



THURBER ENGINEERING LTD.

**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 401 WIDENING, HIGHWAY 16 TO MAITLAND ROAD
HIGHWAY 401W-416N RAMP REHABILITATION, SITE NO. 16X-0306
GWP 4024-20-00 / ASSIGNMENT NO. 4019-E-0010.2**

Geocres No.: 31B-109

Report to:

MTO c/o AECOM Canada Ltd.

Latitude: 44.747701°
Longitude: -75.488351°

March 2023
Thurber File No.: 29381



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PART 1. FACTUAL INFORMATION

1 INTRODUCTION

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

This report addresses the proposed rehabilitation of the Highway 401 and Cedar Grove Road underpass ramp bridge connecting traffic coming from the west on Highway 401 to travel north on Highway 416 (401W-416N). The bridge, Site No. 16X-0306, is located approximately 1.1 km west of Highway 401 Interchange 721 with Highway 16, 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 which included:

- Report prepared by Jacques, Whitford Limited titled, "*Report on Foundation Investigation, W.P. 374-89-02, Site 16-306, Ramp W-N Over Hwy 401 & Cedar Grove Road, Hwy. 401-416 Interchange, District 9, Ottawa*", dated March, 1992 (Geocres No. 31B-74).

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

Highway 401 is generally oriented northeast to southwest and the 401W-416N ramp bridge is oriented roughly north to south with a total length of 163 m and a width of 11.2 m. For project



purposes, Highway 401 and the bridge are herein described as oriented east-west and north-south, respectively.

The land adjacent to the site typically consists of forests, wet ground, and agricultural fields. The terrain is relatively flat, apart from the existing highway and interchange embankments and associated drainage ditches.

Highway 401 in this area consists of a four-lane divided freeway with paved shoulders and a median barrier and median stormwater system. A guiderail is present along the outsides of the highway. Cedar Grove Road is a two-lane, local roadway with narrow granular shoulders and a guiderail on the south side of the road. The ramp consists of one travelled lane with wide, paved shoulders. Steel beam guiderails are present along the approaches and abut the concrete barrier walls along the bridge.

Within the vicinity of the bridge, the embankments are partially retained by RSS walls near the abutments, below which the slopes are at approximately 2H:1V. The slopes are generally vegetated with bushes and small trees growing around the abutments. At the time of the field work, 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 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 bedrock but includes 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 April 1991. The current investigation was carried out in April/May 2021 to collect additional subsurface information near the existing bridge abutments. Summaries of the investigations are provided in the following sections.

3.1 Previous Investigation (1991)

A total of seventeen test holes were put down at the site as part of the 1991 investigation. Boreholes relevant to the current study include those numbered 91-2 to 91-7 which were put down at the then-proposed foundation element locations for the bridge between April 8 and 16, 1991.



The six relevant 1991 boreholes were advanced to depths ranging from 4.5 m to 12.6 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 each of the boreholes, with double installations in Boreholes 91-5 and 91-6.

The locations of the 1991 boreholes were surveyed by others prior to the initiation of the field work, unless they were subsequently relocated due to site constraints, in which case the as-drilled borehole location was subsequently surveyed.

The northings, eastings, and elevations of the boreholes used in this report 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. 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	Northing¹ (Latitude)	Easting¹ (Longitude)	Ground Surface² Elevation (m)	Termination Depth (m)
91-2	Near the North Abutment	4 957 043.0 (44.748900)	384 922.5 (-75.488127)	87.3	10.8
91-3	Near the North Pier	4 957 003.1 (44.748541)	384 920.6 (-75.488157)	87.4	12.6
91-4	Near the Centre Pier	4 956 959.1 (44.748146)	384 916.2 (-75.488220)	85.7	7.7
91-5	Near the South Pier	4 956 915.3 (44.747753)	384 909.1 (-75.488316)	86.1	8.1
91-6	Near the South Abutment	4 956 876.9 (44.747408)	384 900.6 (-75.488430)	85.2	7.0
91-7	South Approach Fill	4 956 847.7 (44.747146)	384 893.2 (-75.488528)	84.3	4.5

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 ramp and bridge.

3.2 Current Investigation (2021)

The current site investigation was carried out in the Spring of 2021. Two boreholes were put down at the site: one near the south abutment on April 19, 2021 (Borehole 306-21-1) and one near the north abutment between April 20, 2021 and May 4, 2021 (Borehole 306-21-2). The boreholes were put down with a truck-mounted CME 55 drill rig.

The locations of 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, and in Table 3-2 below. The site is located within MTM Zone 9.

Table 3-2: Borehole Summary

Borehole No.	Drilled Location	Northing (Latitude)	Easting (Longitude)	Ground Surface Elevation (m)	Termination Depth (m)
306-21-1	Near the South Abutment	4 956 859.3 (44.747256)	384 903.0 (-75.488405)	94.8	16.4
306-21-2	Near the North Abutment	4 957 047.6 (44.748948)	384 928.6 (-75.488052)	95.5	26.5

Soil samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). A standpipe piezometer was installed in Borehole 306-21-1 following completion of the drilling to allow for subsequent groundwater level measurements. It was decommissioned in December 2022. Borehole 306-21-2 was abandoned after drilling by backfilling with bentonite and drill cuttings.

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 306-21-2. Laboratory testing carried out as part of the 1991 investigation included natural moisture content, grain size distribution, Atterberg limit determination, and laboratory vane testing on selected soil samples.

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

5 GENERAL DESCRIPTION OF SUBSURFACE CONDITIONS

Details of the encountered soil stratigraphy are presented on the Record of Borehole sheets included in Appendix B and 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 investigation is in accordance with ASTM D2487. Description of cohesive soils and secondary components of all deposits from the 2021 borehole



are described as outlined in the MTO Guideline for Foundation Engineering Services manual (October 2020). Terminology from the historic information may vary from current practice.

In general, the site is underlain by a layer of clayey silt to clay overlying glacial till at relatively shallow depth. The till is, in turn, underlain by limestone bedrock, which slopes down to the north.

The sections below describe subsurface conditions encountered at the time the boreholes were advanced. The conditions reported in the historic information may have been disturbed or altered, partially or completely during the construction of the W-N Ramp structure.

5.1 Embankment Fill

The asphalt surface was observed to be 150 mm and 125 mm thick in Boreholes 306-21-1 and 306-21-2. Granular embankment fill consisting of silty sand to sand to sand and gravel was encountered at the boreholes put down behind the abutments. At Boreholes 306-21-1 (south abutment) and 306-21-2 (north abutment) the embankment fill is 10.5 m and 9.0 m thick, respectively (extending down to Elevations 84.1 m and 86.4 m)

SPTs conducted in the embankment fill gave N-values ranging from 17 to 53 blows per 0.3 m of penetration, indicating a compact to very dense relative density.

The moisture content of the fill samples tested ranged from about 2 to 9%, and one moisture content of 17% in Borehole 306-21-2. The results of grain size analysis testing conducted on four samples of the embankment fill are summarized below and are illustrated on Figure C1 in Appendix C.1.

Summary of Grain Size Distribution Testing – Granular Fill

Soil Particle	Percentage (%)
Gravel	1 – 48
Sand	42 – 94
Silt and Clay	5 – 10

5.2 Surficial Deposits

Surficial deposits consisting of topsoil, underlain by fill and sand were encountered in Boreholes 91-3, 91-5, 91-6, and 91-7 which were put down prior to construction of the bridge. Though the surficial deposits within the embankment footprints and at the abutments may have been removed prior to construction, they may still be present beyond the embankment footprints and in the vicinity of the bridge piers, and are described in the following sections for information purposes only.

5.2.1 Topsoil

Topsoil was encountered at the ground surface in Boreholes 91-2, and 91-4 to 91-7. Topsoil ranged in thickness from 100 to 300 mm.



5.2.2 Fill

A deposit of fill was encountered in Boreholes 91-3 and 91-5, which were put down close to Cedar Grove Road and Highway 401, respectively. A deposit of sand and gravel fill was encountered at ground surface in Borehole 91-3 and a deposit of silt and sand fill was encountered at ground surface in Borehole 91-5. The fill was approximately 1.4 m thick in Borehole 91-3 and 0.9 m thick in Borehole 91-5.

Two SPTs conducted in the fill gave N-values of 6 and 13 blows per 0.3 m of penetration, indicating a loose to compact relative density. The moisture contents of the two samples tested were 8 and 20%.

5.3 Sand

A deposit of sand, some silt was encountered beneath the embankment fill in Borehole 306-21-1 and beneath the topsoil in Boreholes 91-6 and 91-7. Organics were observed in the deposit in Borehole 306-21-1. The thickness ranged from about 0.5 m to 1.5 m with base elevations ranging from 82.6 to 84.5 m. SPTs conducted in the sand deposit gave N-values of 2 to 14 blows per 0.3 m of penetration, indicating a very loose to compact relative density.

The moisture content of one sample tested was 23%. The results of a grain size analysis test conducted on a sample of this material obtained in Borehole 306-21-1 is summarized below and is illustrated on Figure C2 in Appendix C.1.

Summary of Grain Size Distribution Testing – Sand

Soil Particle	Percentage (%)
Gravel	8
Sand	76
Silt	10
Clay	6

5.4 Clayey Silt to Clay

A deposit of clayey silt to clay was encountered beneath the sand layer in Boreholes 306-21-1, 91-6, and 91-7, beneath the embankment fill in Boreholes 306-21-2, 91-3, and 91-5, and beneath the topsoil in Borehole 91-2, 91-4. The deposit was encountered near the original ground surface and ranged from 0.9 m to 2.8 m thick (base Elevations 81.7 m to 85.8 m).

SPTs conducted within this layer gave N-values generally ranging from 2 to 13 blows per 0.3 m of penetration. Field and laboratory vane tests conducted during the 1991 investigation indicated that the clay has an undrained shear strength ranging from 75 kPa to over 200 kPa, indicating a stiff to hard consistency.

The moisture content of the samples tested ranged from about 21 to 42%.



The results of four grain size analysis tests conducted on this deposit are summarized below and are illustrated on Figure C3 in Appendix C.1 and Figure 2 in Appendix C.2.

Summary of Grain Size Distribution Testing – Clayey Silt to Silty Clay to Clay

Soil Particle	Percentage (%)
Gravel	0 – 1
Sand	0 – 4
Silt	33 – 70
Clay	29 – 67

The results of Atterberg Limits testing carried out on five samples of this deposit are summarized below and are illustrated on Figure C4 in Appendix C.1 and Figure 1 in Appendix C.2. The laboratory results indicate that the material is generally a clay of intermediate plasticity (CI) to high plasticity (CH), with the exception of one test carried out in the relatively thin deposit in Borehole 306-21-2 which plotted as a clay of low plasticity (CL, clayey silt).

Summary of Atterberg Limit Testing – Clayey Silt to Silty Clay to Clay

Parameter	Value
Liquid Limit	28 – 54
Plastic Limit	18 – 27
Plasticity Index	10 – 30

5.5 Glacial Till

A basal till deposit consisting of a heterogeneous mixture of silty sand with to some gravel was encountered beneath the clay deposit in Borehole 306-21-2 and 91-2 to 91-7. Cobbles and boulders were present in the glacial till in all boreholes where the deposit was encountered. The glacial till was encountered at Elevations ranging from 81.7 m to 85.8, with thicknesses ranging from 0.1 m to 12.2 m (base elevation ranging from 72.6 m to 81.7 m). The glacial till deposit generally becomes thicker to the north.

SPTs conducted in this layer gave N-values ranging from 3 blows to greater than 50 blows for 150 mm of penetration but were generally between 10 and 50 blows per 0.3 m of penetration, indicating a compact to very dense relative density. Refusals within this deposit are likely due to presence of cobbles and boulders. Penetration through this layer required the use of coring techniques in a few locations.

Summary of Grain Size Distribution Testing – Glacial Till

Soil Particle	Percentage (%)	
Gravel	10 – 31	
Sand	26 – 39	
Silt	43 – 52	27 – 36
Clay		11 – 22

The moisture content of samples obtained from this unit ranged from 7 to 14%. The results of grain size distribution testing carried out on six samples of the till from the 1991 and 2021 investigations are summarized in the table above. The results from the 2021 investigation are illustrated on Figure C5 in Appendix C.1 and an envelope summarizing the results from the 1991 investigation is illustrated on Figure 3 in Appendix C.2.

The results of Atterberg Limits testing carried out on the fines of three samples of the glacial till from the 1991 boreholes are summarized below and are illustrated on Figure 4 in Appendix C.2. The laboratory results indicate that the fines are non-plastic to slightly plastic (ML to CL-ML).

Summary of Atterberg Limit Testing – Glacial Till Fines

Parameter	Value
Liquid Limit	14 – 15
Plastic Limit	10 – 11
Plasticity Index	3 – 5

5.6 Bedrock

Bedrock was proven by coring in Boreholes 306-21-1, 306-21-2, and 91-3 to 91-7. The bedrock encountered consisted of fresh, very strong, fine grained, grey, interbedded dolostone and limestone. Photographs of the bedrock cores are provided in Appendix C. The following table summarizes the rock core quality:

Table 5-1: Summary of Rock Core Quality

Parameter	Range
Total Core Recovery (TCR), %	93 to 100
Solid Core Recovery (SCR), %	59 to 100
Rock Quality Designation (RQD), %	41 to 100
Fracture Index	0 to >10

The RQD values encountered in the upper run of bedrock core in Boreholes 306-21-1, 306-21-2 and 91-3 were between 41% and 58%. All other RQD values ranged from about 75% to 100%, indicating a bedrock of good to excellent quality.

Unconfined compressive strength (UCS) testing was carried out on a sample of the bedrock from Borehole 306-21-2. The results indicated a UCS value of 220 MPa, indicating a very strong rock. The results of the UCS testing are included in Appendix C.

A summary of the bedrock surface information is provided in Table 5 2, below.

Table 5-2: Summary of Bedrock Depth/Elevation

Borehole No.	Depth to Bedrock Surface (mbgs)¹	Bedrock Surface Elevation (m)
91-2	Refusal at 10.8	Refusal at 76.5
91-3	9.6	77.8
91-4	4.6	81.1
91-5	5.2	80.9
91-6	4.0	81.2
91-7	2.6	81.7
306-21-1	13.1	81.7
306-21-2	22.9	72.6

Note: Depths below ground surface at the time of drilling; ground surface may have changed since.

5.7 Groundwater

Standpipe piezometers or monitoring wells were installed in all of the relevant boreholes put down as part of the 1991 investigation, and in Borehole 306-21-1 put down as part of the current investigation. Groundwater levels recorded are presented in Table 5-3.

Table 5-3: Summary of Groundwater Levels

Borehole No.	Bottom of Screen Elev. (m)	Screened Unit	Depth (mbgs)¹	Groundwater Elevation (m)	Date of Measurement
91-2	76.5	Glacial Till	0.6	86.7	May 10, 1991
91-3	74.8	Bedrock	0.8	86.6	May 10, 1991
91-4	81.3	Glacial Till	0.1	85.6	May 10, 1991
91-5	78.0	Bedrock	0.7	85.4	May 10, 1991
91-6	82.2	Clay	0.3	84.9	May 10, 1991
91-6	78.2	Bedrock	-0.2	85.4	May 10, 1991
91-7	79.9	Bedrock	0.0	84.3	May 10, 1991
306-21-1	82.6	Sand	9.6	85.2	July 1, 2021
			9.4	85.4	December 19, 2022

Note: Depths below ground surface at the time of reading; ground surface may have changed since.

