



THURBER ENGINEERING LTD.

**PRELIMINARY
FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 17 TWINNING, RENFREW AREA
CPR WBL STRUCTURAL CULVERT SITE NO. 29X-0194/B1
WP 4068-09-00 / ASSIGNMENT NO. 4018-E-0009**

Geocres No.: 31F-240

Report to:

Ministry of Transportation Ontario

Latitude: 45.476925°
Longitude: -76.632043°

December 2022
Thurber File No.: 24726

TABLE OF CONTENTS

PART 1. FACTUAL INFORMATION

1	INTRODUCTION.....	1
2	SITE DESCRIPTION	2
2.1	Site Geology.....	2
3	SITE INVESTIGATION AND FIELD TESTING	2
4	LABORATORY TESTING	3
5	GENERAL DESCRIPTION OF SUBSURFACE CONDITIONS	4
5.1	Topsoil/Organics	4
5.2	Clayey Silt (CL to CL-ML).....	4
5.3	Till: Silty Sand to Clayey Sand to Sandy Silt (SM, SC-SM, SC, ML).....	6
5.4	Bedrock	7
5.5	Surface and Groundwater	7
5.6	Analytical Testing	8
6	MISCELLANEOUS	10

PART 2. ENGINEERING DISCUSSION AND RECOMMENDATIONS

7	INTRODUCTION.....	11
7.1	Proposed Structure	12
7.2	Applicable Codes and Design Considerations	12
8	SEISMIC CONSIDERATIONS	13
8.1	Spectral and Peak Acceleration Hazard Values.....	13
8.2	CHBDC Seismic Site Classification.....	13
8.3	Seismic Performance Category.....	13
8.4	Seismic Liquefaction Potential	13
9	DESIGN OPTIONS	14
9.1	Culvert Type / Foundation Alternatives	14
9.2	Construction Methodology.....	14
9.3	Drainage.....	15

10	FOUNDATION DESIGN RECOMMENDATIONS	15
10.1	Culvert Foundation Bearing Resistances	15
10.2	Subgrade Preparation, Bedding and Backfilling.....	16
10.3	Retaining Wall Foundations	17
10.3.1	Shallow Foundations for Concrete Cantilever Walls.....	18
10.3.2	RSS Walls.....	19
10.4	Frost Depth.....	20
10.5	Backfill and Lateral Earth Pressures	20
10.5.1	Static Lateral Earth Pressure	21
10.5.2	Combined Static and Seismic Lateral Earth Pressure	22
10.6	Embankment Fill.....	23
10.6.1	Embankment Stability	23
10.6.2	Embankment Settlement	26
10.7	Cement Type and Corrosion Potential	28
11	CONSTRUCTION CONSIDERATIONS.....	29
11.1	Temporary Excavations.....	29
11.2	Temporary Protection Systems	29
11.3	Surface and Groundwater Control.....	31
11.4	Erosion Control.....	32
12	CONSTRUCTION CONCERNS.....	32
13	CLOSURE.....	34
	REFERENCES.....	35



APPENDICES

Appendix A.	Borehole Location Plan and Stratigraphic Drawings
Appendix B.	Record of Borehole Sheets
Appendix C.	Laboratory Testing
Appendix D.	Site Photographs
Appendix E.	MASW Report
Appendix F.	GSC Seismic Hazard Calculation
Appendix G.	Preliminary General Arrangement Drawings
Appendix H.	Slope Stability Analysis Figures
Appendix I.	List of Referenced Specifications
	Non-Standard Special Provisions



**PRELIMINARY
FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 17 TWINNING, RENFREW AREA
CPR WBL STRUCTURAL CULVERT SITE NO. 29X-0194/B1
WP 4068-09-00 / ASSIGNMENT NO. 4018-E-0009**

Geocres No.: 31F-240

PART 1. FACTUAL INFORMATION

1 INTRODUCTION

Thurber Engineering Ltd. (Thurber) has been engaged by the Ministry of Transportation Ontario (MTO) under Assignment No. 4018-E-0009 to carry out Foundation Investigations to support the design of the Highway 17 Twinning Project which extends from Scheel Drive westerly to 3 km west of Bruce Street in the Renfrew area.

This report addresses the proposed Highway 17 westbound lanes (WBL) crossing of the now abandoned Canadian Pacific Railway (CPR) line (currently known as Algonquin Trail) east of Renfrew, Ontario. The existing Highway 17 alignment will become the future Highway 17 westbound lanes (WBL). Details of the foundation investigations associated with the proposed structure to carry future eastbound traffic over the trail are reported under separate cover.

This section of the report presents the factual findings obtained from a desktop study carried out based on historical information as well as pertinent information collected at nearby sites as part of the overall project and included:

- Memorandum prepared by Department of Highways Ontario titled, “*Foundation Investigation Report for The Overhead Structure at the Crossing of Proposed Hwy. 17 ‘New’ WBL and Canadian Pacific Railway, Twp. Of Horton – Co. of Renfrew, District No. 9 (Ottawa), W.O. 71-11085, W.P. 5-76-01*”, dated December 17, 1971 (Geocres No. 31F00-053); and,
- Report prepared by Thurber titled, “*Preliminary Foundation Investigation and Design Report, Highway 17 Twinning, Renfrew Area, CPR EBL Structural Culvert Site No. 29X-0194/B2, WP 4068-09-00 / Assignment No. 4018-E-0009*”, dated December 2022 (Geocres No. 31F-241).

The purpose of this study 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.

It should be noted that the use of and reliance on Part 1 of the Report is governed by and limited to the terms and conditions set out in the Report and a reliance letter. The Preferred Proponent remains responsible to assess the need for additional investigations and to complete that work.



2 SITE DESCRIPTION

The site is located on Highway 17 approximately 1.2 km east of O'Brien Road (Highway 60). Highway 17 is generally oriented east to west and the trail at the site is oriented roughly northeast to southwest. For project purposes, the highway and trail are herein described as oriented east-west and north-south, respectively.

The land adjacent to the site typically consists of forests, wetlands, and agricultural fields. The terrain is relatively flat apart from the existing highway embankment and drainage ditches adjacent to the trail. Highway 17 in this area consists of a two-lane undivided highway with paved shoulders. The existing embankment side slopes did not show any visible signs of distress at the time of the investigation and were sloped at approximately 2.0H:1V to 2.3H:1V.

The existing bridge is a three-span, pile supported structure. The trail crosses under the highway at a 45 degree skew, the trail is an unpaved pathway at approximate elevation 131.0 m to 131.5 m. A drainage ditch runs along the eastern edge of the trail and crosses beneath the bridge through a culvert in front of the existing east abutment. The area east of the trail and south of the existing embankment is dominated by wet ground and standing water. West of the trail are agricultural fields and areas overgrown with brush.

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

2.1 Site Geology

Based on published geological information in *The Physiography of Southern Ontario* by Chapman and Putnam (1984), the site lies within the physiographic region known as the Ottawa Valley Clay Plains. The Ottawa Valley Clay Plains are characterized primarily by clay plains deposited by the Champlain Sea (Leda Clay) interrupted by ridges of rock or sand.

Base mapping by the Ontario Geological Survey indicates the bedrock in the area is carbonate metasedimentary rocks, marble, calc-silicate rocks, skarn, tectonic breccias of the Grenville Supergroup and Finton Group.

3 SITE INVESTIGATION AND FIELD TESTING

The original site investigation for the existing Highway 17 bridge (future WBL) was carried out between September 7 and October 22, 1971. The field investigation consisted of advancing eight boreholes, identified as BH 1 to BH 8. A recent field investigation was carried out along the proposed new EBL alignment to the south and included a geophysical investigation using Multichannel Analysis of Surface Waves (MASW), completed on June 3, 2020. Details of the MASW testing equipment and methodology are provided in Appendix E.

The 1971 borehole locations were surveyed for elevation and plan location relative to chainage and offset from the then-proposed Highway 17 alignment. The plan coordinates of the boreholes were estimated relative to current base plans. The approximated northing and easting of the

boreholes are shown on the Borehole Location and Soil Strata Drawings in Appendix A and in Table 3-1 below. The drilled location is with reference to the existing bridge structure. The elevation of the boreholes have been converted to a metric geodetic benchmark and are shown on the Borehole Location and Soil Strata Drawings in Appendix A, the individual Record of Borehole sheets in Appendix B, and in Table 3-1 below. The site is located within MTM Zone 9.

Table 3-1: Borehole Summary

Borehole No.	Drilled Location	Northing* (Latitude)	Easting* (Longitude)	Ground Surface Elevation** (m)	Termination Depth (m)
BH 1	West Abutment	5037467 (45.477052)	294443 (-76.632473)	129.7	16.5
BH 2	West Abutment	5037469 (45.47707)	294459 (-76.632268)	129.6	17.6
BH 3	West Pier	5037457 (45.476962)	294457 (-76.632294)	129.8	20.2
BH 4	West Pier	5037459 (45.476981)	294472 (-76.632102)	129.8	21.9
BH 5	East Pier	5037444 (45.476846)	294474 (-76.632076)	130.0	19.8
BH 6	East Abutment	5037435 (45.476765)	294503 (-76.631705)	129.7	19.8
BH 7	West Approach	5037485 (45.477214)	294429 (-76.632652)	129.7	4.7
BH 8	East Approach	5037416 (45.476594)	294518 (-76.631513)	129.7	9.4

* The plan locations of the boreholes were estimated relative to available CAD base plan and area approximate only.

** The ground surface elevation may be significantly different now compared to that of 1971.

Soil samples were obtained at selected intervals using split spoon samplers in conjunction with Standard Penetration Testing (SPT) during the investigation. In-situ vane shear testing was carried out in cohesive soils. Shelby tube samples of the cohesive deposit were collected during the field investigation. In Boreholes BH 1 to BH 6, the bedrock was cored for lengths ranging from about 3.6 to 7.6 m to collect BX sized core samples. Groundwater levels were observed in the open boreholes during the field investigation.

4 LABORATORY TESTING

Geotechnical laboratory testing consisted of natural moisture content determination, grain size distribution testing, Atterberg Limit determination, bulk density, and lab shear vane carried out on selected samples. Consolidation testing was also carried out on two samples of the cohesive deposit.

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



5 GENERAL DESCRIPTION OF SUBSURFACE CONDITIONS

Details of the encountered soil stratigraphy are presented on the Record of Borehole sheets included in Appendix B and the Borehole Location and Soil Strata Drawings included in Appendix A. The MASW data collected as part of the 2021 investigation is provided in Appendix E, for reference. 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 Drawings and the general description. It must be recognized that the soil and groundwater conditions may vary between and beyond borehole locations. Soil classification terminology from the historic borehole information may vary from current practice.

In general terms, the site was found to have a surficial layer topsoil, and/or organics overlying a native clayey silt deposit, which is underlain by a deposit of silty sand glacial till over dolomite bedrock.

The sections below describe subsurface conditions encountered at the time the boreholes were advanced in 1971. Since the boreholes were put down prior to the construction of the existing bridge, it should be noted that surficial deposits within the vicinity of the proposed WBL culvert likely differ from that described below. The surficial deposits would likely have been disturbed, altered, or completely removed during the construction of the existing bridge.

5.1 Topsoil/Organics

Topsoil, organic silt and peat were encountered at the ground surface at all borehole locations. The 1971 investigation report described it as “loose, sandy topsoil intermixed with organic silt.” Boreholes BH 5 and BH 6 indicated that it contained peat. Moisture content testing on one sample of organic silt yielded a value of 45%. Atterberg Limit testing on the same sample indicated a liquid limit of 44% and a plasticity index of 15. This material can be classified as an organic silt of intermediate plasticity. The thickness of the surficial topsoil/organic deposit ranged from about 150 to 600 mm at the time of the investigation. The organic thickness may vary between or beyond the borehole locations and has been assumed to have been subsequently removed within the footprint of the existing bridge approach embankments and foundation elements and, therefore, the thicknesses provided above should not be used for estimating purposes at the site.

5.2 Clayey Silt (CL to CL-ML)

A native, cohesive deposit of clayey silt was encountered below the topsoil or organics in all boreholes. This layer ranged in thickness from 3.2 m to 4.3 m with an underside elevation ranging from 126.5 m to 125.3 m. Occasional sand seams up to about 75 mm thick were reported within this deposit at all borehole locations.

SPTs conducted within this layer gave N-values ranging from 2 to 27. In-situ shear vane tests indicated undrained shear strengths of 62 kPa to greater than 96 kPa, but generally indicated a stiff to very stiff consistency. Sensitivity ranged from 4 to 11.

The moisture content of the samples tested ranged from about 19 to 37%. The results of Atterberg Limits testing carried out on 11 samples of this material are summarized below and are illustrated on Figure C1 in Appendix C. The laboratory results indicate that the material is generally a clayey silt of low plasticity (CL to CL-ML).

Summary of Atterberg Limit Testing – Clayey Silt

Parameter	Value
Liquid Limit	20 – 32
Plastic Limit	14 – 18
Plasticity Index	6 – 15

The results of three grain size analysis tests conducted on samples of this material are summarized below and are illustrated on Figure C2 Appendix C.

Summary of Grain Size Distribution Testing – Clay to Clayey Silt

Soil Particle	Percentage (%)
Gravel	0 – 5
Sand	10 – 37
Silt	47 – 68
Clay	11 – 22

The results of laboratory oedometer (one-dimensional consolidation) tests carried out on two relatively undisturbed clay samples obtained with thin-walled tube samples are presented on Figure C5 in Appendix C and summarized below. The results for the nearby Borehole CPR19-6, drilled and tested as part of the nearby 2021 investigation, are also summarized in the table below.

Table 5-1: Consolidation Test Results – Clayey Silt

Parameter	Results		
Borehole	BH 4	BH 7	CPR19-6
Sample	TW 4	TW 4	ST3
Sample Depth, (m)	3.2	3.4	1.8
Sample Elevation, (m)	126.6	126.3	128.6
Approx. Existing Effective Stress, P_0 , (kPa)	32.3	34.3	25
Moisture Content, (%)	31	19	23
Liquid Limit, (%)	26	24	32
Plastic Limit, (%)	15	13	16
Liquidity Index	1.5	0.5	0.44
Unit Weight, γ (kN/m ³)	19.9*	19.9*	19.9
Specific Gravity, G_s	-	-	2.730
Initial Void Ratio e_0	1.023	0.606	0.653
Pre-consolidation Pressure, P_c' , (kPa)	254	380	270
Over Consolidation Ratio, OCR	7.9	11.1	10.8
Compression Index, C_c	0.69	0.21	0.17

Parameter	Results		
Recompression Index, C_r	0.03	0.02	0.023
Coefficient of consolidation, c_v (mm ² /s)	-	-	0.3
Coefficient of re-consolidation, c_{vr} (mm ² /s)	-	-	0.8

*assumed based on consolidation test results carried out as part of the 2020 investigation

Other test results reported in the historic report for the clayey silt include Bulk Density ranging from 18 to 21 kN/m³ and Laboratory Undrained Shear Strength ranging from 24 to greater than 96 kPa.

5.3 Till: Silty Sand to Clayey Sand to Sandy Silt (SM, SC-SM, SC, ML)

A basal till deposit was encountered beneath the clayey silt in all boreholes. The upper portions were typically silty sand with a trace to some gravel. The lower portions consisted of a heterogeneous mix of silt, sand and gravel with a trace of clay. Cobbles were encountered in deposit in Boreholes BH 1 to BH 6. Boulders should also be anticipated to be present in the deposit. In the boreholes which fully penetrated this deposit, the thickness ranged from 7.9 m to 12.5 m and the underside ranged from elevation 113.5 m to 117.4 m.

SPTs conducted in this layer gave N-values ranging from 5 to greater than 100 blows for 150 mm of penetration, indicating a loose to very dense relative density, although typically compact to dense. Refusals within this deposit were likely due to the presence of cobbles and boulders. Penetration through this layer often required the use of coring techniques.

The moisture content of samples tested from the till unit ranged from about 7 to 16%. The results of grain size distribution testing carried out on 12 samples of the till are summarized below and the envelope that defines the limits of the results is shown on Figure C4 Appendix C.

Summary of Grain Size Distribution Testing – Glacial Till

Soil Particle	Percentage (%)	
Gravel	0 – 42	
Sand	36 – 60	
Silt	54	18 – 44
Clay		4 – 15

The results of Atterberg Limits testing carried out on the fines of six samples of this material are summarized below and are illustrated on Figure C3 in Appendix C. Three additional samples of the fines portion of the deposit were tested and yielded non-plastic results. The laboratory results indicate that the fines are non-plastic to slightly plastic (ML).

Summary of Atterberg Limit Testing – Glacial Till Fines

Parameter	Value
Liquid Limit	15 – 16
Plastic Limit	12 – 13
Plasticity Index	3 – 4

5.4 Bedrock

Bedrock was proven by coring in all boreholes except BH 7 and BH 8. The bedrock encountered was described as white and pink, crystalline dolomite. Occasional vertical seams were noted in Boreholes BH 2, BH 3, BH 4 and BH 6. The upper 2.1 m to 5.5 m of the bedrock, where encountered, was indicated to be fractured. Total core recovery ranged from 5% to 100% but was generally between about 50% and 80%.

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)*	Bedrock Surface Elevation (m)
BH 1	12.3	117.4
BH 2	12.2	117.4
BH 3	13.3	116.5
BH 4	14.3	115.5
BH 5	15.3	114.7
BH 6	16.2	113.5

* Depth relative to ground surface at the time of the 1971 investigation.

5.5 Surface and Groundwater

The groundwater level was measured in the open boreholes during the 1971 drilling investigation. At that time, the groundwater level ranged between about Elevation 129.3 m to 129.9 m.

Standpipe piezometers and monitoring wells were installed in boreholes drilled as part of the 2003 and 2020 investigations at the proposed EBL trail crossing site, located about 40 m to the south. The groundwater measurements taken at those locations as part of that study are summarized in Table 5-1, below. The locations, elevations, and installation details of the piezometers and wells are provided under separate cover (Geocres. No. 31F-241) and reference must be made to the details presented therein.

Based on site visits carried out at both sites, ponded water was observed at or above surface year-round in the area.

The groundwater level at the time of construction may be different from those observations described above and seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after periods of significant and/or prolonged precipitation.

Table 5-1: Summary of Groundwater Levels

Borehole No.	Bottom of Screen Elevation (m)	Screened Unit	Depth (mbgs)	Groundwater Elevation (m)	Date of Measurement
CPR-1	113.1	Bedrock	0.4	129.5	October 22, 2003
			0.6	129.3	February 4, 2004
			0.2	129.7	March 11, 2004
CPR-2	119.8	Till	0.5	129.4	October 22, 2003
			0.0	129.9	February 4, 2004
			0.0	129.9	March 11, 2004
CPR19-2	120.6	Till	1.1	129.0	July 22, 2020
			0.7	129.4	September 29, 2020
			0.8	129.3	September 24, 2021
			0.5	129.6	October 4, 2021
			0.7	129.4	January 21, 2022
CPR19-3	112.4	Bedrock	0.5	129.5	July 22, 2020
			0.1	129.9	September 29, 2020
			0.1	129.9	September 28, 2021
			-0.1 ^a	130.1	October 4, 2021
			-0.3 ^a	130.3	January 21, 2022
CPR19-5	124.8	Clayey Silt	0.8	130.4	April 21, 2020
			1.1	130.1	September 29, 2020
			1.1	130.1	September 28, 2021

Note: ^a negative depth indicates artesian conditions

5.6 Analytical Testing

Analytical testing was not carried out as part of the 1971 investigation. However, four samples of the native soils collected during the 2020 investigation carried out at the proposed EBL trail crossing site, approximately 40 m to the south, were submitted for analysis of pH, water soluble sulphate, sulphide and chloride concentrations, resistivity, and conductivity as part of that investigation. The analysis results are included in that report (Geocres No. 31F-241) and are summarized below in Table 5-3, for reference.

Table 5-3: Summary of Chemical Analysis Results (Geocres No. 31F-241)

Parameter	Result Range
Depth (m)	0.8 – 2.1
Chloride (µg/g)	14 – 183
Sulphate (µg/g)	17 – 64
Sulphide (%)	<0.04
pH (-)	7.4 – 7.7
Resistivity (Ohm-cm)	2,300 – 6,790
Conductivity (µS/cm)	147 – 435

6 MISCELLANEOUS

Overall project management and direction of the field visits were provided by Fred Griffiths, P.Eng.

It is noted that the information provided herein is based on an investigation completed prior to construction of Highway 17 and the existing CPR overhead bridge. It is likely that conditions have changed on site during the intervening years.

Interpretation of the factual data and preparation of this report were carried out by Matt Kennedy, P.Eng. and Fred Griffiths, P.Eng. The report was reviewed by P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.



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



Dr. Fred Griffiths, P.Eng.
Senior Geotechnical Engineer,
Senior Associate



Dr. P.K. Chatterji, P.Eng.
MTO Review Principal,
Senior Geotechnical Engineer



**PRELIMINARY
FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 17 TWINNING, RENFREW AREA
CPR WBL STRUCTURAL CULVERT SITE NO. 29X-0194/B1
WP 4068-09-00 / ASSIGNMENT NO. 4018-E-0009**

Geocres No.: 31F-240

PART 2. ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 INTRODUCTION

This report presents the interpretation of the factual data obtained from a desktop study carried out by Thurber for the construction of a structural culvert and high fills at the proposed Highway 17 westbound lanes (WBL) crossing of the now abandoned Canadian Pacific Railway (CPR) line (currently known as Algonquin Trail).

The site is located on Highway 17 approximately 1.2 km east of O'Brien Road (Highway 60). At the site, the existing Highway 17 will become the future westbound lanes while the new future eastbound alignment will be located approximately 40 m south of the existing alignment. The existing three-span bridge is to be replaced with a structural culvert to carry the new Highway 17 westbound lanes over the trail.

A new structural culvert is also to be constructed to the south, to carry the new Highway 17 eastbound lanes over the trail. The preliminary foundation investigation and design report for that structure is under separate cover.

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 design-build contractors. It should be noted that the use of and reliance on Part 1 of the Report is governed by and limited to the terms and conditions set out in the Report and a reliance letter. The Preferred Proponent remains responsible to assess the need for additional investigations and to complete that work. The Preferred Proponent must make their own interpretation based on the factual data in Part 1 of the report. The information included in Part 2 is not to be relied upon for design purposes and foundation design is the sole responsibility of the Preferred Proponent. No use shall be made of Part 2 or any part thereof. The Preferred Proponent 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 replacement of the existing three-span bridge with the proposed WBL culvert and construction of the associated high fills. The discussions and recommendations presented in this report are based on the information provided by the Ministry of Transportation of Ontario (MTO), the factual data included in the original 1971 investigation (Geocres No. 31F00-053), and on the factual data obtained



during the course of the 2020 investigation carried out nearby for the proposed EBL culvert (Geocres No. 31F-241).

7.1 Proposed Structure

Highway 17 is generally east-west and the trail at the site is oriented roughly northeast to southwest. For project purposes, the highway and trail are herein described as oriented east-west and north-south, respectively.

Based on available General Arrangement (GA) drawing, dated September 24, 2021 (see Appendix G), the new WBL structure is to consist of a closed-bottom concrete box culvert with inside span and height of 5.0 m. The culvert under the WBL is to be about 43.8 m long. It has been assumed that the top slab will be approximately 0.4 m thick and the base slab 0.5 m thick, based on interpretation of the GA. Retaining walls are indicated on the east side at the north end of the culvert and on both sides of the south end connecting the WBL culvert with the proposed EBL culvert to the south.

The following summary of the proposed design, based on the GA drawing, has been developed for foundation purposes:

- The finished grade of the proposed Highway 17 westbound lanes at the culvert will be approximately Elevation 139.9 m.
- The width of the westbound roadway will be about 9.9 m to allow for two travelled lanes and shoulders. The GA also indicates the provision for future widening to the inside for a third lane.
- The fill height over the EBL culvert will be up to about 3.6 m.
- The trail elevation inside the culvert will be similar to the existing trail at approximately 130.9 m.
- The underside of the concrete culvert base slab will be at approximate Elevation 130.4 m.

7.2 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 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, Ontario (MTO).

It is understood that the new structure is being designed to the “Major Route” importance category.



This project has been assigned Typical Consequence Classification, in accordance with Section 6.5.1 of the CHBDC. Accordingly, a consequence factor (Ψ) of 1.0, as per Table 6.1 of the CHBDC, has been used in assessing the factored geotechnical resistances.

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

8 SEISMIC CONSIDERATIONS

8.1 Spectral and Peak Acceleration Hazard Values

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

The site coefficients used to determine the design spectral acceleration and displacement values are a function of the Site Class, the peak ground acceleration (PGA) and $S_a(0.2)$. The PGA for this location for a *reference* Site Class C with a 2% probability of exceedance in 50 years is 0.23 g (1 in 2475 year). This value is to be scaled by the $F(PGA)$ based on the site-specific Site Class as per 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. Based on the measured shear wave velocities from the MASW test (presented in Appendix E), this site has been classified as a Site Class C in accordance with Table 4.1 of the CHBDC.

8.3 Seismic Performance Category

In consideration of the Site Class C spectral values for the site and the designated "Major Route" importance category, the geotechnical systems at the site would fall into either Seismic Performance Category 1, if they have a fundamental period greater than or equal to 0.5 seconds, or Seismic Performance Category 3, if they have a fundamental period less than 0.5 seconds, as per Section 4.4.4 (Table 4.10) of the CHBDC. It is assumed that the WBL culvert will have a fundamental period of less than 0.5 seconds and, therefore, would fall into Seismic Performance Category 3.

8.4 Seismic Liquefaction Potential

The susceptibility of the cohesive soils at this site to experience liquefaction/cyclic softening was assessed following the Boulanger and Idriss (2007)ⁱ criteria using measured undrained shear



strengths. The results of the analysis indicate the cohesive material is not susceptible to cyclic mobility.

The susceptibility of the cohesionless soils at the site to experience liquefaction was assessed using the SPT data following the simplified method for cohesionless soil as outlined in Boulanger and Idriss (2014)ⁱⁱ. The results of the assessment using data from all 1971 boreholes indicated that a portion of the glacial till at Borehole BH 8, between about Elevation 125.0 m and 121.9 m, is potentially liquefiable following the 2,475-year earthquake. However, Borehole BH 8 was put down greater than 50 m east of the proposed WBL culvert site and was, therefore, considered not relevant to the liquefaction assessment and was excluded.

Based on the assessment results from the nearby 1971 boreholes, put down within 50 m of the proposed structure, only one SPT at Borehole BH 4 (Elevation 124.9 m) gave a liquefaction factor of safety of less than 1.0. Since the result was at a single, discrete location, it is not considered to be representative of the anticipated post-seismic behavior of the overall deposit. Further, since the results of the liquefaction assessment carried out at the adjacent proposed EBL culvert using SPT data collected from the 2020 investigation did not identify any potentially liquefiable cohesionless deposits, the cohesionless soils at the CPR WBL culvert site may be considered to be non-liquefiable.

9 DESIGN OPTIONS

9.1 Culvert Type / Foundation Alternatives

At the time of preparation of this report, it is understood that the WBL culvert is to consist of a closed-bottom, concrete box culvert structure with an interior span and height of 5.0 m, and a length of 43.8 m. The invert elevation is understood to be 130.9 m with a bottom slab about 0.5 m thick. It is noted that other culvert types (e.g., closed-bottom arch, and possibly open-bottom box/arch) may also be technically feasible.

9.2 Construction Methodology

At the time of preparation of this report, it is understood that the proposed Highway 17 eastbound lanes (including the new EBL culvert structure and embankment) will be constructed first, then traffic will be detoured onto the new eastbound lanes during replacement of the bridge with a new culvert structure for the future westbound lanes. As such, the foundation recommendations presented herein have been prepared based on the assumption that WBL culvert construction will be carried out under full closure conditions once traffic is fully detoured to the new eastbound lanes and will not require closure of the existing Highway 17 or associated roadway protection.

The existing superstructure and bridge foundation elements will require removal prior to site preparation for and construction of the WBL culvert. Based on the original 1972 GA drawing for the existing CPR overhead bridge (see Appendix G), the undersides of the existing pier pile caps are understood to be at Elevation 128.0 m. The pier pile caps and piles may be left in place, provided that they are outside the zone of influence of the WBL culvert foundation (i.e. outside



the area defined by a 1H:1V line projected outwards and downwards from the edge of the trail culvert footing).

9.3 Drainage

If necessary, it is recommended that drainage across Highway 17 be provided via separate culverts parallel to the new trail culverts. The drainage culverts should be circular concrete pipes placed outside the zone of influence of the trail culvert foundations, i.e. outside the area defined by a 1H:1V line projected outwards and downwards from the edge of the trail culvert footing. Furthermore, if the existing drainage culvert located in front of the east abutment of the existing bridge is within the zone of influence of the new trail culvert foundations, it should be removed and relocated. Flow in ditches should be assessed and the drainage culverts appropriately sized to ensure that the trail structures remain dry at all times.

10 FOUNDATION DESIGN RECOMMENDATIONS

Foundation design aspects for the culvert include subgrade conditions, settlement of the founding soils, imposed loading pressures, and water control. The culvert must be designed to resist loadings including lateral earth pressures, hydrostatic pressure, weight of embankment fill, traffic loading and any surcharge due to construction equipment and activities under static and seismic conditions. Additionally, the culvert must be waterproof, and settlement/heave due to frost action should be minimized such that the culvert can remain serviceable. If the culvert designer determines that the culvert cannot settle or heave due to frost, then the culvert must be protected against frost impacts by replacing all frost-susceptible soil below and adjacent to the culvert with drained, non-frost-susceptible, granular backfill or must be equivalently insulated. It is noted that minor settlements could be accommodated by regrading of a granular substrate layer within the culvert.

10.1 Culvert Foundation Bearing Resistances

A closed-bottom, pre-cast, concrete box culvert may be founded on a bedding layer (see Section 10.2) in a dewatered, temporary excavation overlying the native undisturbed clayey silt to silty clay soils. It is anticipated that the underside of the bedding layer of the culvert will be at approximate elevation 130.1 m. Subgrade preparation should be as described in Section 10.2 and will include removal of unsuitable materials. Frost protection requirements are provided in Section 10.4. Surface water diversion and dewatering may be required to place the bedding material and install the culverts in the dry (Section 11.3).

For a closed-bottom box culvert with a bearing width of 5.7 m and the underside of culvert base slab at or below about approximate elevation 130.4 m, installed on a bedding layer as described in Section 10.2 placed on the undisturbed native silty clay, and constructed after completion of settlement under the full height preloads recommended in Section 10.6.2, the design can be based on the following values:



- Factored Geotechnical Resistance at ULS of 300 kPa
- Factored Geotechnical Resistance at SLS* of 150 kPa

Note (*): The SLS value provided is for settlements up to 25 mm for culvert construction carried out following completion of the preload period described in Section 10.6.2. If the culvert is constructed with a camber to accommodate the anticipated embankment settlement, the SLS resistance will be higher since the culvert is designed for greater settlement.

The factored geotechnical resistances include the following factors:

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

The bearing resistance values are for vertical, concentric loading. In the case of eccentric or inclined loading, the bearing resistance must be reduced in accordance with CHBDC Clause 6.10.3 and Clause 6.10.4. Post-construction foundation settlement, based on the supplied SLS resistance, is expected to be up to 25 mm for the applied load. Additional discussion on settlement induced by embankment construction and its mitigation is presented in Section 10.6.2.

Resistance to lateral forces/sliding resistance between the precast concrete and the underlying Granular A bedding fill (Section 10.2) should be evaluated in accordance with the CHBDC assuming an unfactored coefficient of friction of 0.45. A reduction factor of 0.8 (as per CHBDC Table 6.2) should be used to estimate the sliding resistance between the culvert and Granular A bedding fill. An unfactored coefficient of friction of 0.35 can be assumed for the interface between the Granular A bedding fill and the silty clay. A reduction factor of 0.6 (as per CHBDC Table 6.2) should be used to estimate the sliding resistance between the Granular A and the clay subgrade.

10.2 Subgrade Preparation, Bedding and Backfilling

All organics, soft or loose deposits, disturbed soils, and deleterious materials must be stripped from the footprint of the culvert foundations to expose competent subgrade at or below the desired founding elevations. It is noted that, within the vicinity of the culvert, peat was encountered in the boreholes put down as part of the 1971 investigation to elevations as deep as about 129.2 m. During the 2003 investigation carried out along the proposed Highway 17 EBL alignment, approximately 50 m to the south, peat was encountered as deep as Elevation 129.0 m. Fill was encountered above the peat in one of the 2020 boreholes (CPR19-5) put down within the CPR trail alignment. A similar condition is anticipated at the WBL culvert location.

The fill and peat/organic deposits are considered unsuitable to support the culvert and retaining walls, and should be sub-excavated within the foundation footprints and replaced with OPSS.PROV 1010 Granular A, Granular B Type II, or rock fill material that is placed and compacted up to the bedding level as per OPSS.PROV 501. Rock fill must be properly chinked



prior to placement of bedding backfill. The exposed final subgrade must be inspected to confirm that the subgrade is suitable and uniformly competent.

