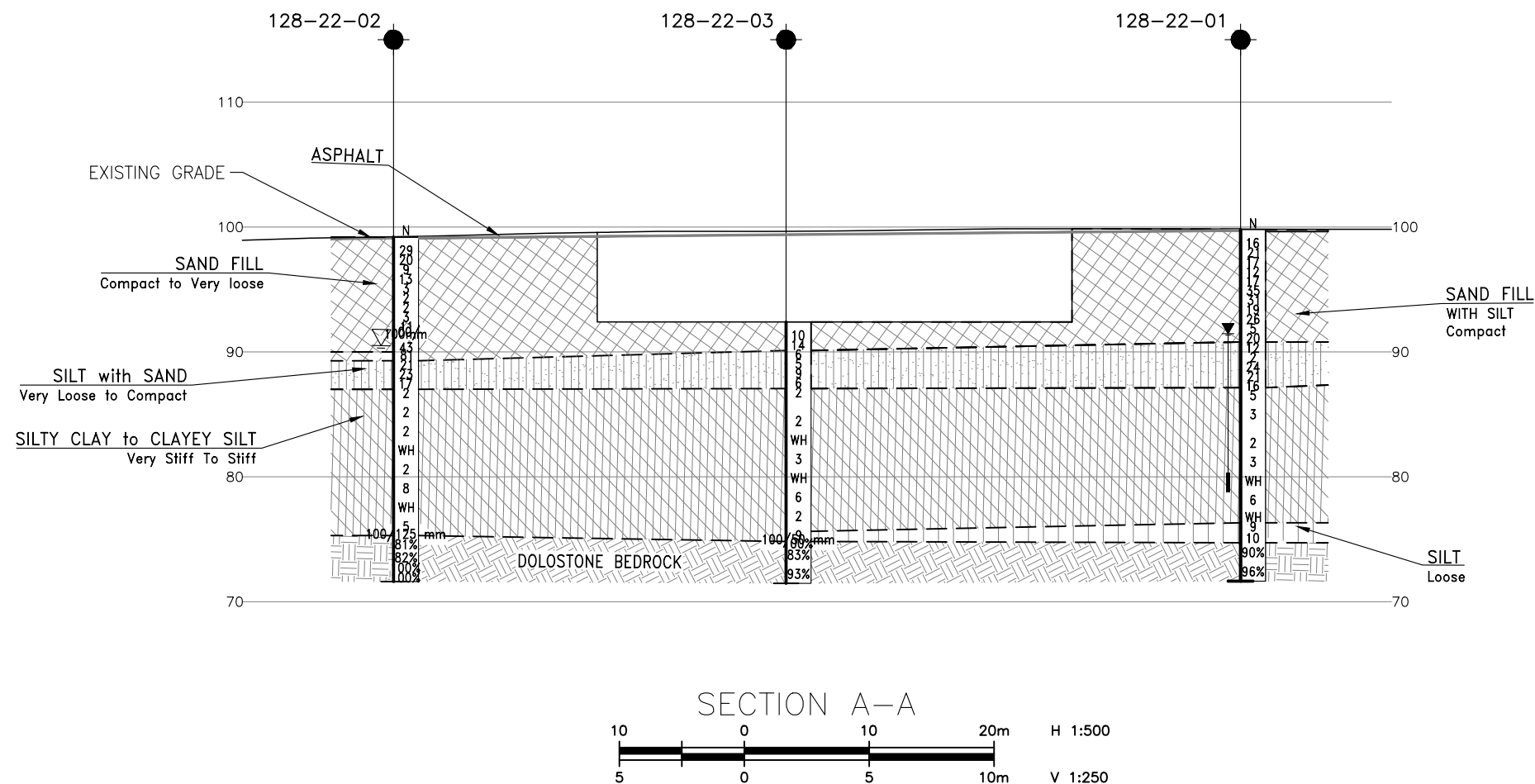
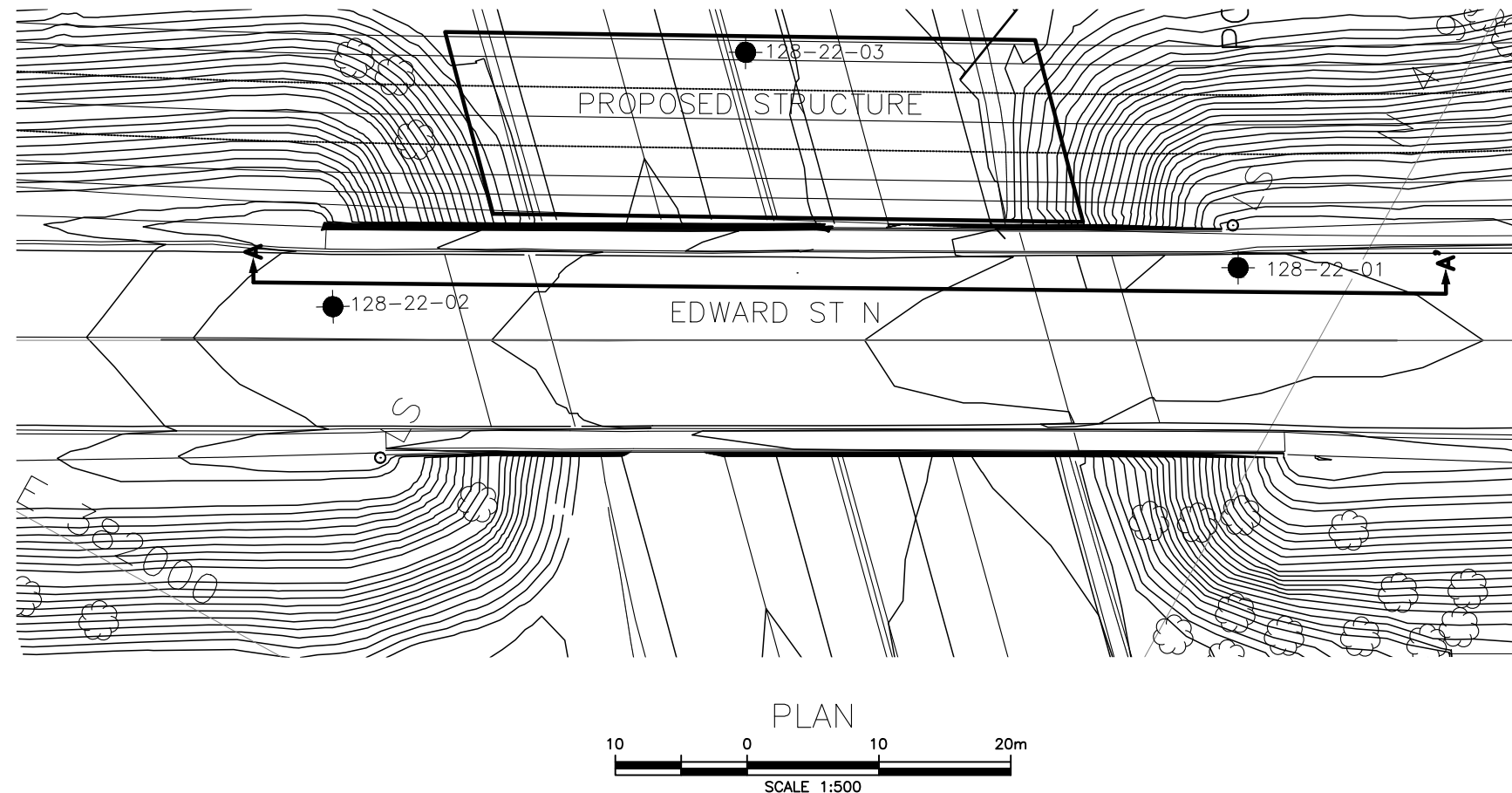
7. INDEPENDENT JUDGEMENTS OF CLIENT

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.

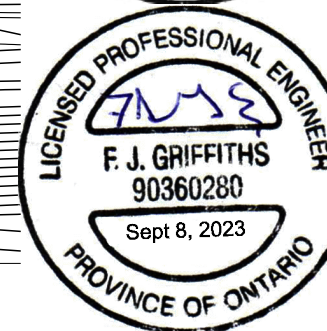


Appendix A.

Borehole Location Plan and Stratigraphic Drawing



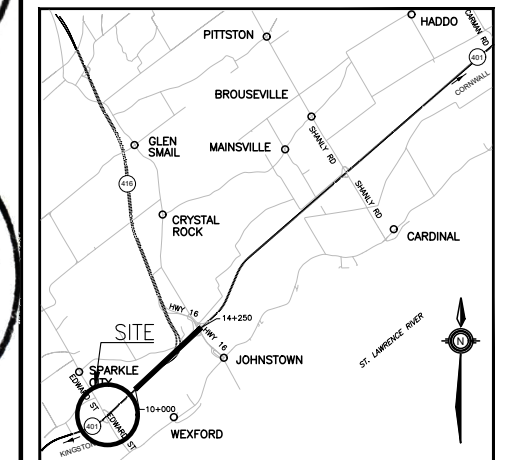
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



CONT No
GWP No 4024-20-00





HIGHWAY 401
EDWARD STREET UNDERPASS
REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

Ontario 



KEYPLAN

LEGEND

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

NO	ELEVATION	NORTHING	EASTING
128-22-01	99.8	4 954 109.0	382 060.9
128-22-02	99.2	4 954 166.8	382 025.2
128-22-03	92.4	4 954 148.4	382 057.1

-NOTES-

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

GEOCRES No. 31B-111

REVISIONS							
	DATE	BY	DESCRIPTION				
DESIGN	MJK	CHK -	CODE	LOAD	DATE	JUNE 203	
DRAWN	JW	CHK MJK	SITE 16X-0128	STRUCT	DWG	1	



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).

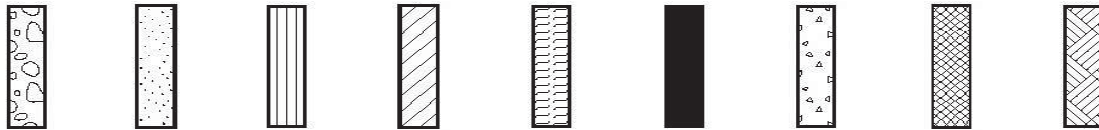
DYNAMIC CONE PENETRATION TEST (DCPT):

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 128-22-01

1 OF 3

METRIC

GWP# 4024-20-00 LOCATION Lat: 44.722821°, Long: -75.524708° Edward Street Underpass, MTM z9: N 4 954 109.0 E 382 060.9 ORIGINATED BY IK
 HWY 401 BOREHOLE TYPE CME 55 Truck Mount HSA / NW Casing / NQ Coring COMPILED BY RH
 DATUM Geodetic DATE 2022.11.22 - 2022.11.24 CHECKED BY CM

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											
99.8	Ground Surface							20	40	60	80	100							
0.0	ASPHALT (150 mm)							20	40	60	80	100							
0.2	SILTY SAND with gravel compact brown FILL						99											18 42 40 (SI+CL)	
98.3																			
1.5	SAND with silt loose to dense brown FILL						98												
							97												
							96											0 94 6 (SI+CL)	
							95												
							94												
							93											0 92 8 (SI+CL)	
							92												
91.6							91												
8.2	WOOD with silty sand and gravel infills FILL																		
91.3																			
90.5	ASPHALT (150 mm)		11	SS	20														
8.7																			
90.8	SILTY SAND compact grey FILL																		
9.0																			
	SANDY SILT (ML) trace clay very loose to compact grey		12	SS	12		90											0 39 53 8	

Continued Next Page

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

DOUBLE LINE 29381 EDWARD STREET UNDERPASS.GPJ 2012TEMPLATE(MTO).GDT 9-5-23

METRIC

[illegible]


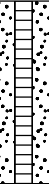
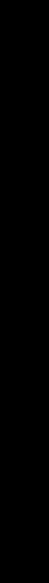



