



THURBER ENGINEERING LTD.

**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 401 WIDENING, HIGHWAY 16 TO MAITLAND ROAD
HWY 416 SBL - CEDAR GROVE RD OVERPASS, SITE NO. 16X-0307
GWP 4024-20-00 / ASSIGNMENT NO. 4019-E-0010.2**

Geocres No.: 31B-104

Report to:

MTO c/o AECOM Canada Ltd.

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PART 1. FACTUAL INFORMATION

1 INTRODUCTION

Thurber Engineering Ltd. (Thurber) has been retained by AECOM Canada Ltd. (AECOM) on behalf of the Ministry of Transportation Ontario (MTO) under Assignment No. 4019-E-0010, Work Item No. 2, to carry out Foundation Investigations to support the Preliminary Design and Environmental Assessment for the widening of Highway 401 from Highway 416 to Maitland Road. The overall scope of work comprises replacement or rehabilitation of 14 existing structures, including ten bridges and four structural culverts.

This report addresses the proposed rehabilitation of Bridge Site No. 16X-0307 which supports Highway 416 southbound traffic as it passes over Cedar Grove Road before connecting to Highway 401 westbound (416N-401W). The bridge is located approximately 500 m northeast of the ramp's termination at Highway 401, near 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 and included:

- Report prepared by Jacques, Whitford Limited titled, "*Report on Foundation Investigation for W.P. 374-89-03, Site 16-307, Ramp N-W Over Cedar Grove Road, Highway 401-416 Interchange, District 9, Ottawa*", dated March, 1992 (Geocres No. 31B-75).

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 rehabilitation 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

Cedar Grove Road is roughly oriented east-west and the bridge is oriented roughly northeast to southwest. For project purposes, Cedar Grove Road and the 416N-401W ramp bridge are herein described as oriented east-west and north-south, respectively.

The land adjacent to the site typically consists of forested lands and agricultural fields. A Trans-Northern Pipelines Inc. pipeline junction is located immediately southeast of the bridge. The terrain is relatively flat apart from the existing highway embankment and associated drainage ditches. Cedar Grove Road in this area consists of a two-lane local roadway, and the 416N-401W ramp consists of two southbound travelled lanes with paved shoulders. The bridge was constructed under Contract 97-68 and is a single span structure with a length of 35 m and a width of 12.5 m.

The embankment side slopes on the original design drawings (Sheet 159) were to be sloped at approximately 2H:1V except for the southwest embankment which is indicated to be at 1.5H:1V and includes a note indicating rock backfill at abutments. The forward slopes are also indicated to be at 1.5H:1V and incorporate rockfill as well as concrete slope paving.

The side slopes were observed during the current field investigation and were generally grass-covered, with bushes and small trees growing around the abutments. Rockfill material was exposed on the upper portions of the west slope of both the north and south approaches (see Photos 1 and 7 in Appendix D). At the time of the field work, the side slopes appeared to be 2H:1V in all quadrants and did not show any visible signs of distress or other performance issues. Steel beam guiderails are present along the approaches and abut the concrete barrier walls along the bridge.

Based on published geological information in *The Physiography of Southern Ontario* by Chapman and Putnam (1984), the site lies on the border of the physiographic regions known as the Smith's Falls Limestone Plain and the Glengarry Till Plain.

The Smith's Falls Limestone Plain is characterized by typically shallow but including a few localized deep areas of highly variable soils consisting of clays, sands, and gravels. The Glengarry Till Plain is characterized by an undulating surface consisting of morainic ridges and intervening clay flats and swamps, overlying till and similar glaciofluvial deposits containing many cobbles and boulders. Both areas are known to be underlain by limestone and sandstone bedrock.

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

3 SITE INVESTIGATIONS AND FIELD TESTING

The original foundation investigation for design of the bridge was carried out in May 1991, prior to its construction. The current investigation was carried out in April/May 2021 to collect additional



subsurface information near the bridge abutments and through the existing embankments. Summaries of the investigations are provided in the following sections.

3.1 Original (1991) Investigation

A total of four boreholes were put down at the site as part of the original investigation. Boreholes 91-1 to 91-4 were put down at the proposed approach embankment and foundation element locations between May 3 and 6, 1991. Boreholes 91-2 and 91-3 were put down near the now-constructed north and south abutments. Boreholes 91-1 and 91-4 were put down beyond the limits of the current study and will not be discussed herein.

Boreholes 91-2 and 91-3 were advanced to depths of 10.7 m and 10.2 m below the existing ground surface at the time of the investigation (prior to construction of the bridge). A standpipe piezometer or monitoring well was installed in each of Boreholes 91-2 and 91-3.

The northing, easting and elevation of the boreholes used in this investigation are shown on the Borehole Location and Soil Strata Drawing No. 1 in Appendix A and in Table 3-1, below. The site is located within MTM Zone 9. Note that the borehole locations were originally surveyed relative to NAD27 horizontal datum and have been converted relative to NAD83 in the drawing, on the Record of Borehole Sheets (where appropriate), and in Table 3-1, below.

Table 3-1: Borehole Summary

Borehole No.	Drilled Location	Northing ¹ (Latitude)	Easting ¹ (Longitude)	Ground Surface ² Elevation (m)	Termination Depth (m)
91-2	North Abutment	4 956 966.7 (44.748233)	384 745.4 (-75.490375)	86.3	10.7
92-3	South Abutment	4 956 926.8 (44.747875)	384 742.2 (-75.490422)	86.4	10.2

Notes: 1) Boreholes were surveyed relative to NAD27; coordinates listed above were converted relative to NAD83.

2) Boreholes were put down prior to construction of the existing approach embankment and foundation.

3.2 Current (2021) Investigation

The current site investigation was carried out on April 26 to 27, 2021. Two boreholes were put down at the bridge site: one near the north abutment (Borehole 307-21-1) and one near the south abutment (Borehole 307-21-2).

The locations of the 2021 boreholes were surveyed by Thurber for both location and elevation with a Trimble Catalyst DA1 antenna with centimeter accuracy. The northing, easting and elevation of the boreholes are shown on the Borehole Location and Soil Strata Drawing No. 1 in Appendix A, the individual Record of Borehole sheets in Appendix B.1, and in Table 3-2 below.

Table 3-2: Borehole Summary

Borehole No.	Drilled Location	Northing (Latitude)	Easting (Longitude)	Ground Surface Elevation (m)	Termination Depth (m)
307-21-1	North Abutment	4 956 968.2 (44.748254)	384 746.3 (-75.490366)	94.3	19.0
307-21-2	South Abutment	4 956 917.5 (44.747799)	384 731.6 (-75.490560)	93.8	18.7

The current investigation was carried out using a truck-mounted CME 55 drill rig equipped with hollow-stem augers and rotary diamond drilling equipment.

Soil samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). The boreholes were advanced approximately 3 m into bedrock, with NQ sized coring equipment.

A monitoring well, 19 mm in diameter, was installed in Borehole 307-21-2 and finished with a flushmount cover at the road surface. The installation details are illustrated on the respective Record of Borehole sheets provided in Appendix B.1. Borehole 307-21-1 was backfilled in accordance with MOE requirements (O.Reg 903, as amended).

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.

4 LABORATORY TESTING

Geotechnical laboratory testing carried out as part of the current investigation included natural moisture content determination and visual identification of all retained soil samples. Testing for grain size distribution and Atterberg Limits was also carried out on selected samples to MTO and ASTM standards. All rock cores were photographed and their total core recovery (TCR), solid core recovery (SCR) and rock quality designation (RQD) were measured. One Unconfined Compressive Strength (UCS) test was conducted on a recovered core sample from Borehole 307-21-1.

Laboratory testing carried out as part of the 1992 investigation on selected soil samples included grain size distribution, Atterberg limit determination, and a one-dimensional consolidation test.

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

5 GENERAL DESCRIPTION OF SUBSURFACE CONDITIONS

Details of the encountered soil stratigraphy are presented on the Record of Borehole sheets included in Appendix B and the Borehole Location and Soil Strata Drawing included in



Appendix A. A general description of the stratigraphy based on the conditions encountered in the boreholes is given in the following sections. However, the factual data presented on the Borehole Records takes precedence over the Soil Strata Drawing and the general description. It must be recognized that the soil and groundwater conditions may vary between and beyond borehole locations. Soil classification for the 2021 boreholes is in accordance with ASTM D2487. Descriptions of cohesive soils and secondary components of all deposits from the 2021 boreholes are described as outlined in the MTO Guideline for Foundation Engineering Services manual (October 2020). Terminology from the historic Geocres information may vary from current.

In general, the subsurface conditions consist of pavement structure and fill over silty clay to clay, which is underlain by a deposit of glacial till consisting of silty sand to sandy clayey silt. The overburden is underlain by interbedded dolostone and sandstone bedrock.

The surficial topsoil and very loose sand deposits in Boreholes 91-2 and 91-3 would have likely been disturbed, altered, or completely removed during the construction of the 416N-401W ramp and bridge structure; these layers are not included in the following descriptions.

5.1 Pavement Structure and Embankment Fill

The 2021 boreholes were advanced through the pavement structure of the 416N-401W ramp. The pavement structure encountered consisted of asphaltic concrete over granular fill. The asphalt ranged in thickness from 200 to 275 mm. A granular embankment fill layer consisting of gravel and sand, trace to some fines to sandy gravel, some silt to silty sand, some gravel was encountered beneath the asphalt in Boreholes 307-21-1 and 307-21-2. The granular embankment fill encountered was 8.9 m and 6.7 m thick with underside elevations of 85.2 m to 86.8 m near the north and south abutments, respectively.

SPTs conducted in the fill gave N-values of 3 blows to greater than 100 blows for 125 mm of penetration, but generally between about 15 and 45 blows per 0.3 m of penetration indicating a compact to dense relative density. It is noted that neither of the 2021 boreholes encountered rock fill. The higher blow counts observed in the boreholes may represent cobbles or a boulder within the fill rather than the relative density of the soil matrix.

The moisture content of the tested embankment fill samples ranged from 1 to 20%. The results of grain size analyses conducted on three samples of the embankment fill are summarized below and are illustrated on Figure C1 in Appendix C.1.

Summary of Grain Size Distribution Testing – Embankment Fill

Soil Particle	Percentage (%)
Gravel	11 – 59
Sand	33 – 70
Silt and Clay	8 – 19



5.2 Silty Sand

A deposit of silty sand was encountered beneath the fill in Borehole 307-21-2. Organics were encountered within the deposit. The layer was approximately 1.2 m thick with a base elevation of 85.6 m.

One SPT carried out in this layer gave an N-value of 21 blows per 0.3 m of penetration, indicating a compact relative density. The sample of the silty sand had a moisture content of 16%.

5.3 Silty Clay to Clay

A native, cohesive deposit ranging in composition from silty clay to clay was encountered below the fill in Borehole 307-21-1, below the silty sand in Borehole 307-21-2, and within 200 mm of the existing ground surface in Boreholes 91-2 and 91-3 at the time of drilling. The silty clay deposit has a thickness of 3.4 m and 6.0 m, with a base elevation of 80.1 to 81.8 m.

The upper portion of the clay has generally been weathered to a grey-brown crust. SPTs conducted in the weathered crust gave N-values ranging from 9 to 17 blows for 0.3 m of penetration. Two field vane tests carried out within the lower portion of the weathered crust in Borehole 91-3 gave undrained shear strengths of approximately 230 kPa to greater than 240 kPa. These in-situ test results indicate a hard consistency. Remolded field vane testing indicates that the weathered crust has a high sensitivity. Recorded moisture contents of the weathered crust ranged from 26 to 31%.

Unweathered grey clay was encountered beneath the weathered crust in all boreholes drilled during the investigations. SPTs conducted in the grey clay gave N-values ranging from 7 to 13 blows for 0.3 m of penetration. Field vane testing carried out in 1991 in the grey clay gave undrained shear strength values ranging from 130 kPa to greater than 240 kPa, indicating a very stiff to hard consistency. Remolded field vane testing indicates that the clay is of medium to high sensitivity. Recorded moisture contents of the grey clay ranged from 22 to 33%.

The results of two grain size analysis tests conducted on the weathered crust and one grain size analysis on the grey clay are summarized below and are illustrated on Figure C2 in Appendix C.1 and Figure 1 in Appendix C.2, respectively.

Summary of Grain Size Distribution Testing – Silty Clay to Clay

Soil Particle	Percentage (%)
Gravel	0
Sand	0 – 3
Silt	38 – 41
Clay	56 – 62

The results of the Atterberg Limits testing carried out on two samples of the weathered crust and one sample of the grey clay are summarized below and are illustrated on Figure C3 in Appendix C.1 and Figure 2 in Appendix C.2, respectively. The laboratory results indicate that the material is generally a clay of intermediate plasticity (CI) to high plasticity (CH).



Summary of Atterberg Limit Testing – Silty Clay to Clay

Parameter	Value
Liquid Limit	47 – 51
Plastic Limit	20 – 22
Plasticity Index	25 – 30

One laboratory oedometer (one-dimensional consolidation) test was carried out on a relatively undisturbed clay sample obtained during the 1991 investigation with a thin-walled tube sampler. The results of that testing are presented on Figure 3 in Appendix C.2 and indicate a material with and over-consolidation ratio (OCR) in excess of 13.

5.4 Till: Sandy Clayey Silt to Sandy Silt to Silty Sand

A glacial till deposit of sandy clayey silt to sandy silt to silty sand with varying amounts of gravel was present beneath the silty clay to clay deposit in all boreholes. Occasional to frequent cobbles and boulders were noted in the till in all boreholes. The thickness of the glacial till deposit ranged from 2.2 to 3.2 m and the underside of this layer ranged from elevation 77.9 to 78.6 m.

SPTs conducted in this layer gave N-values ranging from 15 to 69 blows for 0.3 m of penetration indicating a compact to very dense relative density.

The moisture content of this unit ranged from 7 to 18%. The results of grain size distribution testing carried out on three samples of the till are summarized below and are illustrated on Figure C4 in Appendix C.1 and Figure 4 in Appendix C.2.

Summary of Grain Size Distribution Testing – Glacial Till

Soil Particle	Percentage (%)	
Gravel	7 – 13	
Sand	22 – 41	
Silt	50 - 65	42 – 48
Clay		8 – 17

The results of Atterberg Limits testing carried out on the fines of two samples of this material are summarized below and are illustrated on Figure C5 in Appendix C.1 and Figure 5 in Appendix C.2. The laboratory results indicate that the fines are non-plastic (ML) to low plasticity (CL-ML).

Summary of Atterberg Limit Testing – Glacial Till Fines

Parameter	Value
Liquid Limit	17
Plastic Limit	13 – 14
Plasticity Index	3 – 4

5.5 Bedrock

Bedrock was proven by coring in all boreholes. The bedrock encountered consisted of fresh to slightly weathered, fine to coarse grained, very strong, interbedded dolostone and sandstone encountered at elevations ranging from 77.9 to 78.6 m. The 1991 investigation described the bedrock as dolomitic limestone. Quartz vugs were present in Borehole 307-21-2. Photographs of the bedrock core from the current investigation are provided in Appendix C. The following table summarizes the rock core quality:

Summary of Rock Core Quality

Parameter	Range	Average
Total Core Recovery (TCR), %	97 – 100	99
Solid Core Recovery (SCR), %	65 – 98	86
Rock Quality Designation (RQD), %	53 – 97	80
Fracture Index	0 to >10	-

Based on the average RQD value, the bedrock is classified as good quality. It is noted that vertical fractures were observed in the core recovered from Borehole 307-21-1. Unconfined compressive strength (UCS) testing was carried out on one sample of the bedrock from Borehole 307-21-1. The result of 162 MPa indicates a very strong rock, and is included in Appendix C. A summary of the bedrock surface information is provided in Table 5-1, below:

Table 5-1: Summary of Bedrock Depth/Elevation

Borehole No.	Depth to Bedrock Surface (mbgs)	Bedrock Surface Elevation (m)
307-21-1	15.7	78.6
307-21-2	15.9	77.9
91-2	7.9 ¹	78.4
91-3	8.4 ¹	78.0

Note: 1) The 1991 boreholes were put down prior to construction of the existing approach embankments and foundations and the ground surface at the time of the investigation was lower than the current ground elevation.

