

Foundation Investigation and Design Report

Chippewa Creek Culvert
Replacement
Site No. 46-362, Highway 11,
North Bay, ON

G.W.P. 5143-06-01

Geocres No. 31L-204



Prepared for:
Ministry of Transportation Ontario

Prepared by:
Stantec Consulting Ltd.
400 – 1331 Clyde Avenue
Ottawa, ON K2C 3G4

Project No. 165000836

May 2017

Table of Contents

1.0	INTRODUCTION	1
2.0	SITE DESCRIPTION AND GEOLOGY	2
2.1	SITE DESCRIPTION	2
2.2	SITE GEOLOGY	2
3.0	INVESTIGATION PROCEDURES	3
3.1	REVIEW OF PREVIOUS INVESTIGATION	3
3.2	FIELD INVESTIGATION – CULVERT SITE	3
3.3	LOCATION AND ELEVATION SURVEY	4
3.4	LABORATORY TESTING	4
4.0	SUBSURFACE CONDITIONS	5
4.1	SUBSURFACE PROFILE	5
4.1.1	Pavement Structure	5
4.1.2	Fill	5
4.1.3	Organic Soil	6
4.1.4	Silty Sand to Sand	6
4.1.5	Till	7
4.1.6	Bedrock	8
4.2	GROUNDWATER	8
4.3	CHEMICAL TEST RESULTS	9
5.0	SITE RECONAISSANCE	9
6.0	MISCELLANEOUS	11
7.0	CLOSURE	12
8.0	DISCUSSION AND RECOMMENDATIONS	13
8.1	PROJECT DESCRIPTION AND BACKGROUND	13
8.2	GEOTECHNICAL DESIGN PARAMETERS	15
8.3	FROST PENETRATION	15
8.4	SEISMIC DESIGN CONSIDERATIONS	16
8.4.1	Site Class	16
8.4.2	Peak Ground Acceleration (PGA)	16
8.4.4	Liquefaction Potential	16
8.5	STRUCTURE FOUNDATIONS	17
8.5.1	General	17
8.6	FOUNDATION RECOMMENDATIONS	18
8.6.1	Geotechnical Resistances – Box Culvert and Retaining Wall	18
8.6.2	Sliding Resistance	20
8.7	LATERAL EARTH PRESSURES	20
8.7.1	Lateral Earth Pressures under Static Conditions	20

FOUNDATION INVESTIGATION AND DESIGN REPORT

8.7.2	Lateral Earth Pressures under Seismic Conditions	21
8.8	EMBANKMENT DESIGN.....	22
8.8.1	Embankment Settlement.....	22
8.8.2	Stability of Slopes	23
8.9	EROSION AND SCOUR PROTECTION	23
8.9.1	Erosion Protection.....	23
8.9.2	Scour Protection	23
8.10	CEMENT TYPE AND CORROSION PROTECTION	23
9.0	CONSTRUCTION CONSIDERATIONS	24
9.1	CONSTRUCTION STAGING	24
9.2	TEMPORARY ROADWAY PROTECTION	25
9.2.1	General.....	25
9.2.2	Geotechnical Lateral Resistance.....	27
9.3	EXCAVATION AND BACKFILLING	28
9.4	REUSE OF EXCAVATED MATERIAL.....	28
9.5	TEMPORARY CONSTRUCTION UNWATERING AND DEWATERING	28
10.0	SPECIFICATIONS.....	30
11.0	REFERENCES.....	30
12.0	CLOSURE.....	31

LIST OF TABLES

Table 1.1: Coordinates of Chippewa Creek Culvert on Highway 11, north of North Bay, ON (MTM Zone 10)	1
Table 3.1: Borehole Summary	4
Table 3.2: Laboratory Testing for Culvert Site.....	4
Table 4.1: Inferred and Measured Groundwater Levels	8
Table 4.2: Rising Head Permeability Test Results.....	9
Table 4.3: Results of Chemical Analysis	9
Table 5.1: Soil Samples at Culvert Inlet and Outlet.....	10
Table 8.1: Geotechnical Model at Chippewa Creek Culvert	15
Table 8.2: Peak Ground Acceleration Data.....	16
Table 8.3: Comparison of the Replacement Options for the Original Culvert	17
Table 8.4: Recommended Spread Footing Design Parameters.....	19
Table 8.5: Recommended Non-Seismic Earth Pressure Parameters (Horizontal Backfill)	20
Table 8.6: Recommended Non-Seismic Earth Pressure Parameters (2H:1V Backfill)	21
Table 8.7: Seismic Design Parameters to Estimate Lateral Earth Pressures	22
Table 8.8: Recommended Seismic Earth Pressure Parameters (2H:1V Backfill)	22
Table 9.1: Comparison of Roadway Protection Systems	25
Table 9.2: Soil Parameters - Temporary Roadway Protection	26
Table 10.1: Specifications Referenced in Report	30