+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 128-22-01

3 OF 3

METRIC

GWP# 4024-20-00 LOCATION Lat: 44.722821°, Long: -75.524708° Edward Street Underpass, MTM z9: N 4 954 109.0 E 382 060.9 ORIGINATED BY IK
 HWY 401 BOREHOLE TYPE CME 55 Truck Mount HSA / NW Casing / NQ Coring COMPILED BY RH
 DATUM Geodetic DATE 2022.11.22 - 2022.11.24 CHECKED BY CM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE										
Continued From Previous Page								20 40 60 80 100										
	SILTY CLAY (CI) to CLAYEY SILT (CL) very stiff to stiff grey wet		21	SS	WH													
76.3			22	SS	6													
23.5	SILT (ML) some clay loose to compact grey		24	SS	9													
74.7			25	SS	10													
25.1	DOLOSTONE BEDROCK occasional calcite infilled joints fresh fine to medium grained grey medium bedded very strong		1	RUN	-													
71.7			2	RUN	-													
28.1	End of Borehole Flushmount standpipe piezometer consists of a 19 mm diameter Schedule 40 PVC pipe with a 1.5-m slotted screen. Water level readings: DATE DEPTH (m) ELEV. (m) 2022.12.18 8.3 91.5 2023.04.26 8.2 91.6																	

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

DOUBLE LINE 29381 EDWARD STREET UNDERPASS.GPJ 2012TEMPLATE(MTO).GDT 9-5-23

RECORD OF BOREHOLE No 128-22-02

1 OF 3

METRIC

GWP# 4024-20-00 LOCATION Lat: 44.723344°, Long: -75.52515° Edward Street Underpass, MTM Z9: N 4 954 166.8 E 382 025.2 ORIGINATED BY IK
 HWY 401 BOREHOLE TYPE CME 55 Truck Mount HSA / NW Casing / NQ Coring COMPILED BY RH
 DATUM Geodetic DATE 2022.11.24 - 2022.11.28 CHECKED BY CM

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					
99.2	Ground Surface							20 40 60 80 100					
0.0	ASPHALT (150 mm)							20 40 60 80 100					
0.2	SAND with silt compact to very loose brown FILL							20 40 60 80 100					
			1	SS	29		99						
			2	SS	20		98						
			3	SS	9		97						0 95 5 (SI+CL)
			4	SS	13		96						
			5	SS	3		95						
			6	SS	2		94						
			7	SS	2		93						2 94 4 (SI+CL)
			8	SS	3		92						
			9	SS	11		91						
			10	SS	100/		90						
					100mm								
91.1	- Hollow stem auger refusal at 8.1 m												
8.1	CONCRETE		1	NQ	-		91						
90.7													
8.5	WOOD with silty sand and gravel infills FILL		11	SS	43								
			12	SS	81		90						
89.3													

Continued Next Page

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

RECORD OF BOREHOLE No 128-22-02

2 OF 3

METRIC

GWP# 4024-20-00 LOCATION Lat: 44.723344°, Long: -75.52515° Edward Street Underpass, MTM Z9: N 4 954 166.8 E 382 025.2 ORIGINATED BY IK
 HWY 401 BOREHOLE TYPE CME 55 Truck Mount HSA / NW Casing / NQ Coring COMPILED BY RH
 DATUM Geodetic DATE 2022.11.24 - 2022.11.28 CHECKED BY CM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)								
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)				GR	SA	SI	CL					
								20 40 60 80 100				w _p w w _L												
								○ UNCONFINED + FIELD VANE																
								● QUICK TRIAXIAL × LAB VANE																
								20 40 60 80 100				20 40 60												
9.9	Continued From Previous Page		13	SS	21		89											0	29	65	6			
			14	SS	23																			
			15	SS	17																			
87.0							87																	
12.2	SILTY CLAY (CI) very stiff grey		16	SS	2																			
			17	SS	2																0	1	49	50
			18	SS	2																			
			19	SS	WH																			

Continued Next Page

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

RECORD OF BOREHOLE No 128-22-02

3 OF 3

METRIC

GWP# 4024-20-00 LOCATION Lat: 44.723344°, Long: -75.52515°
Edward Street Underpass, MTM z9: N 4 954 166.8 E 382 025.2 ORIGINATED BY IK
HWY 401 BOREHOLE TYPE CME 55 Truck Mount HSA / NW Casing / NQ Coring COMPILED BY RH
DATUM Geodetic DATE 2022.11.24 - 2022.11.28 CHECKED BY CM

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		
	Continued From Previous Page							SHEAR STRENGTH kPa						
								○ UNCONFINED + FIELD VANE						
								● QUICK TRIAXIAL × LAB VANE						
								WATER CONTENT (%)						
								20	40	60				
75.3	SILTY CLAY (Cl) very stiff grey		21	SS	8		79							0 0 40 60
			22	SS	WH		78							
			23	SS	5		77							
			24	SS	100/		76							
23.9	DOLOSTONE BEDROCK fresh fine to medium grained grey medium bedded very strong				125mm		75							
			1	RUN	-									RUN #1 TCR=100% SCR=92% RQD=81%
			2	RUN	-		74							RUN #2 TCR=100% SCR=71% RQD=82%
			3	RUN	-		73							RUN #3 TCR=100% SCR=94% RQD=100%
			4	RUN	-		72							RUN #4 TCR=100% SCR=100% RQD=100%
71.6	End of Borehole Water level after 60 hours at Elev. 90.5 m on completion of drilling													
27.6														

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

METRIC

[illegible]

+³, ×³: Numbers refer to Sensitivity




DOUBLE LINE 29381 EDWARD STREET UNDERPASS.GPJ 2012TEMPLATE(MTO).GDT 9-5-23

RECORD OF BOREHOLE No 128-22-03

2 OF 3

METRIC

GWP# 4024-20-00 LOCATION Lat: 44.723176°, Long: -75.524749° Edward Street Underpass, MTM z9: N 4 954 148.4 E 382 057.1 ORIGINATED BY IK
 HWY 401 BOREHOLE TYPE CME 55 Truck Mount HSA / NW Casing / NQ Coring COMPILED BY RH
 DATUM Geodetic DATE 2022.12.07 - 2022.12.08 CHECKED BY CM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)			
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE												
	Continued From Previous Page						20	40	60	80	100	20	40	60						
	CLAYEY SILT (CL) very stiff to stiff grey																			
			11	SS	3															
			12	SS	WH															
			13	SS	6															
			14	SS	2															
75.6																				
16.8	SILT (ML) loose grey		15	SS	9															
74.8			16	SS	100/															
17.6	DOLOSTONE BEDROCK fresh fine to medium grained grey medium bedded very strong		1	RUN	50mm -															
			2	RUN	-															

Continued Next Page

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

DOUBLE LINE 29381 EDWARD STREET UNDERPASS.GPJ 2012TEMPLATE(MTO).GDT 9-5-23

3 OF 3

METRIC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE LIQUID CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× LAB VANE						
	Continued From Previous Page						20	40	60	80	100						
71.5	DOLOSTONE BEDROCK fresh fine to medium grained grey medium bedded very strong		3	RUN	-												1
20.9	End of Borehole A representative open-hole groundwater level measurement was not obtained due to the introduction of water during drilling.																1
																	2

+³, ×³: Numbers refer to Sensitivity



Appendix C.

Laboratory Testing

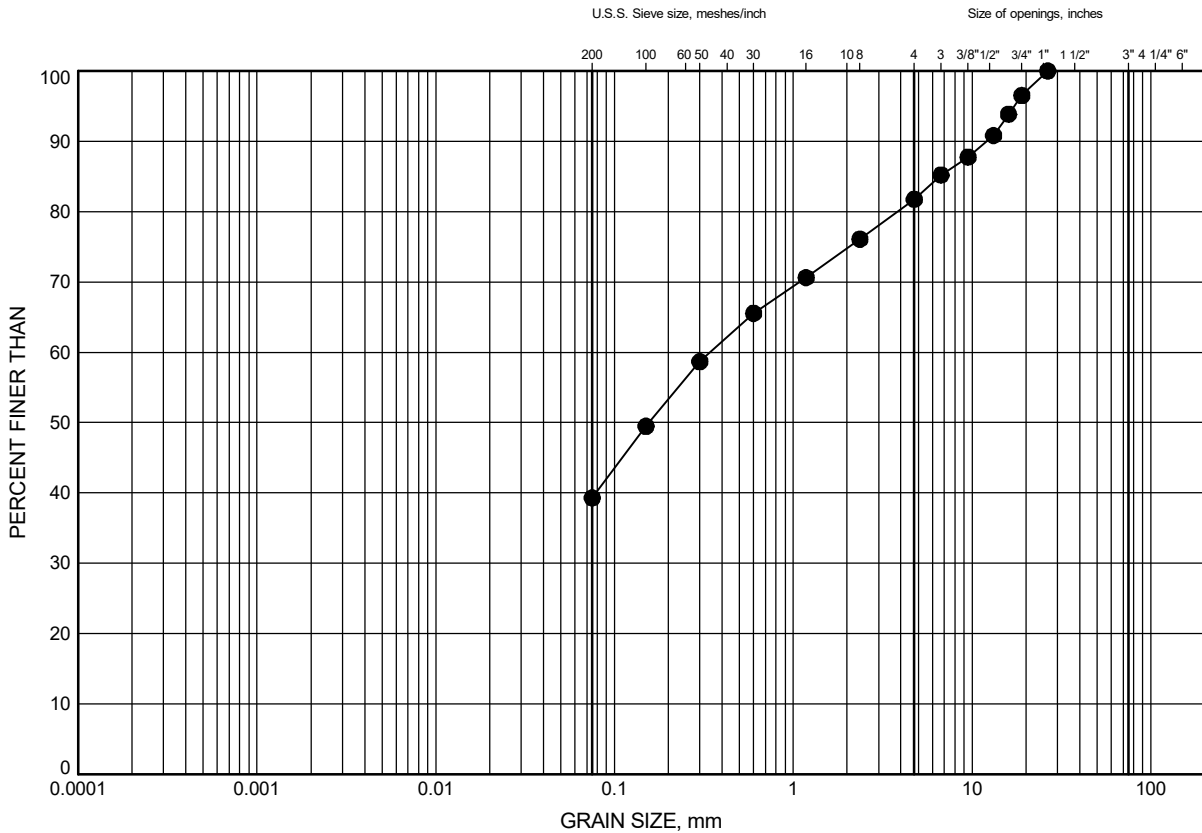


Appendix C.1
Particle Size Analysis Figures
Atterberg Limit Test Results

Highway 401 Edward Street Underpass GRAIN SIZE DISTRIBUTION

FIGURE C1

FILL: Silty Sand with Gravel



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	128-22-01	1.1	98.7

Date March 2023
GWP# 4024-20-00

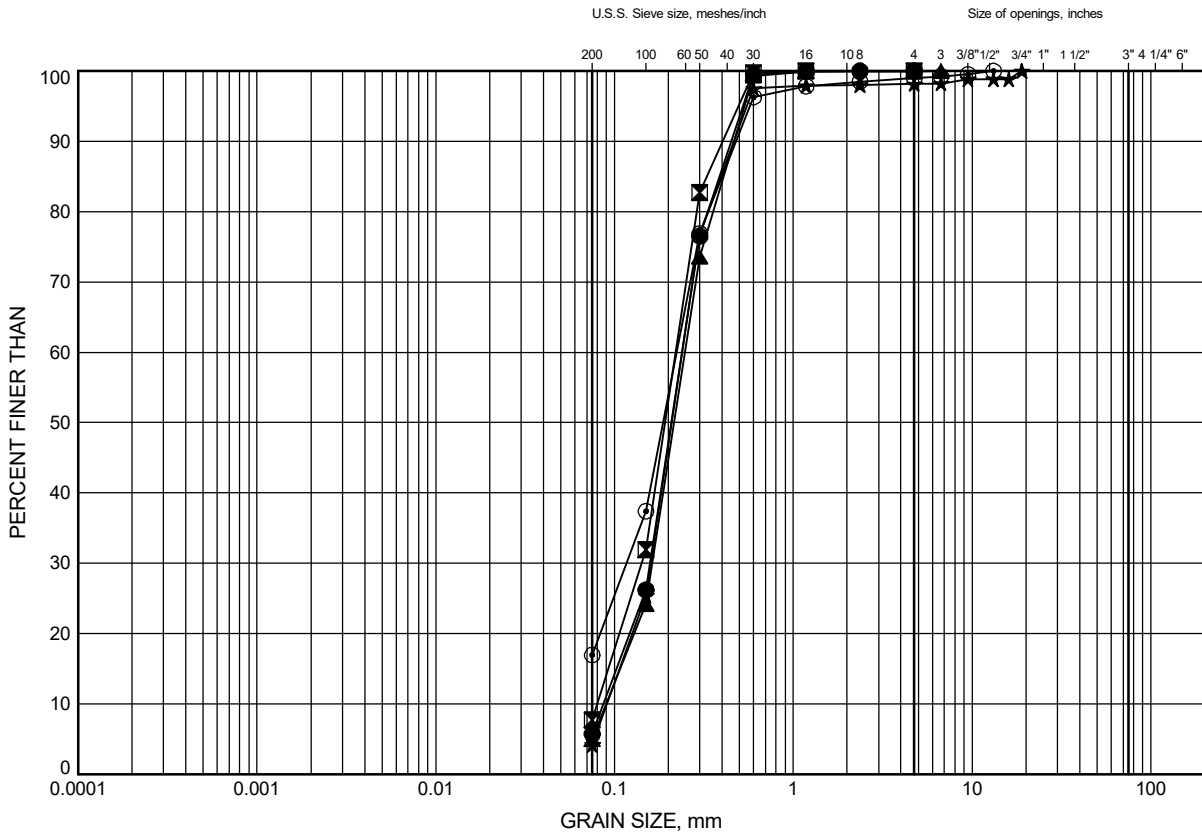


Prep'd RH
Chkd. CM

Highway 401 Edward Street Underpass GRAIN SIZE DISTRIBUTION

FIGURE C2

FILL: Sand with Silt to Silty Sand



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	128-22-01	3.4	96.4
⊠	128-22-01	7.2	92.6
▲	128-22-02	2.6	96.6
★	128-22-02	6.4	92.8
⊙	128-22-03	1.1	91.3