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 level may be at a higher elevation after periods of significant and/or prolonged precipitation.



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 locations were selected by Thurber relative to existing site features. The as-drilled locations and ground surface elevations of the boreholes were surveyed by Thurber following completion of the field program. The elevation survey of the boreholes was carried out with reference to geodetic elevation benchmarks provided by the MTO. Eastern Ontario Diamond Drilling of Hawkesbury, Ontario supplied and operated the drilling equipment and carried out the drilling, soil sampling, in-situ testing, and borehole decommissioning.

The field investigation was supervised on a full-time basis by Jamil Pirani 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.

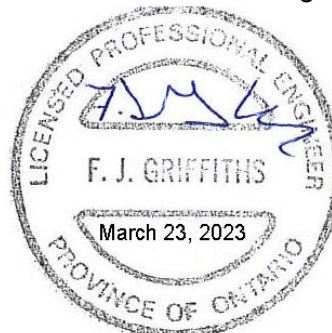
Overall project management and direction of the field investigation was provided by Matt Kennedy, P.Eng. Interpretation of the factual data and preparation of this report was carried out by Sarah Harrold, EIT and Matt Kennedy, P.Eng. The report was reviewed by Paul Carnaffan, P.Eng. and Fred Griffiths, 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 preliminary foundation investigation and a desktop review of the available subsurface information conducted by Thurber for the rehabilitation of the existing Highway 401 and Cedar Grove Road Underpass Ramp Bridge connecting traffic coming from the west on Highway 401 to traffic traveling north on Highway 416 (401W-416N).

The site is located approximately 1.1 km west of Highway 401 Interchange 721 with Highway 16. The bridge carries traffic coming from the west on Highway 401 to travel north on Highway 416. Highway 401 is oriented northeast to southwest and the bridge is oriented roughly north to south. For project purposes, Highway 401 and the 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 preliminary 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

The existing bridge is a four-span structure with a cast-in-place, post-tensioned concrete deck and non-integral abutments with a total length of 163 m and a width of 11.2 m. It is curved to the west and crosses the lanes of the eastbound and westbound Highway 401 and Cedar Grove Road at skew angles of about 34, 36, and 53 degrees to parallel, respectively.



Based on the available structural drawings (Cont. No. 97-68, Sheets 122 and 124), the south abutment is supported on two rows of steel HP 310x132 piles about 10 m long, the north abutment is supported on two rows of steel HP 310x110 piles about 16 m long, and the north pier is supported on steel HP 310x110 piles about 7.5 m long. All piles were to be driven to bedrock.

The south abutment has one row of five and one row of six piles and the north abutment has two rows of five piles. Both abutment pile caps are perched within the existing embankments and are nominally sloped down to the west with top of each pile cap ranging from about Elevation 91.7 m to 93.1 m.

The south and centre piers are supported on concrete spread footings on bedrock. The south pier is 5.0 m square, 1.8 m thick, and founded at approximate Elevation 80.9 m. The centre pier is 7.0 m by 10.5 m, 2.2 m thick, and founded at about Elevation 81.1 m.

7.2 Proposed Work

It is understood that the details of the works are yet to be decided at this time. Preliminary foundation recommendations are required concerning seismic design, foundation bearing resistance, and lateral earth pressure to allow structural assessment of the bridge and the need for rehabilitation treatments. Additional investigation and analysis will likely be required in any subsequent detail design phase of the project.

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.

It is assumed that the fundamental period of vibration for the structure is greater than or equal to 0.5 seconds.

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

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



8 SEISMIC CONSIDERATIONS

8.1 Spectral and Peak Acceleration Hazard Values

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

The site coefficients used to determine the design spectral acceleration and displacement values are a function of the Site Class, the peak ground acceleration (PGA) and $S_a(0.2)$. The PGA for this location for a *reference* Site Class C with a 2% probability of exceedance in 50 years is 0.25 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 91-4, and 91-5 were put down near the spread footings supported on bedrock at the south and centre piers. At these locations, the bedrock consists of good to excellent quality interbedded dolostone and sandstone bedrock. Though the shear wave velocity of the bedrock is anticipated to be greater than 760 m/s which would fall within Site Class B, in the absence of measured velocities at the site a Site Class C should be assumed at these foundation elements.

The south abutment, north pier, and north abutment are supported on piles driven to bedrock. Based on the range of N_{60} values recorded in the embankment fill (17 to 53 blows per 0.3 m of penetration), the interpreted undrained shear strength of the stiff to hard clayey silt to clay, and the N_{60} values recorded in the glacial till (16 to 43 blows per 0.3 m of penetration), the Site Class at these foundation elements may be taken as Site Class D.

The site classification should be confirmed with measurement of the shear wave velocity in the 30 m below the foundation elements at subsequent design stages.

8.3 Seismic Performance Category

In consideration of the Site Class D spectral values for the site and the designated *Major Route* importance category, the bridge structure would fall into Seismic Performance Category 2,

assuming the bridge has a fundamental period greater than or equal to 0.5 seconds, as per Section 4.4.4 (Table 4.10) of the CHBDC.

8.4 Liquefaction Potential

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

The susceptibility of the glacial till at the site to experience liquefaction was also assessed using the SPT data following the simplified method for cohesionless soil as outlined in Boulanger and Idriss (2014)ⁱ. The cohesionless soils anticipated to be below the piled foundations at the site are not considered to be susceptible to liquefaction.

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

9 FOUNDATION DESIGN RECOMMENDATIONS

9.1 Existing Spread Footings

The available structural drawings (Appendix F) indicate that the existing south and centre pier foundations consist of spread footings founded on bedrock. The footing dimensions and founding elevations are summarized in Table 9-1, below.

Table 9-1: Summary of Pier Foundation Footings

	South Pier (Pier #1)	Centre Pier (Pier #2)
Plan Dimensions (m)	5.0 x 5.0	10.5 x 7.0
Footing Thickness (m)	1.8	2.2
Top of Footing Elevation (m)	82.7	83.3
Underside of Footing Elevation (m)	80.9	81.1

Based on the pier footing dimensions, founding elevations, and the assumption that the footings were cast-in-place on the bedrock surface, a Factored Geotechnical Resistance at ULS of 1,800 kPa may be considered. The Factored Geotechnical Resistance at SLS for 25 mm of settlement would exceed the resistance at ULS.

The factored geotechnical resistances include the following factors:

- Consequence factor (Ψ) of 1.0 (as per CHBDC Table 6.1)
- Geotechnical resistance factors (as per CHBDC Table 6.2):
 - $\phi_{gu} = 0.50$ (static analysis; *typical* degree of understanding)
 - $\phi_{gs} = 0.80$ (static analysis; *typical* degree of understanding)

The geotechnical resistances presented are for vertical concentric loading only on cast-in-place footings and would need to be adjusted for the effects of inclined or eccentric loadings, where applicable, in accordance with CHBDC Clause 6.10.5.

The horizontal resistance against sliding between the cast-in-place concrete footings founded on the bedrock can be computed using a unfactored coefficient of friction factor of 0.5. Appropriate resistance factors should be applied for the design.

9.2 Existing Driven Steel Piles

The available structural drawings (Appendix F) indicate that the abutments and north pier are supported on vertical steel piles driven to bedrock. The north pier and north abutment are founded on HP310x110 piles, and the south abutment is founded on HP 310x132 piles.

The 1997 design drawings also indicate that a portion of the abutment piles directly below the underside of pile caps were cased in a concrete-filled, 500 mm diameter Corrugated Steel Pipe (CSP). The abutment foundation dimensions are summarized in Table 9-2, below.

Table 9-2: Summary of Piled Foundations

	South Abutment	North Pier (Pier #3)	North Abutment
Overall Plan Dimensions (m)	11.1 x 3.1	5.4 x 5.4	11.1 x 2.9
Pile Cap Thickness (m)	Up to 1.45	1.8	Up to 1.45
Top of Pile Cap Elevation (m)	91.7 to 92.4	86.5	92.4 to 93.1
Underside of Pile Cap Elevation (m)	90.3 to 91.0	84.7	91.0 to 91.7

9.2.1 Axial Geotechnical Resistance

The boreholes put down behind the abutments as part of the current investigation encountered compact to very dense granular fill to elevations of 84.1 m (south abutment) and 86.4 m (north abutment). Beneath the embankment fill at the south abutment, and nominal surficial fill at the north pier, deposits of silty clay underlain by glacial till were encountered above the bedrock. The embankment fill at the north abutment is underlain by glacial till and a localized clayey silt deposit over the bedrock.

Based on the subsurface conditions encountered and as-built details provided on the structural drawings, the existing steel HP 310x110 or HP 310x132 piles that are driven to bedrock may be considered to have a factored ULS geotechnical resistance greater than 3,000 kN. The factored geotechnical resistance at SLS will not govern for steel H-piles driven to 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 in cohesionless soils is typically taken as 1.0.

9.2.2 Downdrag

The results of the current investigation indicate a layer of clay to clayey silt overlying the glacial till at the north pier and north abutment, and underlying the embankment fill and sand deposit at the south abutment. Downdrag forces (negative skin friction) acting upon the piles supporting the abutments and north pier should be considered during preliminary design as a result of settlement of the clayey silt to clay deposit under the imposed loading from the fill.

For preliminary design purposes, the unfactored downdrag load acting on a single HP 310x132 pile at the south abutment and single HP 310x110 pile at the north abutment is estimated to be about 800 kN and 400 kN, respectively. The neutral plane will be at the base of the clay layer or approximate elevation 81.7 m and 84.8 m for the south and north abutments respectively. These values would be lesser if the embankments were pre-constructed with a preload period prior to the driving of the abutment piles. No significant grade raise was constructed at the piers, therefore downdrag loads are not expected at the pile-supported north pier.

Additional field and laboratory testing may be needed during detailed design to confirm the magnitude of downdrag loads.

9.2.3 Uplift Resistance

The native soils and embankment fill at the piled foundations will provide uplift resistance to the piles. Shaft friction of the embankment fill (where appropriate), sand, clay to clayey silt, and glacial till were calculated, assuming the piles met effective refusal to driving at the bedrock surface.

The factored geotechnical tensile resistance for a single pile at each of the piled foundations may be taken as shown in Table 9-3.

Table 9-3 Uplift Resistance

Condition	Uplift Resistance (kN)		
	South Abutment HP 310x132	North Pier (Pier #3) HP 310x110	North Abutment HP 310x110
Static	250	210	1,000
Seismic	800	700	3,000

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)

9.2.4 Lateral Geotechnical Resistance

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.

9.3 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 at Boreholes 306-21-1 and 306-21-2 is generally a compact to very dense, heterogeneous mix of sand, gravel, and silt of varying amounts. However, the boreholes were put down at a sufficient distance from the expansion joints to avoid penetration of the approach slabs or the RSS wall reinforcement. The 1997 design drawings show two options for RSS wall design. Both options suggest that the retained earth be a Select Backfill material with unit weight of 22 kN/m³ and friction angle of 35 degrees (equivalent to an OPSS Granular A backfill), and the remaining embankment fill to consist of granular material with a friction angle of 30 degrees.

Based on the compact to very dense relative density of the embankment fill encountered in boreholes 306-21-1 and 306-21-2, the embankment fill may be considered to have a friction angle of about 33 degrees. 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 A to embankment fill. It has been assumed that a filter fabric wrapped subdrain was constructed at the base of the walls, as noted on the 1997 design drawings.

Lateral earth pressure parameters provided in Table 9-4 and Table 9-5 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 9-4 and Table 9-5 should be used. If other backfill and wall geometries are to be considered, Thurber will need to calculate the appropriate earth pressure coefficients.

9.3.1 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 9-4.

Table 9-4 Static Earth Pressure Coefficients

Condition	OPSS Granular A $\phi = 35^\circ, \gamma = 22 \text{ kN/m}^3$	Embankment Fill $\phi = 33^\circ, \gamma = 22 \text{ kN/m}^3$
Active, K_A (Yielding Wall)	0.27	0.29
At Rest, K_o (Non-Yielding Wall)	0.43	0.46
Passive, K_P (Movement towards Soil Mass) in front of wall	3.7	3.4

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.

9.3.2 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 9-5 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 1.08 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 9-5 Combined Static and Seismic Earth Pressure Coefficients

Condition	OPSS Granular A $\phi = 35^\circ, \gamma = 22 \text{ kN/m}^3$	Embankment Fill $\phi = 33^\circ, \gamma = 22 \text{ kN/m}^3$
Active, K_{AE} Yielding Wall	0.35	0.38
Active, K_{AE} Non-Yielding Wall	0.45	0.49

9.4 Embankment Stability

Based on the available original structure drawings and observations during the 2021 field investigation, the grade of the travelled lanes ranges from about 94.7 m to 95.5 m with embankments generally on the order of up to about 8 m, with the exception of the east side of the south embankment that ranges up to about 10 m high near the abutment. Embankment retaining walls parallel to the roadway are present behind the abutments with lengths ranging from about 8 m to 26 m along the embankments. Beyond the retaining walls, the embankments are sloped between about 2H:1V and 2.5H:1V, extending for lengths of up to about 23 m horizontally near the abutments.



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

For seismic analysis, Table 6.3 in Section 6.14.4.1 of the CHBDC indicates a minimum resistance factor of 0.95 ($\phi_{gu, static(temporary)} = 0.75 + 0.2$) for force-based design and 1.0 for performance-based design. Based on these values and Ψ of 1.0, a target Factor of Safety of 1.1 for this temporary condition with a typical degree of understanding is appropriate for the pseudo-static seismic analysis. However, as is stated in Section 6.14.9.1, 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 up to 10 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.

Slope stability assessments have been carried out for the highest/critical embankment slope, considered to be the east slope of the southern embankment, 8 m behind the abutment (immediately beyond the retaining wall). Embankment slope stability was evaluated using GeoStudio 2020 Slope/W software for limit equilibrium analysis. Input parameters for the analyses are based on the SPT N-values encountered in the 2021 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 10 m;
- For analysis of seismic events with return periods of 2,475 years and 475 years, site adjusted PGA values of 0.14 g and 0.06 g, respectively, equal to $\frac{1}{2}$ of the site adjusted PGA values (0.25 g and 0.12 g, respectively) were used, as per Section 4.4.3.3, of the CHBDC and outlined in Sections 8.1 and 8.2 above; and,

- A traffic surcharge of 17 kPa applied as a temporary load.

The results of the stability analyses are provided on the figures presented in 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 9-6 Slope Stability Analysis Results

Condition	Case	Factor of Safety
Permanent	Long-Term Static (Drained)	1.6 (Fig G1)
Temporary (traffic loading)	Short-Term Static (Undrained)	1.6 (Fig G2)
Temporary (seismic)	Pseudo-Static Seismic, 2,475-yr (Undrained)	1.1 (Fig G3)
Temporary (seismic)	Pseudo-Static Seismic, 475-yr (Undrained)	1.3 (Fig G4)

All of the static results presented in Table 9-6 achieve the target Factors of Safety described above. All of the pseudo-static results presented in Table 9-6 above meet or exceed 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 2475 year seismic event (Figure G3). As described above, an additional pseudo-static analysis was completed considering the 1 in 475 year seismic event (Figure G4) and the results achieved the target Factor of Safety to satisfy the performance criteria for Major Route embankments at this site.