To provide a more uniform foundation subgrade condition for the culvert and retaining wall foundations, a minimum 300 mm thick layer of well compacted bedding material conforming to OPSS.PROV 1010 Granular A requirements must be provided under the base of the foundations as per OPSS 422 and OPSS 803.010. A leveling course should be provided as per OPSS 803.010 and OPSS 422 to receive the placement of the culvert sections.

Suggested wording for an NSSP alerting the contractor to the fact that the silty clay to clayey silt subgrade may be easily disturbed when saturated and should be protected from disturbance from both construction traffic and weather has been included in Appendix I. Construction equipment should not be permitted to travel on the exposed subgrade. Protection of the subgrade should include a Class II non-woven geotextile with a maximum FOS of 150 µm (OPSS.PROV 1860). The geotextile should be placed as soon as possible after reaching the final subgrade level in accordance with OPSS.PROV 902. Alternatively, a 200mm thick mud slab could be placed as a protective layer; cellular concrete, which would provide enhanced thermal insulation to the founding soils, could also be considered.

The bedding and backfill requirements for the culvert should be consistent with Section 7 of the CHBDC, OPSS.PROV 501, and OPSS.PROV 902 and placed to the extents shown on OPSS 803.010.

Structural backfill and cover adjacent to the culvert and retaining walls should consist of OPSS Granular A meeting the specifications of OPSS.PROV 1010 and SP110S06, placed and compacted in accordance with OPSS.PROV 501.

Care must be exercised when compacting the fill adjacent to and above the culvert in order not to damage the culvert. Heavy compaction equipment used adjacent to the culvert must be restricted in accordance with OPSS.PROV 501.

It is noted that construction is expected to extend below the groundwater elevation during subgrade preparation. Excavation dewatering will likely be required to control groundwater, surface water, any perched water and precipitation runoff. Refer to Section 11.3 for additional comments on water control.

10.3 Retaining Wall Foundations

This section of the report provides foundation design recommendations for concrete cantilever and retained soil systems (RSS) walls. The magnitude of allowable post-construction wall settlement will depend on the type of wall but is anticipated to be less than that predicted to occur from construction of the up to 10 m high embankment between the existing bridge approach embankments. As such, the recommendations provided below are applicable for walls constructed following completion of the settlement under full-height preloads.

10.3.1 Shallow Foundations for Concrete Cantilever Walls

Shallow foundations for the retaining walls up to 5.6 m high at this site and can be founded directly on the native silty clay to clayey silt with a minimum 300 mm thick bedding layer as discussed in Section 10.2. Subgrade preparation should be as described in Section 10.2 and will include removal of unsuitable materials. Frost protection requirements are provided in Section 10.4. Surface water diversion and dewatering may be required to prepare the subgrade, place the bedding material and construct the foundations in the dry (Section 11.3).

Shallow footings between 1.5 m and 3.0 m in width and constructed as outlined above may be designed based on the following factored geotechnical resistances:

- Factored geotechnical resistance at ULS 250 kPa
- Factored geotechnical resistance at SLS* 125 kPa

Note (*): The SLS value provided is for settlements up to 25 mm and assumes that wall construction is carried out following completion of embankment construction and associated settlements. If project sequencing requires construction of the walls prior to embankment preloading, the SLS value must be reviewed and reassessed.

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

The factored geotechnical resistances include the following factors:

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

The bearing resistance values are for vertical, concentric loading. In the case of eccentric or inclined loading, the bearing resistance must be reduced in accordance with CHBDC Clause 6.10.3 and Clause 6.10.4. Post-construction foundation settlement, based on the supplied SLS resistance, is expected to be up to 25 mm for the applied load. Additional discussion on settlement induced by embankment construction and its mitigation is presented in Section 10.6.2.

Resistance to lateral forces/sliding resistance between the concrete and the underlying Granular A bedding fill (Section 10.2) should be evaluated in accordance with the CHBDC assuming an unfactored coefficient of friction of 0.45. A reduction factor of 0.8 (as per CHBDC Table 6.2) should be used to estimate the sliding resistance between the concrete and Granular A. An unfactored coefficient of friction of 0.35 can be assumed for the interface between the Granular 'A' and the clay. A reduction factor of 0.6 (as per CHBDC Table 6.2) should be used to estimate the sliding resistance between the Granular A and the clay subgrade.

10.3.2 RSS Walls

Retained soil systems (RSS) walls up to 5.6 m high are considered feasible at this site. The design of proprietary RSS walls is the responsibility of the supplier. Typically, such systems do not require full frost protection as they are able to tolerate some movement due to frost heave. The RSS system should be designed in accordance with the MTO RSS Design Guidelines. Once the location and height of the wall is established, recommendations will be provided concerning Performance, Appearance and Acceptance. Subgrade preparation should be as described in Section 10.2 and will include removal of peat and unsuitable materials. Frost protection requirements are provided in Section 10.4. Surface water diversion and dewatering may be required to prepare the subgrade and install the engineered fill pads in the dry (Section 11.3).

The lateral pressure comments provided Section 10.5 may be used in RSS design. Please also refer to Section 10.6.1 for comments on global stability.

A minimum 1 m thick engineered fill pad constructed on the underlying undisturbed native soils should be provided below all RSS wall as well as under the reinforced retained soil. Due to the potentially significant wall height and underlying clayey soil, the engineered fill pad is required to provide a suitable bearing capacity for the RSS walls and a similar founding elevation to that of the adjacent culvert structure to provide similar foundation performance. The engineered fill pads should consist of OPSS Granular A placed and compacted in accordance with OPSS.PROV 501. Engineered fill pads should be constructed with 1H:1V sides slopes with the crest of slope a minimum of 1 m from the edge of footing and reinforced retained soil on all sides.

RSS walls with a minimum embedment of 0.8 m and bearing on an engineered fill pad as described above may be designed based on the following factored geotechnical:

- Factored geotechnical resistance at ULS 250 kPa
- Factored geotechnical resistance at SLS* 150 kPa

Note (*): The SLS value provided is for settlements up to 25 mm and assumes that RSS wall construction is carried out following completion of the embankment construction and associated settlement. If project sequencing requires construction of the RSS wall prior to embankment preloading, the SLS value must be reviewed and reassessed.

The factored geotechnical resistances include the following factors:

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



10.4 Frost Depth

The depth of frost penetration at this site is estimated to be 1.9 m (as per OPSD 3090.101). The soils at the site are considered to be moderately frost susceptible. Shallow foundations for concrete retaining walls and the culvert should be founded at or below this depth or provided with equivalent insulation. To minimize the potential for frost impacts, sufficient frost protection along the base and walls of the culvert structure must be provided with either well-draining engineered fill to 1.9 m or insulation. Due to the relatively shallow groundwater level at the site, if the existing soils are replaced with engineered fill, drains must be provided at the base of well-draining engineered fill.

Should insulation be used beneath the culvert or other structures, the grade of the insulation material must be selected such that it does not creep under the anticipated loading. The thickness of the insulation will be determined by the geometry but will likely be in the order of 75 mm.

It is not necessary to found RSS walls at a depth below frost penetration.

Please refer to the pavement design report for frost taper recommendations for the pavement.

10.5 Backfill and Lateral Earth Pressures

Structural backfill material should consist of Granular A or Granular B Type II meeting the OPSS.PROV 1010 and SP110S06 specifications. Large scale direct shear box testing on samples of Granular A and Granular B Type II from numerous nearby aggregate sources was completed for this project. The results indicate that for design of structural backfill for this project, an internal angle of friction of 42 and 40 degrees can be used respectively for Granular B Type II and quarry- sourced Granular A in this area provided the effective vertical pressure on the material is less than 150 kPa (Geocres Memorandum 31F-213). An Operational Constraint will be required in the contract restricting the source of Granular A to quarries. Throughout this report, the term "Granular A" is defined as "Quarry-Source Granular A" unless specifically described as "Pit-Source Granular A".

The backfill must be in accordance with OPSS.PROV 902 and placed to the extents shown on OPSD 803.010 for the culvert and OPSD 3101.150 for the retaining walls. Structural backfill should consist of Granular A or Granular B Type II placed and compacted in accordance with OPSS.PROV 501. Heavy compaction equipment used adjacent to the walls must be restricted in accordance with OPSS.PROV 501.07.02. The design of the retaining walls, where required, must incorporate a subdrain as shown in OPSD 3101.150.

Lateral earth pressure parameters provided in Table 10-1 and Table 10-2. in the sections below are based on the assumptions that the wall is vertical and the backfill is fully drained so that there are no unbalanced hydrostatic pressures above the permanent groundwater level. If adequate drainage cannot be confirmed, the potential for buildup of hydrostatic pressures should be considered in design. Where back slopes are horizontal or 2H:1V, the corresponding coefficients provided in Table 10-1 and Table 10-2 should be used. For other backfill and wall geometries, the

appropriate earth pressure coefficients will need to be calculated once the final geometry is confirmed.

10.5.1 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 10-1.

Table 10-1 Static Earth Pressure Coefficients

Condition	OPSS Granular B Type II $\phi = 42^\circ, \gamma = 22.8 \text{ kN/m}^3$		Quarry Sourced OPSS Granular A $\phi = 40^\circ, \gamma = 22.8 \text{ kN/m}^3$		Pit Sourced OPSS Granular A $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	2H:1V Slope Behind Wall	Horizontal Surface Behind Wall	2H:1V Slope Behind Wall	Horizontal Surface Behind Wall	2H:1V Slope Behind Wall
Coefficient of at Rest Earth Pressure, K_o (Restrained Wall)	0.33	0.48	0.36	0.52	0.43	0.62
Coefficient of Active Earth Pressure, K_A (Unrestrained Wall)	0.20	0.26	0.22	0.30	0.27	0.39

The parameters in the table correspond to full mobilization of active 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 any wingwalls or unrestrained walls. For rigid structures, at-rest horizontal earth pressures would apply for design.

10.5.2 Combined Static and Seismic Lateral Earth Pressure

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

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

The coefficients of horizontal earth pressure for combined static and seismic loading presented in Table 10-2 may be used. The provided earth pressure coefficients are based on a Seismic Site Class C and a PGA with a 2% probability of exceedance in 50 years of 0.23 g (Geological Survey of Canada – Fifth Generation) and a $F(PGA)$ of 1.14 as per Table 4.8 of the CHBDC.

Table 10-2 Combined Static and Seismic Earth Pressure Coefficients

Condition	OPSS Granular B Type II $\phi = 42^\circ, \gamma = 22.8 \text{ kN/m}^3$		Quarry Sourced OPSS Granular A $\phi = 40^\circ, \gamma = 22.8 \text{ kN/m}^3$		Pit Sourced OPSS Granular A $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	2H:1V Slope Behind Wall	Horizontal Surface Behind Wall	2H:1V Slope Behind Wall	Horizontal Surface Behind Wall	2H:1V Slope Behind Wall
Coefficient of Active Earth Pressure, K_{AE} (Restrained Wall)	0.32	0.56	0.35	0.63	0.41	0.88
Coefficient of Active Earth Pressure, K_{AE} (Unrestrained Wall)	0.25	0.37	0.28	0.42	0.33	0.58

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

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

where:

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

10.6 Embankment Fill

The embankments should be constructed in accordance with OPSS.PROV 206. Local marine or any high plastic clay must not be used as embankment fill. Any peat, organics, and any deleterious material should be removed from within the embankment footprint prior to placement of embankment fill. Where the new embankment fill will abut the existing approach embankments, the existing slopes should be benched in accordance with OPSD 208.010 prior to placement and compaction of the new fill.

10.6.1 Embankment Stability

Slope stability assessment of the existing and proposed embankments was carried out using GeoStudio 2020 Slope/W software for limit equilibrium analysis. Input parameters for the embankment fill and foundation soils for the analysis are based on the SPT N-values, shear vane test results, observations in the field, and the results of laboratory testing.

Table 6.2 of Section 6.9.1 of the CHBDC requires minimum Factors of Safety of 1.5 and 1.3 for embankments in permanent and temporary static conditions, respectively, for a typical degree of understanding and a Ψ of 1.0.

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

In addition, Sections 6.14.2.1 and 6.14.2.3 of the CHBDC present performance criteria requirements for Major Route geotechnical systems (embankments) inside and outside the structure interface zone, respectively. Based on Clause 6.14.2.2, the structure interface zone at this site extends to 20 m behind the structural culvert (based on a fill height of up to about 10 m). The performance criteria for the Major Route embankments are as follows:

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



The stability analyses considered site-adjusted (Site Class C) design PGA values of 0.23 g and 0.08 g for ground motions with return periods of 2,475 and 475 years, respectively, as per Section 4.4.3.2 of the CHBDC.

Based on the available general arrangement drawing (Appendix G), the proposed grade of the eastbound travelled lanes is to be maintained at up to about Elevation 139.9 m, requiring embankment fill in between the existing abutments/embankments up to about 10 m high above the existing ground surface, which is at about Elevation 130 m. The existing Highway 17 bridge embankments are sloped at about 2H:1V and extend horizontally up to about 20 m near the existing abutments.

Slope stability assessments of the north embankment slopes have been carried out considering two different embankment materials: Select Subgrade Material (SSM) and Compacted Rock Fill. It is noted that fill geometry for new embankment slopes must include a 2 m wide bench for earth or granular fills equal to or greater than 8 m in height while rock fill must incorporate a 2 m wide bench for fills equal to or greater than 10 m in height.

Embankment slope stability was evaluated using GeoStudio 2020 Slope/W software for limit equilibrium analysis. Input parameters for the analysis are based on the SPT N values and the results of laboratory testing. The following additional parameters were used in the analysis:

- Estimated soil stratigraphy is based on the nearest boreholes.
- Embankment maximum fill height of 10.0 m was considered.
- Side slopes of 2H:1V for SSM fill and 1.25H:1V for rock fill were modelled. It has been assumed that the side slopes of the new embankment will be constructed with a similar inclination to the existing 2H:1V embankment slopes.
- Mid-height 2 m wide benches were used for SSM and rock fill slopes greater than 8 m and 10 m in height, respectively.
- Retained soil at RSS walls are to consist of OPSS Granular B Type II, up to 7.2 m in width (depending on embankment fill material) and supported on a 1 m thick Granular A bedding layer.
- Site-adjusted horizontal PGA values of 0.11 g and 0.04 g, equal to $\frac{1}{2}$ of the site-adjusted horizontal PGA values were used for the 2,475-year and 475-year seismic analyses, respectively, as per Section 4.4.3.3, of the CHBDC and outlined in Sections 8.1 and 8.2 above.
- A traffic surcharge of 17 kPa has been applied as a temporary load.

Copies of the output from the stability analyses are provided on the figures presented in Appendix H. Each output figure shows the slope geometry, groundwater conditions, soil stratigraphy and soil strength parameters utilized in the analysis. The stability analyses generated the factor of safety values presented below in Table 10-3 and Table 10-4.

The outputs from the stability analyses of the *existing* embankment side slopes are presented in Figures H1.1 to H1.4. The results of the static design analyses (temporary/traffic and permanent conditions) do not strictly meet the target Factors of Safety described above. It should be noted

that the analyses assumed that the existing embankments comprise SSM fill since the available historical subsurface information was collected prior to construction of the existing embankments and information on the composition or condition of the existing embankment material was not available. The existing embankment side slopes did not show any visible signs of distress at the time of the 2020 field investigation. If the Factors of Safety for the existing embankment slopes are unacceptable, more detailed analyses of their stability should be carried out based on the results of a supplementary field investigation that includes boreholes put down through the existing embankments.

Table 10-3 Slope Stability Analysis Results – North Embankment Side Slopes

Condition	Case	Factor of Safety		
		Select Subgrade Material 2H:1V (no bench)	Select Subgrade Material 2H:1V (with bench)	Rock Fill 1.25H:1V
Permanent	Long Term (Drained)	1.4 (Fig H1.1)	1.6 (Fig H2.1)	1.5 (Fig H3.1)
Temporary (traffic loading)	Short Term (Undrained)	1.4 (Fig H1.2)	1.6 (Fig H2.2)	1.6 (Fig H3.2)
Temporary (seismic loading)	Pseudo-Static (Undrained) 2,475-year	1.1 (Fig H1.3)	1.2 (Fig H2.3)	1.3 (Fig H3.3)
	Pseudo-Static (Undrained) 475-year	1.3 (Fig H1.4)	1.4 (Fig H2.4)	n/a

Table 10-4 Slope Stability Analysis Results – RSS Retaining Walls

Condition	Case	Factor of Safety	
		RSS Wall [SSM]* 2H:1V	RSS Wall [Rock Fill]* 1.25H:1V
Permanent	Long Term (Drained)	1.5 (Fig H4.1)	1.5 (Fig H5.1)
Temporary (traffic loading)	Short Term (Undrained)	1.8 (Fig H4.2)	1.9 (Fig H5.2)
Temporary (seismic loading)	Pseudo-Static (Undrained) 2,475-year	1.5 (Fig H4.3)	1.6 (Fig H5.3)
	Pseudo-Static (Undrained) 475-year	n/a	n/a

* Long-term stability was governed by the dimensions of the RSS wall; see analysis results

The results of the other static design analyses (temporary/traffic and permanent conditions) presented above for the new embankment construction (Figures H2-1 to H5.3) meet or exceed the target Factors of Safety. The results of the associated seismic analyses meet or exceed the target Factor of Safety for seismic design for the 2475-year seismic event, yielding factors of safety values equal to or greater than 1.3, with the exception of the existing embankment (Figure H1.3) and the embankment constructed with SSM fill and 2H:1V benched side slopes

(Figure H2.3), which yielded factors of safety of 1.1 and 1.2, respectively. Additional analyses were carried out for those cases to determine if performance criteria would be met for the Major Route geotechnical systems inside and outside the structure interface zone. Pseudo-static analyses considering the 475-year earthquake event were completed and yielded factors of safety of 1.3 and 1.4 for those cases (Figure H1.4 and Figure H2.4, respectively) indicating that the performance requirements would be met for those scenarios.

Slope protection and drainage measures will be required to ensure the long-term surficial stability of the embankments. Normally slope vegetation should be established as soon as possible after completion of embankment construction to control surficial erosion in general accordance with OPSS.PROV 804.

10.6.2 Embankment Settlement

Construction of the new embankments will require placement of embankment fill up to about 10.0 m high at the culvert crossing. The loading imposed from the new fill will increase the effective stress in underlying soil deposits and induce settlement in the compressible clayey silt layer. Settlement analyses were carried out using the software Settle3 (Version 5) by Rocscience.

In accordance with MTO's document "Embankment Settlement Criteria for Design" (March 2, 2010), the criteria adopted for embankment design at this site is as follows:

Table 10-5. Summary of MTO Settlement Criteria

Distance from Abutment	0-20 m	20-50 m	50-75 m	>75 m	Post Construction Settlement Period
Settlement Limits Freeway	25 mm	50 mm	75 mm	100 mm	20 years

A representative site stratigraphy was developed based on the available Record of Borehole logs with material properties based on the results of the laboratory testing from the 1971 and 2020 investigations. The design stratigraphy considered material parameters of the weathered clayey silt crust based on the consolidation test results obtained from a sample from Borehole CPR19-6, and material parameters of the underlying clayey silt based on the consolidation test results obtained from samples from Borehole BH 7 and the results of the 2020 investigation. The soil parameters used in the model are summarized in Table 10-6.

Settlement analyses were carried out to calculate the predicted settlement with time, considering a 10.0 m thick SSM embankment, with benched sides slopes inclined at 2H:1V, and a unit weight of 21 k/m³, constructed 50 years after the existing bridge approach embankments. The results of the analyses indicated that the soils beneath the new embankment would settle as much as 160 mm, including about 50 mm of elastic settlement in the glacial till, and the remaining consolidation settlement in the clayey silt. The elastic settlement in the glacial till would occur almost immediately, and the consolidation settlement in the clayey silt would occur over time, with 90% of the total settlement occurring within about 7 months, and approximately 15 mm thereafter.

Table 10-6 Summary of Material Parameters

Soil Type	Thickness (m)	Unit Weight (kN/m ³)	Settlement Parameters				
			P _c ' (kPa)	C _c	C _r	C _v (mm ² /s)	C _{vr} (mm ² /s)
Clayey Silt (Crust)	2.0 – 2.2	20	270	0.17	0.022	0.2	0.8
Clayey Silt	1.5 – 1.7	20	130	0.16	0.022	0.2	0.6
Silty Sand (Till)	11.3	22	E _s = 30 to 100 MPa				

In addition to the settlement described above, there will be self-settlement of the embankment material itself. For embankments constructed with compacted rock fill the short-term settlement will be approximately 130 mm (up to 1 year after completion of construction with 90% of this value occurring in the first six months). In addition, rock fill embankments will continue to settle after the first year with an estimate of an additional 15 mm. Similarly, an embankment constructed of SSM material will undergo approximately 55 to 100 mm of self settlement with the majority of that complete within the one year of completion of construction, with about 90% occurring in the first 6 months. Embankments must be overbuilt to compensate for the estimated settlement. Depending on embankment fill material, the post-preload settlement magnitude may exceed the values summarized in Table 10-5. As such, the need for delaying final paving following regrading, based on settlement results, should be considered.

Full height preloads (embankment pre-construction) could be applied for a duration of 7 months to ensure that post-construction settlement meets the above guidelines. The preload material would then be excavated to construct the permanent trail culvert. It is noted that construction of the culvert following placement of the full height of embankment preload would necessitate excavation depths on the order of 11 to 12 m which would result in significant excavation footprints or temporary shoring of the excavation walls. Consideration could be given to lowering the design profile of the westbound lanes to minimize the new embankment height, associated settlements, and required excavations for culvert installation.

Alternatively, if the culvert is constructed prior to placement of the new embankment fill, it should be designed with a camber or similar structural detail to accommodate the anticipated settlement.

If more rapid settlement is required to meet a construction schedule that cannot accommodate the required preload period, consideration may be given to alternative treatment methods. Surcharging the area and/or installing wick drains would increase the rate of compression settlement and, therefore, reduce the time required for treatment. Construction of the embankments with lightweight fill would reduce the additional load and, therefore, the anticipated total settlement of the embankments.

If preloading or surcharging is selected as the preferred treatment methodology, a settlement monitoring program should be incorporated into the Contract to allow monitoring of the actual rate and magnitude of the embankment settlement and determination of the end of treatment. The settlement monitoring program should consist of suitable instrumentation installed at regular intervals within the footprint of the preload/surcharge fill to allow regular monitoring of the ground movement throughout the treatment period.

As discussed in Section 10.3, concrete cantilever or RSS walls may not be able to accommodate the magnitude of settlements described above and, as such, should only be constructed following completion of the settlement from the preloaded embankments. Consideration should be given to the following options to reduce the impacts of embankment settlement on construction sequencing:

- Lower the design embankment height and overall WBL grade.
- Eliminate the requirement for retaining walls by extending the WBL culvert north (to the embankment slope toe).
- Construct the culvert with a camber, prior to placement of embankment fill, such that it can accommodate the predicted settlement.

Nonetheless, accommodation of some embankment settlement will be required and challenges with construction sequencing of the retaining walls must be considered, particularly within the median.

10.7 Cement Type and Corrosion Potential

Chemical analysis for determination of pH, water soluble sulphate, sulphides, chloride concentrations, resistivity and electrical conductivity was carried out on samples of the native materials collected as part of the 2020 investigation at the proposed EBL culvert, reported under separate cover (Geocres No. 31F-241). The analysis results from that investigation are summarized in Section 5.6, above.

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. The test results provided in Table 5-3 were compared with Table 3.2 of the MTO Gravity Pipe Design Guideline and indicates a very low to moderately corrosive environment. The concentration of soluble sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with the soil and groundwater at the site. The sulphate results in Table 5-3 were compared with Table 3 of Canadian Standards Association Standards A23.1-19 (CSA A23.1) and indicate a low degree of sulphate attack potential on concrete structures at this site.

The test results provided in Section 5.6 may be used as a general indication of anticipated conditions at the site; however, additional testing of samples collected from boreholes put down at the WBL culvert site should be carried out by the Preferred Proponent to confirm the selection of coatings and corrosion protection systems for buried steel objects, if required, and to assess the degree of sulphate attack potential on concrete structures.



The corrosive effects of road de-icing salts should also be considered in the design and selection of structural materials.

11 CONSTRUCTION CONSIDERATIONS

11.1 Temporary Excavations

All temporary excavations must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of OHSA, the existing fills below the groundwater table and organic deposits at the site may be classified as Type 4 soil. The native stiff clayey silt to clay may be classified as Type 3 soil. Side slopes for excavations through more than one soil type must be entirely based on the highest soil type number. Unsupported excavations in Type 4 soil must have side slopes no steeper than 3H:1V from the base of the excavation. Unsupported excavations made in Type 3 soils must have side slopes no steeper than 1H:1V from the base of the excavation. For deep excavations that may be required at this site to install the culvert through pre-constructed preload embankment fill, additional slope stability analyses must be carried out to assess safe temporary side slopes. Alternatively, shoring should be used for deep excavations.

Excavations for the culvert and retaining wall foundations must be carried out in accordance with OPSS.PROV 902 and will extend through the existing fill and organics and into the underlying native clayey silt deposit. Selection of the equipment and methodology to excavate and prepare the founding surface is the responsibility of the Contractor. Stockpiling or surface surcharge should not be allowed on the embankment or side slopes.

The management and disposal of excess material shall be in accordance with OPSS.PROV 180.

At locations where there are space restrictions or where a slope must be retained, the excavations will need to be carried out within a protection system. Further discussion on temporary protection systems (TPS) is presented in Section 11.2 below.

11.2 Temporary Protection Systems

Should temporary protection systems be required for excavation support or groundwater control, they must be implemented in accordance with OPSS.PROV 539 and designed for Performance Level 2. A temporary protection system consisting of either a soldier pile and lagging system or an interlocking sheet pile system would be feasible to install at this site. An interlocking sheet pile system could also aid in groundwater control, as described further in Section 11.3 below. The soldier pile and lagging or sheet piling system would have to be driven to a sufficient depth to provide the necessary passive resistance for the retained soil height. The presence of cobbles and boulders in the glacial till may impede the penetration of sheet piles and should be considered in the design of the temporary protection system. Ground anchors or struts may be required as part of the temporary protection system for deep excavations.

As part of the 2020 investigation carried out at the proposed EBL culvert site (Geocres No. 31F-241), groundwater levels were measured within the overburden to be within the upper 0.8 m below the existing ground surface. The well installed in bedrock at Borehole CPR19-3 indicated



measured groundwater levels ranging from 0.5 m below the existing ground surface to 0.3 m above (up to Elevation 130.3 m, see Section 5.5). Driving of soldier piles or sheet piles for construction of temporary protection systems are not anticipated to require penetration into the bedrock. If nominal artesian conditions similar to those described above are encountered in the lower layers of overburden, the overlying cohesive clayey silt deposit is expected to reduce the potential for piping of groundwater to the surface. However, the design of temporary protection systems should consider the potential for nominal artesian conditions with groundwater levels at/near the existing ground surface.

The actual pressure distribution acting on the shoring system is a function of the construction sequence and the relative flexibility of the wall, and these factors must be considered when designing the shoring system. The protection system should be installed at a suitable distance away from the new structure to limit the disturbance to subgrade associated with removal of the protection system following completing of construction. Alternatively, the protection system near the structures could be left in place and cut off in accordance with OPSS.PROV 903 to limit the disturbance of subgrade during removal of the temporary protection system.

Lateral earth pressure coefficients, under fully mobilized conditions, that can be used in design for the structural backfill are provided in Table 10-1.

The lateral earth pressure coefficients for the underlying native soils are given below for a vertical wall and a horizontal backslope:

SSM Fill:

$$\begin{aligned}\gamma &= 21 \text{ (kN/m}^3 \text{ bulk unit weight of soil, to be adjusted below water)} \\ K_A &= 0.31 \\ K_P &= 3.3\end{aligned}$$

Rock Fill:

$$\begin{aligned}\gamma &= 20 \text{ (kN/m}^3 \text{ bulk unit weight of soil, to be adjusted below water)} \\ K_A &= 0.20 \\ K_P &= 5.0\end{aligned}$$

Native clayey silt:

$$\begin{aligned}\gamma &= 20 \text{ (kN/m}^3 \text{ bulk unit weight of soil, to be adjusted below water)} \\ K_A &= 0.36 \\ K_P &= 2.8 \\ S_u &= 50 \text{ kPa}\end{aligned}$$

Native glacial till:

$$\begin{aligned}\gamma &= 22 \text{ (kN/m}^3 \text{ bulk unit weight of soil, to be adjusted below water)} \\ K_A &= 0.27 \\ K_P &= 3.7\end{aligned}$$



Submerged unit weight should be used below the groundwater level. If the backslope behind, or if the ground surface in front of the temporary protection systems is not horizontal, the lateral earth pressure parameters provided above do not apply and recalculation of the earth pressure parameters will be required.

The design of roadway protection is the responsibility of the Contractor. All protection systems should be designed by a licensed Professional Engineer experienced in such designs and retained by the Contractor. The design of the roadway protection system must incorporate traffic loading and surcharge loading due to construction equipment and operations.

It is recommended that an NSSP be included in the tender documents to alert the Contractor to the potential for cobbles and boulders and obstructions within the glacial till as well as the potential for nominal artesian conditions at this site. Suggested wording for the NSSP has been included in Appendix I.

11.3 Surface and Groundwater Control

Culvert and retaining wall construction, subgrade preparation and placement and compaction of granular bedding must be carried out in the dry. The natural groundwater level is at about the existing ground surface (approximate Elevation 130 m). The excavations required to construct the culvert are expected to be at or below this elevation.

The Contractor must control groundwater, perched groundwater, and surface water flow at the site to permit the construction in a dry and stable excavation. Depending on the size of the excavation, groundwater may be controlled with sump pumps in the bottom of the excavation or, for larger excavations, groundwater control may also require sheet piling or similar to further control groundwater infiltration into the excavation. The groundwater level within the work zone should be lowered by pumping from sumps to a minimum of 0.5 m below the underside of the planned excavation base prior to each stage of excavation. Given the presence of granular materials beneath a less permeable clay to clayey silt of limited thickness combined with a high groundwater elevation, there is a risk of basal heave of excavations at this site. Depending on the depth of excavation and the construction methodology selected by the contractor, depressurizing the lower granular till may be required.

The design of dewatering systems is the responsibility of the Contractor. The Contract Documents must alert the Contractor to this responsibility and to design the system in accordance with SP No. FOUN0003 which amends OPSS 902. The Designer Fill-In SP No. FOUN0003 should be a minimum of 100 metres from the dewatering system to monitor the impact of the groundwater control system on adjacent properties and structures.

The water level will fluctuate and the minimum groundwater elevation for the site at the time of the excavation should be taken as the expected high water level defined in SP No. FOUN0003.

Further assessment of dewatering requirements and the need for a Permit to Take Water (PTTW) should be carried out by specialists experienced in this field.



It is noted that a Hydrogeological Investigation and Design Report is under preparation for the Highway 17 Twinning Project. Please refer to that document for additional discussion on dewatering with respect to this assignment.

11.4 Erosion Control

The Contractor should provide silt fences and erosion control blankets as per OPSS.PROV 805 throughout the duration of construction to prevent transport of silt/sediment.

Slope protection and drainage measures will be required to ensure the long-term surficial stability of the embankment slopes. Slope vegetation should be established as soon as possible after completion of the embankment fills to limit surficial erosion. A vegetation cover should be established on all exposed earth surfaces to protect against surficial erosion in general accordance with OPSS.PROV 804.

12 CONSTRUCTION CONCERNS

The likely construction methodology includes open cut excavations for the construction of the new WBL culvert. Potential construction concerns include, but are not necessarily limited to:

- Only historical subsurface information at the proposed WBL culvert site was available at the time of preparation of this report. Additional subsurface information should be collected along the proposed culvert alignment and through the existing embankments at a subsequent design stage. Chemical analysis for determination of pH, water soluble sulphate, sulphides, chloride concentrations, resistivity and electrical conductivity should be carried out by the Preferred Proponent on samples of the native materials collected as part of the investigation to assess cement type and corrosion potential.
- Mitigation of the settlement induced by the new westbound embankment will require preload or a structure designed to accommodate the movements. If preloading is employed, an instrumentation and monitoring program will be required to assess the progress of the preload. Given the limited project length, the monitoring program would include approximately six settlement rods located on the westbound alignment with a nominal spacing of 25 m. The base plates should be installed prior to fill placement and the rods will require extension as fill is placed around them. The top of the settlement rods should be surveyed every week during preload construction, then every two weeks for four months, then every month for the duration of the anticipated preload period (see Section 10.6.2). The installation of the monitoring equipment and surveying would typically be carried out by the Contractor, with the results evaluated by the Contract Administration team.
- Construction may extend below the groundwater level. An adequate and effective surface water management and dewatering plan must be implemented to construct the culvert and retaining wall foundations in the dry.