FOUNDATION INVESTIGATION AND DESIGN REPORT

LIST OF APPENDICES

- APPENDIX A** Drawing No. 1 – Borehole Location Plan and Soil Strata Plot
 Drawing No. 2 – Borehole Location Plan and Soil Strata Plot
 Drawing No. P1 – Preliminary General Arrangement
 Site Photos
- APPENDIX B** Symbols and Terms Used on Borehole Records
 Stantec Borehole Records
 Borehole Records from 1996 Memorandum 'WP 25-84-01, Chippewa Culvert
 Replacement, Hwy. 11, District 54, North Bay'
- APPENDIX C** Laboratory Test Results
- APPENDIX D** Design Parameters
 Slope Stability Analysis Results
- APPENDIX E** NBC Seismic Hazard Calculation Sheet
- APPENDIX F** Notice to Contractor – Cobbles and Boulders
 Notice to Contractor – Groundwater Control

FOUNDATION INVESTIGATION AND DESIGN REPORT

May 2017

FOUNDATION INVESTIGATION REPORT

For

G.W.P 5143-06-01

Geocres No. 31L-204

Chippewa Creek Culvert Replacement,

Site No. 46-362, Highway 11

North Bay, Ontario

1.0 INTRODUCTION

Stantec Consulting Ltd. (Stantec) was retained by the Ministry of Transportation, Ontario, (MTO) to complete a foundation investigation of the Chippewa Creek Culvert on Highway 11 north of North Bay, Ontario. The foundation investigation is required to support the design for the replacement of the Chippewa Creek Culvert. The northing and easting coordinates of the existing culvert are found in Table 1.1 below.

Table 1.1: Coordinates of Chippewa Creek Culvert on Highway 11, north of North Bay, ON (MTM Zone 10)

Culvert Name	Easting	Northing
Chippewa Creek Culvert	307401.7	5136644.9

This Foundation Investigation Report has been prepared specifically and solely for the foundation analysis of the Chippewa Creek Culvert on Highway 11.

Project Number: G.W.P. 5143-06-01, Geocres No. 31L-204

Project Location: Highway 11, approximately 4.6 km north of the intersection of Highway 17 and 11, north of North Bay, Ontario

2.0 SITE DESCRIPTION AND GEOLOGY

2.1 SITE DESCRIPTION

Site Location

The site location is shown on the Key Plan inset to Drawing No. 1, provided in Appendix A. The existing Chippewa Creek Culvert crosses beneath Highway 11 approximately 4.6 km north of the intersection of Highway 17 and Highway 11. Photographs showing the general site conditions of the culvert are provided in Appendix A.

General Site Description

Highway 11 runs north-south at the project location; chainage increases in the northern direction. Within the vicinity of the culvert, Highway 11 has a four lane cross-section and approximately 3 m wide shoulders with cable guide rails.

Chippewa Creek flows beneath Highway 11, via the existing culvert, from west to east. The road embankment has side slopes of approximately 2H: 1V. The paved surface of the highway is approximately 5.0 m higher than the ground surface on both sides of the road. The area beyond the water course is covered with brush and trees. Site photographs are shown in Appendix A.

Existing Culvert

The existing culvert is a combination of precast concrete box culvert, concrete culvert, and concrete slab with stone walls. The 5.0 m long section of concrete slab with stone walls is the major poor performing area of the culvert. The culvert has a width of 3.0 m and a length of 45.7 m. The culvert is covered with approximately 3.0 m of fill.

Minor repairs to the stone wall sections of the culvert were carried out in 2016. The repairs included placement of mass concrete against the interior face of the stone walls.

The approximate alignment of the existing culvert is shown on Drawing No. 1 in Appendix A. The proposed culvert location south of the existing culvert at approximate station 14+615 is also shown on the drawing.

2.2 SITE GEOLOGY

Physiographic Description

The project site is located within the Canadian Shield. Soil and bedrock rock mapping published by the Ontario Geological Survey suggests that the subsurface conditions at the site consist of gravel and sand of glaciofluvial origin underlain by Mesoproterozoic migmatitic rock and gneisses of the Grenville Province's Central Gneiss Belt.