10 RECOMMENDED SCOPE FOR DETAIL DESIGN

The recommendations provided above are in support of the preliminary design of the proposed rehabilitation of the Highway 401W-416N Ramp Overpass (Site No. 16X-0306) 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



- Additional field and laboratory testing to acquire compressibility characteristics of the cohesive soils near the abutments to allow estimation of settlement and downdrag loads on abutment piles. This additional work should be carried out if:
 - the approach fills are to be widened or the grades raised
 - the structural assessment based on the preliminary recommendations presented herein indicates concerns with existing pile capacity

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



11 CLOSURE

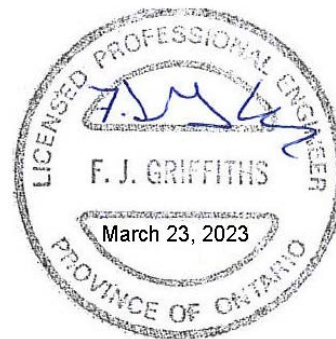
Engineering analysis and preparation of this report was carried out by Mr. Matt Kennedy, P.Eng. The report was reviewed by Paul Carnaffan, P.Eng. and Fred Griffiths, P.Eng., a Designated Principal Contact for MTO Foundation Projects.

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ⁱ 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

ⁱⁱ 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

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- 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.
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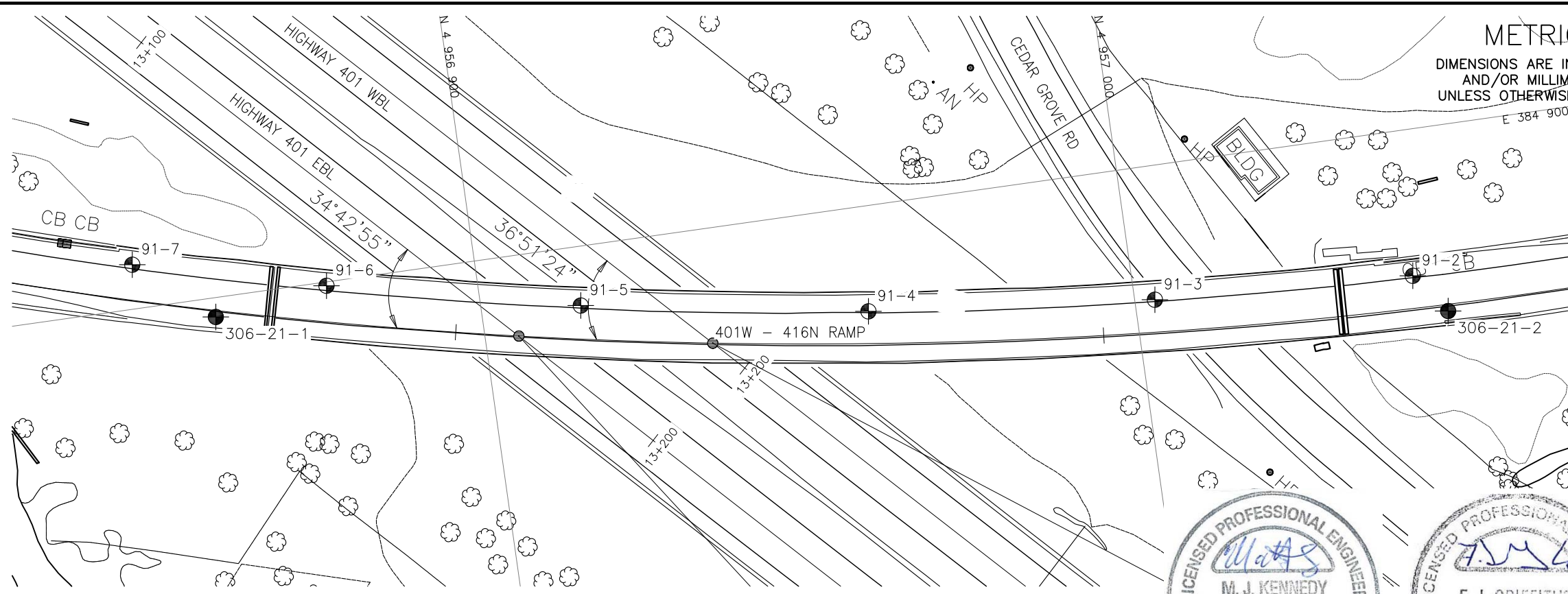
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Appendix A.

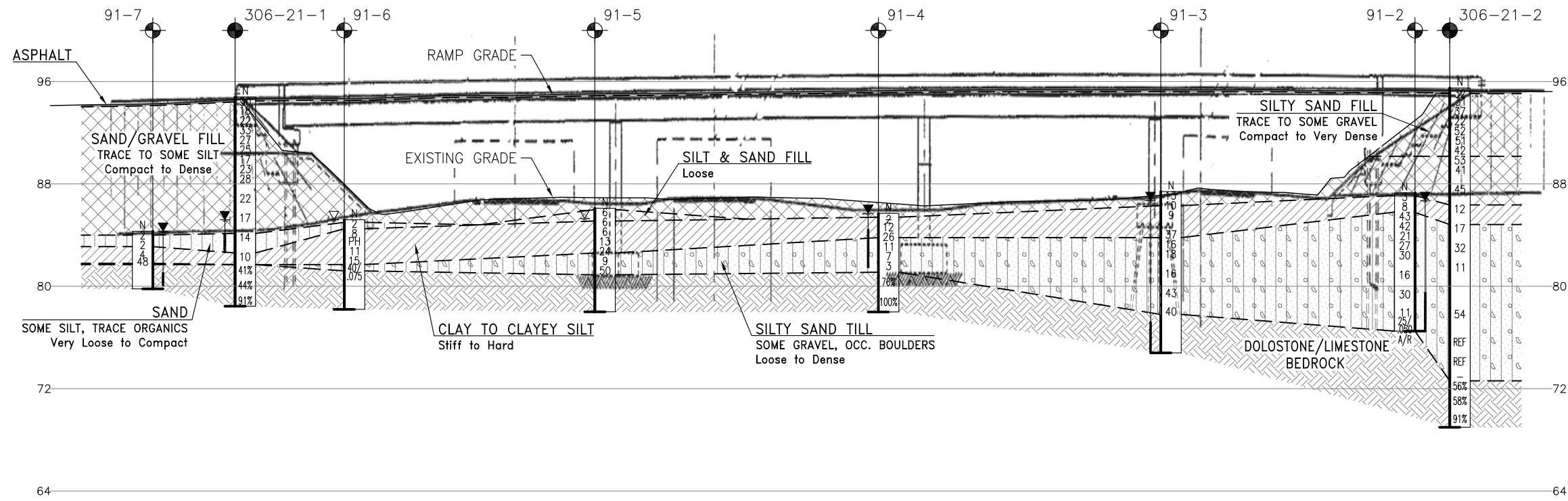
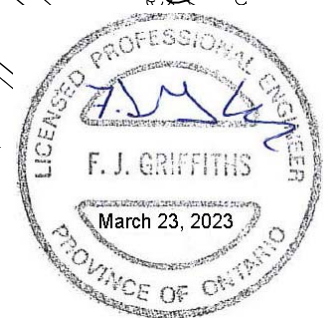
Borehole Location Plan and Stratigraphic Drawings



PLAN

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AND/OR MILLIMETRES
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PROFILE ALONG C 401W - 416N RAMP

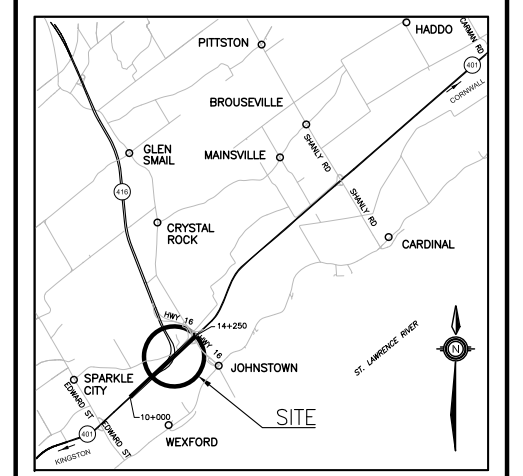
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CONT No
GWP No 4024-20-00

HIGHWAY 401
401W - 416N RAMP
BRIDGE REHABILITATION
BOREHOLE LOCATIONS AND SOIL STRATA

Ontario

SHEET



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
306-21-1	94.8	4 956 859.3	384 903.0
306-21-2	95.5	4 957 047.6	384 928.6
91-2	87.3	4 957 043.0	384 922.5
91-3	87.4	4 957 003.1	384 920.6
91-4	85.7	4 956 959.1	384 916.2
91-5	86.1	4 956 915.3	384 909.1
91-6	85.2	4 956 876.9	384 900.6
91-7	84.3	4 956 847.7	384 893.2

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-109

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	MJK	CHK -	CODE
DRAWN	MFA	CHK MK	SITE 16-306
LOAD			STRUCT
DATE	MAR 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 306-21-1

1 OF 2

METRIC

GWP# 4024-20-00 LOCATION Lat: 44.747256°, Long: -75.488405°
N 4 956 859.3 E 384 903.0 ORIGINATED BY JP
HWY 401 BOREHOLE TYPE CME 55 Truckmount, HSA/NQ Coring COMPILED BY SH
DATUM Geodetic DATE 2021.04.19 - 2021.04.19 CHECKED BY MJK

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
94.8								20 40 60 80 100						
0.0	ASPHALT (150 mm)							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
0.2	GRAVELLY SAND, trace fines Grey-brown to brown Compact to dense FILL		1	SS	30		94	20 40 60 80 100						28 65 7 (SI+CL)
			2	SS	18									
			3	SS	22		93							
			4	SS	33		92							
91.8														
3.0	SAND, trace to some gravel Trace fines Brown Compact FILL		5	SS	27		91							17 75 8 (SI+CL)
			6	SS	25		90							
			7	SS	17									
89.5														
5.3	SAND and GRAVEL Trace to some silt Grey-brown to brown Compact FILL		8	SS	23		89							48 42 10 (SI+CL)
			9	SS	28		88							
			10	SS	22		87							
							86							
			11	SS	17									
							85							

Continued Next Page

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

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

RECORD OF BOREHOLE No 306-21-1

2 OF 2

METRIC

GWP# 4024-20-00 LOCATION Lat: 44.747256°, Long: -75.488405° N 4 956 859.3 E 384 903.0 ORIGINATED BY JP
 HWY 401 BOREHOLE TYPE CME 55 Truckmount, HSA/NQ Coring COMPILED BY SH
 DATUM Geodetic DATE 2021.04.19 - 2021.04.19 CHECKED BY MJK

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
	Continued From Previous Page							20	40	60	80	100					GR SA SI CL
84.1	SAND and GRAVEL Trace to some silt Grey-brown to brown Compact FILL																
10.7	SAND some silt Trace organics Grey Compact		12	SS	14		84							o			8 76 10 6
82.6							83										
12.2	CLAYEY SILT Grey Compact/very stiff		13	SS	10		82							⊖-			0 1 70 29
81.7																FI	
13.1	Interbedded DOLOSTONE and LIMESTONE Fresh Grey Smooth Fine grained Very strong		1	RUN			81									4	RUN #1 TCR=97% SCR=59% RQD=41%
																4	
			2	RUN			80									3	RUN #2 TCR=100% SCR=92% RQD=44%
																5	
																0	
																5	
																4	
																3	
			3	RUN			79									2	RUN #3 TCR=97% SCR=91% RQD=91%
																1	
78.4																2	
16.4	End of Borehole Flushmount 19 mm diameter PVC monitoring well installed. Well Readings: Date: Depth (m): Elev. (m): 2021/07/01 9.6 85.2 2022/12/19 9.4 85.4																

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

RECORD OF BOREHOLE No 306-21-2

1 OF 3

METRIC

GWP# 4024-20-00 LOCATION Lat: 44.748948°, Long: -75.488052° N 4 957 047.6 E 384 928.6 ORIGINATED BY JP
 HWY 401 BOREHOLE TYPE CME 55 Truckmount, HSA/NQ Coring COMPILED BY SH
 DATUM Geodetic DATE 2021.04.20 - 2021.05.05 CHECKED BY MJK

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
95.5														
0.0	ASPHALT (125 mm)													
0.1	SILTY SAND, trace to some gravel Grey-brown to brown Compact to very dense FILL		1	SS	32		95							
			2	SS	31		94							
			3	SS	37		93							
			4	SS	22		92							
			5	SS	52		91							
			6	SS	51		90							
			7	SS	42		89							
90.2	SAND, trace to some silt Trace organics Brown Very dense to dense		8	SS	53		88							
5.3			9	SS	41		87							
			10	SS	45		86							
86.4	SILTY CLAY grey-brown Very stiff WEATHERED CRUST		11	SS	12									
9.1														

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
 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 306-21-2

2 OF 3

METRIC

GWP# 4024-20-00 LOCATION Lat: 44.748948°, Long: -75.488052° N 4 957 047.6 E 384 928.6 ORIGINATED BY JP
 HWY 401 BOREHOLE TYPE CME 55 Truckmount, HSA/NQ Coring COMPILED BY SH
 DATUM Geodetic DATE 2021.04.20 - 2021.05.05 CHECKED BY MJK

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
	Continued From Previous Page							SHEAR STRENGTH kPa						
								○ UNCONFINED + FIELD VANE						
								● QUICK TRIAXIAL × LAB VANE						
								WATER CONTENT (%)						
								20	40	60	80	100		
84.8	SILTY CLAY Grey-brown Very stiff WEATHERED CRUST						85							
10.7	SILTY SAND some gravel Grey Compact to dense Frequent cobbles/boulders GLACIAL TILL		12	SS	17		84							22 33 34 11
			13	SS	32		83							
81.8							82							
13.7	SILTY SAND some gravel Grey Compact GLACIAL TILL		14	SS	11									15 38 36 11
81.2							81							
14.3	SILTY SAND some gravel Grey Very dense Frequent cobbles/boulders GLACIAL TILL						80							
							79							
			15	SS	54		78							
							77							
							76							
			16	SS	REF									

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

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

METRIC

Lat: 44.748948°, Long: -75.488052°
N 4 957 047.6 E 384 928.6

+³, ×³: Numbers refer to Sensitivity

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



Appendix B.2

Previous (1991) Investigation

METRIC

W P 374-89-02 LOCATION Co-ords: N: 4 957 043.0 E: 384 922.5 ORIGINATED BY Y.L.
DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Auger COMPILED BY C.K.K.
DATUM Geodetic DATE April 9, 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	LQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH kPa						WATER CONTENT (%)		
								20 40 60 80 100								
87.3	Ground Surface							○ UNCONFINED + FIELD VANE								
87.1	Topsoil							● QUICK TRIAXIAL x LAB VANE								
0.2	Clay Stiff to Very Stiff Brown/Grey		1	SS	3		87									
							May 10, 1991									
			2	SS	8			x								
85.8							86									
1.5	Het. Mixture of Silty Sand, some clay & gravel, occ. boulders (Glacial Till) Compact to Dense Brown Grey		3	SS	43		85									
			4	SS	42		Native Backfill									
			5	SS	21		84						15 39 (46)			
			6	SS	27		83									
			7	SS	30		82									
			8	SS	16		81									
			9	SS	30		80									
							79									
							Seal									
							Sand Backfill									
			10	SS	11		78									
							Piezometer									
							77									
76.5							Seal									
10.8	End of Borehole Refusal on probable bedrock		11	SS	25/50mm											