Date March 2023
GWP# 4024-20-00

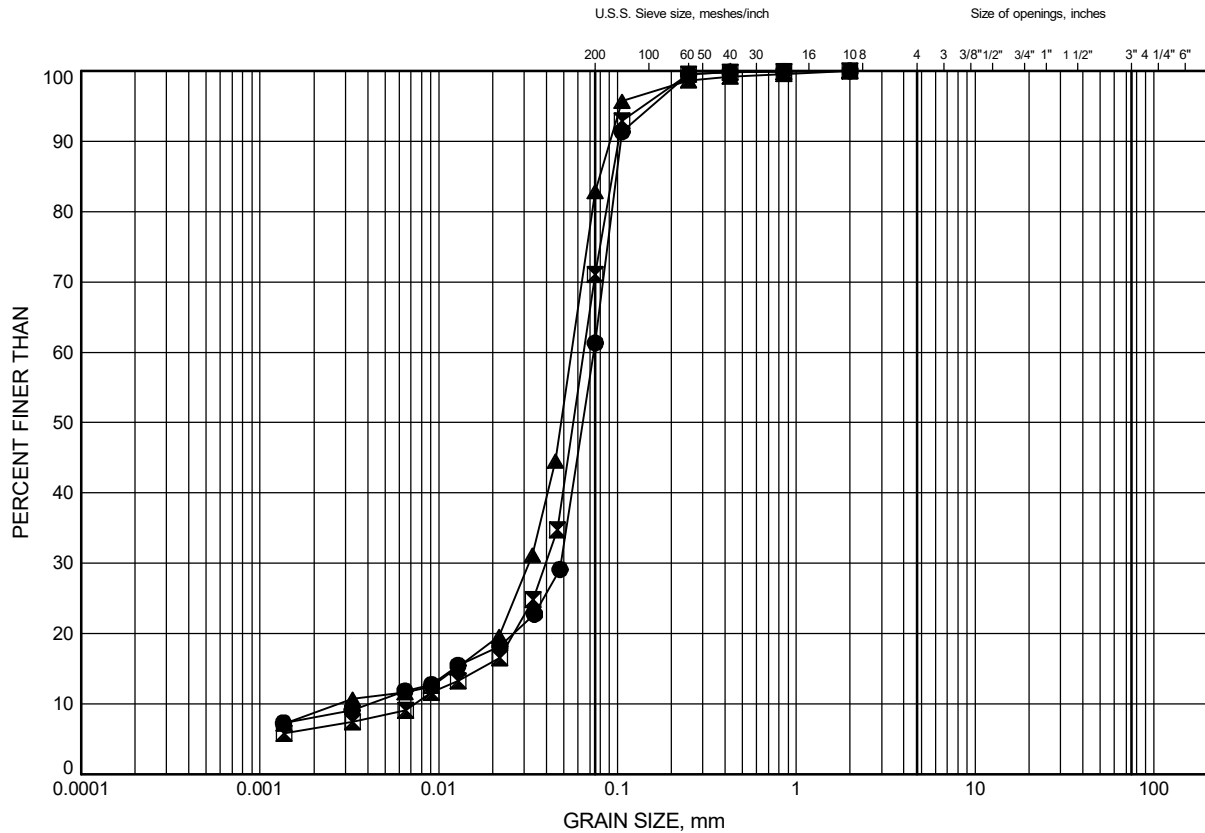


Prep'd RH
Chkd. CM

Highway 401 Edward Street Underpass GRAIN SIZE DISTRIBUTION

FIGURE C3

Sandy Silt to Silt with Sand



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	128-22-01	9.4	90.4
⊠	128-22-02	10.2	89.0
▲	128-22-03	4.9	87.5

Date March 2023
GWP# 4024-20-00

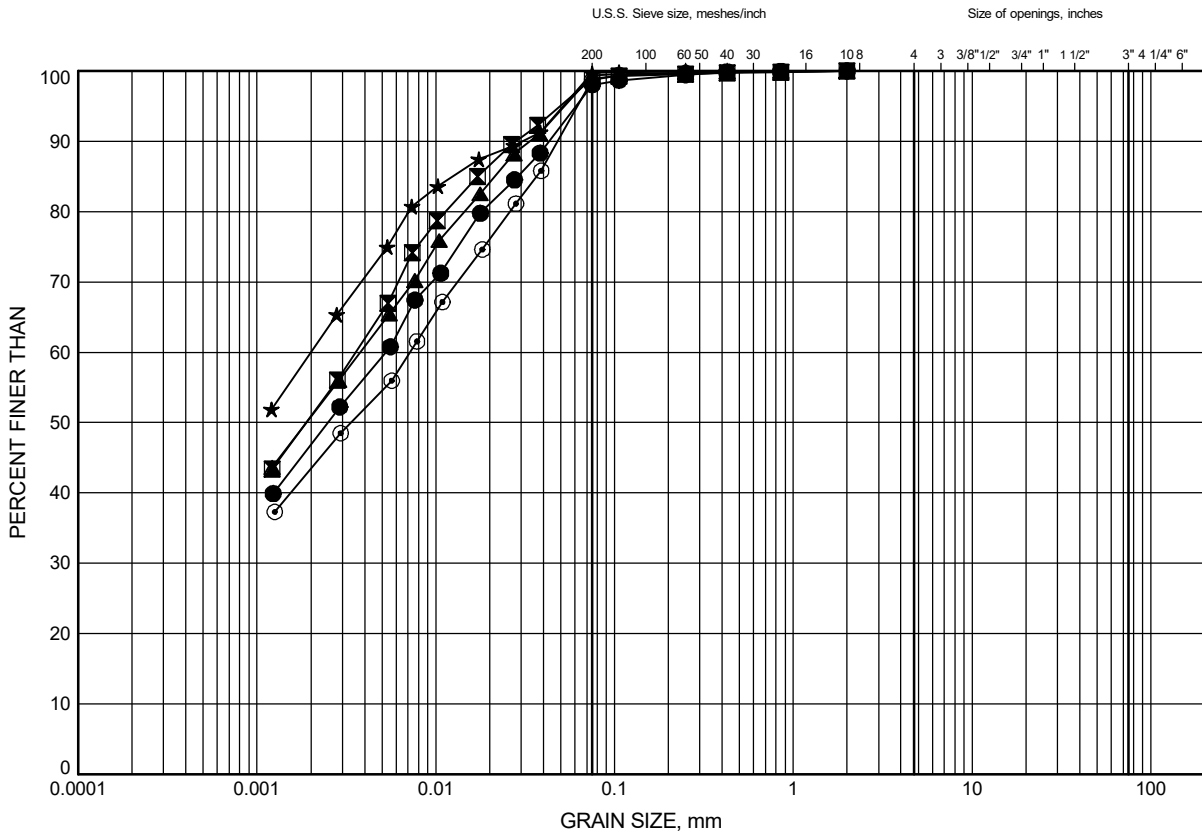


Prep'd RH
Chkd. CM

Highway 401 Edward Street Underpass GRAIN SIZE DISTRIBUTION

FIGURE C4

Silty Clay to Clayey Silt



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	128-22-01	13.3	86.5
⊠	128-22-01	23.0	76.8
▲	128-22-02	14.0	85.2
★	128-22-02	20.1	79.1
⊙	128-22-03	5.6	86.8

Date March 2023

GWP# 4024-20-00



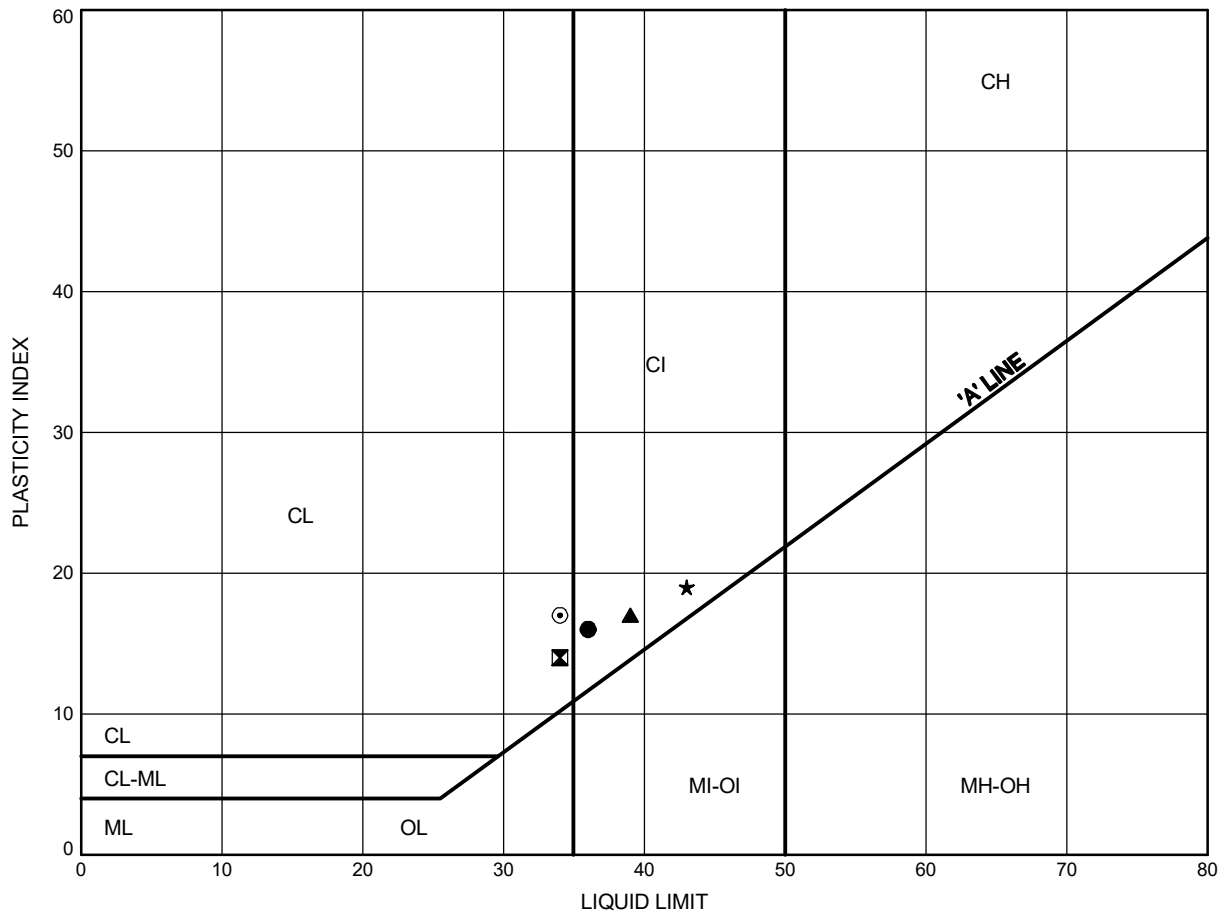
Prep'd RH

Chkd. CM

Highway 401 Edward Street Underpass ATTERBERG LIMITS TEST RESULTS

FIGURE C5

Silty Clay to Clayey Silt



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	128-22-01	13.3	86.5
⊠	128-22-01	23.0	76.8
▲	128-22-02	14.0	85.2
★	128-22-02	20.1	79.1
⊙	128-22-03	5.6	86.8