3.0 INVESTIGATION PROCEDURES

3.1 REVIEW OF PREVIOUS INVESTIGATION

A review of the 1996 'WP 25-84-01, Chippewa Culvert Replacement Hwy. 11, District 54, North Bay' memorandum suggests that the surficial geology of the site consists of sand and silt fill, over an organic sand layer, over silty sand deposits. The depth of bedrock was found to be 11.2 to 13.3 m below the grade of the highway. The groundwater depth was recorded at approximately 5 m below the grade of the highway.

3.2 FIELD INVESTIGATION – CULVERT SITE

A field investigation consisting of four boreholes was carried out for this assignment. The boreholes were designated BH15-1, BH15-2, BH16-1 and BH16-2 and their locations are shown on the Borehole Locations and Soil Strata, Drawing No.1 and Drawing No. 2 in Appendix A.

Prior to carrying out the investigation, Stantec contacted the public utility authorities to clear the borehole locations of public utilities.

The field drilling programs were carried out on May 21, 2015, and November 22 and 23, 2016. Boreholes BH15-1 and BH15-2 were advanced with 'NW' size casing using a rubber tire CME 550 drill rig equipped for soil sampling. The drill rig for the 2015 field program was owned and operated by Landcore Drilling. Borehole BH16-1 was advanced with portable drilling equipment and 'HQ' size casing. Borehole BH16-2 was advanced with a CME truck-mounted drill rig equipped for soil sampling. The drill rig and portable drilling equipment for the 2016 field program were owned and operated by George Downing Estate Drilling Ltd.

The subsurface stratigraphy encountered in each borehole was recorded in the field by experienced Stantec personnel. Split spoon samples were collected at regularly spaced intervals (typically every 760 mm for road boreholes and every 610 mm for the portable borehole) during the course of Standard Penetration Testing (ASTM D1586). All samples recovered were returned to Stantec's Ottawa laboratory for detailed classification and testing.

A monitoring well was installed in BH16-2 at 7.62 m below ground surface. The collection zone in the monitoring well was 3.1 m of slotted 50 mm PVC pipe set between elevation 304.3 to 307.3 and backfilled with sand. The portion of the borehole above and below the collection zone was backfilled with a bentonite seal and auger cuttings. Groundwater levels were estimated in the open boreholes and measured in the monitoring well on November 23, 2016. A rising head permeability test was completed in the monitoring well on November 23, 2016. Boreholes were backfilled with auger cuttings or with sand and bentonite for the monitoring well. Road holes were topped with cold patch asphalt when applicable.

A site reconnaissance was carried out on May 14, 2015, by Zachary Popper, P.Eng., of Stantec to observe the condition of the culvert and the soil sediment inside the culvert. Photographs from

the site visit are presented in Appendix A. Soil samples were collected from the interior of the culvert, the creek bed, and beside the creek by hand auger and shovel.

3.3 LOCATION AND ELEVATION SURVEY

The borehole locations and geodetic elevations were surveyed in the field by Stantec personnel using a Trimble Geo XH GPS. The elevations are accurate to 0.1 m. Table 3.1 summarizes the borehole information.

Table 3.1: Borehole Summary

	Boreholes			
	BH15-1	BH15-2	BH16-1	BH16-2
MTM Zone 10 Coordinates				
Northing	5136651	5136639	5136636	5136627
Easting	307402	307402	307386	307407
Latitude	46° 22' 10.41"	46° 22' 10.02"	46° 22' 9.92"	46° 22' 9.63"
Longitude	79° 27' 58.25"	79° 27' 58.25"	79° 27' 59.00"	79° 27' 58.02"
Ground Surface Elevation, m	311.9	311.8	307.3	311.9
Total Depth Drilled, m	10.5	10.5	5.8	11.6
End of Borehole Elevation, m	301.4	301.3	301.5	300.3
Depth of Casing, m	9.9	9.9	5.8	0
Number of Soil Samples	13	14	9	14

3.4 LABORATORY TESTING

All samples were taken to the Stantec Ottawa laboratory where they were subjected to a detailed visual examination by a Geotechnical Engineer. Selected soil samples underwent gradation analysis, organic content and moisture content testing. The laboratory testing summary is shown in Table 3.2 below.

Table 3.2: Laboratory Testing for Culvert Site

Laboratory Testing	Moisture Content	Gradation Analysis	Organic Content
Number of Tests	57	19	2

Note: Moisture content includes seven samples from the culvert inlet and outlet which are not on the borehole logs. Gradation analysis includes one culvert sediment sample not on borehole logs.