- The native soil which will be exposed beneath culvert bedding layers or retaining wall spread footings is sensitive and readily disturbed. A suggested Notice to Contractor is provided in Appendix I.
- The culvert must be protected from frost with removal of frost-susceptible soil down to 1.9 m depth and replacement with well-drained non-frost-susceptible granular fill or suitably insulated.
- The Contractor's selection of construction equipment and methodology must include assessment of the capability of the existing soils to support the proposed construction equipment and supplies.
- Obstructions could be encountered in the existing embankment fill and may limit choice of equipment and methods. A suggested Notice to Contractor is provided in Appendix I.
- Excavation through significant embankment thickness will be required for culvert installation if carried out following embankment preload construction.

The successful performance of the structure installations will depend largely upon good workmanship and quality control during construction. Observation of the excavation and backfilling operations will be required during construction as per OPSS.PROV 902 to confirm that the foundation recommendations are correctly implemented, and material specifications are met.



13 CLOSURE

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

Thurber Engineering Ltd.
Report Prepared By:



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

Dr. Fred Griffiths, P.Eng.
Senior Geotechnical Engineer,
Senior Associate



Dr. P.K. Chatterji, P.Eng.
MTO Review Principal,
Senior Geotechnical Engineer



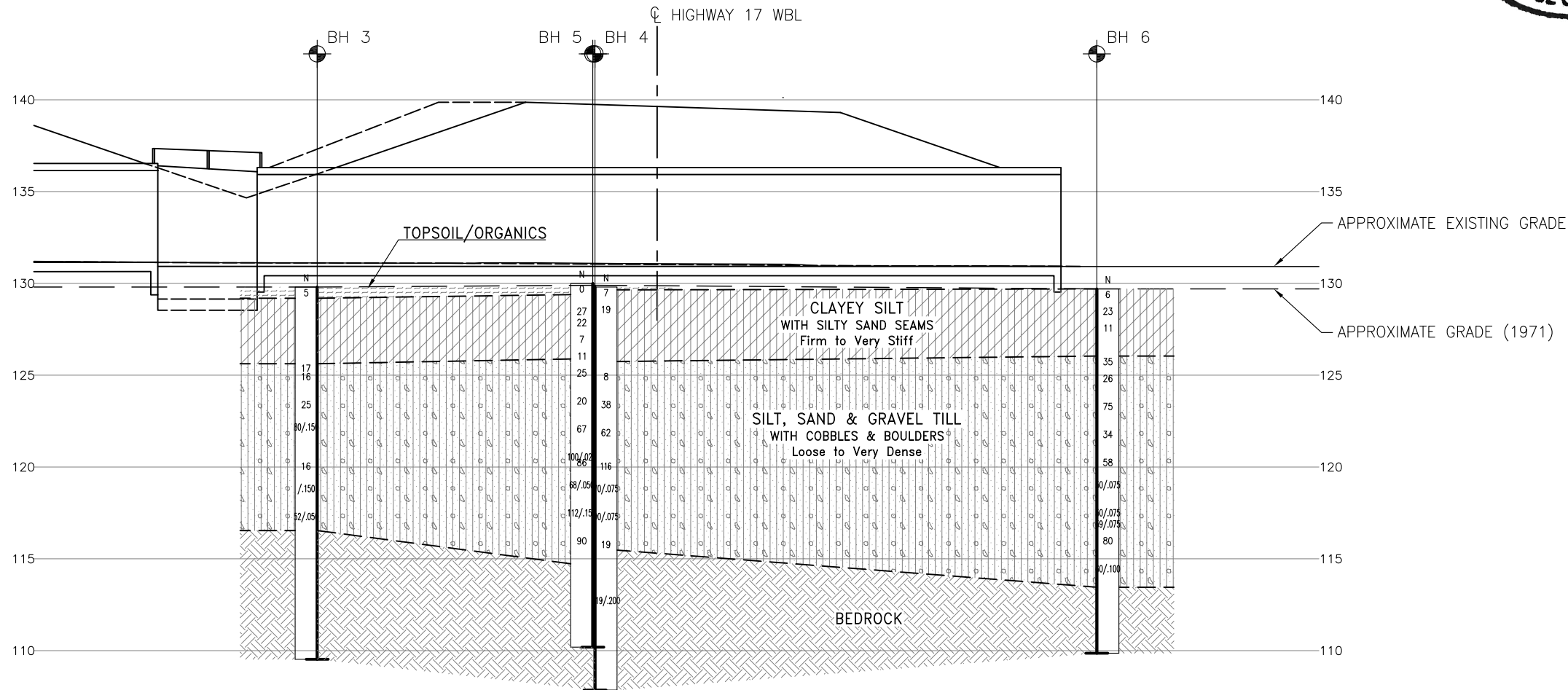
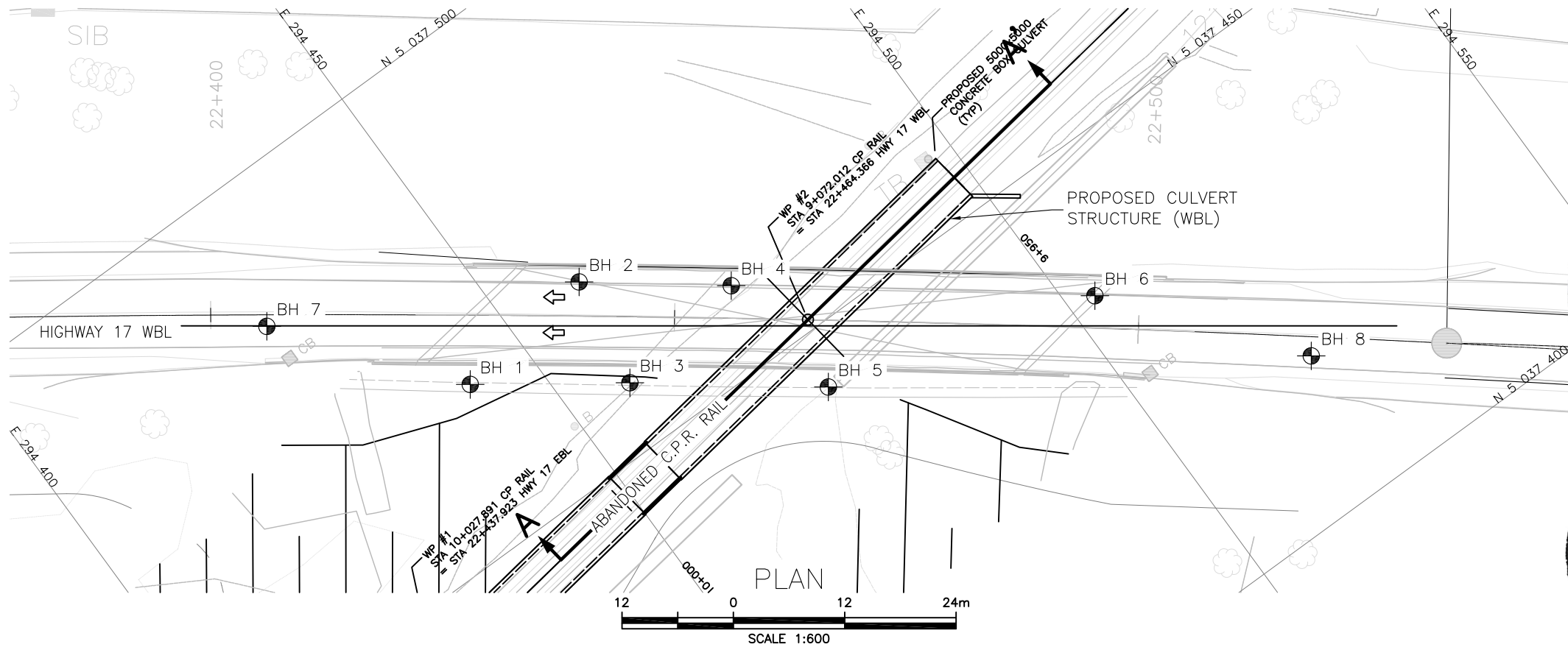
REFERENCES

- ⁱ Boulanger, R. W. and Idriss, I. M. (2007). Evaluation of cyclic softening in silts and clays, ASCE, Journal of Geotechnical and Geoenvironmental Engineering, 133(6), 641-652.
- ⁱⁱ Boulanger, R. W., and Idriss, I. M. (2014). CPT and SPT based liquefaction triggering procedures, Report No. UCD/CGM-14/01, Center for Geotechnical Modeling, Department of Civil and Environmental Engineering, University of California, Davis, CA, 134 pp.

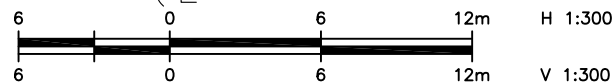


Appendix A.

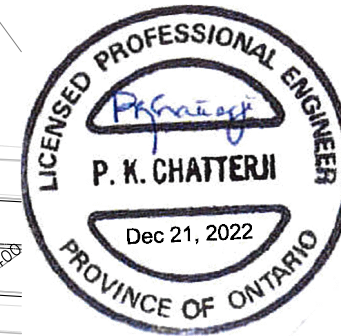
Borehole Location Plan and Stratigraphic Drawings



SECTION A-A' (CL PROPOSED CULVERT)



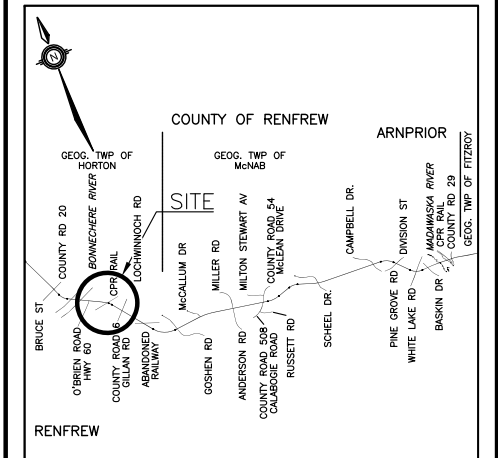
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



CONT No
WP No 4068-09-00

HIGHWAY 17 TWINNING
C.P.R. TRAIL
WBL STRUCTURE
BOREHOLE LOCATIONS AND SOIL STRATA

Ontario



KEYPLAN

LEGEND

- Borehole
- Borehole (1972 Investigation)
- Blows /0.3m (Std Pen Test, 475J/blow)
- Blows /0.3m (60' Cone, 475J/blow)
- CONE
- PH
- Pressure, Hydraulic
- Water Level
- Head Artesian Water
- Piezometer
- 90%
- Rock Quality Designation (RQD)
- A/R
- Auger Refusal

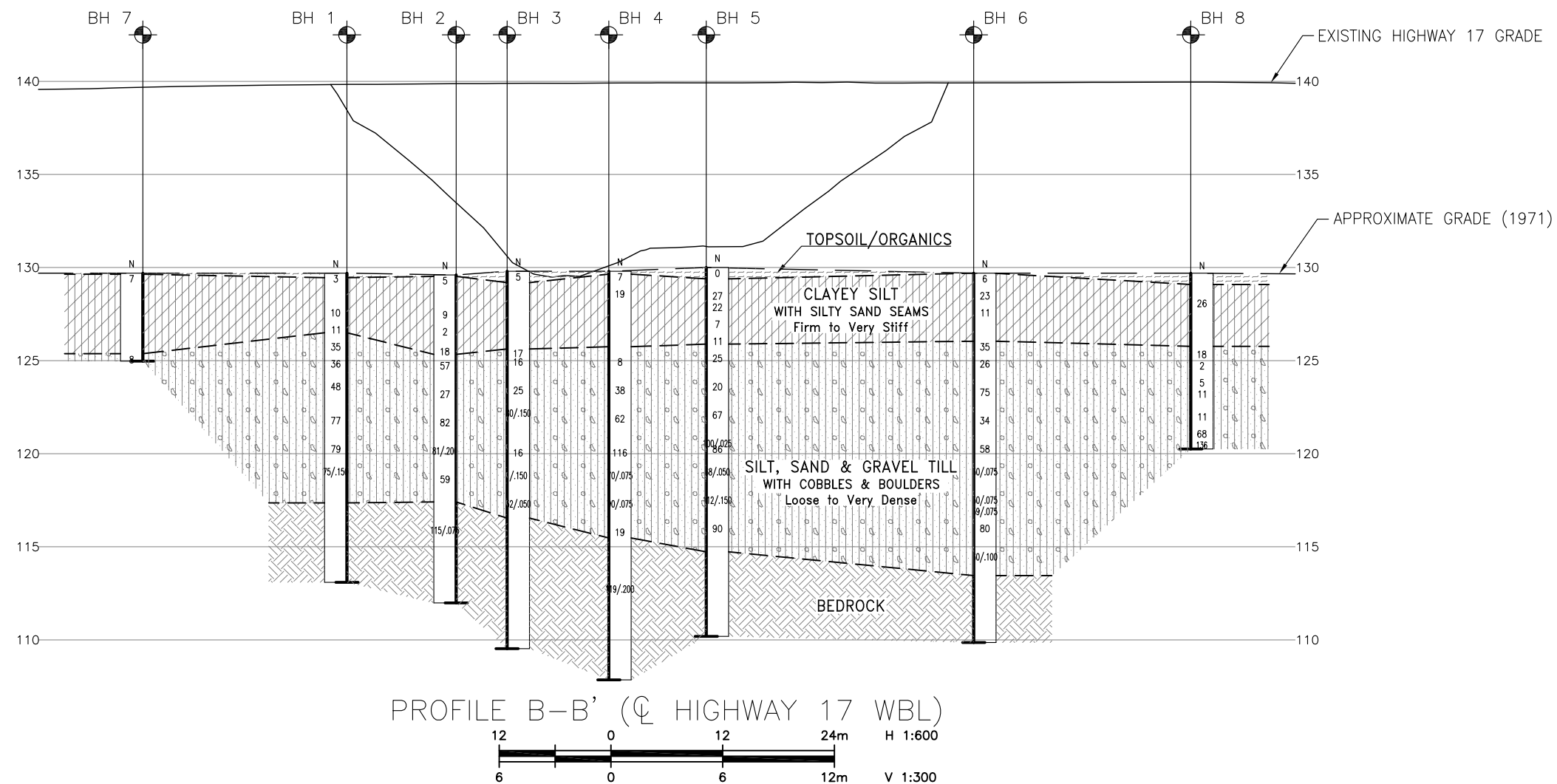
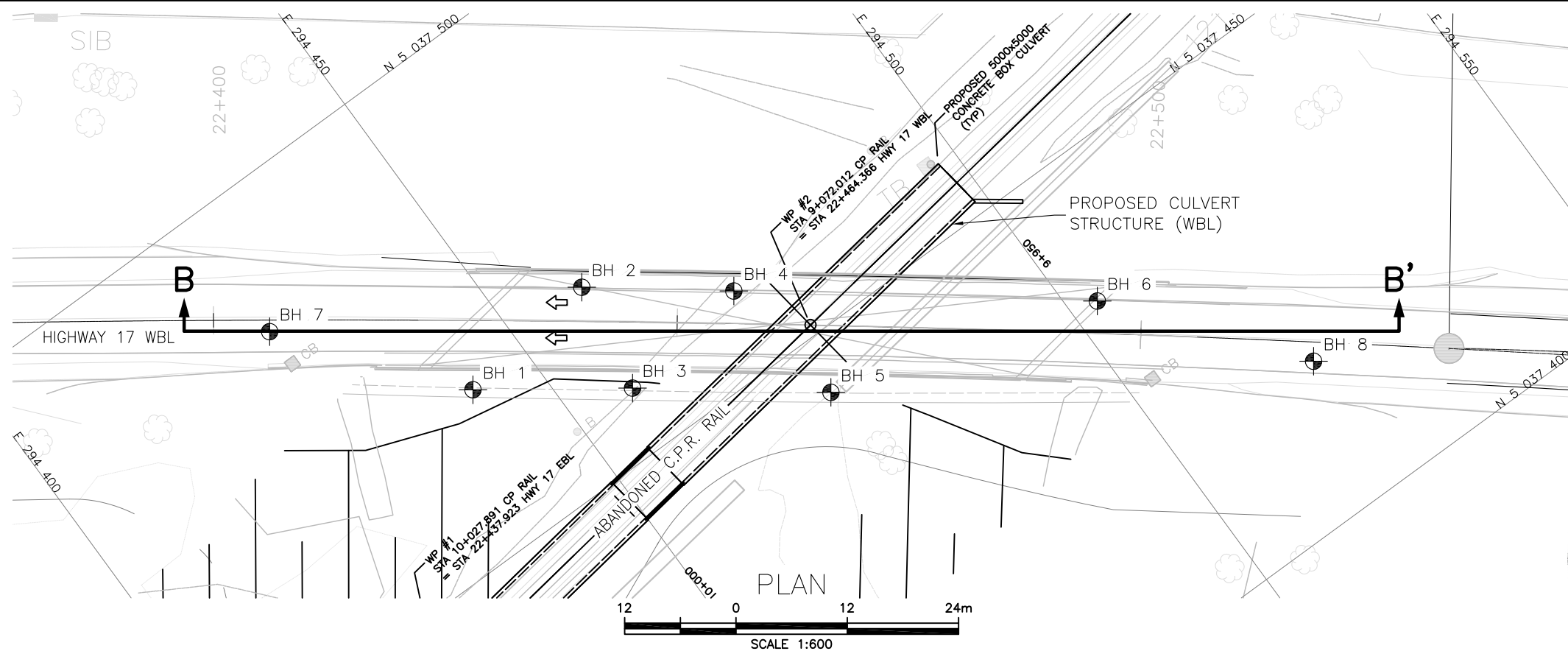
NO	ELEVATION	NORTHING	EASTING
BH 1	129.7	5 037 467.0	294 443.0
BH 2	129.6	5 037 469.0	294 459.0
BH 3	129.8	5 037 457.0	294 457.0
BH 4	129.8	5 037 459.0	294 472.0
BH 5	130.0	5 037 444.0	294 474.0
BH 6	129.7	5 037 435.0	294 503.0
BH 7	129.7	5 037 485.0	294 429.0
BH 8	129.7	5 037 416.0	294 518.0

-NOTES-

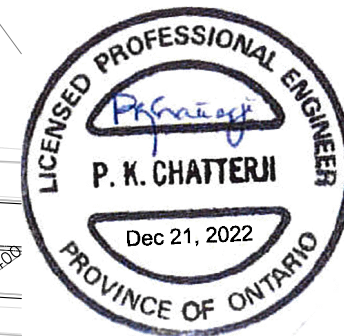
- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- This drawing is for subsurface information only. Structural elements, surface details and features are for conceptual illustration.
- Coordinate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

GEOCRES No. 31F-240

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	JG	CHK MJK	CODE
DRAWN	MFA	CHK FG	SITE
LOAD	DATE	DEC 2022	
STRUCT	DWG	1	



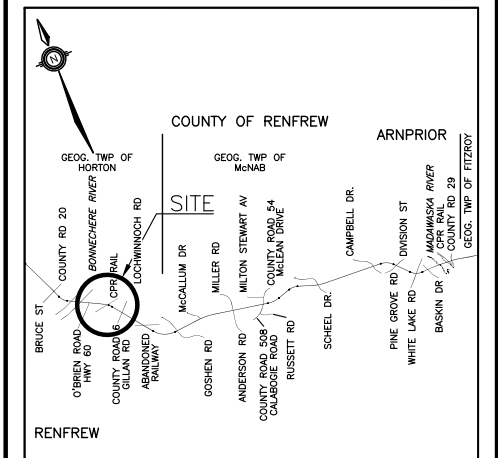
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



CONT No
WP No 4068-09-00






HIGHWAY 17 TWINNING
C.P.R. TRAIL
WBL STRUCTURE
BOREHOLE LOCATIONS AND SOIL STRATA

Ontario 



KEYPLAN

LEGEND

	Borehole
	Borehole (1972 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
BH 1	129.7	5 037 467.0	294 443.0
BH 2	129.6	5 037 469.0	294 459.0
BH 3	129.8	5 037 457.0	294 457.0
BH 4	129.8	5 037 459.0	294 472.0
BH 5	130.0	5 037 444.0	294 474.0
BH 6	129.7	5 037 435.0	294 503.0
BH 7	129.7	5 037 485.0	294 429.0
BH 8	129.7	5 037 416.0	294 518.0

-NOTES-

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

GEOCRES No. 31F-240

REVISIONS									
	DATE	BY	DESCRIPTION						
DESIGN	JG		CHK	MJK		LOAD		DATE	DEC 2022
DRAWN	MFA		CHK	FG	SITE	STRUCT	DWG	2	



Appendix B.

Record of Borehole Sheets

FOUNDATION SECTION

ORIGINATED BY WE

COMPILED BY SO

CHECKED BY MA

[illegible]

FOUNDATION SECTION

ORIGINATED BY WH

COMPILED BY SO

CHECKED BY *E. D.*

SOIL PROFILE		SAMPLES	DYNAMIC PENETRATION BLOWS / FOOT	RESISTANCE PS.F.	LIQUID LIMIT ——— w _L PLASTIC LIMIT ——— w _p WATER CONTENT ——— w	BULK DENSITY	REMARKS
ELEV. DEPTH	DESCRIPTION	NUMBER TYPE BLOWS/FOOT				Y	
425.3	Ground Level						
0.0	Topsoil	1 SS 6					
	Clayey silt	2 SS 8					
	Silty sand seams up to 2" thick below el. 420.	3 SS 9					
	Firm to Very Stiff Grey	4 SS 2					
411.3		5 SS 7					
41.0	Glacial Till	6 SS 18					
	Het. mix. of silt, sand & gravel, trace of clay.	7 SS 57					
	Boulders up to 7" in size throughout	8 RC 22%					
	Compact to Very Dense	9 SS 27					
		10 SS 52					
		11 SS 59					
385.3	Boulder	12 RC 93%					
40.0	Bedrock	13 RC 63%					
360.5	Fractured	14 RC 83%					
357.8	Cavity silty sand layer of clayey silt 1" thick. V. Dense	15 SS 115/8"					
348.0	Crystalline Dolomite Occ. vertical seams. Sound.	16 RC 95%					
367.5	White with Pink Zones	17 RC 100%					
57.8	End of Borehole						

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 11

FOUNDATION SECTION

JOB 71-11085

LOCATION Sta. 509 + 16 20' Lt.

W.P. 7-67-03

BORING DATE Sept. 7, 8, 9, 10 & 15, 1971

ORIGINATED BY: JH

DATUM Geodetic

BOREHOLE TYPE Diamond Drill Washboring

COMPILED BY SO

CHECKED BY ELI

[illegible]

DEPARTMENT OF HIGHWAYS- ONTARIO
 MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 5

FOUNDATION SECTION

 JOB 71-11085 LOCATION Sta. 509 + 51 20' Rt. ORIGINATED BY WH
 W.P. 7-67-03 BORING DATE Oct. 18, 19 & 20, 1971 COMPILED BY SO
 DATUM Geodetic BOREHOLE TYPE Diamond Drill Washboring CHECKED BY E.D.

SOIL PROFILE		STRAT. PLOT	SAMPLES		ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE		LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w		BULK DENSITY γ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION		NUMBER	TYPE		BLOWS / FOOT	SHEAR STRENGTH P.S.F.	WATER CONTENT %			
426.6	Ground Level										
0.0	Organic silt & peat. Black		1	SS	0	*					
2.0	Clayey silt, silty sand seams up to 3/4" thick throughout. Stiff to Very Stiff		2	SS	18						
			3	SS	27						
			4	SS	22						
			5	SS	7						
113.1	Grey		6	SS	11						
13.5	Silty sand, some gravel										
408.6	Compact Grey		7	SS	25						
18.0	Glacial Till										
	Het. mix. of silt, sand and gravel, trace of clay.		8	SS	20						
			9	SS	67						
			10	SS	100 11"						
			11	SS	86						
	Boulders up to 4" in size below el. 328.		12	SS	68 1/2"						
			13	SS	112 1/2"						
	Compact to Very Dense		14	SS	90						
376.5	Grey										
50.1	Bedrock		15	RC	30%						
	Fractured										
369.4			16	RC	18%						
57.2	Crystalline Dolomite		17	RC	93%						
	Sound										
	White		18	RC	94%						
361.6											
65.0	End of Borehole										

 * Sampler sank under wt. of hammer
 0 10 68 22
 5 28 56 11

18 39 33 10

FOUNDATION SECTION

ORIGINATED BY WH

COMPILED BY SO

CHECKED BY ELID

SOIL PROFILE		SAMPLES		ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT	LQUID LIMIT ———— w _L PLASTIC LIMIT ———— w _p WATER CONTENT ———— w	BULK DENSITY γ	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER		TYPE	BLOWS / FOOT		
625.5	Ground Level							
0.0	Organic Silt. Brown		1	C.S.				
2.0	Clayey silt silty sand seams up to 1/4" thick throughout.		2	TW				
			3	SS				
			4	TW				
612.6	Gray		5	TW				
12.9	Silty sand, some gravel		6	TW				
			7	SS				
	Glacial Till		8	SS				
	Het. mix. of silt, sand & gravel, trace of clay		9	SS				
	Loose to Very Dense		10	SS				
			11	SS				
	Gray		12	SS				
394.5			13	SS				
31.0	End of Borehole							



Appendix C.