5.6 Groundwater

The groundwater levels measured in the monitoring well and standpipe piezometers installed during the current and the 1991 field investigations are presented in Table 5-2.

These observations are considered short term and it should be noted that the groundwater level may vary with season and fluctuations of the groundwater level are to be expected. In particular, the groundwater may be at a higher elevation after periods of significant and/or prolonged precipitation.

Table 5-2: Summary of Groundwater Levels

Borehole No.	Bottom of Screen Elevation (m)	Screened Unit	Depth (mbgs)	Groundwater Elevation (m)	Date of Measurement
307-21-2	78.7	Silty Clay/ Glacial Till	7.8	86.0	2021/07/01
			2.0 ¹	91.8 ¹	2022/12/19
91-2	81.6	Silty Clay	0.5	85.8	1991/06/25
	78.4	Glacial Till	0.5	85.8	1991/06/25
91-3	78.2	Silty Clay/ Glacial Till	0.5	85.9	1991/06/25

Note: 1) Measured groundwater level may be a result of ice blockage due to frozen surficial inflow and is not considered to be representative of typical conditions at the site. Consideration may be given to carrying out additional groundwater level measurements during subsequent stages of design.



6 MISCELLANEOUS

It is noted that the conditions reported on the 1991 borehole records may not reflect current conditions due to construction or other activities in the area subsequent to those investigations.

The 2021 borehole location was selected by Thurber relative to existing site features. The as-drilled location and ground surface elevation of the borehole was surveyed by Thurber following completion of the field program. The elevation survey of the boreholes was carried out with reference to geodetic elevation benchmarks provided by the MTO.

Eastern Ontario Diamond Drilling of Hawkesbury, Ontario supplied and operated the drilling equipment and carried out the drilling, soil sampling, in-situ testing, piezometer installation and borehole decommissioning. The field investigation was supervised on a full-time basis by Jamil Pirani, EIT, of Thurber.

Routine geotechnical laboratory testing was completed by Thurber's laboratory in Ottawa, Ontario. Unconfined compressive strength testing of the bedrock was carried out by Stantec's laboratory in Ottawa.

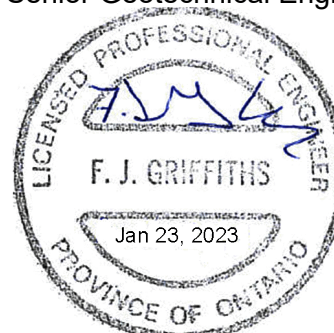
Overall project management and direction of the field investigation was provided by Matt Kennedy, P.Eng. Interpretation of the factual data and preparation of this report was carried out by Sarah Harrold, EIT and Matt Kennedy, P.Eng. The report was reviewed by Paul Carnaffan, P.Eng. and Fred Griffiths, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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PART 2. ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 INTRODUCTION

This report presents the interpretation of the factual data obtained from a foundation investigation and a desktop review of the available subsurface information conducted by Thurber for the rehabilitation of the existing Highway 416 southbound Overpass of Cedar Grove Road at the Highway 401-416 Interchange.

The site is located approximately 500 m northeast of the NW Ramp termination at Highway 401. The overpass bridge carries traffic coming from the north on Highway 416 over Cedar Grove Road, to connect to westbound Highway 401. Cedar Grove Road is generally oriented east-west and the bridge is oriented roughly northeast to southwest. For project purposes, Cedar Grove Road and the 416N-401W Bridge are herein described as oriented east-west and north-south, respectively.

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

The following sections provide geotechnical recommendations for the design of the foundation elements as part of the structural assessment and rehabilitation. The discussions and recommendations presented in this report are based on the information provided by the Ministry of Transportation of Ontario (MTO) and on the factual data obtained throughout this investigation.

7.1 Existing Structure

Cedar Grove Road in this area consists of a two-lane local roadway, and the 416N-401W ramp consists of two southbound travelled lanes with paved shoulders. The bridge was constructed under Contract 97-68 and is a single span structure with a length of 35 m and a width of 12.5 m. It has a cast-in-place, post-tensioned concrete deck and non-integral abutments. The bridge crosses Cedar Grove Road at a skew angle of approximately 55 degrees from parallel.



Based on the available structural drawings (Cont. No. 97-68, Sheets 159 and 161), the abutments are each supported on steel HP 310x110 piles up to about 11 m long, driven to bedrock. There are two rows of piles at each abutment. The front (interior) rows of piles include 11 piles which are battered forward at a slope of 1H:3V and the back (exterior) rows of piles include 7 piles which are battered to the rear at a slope of 1H:12V. Both abutment pile caps are perched within the existing embankments with the top of pile cap at Elevation 89.5 m (north) or 89.0 m (south). The north pile cap is approximately 4.0 m by 16.3 m and 1.0 m thick, with underside at an elevation of approximately 88.5 m. The south pile cap is approximately 4.0 m by 16.9 m and 1.0 m thick, with underside at an elevation of approximately 88.0 m. The original structural drawings indicate that the pile caps were to be underlain by fill with “maximum grain size 75 mm” from the underside of pile cap to existing grade.

The drawings indicate 1.5H:1V side slopes on the west side of the south approach and includes a note indicating “See roadway drawings for details of rock backfill at abutments.” However, based on observation of the approximate slope and location of the embankment toe during the site reconnaissance visit, the west side of the south approach embankment appears to have been constructed at about 2H:1V, like the other embankment side slopes.

The slopes in front of the abutments are at approximately 1.5H:1V and include concrete slope paving.

7.2 Applicable Codes and Design Considerations

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

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

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

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

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



8 SEISMIC CONSIDERATIONS

8.1 Spectral and Peak Acceleration Hazard Values

The seismic hazard data for the CHBDC is based on the fifth-generation seismic model developed by the Geological Survey of Canada (GSC). Seismic hazard data for this site has been obtained from the GSC's seismic hazard calculator. The data includes peak ground acceleration (PGA), peak ground velocity (PGV), and the 5% damped spectral response acceleration values ($S_a(T)$) for the reference ground condition (Site Class C) for a range of periods (T) and for a range of return periods including the 475-year, 975-year and 2475-year events. The GSC seismic hazard calculation data sheet for this site is presented in Appendix E.

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

8.2 CHBDC Seismic Site Classification

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

Boreholes 307-21-1, 307-21-2, 91-2, and 91-3 were put down near the abutments. At the boreholes put down through the embankment fill (Boreholes 307-21-1 and 307-21-2), the average N_{60} values in the embankment fill and sand below the undersides of the pile caps (Elevations 88.5 m and 88.0 m at the north and south pile caps, respectively) were between 15 and 50 blows per 0.3 m of penetration. Similarly, the average N_{60} values in the glacial till deposit that underlies the silty clay at the site was between 15 and 50 blows per 0.3 m of penetration. Based on the average N_{60} values in the upper granular soils and lower glacial till, the seismic site class would be Site Class D.

The average undrained shear strength measured in the silty clay deposit was approximately 180 kPa, with all shear vane readings recorded in Boreholes 91-2 and 91-3 being greater than 100 kPa. Based on the undrained shear strength of the silty clay, the seismic site class would be Site Class C. Further, the site is underlain by very strong interbedded dolostone and sandstone with a shear wave velocity expected to be greater than 1000 m/s (i.e. Site Class B or A).

However, guidance provided in the Commentary to the National Building Code of Canada (2015), on which the CHBDC seismic site class determination is based, indicates that where there is "a number of distinctly different soil layers, the standard penetration resistance or undrained shear

strength for *each* layer should be averaged” and, in the absence of measured shear wave velocity data, the more conservative site class be used for design.

Therefore, for preliminary design of the rehabilitation of the structure at the site, a Site Class D should be used. The site classification should be confirmed with measurement of the shear wave velocity in the upper 30 m of the site at subsequent design stages.

As per Table 4.8 of the CHBDC, Site Class D yields a PGA_{ref} of 0.20 and $F(PGA)$ of 1.108 for the site. These values give a site-adjusted PGA of 0.27 g.

8.3 Seismic Performance Category

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

8.4 Liquefaction Potential

The susceptibility of the cohesive soils at this site to experience liquefaction/cyclic softening was assessed following the Boulanger and Idriss (2007)ⁱ criteria using measured undrained shear strengths. The results of the analysis indicate the cohesive material is not susceptible to cyclic mobility.

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)ⁱⁱ. The cohesionless soil at the site is not considered to be susceptible to liquefaction.

9 FOUNDATION DESIGN RECOMMENDATIONS

9.1 Existing Driven Steel Piles

Based on the available 1997 structural design drawings for the structure (Cont. No. 97-68, Sheets 159 and 161), the existing abutment foundations consist of perched pile caps supported on battered steel HP 310x110 piles driven to bedrock.

Table 9-1: Summary of Abutment Foundations

	North Abutment Pile Cap	South Abutment Pile Cap
Overall Plan Dimensions (m)	4.0 x 16.3	4.0 x 16.9
Pile Cap Thickness (m)	1.00 m	
Top of Pile Cap Elevation (m)	89.5	89.0
Underside of Pile Cap Elevation (m)	88.5	88.0

The abutment foundation dimensions are summarized in Table 9-1, above.

9.1.1 Axial Geotechnical Resistance

The boreholes put down behind the abutments as part of the current investigation encountered up to about 3.3 m of embankment fill beneath the pile cap elevations. A 1.2 m thick deposit of silty sand was encountered below the fill in Borehole 307-21-2. Underlying the fill at the north abutment and the silty sand at the south abutment is a native, cohesive deposit of silty clay that is 3.4 to 5.5 m thick (underside Elevation 80.1 to 81.8 m). The silty clay is underlain by glacial till that ranges from 2.2 to 3.2 m thick (underside Elevation 77.9 to 79.1 m), over bedrock.

The 1997 design drawings indicate that the steel HP 310x110 piles at the north and south abutments were anticipated to be between 10.5 and 11.0 m long, fitted with driving shoes, and driven to the bedrock.

Based on the subsurface conditions encountered at the abutment boreholes, the HP 310x110 piles driven to practical refusal in the bedrock may be considered to have a factored geotechnical Ultimate Limit States (ULS) resistance for axial compression of 2,500 kN. The factored geotechnical resistance at Serviceability Limit States (SLS, for 25 mm of settlement) will not govern for steel piles founded in the bedrock. The structural resistance of the pile under static and seismic conditions must be checked by a structural engineer. The factored geotechnical resistances include the following factors:

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

As outlined in Section 6.11.3.2 of the Commentary to the CHBDC, the group efficiency of driven *end-bearing* piles are typically taken as 1.0.

9.1.2 Downdrag

The results of the current investigation indicate that the glacial till is directly overlain by silty clay, overlain by the granular embankment fill at the abutments. Downdrag forces (negative skin friction) acting upon the piles supporting the abutments, developed as a result of settlement of the silty clay to clay deposit under the imposed loading from the fill, should be considered. Downdrag was considered to be an issue in the original foundation design for the bridge and it was recommended that the approach fills be allowed to settle for a minimum period of two months prior to construction, as outlined in the Foundation Investigation and Design Report (1992, Geocres No. 31B-075). If initiation of the driving of the abutment piles was delayed to occur after the end of the preload/surcharge period, downdrag forces need not be applied.

However, if the abutment piles were driven and seated prior to completion of the preload period and associated settlement, the resulting downdrag loads should be considered in the structural assessment of the piles. The unfactored downdrag load acting on a single HP 310x110 abutment pile is estimated to be about 550 kN.



9.1.3 Uplift Resistance

The glacial till, silty clay to clay, and embankment fill at the abutments will provide uplift resistance to the piles. Shaft friction of the embankment fill, silty clay to clay, and glacial till along the piles were calculated, assuming the piles met effective refusal to driving at Elevation 77.9 m.

The factored geotechnical tensile resistance for a single HP 310x110 pile at either abutment may be taken as 320 kN under static conditions and 1080 kN 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)

9.1.4 Lateral Geotechnical Resistance and Group Effects

The front (interior) row of abutment piles is battered at 1H:3V and the back (exterior) row is battered at 1H:12V. It is assumed that the batter of the piles is sufficient to resist the lateral loads on the foundation.

Alternatively, the lateral response of the existing pile foundations can be analyzed considering the soil-structure interaction between the pile(s) and the surrounding soils or bedrock using the load transfer method. The lateral load-displacement behaviour of the soils and bedrock developed on the face of a given pile can be modeled using p-y curves as described in Section C6.11.2.2.1 of the Commentary to the CHBDC. Thurber can provide p-y curves, if required.

9.2 Backfill and Lateral Earth Pressures

The lateral earth pressures acting on the abutment walls will depend on the type and method of placement of the backfill behind the abutment, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls. Based on the results of the current investigation, the embankment fill generally consists of gravel and sand, trace to some fines to sandy gravel, some silt to silty sand, some gravel. The granular embankment fill was approximately 6.7 to 8.9 m thick with an underside elevation ranging from 85.2 to 86.8 m.

The observed conditions are generally consistent with the original Foundation Investigation and Design Report (1992, Geocres No. 31B-075) which indicated that the abutments were to be backfilled with free-draining engineered fill consisting of either OPSS Granular A or Granular B fill. For consideration of the lateral earth pressures on the abutment walls, the behaviour of the existing embankment fill is expected to range from that with OPSS Granular B Type II to OPSS Granular B Type I. The gradation results of two samples of the more gravelly portions of the embankment fill (Borehole 307-21-1 at 1.8 m depth and Borehole 307-21-2 at 3.6 m depth, see Figure C1) fall within the gradation envelope of OPSS Granular B Type II. The gradation results



of a sample of the sandier portion of the embankment fill encountered in Borehole 307-21-1 at 4.9 m depth (see Figure C1) fall within the gradation envelope of OPSS Granular B Type I.

Lateral earth pressure parameters provided in Table 9-2 and Table 9-3 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 9-2 and Table 9-3 should be used. For other backfill and wall geometries, Thurber will need to calculate the appropriate earth pressure coefficients.

9.2.1 Static Lateral Earth Pressure

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

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

where:

σ_h	=	horizontal pressure on the wall at depth h (kPa)
K	=	earth pressure coefficient (see table below) (K_a for yielding walls, K_o for non-yielding walls)
γ	=	unit weight of retained soil (see table below), 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 earth pressure coefficients for backfill are shown in Table 9-2.

Table 9-2 Static Earth Pressure Coefficients

Condition	OPSS Granular A or Granular B Type II $\phi = 35^\circ, \gamma = 22 \text{ kN/m}^3$	OPSS Granular B Type I $\phi = 30^\circ, \gamma = 21 \text{ kN/m}^3$
Active, K_A (Yielding Wall)	0.27	0.33
At Rest, K_o (Non-Yielding Wall)	0.43	0.50
Passive, K_P (Movement towards Soil Mass) in front of wall	3.7	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.

9.2.2 Combined Static and Seismic Lateral Earth Pressure

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

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

The coefficients of horizontal earth pressure for combined static and seismic loading presented in Table 9-3 may be used. The provided earth pressure coefficients are calculated using a site-adjusted PGA of 0.27 g, based on a Seismic Site Class D, a reference (Site Class C) PGA with a 2% probability of exceedance in 50 years of 0.25 g (Geological Survey of Canada – Fifth Generation) and a $F(PGA)$ of 1.108 as per Table 4.8 of the CHBDC.