OFFICE REPORT ON SOIL EXPLORATION

+3, x⁵: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 91-3

METRIC

W P 374-89-02 LOCATION Co-ords: N: 4 957 003.1 E: 384 920.6 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 April 8, 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 (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100					
87.4	Ground Surface															GR SA SI CL
0.0	Sand and Gravel, some Silt (Fill) Compact Brown to Black		1	SS	13	Seal										
86.0						May 10, 1991										
86.4	Clay Stiff to Hard Brown/Grey		2	SS	10											0 0 33 67
			3	SS	9											
83.8			4	SS	37											
3.6	Het. Mixture of Silty Sand, some clay & gravel, occ. boulders (Glacial Till) Compact to Dense Grey		5	SS	16											
			6	SS	18											
			7	SS	16											15 37 27 21
			8	SS	43											
77.8			9	Ss	40											
9.6	Bedrock Limy Dolostone Fair to Excellent		10	NX RC	REC 80%	Seal										RQD = 53%
			11	NX RC	REC 100%	Sand Backfill										RQD = 95%
74.8						Piezometer										
12.6	End of borehole					Seal										

OFFICE REPORT ON SOIL EXPLORATION

+3, x5: Numbers refer to Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10



RECORD OF BOREHOLE No 91-4

METRIC

W P 374-89-02 LOCATION Co-ords: N: 4 956 959.1 E: 384 916.2 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 April 12, 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 (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100					
85.7	Ground Surface															GR SA SI CL
0.3	Topsoil					May 10, 1991										
	Clay Stiff to Hard Brown/Grey		1	SS	2	Seal		x					o			
			2	SS	12					*			o			
83.8						84										
1.9	Het. Mixture of Silty Sand, Brown some clay & Grey gravel, occ. boulders (Glacial Till) Loose to Compact		3	SS	26	Sand Backfill										
			4	SS	11	Piezometer							o			
			5	SS	7								o			10 38 30 22
			6	SS	3								H			
81.1						Seal										
4.6	Bedrock Dolostone Good to Excellent		7	NX RC	REC 98%	81										RQD = 76%
			8	NX RC	REC 100%	80 Native Backfill										RQD = 100%
78.0						79										
7.7	End of Borehole															

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 91-5

METRIC

W P 374-89-02 LOCATION Co-ords: N: 4 956 915.3 E: 384 909.1 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 April 15, 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 (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
86.1	Ground Surface																GR SA SI CL
0.1	Topsoil						Native Backfill Seal										
85.1	Silt and Sand (Fill) Loose	Brown	1	SS	6		May 10, 1991										
1.0	Clay Stiff to Hard		2	SS	6		85		x								
			3	SS	6		Sand Backfill		x								
		Brown/Grey Brown	4	SS	13		84			x							
							Seal				x						
82.6			5	SS	24		83						x				
3.5	Het. Mixture of Silty Sand, some clay & gravel, occ. boulders (Glacial Till) Loose to Dense Grey		6	SS	9		Native Backfill										
			7	SS	50		82										
80.9							Seal										31 26 (43)
5.2	Bedrock Dolostone interbedded with shale Excellent		8	NX RC	REC 88%		Sand Backfill										RQD = 86%
			9	NX RC	REC 100%		79										RQD = 90%
							Piezometer										
78.0							Seal										
8.1	End of borehole * Standpipe Damaged																

OFFICE REPORT ON SOIL EXPLORATION

+3, x5: Numbers refer to Sensitivity

20
15
10
5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 91-6

METRIC

W P 374-89-02 LOCATION Co-ords: N: 4 956 876.9 E: 384 900.6 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 April 16, 1991 CHECKED BY G.J.K.

OFFICE REPORT ON SOIL EXPLORATION

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					
85.2	Ground Surface																
85.0	Topsoil																
0.2	Sand, some silt	Brown	1	SS	2		May 10, 1991										
84.5	Very Loose						Seal										
0.7	Clay Stiff to Very Stiff		2	SS	8		Native Backfill										
	Brown/Grey		3	TW	PH		Seal										0 0 37 63
			4	SS	11		83 Sand Backfill										
			5	SS	15		82 Seal										
3.5	Het. Mixture of Silty Sand, some clay & gravel, occ. boulders (Glacial Till)		6	SS	40/75		Native Backfill										
81.2							Seal										
4.0	Dense	Grey	7	NX RC	REC 100%		81 Sand Backfill										RDQ = 87%
	Bedrock Dolostone with large particles of sparry calcite Good to Excellent		8	NX RC	REC 92%		80 Piezometer										RDQ = 80%
			9	NX RC	REC 93%		79 Seal										RDQ = 93%
78.2																	
7.0	End of borehole																
	* Artesian head 0.2 m above ground surface encountered on May 10, 1991																

RECORD OF BOREHOLE No 91-7

METRIC

W P 374-89-02 LOCATION Co-ords: N: 4 956 847.7 E: 384 893.2 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 April 16, 1991 CHECKED BY G.J.K.

OFFICE REPORT ON SOIL EXPLORATION

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 (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			20	40	60	80	100					
84.3	Ground Surface															GR SA SI CL
84.0	Topsoil					May 10, 1991										
0.3	Sand, some Silt Very Loose	Brown	1	SS	2	Seal										
83.1			2	SS	2											
1.2	Clay Stiff to Very Stiff Grey		3	SS	4											
81.8			4	SS	48											
2.5	Het Mixture of Silty															
2.6	Sand, some clay & gravel, occ. boulders (Glacial Till)		5	NX RC	REC 99%											RQD = 99%
	Bedrock Limy Dolostone Good to Excellent		6	NX RC	REC 95%											RQD = 80%
79.8						Seal										
4.5	End of borehole															



Appendix C.

Laboratory Testing

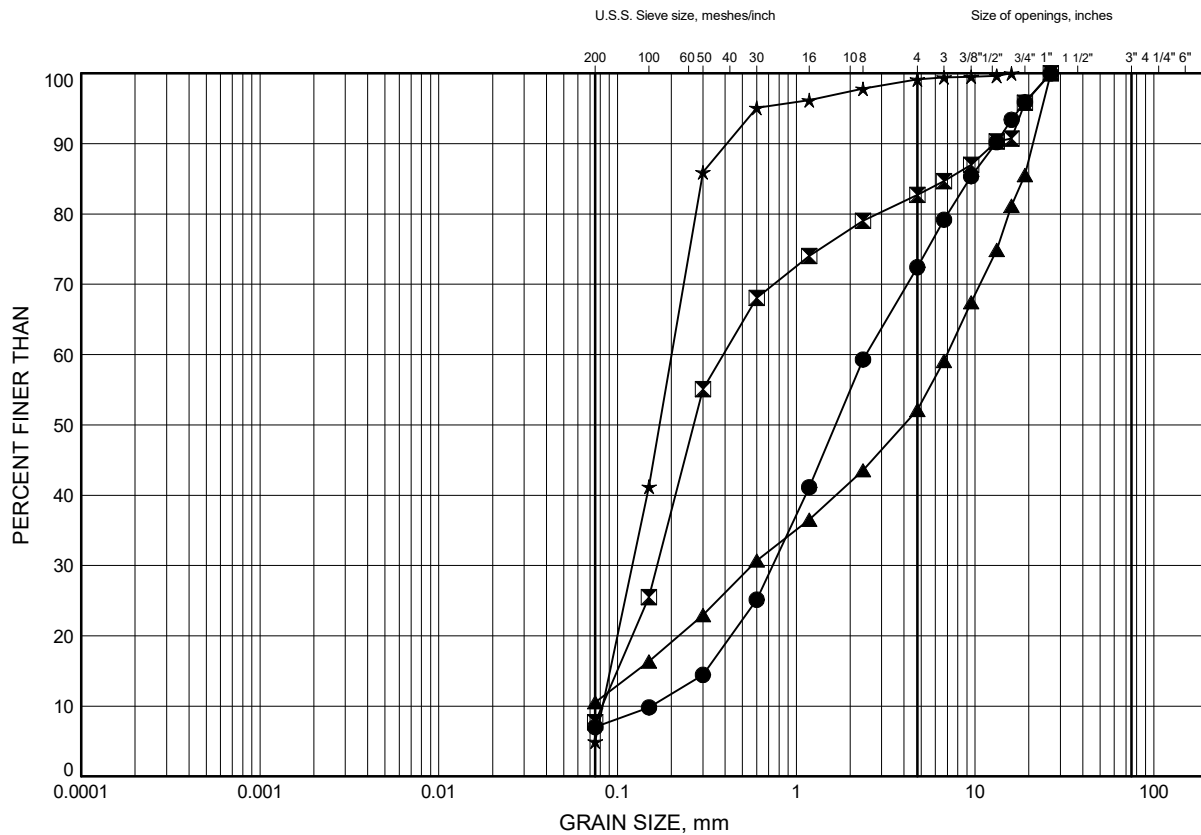


Appendix C.1
Particle Size Analysis Figures (2021)
Atterberg Limit Test Results (2021)

Hwy 401W - 416N Ramp (Site No. 16X-0306)
GRAIN SIZE DISTRIBUTION

FIGURE C1

EMBANKMENT FILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	306-21-1	1.1	93.7
⊠	306-21-1	4.1	90.7
▲	306-21-1	6.4	88.4
★	306-21-2	6.4	89.1

Date December 2021
 WP# 4024-20-00



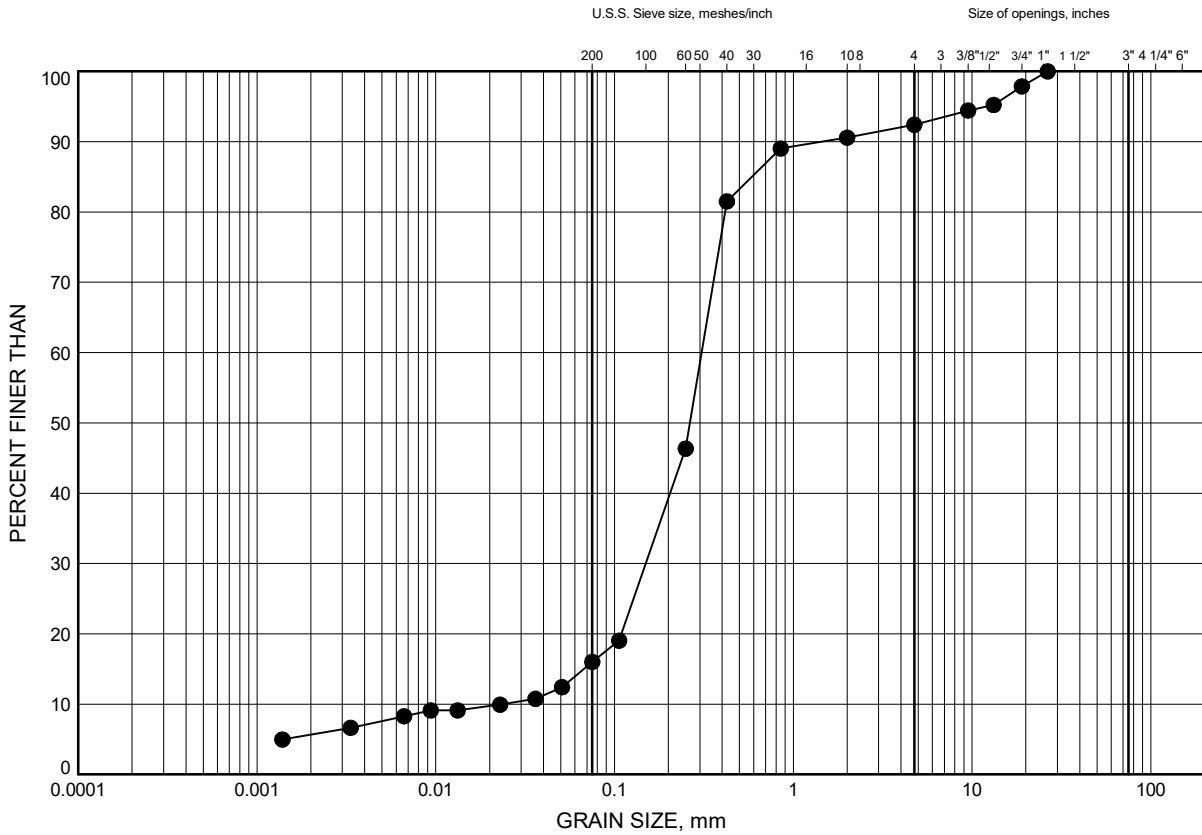
Prep'd SH
 Chkd. MJK

Hwy 401W - 416N Ramp (Site No. 16X-0306)

GRAIN SIZE DISTRIBUTION

FIGURE C2

SAND



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	306-21-1	11.0	83.8

Date December 2021

WP# 4024-20-00

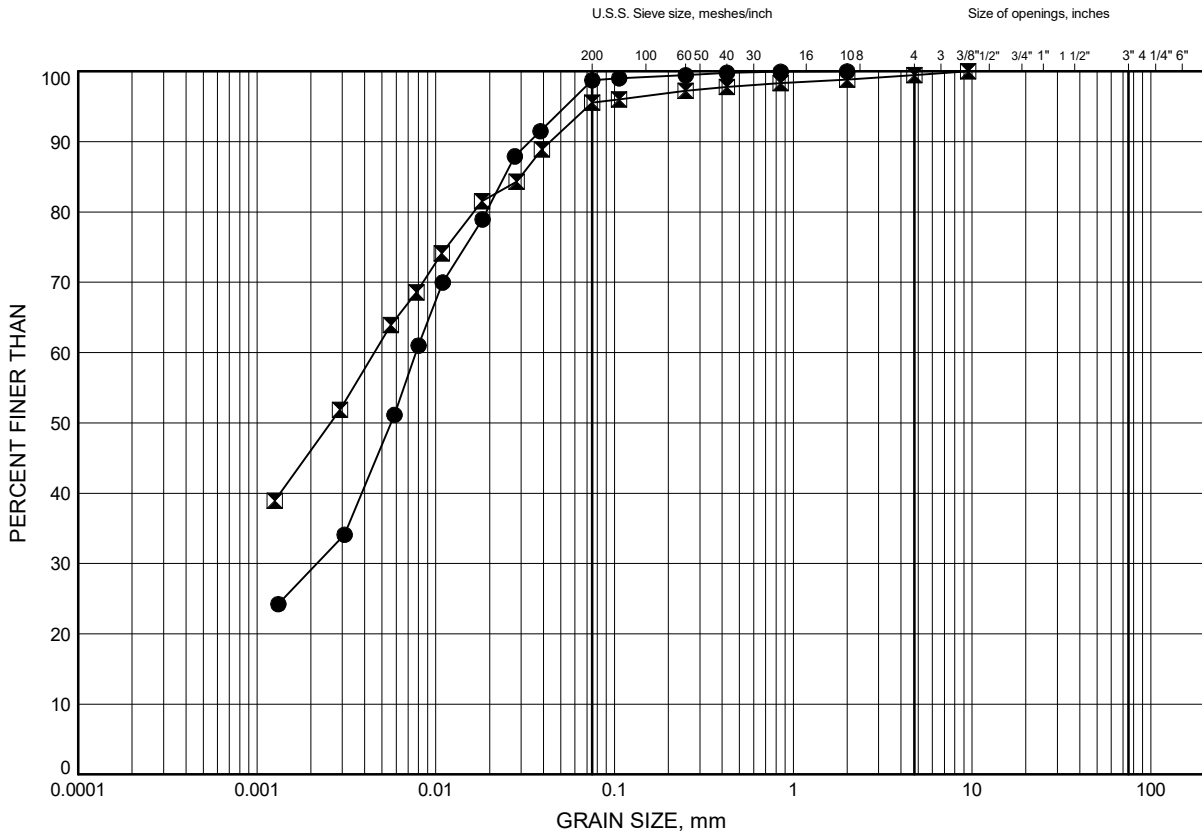


Prep'd SH

Chkd. MJK

GRAIN SIZE DISTRIBUTION

CLAYEY SILT TO SILTY CLAY



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	306-21-1	12.5	82.3
⊠	306-21-2	9.4	86.0