Laboratory Testing

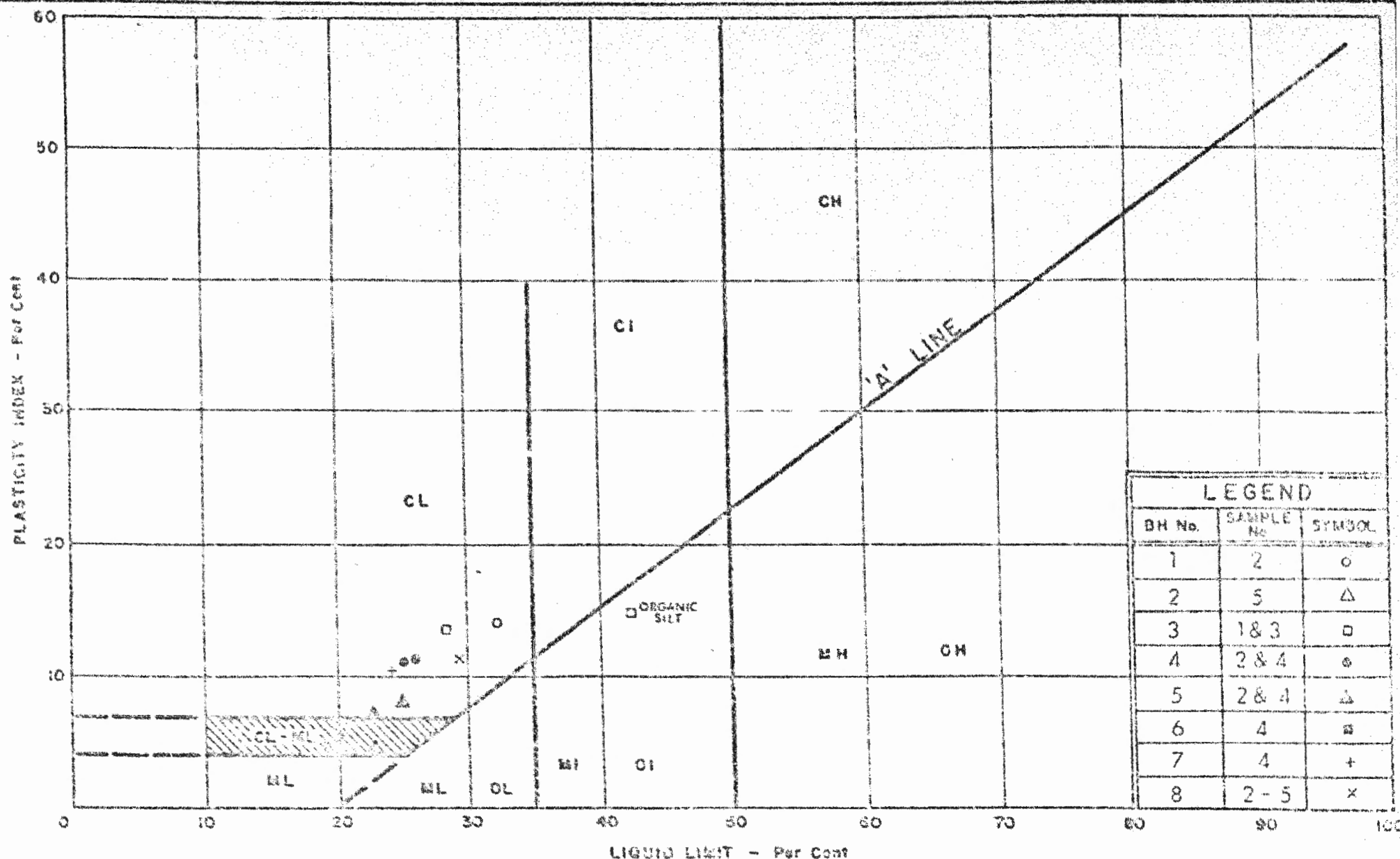


Appendix C.1

Atterberg Limit Test Results

Particle Size Analysis Figures

One-Dimensional Consolidation Test Results

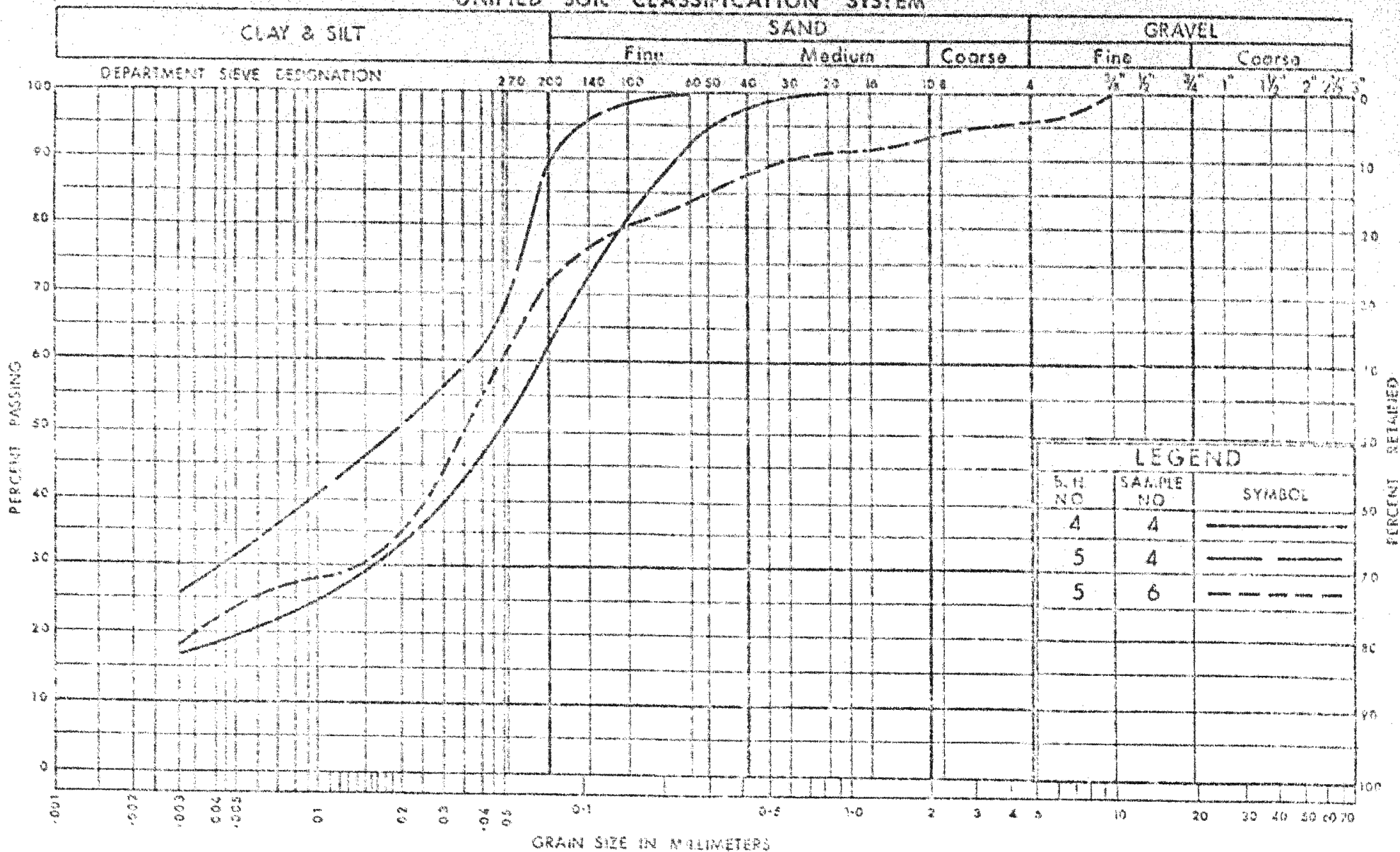


DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

PLASTICITY CHART CLAYEY SILT THIN SILTY SAND SEAMS

WP. No. 7-57-03
JOB No. 71-11085
FIG C1

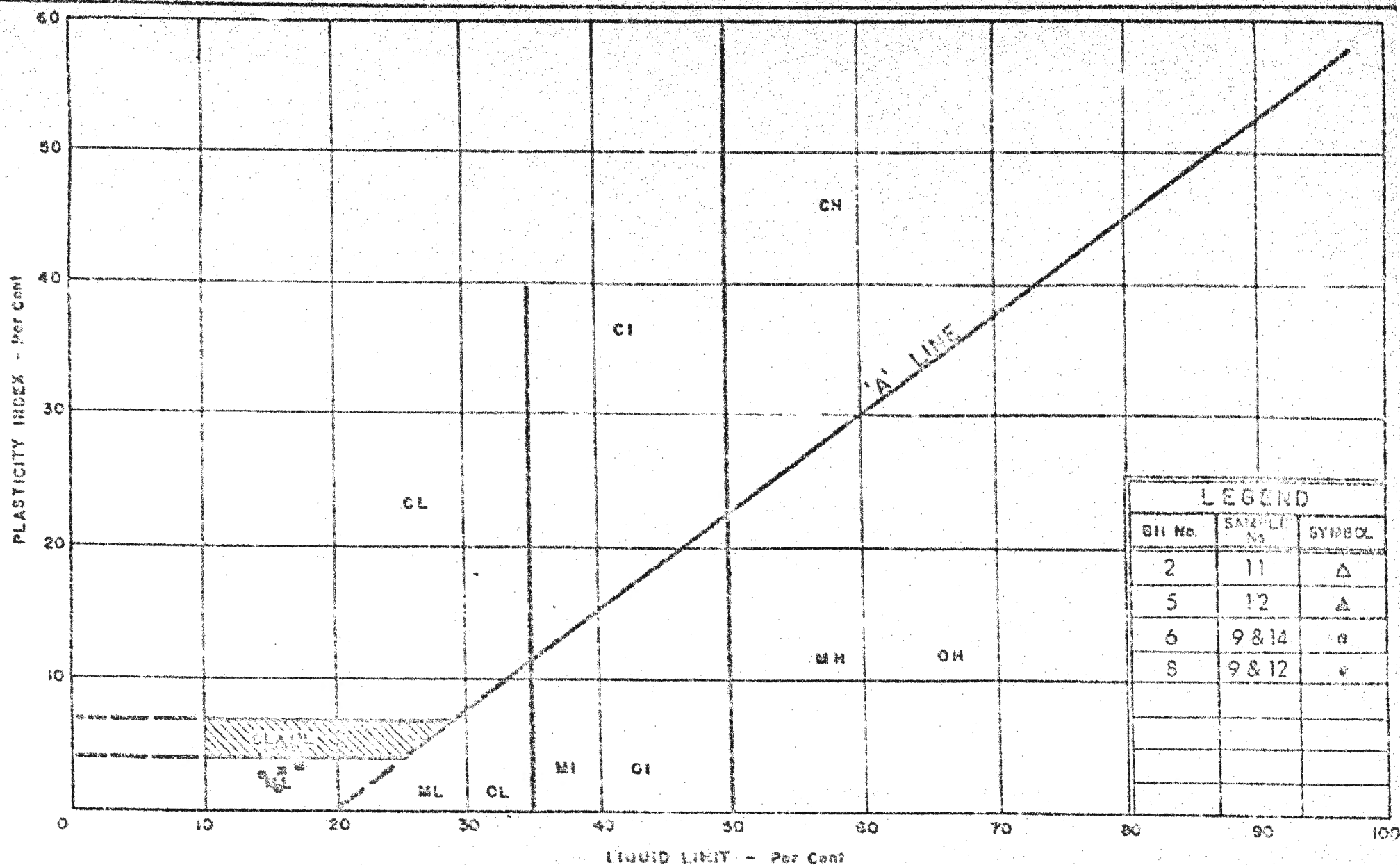
UNIFIED SOIL CLASSIFICATION SYSTEM



DEPARTMENT
OF
TRANSPORTATION AND COMMUNICATIONS
 DESIGN SERVICES
BRANCH

GRAIN SIZE DISTRIBUTION
CLAYEY SILT
WITH SILTY SAND SEAMS

W.P. No. 7-67-03
JOB No. 71-11085
FIG C2



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

PLASTICITY CHART GLACIAL TILL

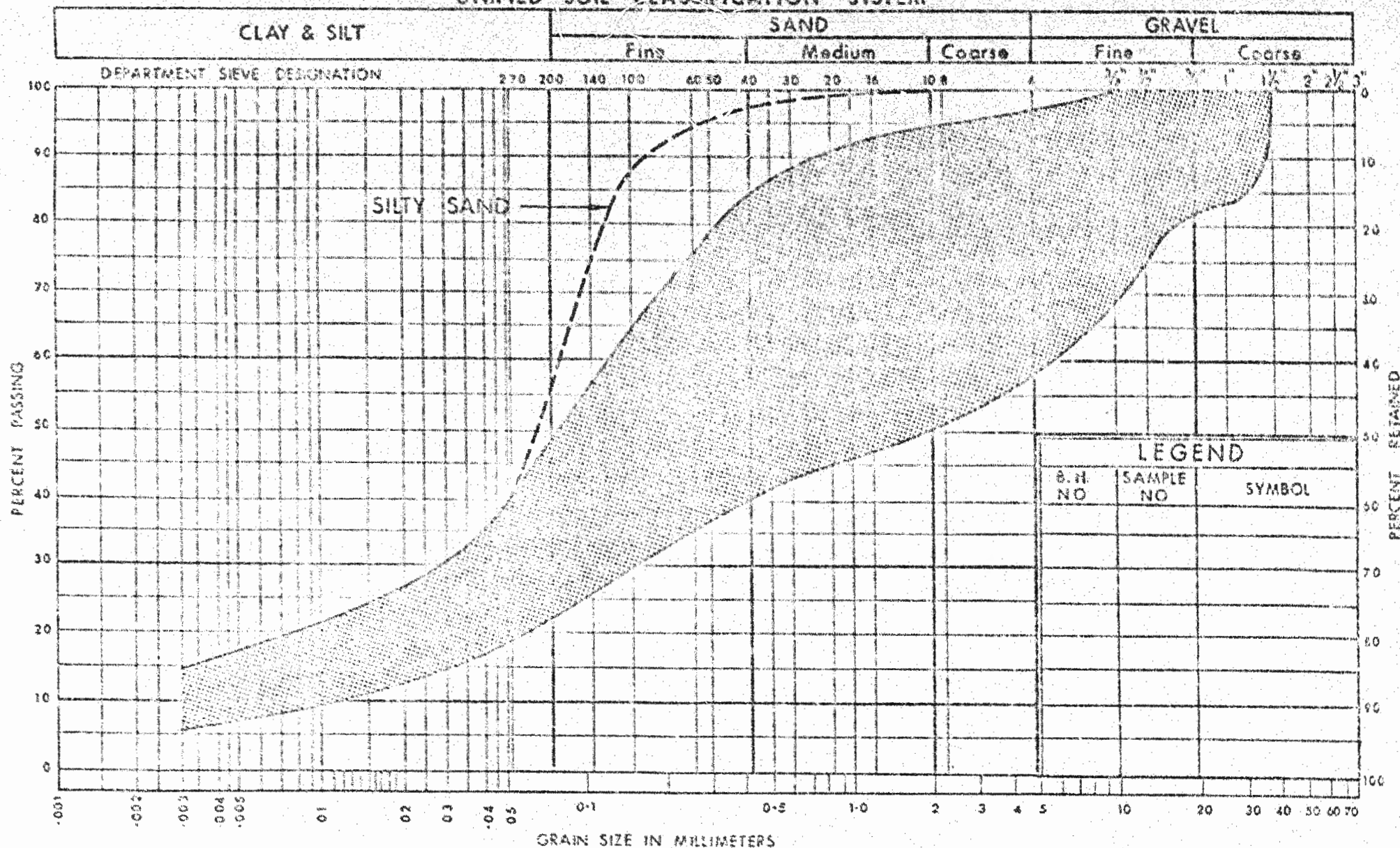
HET. MIXTURE OF SILT, SAND & GRAVEL, TRACE OF CLAY

WP No. 7-67-03

JOB No. 71-11085

FIG. C3

UNIFIED SOIL CLASSIFICATION SYSTEM



DEPARTMENT
OF
TRANSPORTATION AND COMMUNICATIONS



DESIGN SERVICES
BRANCH

GRAIN SIZE DISTRIBUTION
GLACIAL TILL
HET. MIXTURE OF SILT, SAND & GRAVEL, TRACE OF CLAY

W.P. No. 7 - 67 - 03

JOB No. 71-11085

FIG C4

VOID RATIO - PRESSURE CURVES

JOB NO. 71-11085

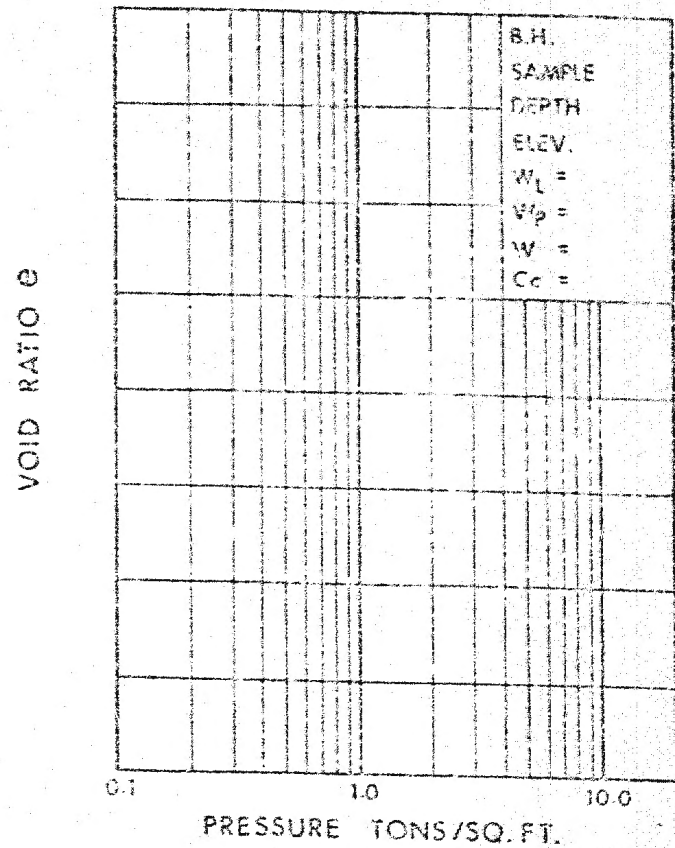
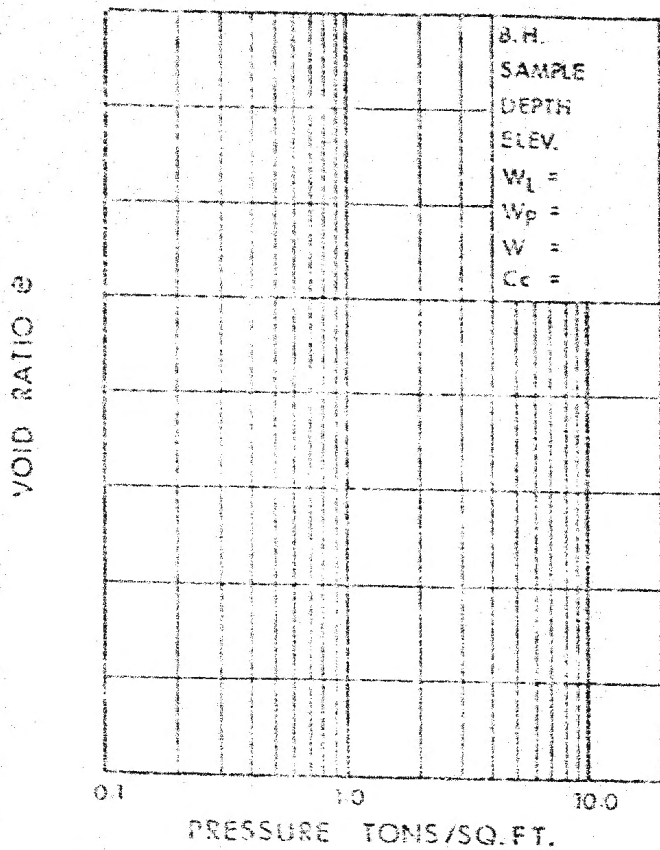
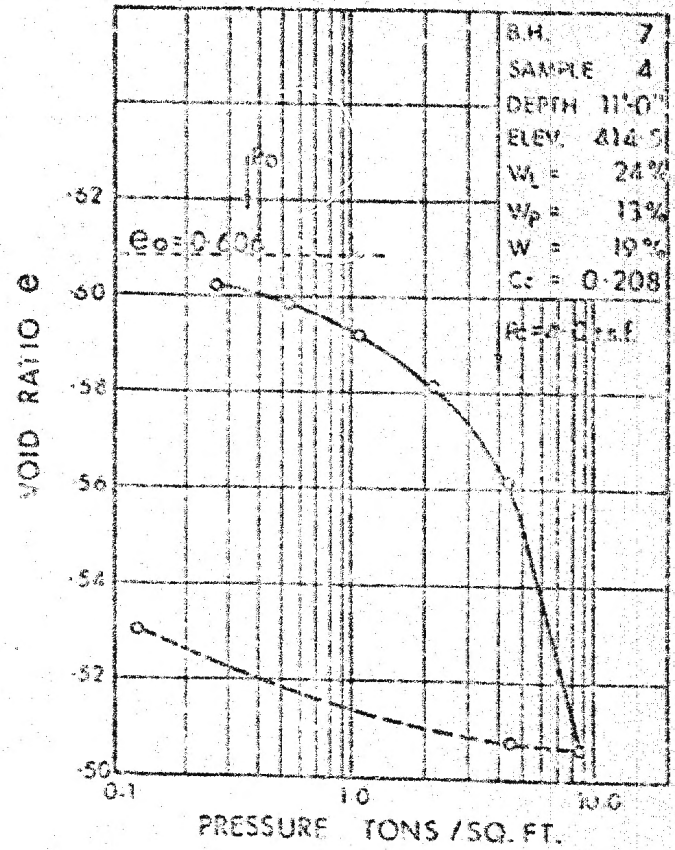
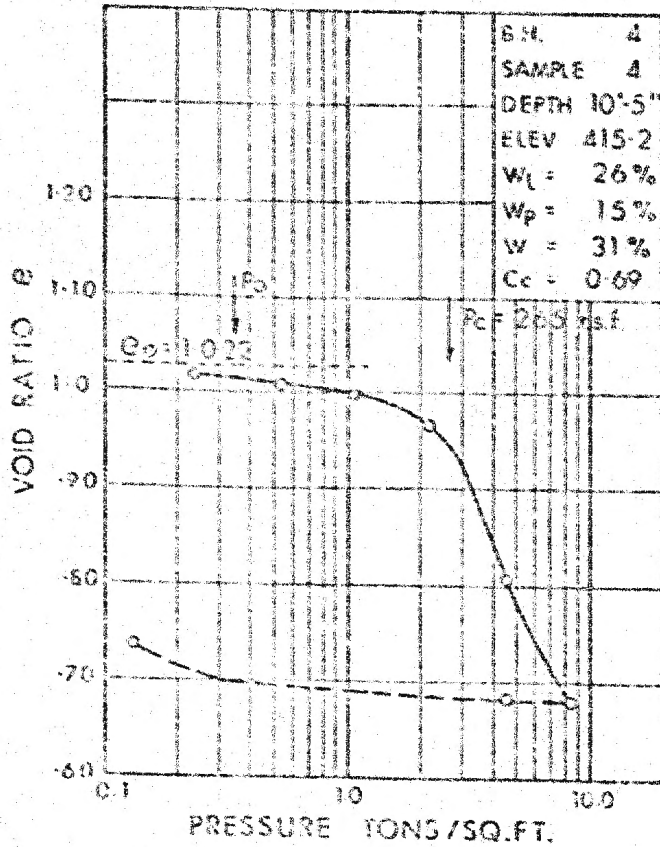


FIG. C5



Appendix D.
Site Photographs



Photo 1. Looking north along trail towards existing bridge (2020/04/22)



Photo 2. Looking south along trail towards existing east pier (2021/11/05)



Photo 3. Looking south from the trail towards east approach and existing drainage culvert (2021/11/05)



Photo 4. Looking west from existing Highway 17 shoulder (2019/09/24)



Appendix E.

MASW Report



GEOPHYSICS GPR INTERNATIONAL INC.

100 – 2545 Delorimier Street Tel. : (450) 679-2400
Longueuil (Québec) Fax : (514) 521-4128
Canada J4K 3P7 info@geophysicsgpr.com
www.geophysicsgpr.com

June 24th, 2020

Transmitted by email: jgray@thurber.ca
Our Ref.: GPR-19-01787

Mr. Justin Gray, P.Eng.
Geotechnical Engineer
Thurber Engineering Ltd.
Suite 104, 2460 Lancaster Road
Ottawa, Ontario K1B 4S5

Subject: Shear Wave Velocity Soundings for Determining Site Classifications
Three locations along Highway 17, in Renfrew County (ON)

Dear Sir,

Geophysics GPR International inc. has been mandated by Thurber Engineering Ltd. to carry out seismic shear wave surveys for three locations along the Trans-Canada Highway, in the Renfrew County (ON). The geophysical investigation used the Multi-channel Analysis of Surface Waves (MASW), the Extended Spatial AutoCorrelation (ESPAC), and the seismic refraction methods. From the subsequent results, the seismic shear wave velocity values were calculated for the soil and the rock, to determine the Site Classes for the different locations.

The surveys were carried out on June 3rd and 4th, by Mr. Mario Nucciarone, B.Sc. geoph. and Mr. Ange Alexandre Forestier, trainee. Figure 1 shows the regional location of the different sites and the Figures 2a to 2c illustrate the locations of the five seismic lines. These figures are presented in the Appendix.

The following paragraphs briefly describe the survey design, the principles of the testing methods, and the results presented in tables and graphs.

MASW PRINCIPLE

The *Multi-channel Analysis of Surface Waves* (MASW) and the *Extended SPatial AutoCorrelation* (ESPAC or MAM for *Microtremors Array Method*) are seismic methods used to evaluate the shear wave velocities of subsurface materials through the analysis of the dispersion properties of the Rayleigh surface waves (“ground roll”). The MASW is considered an “active” method, as the seismic signal is induced at known location and time in the geophones’ spread axis. Conversely, the ESPAC is considered a “passive” method, using the low frequency “signals” produced far away. The method can also be used with “active” seismic source records. The dispersion properties are expressed as a change of phase velocities with respect to frequencies. Surface wave energy will decay exponentially with depth. Lower frequency surface waves will travel deeper and thus be more influenced by deeper velocity layering than the shallow higher frequency waves. The inversion of the Rayleigh wave dispersion curve yields a shear wave (V_s) velocity depth profile (sounding). Figure 3 schematically outlines the basic operating procedure for the MASW method.

Figure 4 illustrates an example of one of the MASW/ESPAC records, the corresponding spectrogram analysis and resulting 1D V_s model. The ESPAC method allows deeper V_s soundings, but generally with a lower resolution for the surface portion. Its dispersion curve can then be merged with the one of higher frequency from the MASW to calculate a more complete inversion.

INTERPRETATION

The main processing sequence involved data inspection and edition when required; spectral analysis (“phase shift” for MASW, and “cross-correlation” for ESPAC); picking the fundamental mode; and 1D inversion of the MASW and ESPAC shot records using the SeisImagerSW™ software. The data inversions used a nonlinear least squares algorithm.

In theory, all the shot records for a given seismic spread should produce a similar shear-wave velocity profile. In practice, however, differences can arise due to energy dissipation, local surface seismic velocities variations, and/or dipping of overburden layers or rock. In general, the precision of the calculated seismic shear wave velocities (V_s) is of the order of 15% or better.

More detailed descriptions of these methods are presented in *Shear Wave Velocity Measurement Guidelines for Canadian Seismic Site Characterization in Soil and Rock*, Hunter, J.A., Crow, H.L., et al., Geological Surveys of Canada, General Information Product 110, 2015.



SURVEY DESIGN

The seismic acquisition spreads were laid out with basic geophone spacings of 3 metres for the main spread, using 24 geophones. One to two shorter seismic spreads, with geophone spacings of 0.5 and 1.0 metre, were dedicated to the near surface materials.

The seismic records counted 4096 data, sampled at 1000 μ s for the MASW surveys, and 50 μ s for the seismic refraction. The records included a pre-triggerring portion of 10 ms. A stacking procedure was also used to improve the Signal / Noise ratio for the seismic records.

The shear wave depth sounding can be considered as the average of the bulk area within the geophone spread, especially for its central half-length.

The seismic records were produced with a seismograph Terraloc MK6 (from ABEM Instrument), and the geophones were 4.5 Hz. An 8 kg sledgehammer was used as the energy source with impacts being recorded off both ends of the seismic spreads.

RESULTS

The \bar{V}_{S30} value results from the harmonic mean of the shear wave velocities, from the surface to 30 metres deep. It is calculated by dividing the total depth of interest (30 metres) by the sum of the time spent in each velocity layer from the surface down to 30 metres, as:

$$\bar{V}_{S30} = \frac{\sum_{i=1}^N H_i}{\sum_{i=1}^N H_i / V_i} \quad | \quad \sum_{i=1}^N H_i = 30 \text{ m}$$

(N: number of layers; H_i : thickness of layer "i" ; V_i : V_s of layer "i")

Thus, the \bar{V}_{S30} value represents the seismic shear wave velocity of an equivalent homogeneous single layer response, between the surface and 30 metres deep.



Lochwinnoch & Gillan Roads intersection

The first geophysical investigations were carried out on Lochwinnoch Road, north-east of HW-17, and on Gillan Road, south-west of Highway 17.

The Lochwinnoch Road seismic spread (SL-1):

The Lochwinnoch Road seismic line was placed northeast of the intersection between Lochwinnoch Road and Highway 17 (cf. Figure 2a).

The MASW calculated V_s results are illustrated at Figure 6a also presented at Table 1, for the \bar{V}_{s30} calculation.

From the seismic refraction data, the rock was calculated between 2 and 3.5 metres deep, ± 1 metre. Its seismic velocity was calculated between 2280 and 2295 m/s for its shallow portion (cf. Figure 5a).

The calculated \bar{V}_{s30} value of the actual site is 1340.1 m/s (cf. Table 1), corresponding to the Site Class “B”. However, the Site Classes A and B are not to be used if there is 3 metres or more of unconsolidated material between the rock and the bottom of the spread footing or mat foundation.

If the foundation is 2.25 metres or less from the rock surface, the minimal \bar{V}_{s30}^* value would be 1513.6 m/s or higher, allowing to use the Site Class “A”.

The Gillan Road seismic spread (SL-2):

The Gillan Road seismic line was placed southwest of the intersection between Gillan Road and Highway 17 (cf. Figure 2a).

The MASW calculated V_s results are illustrated at Figure 6a and they are also presented at Table 2, for the \bar{V}_{s30} calculation. The Seismic Site Class for the actual site is 592.2 m/s, corresponding to the Site Class “C”. Some low seismic velocities were calculated from the surface to approximately 2 metres deep.



From the seismic refraction data, the rock was calculated between 9.5 and 13.5 metres deep ($\pm 10\%$), dipping South-West. The seismic velocity of the shallow portion was calculated to be 2365 m/s (cf. Figure 5a).

Algonquin Trail seismic spread (SL-3):

Seismic-Line 3 was placed on the Algonquin Trail, west of the intersection with the Highway 17 (cf. Figure 2b).

The MASW calculated V_s results are illustrated in Figure 6b and they are also presented at Table 3, for the \bar{V}_{s30} calculation. The Seismic Site Class for the actual site is 566.7 m/s, corresponding to the Site Class "C". Some low seismic velocities were calculated from the surface to approximately 2 metres deep.

From the seismic refraction data, the rock was calculated between 11 and 13 metres deep ($\pm 10\%$). Its seismic velocity was calculated between 2440 and 2565 m/s for its shallow portion (cf. Figure 5b).

The Bonnechere River Bridge:

Two seismic lines were localised on each side of the Bonnechere River Bridge. Both were placed on the west side of the highway 17.

Bonnechere River Bridge, North Side (SL-4):

The Bonnechere River Bridge north side was investigated from the seismic line 4, installed south-west to the Highway 17 (cf. Figure 2c).

The MASW calculated V_s results are illustrated at Figure 6c and they are also presented at Table 4, for the \bar{V}_{s30} calculation. This value for the actual site is 260.3 m/s, corresponding to the Site Class "D". Some low seismic velocities were calculated between 1.5 and 5 metres deep.

From the seismic refraction data, the rock was calculated between 37 and 42 metres deep ($\pm 10\%$), dipping South-East.



Bonnechere River Bridge, South Side (SL-5):

Seismic line 5, placed southwest to Highway 17, was used to investigate the Bonnechere River Bridge south side (cf. Figure 2c).

The MASW calculated V_s results are illustrated in Figure 6c and they are also presented at Table 5, for the \bar{V}_{s30} calculation. The Seismic Site Class for the actual site is 372.1 m/s, corresponding to the Site Class “C”. Some low seismic velocities were calculated from the surface to nearly 1 metre deep.

From the seismic refraction and seismic resonance results, the rock was calculated between 27 and 30 metres deep ($\pm 10\%$).



CONCLUSION

Geophysical surveys were carried out in the vicinity of Highway 17, from the Road 6 to the Bonnechere Bridge, in Renfrew County (ON). The seismic surveys used the MASW and ESPAC analysis, as well as seismic refraction method, to calculate the \bar{V}_{S30} values so as to determine the Site Classes.

The \bar{V}_{S30} calculations for the actual sites are presented in Table 1 to 5. They were determined through the MASW, ESPAC and seismic refraction methods, Table 4.1.8.4.A of the NBC, and the Building Code, O. Reg. 332/12.

It must be noted that other geotechnical information gleaned on site; including the presence of liquefiable soils, very soft clays, high moisture content etc. can supersede the Sites Classifications provided in this report based on the \bar{V}_{S30} values.

The V_s values calculated are representative of the in-situ materials and are not corrected for the total and effective stresses.

Hoping the whole to your satisfaction, we remain yours truly.

Jean-Luc Arsenault, M.A.Sc., P.Eng.
Senior Project Manager





Figure 1: Regional location of the Sites
(source: OpenStreetMap©)

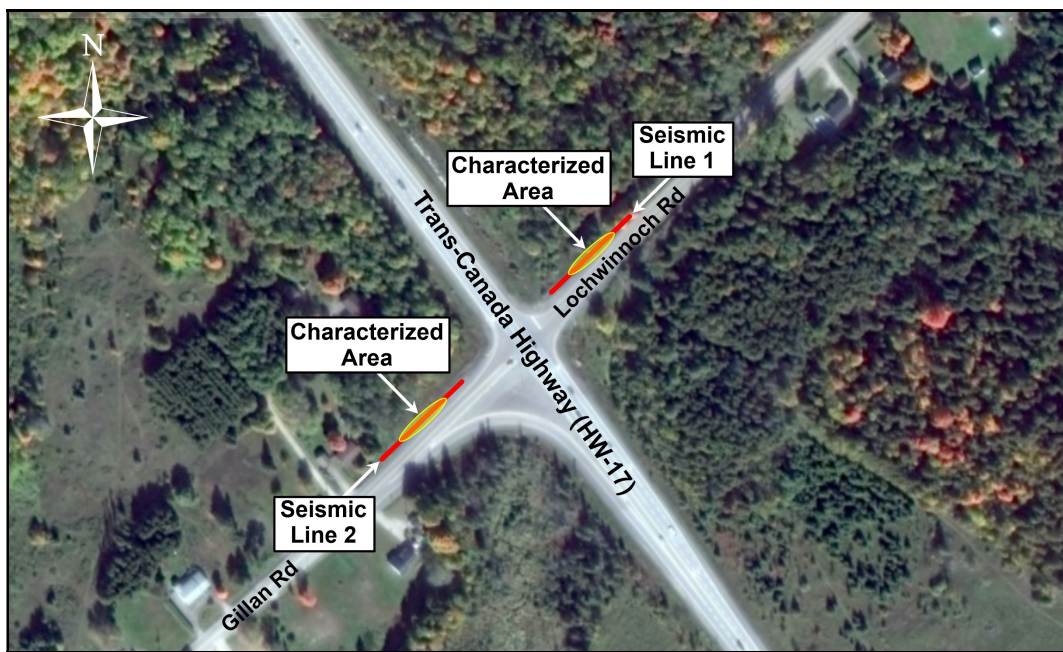


Figure 2a: Location of the seismic spreads SL-1 and SL-2
(source: Google Earth™)





Figure 2b: Location of the seismic spread SL-3
(source: Google Earth™)

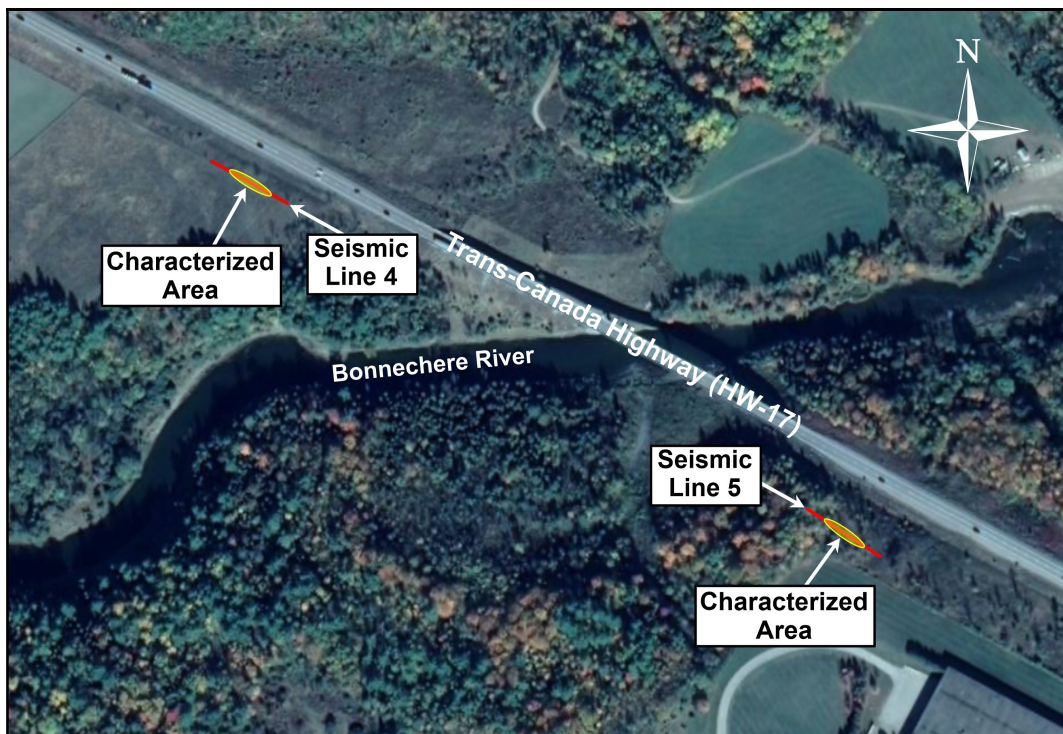


Figure 2c: Location of the seismic spreads SL-4 and SL-5
(source: Google Earth™)