Date March 2023

GWP# 4024-20-00

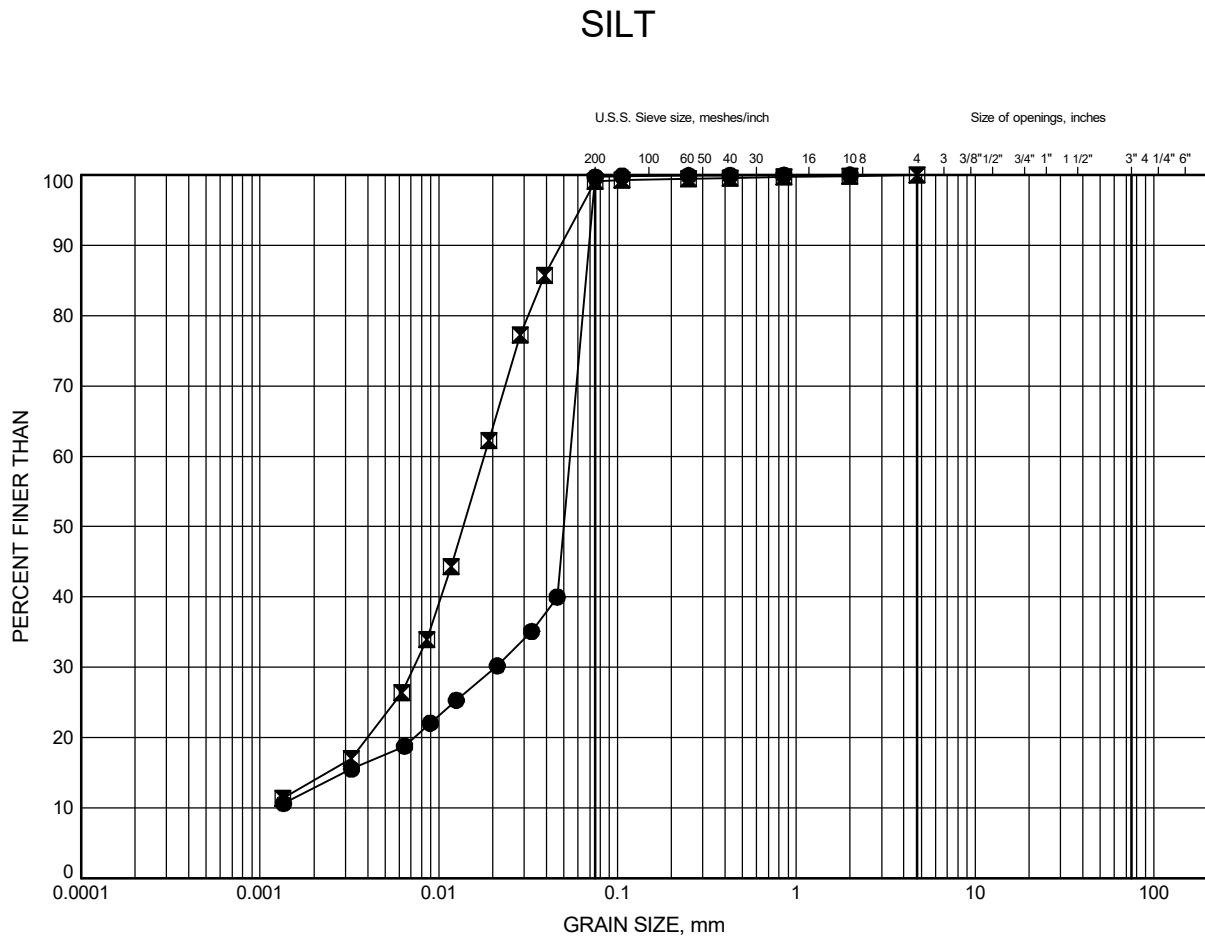


Prep'd RH

Chkd. CM

Highway 401 Edward Street Underpass GRAIN SIZE DISTRIBUTION

FIGURE C6



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	128-22-01	24.7	75.1
⊠	128-22-03	17.5	74.9

Date March 2023
GWP# 4024-20-00



Prep'd RH
Chkd. CM



Appendix C.2

UCS Test Results



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

March 2, 2023
File: 122410864

Client: Thurber Engineering, File #29381

**Reference: ASTM D7012, Method C, Unconfined Compressive Strength of Intact Rock Core
Hwy 401, Maitland**

The following table summarizes unconfined compressive strength results for one intact rock core.

Location	Sample Depth	Compressive Strength (MPa)	Description of Break
BH128.22.1 Run-2	89'-89'7"	174.3	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 128-22-01

RUN 1

Depth 25.1 m to 26.6 m

Elevation 74.7 m to 73.2 m

Dry Sample

Run 1 Start
elev. 74.7 m



Run 1 End
elev. 73.2 m



THURBER ENGINEERING LTD.

Highway 401 Underpass at Edward Street
(Site No. 16X-0128)
Assignment No. 4019-E-0010.2, GWP 4024-20-00

BH 128-22-01
Project No.: 29381

Borehole 128-22-01

RUN 1

Depth 25.1 m to 26.6 m

Elevation 74.7 m to 73.2 m

Wet Sample

Run 1 Start
elev. 74.7 m



Run 1 End
elev. 73.2 m



THURBER ENGINEERING LTD.

Highway 401 Underpass at Edward Street
(Site No. 16X-0128)
Assignment No. 4019-E-0010.2, GWP 4024-20-00

BH 128-22-01
Project No.: 29381

Borehole 128-22-01

RUN 2

Depth 26.6 m to 28.1 m

Elevation 73.2 m to 71.7 m

Dry Sample

Run 1 Start
elev. 73.2 m



Run 1 End
elev. 71.7 m



THURBER ENGINEERING LTD.

Highway 401 Underpass at Edward Street
(Site No. 16X-0128)
Assignment No. 4019-E-0010.2, GWP 4024-20-00

BH 128-22-01
Project No.: 29381

Borehole 128-22-01

RUN 2

Depth 26.6 m to 28.1 m

Elevation 73.2 m to 71.7 m

Wet Sample

Run 1 Start
elev. 73.2 m



Run 1 End
elev. 71.7 m



THURBER ENGINEERING LTD.

Highway 401 Underpass at Edward Street
(Site No. 16X-0128)
Assignment No. 4019-E-0010.2, GWP 4024-20-00

BH 128-22-01
Project No.: 29381

Borehole 128-22-02

NQ1 – Concrete Sample

Depth 8.1 m to 8.5 m

Elevation 91.1 m to 90.7 m

Dry Sample

NQ1 Start
elev. 91.1 m

NQ1 End
elev. 90.7 m



Borehole 128-22-02

RUN 1

Depth 23.9 m to 25.3 m

Elevation 75.3 m to 73.9 m

Dry Sample

Run 1 Start
elev. 75.3 m



Run 1 End
elev. 73.9 m

Borehole 128-22-02

RUN 1

Depth 23.9 m to 25.3 m

Elevation 75.3 m to 73.9 m

Wet Sample

Run 1 Start
elev. 75.3 m



Run 1 End
elev. 73.9 m



THURBER ENGINEERING LTD.

Highway 401 Underpass at Edward Street
(Site No. 16X-0128)
Assignment No. 4019-E-0010.2, GWP 4024-20-00

BH 128-22-02
Project No.: 29381

Borehole 128-22-02

RUNS 2, 3, and 4

Depth 25.3 m to 27.6 m

Elevation 73.9 m to 71.6 m

Dry Sample

Run 2 Start
elev. 73.9 m



Run 2 End
elev. 73.2 m

Run 3 Start
elev. 73.2 m

Run 3 End
elev. 72.3 m



Run 4 Start
elev. 72.3 m

Run 4 End
elev. 71.6 m

Borehole 128-22-02

RUNS 2, 3, and 4

Depth 25.3 m to 27.6 m

Elevation 73.9 m to 71.6 m

Wet Sample

Run 2 Start
elev. 73.9 m



Run 2 End
elev. 73.2 m

Run 3 Start
elev. 73.2 m

Run 3 End
elev. 72.3 m



Run 4 Start
elev. 72.3 m

Run 4 End
elev. 71.6 m



THURBER ENGINEERING LTD.

Highway 401 Underpass at Edward Street
(Site No. 16X-0128)
Assignment No. 4019-E-0010.2, GWP 4024-20-00

BH 128-22-02
Project No.: 29381

Borehole 128-22-03

RUNS 1 and 2

Depth 17.6 m to 19.4 m

Elevation 74.8 m to 73.0 m

Dry Sample

Run 1 Start
elev. 74.8 m

Run 1 End
elev. 74.5 m



Run 2 Start
elev. 74.5 m



Run 3 End
elev. 73.0 m



THURBER ENGINEERING LTD.

Highway 401 Underpass at Edward Street
(Site No. 16X-0128)
Assignment No. 4019-E-0010.2, GWP 4024-20-00

BH 128-22-03
Project No.: 29381

Borehole 128-22-03

RUNS 1 and 2

Depth 17.6 m to 19.4 m

Elevation 74.8 m to 73.0 m

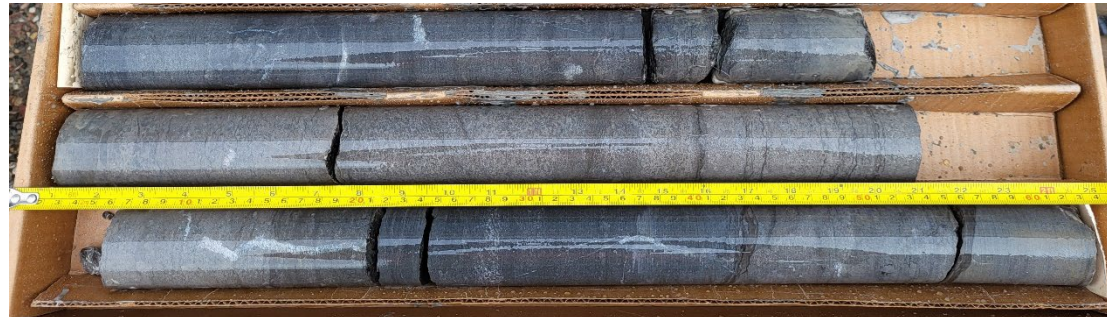
Wet Sample

Run 1 Start
elev. 74.8 m

Run 1 End
elev. 74.5 m



Run 2 Start
elev. 74.5 m



Run 3 End
elev. 73.0 m

Borehole 128-22-03

RUN 3

Depth 19.4 m to 20.9 m

Elevation 73.0 m to 71.5 m

Dry Sample

Run 3 Start
elev. 73.0 m



Run 3 End
elev. 71.5 m

Borehole 128-22-03

RUN 3

Depth 19.4 m to 20.9 m

Elevation 73.0 m to 71.5 m

Wet Sample

Run 3 Start
elev. 73.0 m



Run 3 End
elev. 71.5 m



Appendix D.