Table 9-3 Combined Static and Seismic Earth Pressure Coefficients

Condition	OPSS Granular A or Granular B Type II $\phi = 35^\circ, \gamma = 22 \text{ kN/m}^3$	OPSS Granular B Type I $\phi = 30^\circ, \gamma = 22 \text{ kN/m}^3$
Active, K_{AE} Yielding Wall	0.35	0.42
Active, K_{AE} Non-Yielding Wall	0.45	0.54

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

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)



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 equation above that includes consideration of material properties and the soils profile.

9.3 Embankment Stability

Based on the general arrangement drawing included as part of the original design drawings, the grade of the travelled lanes is at about Elevation 94.0 m to 94.5 m, requiring embankment fill up to 8.5 m high above the existing ground surface, which is at about Elevation 86.5 m. The existing embankments are sloped at about 2H:1V and extend horizontally as much as 17 m near the abutments.

Table 6.2 of the CHBDC for embankment fills with a typical degree of understanding and a Ψ of 1.0 generates minimum Factors of Safety of 1.5 and 1.3 for static permanent and static temporary conditions respectively.

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

Typically, where the initial 1 in 2475 year pseudo-static analyses generates a Factor of Safety less than 1.3, a screening level deformation check should be completed where there are potential implications to the bridge foundations or embankment slopes.

In addition, Sections 6.14.2.1 and 6.14.2.3 of the CHBDC present performance criteria requirements for Major Route geotechnical systems (embankments) inside and outside the bridge interface zone, respectively. Based on Clause 6.14.2.2, the bridge interface zone at this site extends to 20 m behind the abutment (based on a fill height of 8.5 m). The performance criteria for Major Route embankments are as follows:

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

A slope stability assessment was carried out to assess the stability of the existing slopes. Embankment slope stability was evaluated using GeoStudio 2020 Slope/W software for limit



equilibrium analysis. Input parameters for the analysis are based on the SPT N-values and the results of laboratory testing. The following additional parameters were used in the analysis:

- Estimated soil stratigraphy based on the nearest boreholes;
- Side slopes of 2H:1V for the embankment fill;
- A site adjusted PGA value of 0.14 g, equal to $\frac{1}{2}$ of the site adjusted PGA value (0.27 g), was used for the seismic analysis, as per Section 4.4.3.3, of the CHBDC and outlined in Section 8.2 above; and,
- A traffic surcharge of 17 kPa applied as a temporary load.

Copies of the output from the stability analyses are provided on the figures presented in Appendix G. Each output 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:

Table 9-4 Slope Stability Analysis Results

Condition	Case	Factor of Safety
Permanent (no traffic loading)	Long Term Static (Drained)	1.5 (Fig G1)
Temporary (includes seismic)	Pseudo-Static Seismic (Undrained)	1.5 (Fig G2)
Temporary (traffic loading)	Short Term Static (Undrained)	2.0 (Fig G3)

All of the static results presented in the Table 9-4 achieve the target Factors of Safety described above. In addition, the pseudo-static analysis for a 1 in 2475 year seismic event yields a value of 1.5 which achieves the target stability value and indicates that a screening level deformation check is not required. Furthermore, the 1 in 2475 year pseudo-static results suggest that the performance criteria requirements for this embankment (which would be based on a 1 in 475 year event) are also achieved.



10 RECOMMENDED SCOPE FOR DETAIL DESIGN

The recommendations provided above are in support of the preliminary design of the proposed rehabilitation of the Highway 416 SBL Cedar Grove Overpass (Site No. 16X-0307) as part of the overall Preliminary Design and Environmental Assessment for the widening of Highway 401 from Highway 416 to Maitland Road. Depending on the scope of the design rehabilitation works, 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 in the 30 m below the foundation elements to confirm Seismic Site Classification
- Testing of soil and/or groundwater at the site to determine degree of corrosiveness of the sub-surface environment and potential for sulphate attack on steel and concrete elements in contact with the soil and groundwater at the site
- Installation of additional piezometer(s) and confirmation of design groundwater level(s) at the site

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



11 CLOSURE

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

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ⁱ Boulanger, R. W. and Idriss, I. M. (2007). Evaluation of cyclic softening in silts and clays, ASCE, Journal of Geotechnical and Geoenvironmental Engineering, 133(6), 641-652.

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

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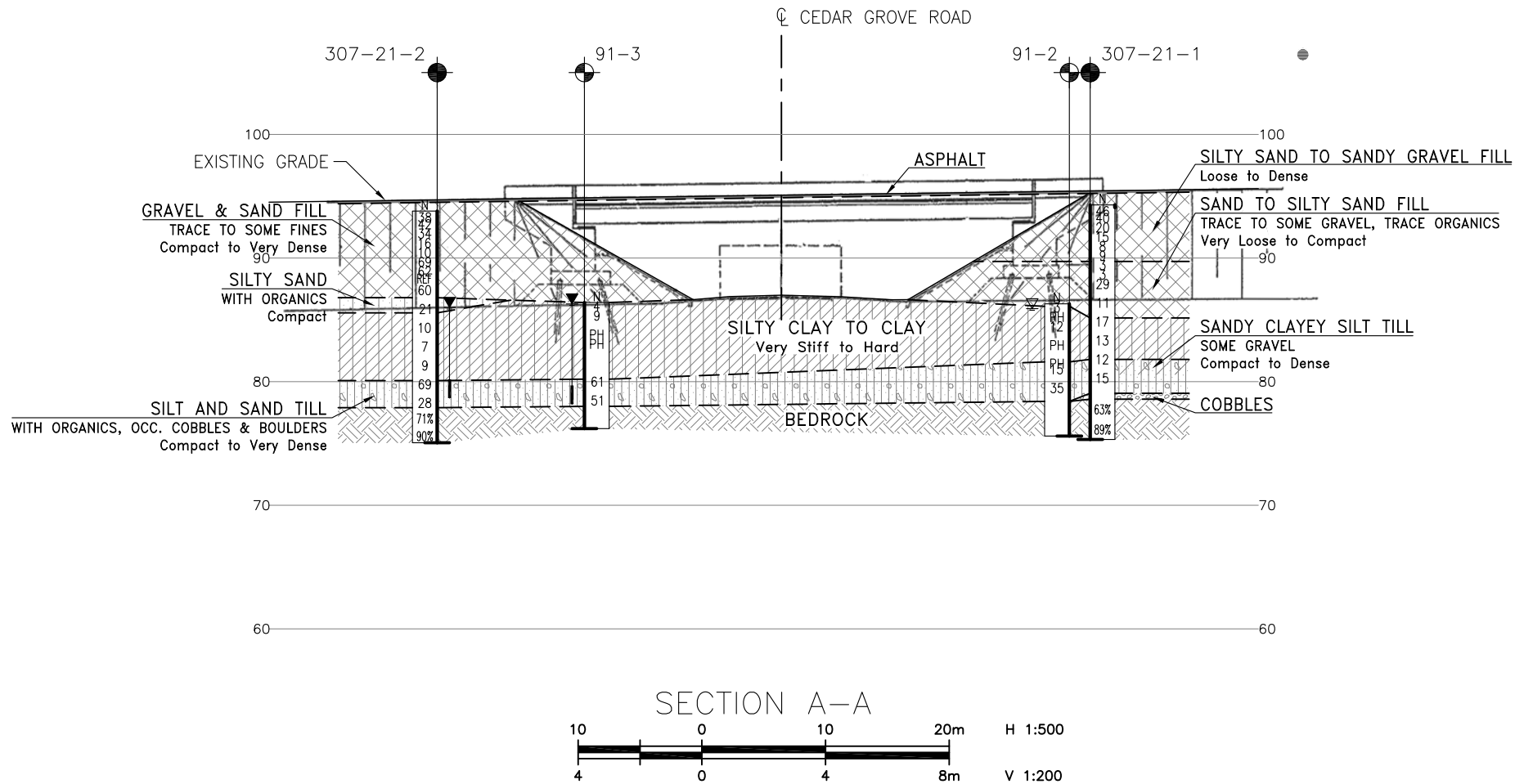
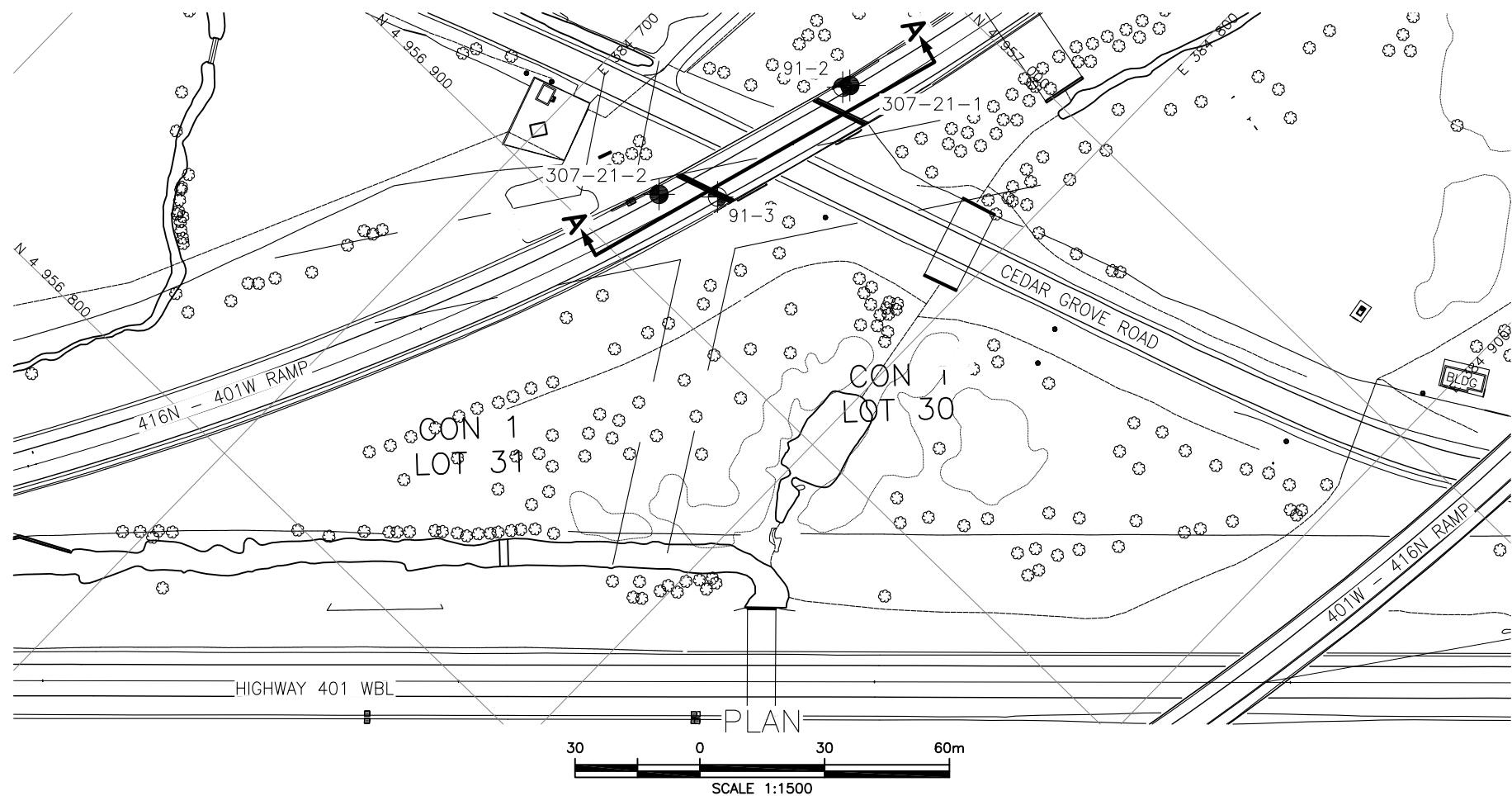
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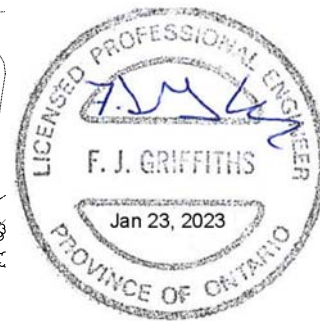


Appendix A.

Borehole Location Plan and Stratigraphic Drawings



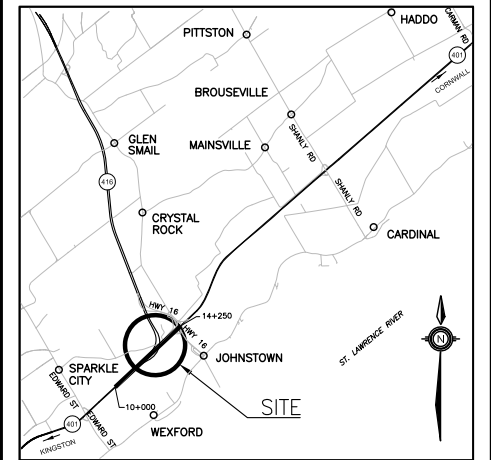
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AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



CONT No
GWP No 4024-20-00

HIGHWAY 401
416N - 401W RAMP
BRIDGE OVER CEDAR GROVE RD
BOREHOLE LOCATIONS AND SOIL STRATA

Ontario



KEYPLAN

LEGEND

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

NO	ELEVATION	NORTHING	EASTING
307-21-1	94.3	4 956 968.2	384 746.3
307-21-2	93.8	4 956 917.5	384 731.6
91-2	86.3	4 956 966.7	384 745.4
91-3	86.4	4 956 926.8	384 742.2

-NOTES-

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

GEOCRES No. 31B-104

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	MJK	CHK -	CODE
DRAWN	MFA	CHK MK	SITE 16-307
			LOAD
			DATE JAN 2023
			DWG 1



Appendix B.

Record of Borehole Sheets



Appendix B.1

Current (2021) Investigation



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).