One soil sample was tested for pH, soluble sulphate content, chloride content, and resistivity.

Samples remaining after testing will be placed in storage for a period of one year after issuance of the final report. After the storage period, the samples will be discarded unless we are directed otherwise by MTO.

4.0 SUBSURFACE CONDITIONS

4.1 SUBSURFACE PROFILE

The subsurface conditions observed in the Stantec boreholes are presented in detail on the Borehole Records provided in Appendix B. Borehole Records from the 1996 memorandum titled 'WP 25-84-01, Chippewa Culvert Replacement Hwy. 11, District 54, North Bay' have also been included. The boreholes from the 1996 investigation will be referred to as Borehole No. 1, Borehole No. 2, and Borehole No. 3. An explanation of the symbols and terms used to describe the Borehole Records is provided in Appendix B.

In general, the subsurface stratigraphy for road boreholes consisted of asphalt, over sand fill with varying amounts of gravel and silt, over a thin organic soil layer, over sand and silty sand deposits, over silty sand trace gravel till, underlain by bedrock. Cobbles and boulders were inferred in the fill and in the till.

A borehole location plan and profile with stratigraphic sections of the soil encountered within the boreholes is provided on Drawing No. 1 and Drawing No. 2 in Appendix A.

4.1.1 Pavement Structure

A 200 mm asphalt layer was encountered in boreholes BH15-1 and BH15-2. A 150 mm asphalt layer was encountered in borehole BH16-2. Asphalt was encountered at the surface of Borehole No. 2 and Borehole No. 3.

4.1.2 Fill

Fill material was encountered in boreholes BH15-1, BH15-2, BH16-2, Borehole No. 2 and Borehole No. 3 beneath the asphalt. The fill was variable and consisted of gravel with sand (pavement structure granular material), sand with varying amounts of gravel and silt, and sandy silt. A trace of organics was noted within the fill in the 1996 investigation. The fill was approximately 4.7 to 5.6 m thick and the bottom extended to elevations 306.1 m to 307.0 in the boreholes.

The Standard Penetration Test (SPT) N-values observed within the fill ranged from 4 to more than 50 blows per 0.3 m suggesting a loose to very dense state. Typical SPT N-Values within the fill ranged from 4 to 32 blows per 0.3 m suggesting a loose to dense state.

Cobbles and boulders were inferred within the fill material based on the SPT "N" values for samples SS3 and SS6 in borehole BH16-2, SS2 in Borehole No. 2 and SS2 in Borehole No. 3.

FOUNDATION INVESTIGATION AND DESIGN REPORT

May 2017

Moisture content and grain size distribution tests carried out on representative samples of the fill yielded the following results:

Gravel:	0 to 19%
Sand:	41 to 89%
Silt and Clay:	8 to 59%
Moisture content:	1 to 20%

The grain size distribution curves for the fill layer are provided in Figure No. 1 of Appendix C.

4.1.3 Organic Soil

A layer of organic soil was encountered in boreholes BH15-1, BH15-2, BH16-2, Borehole No. 2 and Borehole No. 3 beneath the road embankment fill. Organic soil was encountered from 0 to 2.3 m in Borehole No. 1 and was also noted from 0 to 1.8 m below ground surface in borehole BH16-1. The organic soil contained wood. The organic soil had a thickness ranging from 0.6 m to 1.4 m and extended to bottom elevations ranging from 306.4 to 305.0 m.

The Standard Penetration Test (SPT) N-values observed within the organic soil ranged from 0 to 19 blows per 0.3 m suggesting a very loose to compact state.

Organic content testing (ASTM D2974) carried out on representative samples from the organic soil yielded organic contents of 3% to 7%.

4.1.4 Silty Sand to Sand

A sand layer with varying amounts of gravel and silt (fines) was encountered in all of the boreholes.

The sand had a lesser fines content in borehole BH15-2 and in borehole BH16-2 beneath the organic soil. Borehole BH15-2 was terminated within the sand deposit at a depth of 10.5 m at elevation 301.3 m. The sand in these two boreholes is described as poorly graded sand (SP), well graded sand with silt (SW-SM) and silty sand (SM) based on the Unified Soil Classification System (USCS). The sand deposit contained some organic soil and wood directly beneath the organic soil layer.