Date December 2021

WP# 4024-20-00



Prep'd SH

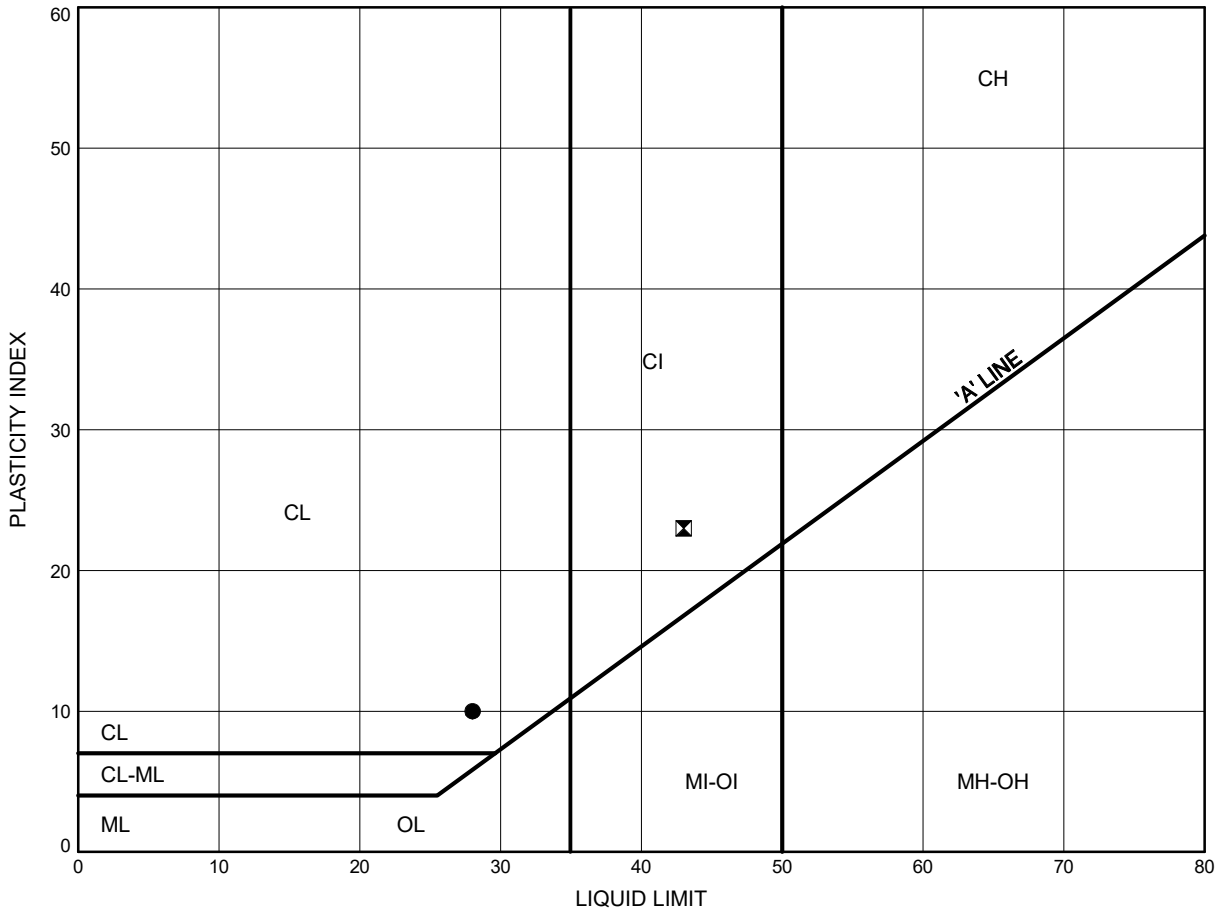
Chkd. MJK

Hwy 401W - 416N Ramp (Site No. 16X-0306)

ATTERBERG LIMITS TEST RESULTS

FIGURE C4

CLAYEY SILT TO SILTY CLAY



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	306-21-1	12.5	82.3
⊠	306-21-2	9.4	86.0

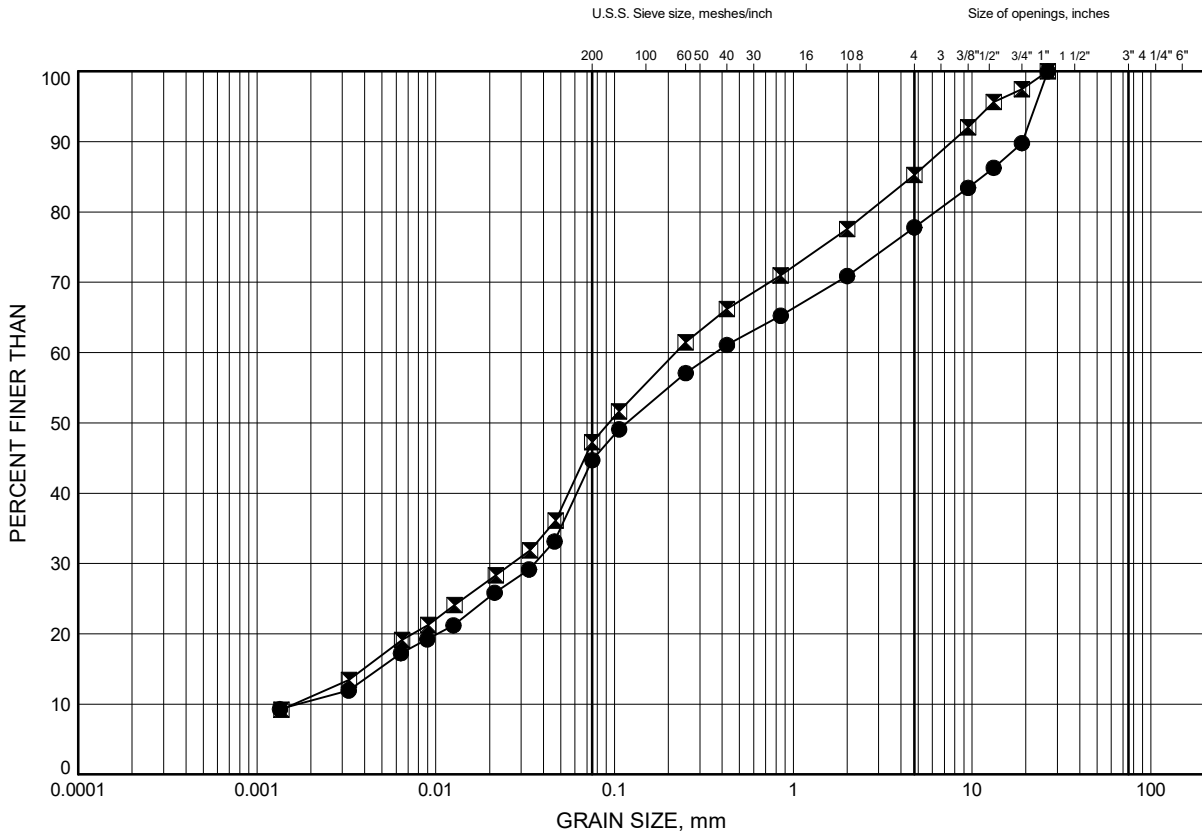
Date December 2021
 WP# 4024-20-00



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)
●	306-21-2	11.0	84.5
⊠	306-21-2	14.0	81.4

Date December 2021

WP# 4024-20-00

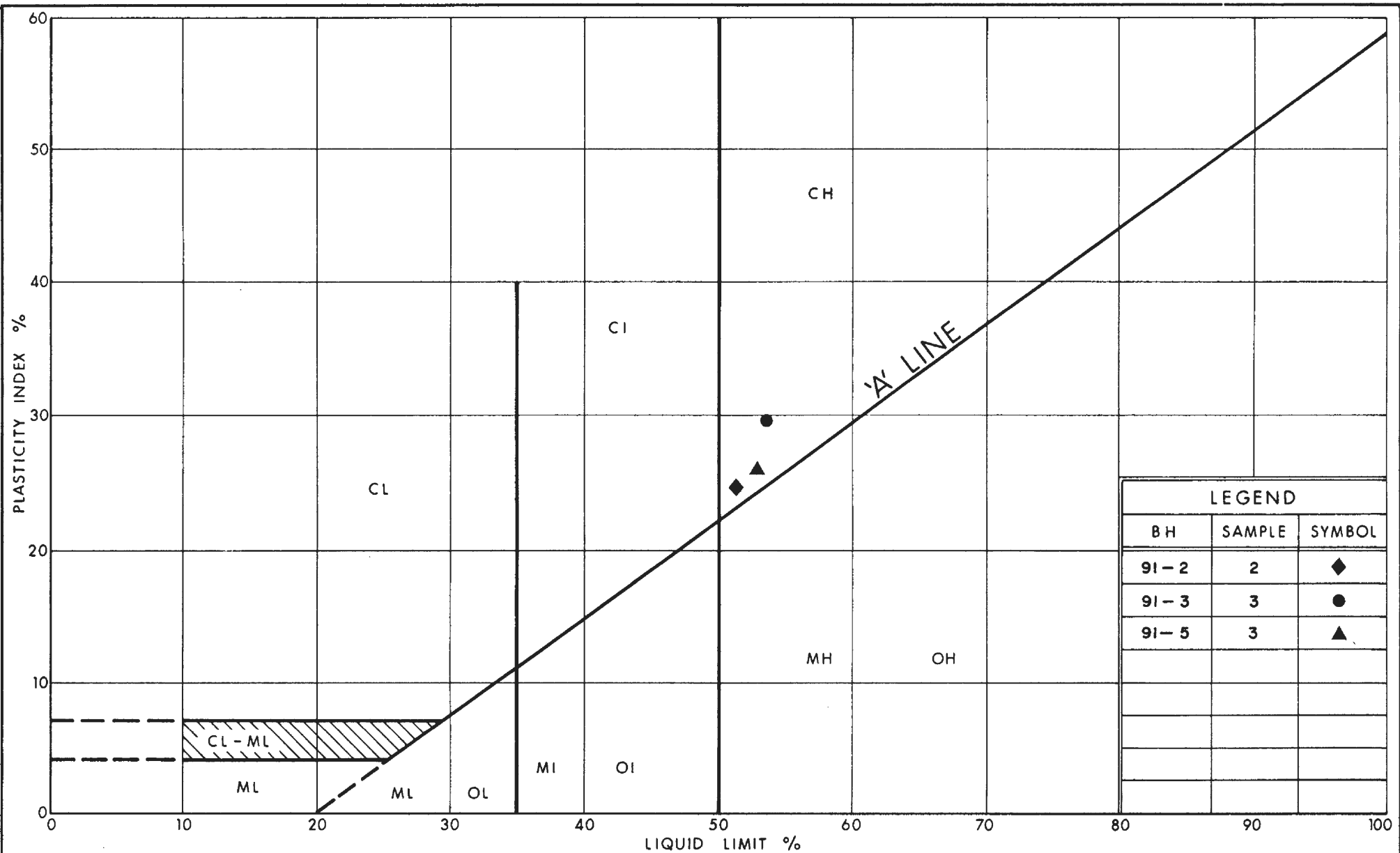


Prep'd SH

Chkd. MJK



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



Ministry of
Transportation

PLASTICITY CHART CLAY

FIG No 1

W P 374 - 89 - 02

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT

SAND

GRAVEL

Fine

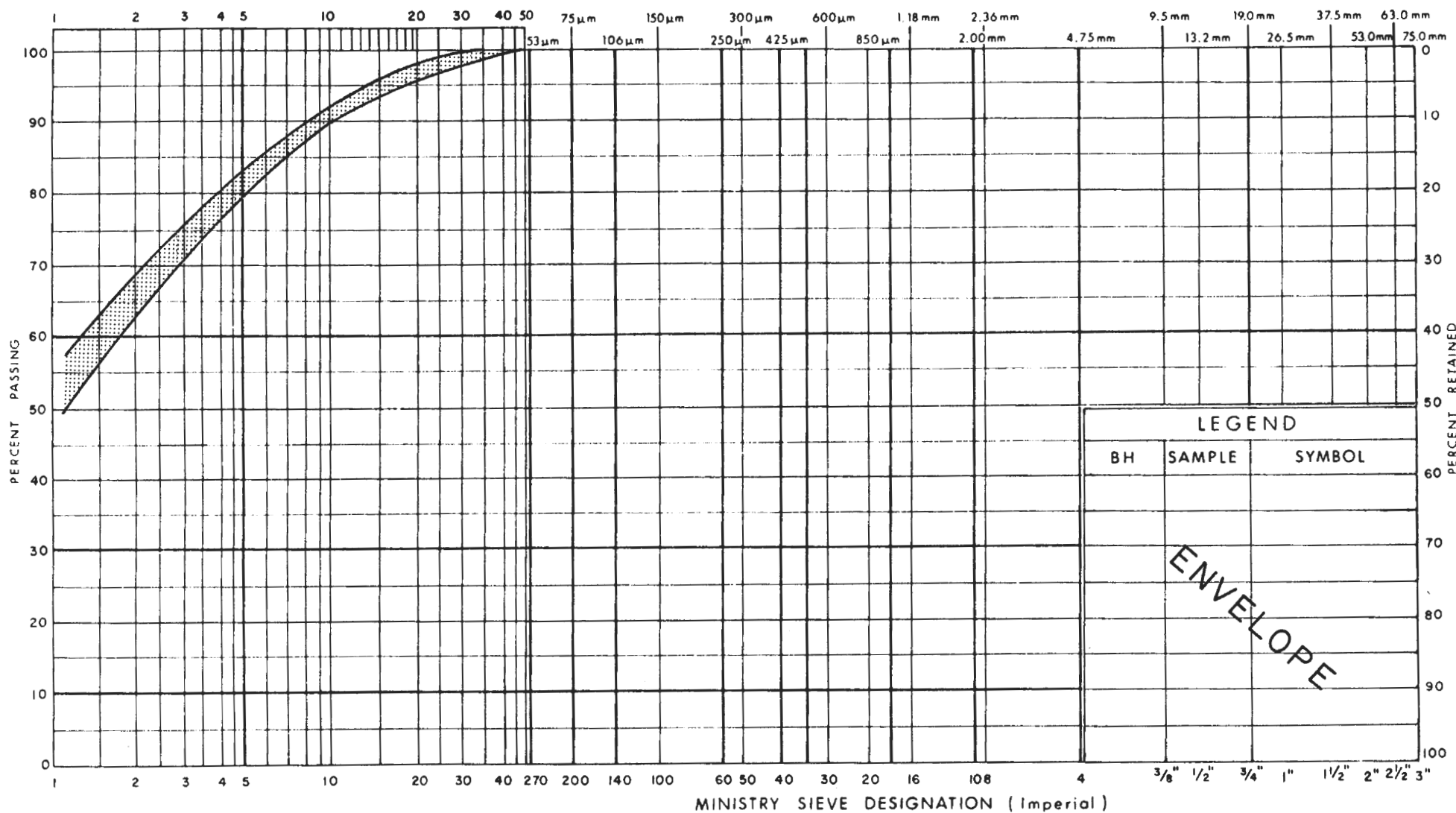
Medium

Coarse

Fine

Coarse

GRAIN SIZE IN MICROMETERS

[illegible]