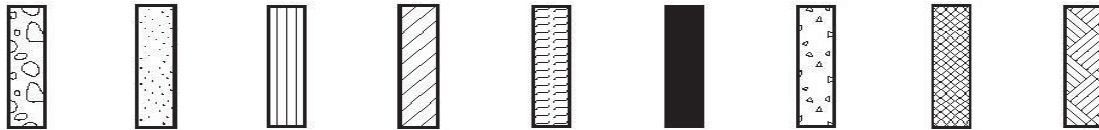
DYNAMIC CONE PENETRATION TEST (DCPT):

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 307-21-1

1 OF 2

METRIC

GWP# 4024-20-00 LOCATION Lat: 44.748254°, Long: -75.490366° N 4 956 968.2 E 384 746.3 ORIGINATED BY JP
 HWY 401 BOREHOLE TYPE CME 55 Truckmount, HSA/NQ Coring COMPILED BY SH
 DATUM Geodetic DATE 2021.04.26 - 2021.04.27 CHECKED BY MJK

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			20 40 60 80 100	W _P W W _L					
								SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE		WATER CONTENT (%)				
94.3														GR SA SI CL
0.0	ASPHALT (200 mm)													
0.2	SILTY SAND, some gravel to SANDY GRAVEL, some silt Brown Loose to dense FILL		1	SS	46		94							
			2	SS	40									
			3	SS	20									
			4	SS	15									
			5	SS	8									
			6	SS	9									
89.7														
4.6	SAND, some silt to SILTY SAND, trace to some gravel No to trace organics Brown to brown with black Very loose to compact FILL		7	SS	3		90							
8	SS		3											
9	SS		29											
10	SS		11											
85.2														
9.1	SILTY CLAY to CLAY Grey-brown Hard WEATHERED CRUST		11	SS	17		85							

DOUBLE LINE 29381 BOREHOLE LOGS REHAB SITES.GPJ 2012TEMPLATE(MTO).GDT 12-23-22

Continued Next Page

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

RECORD OF BOREHOLE No 307-21-1

2 OF 2

METRIC

GWP# 4024-20-00 LOCATION Lat: 44.748254°, Long: -75.490366° N 4 956 968.2 E 384 746.3 ORIGINATED BY JP
 HWY 401 BOREHOLE TYPE CME 55 Truckmount, HSA/NQ Coring COMPILED BY SH
 DATUM Geodetic DATE 2021.04.26 - 2021.04.27 CHECKED BY MJK

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
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DOUBLE LINE 29381 BOREHOLE LOGS REHAB SITES.GPJ 2012TEMPLATE(MTO).GDT 12-23-22

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

RECORD OF BOREHOLE No 307-21-2

1 OF 2

METRIC

GWP# 4024-20-00 LOCATION Lat: 44.747799°, Long: -75.49056° N 4 956 917.5 E 384 731.6 ORIGINATED BY JP
 HWY 401 BOREHOLE TYPE CME 55 Truckmount, HSA/NQ Coring COMPILED BY SH
 DATUM Geodetic DATE 2021.04.27 - 2021.04.27 CHECKED BY MJK

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 (%)					
93.8							20 40 60 80 100	○ UNCONFINED + FIELD VANE	W _P W W _L					GR SA SI CL	
0.0	ASPHALT (275 mm)						20 40 60 80 100	● QUICK TRIAXIAL × LAB VANE							
93.5															
0.3	GRAVEL and SAND, trace to some fines Grey-brown to brown Compact to very dense FILL		1	SS	38										
			2	SS	42										
			3	SS	34										
			4	SS	16										
			5	SS	10										
			6	SS	69										
			7	SS	62										
			8	SS	REF										
			9	SS	60										
86.8															
7.0	SILTY SAND with organics Brown Compact														
			10	SS	21										
85.6															
8.2	SILTY CLAY Grey-brown Very stiff WEATHERED CRUST														
			11	SS	10										
								</							

DOUBLE LINE 29381 BOREHOLE LOGS REHAB SITES.GPJ 2012TEMPLATE(MTO).GDT 12-23-22

Continued Next Page

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

RECORD OF BOREHOLE No 307-21-2

2 OF 2

METRIC

GWP# 4024-20-00 LOCATION Lat: 44.747799°, Long: -75.49056° N 4 956 917.5 E 384 731.6 ORIGINATED BY JP
 HWY 401 BOREHOLE TYPE CME 55 Truckmount, HSA/NQ Coring COMPILED BY SH
 DATUM Geodetic DATE 2021.04.27 - 2021.04.27 CHECKED BY MJK

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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE									
	Continued From Previous Page						20 40 60 80 100										
83.1	SILTY CLAY Grey-brown Very stiff WEATHERED CRUST																
10.7	SILTY CLAY Grey Very stiff		12	SS	7		83						○				
							82										
			13	SS	9		81						○				
80.1							80										
13.7	SILTY SAND Grey Compact to very dense Occasional cobbles/boulders GLACIAL TILL		14	SS	69		79						○				
							78										
77.9			15	SS	28		77						○				
15.9	DOLOSTONE with quartz vugs Fresh Grey Fine Grained		1	RUN			76										
							75										
18.7	End of Borehole Flushmount 19 mm diameter PVC monitoring well installed. Well Readings: Date: Depth (m): Elev. (m): 2021/07/01 7.8 86.0 2022/12/19 2.0 91.8																

DOUBLE LINE 29381 BOREHOLE LOGS REHAB SITES.GPJ 2012TEMPLATE(MTO).GDT 12-23-22

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



Appendix B.2

Previous (1991/1992) Investigation



RECORD OF BOREHOLE No 91-2

METRIC

W P 374-89-03 LOCATION Co-ords: N: 4 956 966.7 E: 384 745.4 ORIGINATED BY Y.L.
DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Auger, N-Casing, Rock Coring COMPILED BY C.K.K.
DATUM Geodetic DATE May 3, 1991 CHECKED BY G.J.K.

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	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100													
								SHEAR STRENGTH kPo					WATER CONTENT (%)								
86.3	Ground Surface							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE													
	Topsoil						Seal														
0.2	Sand, trace silt		1	SS	3		86														
0.4	Silty Clay Very Stiff to Hard						June 25, 1991														
			2	TW	PH		Native Material														
							85														
			3	SS	12																
							84														
	Brown Grey						Seal														
			4	TW	PH		83														
							Sand Backfill														
							Piezometer														
81.6							82														
4.7	Het. Mixture of Sandy Silt, some clay and gravel, occ. boulders (Glacial Till) Compact to Dense Grey		5	TW	PH		Seal														
			6	SS	15		Native Material														
							81														
							Seal														
			7	SS	35		Sand Backfill														
							80														
							79														
							Piezometer														
78.4																					
7.9	Bedrock Dolomitic Limestone Fair to Excellent		8	NQ RC	REC 100%		Native Material										RQD = 53%				
			9	NQ RC	REC 100%												RQD = 97%				
							76														
75.6																					
10.7	End of Borehole																				

OFFICE REPORT ON SOIL EXPLORATION

+3, x5: Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10

METRIC

W P 374-89-03 LOCATION Co-ords: N: 4 956 926.8 E: 384 742.2 ORIGINATED BY Y.L.
DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Auger, N-Casing, Rock Coring COMPILED BY C.K.K.
DATUM Geodetic DATE May 3, 1991 CHECKED BY G.J.K.

[illegible]

+3, x5: Numbers refer to Sensitivity



Appendix C.

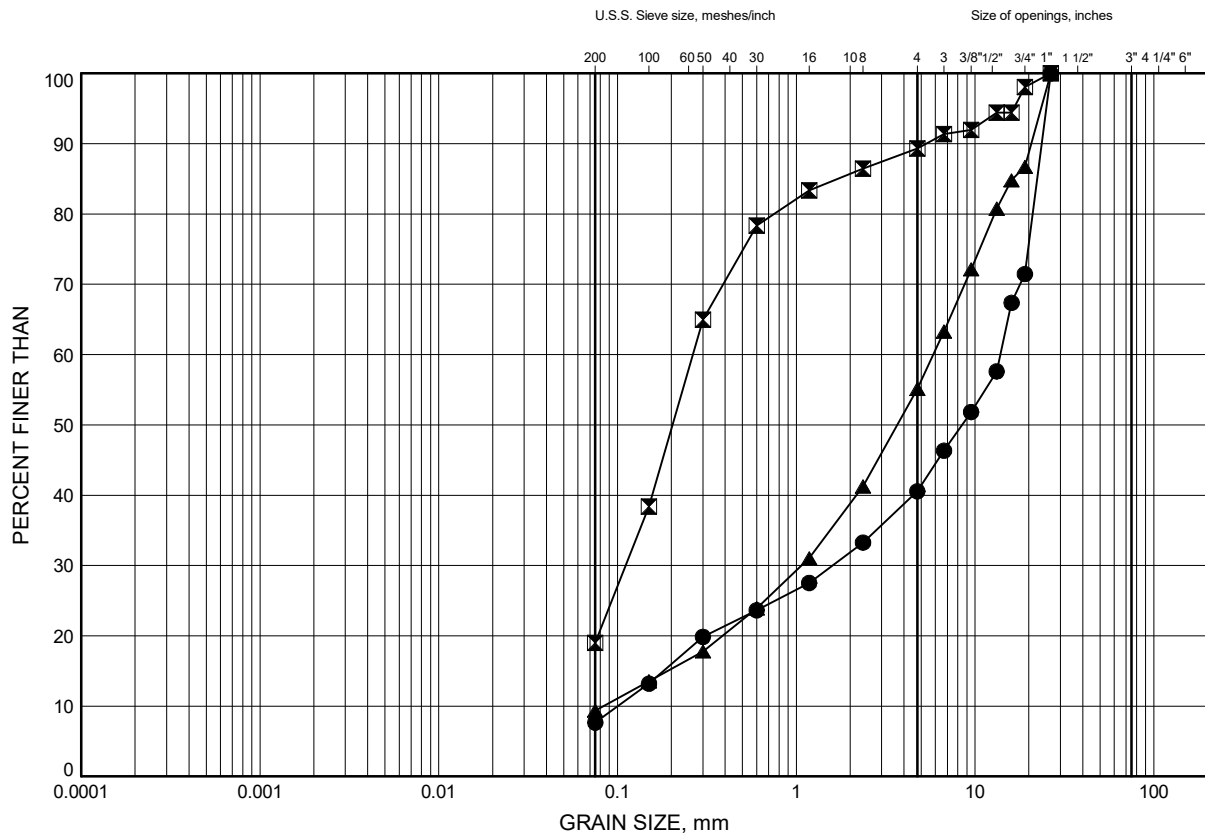
Laboratory Testing



Appendix C.1
Particle Size Analysis Figures (2021)
Atterberg Limit Test Results (2021)

GRAIN SIZE DISTRIBUTION

EMBANKMENT FILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	307-21-1	1.8	92.5
⊠	307-21-1	4.9	89.5
▲	307-21-2	3.6	90.2

Date November 2021

WP# 4024-20-00

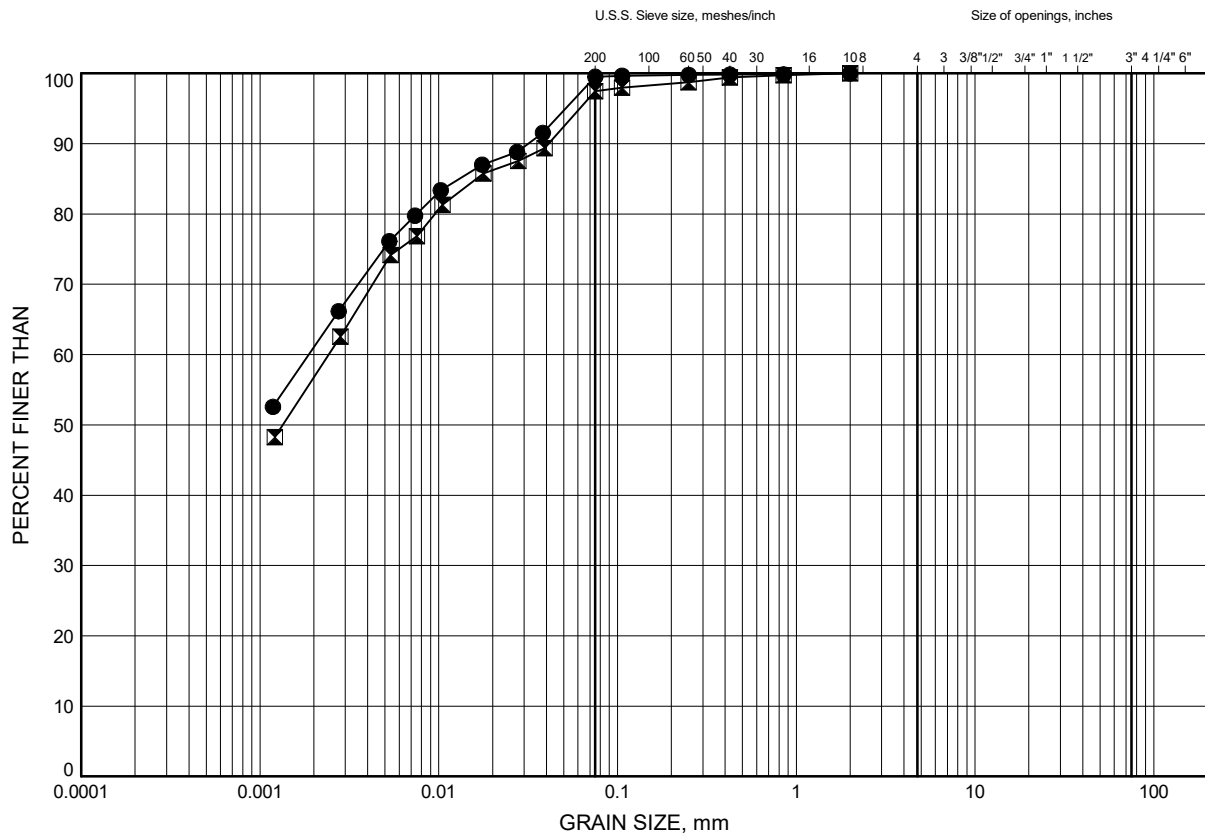


Prep'd SH

Chkd. MJK

GRAIN SIZE DISTRIBUTION

SILTY CLAY TO CLAY



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	307-21-1	11.0	83.4
⊠	307-21-2	9.4	84.3

Date November 2021

WP# 4024-20-00

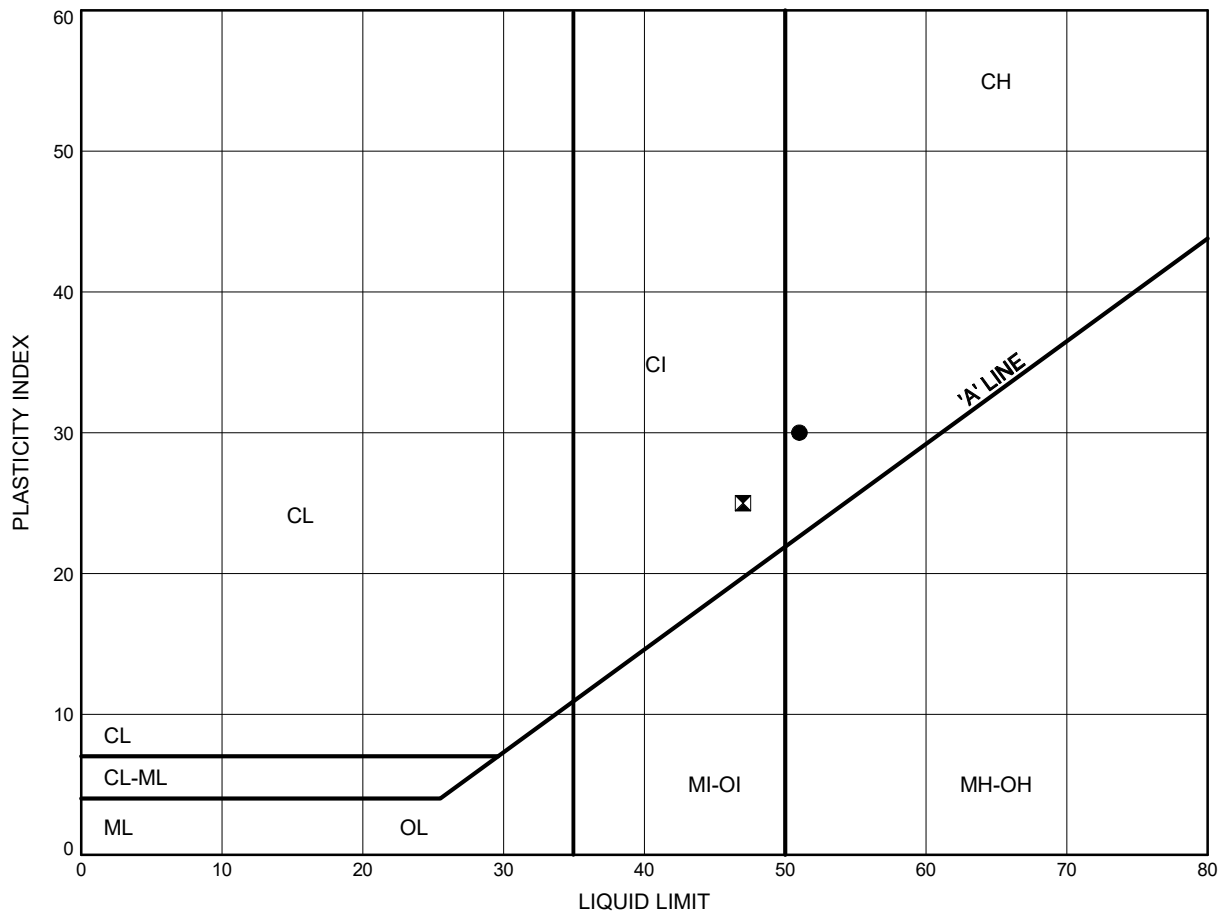


Prep'd SH

Chkd. MJK

ATTERBERG LIMITS TEST RESULTS

SILTY CLAY TO CLAY



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	307-21-1	11.0	83.4
⊠	307-21-2	9.4	84.3