Selected Site Photographs



Photo 1: Looking southwest at bridge (2022/09/01)



Photo 2: Looking north along west side of bridge (2022/12/19)



Photo 3: Looking north along northbound Edward Street (2022/12/08)



Photo 4: Looking south along Edward Street (2022/09/01)



Photo 5: Looking west at northeast embankment side slope (2022/09/01)



Photo 6: Looking east at southwest embankment side slope (2022/12/19)



Photo 7: Looking north from east side of bridge (2022/12/19)



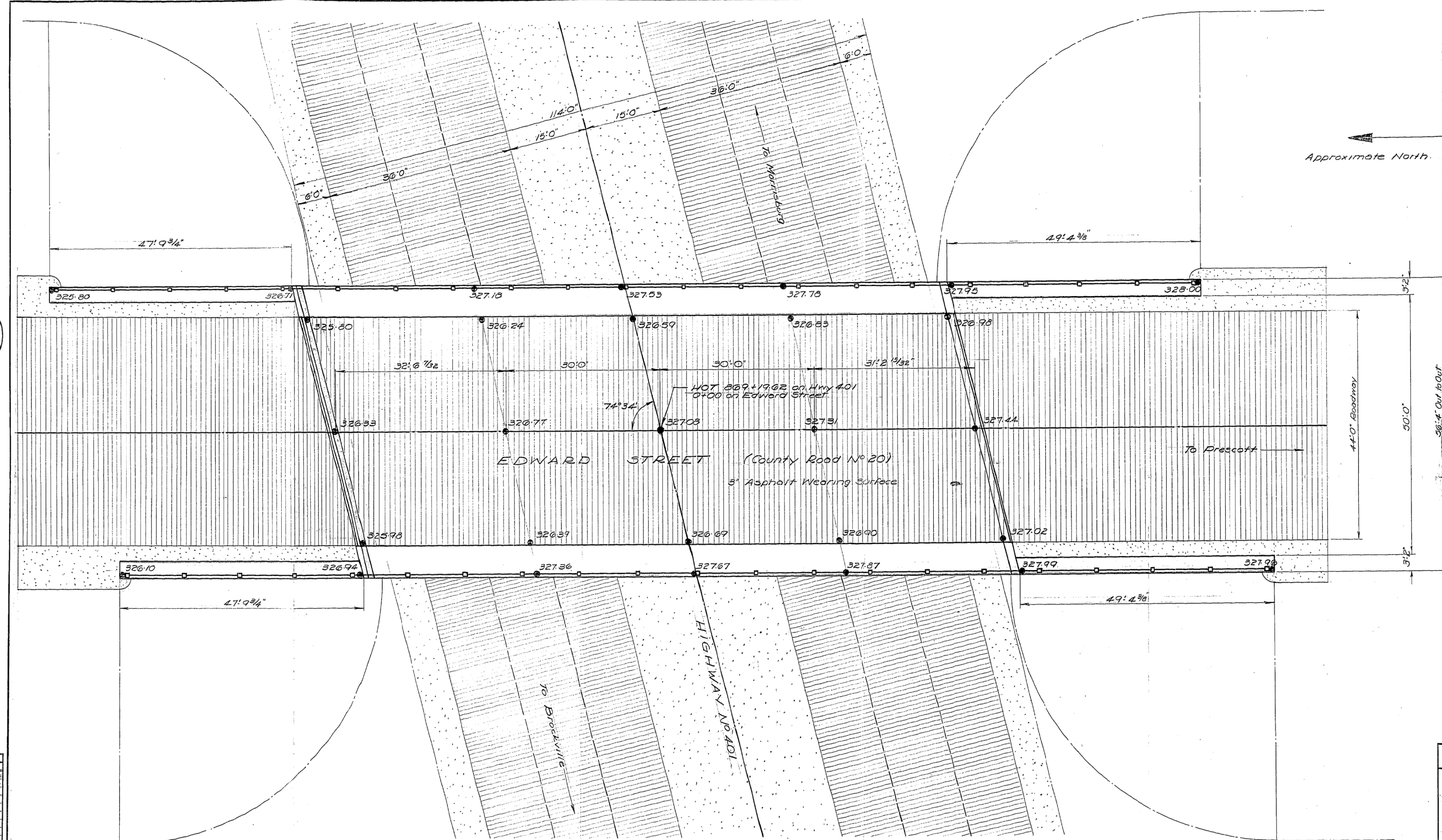
Appendix E.

Existing Structure – General Arrangement and Elevation Drawings Historical GEOCRETS Borehole Information

J.D. LEE & COMPANY LTD.	
CONSULTING ENGINEERS	KINGSTON, ONTARIO
DESIGN	SCALE
DWN.	DATE
CHK.	FILE
TRCD.	TOTAL
DATE	2



PRINT RECORD		
NO.	FOR	DATE
1	DESIGNED	17-10-55
2	CHECKED	12-2-57



PLAN
Scale 1/8 inch = 1 foot

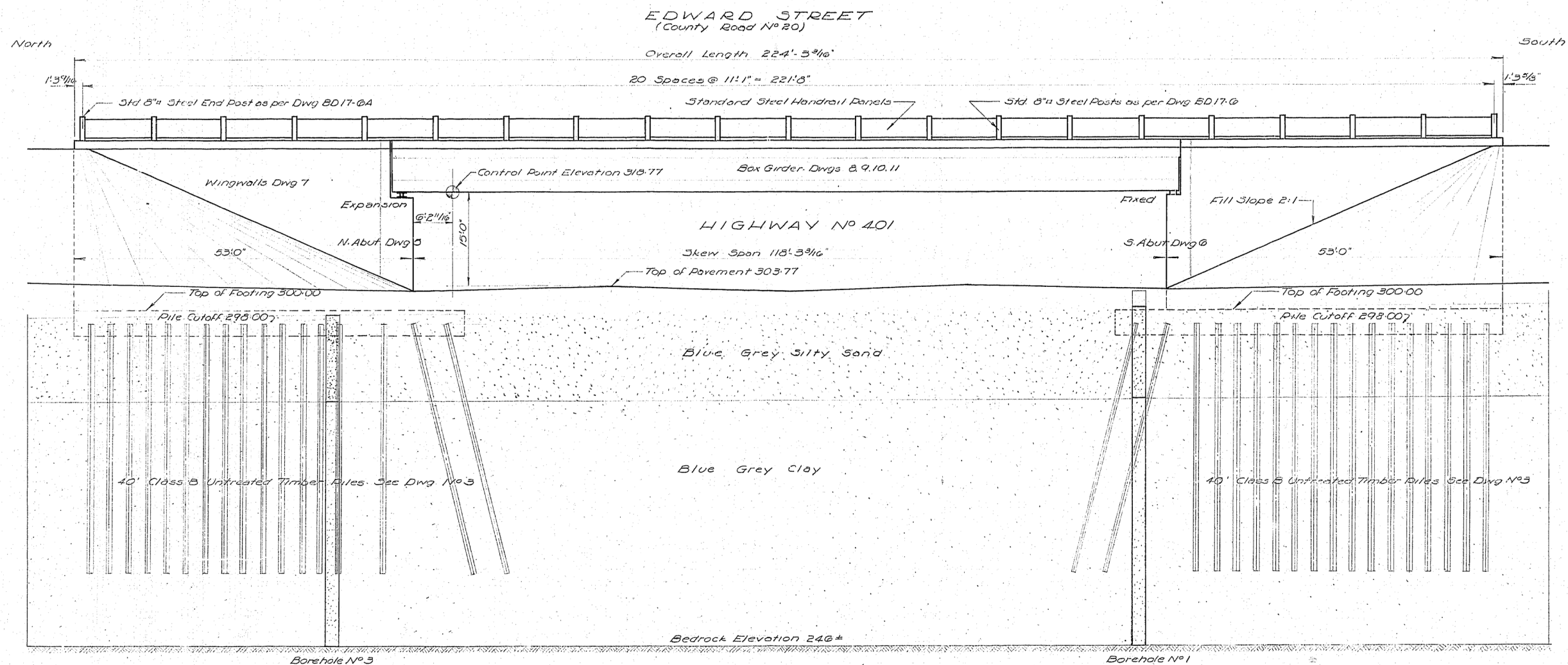
CONTR. No. 78-357
SHEET No. 17

DEPARTMENT OF HIGHWAYS-ONTARIO			
BRIDGE OFFICE-TORONTO			
AUGUSTA TOWNSHIP No. 5			
UNDERPASS			
HWY No. 401 & EDWARD STREET			
THE KING'S HIGHWAY No. 401		DIV. No. 5	
CO. GRENVILLE			
TWP. AUGUSTA		LOT 263	CON. I
GENERAL PLAN			
APPROVED <i>[Signature]</i>			
BRIDGE ENGINEER		CHIEF ENGINEER	
DESIGN	N.D.G.	CHECK	J.D.L.
DRAWING	N.D.G.	CHECK	J.F.L.
TRACING	N.D.G.	CHECK	J.F.L.
DATE	5-10-56	LOADING	H20-S16
CONTRACT NUMBER		DRAWING NUMBER	
		D3761-2	

30-12-1
T.O.D.

PROVISIONAL
AS LEE
ON

COND.	DATE
1	12-12-55
2	12-12-55
3	12-12-55
4	12-12-55
5	12-12-55
6	12-12-55
7	12-12-55
8	12-12-55
9	12-12-55
10	12-12-55



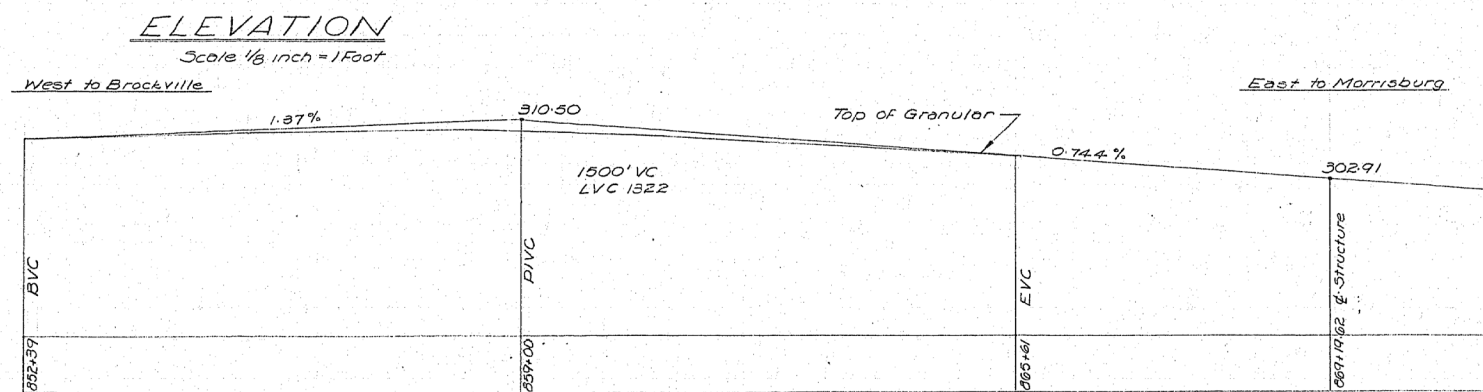
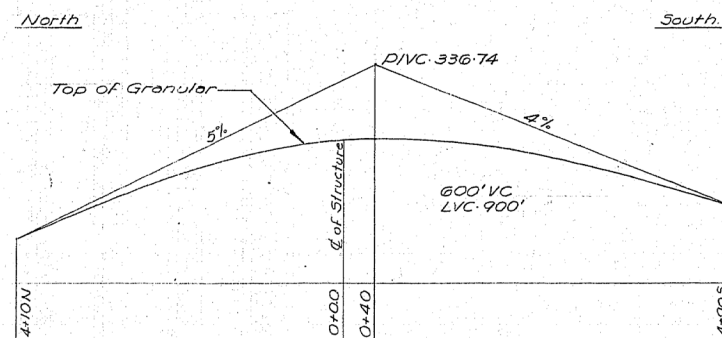
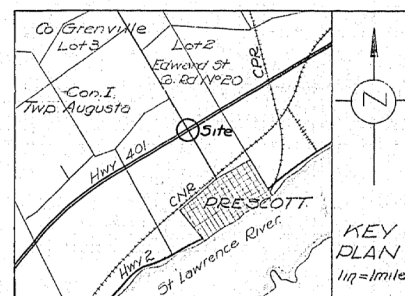
NOTES:
NOTE TO DIVISION ENGINEER
Construction work on this structure must not be commenced until monuments to fix control points have been erected and checked by the Division Engineer.