Ontario

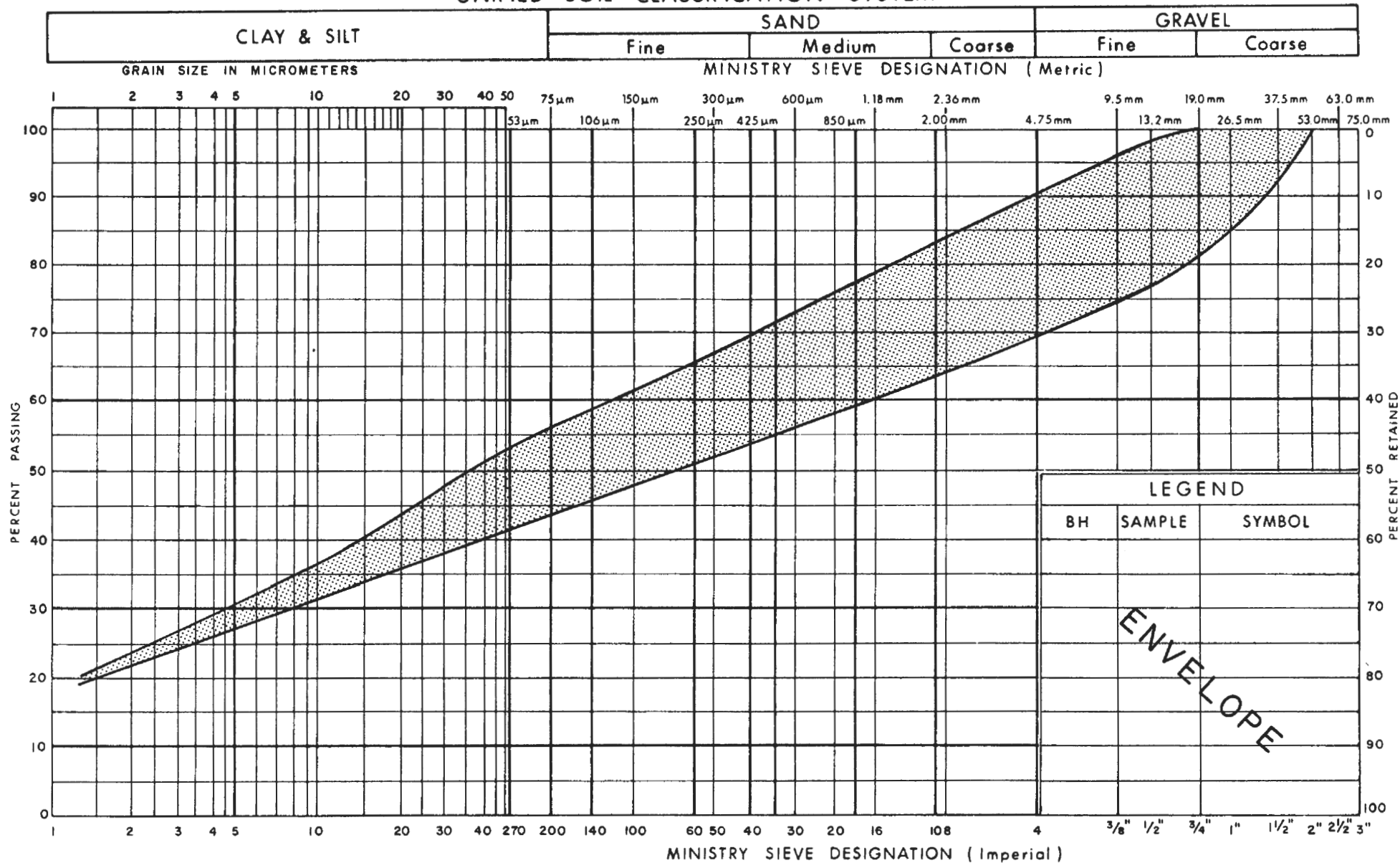
Ministry of
Transportation

GRAIN SIZE DISTRIBUTION CLAY

FIG No 2

W P 374 - 89 - 02

UNIFIED SOIL CLASSIFICATION SYSTEM

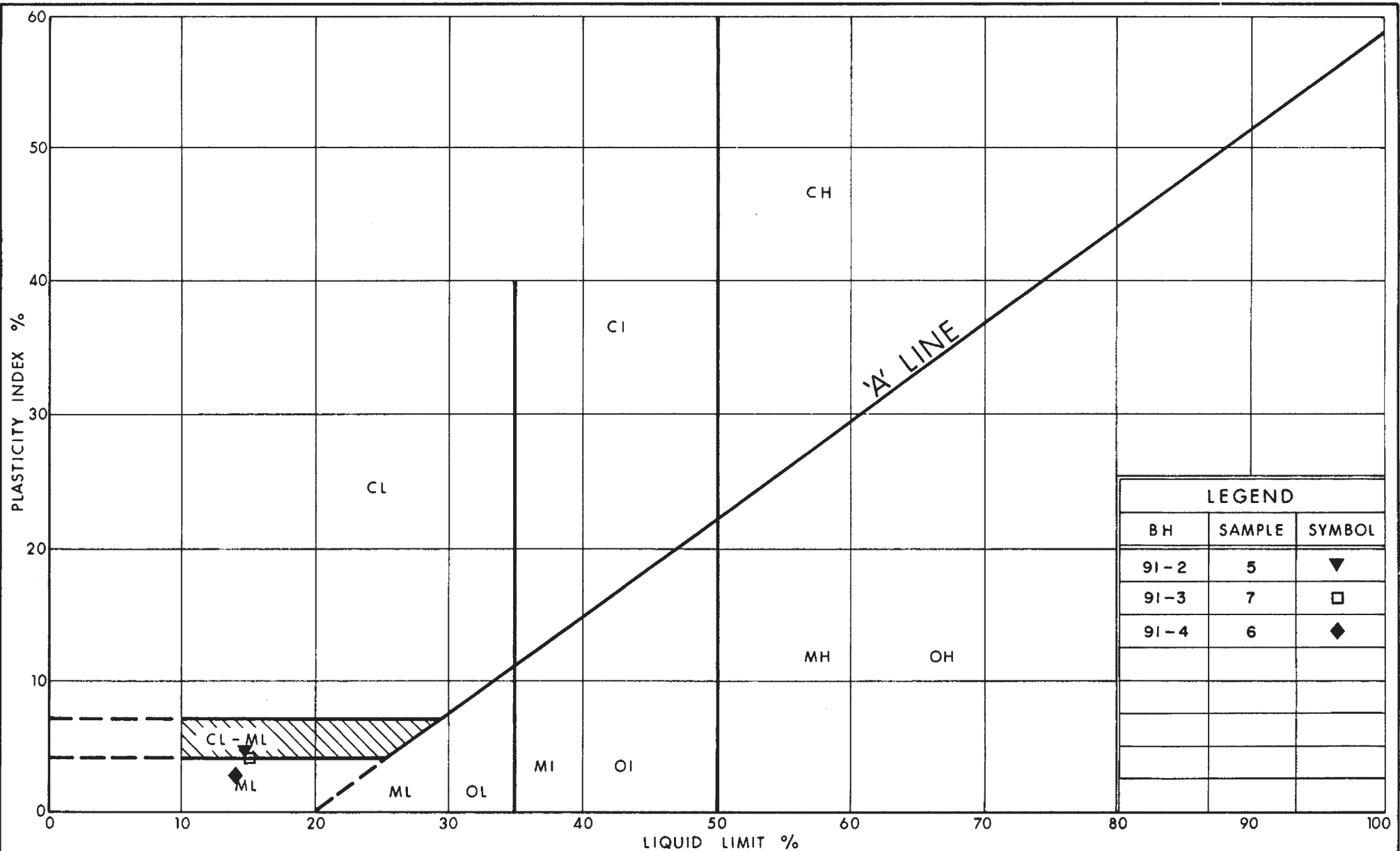


Ministry of
Transportation

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

FIG No 3

W P 374-89-02



Ministry of
Transportation

Ontario

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

FIG No 4

W P 374 - 89 - 02



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

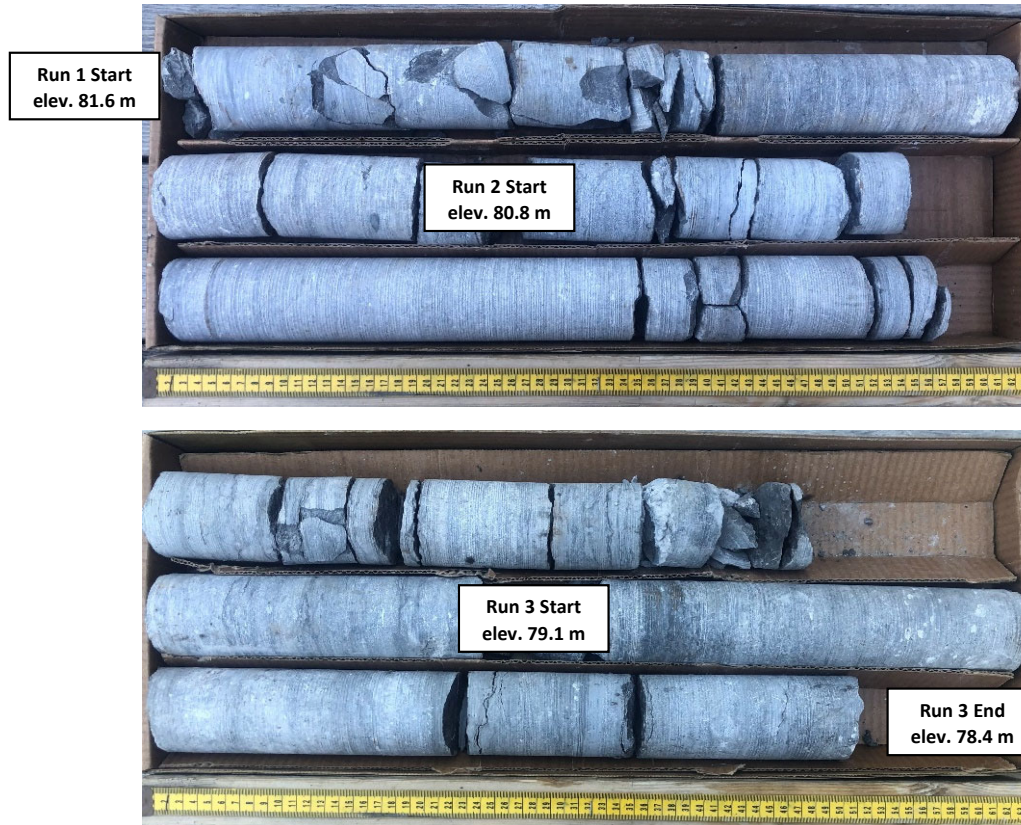
Brian Prevost
Laboratory Supervisor
Tel: 613-738-6075
brian.prevost@stantec.com



Appendix C.4

Bedrock Core Photographs

Borehole 306-21-1
Run 1 to 3 (of 3)
Elevation 81.6 m to 78.4 m
Dry

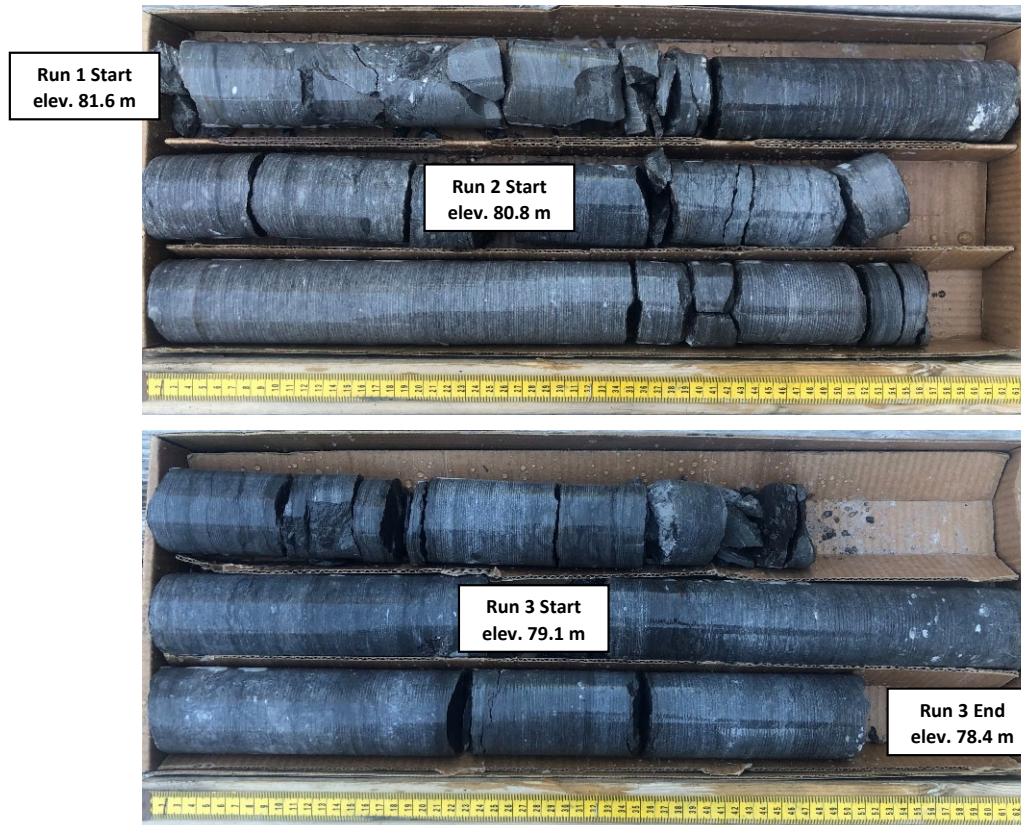


THURBER ENGINEERING LTD.

Highway 401/416 Interchange
Hwy 401W – 416N Ramp (Site No. 16X-0306)
Assignment No. 4019-E-0010.2, GWP 4024-20-00

BH 306-21-1
Project No.: 29381

Borehole 306-21-1
Run 1 to 3 (of 3)
Elevation 81.6 m to 78.4 m
Wet

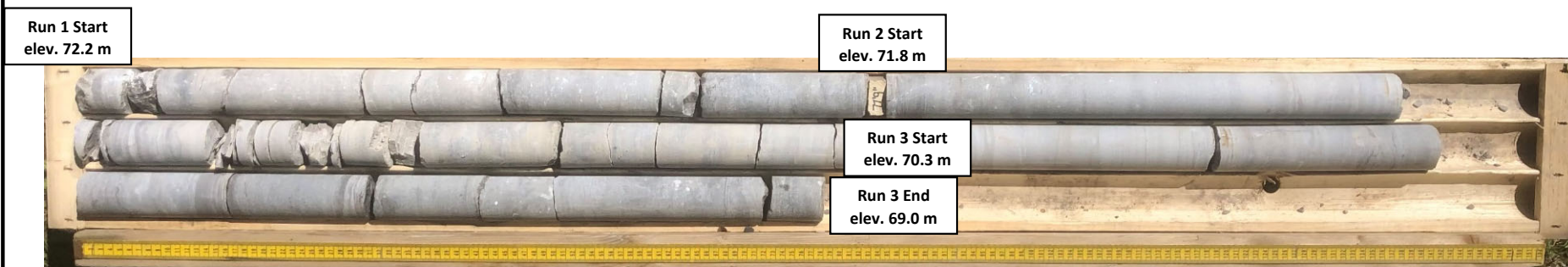


THURBER ENGINEERING LTD.