Date November 2021

WP# 4024-20-00

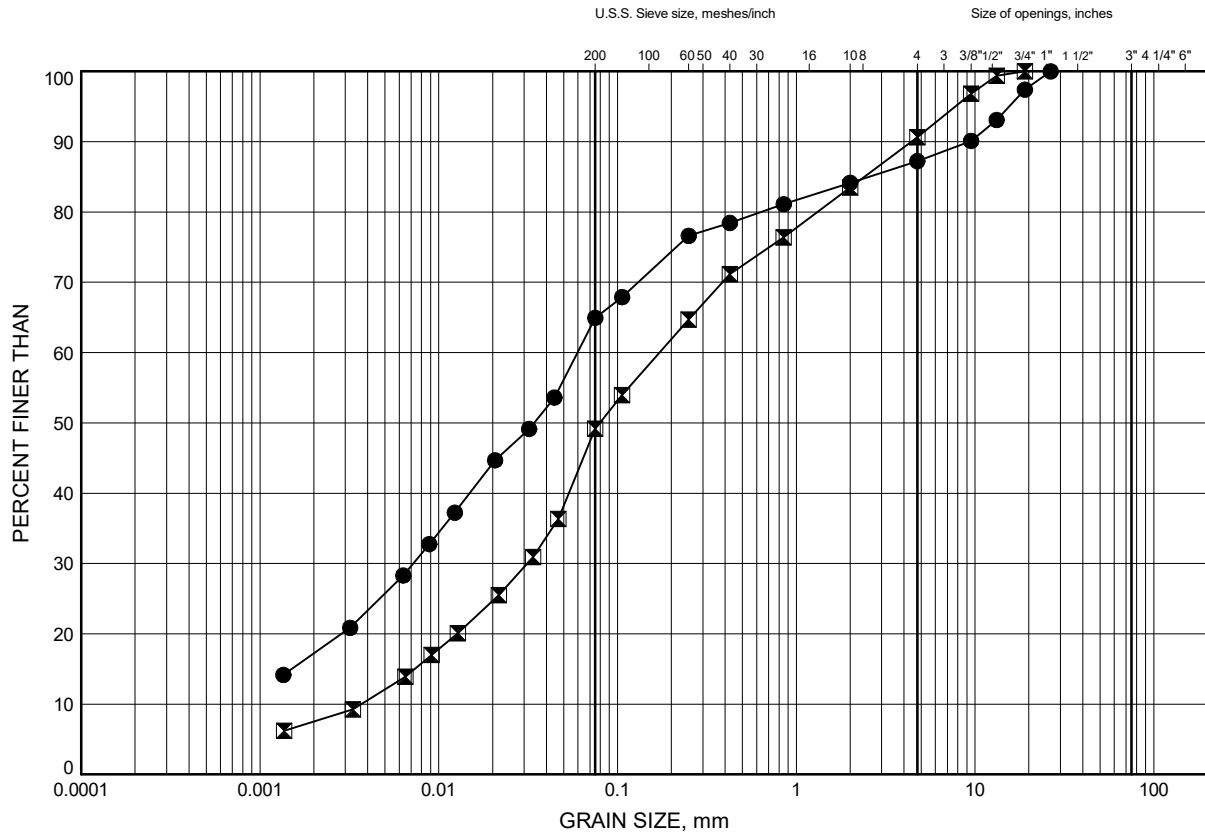


Prep'd SH

Chkd. MJK

GRAIN SIZE DISTRIBUTION

GLACIAL TILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	307-21-1	12.6	81.7
⊠	307-21-1	14.0	80.3

Date November 2021

WP# 4024-20-00

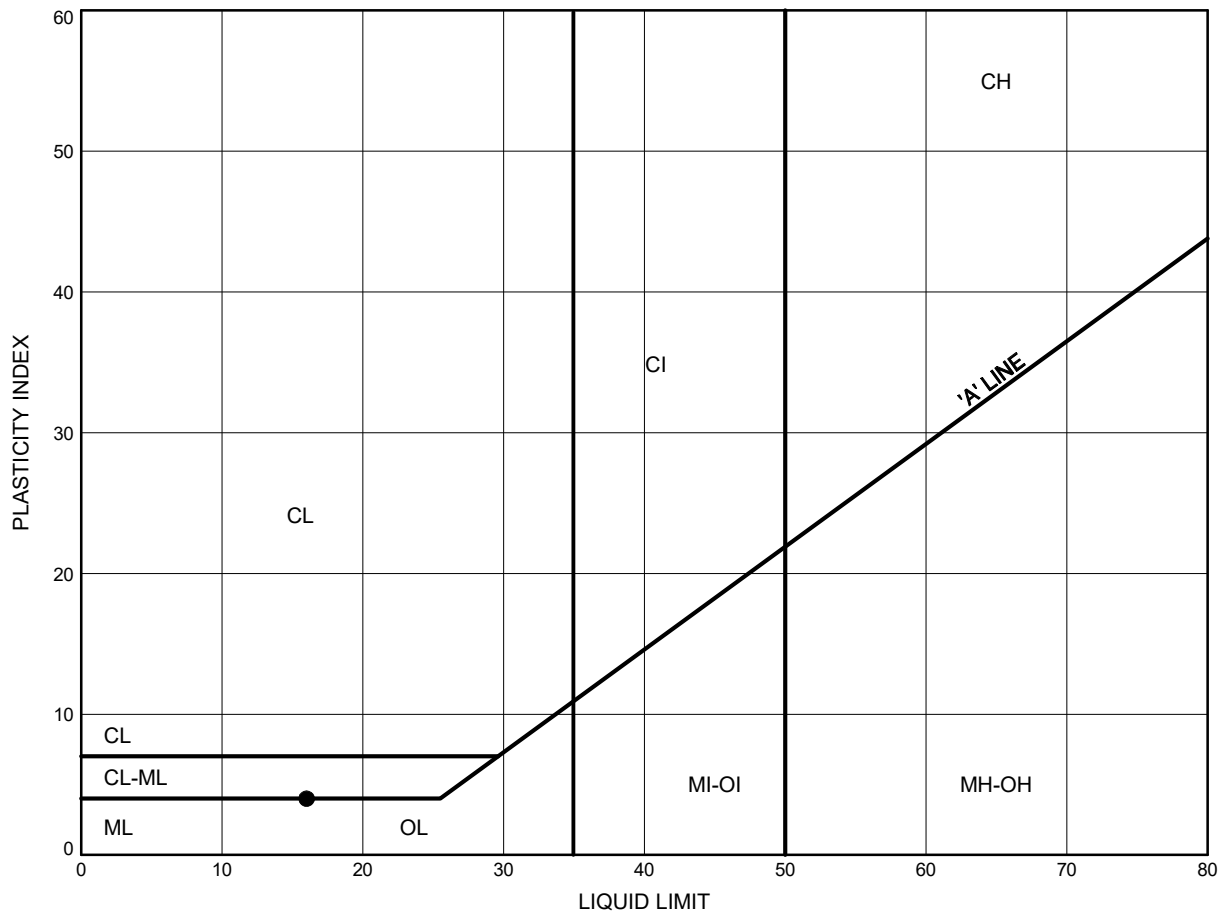


Prep'd SH

Chkd. MJK

ATTERBERG LIMITS TEST RESULTS

GLACIAL TILL



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	307-21-1	12.6	81.7

Date November 2021

WP# 4024-20-00



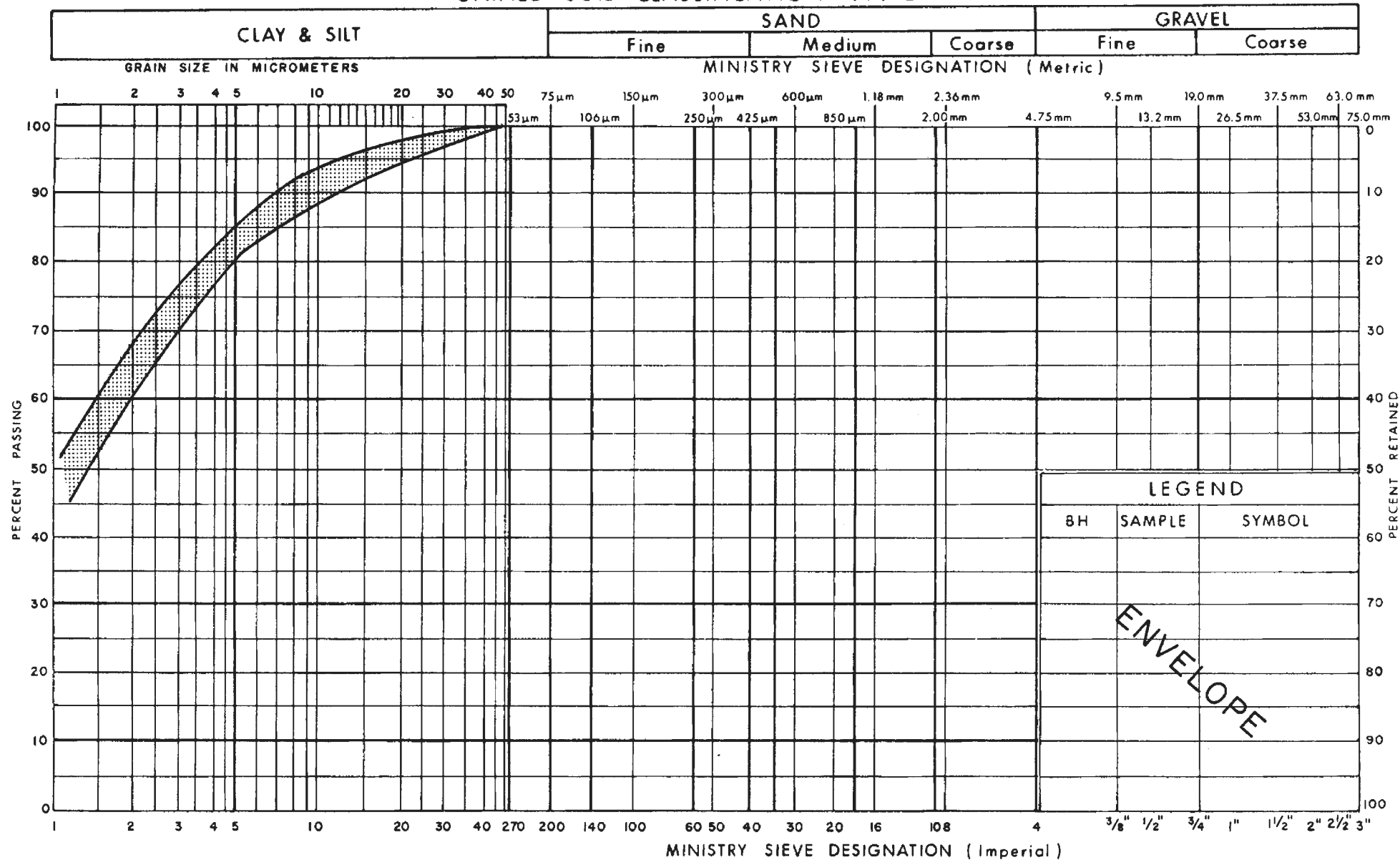
Prep'd SH

Chkd. MJK



Appendix C.2
Particle Size Analysis Figures (1991)
Atterberg Limit Test Results (1991)