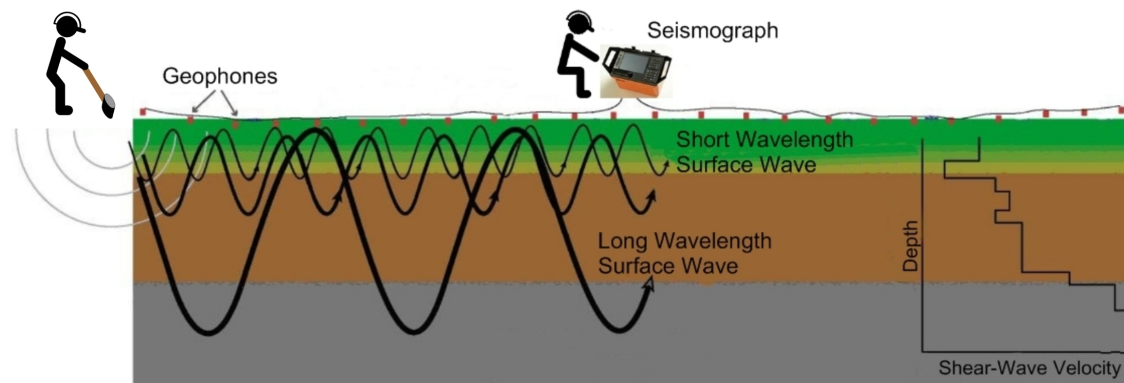


Figure 3: MASW Operating Principle

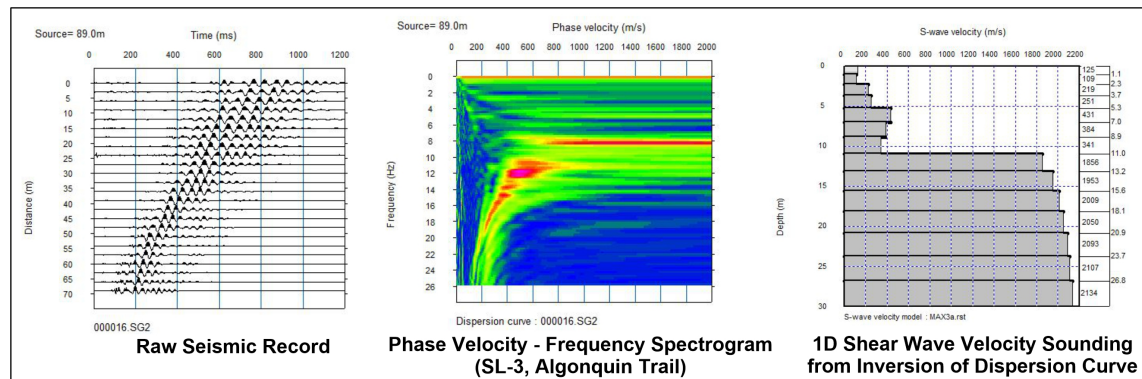
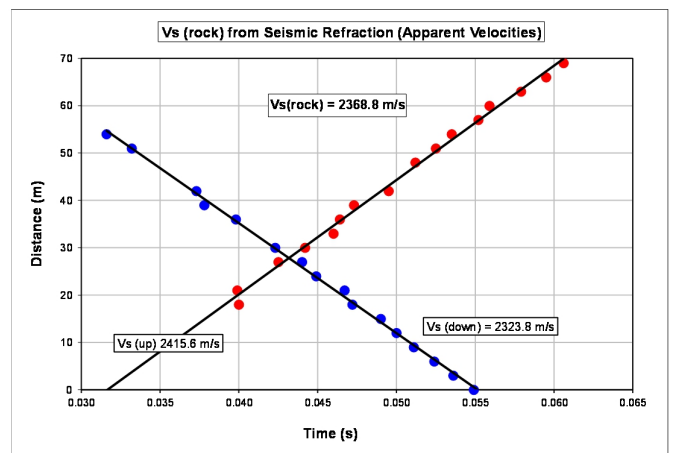
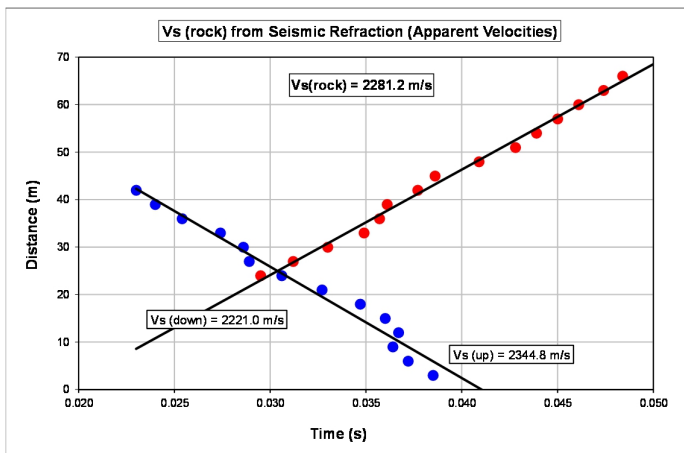
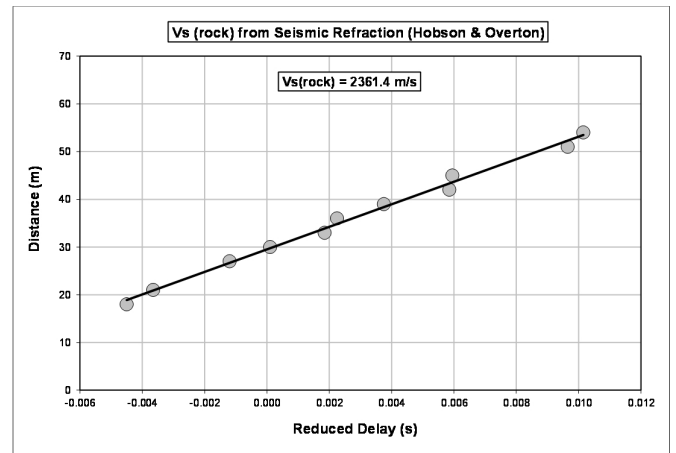
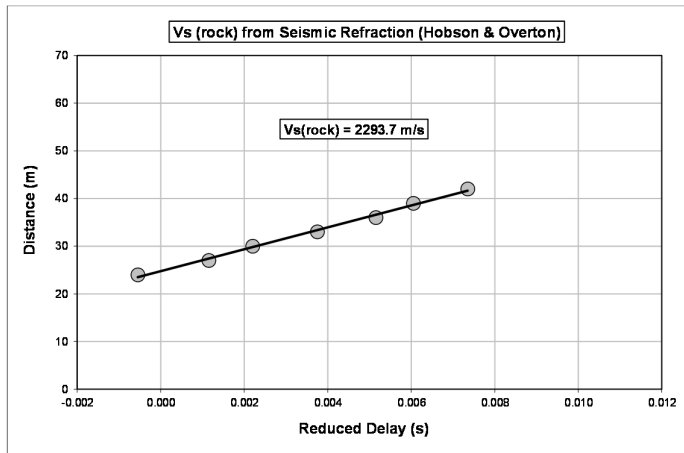


Figure 4: Example of a MASW / ESPAC record, Phase Velocity - Frequency curve and resulting 1D Shear Wave Velocity Model



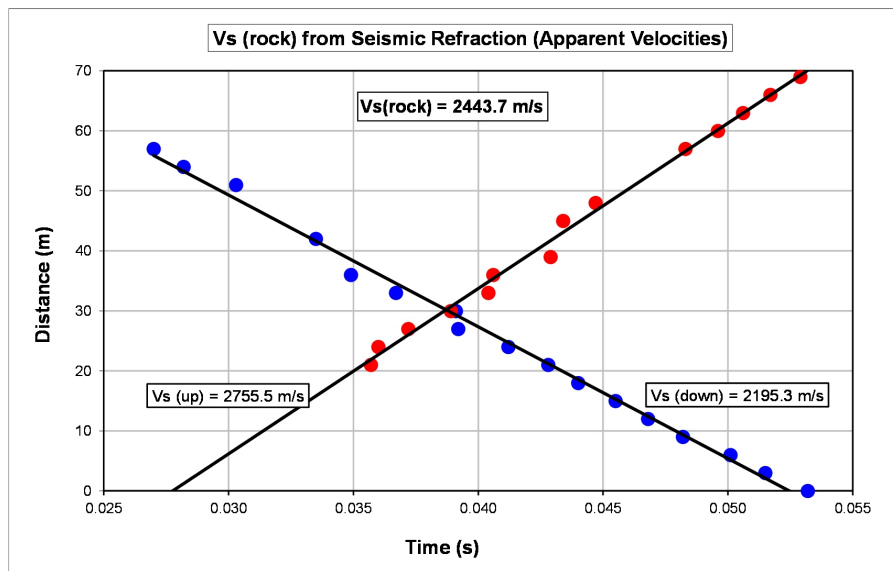
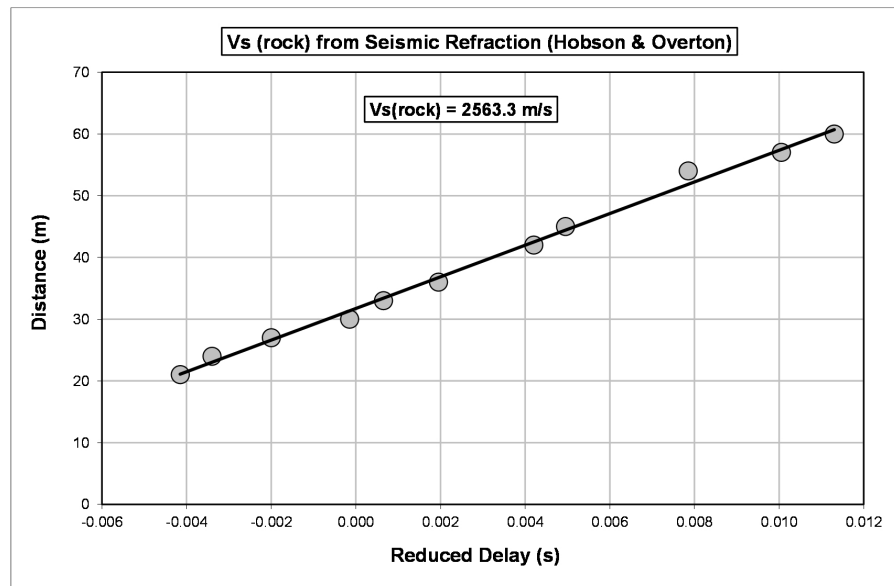


SL-1

SL-2

Figure 5a: Rock V_s from Seismic Refraction





SL-3

Figure 5b: Rock V_s from Seismic Refraction



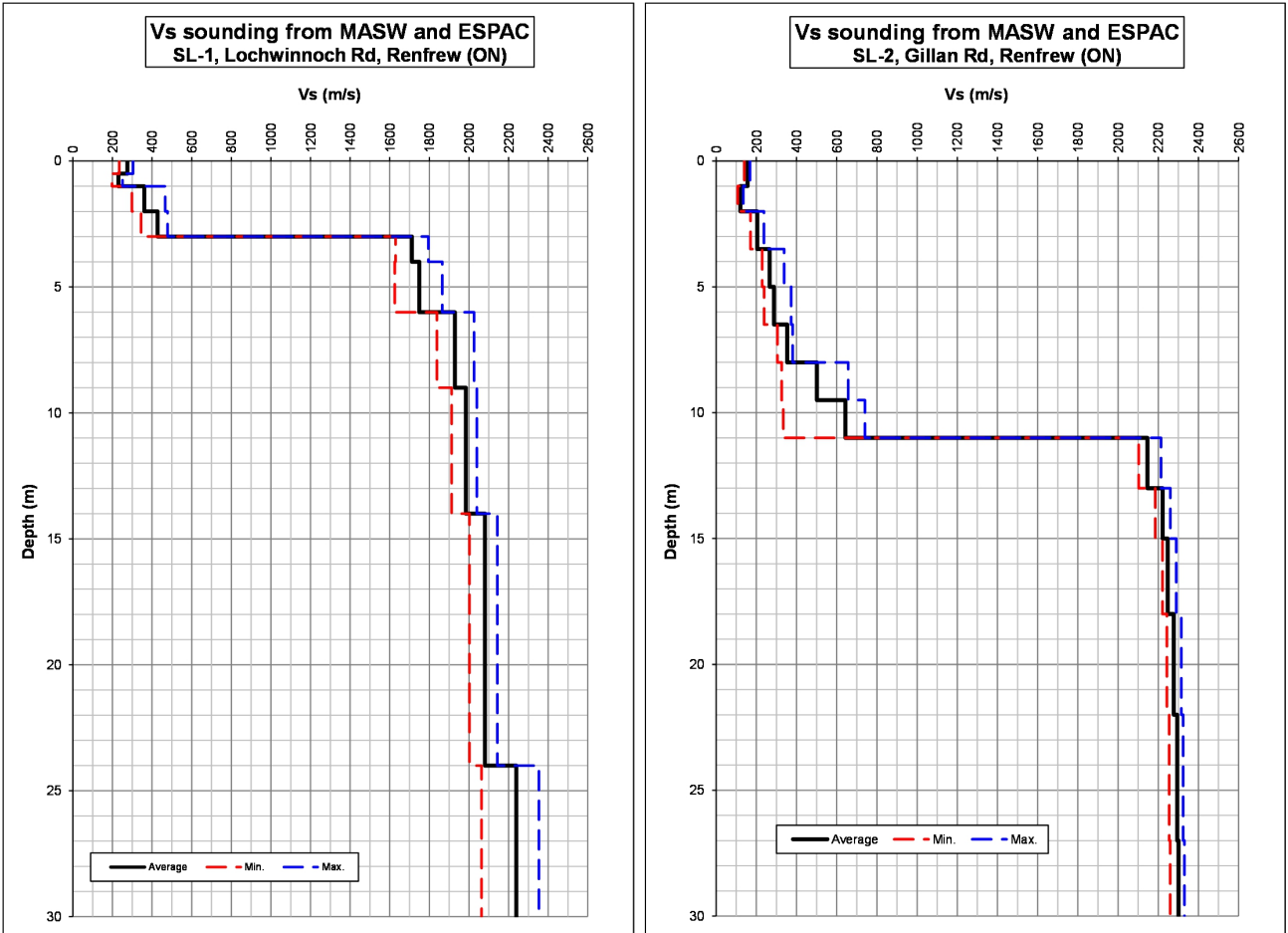


Figure 6a: MASW Shear-Wave Velocity Soundings



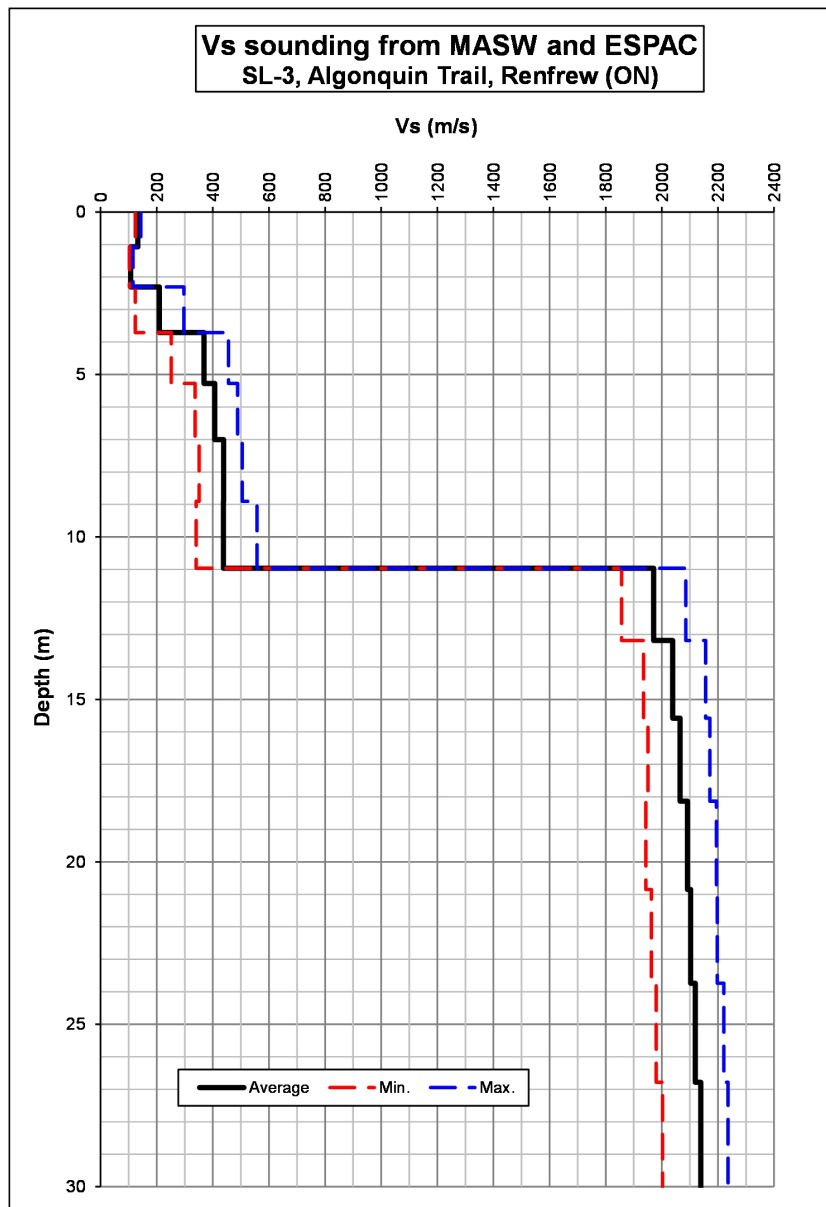


Figure 6b: MASW Shear-Wave Velocity Sounding



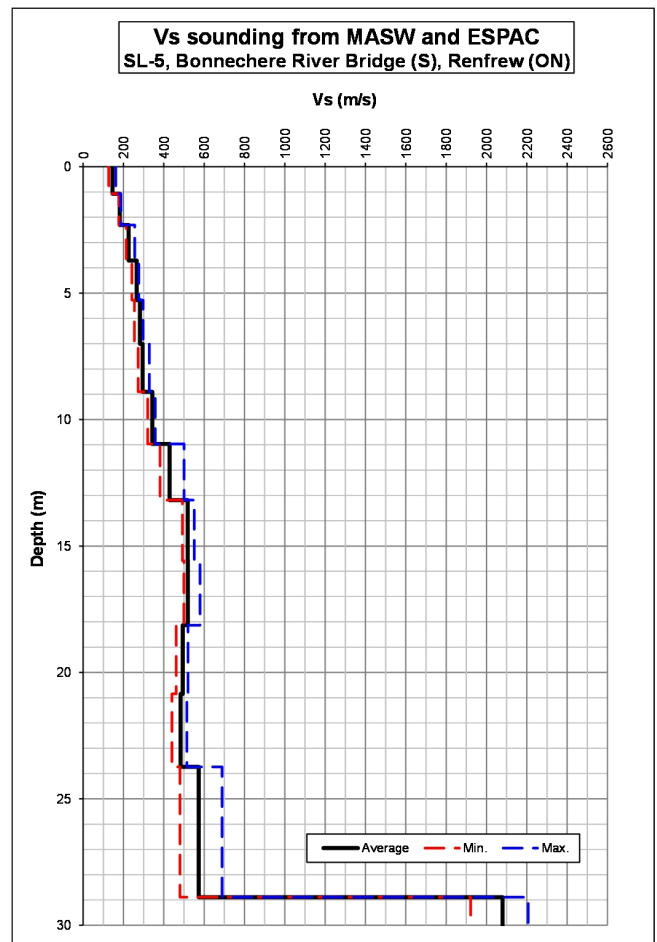
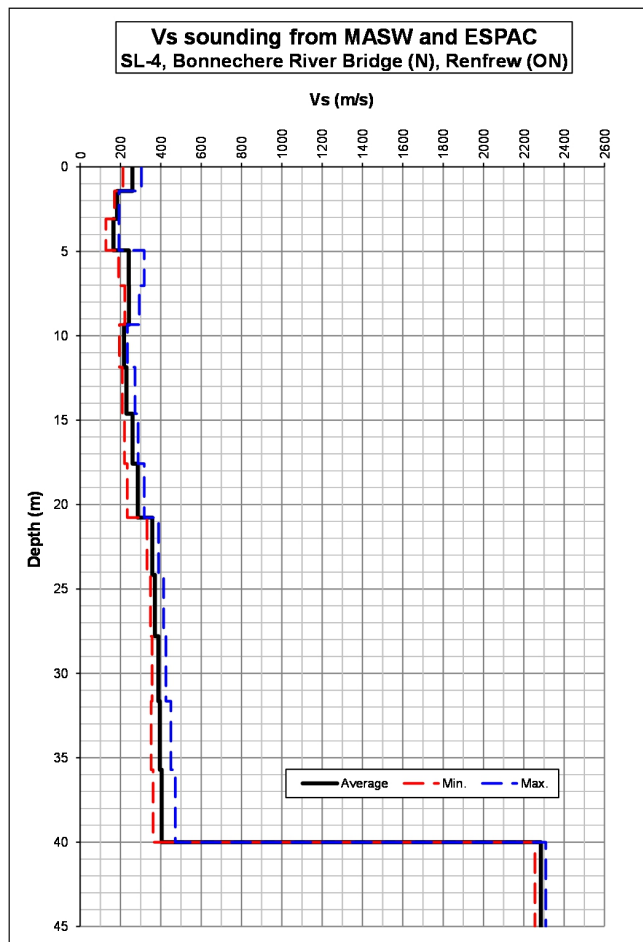


Figure 6c: MASW Shear-Wave Velocity Soundings



TABLE 1
Lochwinnoch Rd (SL-1) V_{S30} Calculation for the Site Class (actual site)

Depth	Vs			Thickness	Cumulative Thickness	Delay for Avg. Vs	Cumulative Delay	Vs at given Depth
	Min.	Average	Max.					
(m)	(m/s)	(m/s)	(m/s)	(m)	(m)	(s)	(s)	(m/s)
0	234.4	275.3	303.8	Ground level while seismic surveys (June 3rd, 2020)				
0.5	198.0	230.3	250.5	0.50	0.50	0.001816	0.001816	275.3
1.0	297.7	360.7	466.1	0.50	1.00	0.002171	0.003987	250.8
2.0	344.4	427.5	478.2	1.00	2.00	0.002772	0.006759	295.9
3.0	1629.2	1712.1	1795.0	1.00	3.00	0.002339	0.009099	329.7
4.0	1625.1	1749.3	1864.8	1.00	4.00	0.000584	0.009683	413.1
6.0	1838.2	1929.0	2025.7	2.00	6.00	0.001143	0.010826	554.2
9.0	1912.5	1984.7	2039.6	3.00	9.00	0.001555	0.012381	726.9
14.0	2002.4	2080.9	2143.6	5.00	14.00	0.002519	0.014901	939.6
24.0	2063.2	2238.6	2353.2	10.00	24.00	0.004806	0.019706	1217.9
30				6.00	30.00	0.002680	0.022387	1340.1

Vs30 (m/s)	1340.1
Class	B ⁽¹⁾

- (1) The Site Classes A and B are not to be used if there is 3 metres or more of unconsolidated materials between the rock surface and the bottom of the foundation.

TABLE 2
Gillan Rd (SL-2) V_{S30} Calculation for the Site Class (actual site)

Depth	Vs			Thickness	Cumulative Thickness	Delay for Avg. Vs	Cumulative Delay	Vs at given Depth
	Min.	Average	Max.					
(m)	(m/s)	(m/s)	(m/s)	(m)	(m)	(s)	(s)	(m/s)
0	140.3	156.0	168.6	Ground level while seismic surveys (June 3rd, 2020)				
1.0	107.2	120.8	135.1	1.00	1.00	0.006409	0.006409	156.0
2.0	171.1	204.6	237.6	1.00	2.00	0.008276	0.014685	136.2
3.5	228.7	266.4	338.6	1.50	3.50	0.007333	0.022018	159.0
5.0	239.0	287.7	373.5	1.50	5.00	0.005631	0.027649	180.8
6.5	304.6	353.8	380.7	1.50	6.50	0.005213	0.032862	197.8
8.0	325.9	501.5	657.2	1.50	8.00	0.004239	0.037101	215.6
9.5	334.5	643.0	740.8	1.50	9.50	0.002991	0.040093	237.0
11.0	2103.0	2145.7	2213.9	1.50	11.00	0.002333	0.042426	259.3
13.0	2185.2	2222.2	2259.8	2.00	13.00	0.000932	0.043358	299.8
15.0	2221.1	2247.1	2289.5	2.00	15.00	0.000900	0.044258	338.9
18.0	2243.0	2277.5	2314.5	3.00	18.00	0.001335	0.045593	394.8
22.0	2254.7	2294.7	2323.6	4.00	22.00	0.001756	0.047349	464.6
27.0	2259.2	2300.9	2330.0	5.00	27.00	0.002179	0.049528	545.1
30				3.00	30.00	0.001304	0.050832	590.2

Vs30 (m/s)	590.2
Class	C



TABLE 3
Algonquin Trail (SL-3) V_{S30} Calculation for the Site Class (actual site)

Depth	Vs			Thickness	Cumulative Thickness	Delay for Avg. Vs	Cumulative Delay	Vs at given Depth
	Min.	Average	Max.					
(m)	(m/s)	(m/s)	(m/s)	(m)	(m)	(s)	(s)	(m/s)
0	123.5	132.9	143.7	Ground level while seismic surveys (June 4th, 2020)				
1.1	102.9	107.4	115.5	1.07	1.07	0.008062	0.008062	132.9
2.3	123.6	209.6	296.8	1.24	2.31	0.011506	0.019568	117.9
3.7	251.7	368.8	455.6	1.40	3.71	0.006686	0.026254	141.3
5.3	336.8	407.1	488.2	1.57	5.27	0.004246	0.030501	172.9
7.0	351.1	438.4	504.6	1.73	7.01	0.004252	0.034752	201.6
8.9	341.0	437.7	557.3	1.90	8.90	0.004323	0.039076	227.8
11.0	1856.7	1971.2	2085.7	2.06	10.96	0.004708	0.043784	250.4
13.2	1934.5	2038.4	2156.3	2.23	13.19	0.001129	0.044912	293.6
15.6	1950.8	2065.2	2171.4	2.39	15.58	0.001173	0.046085	338.0
18.1	1942.8	2091.4	2194.3	2.55	18.13	0.001237	0.047322	383.2
20.9	1962.4	2103.1	2197.6	2.72	20.85	0.001300	0.048623	428.8
23.7	1980.0	2119.6	2220.9	2.88	23.74	0.001372	0.049994	474.8
26.8	2002.4	2138.5	2235.6	3.05	26.79	0.001439	0.051433	520.8
30	2100.0	2160.9	2246.5	3.21	30.00	0.001503	0.052936	566.7

V_{S30} (m/s)	566.7
Class	C



TABLE 4
Bonnechere River Bridge (SL-4) V_{S30} Calculation for the Site Class (actual site)

Depth	Vs			Thickness	Cumulative Thickness	Delay for Avg. Vs	Cumulative Delay	Vs at given Depth
	Min.	Average	Max.					
(m)	(m/s)	(m/s)	(m/s)	(m)	(m)	(s)	(s)	(m/s)
0	212.3	258.7	303.8	Ground level while seismic surveys (June 4th, 2020)				
1.4	169.6	183.0	193.0	1.43	1.43	0.005521	0.005521	258.7
3.1	127.5	164.7	192.4	1.65	3.08	0.009009	0.014530	211.8
4.9	190.8	241.1	317.2	1.87	4.95	0.011340	0.025871	191.1
7.0	221.6	242.1	293.4	2.09	7.03	0.008661	0.034531	203.7
9.3	194.6	217.5	234.1	2.31	9.34	0.009531	0.044062	212.0
11.9	208.9	229.3	272.3	2.53	11.87	0.011620	0.055682	213.1
14.6	220.0	259.3	287.0	2.75	14.62	0.011982	0.067665	216.0
17.6	233.3	285.8	318.0	2.97	17.58	0.011441	0.079106	222.3
20.8	331.6	357.8	388.3	3.19	20.77	0.011150	0.090256	230.1
24.2	348.8	370.4	413.5	3.41	24.18	0.009521	0.099776	242.3
27.8	357.1	388.0	425.7	3.63	27.80	0.009791	0.109567	253.7
30				2.20	30.00	0.005665	0.115232	260.3

V_{S30} (m/s)	260.3
Class	D

TABLE 5
Bonnechere River Bridge (SL-5) V_{S30} Calculation for the Site Class (actual site)

Depth	Vs			Thickness	Cumulative Thickness	Delay for Avg. Vs	Cumulative Delay	Vs at given Depth
	Min.	Average	Max.					
(m)	(m/s)	(m/s)	(m/s)	(m)	(m)	(s)	(s)	(m/s)
0	126.4	145.8	161.5	Ground level while seismic surveys (June 4th, 2020)				
1.1	177.4	181.2	186.9	1.07	1.07	0.007351	0.007351	145.8
2.3	212.9	225.6	255.8	1.24	2.31	0.006823	0.014174	162.8
3.7	242.0	265.3	276.4	1.40	3.71	0.006211	0.020385	181.9
5.3	253.9	281.9	297.1	1.57	5.27	0.005902	0.026287	200.7
7.0	272.9	294.5	327.8	1.73	7.01	0.006140	0.032427	216.0
8.9	319.8	342.6	358.0	1.90	8.90	0.006437	0.038864	229.0
11.0	381.7	428.2	500.3	2.06	10.96	0.006013	0.044877	244.3
13.2	492.2	518.2	551.1	2.23	13.19	0.005196	0.050074	263.3
15.6	499.5	518.9	579.8	2.39	15.58	0.004612	0.054686	284.8
18.1	461.3	493.7	520.1	2.55	18.13	0.004924	0.059610	304.2
20.9	440.3	483.0	514.4	2.72	20.85	0.005509	0.065119	320.2
23.7	479.7	573.1	688.9	2.88	23.74	0.005972	0.071091	333.9
28.9	1921.1	2079.6	2207.2	5.15	28.89	0.008992	0.080083	360.7
30	1939.3	2103.1	2237.5	1.11	30.00	0.000534	0.080617	372.1

V_{S30} (m/s)	372.1
Class	C





Appendix F.

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: 45.477N 76.633W

User File Reference: CPR Overhead

2020-10-14 18:47 UT

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)	0.355	0.183	0.105	0.032
Sa (0.1)	0.421	0.228	0.137	0.045
Sa (0.2)	0.352	0.198	0.123	0.043
Sa (0.3)	0.268	0.154	0.098	0.035
Sa (0.5)	0.192	0.112	0.072	0.026
Sa (1.0)	0.098	0.059	0.038	0.013
Sa (2.0)	0.047	0.028	0.018	0.005
Sa (5.0)	0.013	0.007	0.004	0.001
Sa (10.0)	0.005	0.003	0.002	0.001
PGA (g)	0.226	0.125	0.076	0.025
PGV (m/s)	0.160	0.090	0.055	0.018

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

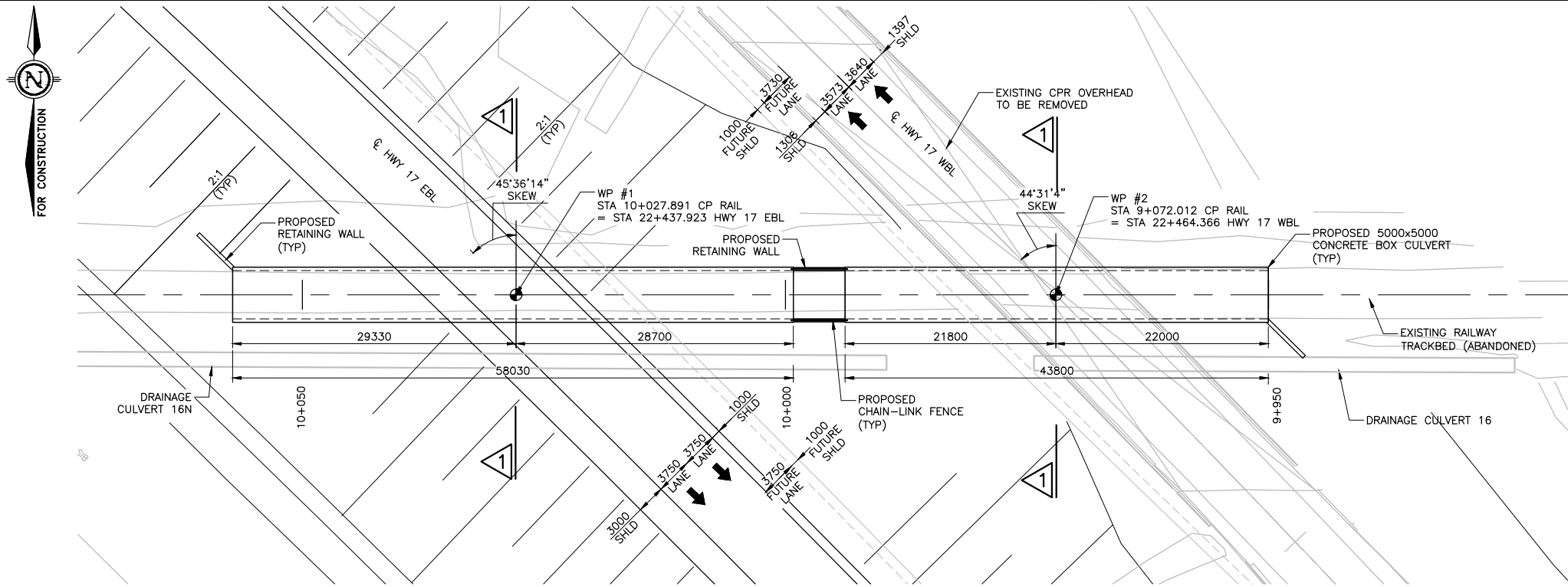
Ressources naturelles
Canada

Canada

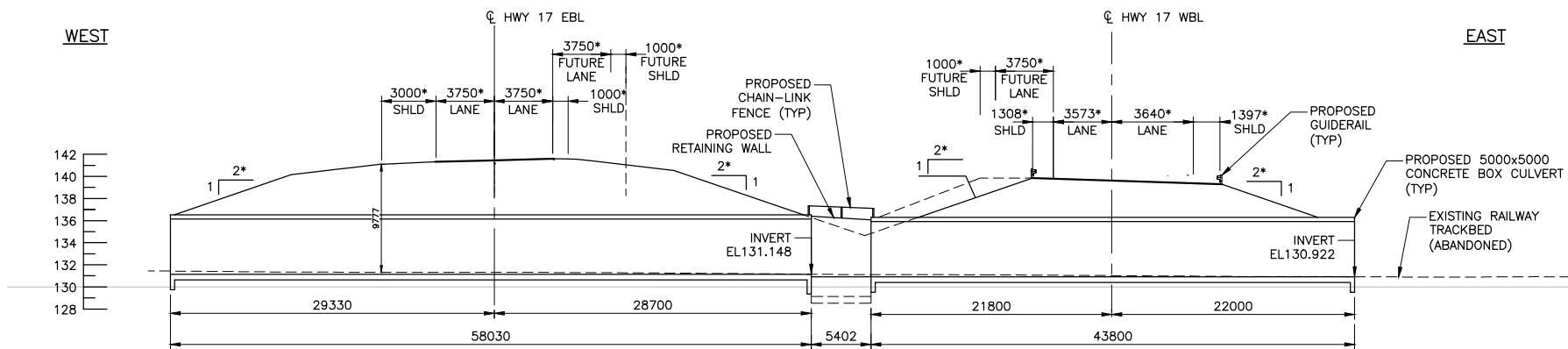


Appendix G.