Highway 401/416 Interchange
Hwy 401W – 416N Ramp (Site No. 16X-0306)
Assignment No. 4019-E-0010.2, GWP 4024-20-00

BH 306-21-1
Project No.: 29381

Borehole 306-21-2
Run 1 to 3 (of 3)
Elevation 72.6 m to 69.0 m
Dry



THURBER ENGINEERING LTD.

Highway 401/416 Interchange
Hwy 401W – 416N Ramp (Site No. 16X-0306)
Assignment No. 4019-E-0010.2, GWP 4024-20-00

BH 306-21-2
Project No.: 29381

Borehole 306-21-2
Run 1 to 3 (of 3)
Elevation 72.6 m to 69.0 m
Wet

Run 1 Start
elev. 72.2 m

Run 2 Start
elev. 71.8 m

Run 3 Start
elev. 70.3 m

Run 3 End
elev. 69.0 m





Appendix D.

Site Photographs



Photo 1. Looking north at northwest embankment (2021/03/29)



Photo 2. Looking north at northeast embankment (2021/03/29)



Photo 3. Looking south along deck. (2021/03/29)



Photo 4. Looking north at southeast embankment and culvert 16-259/C outlet (below/right). (2021/03/29)



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



Photo 6. Looking west at south abutment bearings. (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 75.488W

User File Reference: Bridge Site 16-306

2021-07-05 18:01 UT

Requested by: Sarah Harrold, EIT, 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

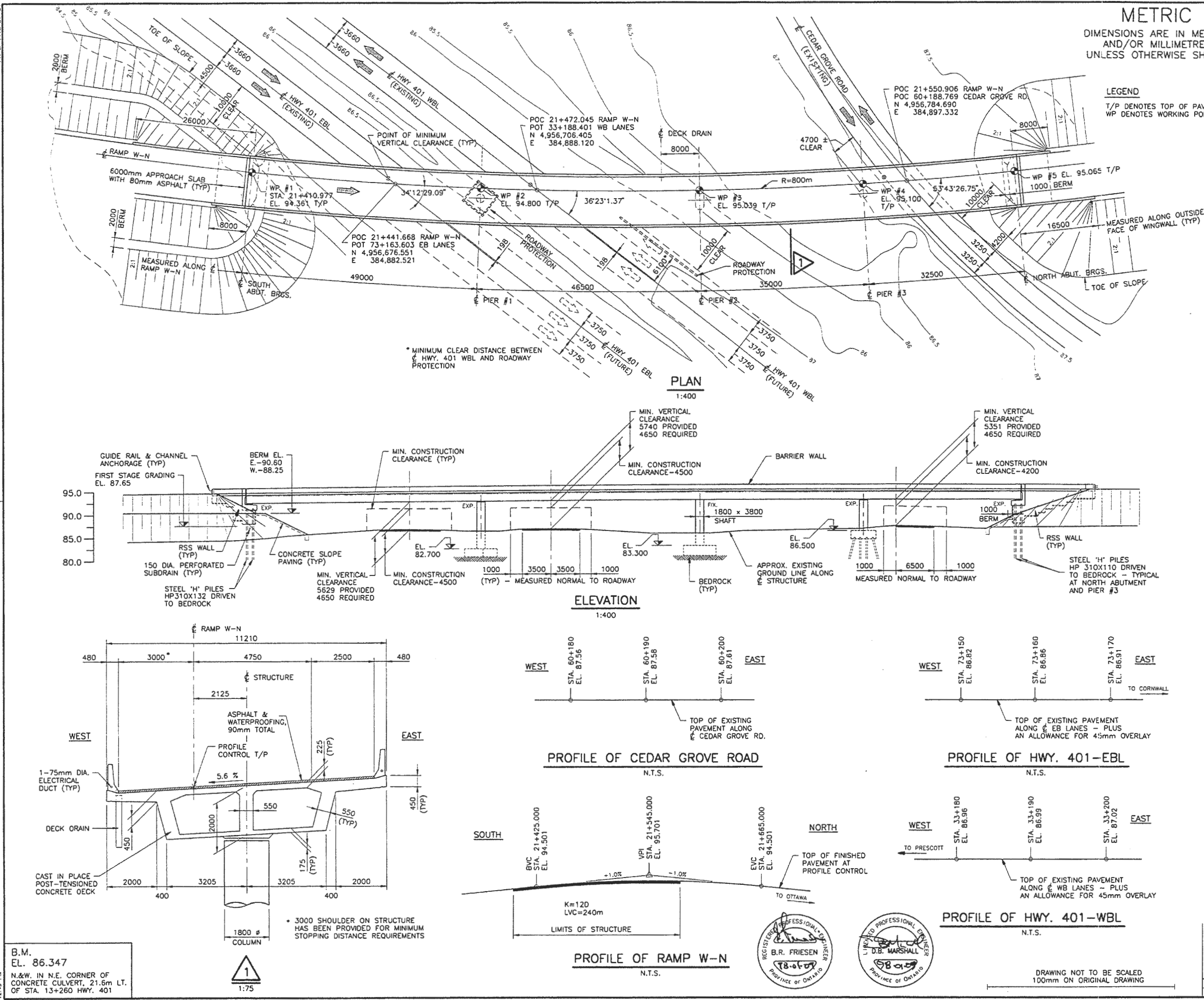


Appendix F.

General Arrangement Drawing (1997)

Foundation Layout Drawing (1997)

10-JAN-98 FILE NAME: P:\02146\B00\CA0\B500-001.DWG 12:45 PM



METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

DISTRICT NO. 42

CONT No 97-68

WP No 374-89-02

HWY. 401/416 INTERCHANGE - RAMP W-N OVER HWY 401 & CEDAR GROVE ROAD

GENERAL ARRANGEMENT

DELCAN

ENGINEERS PLANNERS

SHEET

122

GENERAL NOTES:

CLASS OF CONCRETE

CLEAR COVER TO REINFORCING STEEL

REINFORCING STEEL

CONSTRUCTION NOTES

LIST OF DRAWINGS

APPLICABLE STANDARD DRAWINGS:

REVISIONS

DATE	BY	DESCRIPTION
DESIGN BR	CHK DBM	CODE OHBDC-91 LOAD OHBD
DRAWN KRS	CHK BR	SITE 16-306

DATE NOV. 1997

DWG. 1

DRAWING NOT TO BE SCALED

100mm ON ORIGINAL DRAWING

PROFESSIONAL ENGINEER

B.R. FRIESEN

PROVINCE OF ONTARIO

PROFESSIONAL ENGINEER

D.B. MARSHALL

PROVINCE OF ONTARIO

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

CONT No 97-68
WP No 374-89-02



HWY. 401/416 INTERCHANGE - RAMP W-N
OVER HWY 401 & CEDAR GROVE ROAD

SHEET
124

DELCAN ENGINEERS
PLANNERS

NOTES:

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

PILE DESIGN DATA

STEEL 'H' PILES:

HP 310x110:
CAPACITY AT SLS = 1150 kN
FACTORED CAPACITY AT ULS = 1600 kN

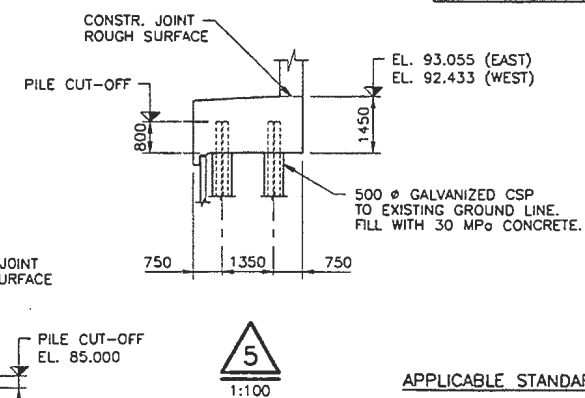
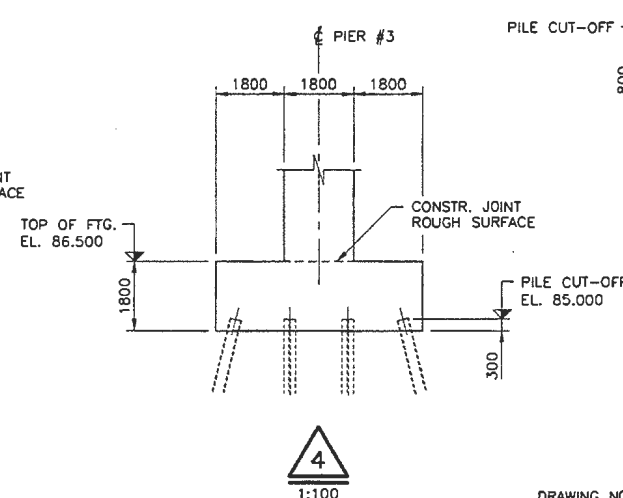
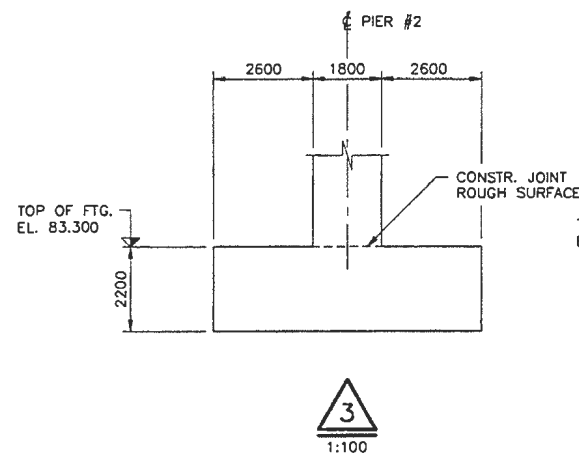
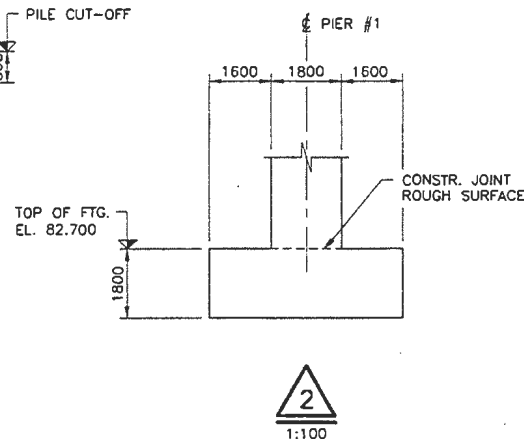
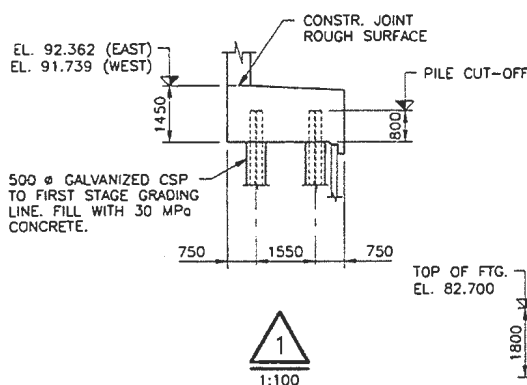
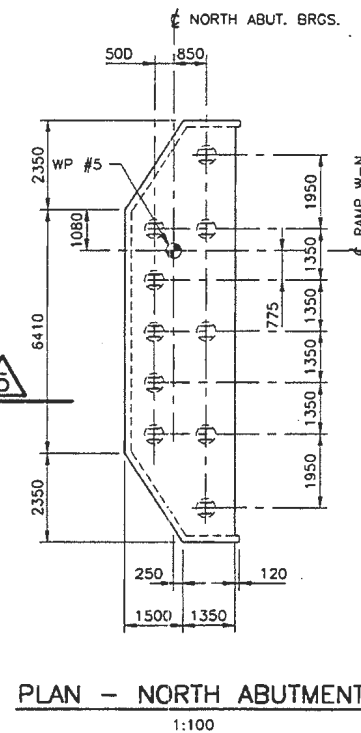
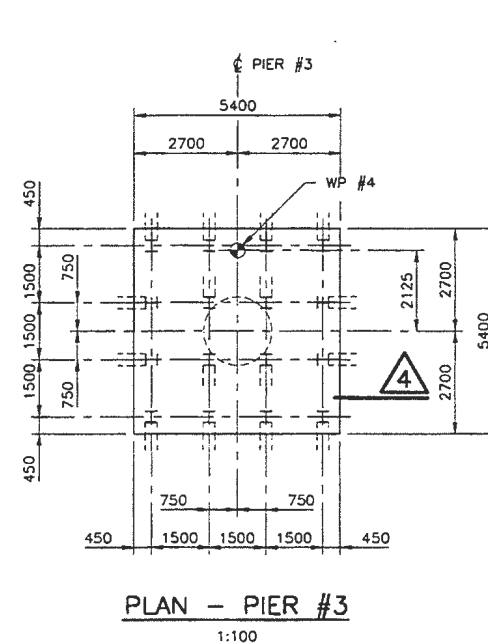
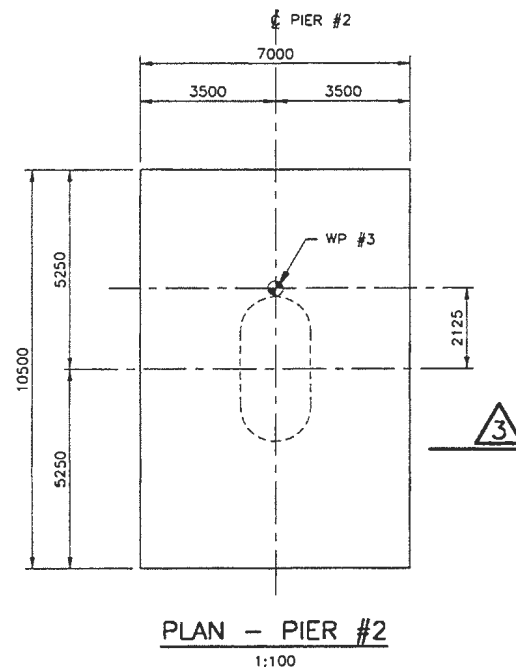
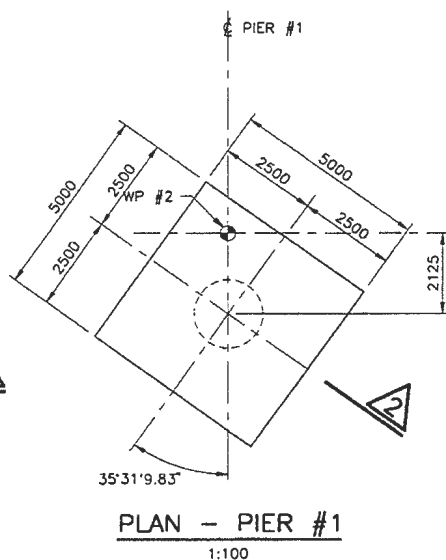
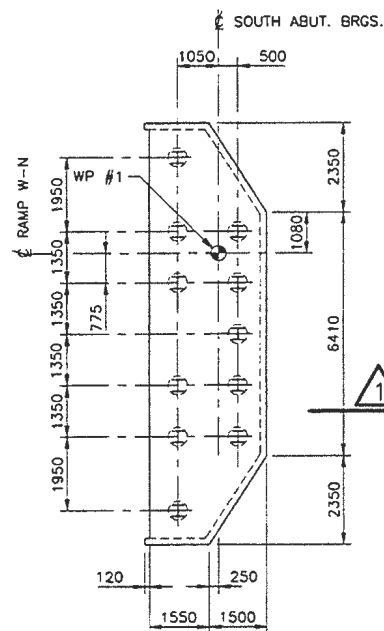
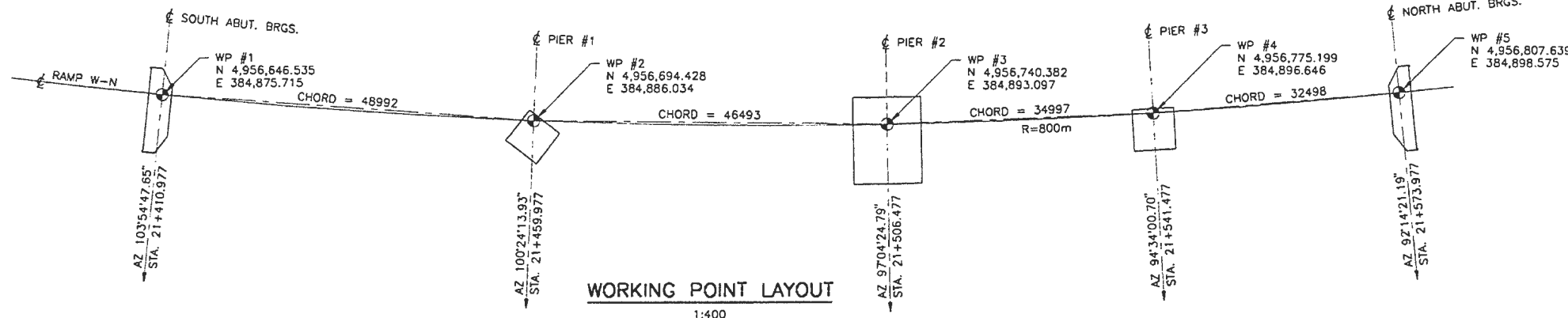
HP 310x132:
CAPACITY AT SLS = 1370 kN
FACTORED CAPACITY AT ULS = 1910 kN