NOTE TO CONTRACTOR
Structure to be built in accordance with the "General Specifications for Highway Bridges, Ontario, 1935" Form No 9 and the Special Specifications attached to the "Information to Bidders" sheet, extra copies of which may be obtained from the Division Engineer.

CONCRETE MIX:
All concrete shall test 5000 psi at 28 days.
Add 1/4 lb Presolith "S" per bag of cement.
Max Aggregate sizes "Gravel 3/4"
All other 1 1/2"

FUNCTIONS OF 15° 20'
Sine 0.26012
Cosine 0.96394
Tangent 0.27007
Handrail panels and posts at expansion joint to be set to take changes in length due to temperature. Secure with lock-nuts.

NOTE:
1. Approach fill to be placed in stages either before or after construction of the structure.
2. First stage of the approach fill not to exceed a height of 20 feet.



DEPARTMENT OF HIGHWAYS-ONTARIO
BRIDGE OFFICE-TORONTO

AUGUSTA TOWNSHIP NO UNDERPASS
HWY No 401 & EDWARD STREET

THE KING'S HIGHWAY No. 401 DIV. N
CO. GRENVILLE
TWP. AUGUSTA LOT 2 & 3 CON. I

ELEVATION & PROFILES

APPROVED
[Signature]
BRIDGE ENGINEER

CHIEF ENGINEER

DESIGN	N.D.G.	CHECK	J.D.L.
DRAWING	N.D.G.	CHECK	J.F.L.
TRACING	N.D.G.	CHECK	J.F.L.

DATE 5-10-56

REVISIONS

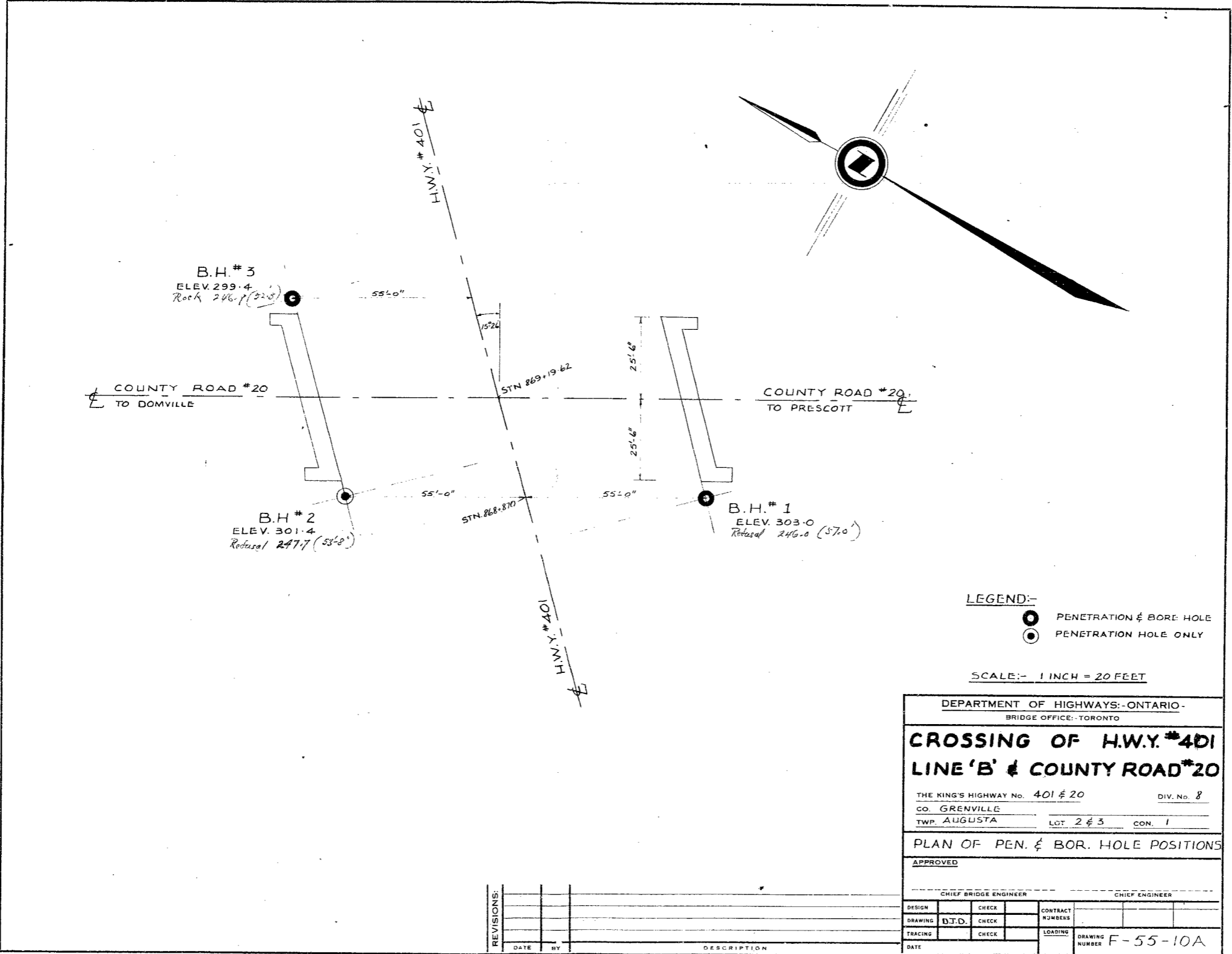
NO.	DATE	BY	DESCRIPTION
1			
2			
3			
4			
5			
6			
7			
8			
9			
10			

REFERENCE PLANS
E-2608-1
F-3417-4
F-3417-5

CONTRACT NUMBERS
H20-5/1

DRAWING NUMBER
D3761

TWP# 26-123-1-A



PRINT RECORD		
NO.	FOR	DATE

REVISIONS:	DATE		BY	DESCRIPTION

LEGEND:-
● PENETRATION & BORE HOLE
○ PENETRATION HOLE ONLY

SCALE:- 1 INCH = 20 FEET

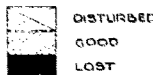
DEPARTMENT OF HIGHWAYS-ONTARIO-			
BRIDGE OFFICE-TORONTO			
CROSSING OF H.W.Y. #401			
LINE 'B' & COUNTY ROAD #20			
THE KING'S HIGHWAY No. 401 & 20		DIV. No. 8	
CO. GRENVILLE			
TWP. AUGUSTA		LOT 2 & 3	CON. 1
PLAN OF PEN. & BOR. HOLE POSITIONS			
APPROVED			
CHIEF BRIDGE ENGINEER		CHIEF ENGINEER	
DESIGN	CHECK	CONTRACT	
DRAWING	CHECK	NUMBERS	
TRACING	CHECK	LOADING	
DATE		DRAWING	
		NUMBER	F-55-10A

MATERIALS LABORATORY - DEPARTMENT OF HIGHWAYS - ONTARIO
OFFICE REPORT ON SOIL EXPLORATION

DRILL RIG *CORE DRILL #1*
CASING *2 1/2" (STANDARD SAMPLERS TO FIT UNLESS NOTED)*
SAMPLER HAMMER WT *250* DROP *22* INCHES

JOB *55-F-10*
DATUM *278.88 ± 0.10 at 25' 0" depth*
COMPILED BY *B.H.* CHECKED BY *B.H.*
BORING NO. *1*
DATE REPORT *21 May 1955*
BORING DATE *24 May 1955*

SAMPLE CONDITION



SAMPLE TYPES

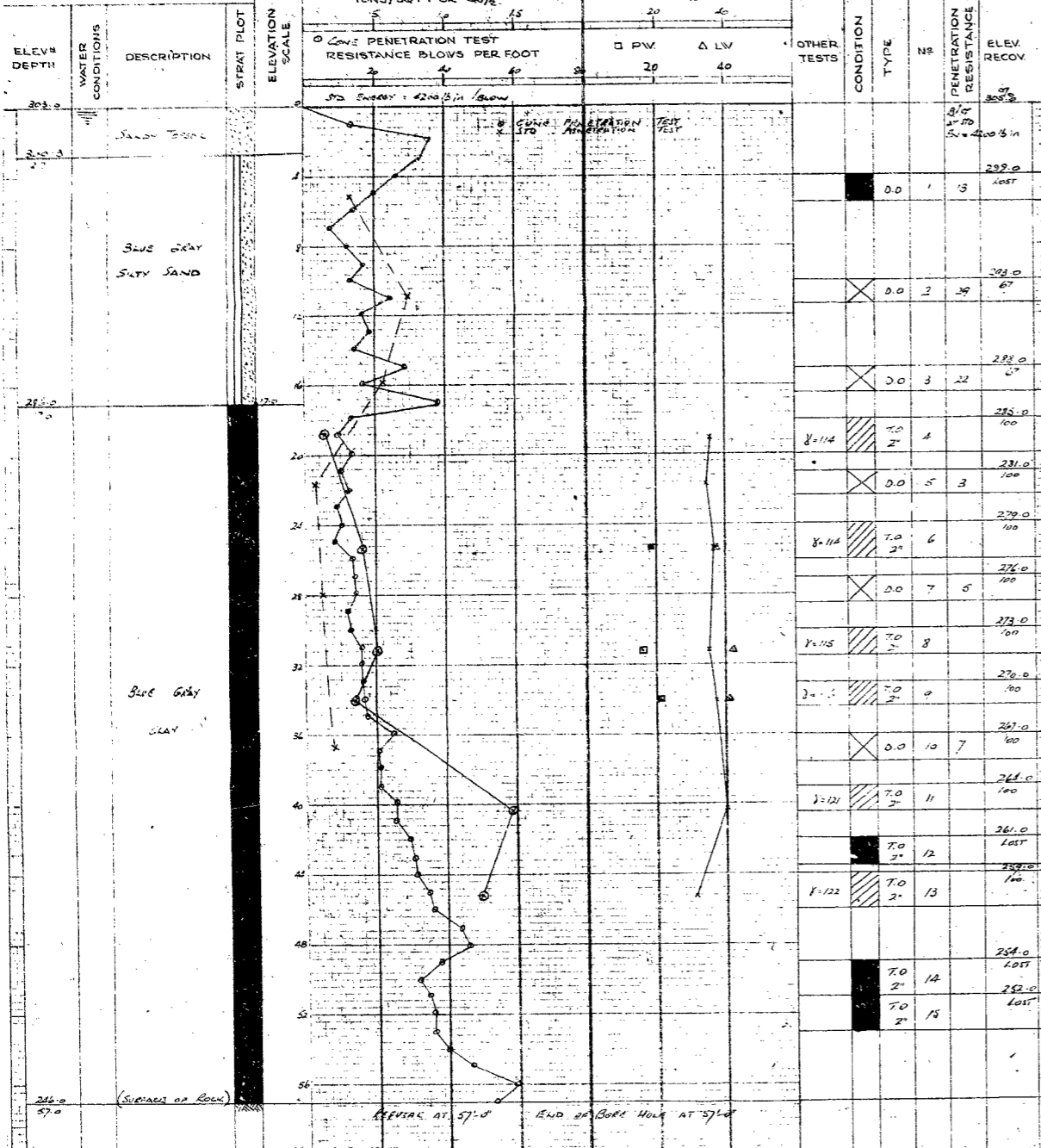
CS - CHUNK
DO - DRIVE OPEN
DF - DRIVE FOOT VALVE
TO - THIN WALLED OPEN

WS - WASHED SAMPLE
RC - ROCK CORE

ABBREVIATIONS

V - INSITU VANE SHEAR TEST
M - MECHANICAL ANALYSIS
U - UNCONFINED COMPRESSION
Qc - TRIAXIAL CONSOLIDATED QUICK
Q - TRIAXIAL QUICK
S - TRIAXIAL SLOW
UNIT WEIGHT
K - PERMEABILITY
C - CONSOLIDATION
CA - CASING
WL - WATER LEVEL IN CASING
WT - WATER TABLE IN SOIL

SOIL PROFILE

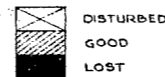


MATERIALS LABORATORY - DEPARTMENT OF HIGHWAYS - ONTARIO
OFFICE REPORT ON SOIL EXPLORATION

DRILL RIG *CORE DRILL #1*
CASING *2 1/2" (STANDARD SAMPLERS TO FIT UNLESS NOTED)*
SAMPLER HAMMER WT *250* DROP *22* INCHES