Preliminary General Arrangement Drawings

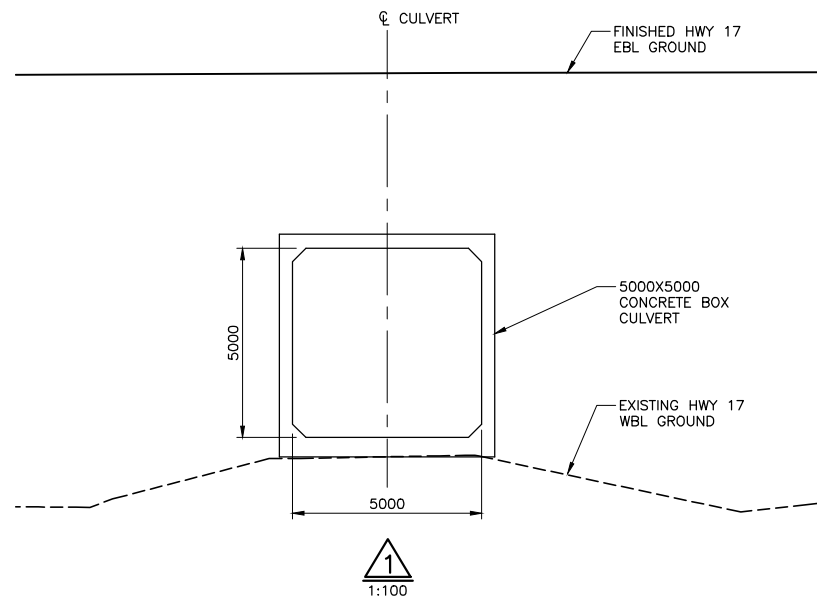


PLAN
1:300

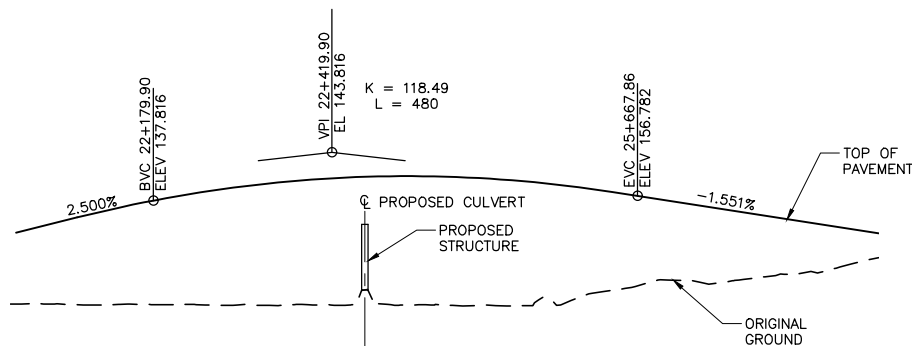


ELEVATION
1:300

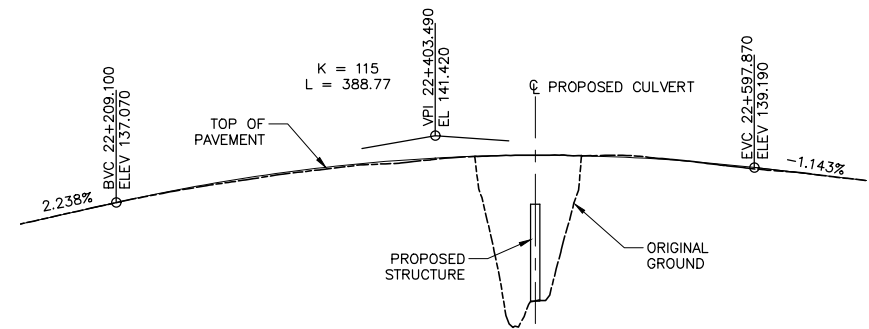
(* MARKED DIMENSIONS MEASURED SQUARE TO HWY 17 CENTRE LINE)



(EBL CROSS SECTION SHOWN, WBL CROSS SECTION SIMILAR, UNLESS NOTED OTHERWISE)



PROPOSED HWY 17 EBL PROFILE
NTS



EXISTING HWY 17 WBL PROFILE
NTS

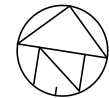
METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN
DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

PARSONS

CONT No
WP

HWY 17 TWINNING
C.P.R. CULVERTS

GENERAL ARRANGEMENT



SHEET
-

GENERAL NOTES:

- CLASS OF CONCRETE:
PRECAST CONCRETE 45 MPa
MASS CONCRETE 20 MPa
REMAINDER 30 MPa
UNLESS OTHERWISE NOTED
- CLEAR COVER TO REINFORCING STEEL:
CULVERT TOP SURFACE 50±10
CULVERT INSIDE WALLS 50±10
REMAINDER 45±10
UNLESS OTHERWISE NOTED
- REINFORCING STEEL
REINFORCING STEEL SHALL BE GRADE 400W UNLESS OTHERWISE SPECIFIED.
UNLESS SHOWN OTHERWISE, TENSION LAP SPLICES SHALL BE CLASS B.
BAR HOOKS SHALL HAVE STANDARD HOOK DIMENSIONS USING MINIMUM BEND DIAMETERS, WHILE STIRRUPS AND TIES SHALL HAVE MINIMUM HOOK DIMENSIONS. ALL HOOKS SHALL BE IN ACCORDANCE WITH THE STRUCTURAL STANDARD DRAWING SS12-1 UNLESS INDICATED OTHERWISE.

CONSTRUCTION NOTES:

- BACKFILL SHALL BE PLACED SIMULTANEOUSLY BEHIND BOTH CONCRETE WALLS KEEPING THE HEIGHT OF THE BACKFILL APPROXIMATELY THE SAME. AT NO TIME SHALL THE DIFFERENCE IN ELEVATION BE GREATER THAN 500mm.
- TEMPORARY ROADWAY PROTECTION SHALL BE DESIGNED TO PERFORMANCE LEVEL 2 CRITERIA.

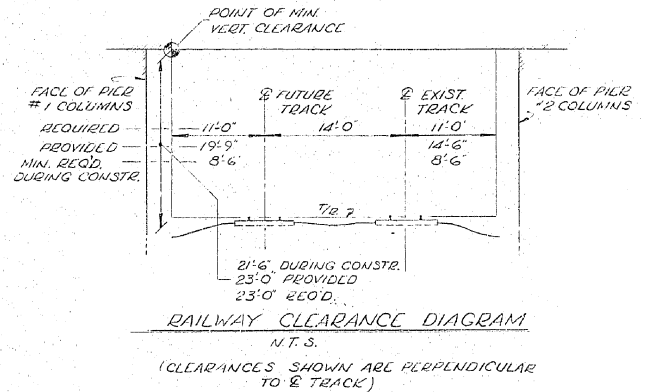
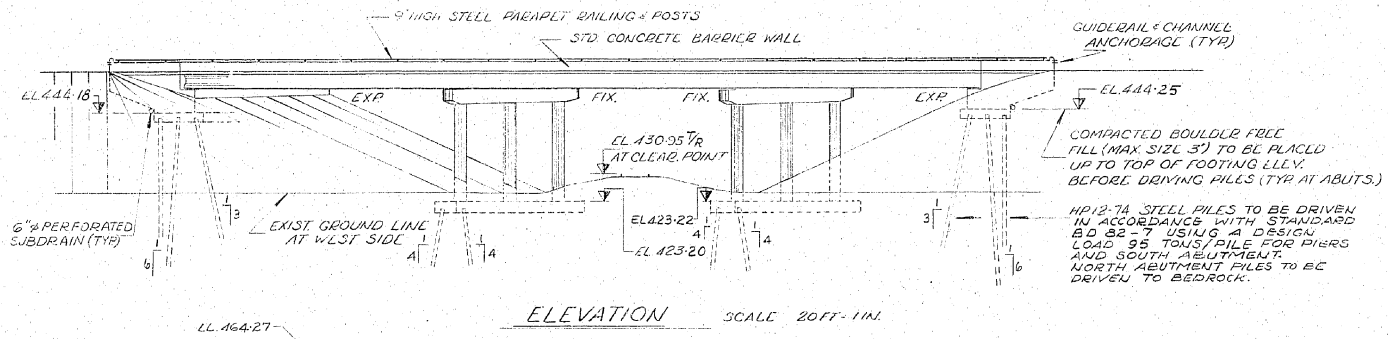
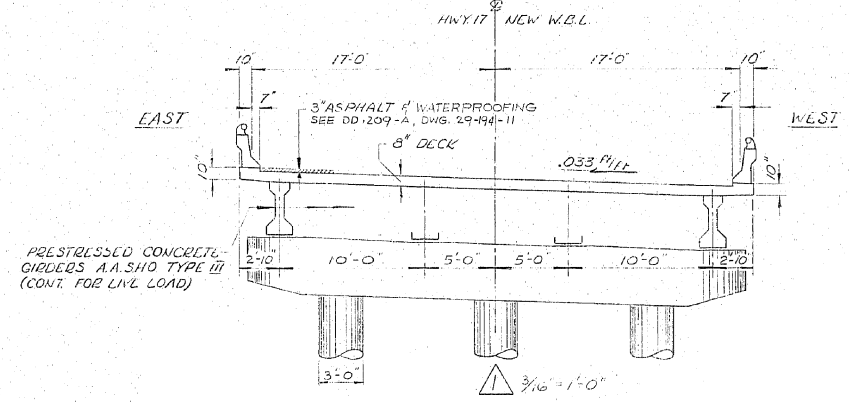
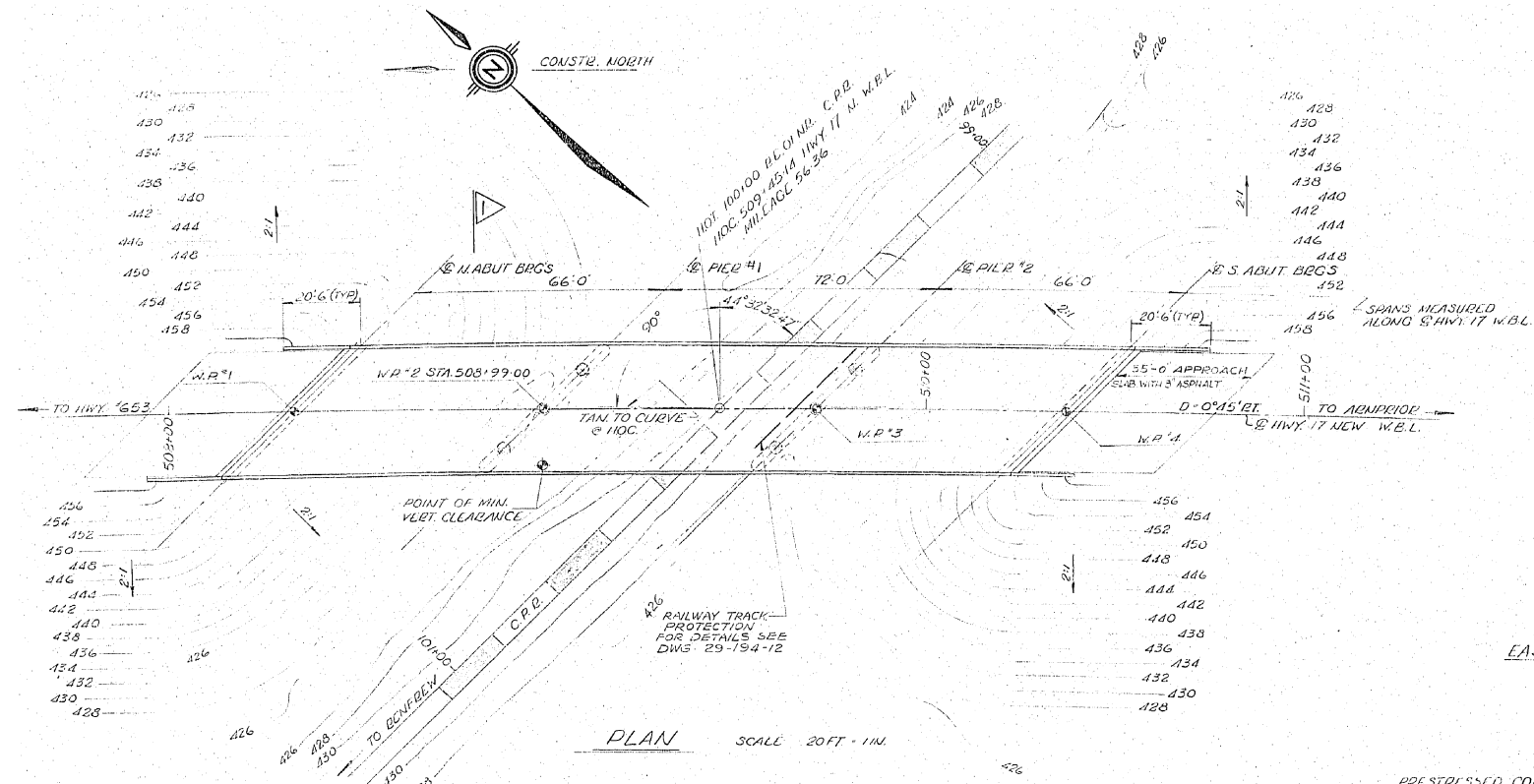
APPLICABLE STANDARD DRAWINGS:

- | | |
|---------------|---|
| OPSD 3101.150 | WALLS, ABUTMENT, BACKFILL, MINIMUM GRANULAR REQUIREMENT |
| OPSD 3370.100 | DECK, WATERPROOFING HOT APPLIED ASPHALT MEMBRANE WITH PROTECTION BOARD |
| OPSD 3370.101 | DECK, WATERPROOFING HOT APPLIED ASPHALT MEMBRANE AT ACTIVE CRACKS GREATER THAN 2mm WIDE AND CONSTRUCTION JOINTS |
| MTOD 3941.210 | FIGURES IN CONCRETE, SITE NUMBER AND DATE, LAYOUT |
| OPSD-3329.100 | DECK, REINFORCEMENT, SUPPORTS FOR REINFORCING STEEL FOR SLAB DEPTHS 300mm OR LESS |
| OPSD-3329.101 | DECK, REINFORCEMENT, SUPPORTS FOR REINFORCING STEEL FOR SLAB DEPTHS GRATER THAN 300mm |

REV	REVISIONS			
	NO	DATE	BY	DESCRIPTION
DESIGN	AL	CHK	CODE	CAN/CSA S6-19
DRAWN	FP	CHK	AL	SITE 29X-0194B1&B2
			LOAD	CL-625-ONT
			DATE	DWG

DOCUMENT CODE:

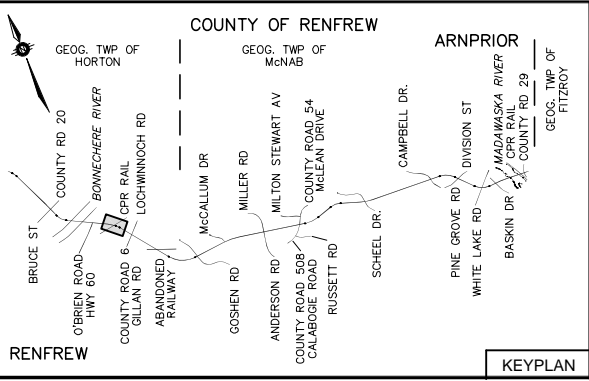
29-194.dwg - 26 August 2004 - 13:02:51



PROFILE OF HWY 17 (NEW) W.B.L. N.T.S.

PROFILE OF C.P.R. N.T.S.

REFERENCE DRAWINGS:
THIS SKETCH IS A PARTIAL COPY OF THE GENERAL PLAN (SHEET 49) OF THE CANADIAN PACIFIC RAILWAY OVERHEAD, APPROXIMATELY 2.6 MILES (4.2 km) EAST OF RENFREW, SITE No. 29-194, KING'S HIGHWAY No. 17 (NEW) W.B.L., DATED SEPTEMBER 1972. IT IS INTENDED ONLY TO ILLUSTRATE THE TYPE OF STRUCTURE WHICH EXISTS AT THIS SITE.



GENERAL ARRANGEMENT

C.P.R. OVERHEAD

HIGHWAY 17 TWINNING, COUNTY ROAD 29 TO 3.0 km WEST OF BRUCE STREET
PRELIMINARY DESIGN/ ENVIRONMENTAL ASSESSMENT STUDY
WP 647-92-00

SCALE: N.T.S.






Ministry of Transportation

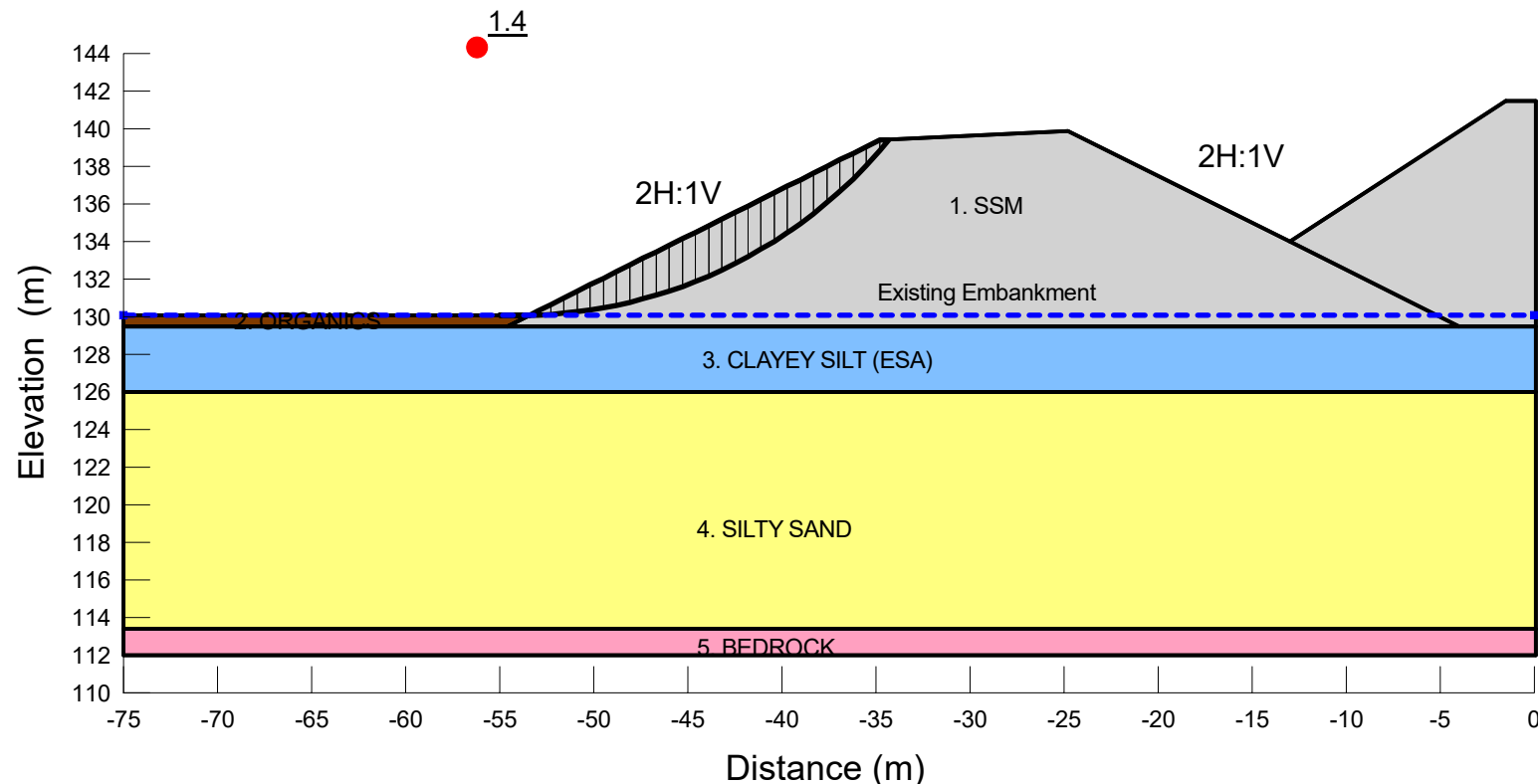
engineers architects planners



Appendix H.

Slope Stability Analysis Figures






Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	1. SSM FILL	Mohr-Coulomb	21	0	32
	2. ORGANICS	Mohr-Coulomb	11	10	10
	3. CLAYEY SILT (ESA)	Mohr-Coulomb	17.5	5	28
	4. SILTY SAND	Mohr-Coulomb	21	0	35
	5. BEDROCK	Bedrock (Impenetrable)			

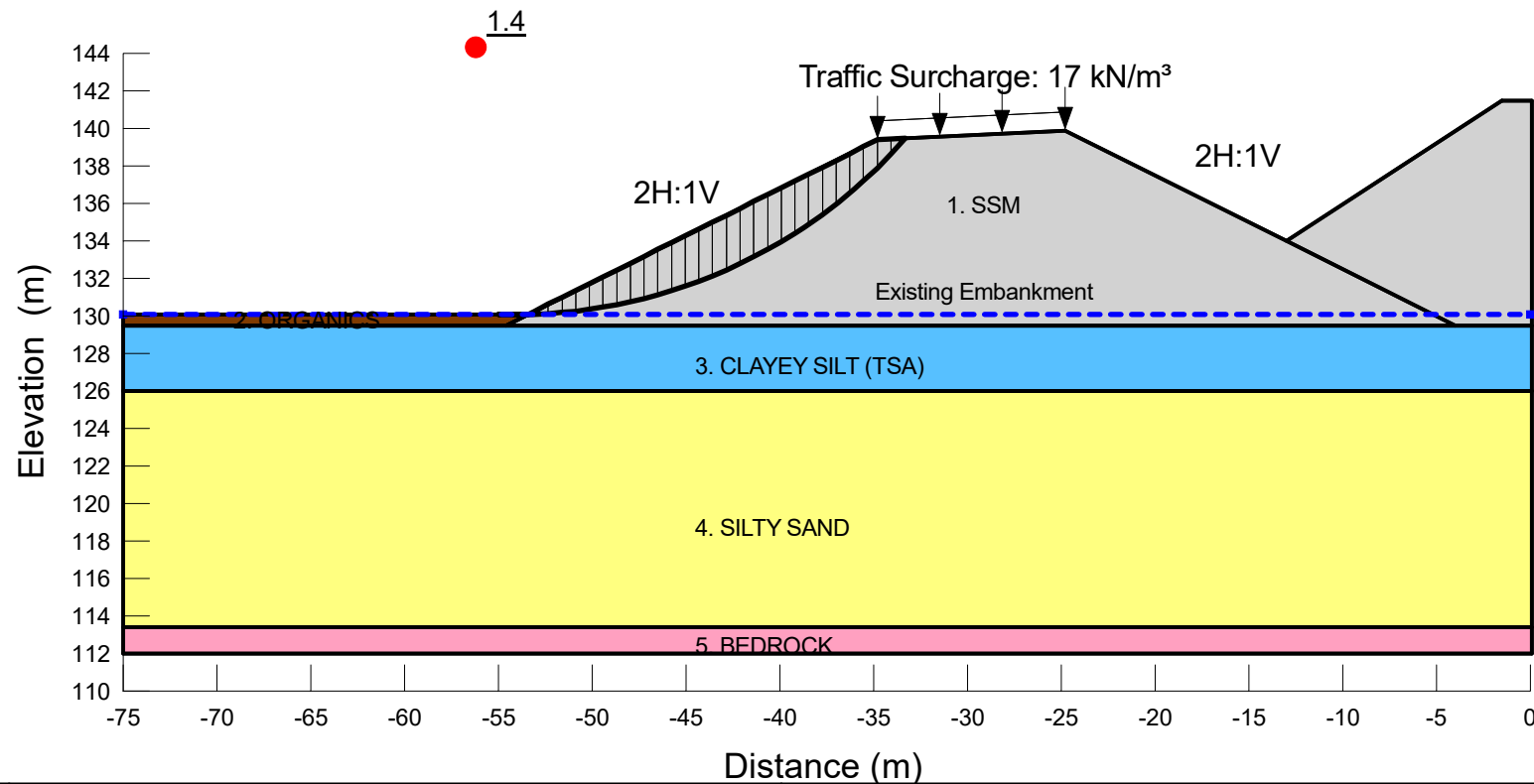


Project		
CPR Westbound Lanes Embankment		
Analysis		
1.1 Permanent Long Term (Drained)		
Seismic Coefficient	Last Run	Scale
H: 0g, V: 0g	2022/07/14, 04:53:45 PM	1:400

Additional Details
Name: 1 CPR Culvert: 2H:1V Earth Embankment
Comments:
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
Entry: (-34.3, 139.425) m, Exit: (-55, 130.1) m
Center: (-53.982432, 155.479) m, Radius: 25.399389 m

Figure H1.1

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	1. SSM FILL	Mohr-Coulomb	21	0	32
	2. ORGANICS	Mohr-Coulomb	11	10	10
	3. CLAYEY SILT (TSA)	Mohr-Coulomb	17.5	90	0
	4. SILTY SAND	Mohr-Coulomb	21	0	35
	5. BEDROCK	Bedrock (Impenetrable)			



Project		
CPR Westbound Lanes Embankment		
Analysis		
1.2 Temporary Short Term - Traffic (Undrained)		
Seismic Coefficient	Last Run	Scale
H: 0g, V: 0g	2022/07/14, 04:53:53 PM	1:400

Additional Details

Name: 1 CPR Culvert: 2H:1V Earth Embankment

Comments:






Method: Morgenstern-Price, Half-Sine

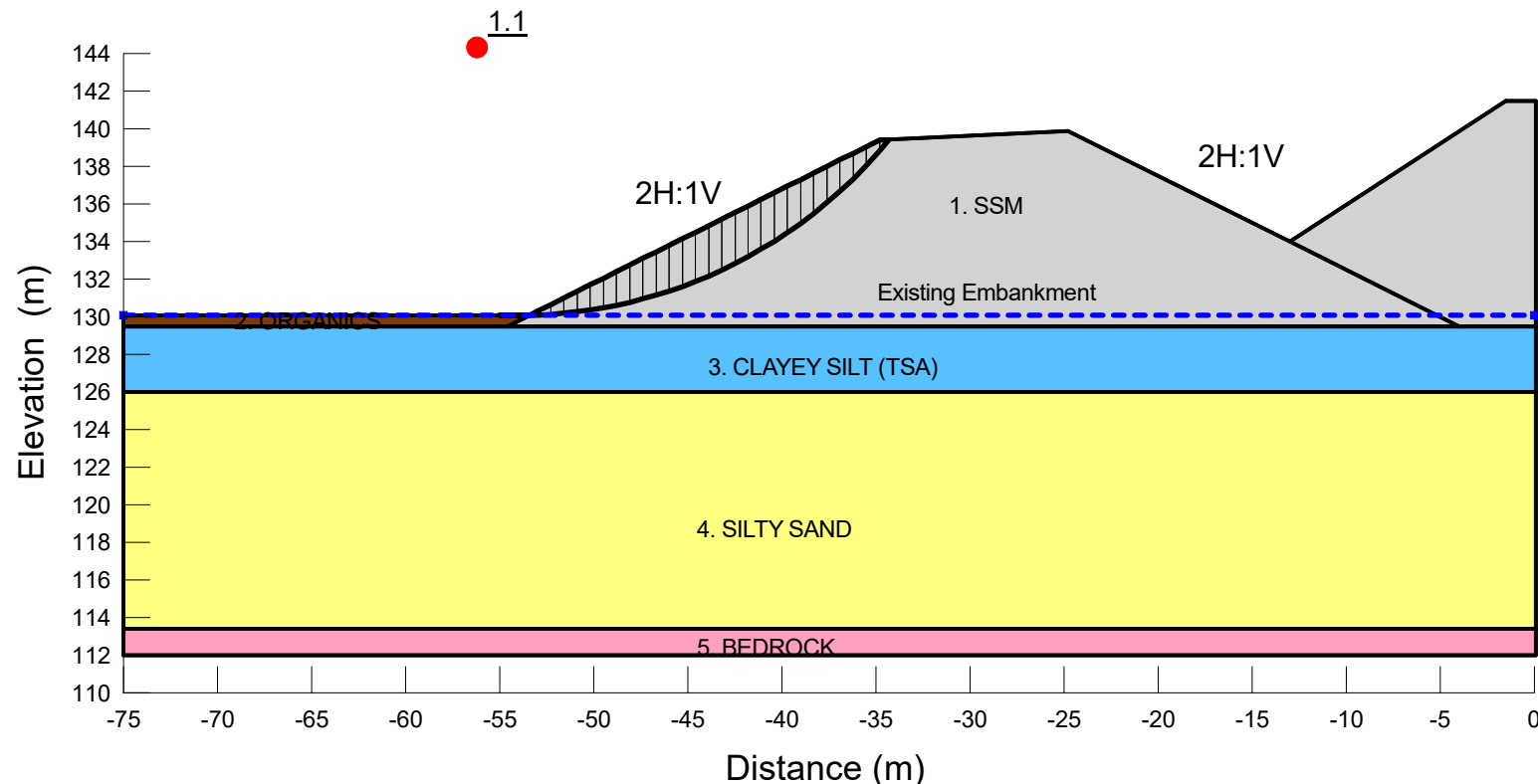
Minimum Slip Surface Depth: 1.52 m

Entry: (-33.3, 139.475) m, Exit: (-55, 130.1) m

Center: (-54.033133, 157.66366) m, Radius: 27.580612 m

Figure H1.2

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	1. SSM FILL	Mohr-Coulomb	21	0	32
	2. ORGANICS	Mohr-Coulomb	11	10	10
	3. CLAYEY SILT (TSA)	Mohr-Coulomb	17.5	90	0
	4. SILTY SAND	Mohr-Coulomb	21	0	35
	5. BEDROCK	Bedrock (Impenetrable)			



Project		
CPR Westbound Lanes Embankment		
Analysis		
1.3 Temporary - Pseudo-Static 2,475-yr (Undrained)		
Seismic Coefficient	Last Run	Scale
H: 0.113g, V: 0g	2022/07/14, 04:53:49 PM	1:400

Additional Details

Name: 1 CPR Culvert: 2H:1V Earth Embankment

Comments:






Method: Morgenstern-Price, Half-Sine

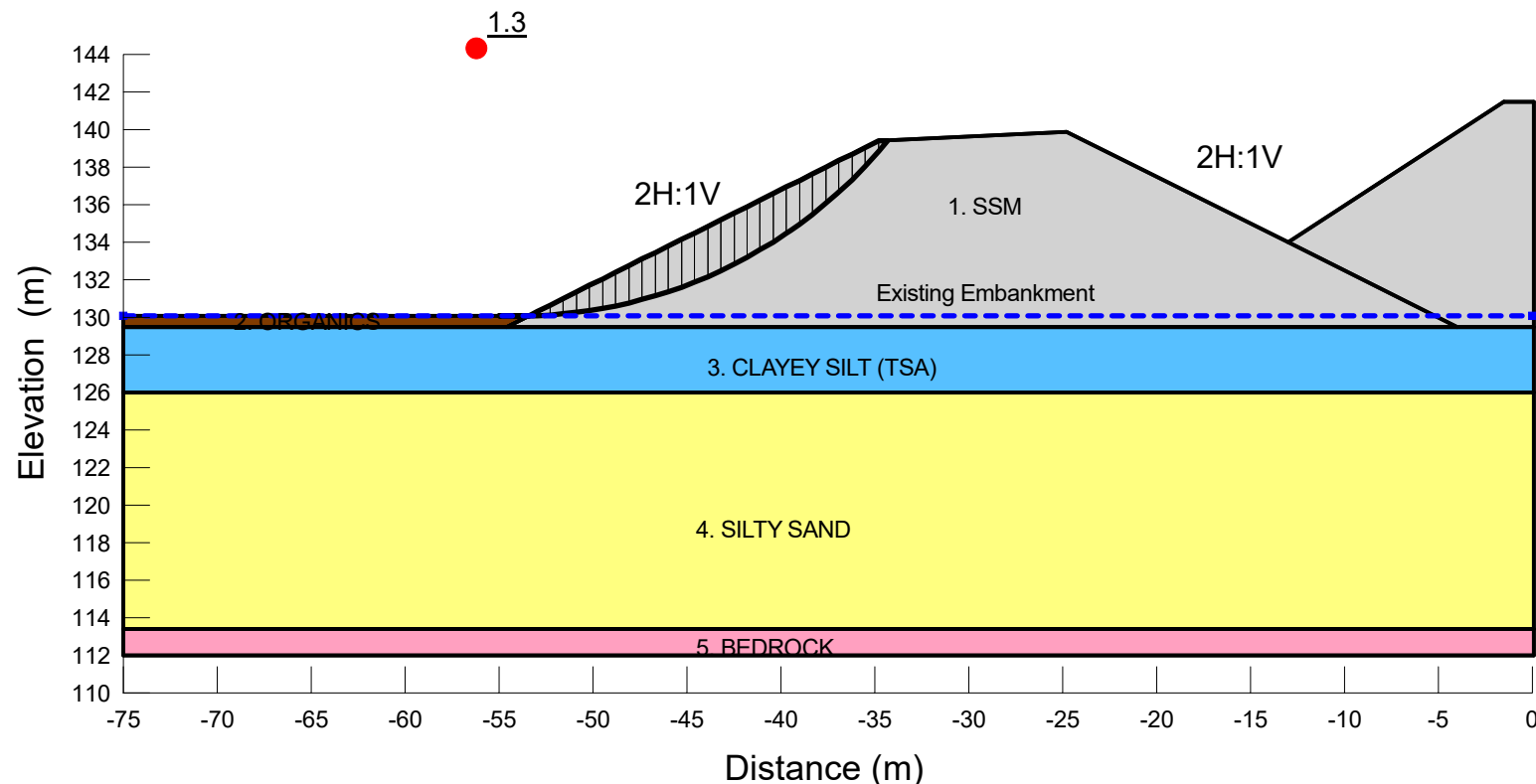
Minimum Slip Surface Depth: 1.52 m

Entry: (-34.3, 139.425) m, Exit: (-55, 130.1) m

Center: (-53.982432, 155.479) m, Radius: 25.399389 m

Figure H1.3






Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	1. SSM FILL	Mohr-Coulomb	21	0	32
	2. ORGANICS	Mohr-Coulomb	11	10	10
	3. CLAYEY SILT (TSA)	Mohr-Coulomb	17.5	90	0
	4. SILTY SAND	Mohr-Coulomb	21	0	35
	5. BEDROCK	Bedrock (Impenetrable)			