ABUTMENT CONSTRUCTION SEQUENCE

1. COMPLETE FIRST STAGE GRADING AT SOUTH ABUTMENT.
2. DRIVE PILES TO REFUSAL; CUT-OFF AT ELEVATIONS INDICATED.
3. INSTALL 500mm Ø GALVANIZED CSP'S AND FILL WITH CONCRETE.
4. CONSTRUCT FIRST STAGE OF RSS WALL TO BOTTOM OF PILE CAP/ABUTMENT BEARING SEAT.
5. CONSTRUCT WIRE MESH FACING INCLUDING REINFORCING STRIPS/MESH AND EARTH VOLUME TO "FILL LINE AT TIME OF POST TENSIONING". SEE RSS WALL DRAWINGS.
6. EXTEND RSS WALLS TO THIS ELEVATION.
7. CONSTRUCT PILE CAP/ABUTMENT BEARING SEAT.
8. CONSTRUCT CONCRETE DECK, INCLUDING LONGITUDINAL POST-TENSIONING.
9. CONSTRUCT REMAINING WIRE MESH FACING FOR BALLAST WALLS INCLUDING REINFORCING STRIPS/MESH AND EARTH VOLUME, AND THE REMAINING RSS WALLS TO THE TOP OF THE WIRE MESH FACING.
10. CONSTRUCT BALLAST WALLS.
11. CONSTRUCT REMAINING RSS WALLS TO THEIR FINAL ELEVATIONS.

STEEL H-PILE DATA

LOCATION	BATTER	NO. REQ'D	LENGTH (m)	TYPE
SOUTH ABUT	VERTICAL	11	10.2	HP 310 x 132 WITH DRIVING SHOES
PIER #3	1:4	16	7.4	HP 310 x 110 WITH DRIVING SHOES
NORTH ABUT	VERTICAL	10	16.2	



APPLICABLE STANDARD DRAWINGS:

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






DATE	BY	DESCRIPTION
DESIGN FL/BRF	CHK DBM	CODE OHBDC-91 LOAD OHBD
DRAWN GG	CHK BRF	SITE 16-306
		DATE NOV 1997
		DWG. 3

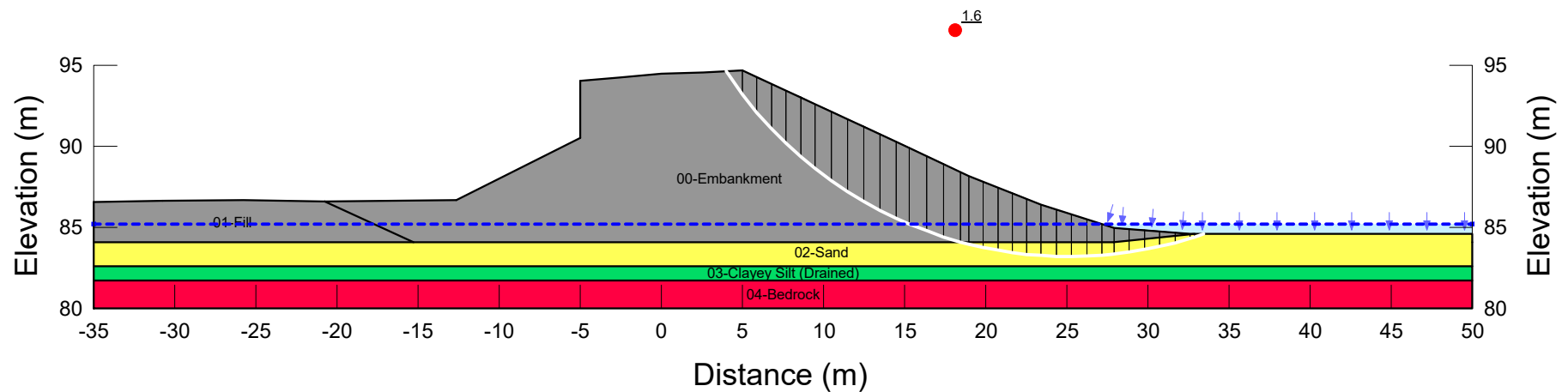
DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING



Appendix G.

Slope Stability Analysis Figures






Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	00-Embankment	Mohr-Coulomb	21	0	33
	01-Fill	Mohr-Coulomb	20	0	30
	02-Sand	Mohr-Coulomb	20	0	30
	03-Clayey Silt (Drained)	Mohr-Coulomb	18	7	29
	04-Bedrock	Bedrock (Impenetrable)			

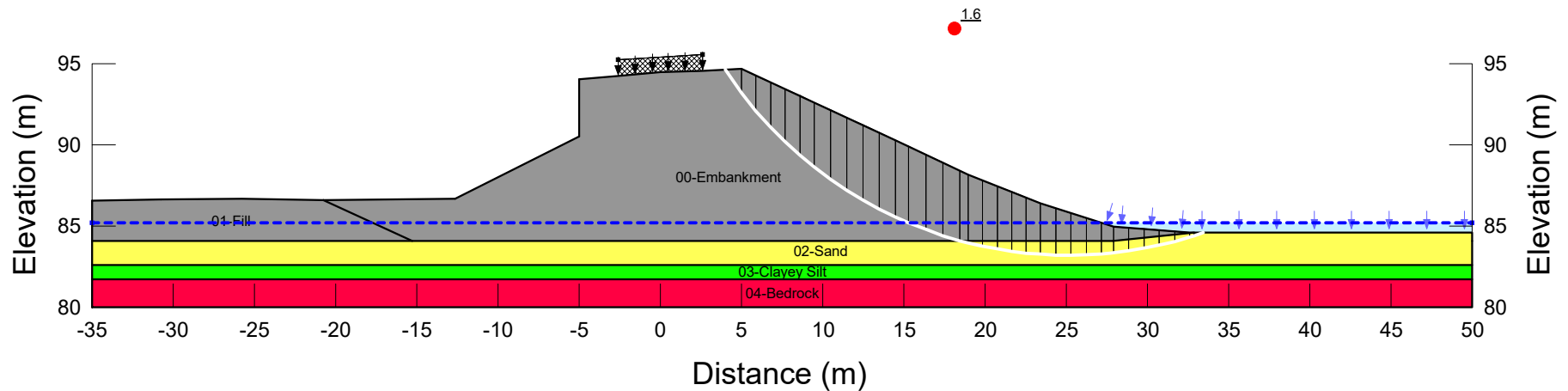


Project 401W-416N Ramp (Site 16-306) South Embankment		
Analysis Permanent		
Seismic Coefficient H: g, V: g	Last Run 01/04/2022, 09:47:12 AM	Scale 1:400

Additional Details
 Name: 2D Geometry
 Comments:
 Method: Morgenstern-Price, Half-Sine
 Minimum Slip Surface Depth: 0.1 m
 Entry: (3.996, 94.624167) m, Exit: (33.456467, 84.61) m
 Center: (25.135863, 108.47344) m, Radius: 25.272437 m

Figure G1






Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Total Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	00-Embankment	Mohr-Coulomb	21		0	33
	01-Fill	Mohr-Coulomb	20		0	30
	02-Sand	Mohr-Coulomb	20		0	30
	03-Clayey Silt	Undrained (Phi=0)	18	100		
	04-Bedrock	Bedrock (Impenetrable)				

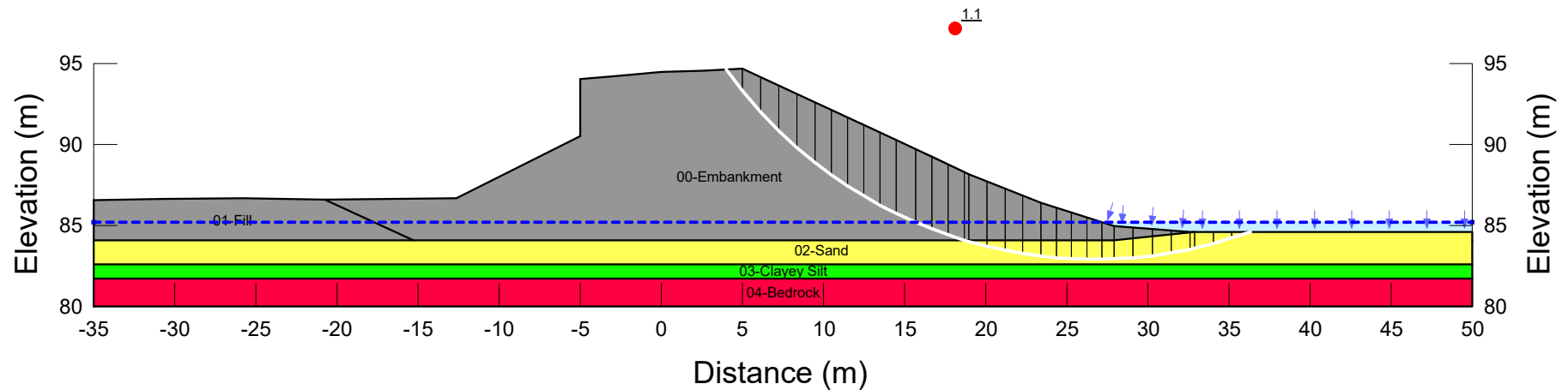


Project 401W-416N Ramp (Site 16-306) South Embankment		
Analysis Temporary - Traffic		
Seismic Coefficient H: g, V: g	Last Run 01/04/2022, 09:47:08 AM	Scale 1:400

Additional Details
Name: 2D Geometry
Comments:
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 0.1 m
Entry: (3.996, 94.624167) m, Exit: (33.456467, 84.61) m
Center: (25.135863, 108.47344) m, Radius: 25.272437 m

Figure G2






Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Total Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	00-Embankment	Mohr-Coulomb	21		0	33
	01-Fill	Mohr-Coulomb	20		0	30
	02-Sand	Mohr-Coulomb	20		0	30
	03-Clayey Silt	Undrained (Phi=0)	18	100		
	04-Bedrock	Bedrock (Impenetrable)				

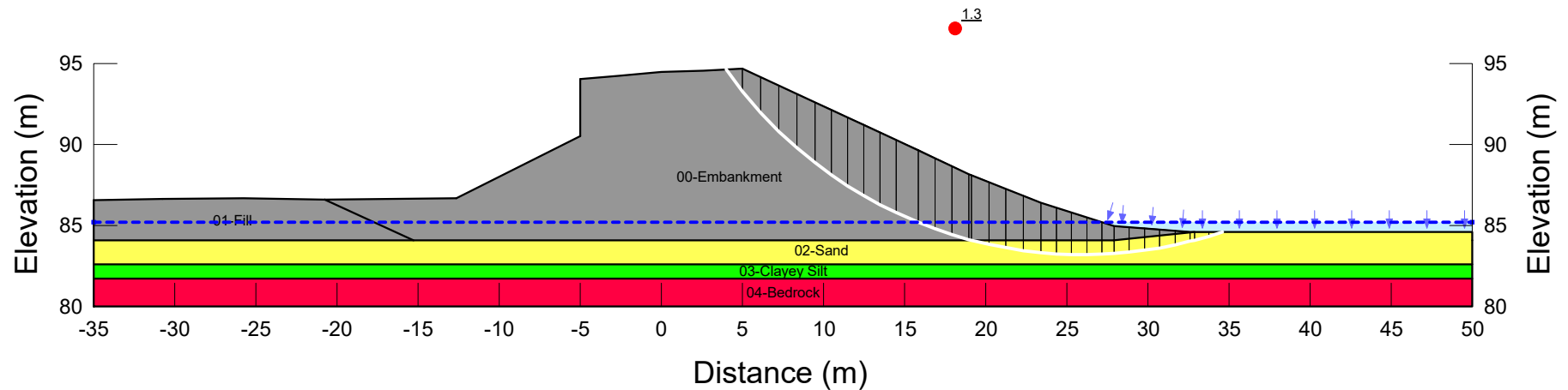


Project 401W-416N Ramp (Site 16-306) South Embankment		
Analysis Temporary - Pseudo-Static (2,475-year EQ)		
Seismic Coefficient H: 0.14g, V: g	Last Run 01/04/2022, 09:47:16 AM	Scale 1:400

Additional Details
 Name: 2D Geometry
 Comments:
 Method: Morgenstern-Price, Half-Sine
 Minimum Slip Surface Depth: 0.1 m
 Entry: (3.996, 94.624167) m, Exit: (36.3088, 84.61) m
 Center: (26.728943, 110.83767) m, Radius: 27.922471 m

Figure G3

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Total Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	00-Embankment	Mohr-Coulomb	21		0	33
	01-Fill	Mohr-Coulomb	20		0	30
	02-Sand	Mohr-Coulomb	20		0	30
	03-Clayey Silt	Undrained (Phi=0)	18	100		
	04-Bedrock	Bedrock (Impenetrable)				



Project 401W-416N Ramp (Site 16-306) South Embankment		
Analysis Temporary - Pseudo-Static (475-year EQ)		
Seismic Coefficient H: 0.06g, V: g	Last Run 01/04/2022, 09:47:18 AM	Scale 1:400

Additional Details
Name: 2D Geometry
Comments:
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 0.1 m
Entry: (3.996, 94.624167) m, Exit: (34.5974, 84.61) m
Center: (26.003724, 110.11248) m, Radius: 26.911479 m

Figure G4