JOB *55-F-10*
DATUM *278.88 ± 0.10 at 25' 0" depth*
COMPILED BY *B.H.* CHECKED BY *B.H.*
BORING NO. *3*
DATE REPORT *19 May 1955*
BORING DATE *24 May 1955*

SAMPLE CONDITION



SAMPLE TYPES

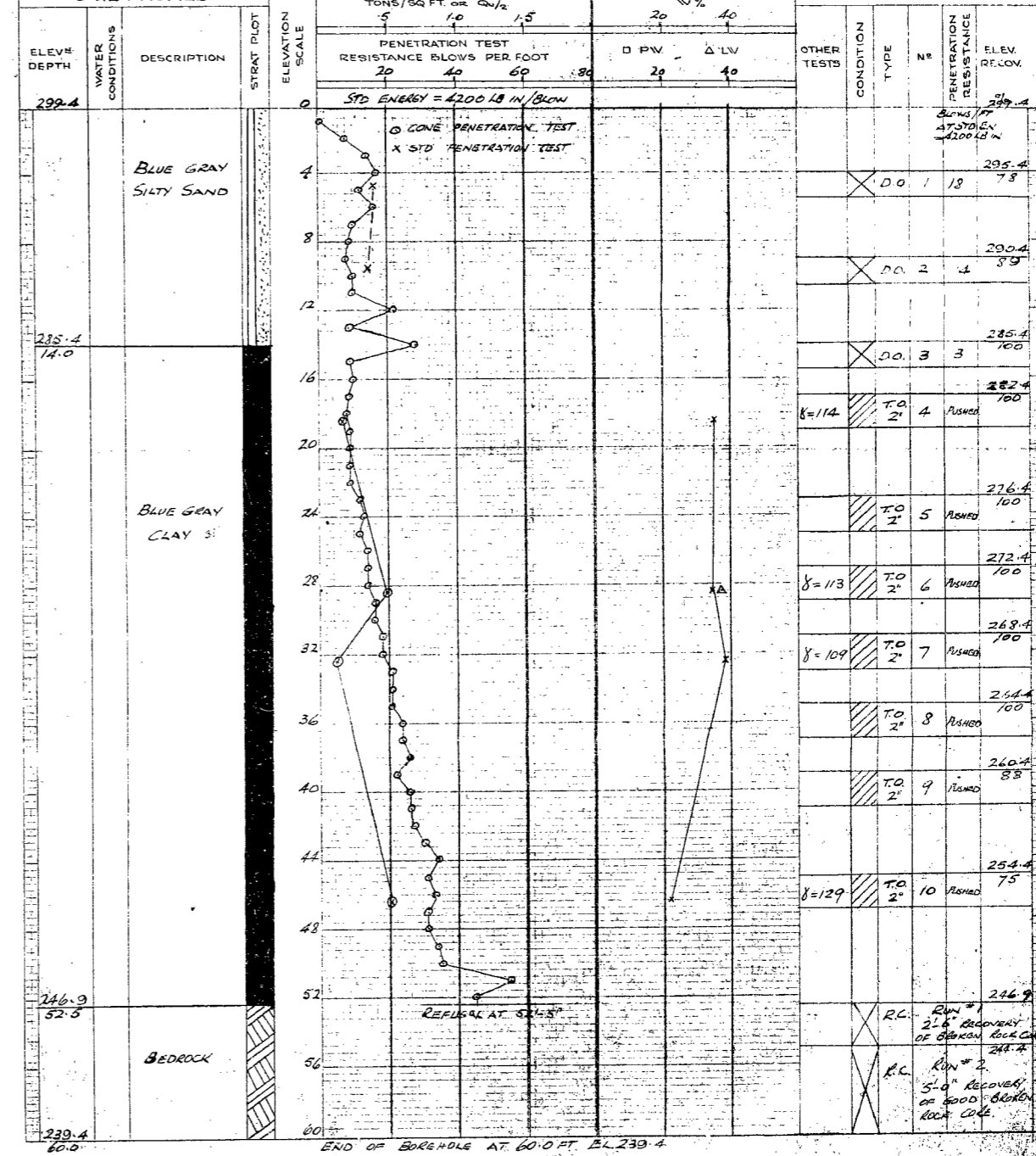
CS - CHUNK
DO - DRIVE OPEN
DF - DRIVE FOOT VALVE
TO - THIN WALLED OPEN

WS - WASHED SAMPLE
RC - ROCK CORE

ABBREVIATIONS

V - INSITU VANE SHEAR TEST
M - MECHANICAL ANALYSIS
U - UNCONFINED COMPRESSION
Qc - TRIAXIAL CONSOLIDATED QUICK
Q - TRIAXIAL QUICK
S - TRIAXIAL SLOW
UNIT WEIGHT
K - PERMEABILITY
C - CONSOLIDATION
CA - CASING
WL - WATER LEVEL IN CASING
WT - WATER TABLE IN SOIL

SOIL PROFILE

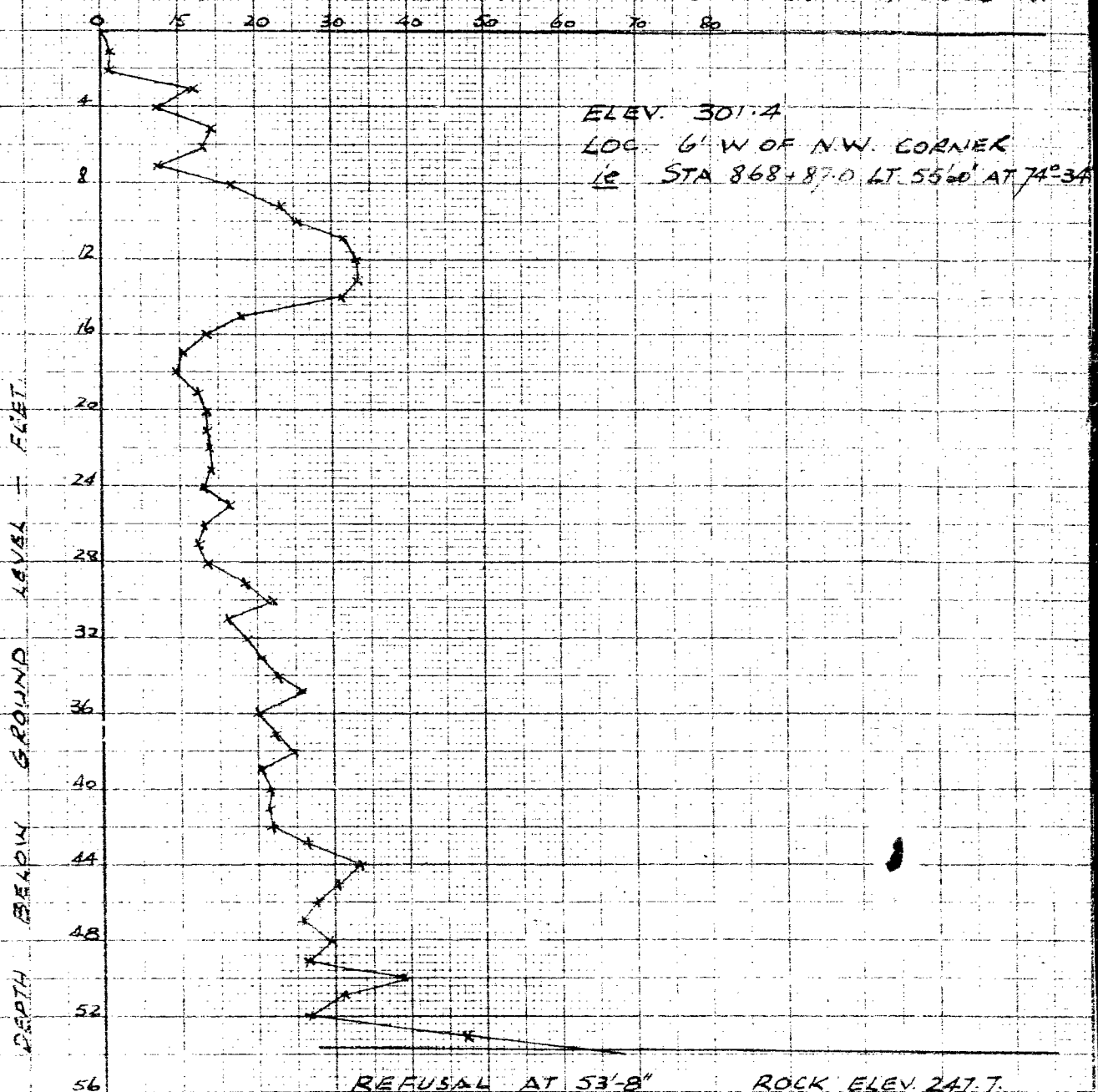


(PRESCOTT) 55-F-10

BH-2

GRAPH OF CONE PENETRATION TEST

No. OF BLOWS AT STANDARD ENERGY 4,200 LB. IN.





Appendix F.

Foundation Alternative Comparisons









COMPARISON OF BRIDGE FOUNDATION ALTERNATIVES

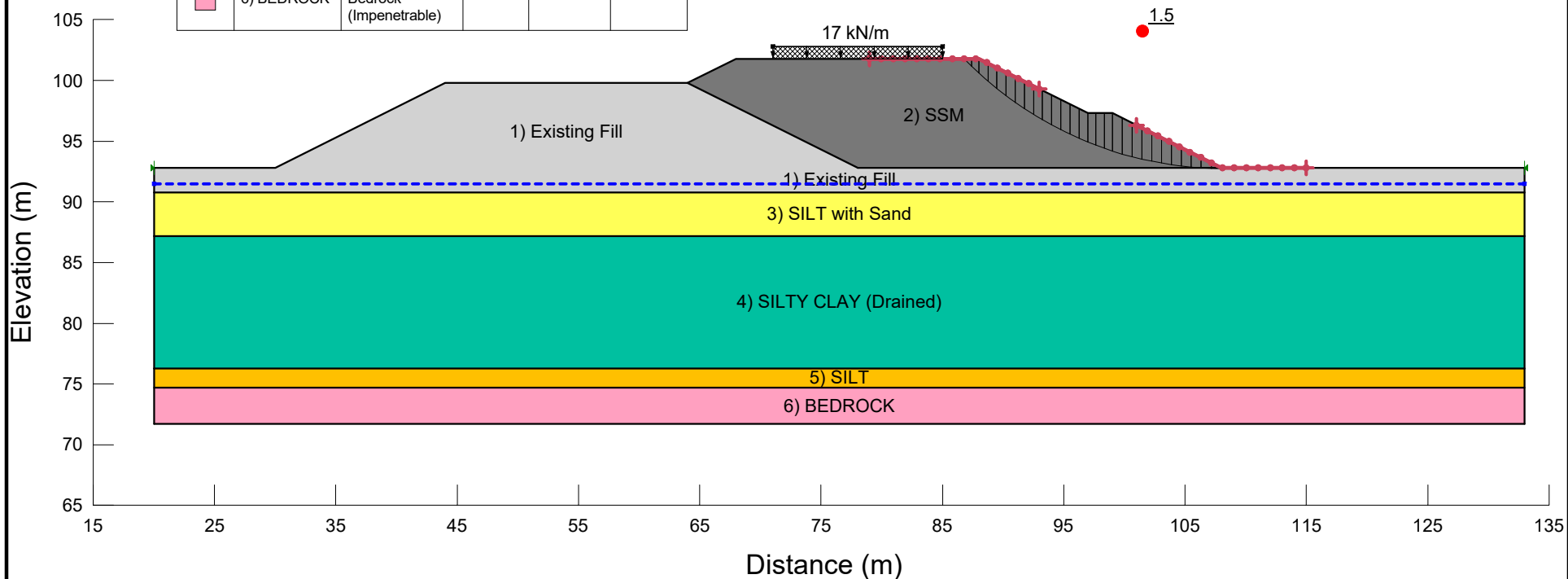
	<i>Driven Steel H-Piles</i>	<i>Concrete Caissons</i>	<i>Drilled in Pipe Piles</i>
Description	<ul style="list-style-type: none">• The abutments would be supported by a single row of steel H-piles driven to refusal on bedrock. The piers would be supported on multiple rows of piles depending on loading requirements 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	<ul style="list-style-type: none">• Steel casing is advanced using a down-the-hole hammer with the cuttings/tailings flushed back to surface inside the drill string. Steel casing keeps hole open during drilling and prevents adjacent soil from collapsing into the hole. The casing is left in place as the bearing structure and would be filled with concrete (no rebar).
Advantages	<ul style="list-style-type: none">• Steel H-piles are well suited for use in integral abutment design.• Requires less concrete than caissons and drilled-in pipe piles.	<ul style="list-style-type: none">• Moderate to high axial geotechnical resistance.• Can handle oversized obstructions.• Suitable for semi-integral abutment design approach.	<ul style="list-style-type: none">• Moderate geotechnical resistance due to end-bearing only capacity in bedrock• Drilling system is well suited for penetrating through most obstructions (except wood).• High drilling production rates.
Disadvantages	<ul style="list-style-type: none">• Has potential to encounter obstructions in the fill.• Lower geotechnical resistance the caissons and drilled-in pipe piles.	<ul style="list-style-type: none">• Not compatible with integral abutment design approach.• Likely requires concrete to be placed using tremie techniques.• Temporary steel casing required to keep hole open during drilling.• The base of the caisson would need to be inspected to ensure end bearing capacity.	<ul style="list-style-type: none">• Smaller number of contractors with suitable equipment.• Not commonly used for integral abutments
Risks / Consequences	<ul style="list-style-type: none">• Difficult advancing through obstructions	<ul style="list-style-type: none">• Difficulty penetrating through obstructions such as concrete and wood can cause construction delays.• Increased concrete volume may be required if additional soil is pulled in from sidewall while advancing through obstructions.• Position and alignment could be affected by obstructions.	<ul style="list-style-type: none">• Lateral response of pile difficult to predict since some equipment uses a drill head with greater diameter than pile and the width of this annular space may vary.
Relative Cost	<ul style="list-style-type: none">• Lower	<ul style="list-style-type: none">• Higher	<ul style="list-style-type: none">• Higher
Conclusion	<ul style="list-style-type: none">• Recommended	<ul style="list-style-type: none">• Feasible, but not recommended	<ul style="list-style-type: none">• Feasible, but not recommended