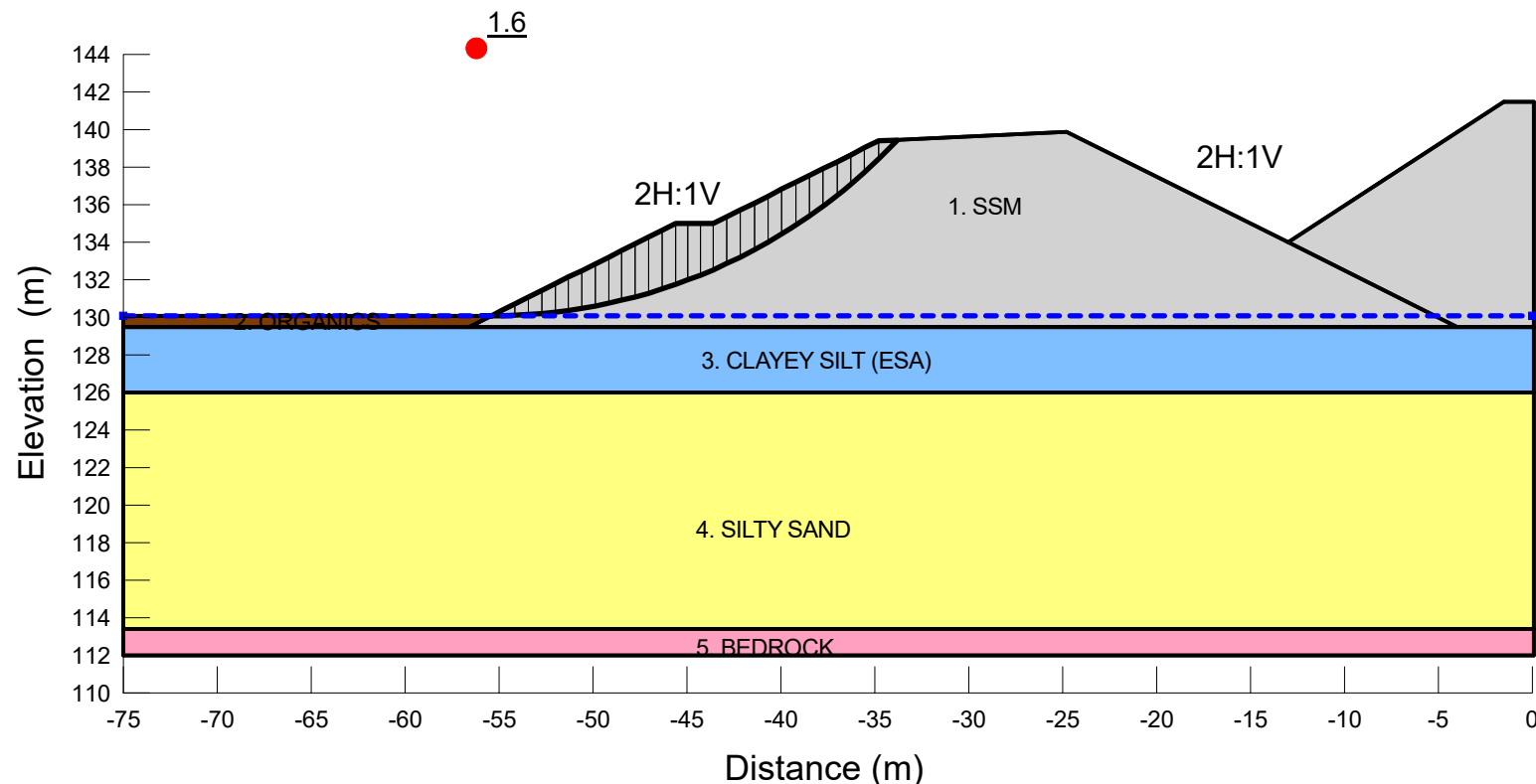


Project CPR Westbound Lanes Embankment		
Analysis 1.4 Temporary - Pseudo-Static 475-yr (Undrained)		
Seismic Coefficient H: 0.04g, V: 0g	Last Run 2022/07/14, 04:53:57 PM	Scale 1:400

Additional Details
Name: 1 CPR Culvert: 2H:1V Earth Embankment
Comments:
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
Entry: (-34.3, 139.425) m, Exit: (-55, 130.1) m
Center: (-53.982432, 155.479) m, Radius: 25.399389 m

Figure H1.4






Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	1. SSM FILL	Mohr-Coulomb	21	0	32
	2. ORGANICS	Mohr-Coulomb	11	10	10
	3. CLAYEY SILT (ESA)	Mohr-Coulomb	17.5	5	28
	4. SILTY SAND	Mohr-Coulomb	21	0	35
	5. BEDROCK	Bedrock (Impenetrable)			

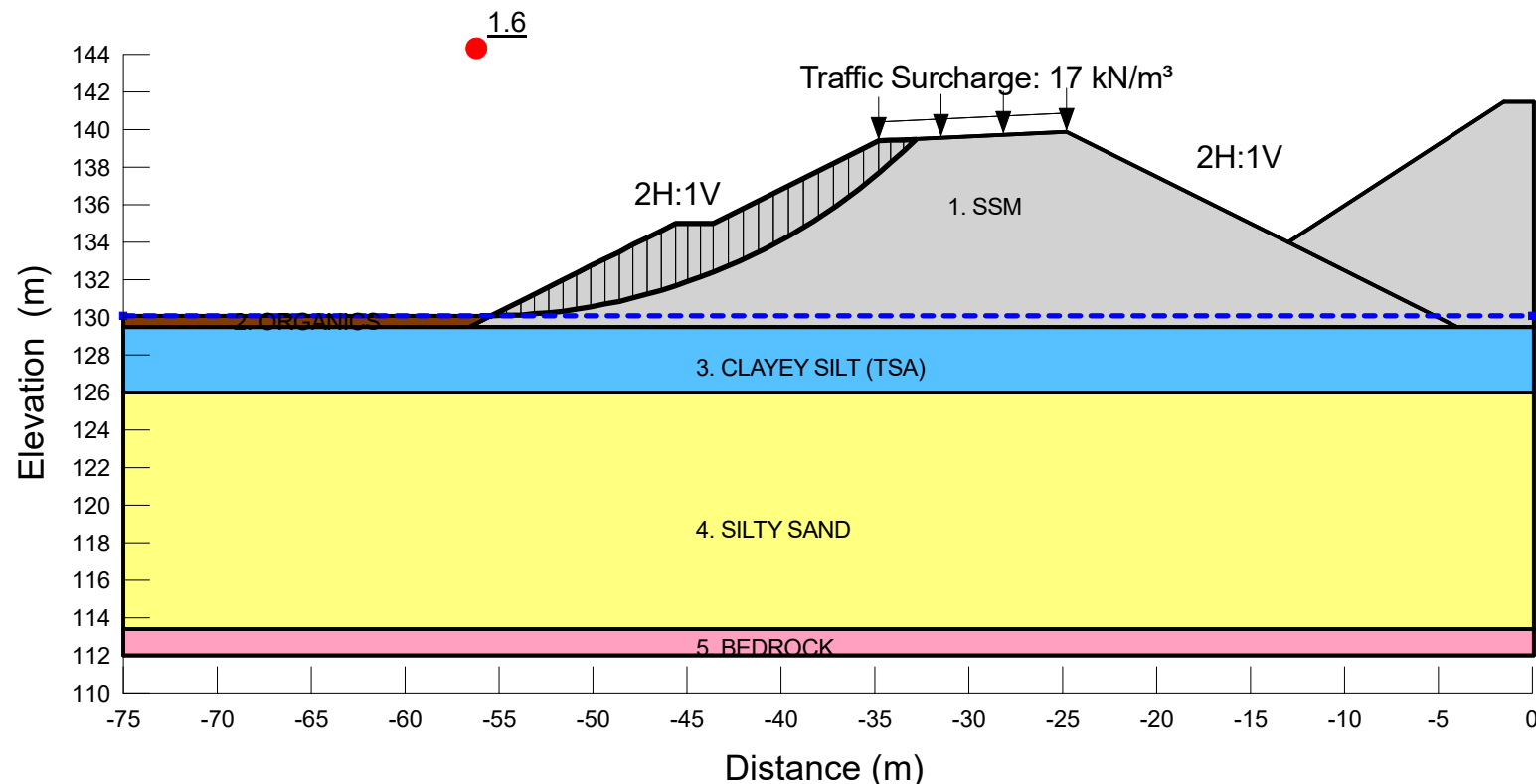


Project		
CPR Westbound Lanes Embankment		
Analysis		
2.1 Permanent Long Term (Drained)		
Seismic Coefficient	Last Run	Scale
H: 0g, V: 0g	2022/07/14, 04:54:31 PM	1:400

Additional Details
Name: 2 CPR Culvert: 2H:1V Earth Embankment (Benched)
Comments:
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
Entry: (-33.8, 139.45) m, Exit: (-55.9, 130.1) m
Center: (-55.376255, 159.65524) m, Radius: 29.55988 m

Figure H2.1






Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	1. SSM FILL	Mohr-Coulomb	21	0	32
	2. ORGANICS	Mohr-Coulomb	11	10	10
	3. CLAYEY SILT (TSA)	Mohr-Coulomb	17.5	90	0
	4. SILTY SAND	Mohr-Coulomb	21	0	35
	5. BEDROCK	Bedrock (Impenetrable)			

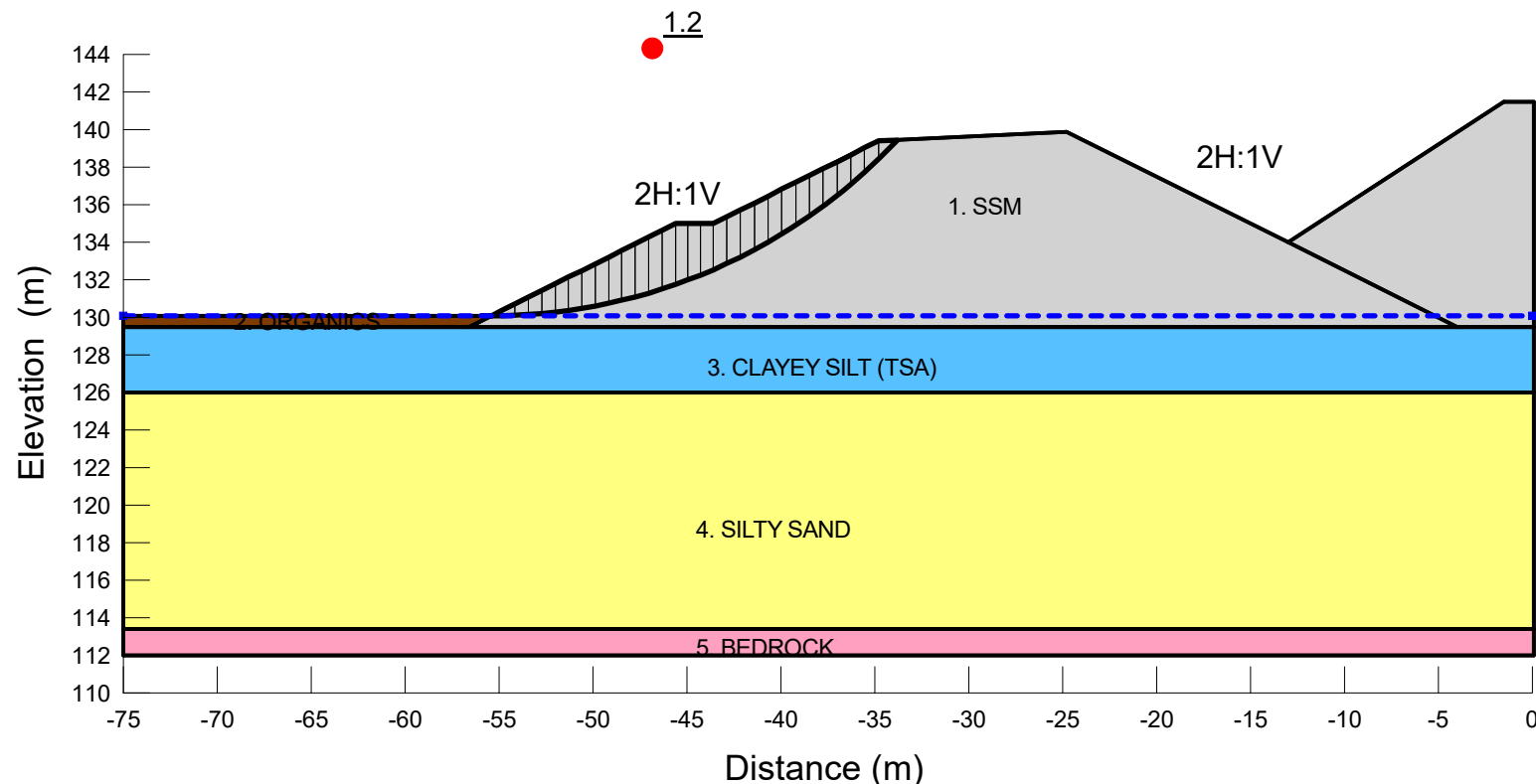


Project		
CPR Westbound Lanes Embankment		
Analysis		
2.2 Temporary Short Term - Traffic (Undrained)		
Seismic Coefficient	Last Run	Scale
H: 0g, V: 0g	2022/07/14, 04:54:35 PM	1:400

Additional Details
Name: 2 CPR Culvert: 2H:1V Earth Embankment (Benched)
Comments:
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
Entry: (-32.8, 139.5) m, Exit: (-55.9, 130.1) m
Center: (-55.622203, 162.50084) m, Radius: 32.40203 m

Figure H2.2






Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	1. SSM FILL	Mohr-Coulomb	21	0	32
	2. ORGANICS	Mohr-Coulomb	11	10	10
	3. CLAYEY SILT (TSA)	Mohr-Coulomb	17.5	90	0
	4. SILTY SAND	Mohr-Coulomb	21	0	35
	5. BEDROCK	Bedrock (Impenetrable)			

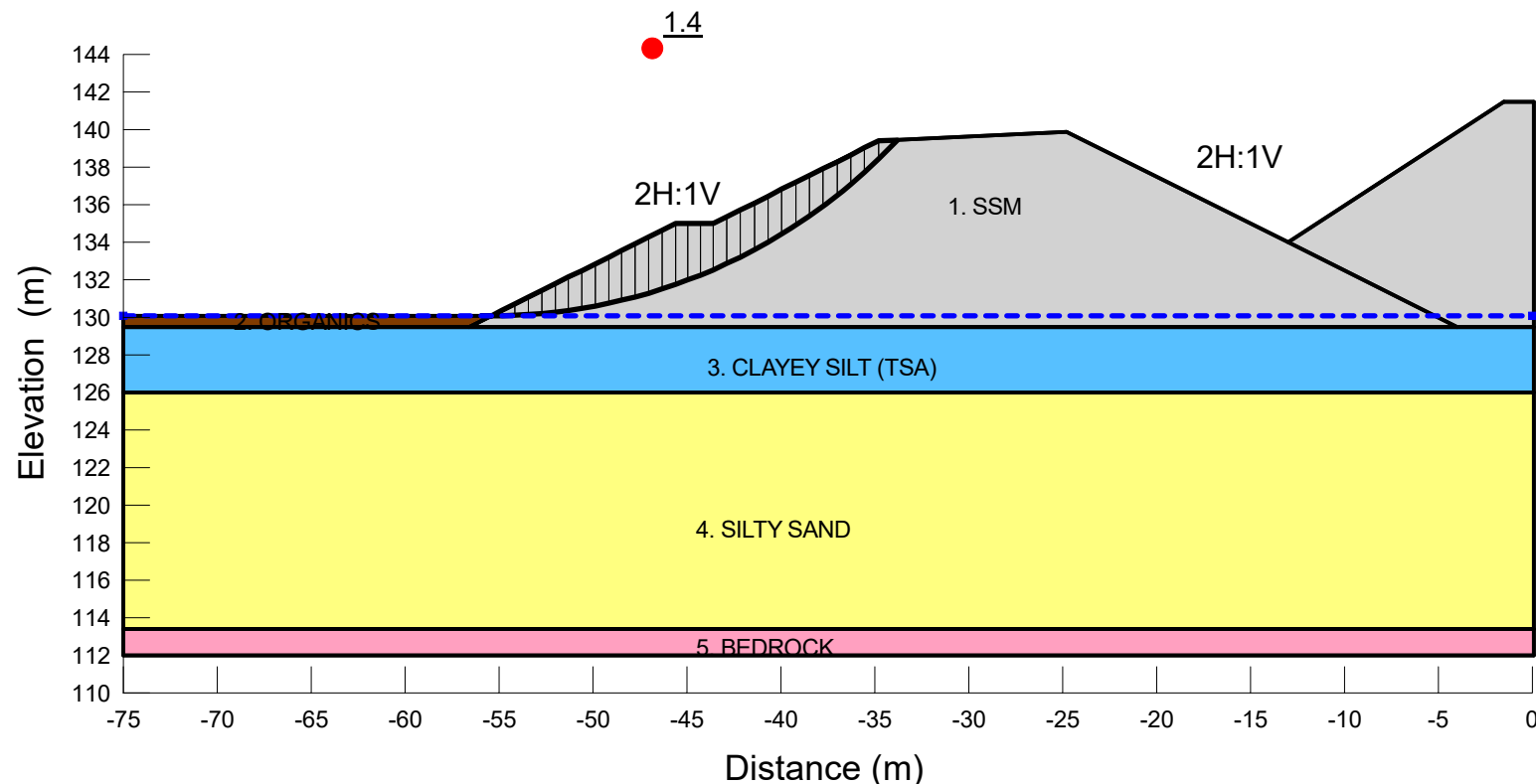


Project		
CPR Westbound Lanes Embankment		
Analysis		
2.3 Temporary - Pseudo-Static 2,475-yr (Undrained)		
Seismic Coefficient	Last Run	Scale
H: 0.113g, V: 0g	2022/07/14, 04:55:04 PM	1:400

Additional Details
Name: 2 CPR Culvert: 2H:1V Earth Embankment (Benched)
Comments:
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
Entry: (-33.8, 139.45) m, Exit: (-55.9, 130.1) m
Center: (-55.376255, 159.65524) m, Radius: 29.55988 m

Figure H2.3

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	1. SSM FILL	Mohr-Coulomb	21	0	32
	2. ORGANICS	Mohr-Coulomb	11	10	10
	3. CLAYEY SILT (TSA)	Mohr-Coulomb	17.5	90	0
	4. SILTY SAND	Mohr-Coulomb	21	0	35
	5. BEDROCK	Bedrock (Impenetrable)			

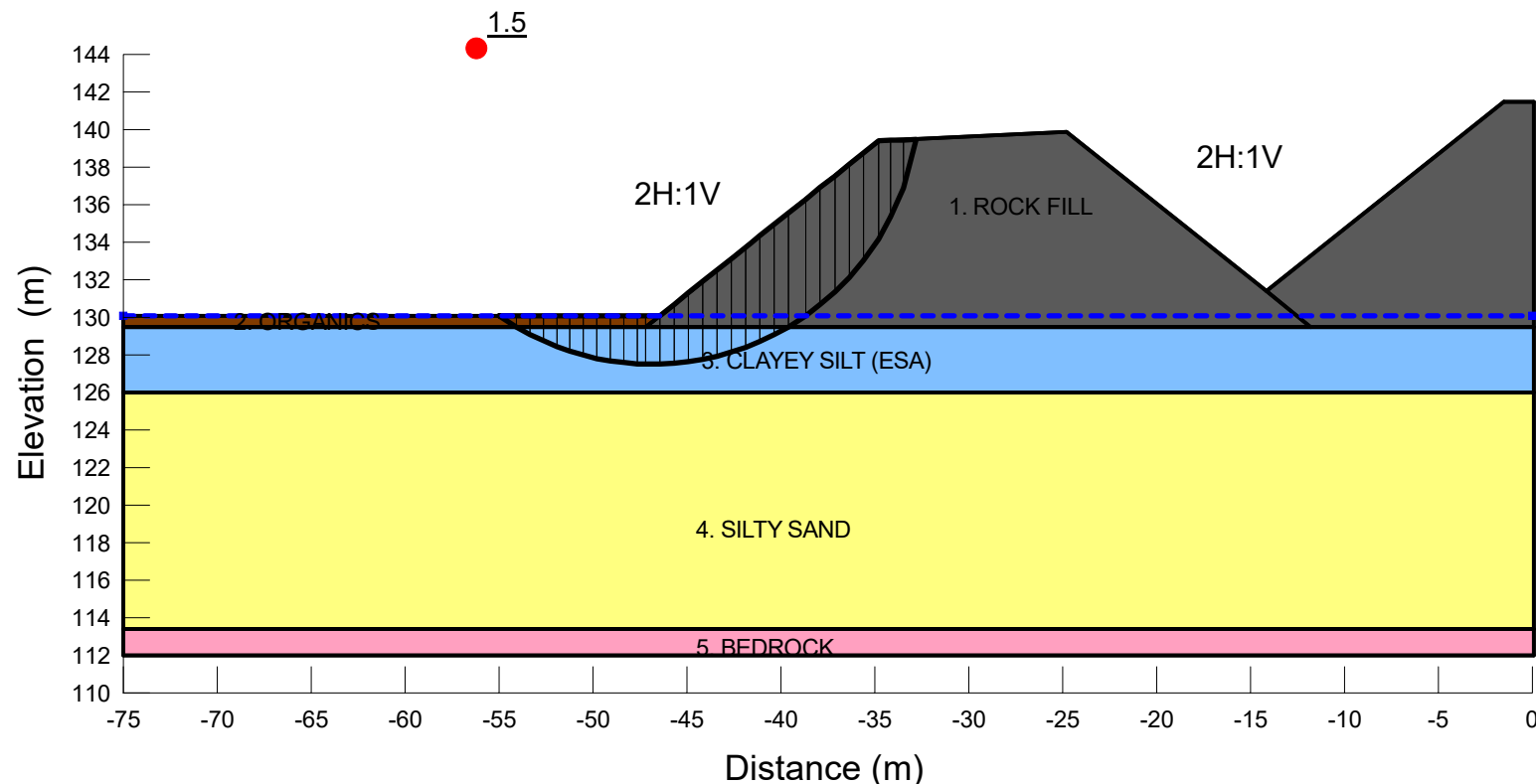


Project		
CPR Westbound Lanes Embankment		
Analysis		
2.4 Temporary - Pseudo-Static 475-yr (Undrained)		
Seismic Coefficient	Last Run	Scale
H: 0.04g, V: 0g	2022/07/14, 04:55:04 PM	1:400

Additional Details
Name: 2 CPR Culvert: 2H:1V Earth Embankment (Benched)
Comments:
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
Entry: (-33.8, 139.45) m, Exit: (-55.9, 130.1) m
Center: (-55.376255, 159.65524) m, Radius: 29.55988 m

Figure H2.4

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
■	1. ROCK FILL	Mohr-Coulomb	20	0	42
■	2. ORGANICS	Mohr-Coulomb	11	10	10
■	3. CLAYEY SILT (ESA)	Mohr-Coulomb	17.5	5	28
■	4. SILTY SAND	Mohr-Coulomb	21	0	35
■	5. BEDROCK	Bedrock (Impenetrable)			

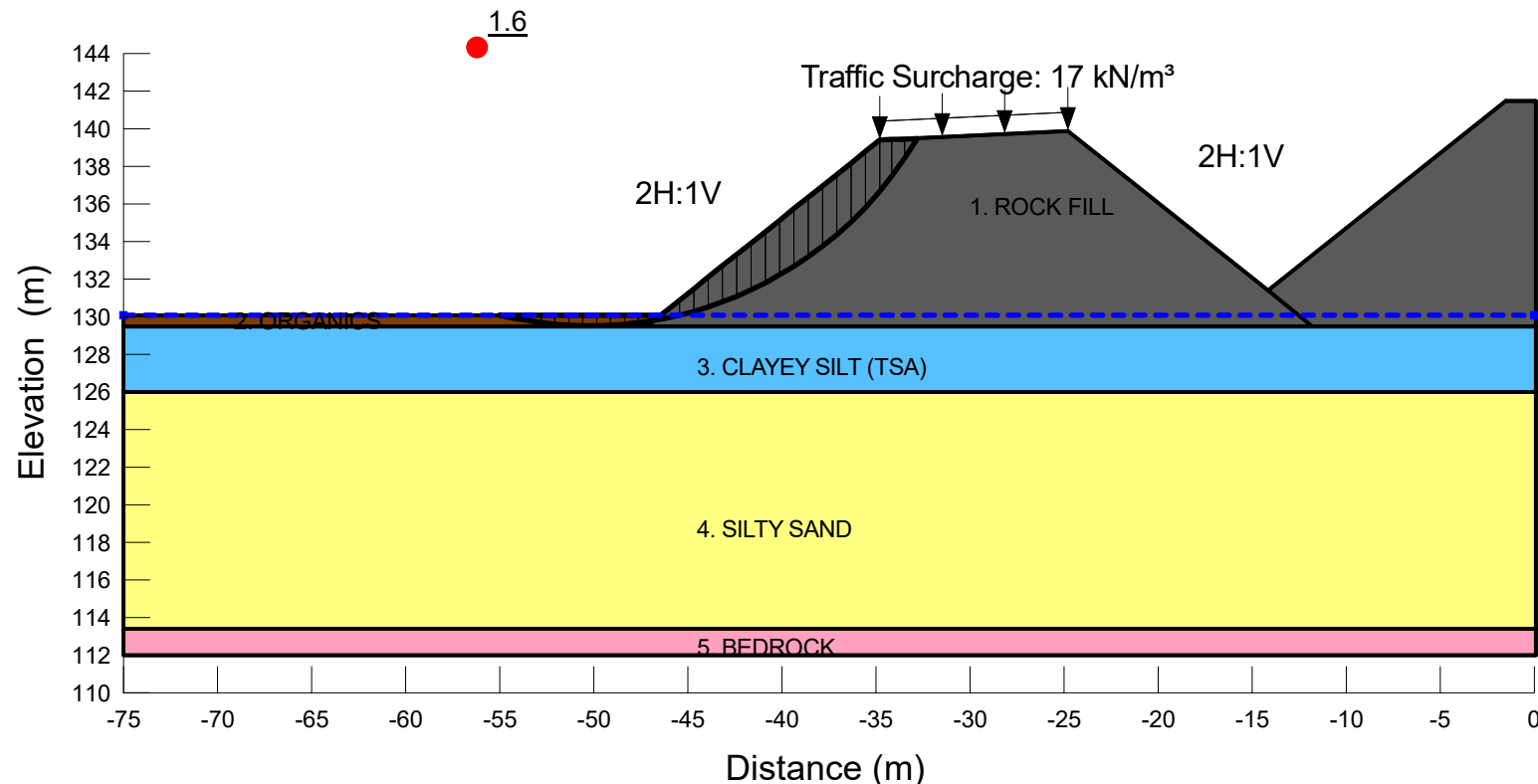


Project CPR Westbound Lanes Embankment		
Analysis 3.1 Permanent Long Term (Drained)		
Seismic Coefficient H: 0g, V: 0g	Last Run 2022/07/14, 04:54:39 PM	Scale 1:400

Additional Details
Name: 3 CPR Culvert: 125H:1V Rock Fill Embankment
Comments:
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
Entry: (-32.8, 139.5) m, Exit: (-55, 130.1) m
Center: (-46.831322, 141.72291) m, Radius: 14.206312 m

Figure H3.1

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
■	1. ROCK FILL	Mohr-Coulomb	20	0	42
■	2. ORGANICS	Mohr-Coulomb	11	10	10
■	3. CLAYEY SILT (TSA)	Mohr-Coulomb	17.5	90	0
■	4. SILTY SAND	Mohr-Coulomb	21	0	35
■	5. BEDROCK	Bedrock (Impenetrable)			

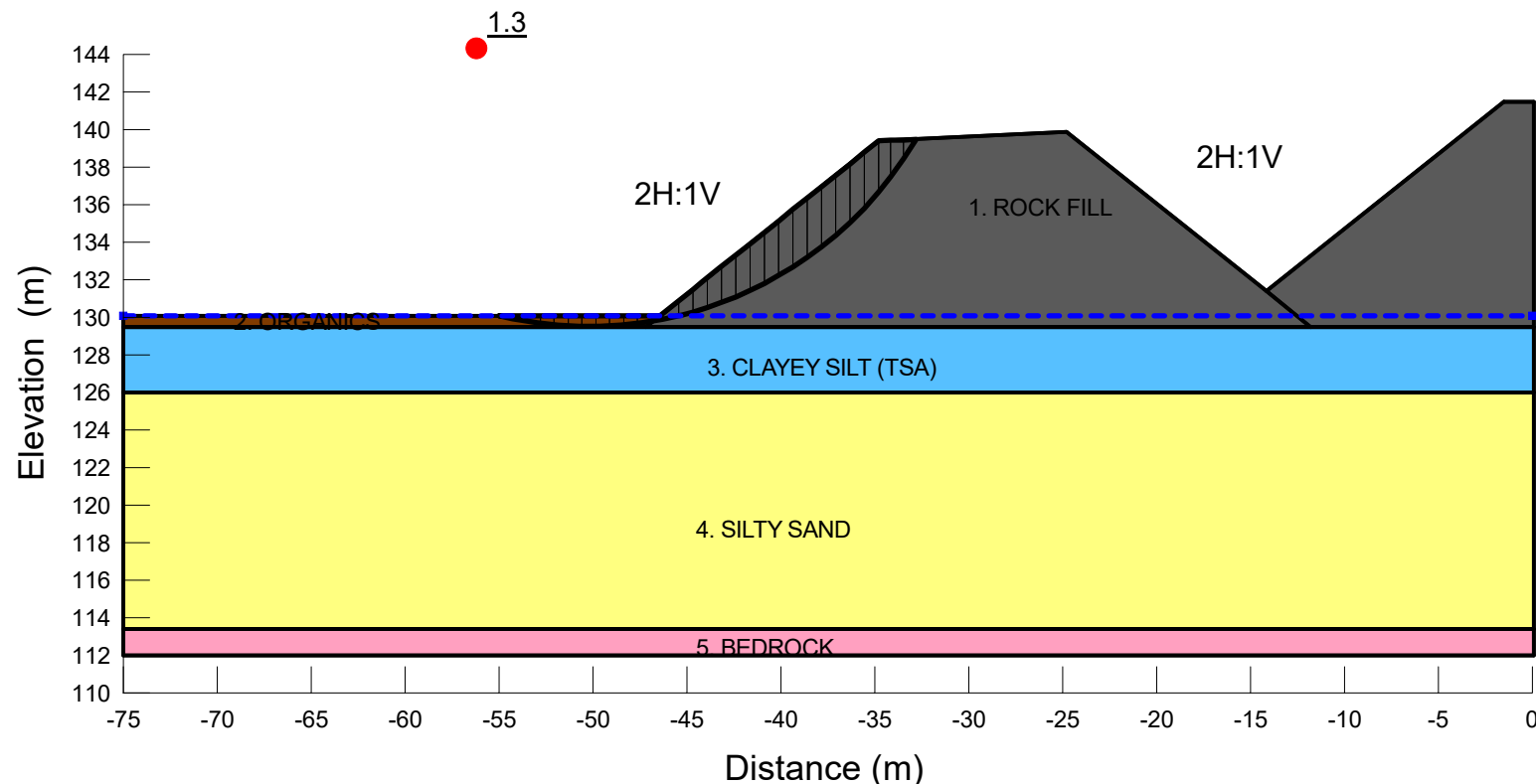


Project		
CPR Westbound Lanes Embankment		
Analysis		
3.2 Temporary Short Term - Traffic (Undrained)		
Seismic Coefficient	Last Run	Scale
H: 0g, V: 0g	2022/07/14, 04:54:43 PM	1:400