UNIFIED SOIL CLASSIFICATION SYSTEM

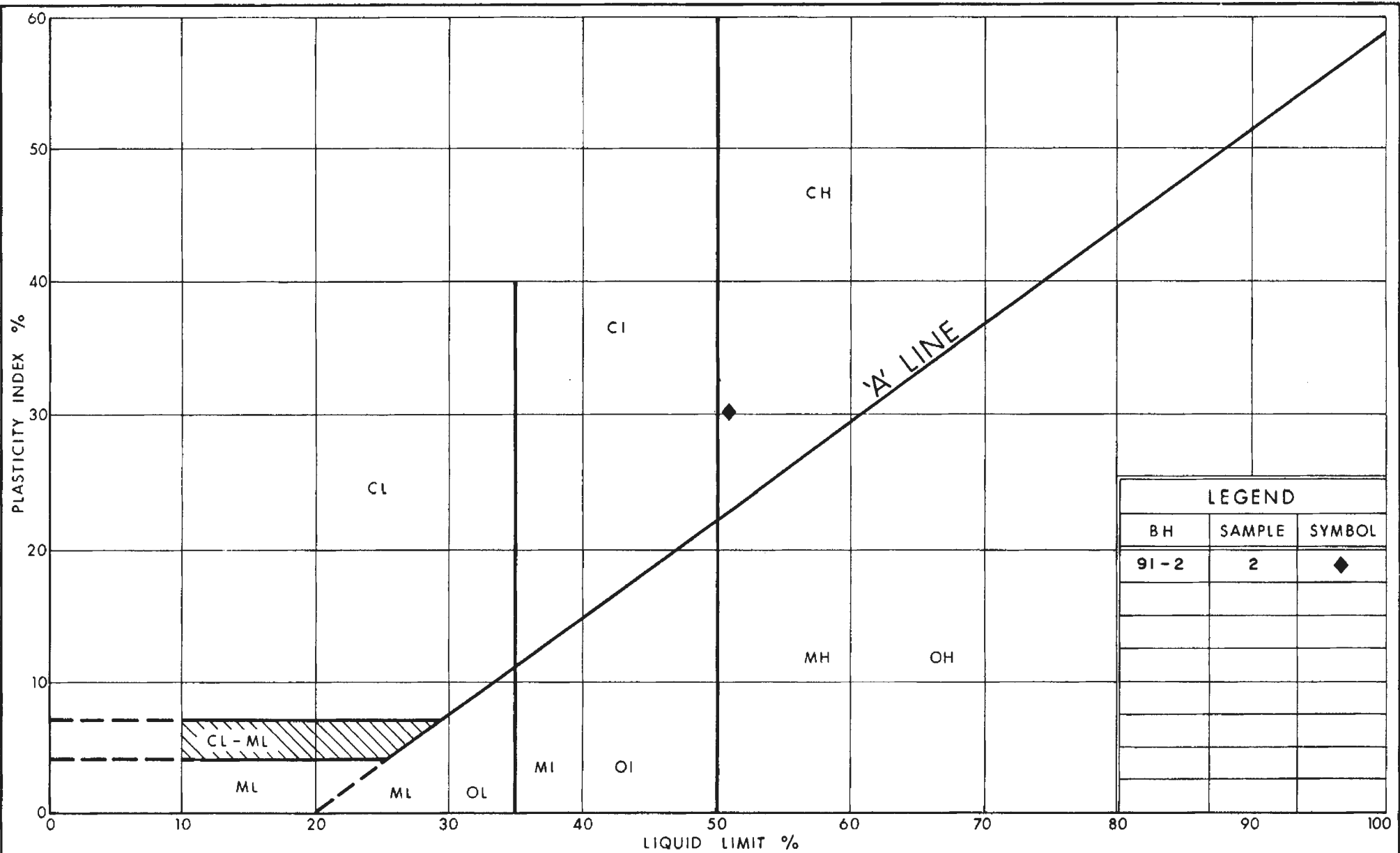


Ministry of
Transportation

GRAIN SIZE DISTRIBUTION
SILTY CLAY

FIG No 1

W P 374-89-03



LEGEND		
BH	SAMPLE	SYMBOL
91-2	2	◆



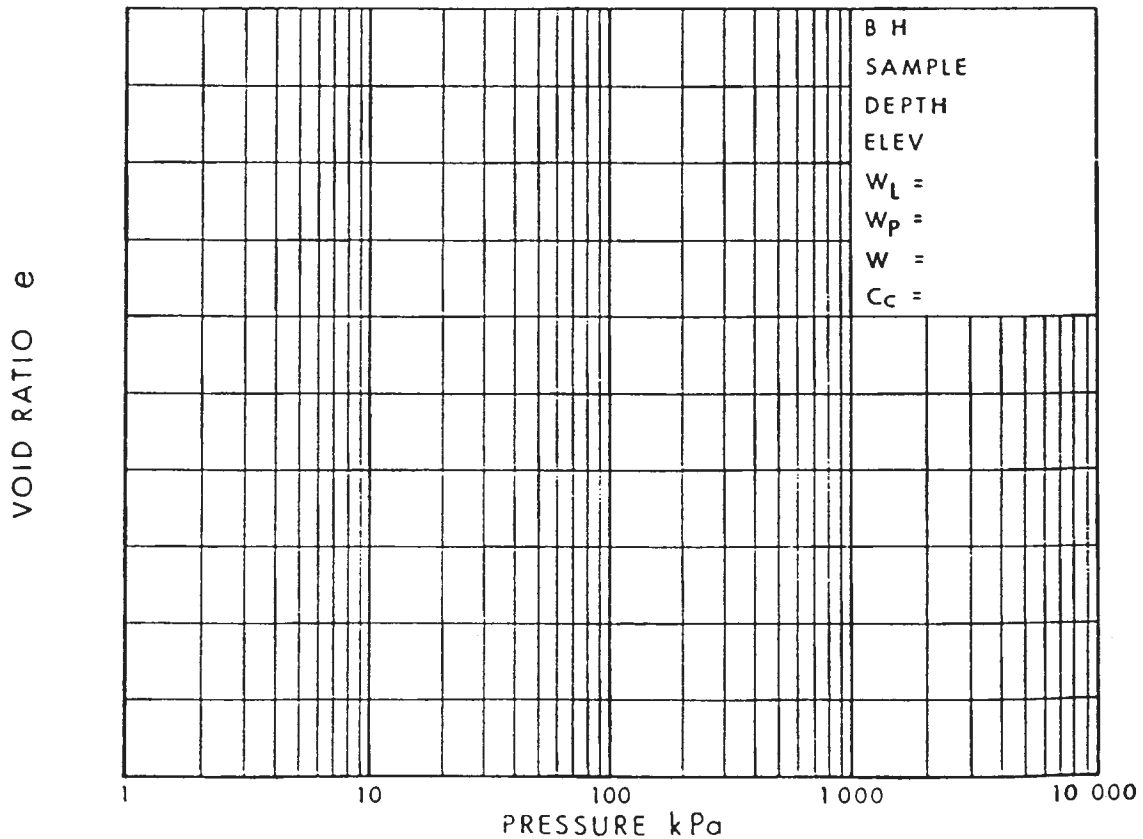
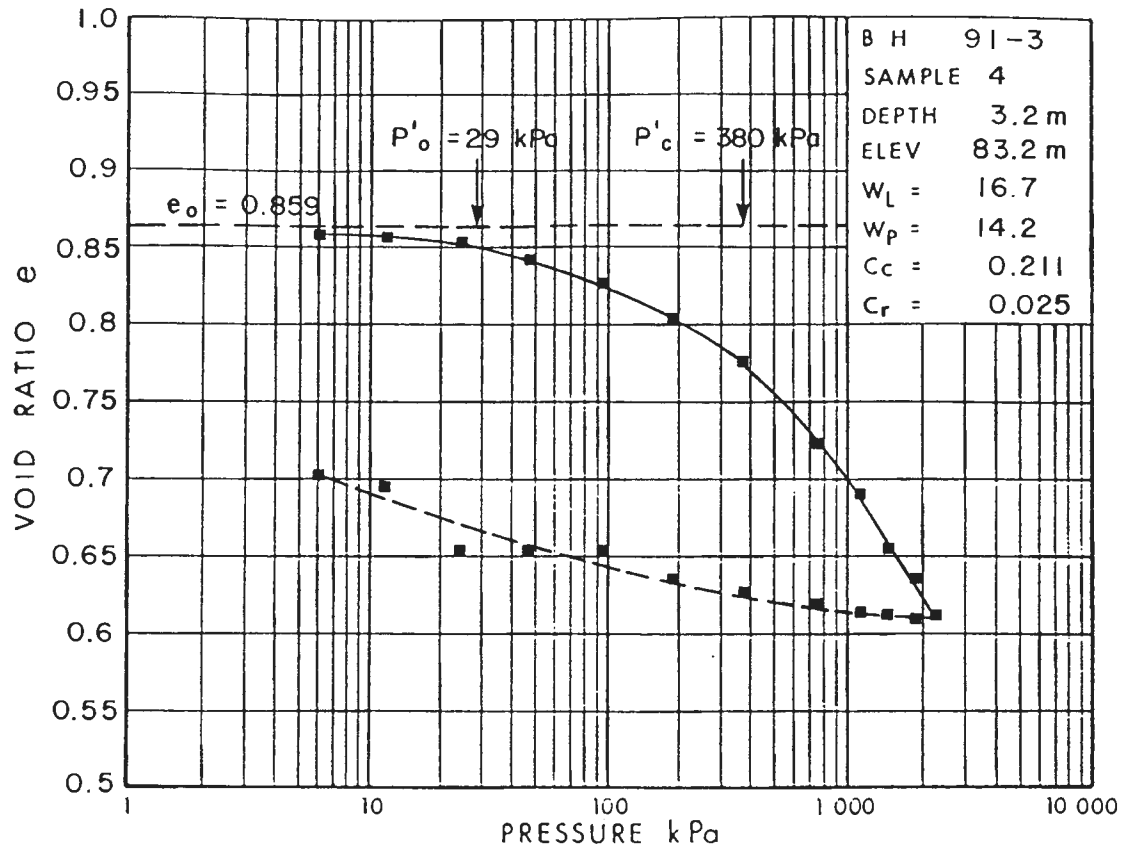
Ministry of
Transportation
Ontario

PLASTICITY CHART SILTY CLAY

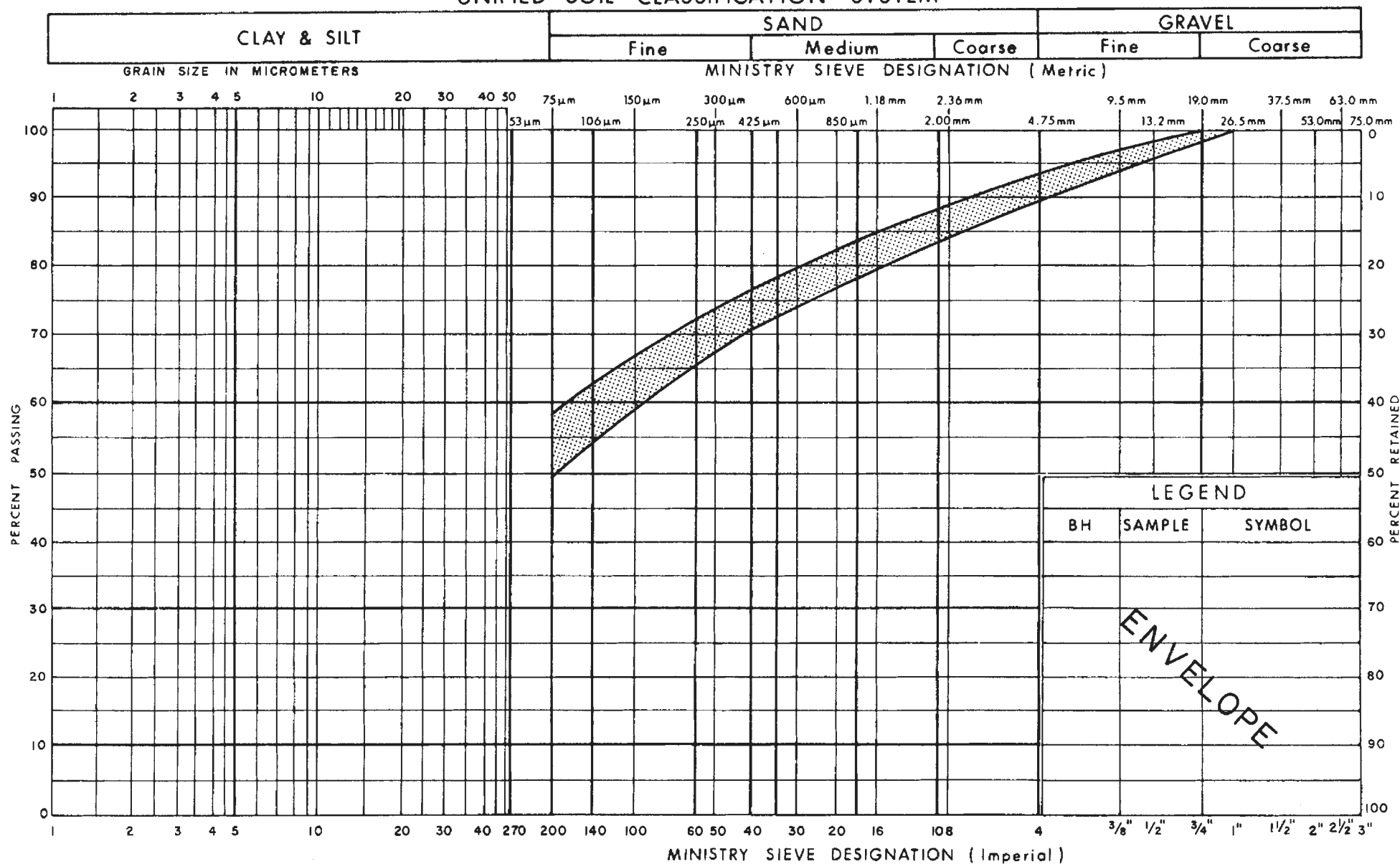
FIG No 2

W P 374-89-03

VOID RATIO - PRESSURE CURVES



UNIFIED SOIL CLASSIFICATION SYSTEM

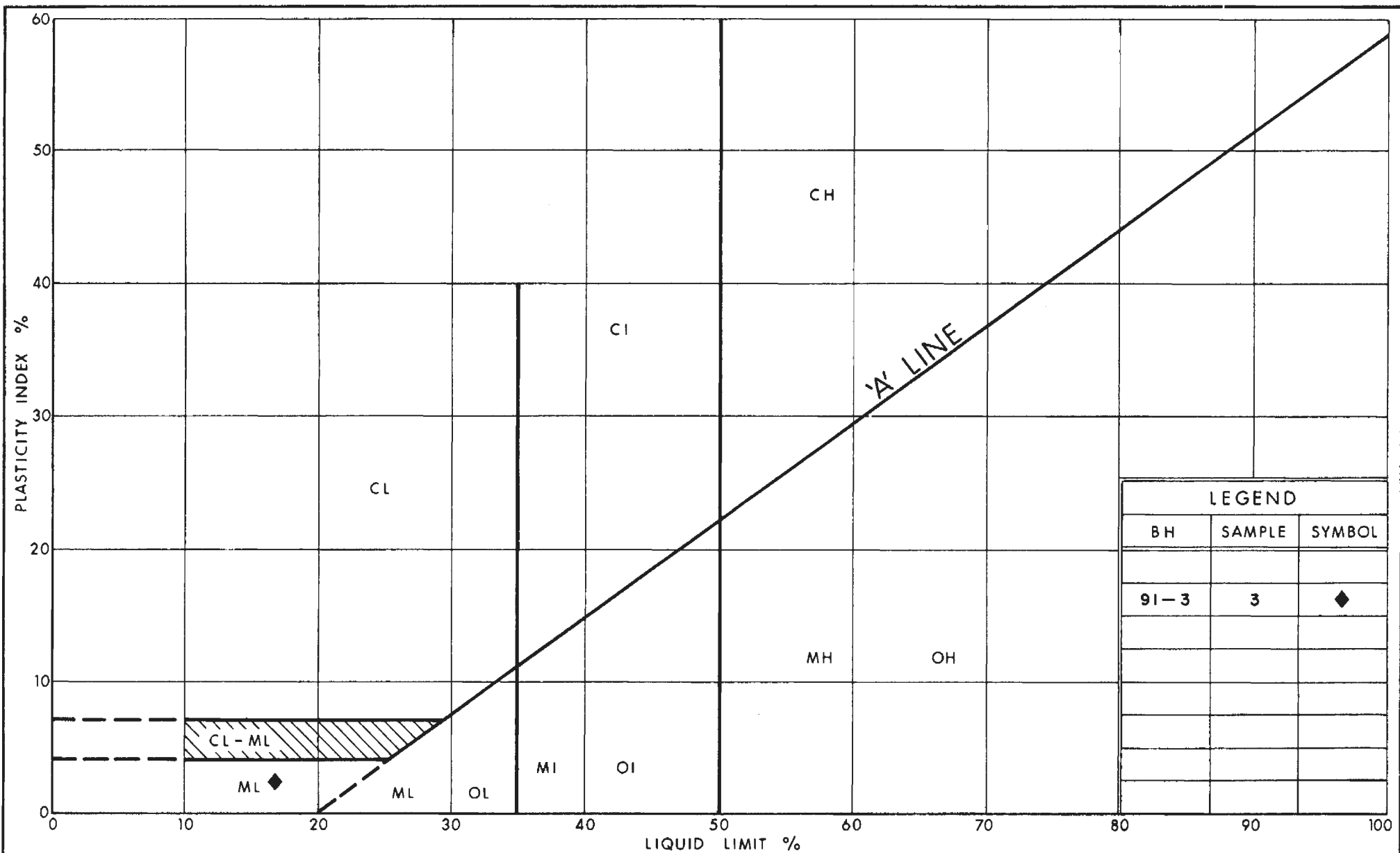


Ministry of
Transportation

GRAIN SIZE DISTRIBUTION
HET MIXTURE OF SANDY SILT,
SOME CLAY & GRAVEL, OCCASIONAL BOULDERS (Glacial Till)

FIG No. 4

WP 374-89-03



Ministry of
Transportation

Ontario

PLASTICITY CHART
HET MIXTURE OF SANDY SILT,
SOME CLAY & GRAVEL, OCCASIONAL BOULDERS (Glacial Till)

FIG No 5

W P 374 - 89 - 03



Appendix C.3

UCS Test Results



Stantec

Stantec Consulting Ltd
2781 Lancaster Rd, Suite 100 A&B
Ottawa, ON K1B 1A7
Tel: (613) 738-6075
Fax: (613) 722-2799

May 25, 2021
File: 122410864

Attention: Thurber Engineering, File #29381

Reference: ASTM D7012, Method C, Unconfined Compressive Strength of Intact Rock Core
Highway 401/416 Interchange