Appendix G.

Slope Stability Analysis Figures

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	1) Existing Fill	Mohr-Coulomb	20	0	30
	2) SSM	Mohr-Coulomb	20	0	30
	3) SILT with Sand	Mohr-Coulomb	19	0	28
	4) SILTY CLAY (Drained)	Mohr-Coulomb	17.5	1	29
	5) SILT	Mohr-Coulomb	19	0	28
	6) BEDROCK	Bedrock (Impenetrable)			










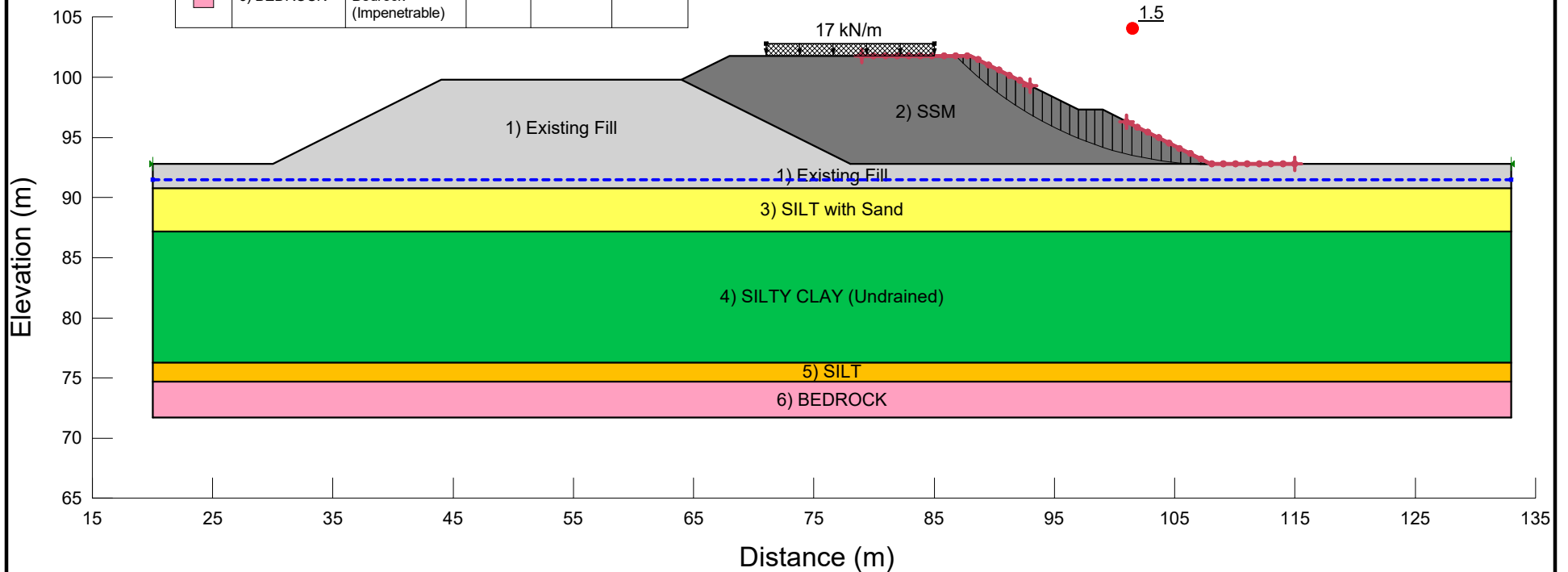
	Project			Additional Details	
	Hwy 401 Edward Street Underpass			Name: Hwy 401 Edward Street U/P - New Alignment	
	Analysis			Comments:	
	01) Static Drained			Method: Morgenstern-Price, Half-Sine	
Seismic Coefficient		Last Run		Minimum Slip Surface Depth: 3.05 m	
H: g, V: g		03/14/2023, 03:16:10 PM		Entry: (86.781424, 101.8) m, Exit: (110.05792, 92.8) m	
		Scale		Center: (108.18334, 122.55155) m, Radius: 29.81055 m	
		1:507			

Figure G1

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	1) Existing Fill	Mohr-Coulomb	20	0	30
	2) SSM	Mohr-Coulomb	20	0	30
	3) SILT with Sand	Mohr-Coulomb	19	0	28
	4) SILTY CLAY (Undrained)	Mohr-Coulomb	17.5	100	0
	5) SILT	Mohr-Coulomb	19	0	28
	6) BEDROCK	Bedrock (Impenetrable)			










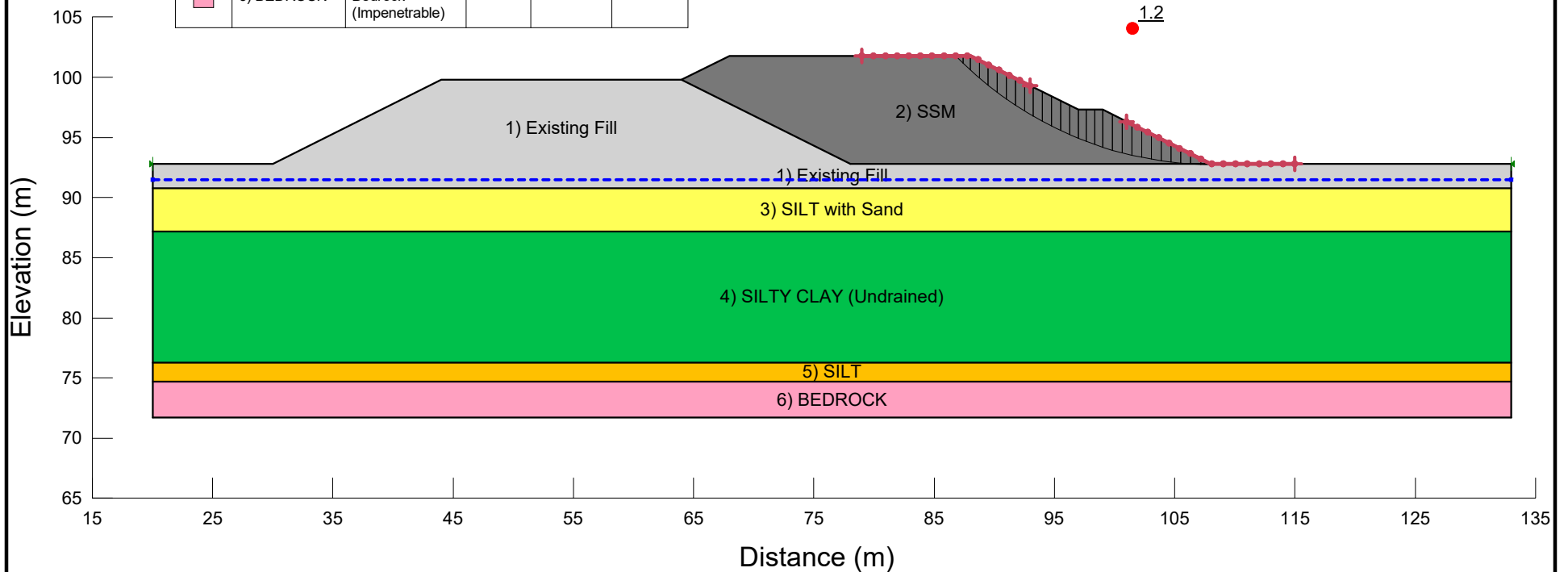
	Project			Additional Details	
	Hwy 401 Edward Street Underpass			Name: Hwy 401 Edward Street U/P - New Alignment	
	Analysis			Comments:	
	02) Static Undrained			Method: Morgenstern-Price, Half-Sine	
	Seismic Coefficient			Minimum Slip Surface Depth: 3.05 m	
	Last Run			Entry: (86.781424, 101.8) m, Exit: (110.05792, 92.8) m	
H: g, V: g	03/14/2023, 03:16:10 PM			Center: (108.18334, 122.55155) m, Radius: 29.81055 m	
	Scale				
	1:507				

Figure G2

Figure G2

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	1) Existing Fill	Mohr-Coulomb	20	0	30
	2) SSM	Mohr-Coulomb	20	0	30
	3) SILT with Sand	Mohr-Coulomb	19	0	28
	4) SILTY CLAY (Undrained)	Mohr-Coulomb	17.5	100	0
	5) SILT	Mohr-Coulomb	19	0	28
	6) BEDROCK	Bedrock (Impenetrable)			




	Project			Additional Details		
	Hwy 401 Edward Street Underpass			Name: Hwy 401 Edward Street U/P - New Alignment		
	Analysis			Comments:		
	03) 475yr Pseudo-static			Method: Morgenstern-Price, Half-Sine		
	Seismic Coefficient			Minimum Slip Surface Depth: 3.05 m		
	H: 0.078g, V: g		Last Run		Entry: (86.781424, 101.8) m, Exit: (110.05792, 92.8) m	
			03/14/2023, 03:16:10 PM		Center: (108.18334, 122.55155) m, Radius: 29.81055 m	
				Scale		
				1:507		

Figure G3

Figure G3



Appendix H.

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: 44.723N 75.525W

User File Reference: Highway 401 Edward Street Underpass

2023-02-02 15:01 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)	0.350	0.200	0.122	0.036
Sa (0.1)	0.416	0.248	0.157	0.050
Sa (0.2)	0.354	0.214	0.138	0.047
Sa (0.3)	0.273	0.166	0.108	0.038
Sa (0.5)	0.197	0.120	0.078	0.028
Sa (1.0)	0.101	0.062	0.041	0.014
Sa (2.0)	0.049	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)	0.227	0.136	0.086	0.027
PGV (m/s)	0.164	0.096	0.060	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 I.

List of Referenced Specifications and NSSPs



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

OPSD 3090.101	Foundation, Frost Penetration Depths for Southern Ontario
OPSD 3101.150	Walls, Abutment, Backfill Minimum Requirements
OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS.PROV 1010	Material Specification for Aggregates - Base, Subbase, Select Subgrade, and Backfill Material