Additional Details
Name: 3 CPR Culvert: 125H:1V Rock Fill Embankment
Comments:
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
Entry: (-32.8, 139.5) m, Exit: (-55, 130.1) m
Center: (-50.215586, 149.71553) m, Radius: 20.190586 m

Figure H3.2

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
■	1. ROCK FILL	Mohr-Coulomb	20	0	42
■	2. ORGANICS	Mohr-Coulomb	11	10	10
■	3. CLAYEY SILT (TSA)	Mohr-Coulomb	17.5	90	0
■	4. SILTY SAND	Mohr-Coulomb	21	0	35
■	5. BEDROCK	Bedrock (Impenetrable)			

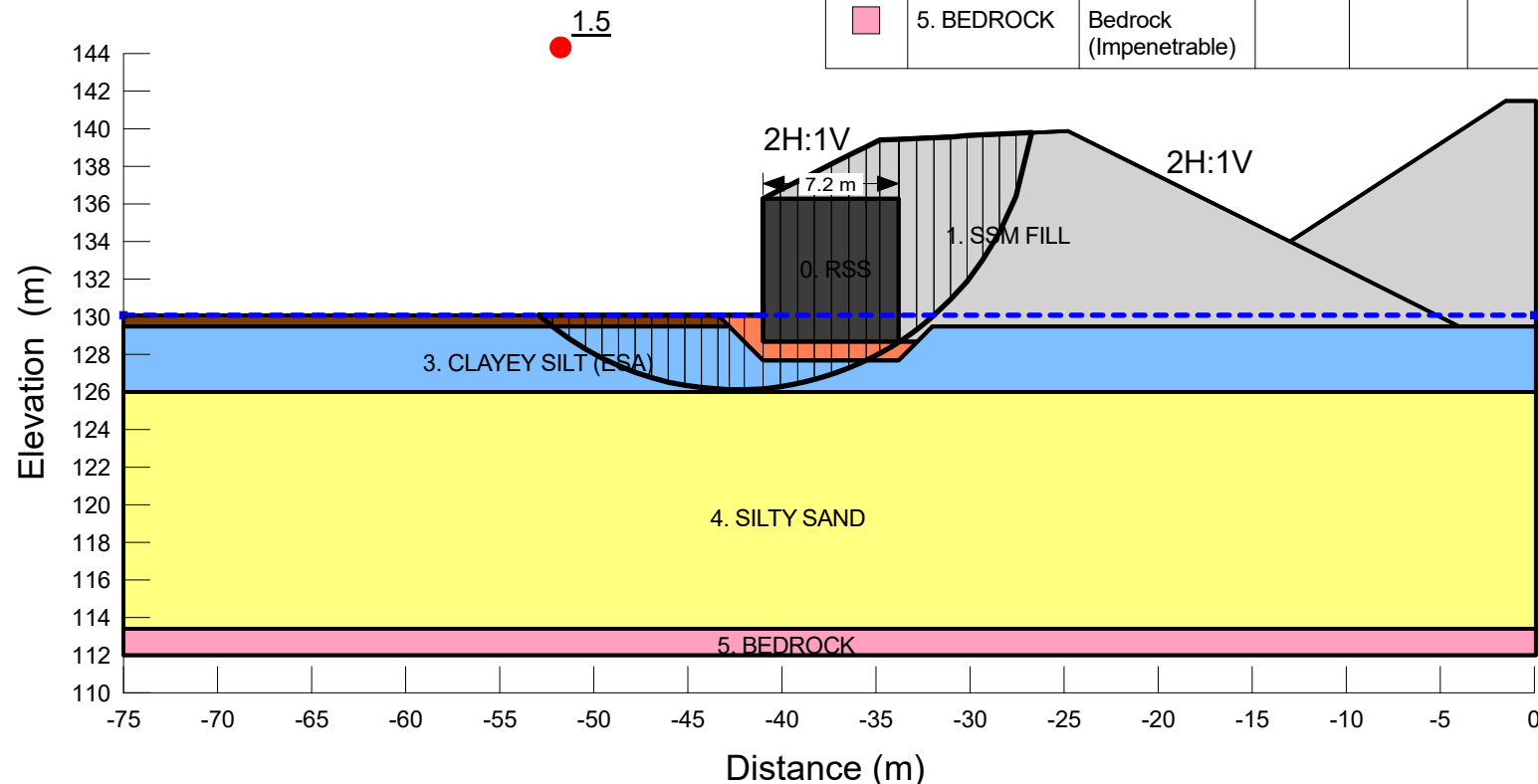


Project		
CPR Westbound Lanes Embankment		
Analysis		
3.3 Temporary - Pseudo-Static 2,475-yr (Undrained)		
Seismic Coefficient	Last Run	Scale
H: 0.113g, V: 0g	2022/07/14, 04:54:47 PM	1:400

Additional Details
Name: 3 CPR Culvert: 125H:1V Rock Fill Embankment
Comments:
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
Entry: (-32.8, 139.5) m, Exit: (-55, 130.1) m
Center: (-50.215586, 149.71553) m, Radius: 20.190586 m

Figure H3.3

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
■	0-GRANULAR A	Mohr-Coulomb	22.8	0	40
■	0-RSS	Mohr-Coulomb	22.8	250	42
■	1. SSM FILL	Mohr-Coulomb	21	0	32
■	2. ORGANICS	Mohr-Coulomb	11	10	10
■	3. CLAYEY SILT (ESA)	Mohr-Coulomb	17.5	5	28
■	4. SILTY SAND	Mohr-Coulomb	21	0	35
■	5. BEDROCK	Bedrock (Impenetrable)			

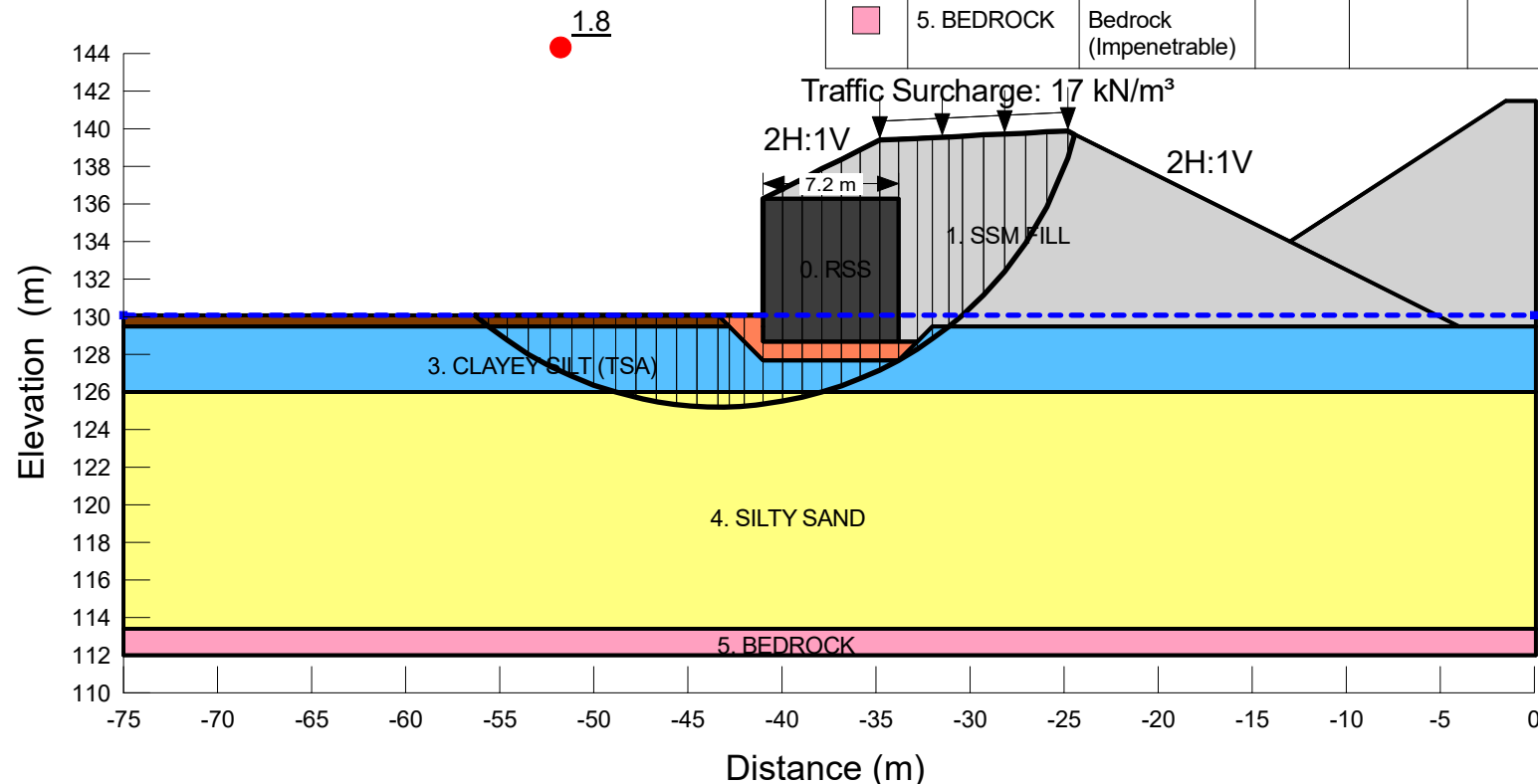


Project CPR Westbound Lanes Embankment		
Analysis 4.1 Permanent Long Term (Drained)		
Seismic Coefficient H: 0g, V: 0g	Last Run 2022/07/14, 04:54:04 PM	Scale 1:400

Additional Details
Name: 4 CPR Culvert: 2H:1V Earth Embankment & RSS Wall
Comments:
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
Entry: (-26.688281, 139.80559) m, Exit: (-52.9, 130.1) m
Center: (-42.39493, 141.9767) m, Radius: 15.855995 m

Figure H4.1








Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
■	0-GRANULAR A	Mohr-Coulomb	22.8	0	40
■	0-RSS	Mohr-Coulomb	22.8	250	42
■	1. SSM FILL	Mohr-Coulomb	21	0	32
■	2. ORGANICS	Mohr-Coulomb	11	10	10
■	3. CLAYEY SILT (TSA)	Mohr-Coulomb	17.5	90	0
■	4. SILTY SAND	Mohr-Coulomb	21	0	35
■	5. BEDROCK	Bedrock (Impenetrable)			

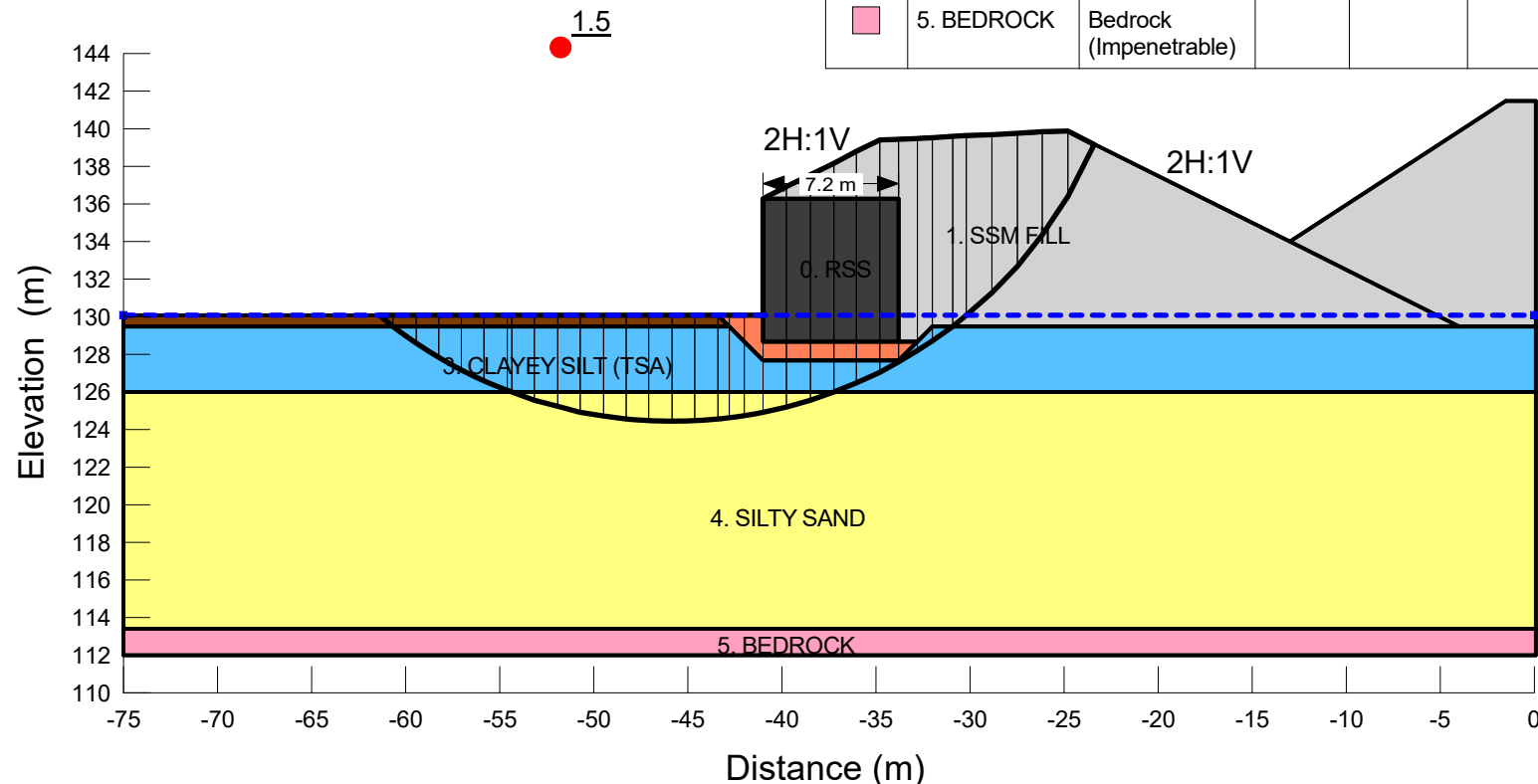


Project		
CPR Westbound Lanes Embankment		
Analysis		
4.2 Temporary Short Term - Traffic (Undrained)		
Seismic Coefficient	Last Run	Scale
H: 0g, V: 0g	2022/07/14, 04:54:11 PM	1:400

Additional Details
Name: 4 CPR Culvert: 2H:1V Earth Embankment & RSS Wall
Comments:
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
Entry: (-24.415495, 139.70775) m, Exit: (-56.3, 130.1) m
Center: (-43.347847, 144.82689) m, Radius: 19.61223 m

Figure H4.2

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	0-GRANULAR A	Mohr-Coulomb	22.8	0	40
	0-RSS	Mohr-Coulomb	22.8	250	42
	1. SSM FILL	Mohr-Coulomb	21	0	32
	2. ORGANICS	Mohr-Coulomb	11	10	10
	3. CLAYEY SILT (TSA)	Mohr-Coulomb	17.5	90	0
	4. SILTY SAND	Mohr-Coulomb	21	0	35
	5. BEDROCK	Bedrock (Impenetrable)			



Project		
CPR Westbound Lanes Embankment		
Analysis		
4.3 Temporary - Pseudo-Static 2,475-yr (Undrained)		
Seismic Coefficient	Last Run	Scale
H: 0.113g, V: 0g	2022/07/14, 04:54:19 PM	1:400

Additional Details

Name: 4 CPR Culvert: 2H:1V Earth Embankment & RSS Wall

Comments:








Method: Morgenstern-Price, Half-Sine

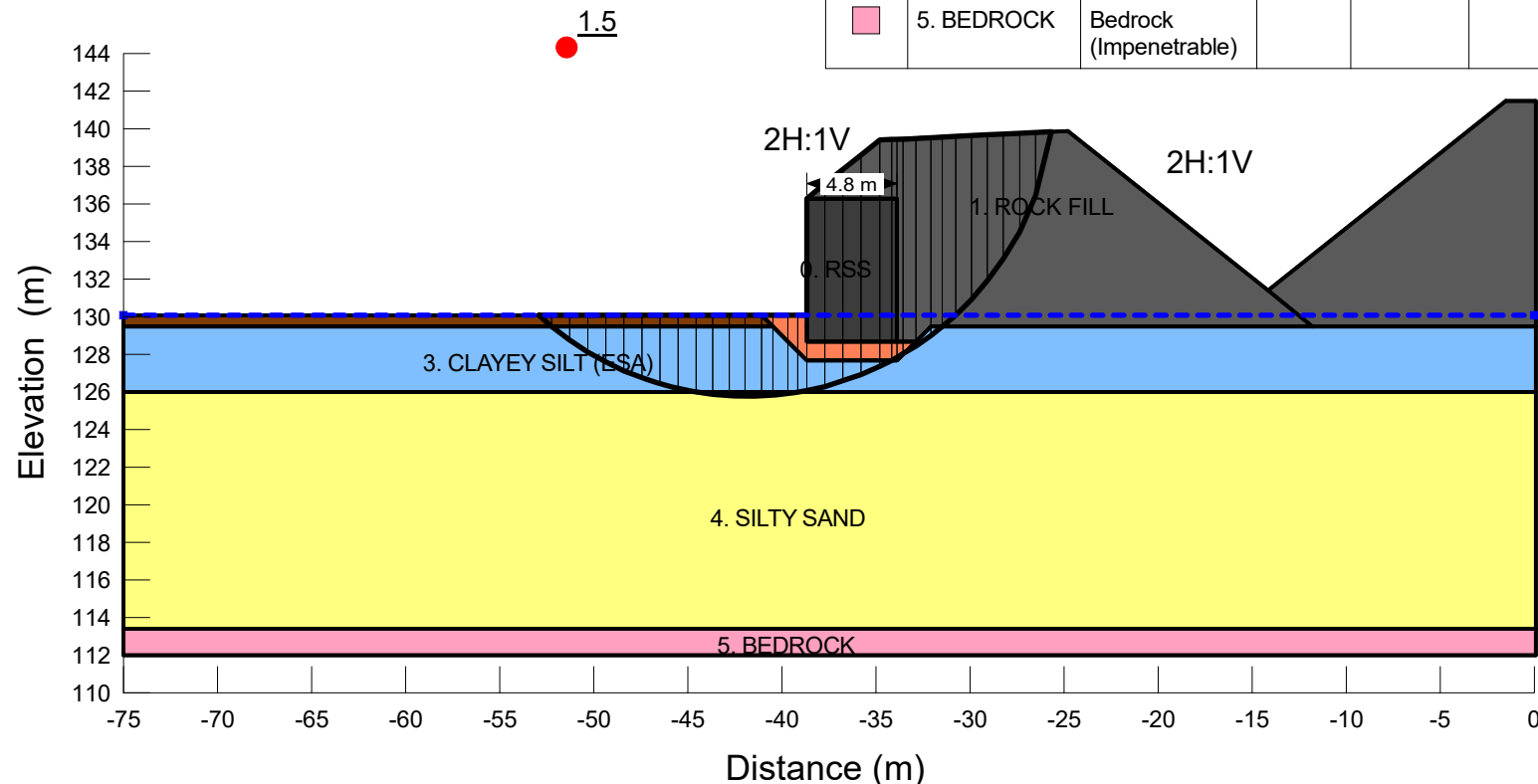
Minimum Slip Surface Depth: 1.52 m

Entry: (-23.377723, 139.18886) m, Exit: (-61.4, 130.1) m

Center: (-45.784716, 148.85063) m, Radius: 24.401292 m

Figure H4.3








Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	0-GRANULAR A	Mohr-Coulomb	22.8	0	40
	0-RSS	Mohr-Coulomb	22.8	250	42
	1. ROCK FILL	Mohr-Coulomb	20	0	42
	2. ORGANICS	Mohr-Coulomb	11	10	10
	3. CLAYEY SILT (ESA)	Mohr-Coulomb	17.5	5	28
	4. SILTY SAND	Mohr-Coulomb	21	0	35
	5. BEDROCK	Bedrock (Impenetrable)			

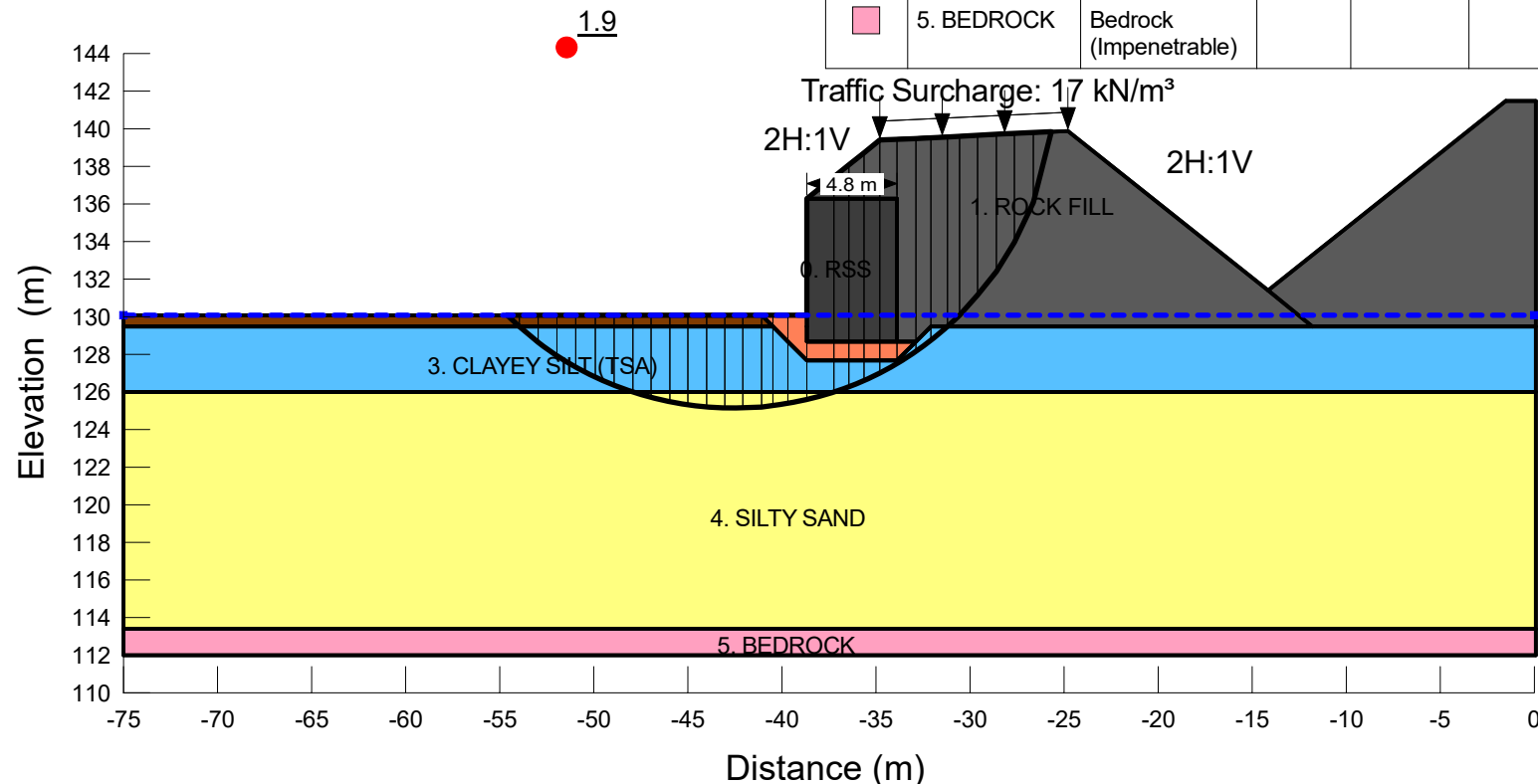


Project CPR Westbound Lanes Embankment		
Analysis 5.1 Permanent Long Term (Drained)		
Seismic Coefficient H: 0g, V: 0g	Last Run 2022/07/14, 04:54:55 PM	Scale 1:400

Additional Details
Name: 5 CPR Culvert: 1.25H:1V Rock Fill Embankment & RSS Wall
Comments:
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
Entry: (-25.658867, 139.85706) m, Exit: (-52.9, 130.1) m
Center: (-41.831239, 142.10302) m, Radius: 16.327583 m

Figure H5.1








Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	0-GRANULAR A	Mohr-Coulomb	22.8	0	40
	0-RSS	Mohr-Coulomb	22.8	250	42
	1. ROCK FILL	Mohr-Coulomb	20	0	42
	2. ORGANICS	Mohr-Coulomb	11	10	10
	3. CLAYEY SILT (TSA)	Mohr-Coulomb	17.5	90	0
	4. SILTY SAND	Mohr-Coulomb	21	0	35
	5. BEDROCK	Bedrock (Impenetrable)			

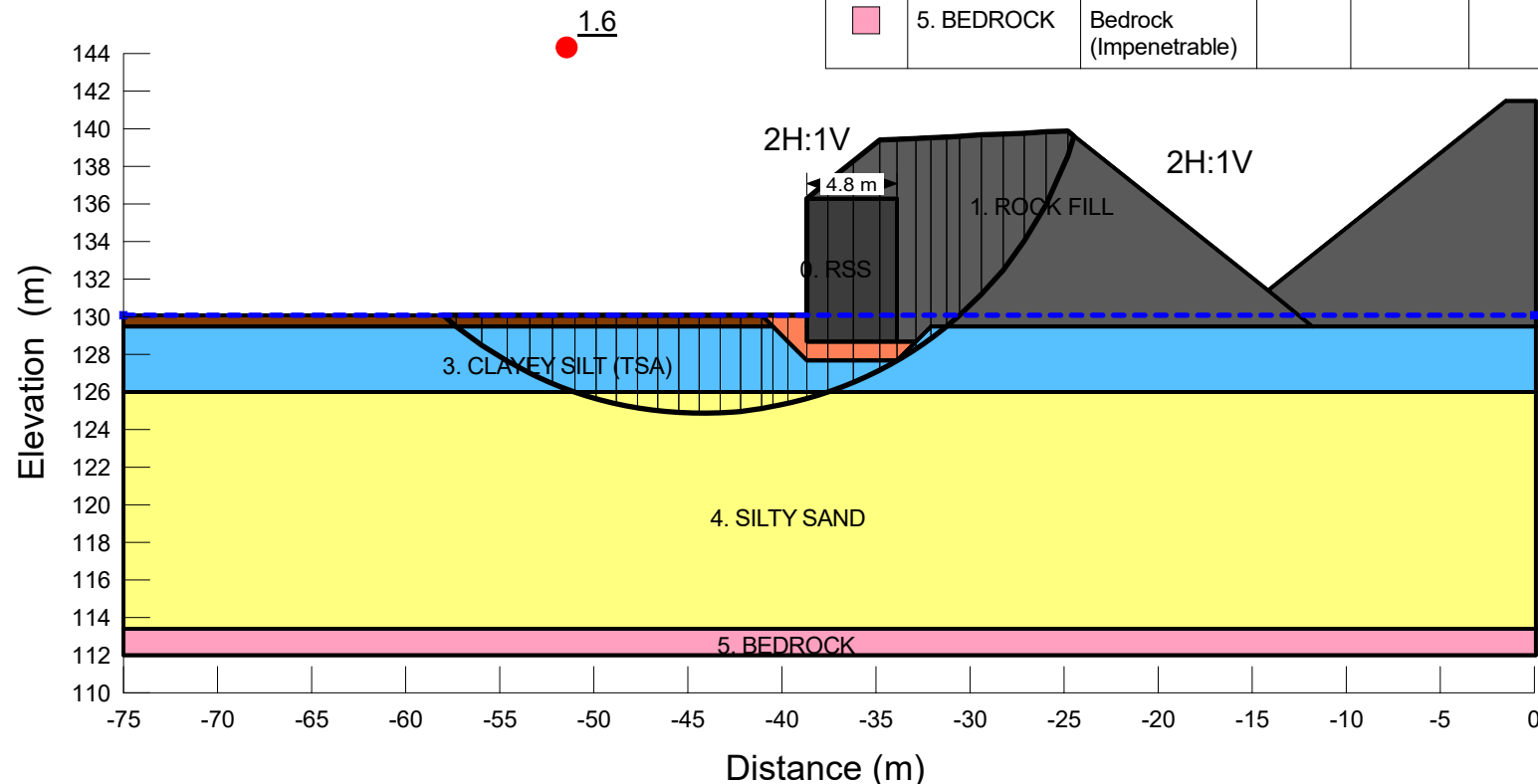


Project CPR Westbound Lanes Embankment		
Analysis 5.2 Temporary Short Term - Traffic (Undrained)		
Seismic Coefficient H: 0g, V: 0g	Last Run 2022/07/14, 04:54:59 PM	Scale 1:400

Additional Details
Name: 5 CPR Culvert: 1.25H:1V Rock Fill Embankment & RSS Wall
Comments:
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
Entry: (-25.658867, 139.85706) m, Exit: (-54.6, 130.1) m
Center: (-42.572327, 142.22458) m, Radius: 17.078357 m

Figure H5.2

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	0-GRANULAR A	Mohr-Coulomb	22.8	0	40
	0-RSS	Mohr-Coulomb	22.8	250	42
	1. ROCK FILL	Mohr-Coulomb	20	0	42
	2. ORGANICS	Mohr-Coulomb	11	10	10
	3. CLAYEY SILT (TSA)	Mohr-Coulomb	17.5	90	0
	4. SILTY SAND	Mohr-Coulomb	21	0	35
	5. BEDROCK	Bedrock (Impenetrable)			



Project CPR Westbound Lanes Embankment		
Analysis 5.3 Temporary - Pseudo-Static 2,475-yr (Undrained)		
Seismic Coefficient H: 0.113g, V: 0g	Last Run 2022/07/14, 04:55:04 PM	Scale 1:400

Additional Details
Name: 5 CPR Culvert: 1.25H:1V Rock Fill Embankment & RSS Wall
Comments:
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
Entry: (-24.450007, 139.62104) m, Exit: (-58, 130.1) m
Center: (-44.265362, 145.57406) m, Radius: 20.690258 m

Figure H5.3



Appendix I.

List of Referenced Specifications Non-Standard Special Provisions



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

OPSD 208.010	Benching of Earth Slopes
OPSD 803.010	Backfill and Cover for Concrete Culverts with Spans Less than or Equal to 3.0m
OPSD 3090.101	Foundation Frost Depths for Southern Ontario
OPSD 3101.150	Walls Abutment, Backfill Minimum Granular Requirement
OPSS.PROV 180	General Specification for the Management of Excess Materials
OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 422	Precast Reinforced Concrete Box Culverts and Box Sewers in Open Cut
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 517	Construction Specification for Dewatering
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS.PROV 805	Construction Specification for Temporary Erosion and Sediment Control Measures
OPSS.PROV 902	Construction Specification for Excavating and Backfilling Structures
OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS.PROV 1010	Material Specification for Aggregates Base, Subbase, Select Subgrade, and Backfill Material
OPSS.PROV 1860	Material Specification for Geotextiles
SP 110S06	Amendment to OPSS 1010 - Material Specification for Aggregates Base, Subbase, Select Subgrade, and Backfill Material
SP 517F01	Amendment to OPSS 517 - Construction Specification for Dewatering



2. Suggested wording for NSSPs

Notice to Contractor: “Obstructions”

The Contractor is hereby notified that the native discontinuous tills at the site and as inferred from available information should be expected to contain cobbles and boulders. Considerations of these obstructions must be made in the selection of appropriate equipment and procedures for excavations, installations of deep foundations and temporary protection systems.

Notice to Contractor: “Artesian Conditions”

The Contractor is advised that artesian conditions may be present at the site that could influence design and construction of driven temporary protection systems such as soldier piles or sheet piles. The Contractor shall be responsible for implementing adequate groundwater control measures to manage artesian conditions encountered.

Notice to Contractor: “Protection of Sensitive Foundation Soils”

The Contractor is advised that the native clay to clayey silt that will be exposed at the subgrade is moisture sensitive and may become disturbed or otherwise negatively impacted when subjected to construction or personnel traffic, freeze-thaw actions, ingress or ponding water. The Contractor shall be responsible for implementing adequate groundwater control measures and to minimize construction and personnel traffic on the founding subgrade.

“Structural Backfill”

Structural backfill for the culvert and retaining walls shall consist of OPSS Granular B Type II or Quarry Sourced OPSS Granular A material.