Moisture content and grain size distribution tests carried out on representative samples of the sand with a lower fines content yielded the following results:

Gravel:	4 to 10%
Sand:	82 to 92%
Fines (silt and clay):	4 to 8%
Moisture Content:	16 to 39%

Sand with a higher fines content was encountered in all of the boreholes except for BH15-2 and is described as silty sand (SM) based on the USCS. The silty sand layer was encountered in borehole BH15-1 and in Boreholes No. 1 to 3 below the organic soil, in borehole BH16-1 at the

FOUNDATION INVESTIGATION AND DESIGN REPORT

May 2017

surface and in borehole BH16-2 below the fill. Borehole BH15-1 was terminated in the silty sand deposit at a depth of 10.5 m at elevation 301.4 m. The bottom of the silty sand deposit was at depths of 5.2 m and 11.1 m, elevations 302.1 m and 300.8 m, in boreholes BH16-1 and BH16-2, respectively. The bottom of the silty sand in the 1996 investigation ranged from depths of 8.2 m to 10.8 m, elevations 299.1 m to 302.5 m.

The SPT N-values for this deposit ranged from 1 to 25 blows per 0.3 m suggesting a very loose to compact state; typically, N-values of 4 to 8 were recorded suggesting a generally loose stratum. An SPT N-value of 54 at the bottom of the silty sand layer in Borehole No. 1 suggests that the SPT was likely done within the till layer described below.

Moisture content and grain size distribution tests carried out on representative samples of the silty sand yielded the following results:

Gravel:	0 to 1%
Sand:	67 to 85%
Fines (silt and clay):	15 to 33%
Silt size:	19 to 33%
Clay size:	0 to 1%
Moisture Content:	17 to 45%

The grain size distribution curve for the silty sand material is provided in Figure No. 2 of Appendix C.

4.1.5 Till

A till layer consisting of silty sand to sandy silt with varying amounts of gravel was encountered below the silty sand in boreholes BH16-1, BH16-2, Borehole No. 2 and Borehole No. 3. Cobbles and boulders were also inferred in the till. Drilling refusal occurred within the till layer at depths of 11.6 m, and 11.2 m, elevations 300.3 m and 300.4 m, in boreholes BH16-2, and Borehole No. 3, respectively. The bottom of the till layer was at a depth of 13.3 m, elevation 298.6 m in Borehole No. 2.

The SPT N-values for this deposit ranged from 6 to 57 blows per 0.3 m suggesting a loose to very dense state.

Moisture content and grain size distribution tests carried out on representative samples of the till yielded the following results:

Gravel:	0 to 3%
Sand:	44 to 73%
Fines (silt and clay):	27 to 53%
Silt size:	43%
Clay size:	10%
Moisture Content:	14 to 18%

The grain size distribution curve for the sandy silt till material is provided in Figure No. 3 of Appendix C.

4.1.6 Bedrock

Bedrock level was proven by coring in Borehole No. 1 and Borehole No. 2. The bedrock was described in the 1996 memo as biotite gneiss, greyish orange pink to greyish red to dark grey; medium to coarse grained; strong; unweathered to slightly weathered.

The bedrock surface was encountered at depths of 8.2 m and 13.3 m below ground surface, corresponding to approximate elevations of 299.1 m and 298.6 m, respectively in Borehole No. 1 and Borehole No. 2. Auger refusal on bedrock was encountered at elevation 300.3 m and 300.4 m in boreholes BH16-2 and Borehole No. 3, respectively.

Rock Quality Designation (RQD) values measured on the retrieved bedrock core ranged between 25% and 94%, indicating a poor to excellent rock mass quality. Core recovery ranged from 89% to 95%. Details are provided in the Rock Core Description sheet from the 1996 memo in Appendix B.

4.2 GROUNDWATER

The inferred groundwater levels in the open boreholes at the time of drilling and measured groundwater level in the monitoring well are provided in Table 4.1 below. The groundwater levels are approximately at the depth of the creek water level. The creek water level in the existing culvert measured on November 23, 2016, was at elevation 306.6 m.

Table 4.1: Inferred and Measured Groundwater Levels

Borehole No.	Observation/Measurement Date	Groundwater Depth (m)	Ground Surface Elevation(m)	Groundwater Elevation (m)
BH15-1	May 21, 2015	5.2 (inferred)	311.9	306.7
BH15-2	May 21, 2015	5.2 (inferred)	311.8	306.6
BH16-1	November 22, 2016	1.2 (inferred)	307.3	306.1
BH16-2	November 23, 2016	5.2 (inferred)	311.9	306.7
BH16-2	November 23, 2016	5.2 (measured)	311.9	306.7
Borehole No.1	June 27, 1996	1.0 (inferred)	307.3	306.3
Borehole No.2	June 26, 1996	4.9 (inferred)	311.8	306.9
Borehole No.3	June 26, 1996	4.8 (inferred)	311.6	306.8

The above measurements suggest that the groundwater levels within the sand is essentially the same as the Chippewa Creek water level.