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

Location	Sample Depth	Compressive Strength (MPa)	Description of Break
259-21-1 Run-2	8'6"-9'1"	205.3	Well-formed cone at both ends
306-21-2 Run-1	77'2"-77'9"	219.8	Well-formed cone at both ends
307-21-1 Run-1	55'-55'7"	162.4	Well-formed cone at both ends
308-21-1 Run-2	72'6"-73'3"	216.9	Vertical cracking throughout, no well-formed cones.
250-21-21 Run-2	24'8"-25'3"	181.6	Well-formed cone at both ends

Sincerely,

Stantec Consulting Ltd

Brian Prevost

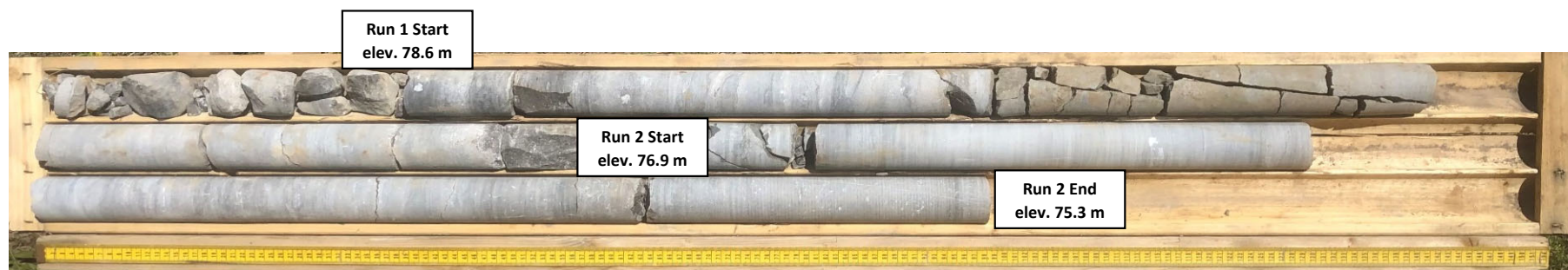
Brian Prevost
Laboratory Supervisor
Tel: 613-738-6075
brian.prevost@stantec.com



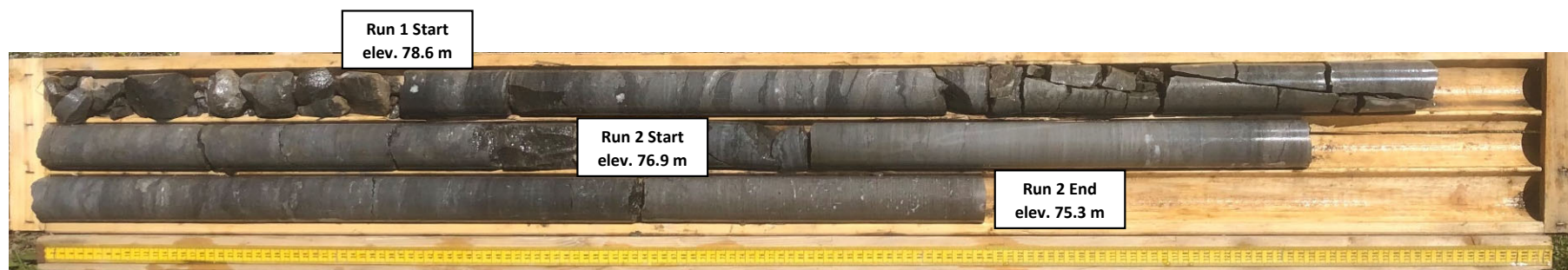
Appendix C.4

Bedrock Core Photographs

Borehole 307-21-1
Run 1 to 2 (of 2)
Elevation 78.8 m to 75.4 m
Dry



Borehole 307-21-1
Run 1 to 2 (of 2)
Elevation 78.8 m to 75.4 m
Wet



Borehole 307-21-2
Run 1 to 2 (of 2)
Elevation 77.9 m to 75.0 m

Run 1 Start
elev. 77.9 m



Borehole 307-21-2
Run 1 to 2 (of 2)
Elevation 77.9 m to 75.0 m

Run 1 Start
elev. 77.9 m





Appendix D.

Site Photographs



Photo 1. Looking north at the west side of the north embankment (2021/03/29)



Photo 2. West side of north abutment (2021/03/29)



Photo 3. North deck and abutment (2021/03/29)



Photo 4. Asphalt paved surface of 416N-401W ramp bridge (2021/03/29)



Photo 5. Looking south at south abutment (2021/03/29)



Photo 6. South deck and abutment looking east (2021/03/29)



Photo 7. Looking south at the west side of the south embankment (2021/03/29)



Photo 8. Looking north 416N-401W ramp bridge (2021/03/29)



Appendix E.

GSC Seismic Hazard Calculation

2015 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

Site: 44.748N 75.490W

User File Reference: Site 16-307

2021-06-09 19:00 UT

Requested by: Sarah Harrold, 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)	0.384	0.216	0.129	0.037
Sa (0.1)	0.453	0.265	0.165	0.052
Sa (0.2)	0.382	0.227	0.145	0.048
Sa (0.3)	0.292	0.175	0.113	0.039
Sa (0.5)	0.209	0.126	0.081	0.028
Sa (1.0)	0.106	0.064	0.042	0.014
Sa (2.0)	0.051	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)	0.245	0.145	0.090	0.028
PGV (m/s)	0.173	0.100	0.062	0.019

Notes: Spectral ($S_a(T)$, where T is the period in seconds) and peak ground acceleration (PGA) values are given in units of g (9.81 m/s^2). Peak ground velocity is given in m/s . Values are for "firm ground" (NBCC2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are highlighted in yellow. Three additional periods are provided - their use is discussed in the NBCC2015 Commentary. Only 2 significant figures are to be used. **These values have been interpolated from a 10-km-spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the directly calculated values.**

References

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

Structural Commentaries (User's Guide - NBC 2015: Part 4 of Division B)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File 7893 Fifth Generation Seismic Hazard Model for Canada: Grid values of mean hazard to be used with the 2015 National Building Code of Canada

See the websites www.EarthquakesCanada.ca and www.nationalcodes.ca for more information



Natural Resources
Canada

Ressources naturelles
Canada

Canada



Appendix F.

General Arrangement and Foundation Layout Drawings (July 1992)

DELCAN ENGINEER
PLANNER
ARCHITECTS

GENERAL NOTES:

CLASS OF CONCRETE

DECK	35 MPa
REMAINDER	30 MPa

CLEAR COVER TO REINFORCING STEEL

FOOTINGS	100 ± 25 mm
ABUTMENTS AND WINGWALLS	
FRONT FACE	80 ± 20 mm
BACK FACE	70 ± 20 mm
DECK	
TOP	70 ± 20 mm
BOTTOM AND SIDES	50 ± 10 mm
REMAINDER	70 ± 20 mm

UNLESS OTHERWISE SPECIFIED

REINFORCING STEEL

REINFORCING STEEL SHALL BE GRADE 400 UNLESS OTHERWISE SPECIFIED. BAR MARKS WITH SUFFIX "C" DENOTE COATED BARS.

CONSTRUCTION NOTES

THE CONTRACTOR SHALL ESTABLISH THE BEARING SEAT ELEVATIONS BY DEDUCTING THE ACTUAL BEARING THICKNESSES FROM THE TOP OF BEARING ELEVATIONS. IF THE ACTUAL BEARING THICKNESSES ARE DIFFERENT FROM THOSE GIVEN WITH THE BEARING DESIGN DATA, THE CONTRACTOR SHALL ADJUST THE REINFORCING STEEL TO SUIT.

FIRST STAGE FILL, MAXIMUM GRAIN SIZE 75 mm SHALL BE PLACED UP TO THE BOTTOM OF FOOTING ELEVATION PRIOR TO DRIVING PILES.

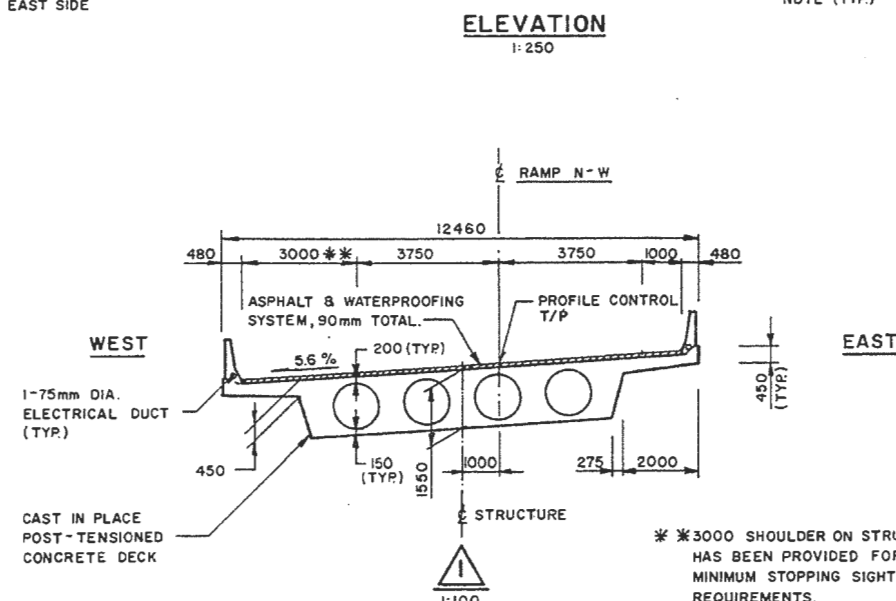
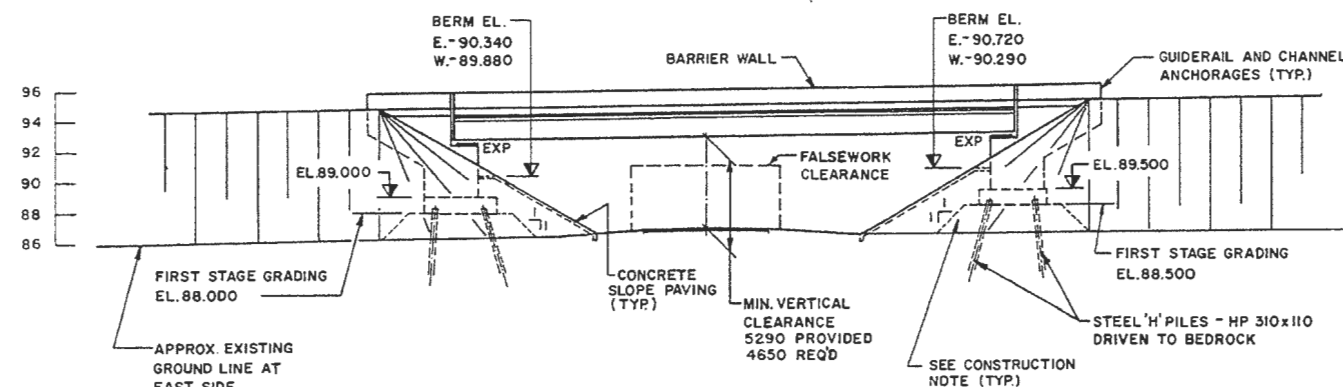
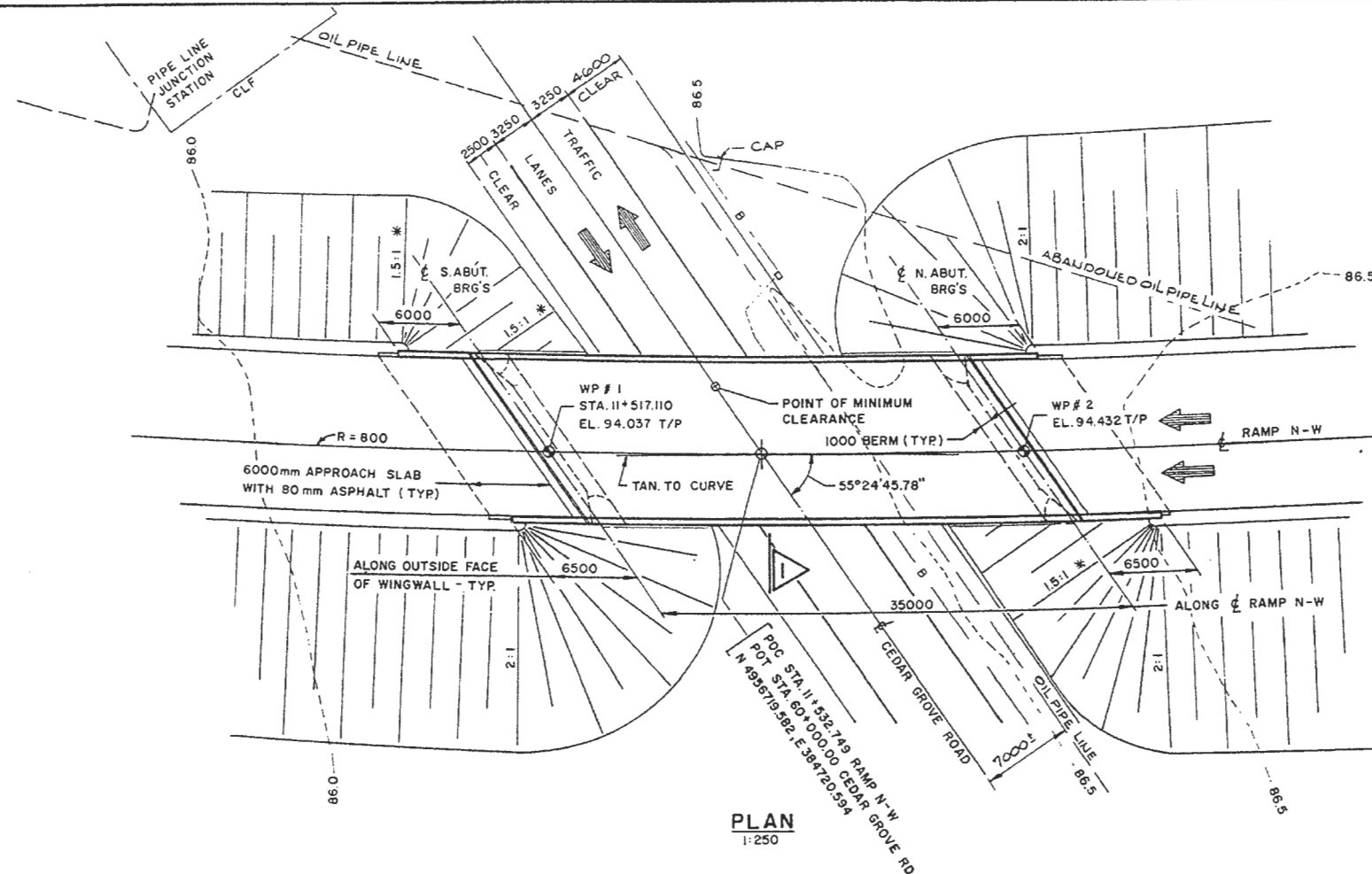
LIST OF DRAWINGS

1. GENERAL ARRANGEMENT
2. BORE HOLE LOCATIONS & SOIL STRATA
3. FOOTING LAYOUT
4. FOOTING REINFORCING
5. SOUTH ABUTMENT I
6. SOUTH ABUTMENT II
7. NORTH ABUTMENT I
8. NORTH ABUTMENT II
9. DECK DETAILS
10. LONGITUDINAL TENDONS
11. TRANSVERSE TENDONS
12. DECK REINFORCING I
13. DECK REINFORCING II
14. JOINT ANCHORAGE AND ARMOURING - ASSEMBLY
15. JOINT ANCHORAGE AND ARMOURING - DETAILS
16. BARRIER WALL W/O RAILING
17. 600mm CURB EACH A.B.
18. DETAILS OF CONCRETE SLOPE PAVING
19. ELECTRICAL EMBEDDED WORK
20. QUANTITIES - STRUCTURE I
21. QUANTITIES - STRUCTURE II

APPLICABLE STANDARD DRAWINGS

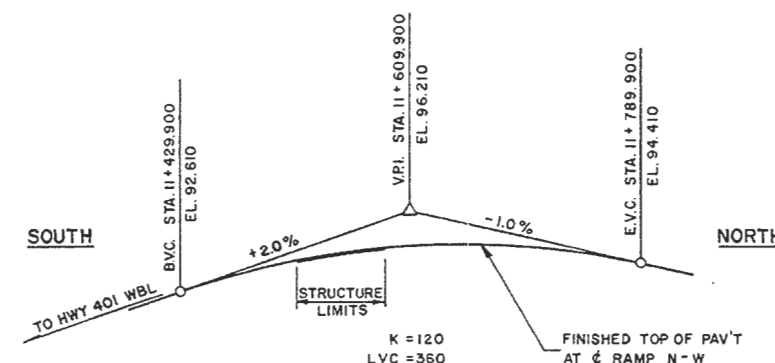
OPSD-3505.000	ROCK BACKFILL REQUIREMENTS - ABUTMENTS
OPSD-3906.02	BRIDGE DECK WATERPROOFING
OPSD-4010.00	GUIDE RAIL AND CHANNEL ANCHORAGE

REVISED										
	DATE	BY	DESCRIPTION							
	DESIGN G.S.S.	CHK J.W.H.	CODE OHBDC - 83	LOAD CLASS A	DATE JULY 1970					
	DRAWN W.M.K.	CHK G.S.S.	SITE 16-307	STRUCT	SCHEME	DWG.				

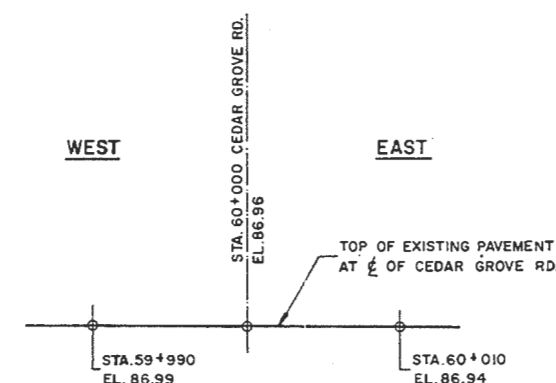


B.M.
EL. 86.347
N. & W. in N.E. CORNER OF
CONCRETE CULVERT
21.6m LT. OF 13+260 HWY 401

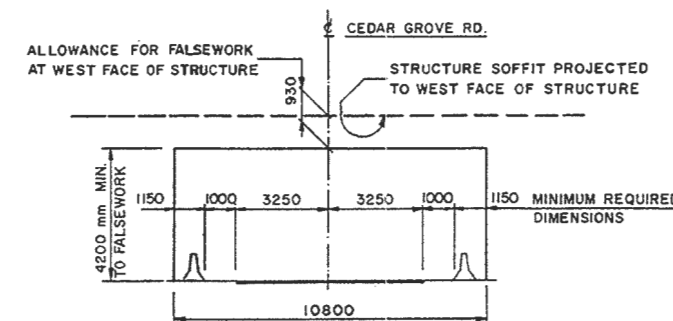
NOTES
W.P DENOTES WORKING POINT
T/P DENOTES TOP OF PAYEMENT
* SEE ROADWAY DRAWINGS FOR
DETAILS OF ROCK BACKFILL
AT ABUTMENTS.



PROFILE OF RAMP N - W
N.T.S.



PROFILE OF CEDAR GROVE ROAD
N.T.S.



FALSEWORK CLEARANCE DIAGRAM
NTS



DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

ENGINEERS
PLANNERS
ARCHITECTS

1. PILE SPACING IS MEASURED AT THE UNDERSIDE OF FOOTINGS.
2. PILE LENGTHS SHOWN ARE THE THEORETICAL LENGTH BELOW CUT-OFF.
3. PILES TO BE DRIVEN TO BEDROCK.

MAX. COMBINED FACTORED LOADS:

- SLS	11	1050kN
- ULS		1450kN

MAX. COMBINED FACTORED LOADS:

- SLS	11	1050kN
- ULS		1450kN



DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING






· OPSD-3301.00 SPLICE AND DRIVING SHOE DETAILS
FOR STEEL H-PILES

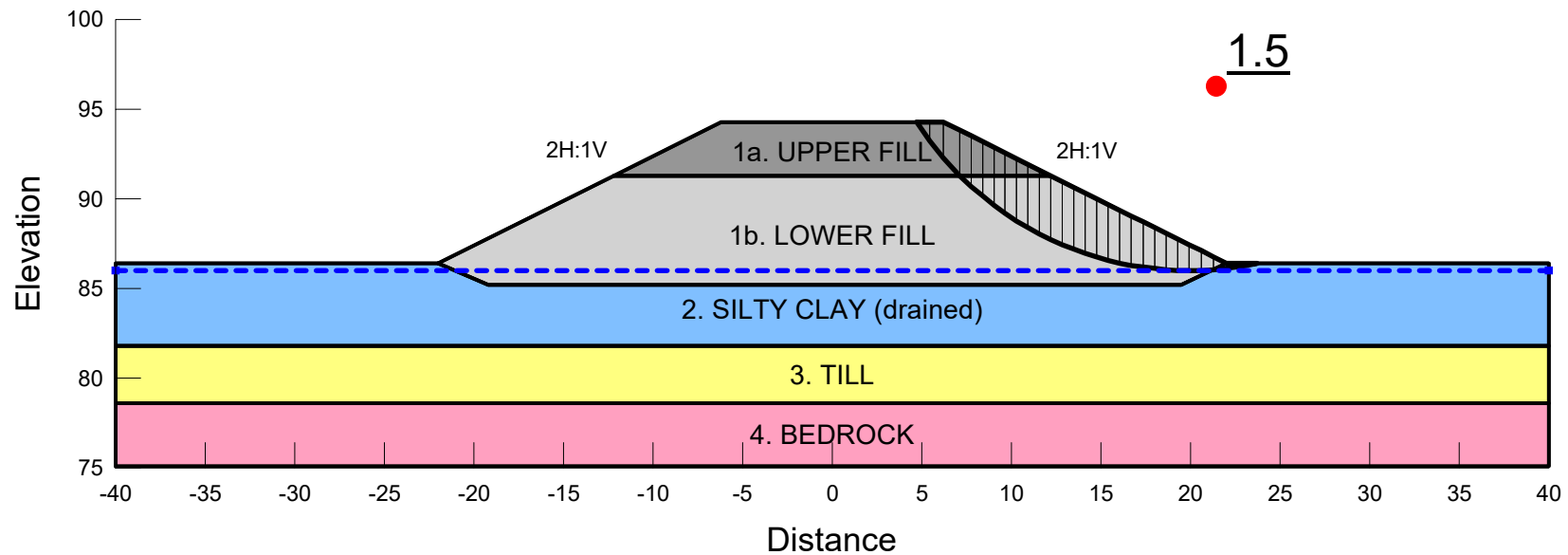
REVISIONS						
	DATE	BY	DESCRIPTION			
	DESIGN J.W.H.	CHK M.E.H.	CODE OHBDC-83	LOAD CLASS A	DATE JULY 1992	
	DRAWN G.G.	CHK J.W.H.	SITE 16-307	STRUCT	SCHEME	DWG. 3



Appendix G.

Slope Stability Analysis Figures






Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	1a. UPPER FILL	Mohr-Coulomb	21	0	35
	1b. LOWER FILL	Mohr-Coulomb	21	0	30
	2. SILTY CLAY (drained)	Mohr-Coulomb	18.8	7	28
	3. TILL	Mohr-Coulomb	22	0	35
	4. BEDROCK	Bedrock (Impenetrable)			

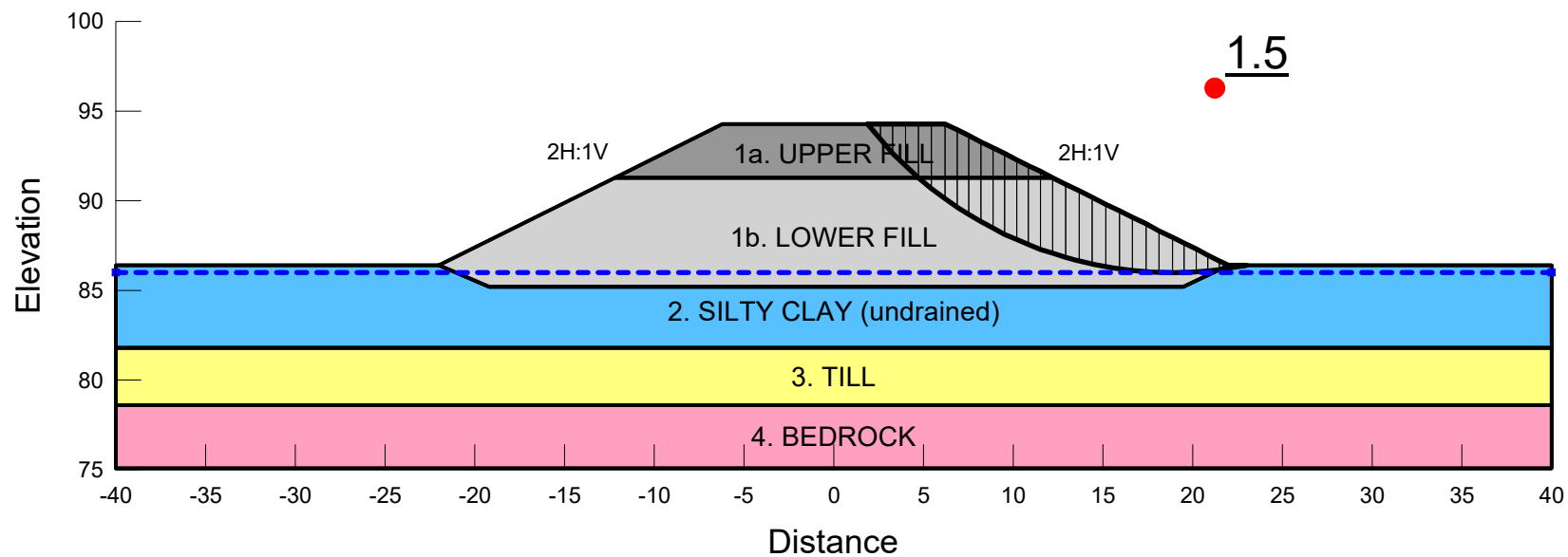


Project Hwy 416 Cedar Grove Road Overpass SBL (Site 16-307)		
Analysis Permanent (Drained)		
Seismic Coefficient H: 0g, V: g	Last Run 11/24/2021, 11:08:55 AM	Scale 1:400

Additional Details
Name: 16-307: 2H:1V Existing Embankment
Comments:
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.5 m
Entry: (4.7, 94.3) m, Exit: (23.705, 86.4) m
Center: (19.920271, 104.10522) m, Radius: 18.105219 m

Figure G1






Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	1a. UPPER FILL	Mohr-Coulomb	21	0	35
	1b. LOWER FILL	Mohr-Coulomb	21	0	30
	2. SILTY CLAY (undrained)	Mohr-Coulomb	18.8	130	0
	3. TILL	Mohr-Coulomb	22	0	35
	4. BEDROCK	Bedrock (Impenetrable)			

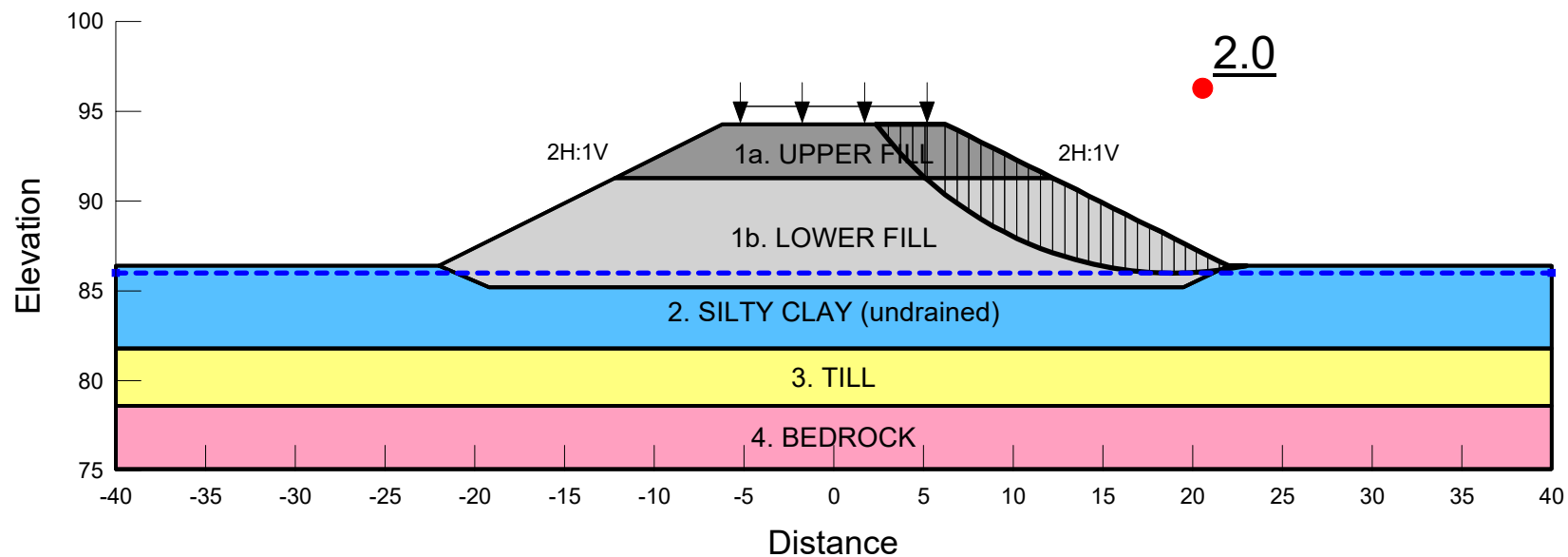


Project		
Hwy 416 Cedar Grove Road Overpass SBL (Site 16-307)		
Analysis		
Temporary - Pseudo-Static (2,475-year EQ)		
Seismic Coefficient	Last Run	Scale
H: 0.14g, V: g	11/24/2021, 11:08:59 AM	1:400

Additional Details
Name: 16-307: 2H:1V Existing Embankment
Comments:
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.5 m
Entry: (1.88, 94.3) m, Exit: (23, 86.4) m
Center: (18.868542, 107.53618) m, Radius: 21.536179 m

Figure G2

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	1a. UPPER FILL	Mohr-Coulomb	21	0	35
	1b. LOWER FILL	Mohr-Coulomb	21	0	30
	2. SILTY CLAY (undrained)	Mohr-Coulomb	18.8	130	0
	3. TILL	Mohr-Coulomb	22	0	35
	4. BEDROCK	Bedrock (Impenetrable)			



Project Hwy 416 Cedar Grove Road Overpass SBL (Site 16-307)		
Analysis Temporary - Traffic		
Seismic Coefficient H: 0g, V: 0g	Last Run 11/24/2021, 11:08:51 AM	Scale 1:400

Additional Details
Name: 16-307: 2H:1V Existing Embankment
Comments:
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.5 m
Entry: (2.35, 94.3) m, Exit: (23, 86.4) m
Center: (18.946135, 106.74227) m, Radius: 20.742272 m

Figure G3