Fluctuations in the groundwater due to seasonal variations or in response to a particular precipitation event should be anticipated.

A rising head permeability test was completed in monitoring well BH16-2 on November 23, 2016 to determine the hydraulic conductivity of the soil. The initial drawdown was recorded to be 5.6 m below ground surface. The static water level was recorded to be 5.2 m below ground surface. The total volume of water removed to allow of the initial drawdown was 70 L. The test results are provided in Table 4.2.

Table 4.2: Rising Head Permeability Test Results

Elapsed Time (min)	Water Level Below Grade (m)	Elapsed Time (min)	Water Level Below Grade (m)
0	5.60	4	5.51
1	5.54	5	5.49
2	5.53	8	5.42
3	5.52	10	5.20

The test results were analyzed using the Bouwer-Rice solution method for an unconfined aquifer model. The analysis yielded a hydraulic conductivity 'K' value of 2.8×10^{-4} cm/s, which falls within the range of values typically found for poorly graded sand.

4.3 CHEMICAL TEST RESULTS

One sample of the native material was tested for pH, water soluble sulphate and chloride concentrations, and resistivity. The analysis results are provided in Table 4.3.

Table 4.3: Results of Chemical Analysis

Borehole No	Sample No.	Depth (m)	pH	Chloride (µg/g)	Sulphate (µg/g)	Resistivity (Ohm-m)
BH16-1	SS4	1.83 to 2.44	7.2	682	61	10

5.0 SITE RECONNAISSANCE

A site visit was carried out on May 14, 2015, by Zachary Popper, P.Eng., of Stantec to observe the condition of the culvert and the soil sediment inside the culvert. Photographs from the site visit can be found in Appendix A. It should be noted that minor repairs to the culvert were completed in 2016 and the photographs and observations noted herein are not necessarily representative of the site condition. Soil samples were collected from the interior of the culvert, the creek bed, and beside the creek by hand auger and shovel. The following observations were made during the site visit:

West Side – Culvert Inlet

- The creek bed consists of poorly graded sand with gravel, cobbles and boulders.
- The groundwater level is 0.3 m above the culvert invert and 0.6 m above the creek bottom.
- Inside the culvert at the inlet there is a 25 mm to 150 mm thick layer of sand sediment on the south side of the culvert. The sand sediment layer varies from 0.3 m to 1.0 m wide from the south edge of the culvert.
- Cobbles and boulders are present approximately 5 m into the culvert from the inlet.
- The surface of the slope from Highway 11 to the culvert inlet consists of sand, gravel, cobbles and boulders (see Photo No. 3).
- There is evidence of concrete and stone wall deterioration including fallen concrete, loose stone fragments and deterioration of walls in the middle-west area of the culvert (see Photo No. 6, No. 9, No. 10 and No. 11).

FOUNDATION INVESTIGATION AND DESIGN REPORT

May 2017

East Side – Culvert Outlet

- The creek bed consists of sand, gravel, and frequent cobbles and boulders at the culvert outlet (see Photo No. 8).
- The water level is 0.3 m above the culvert outlet.
- The water level is 50 mm above the creek bed on the south side of the culvert to 300 mm above creek bed on the north side.
- 3 m into the culvert from the outlet there are frequent boulders and sand sediment on the south half of the width of the culvert. Wood branches are also present (see Photo No. 8).
- Sand, gravel, cobbles and boulders are present on the slope from Highway 11 to the culvert outlet; frequent boulders are evident on the slope to the south side of culvert (see Photo No. 4).

The table below summarizes soil samples taken from the interior of the culvert, the creek bed, and beside the creek by hand auger and by shovel:

Table 5.1: Soil Samples at Culvert Inlet and Outlet

Sample Number	Depth (m)	Hand Auger or Shovel Sample	Location of Sample	Soil Type
GS1	0 - 0.30	Shovel	Creek bed at culvert inlet	Brown poorly graded SAND (SP) with gravel, cobbles and boulders
GS2	0 - 0.15	Shovel	Soil sediment from culvert interior at inlet	Brown poorly graded SAND (SP)
GS3	0 - 0.30	Hand Auger	1 m west, 1 m north of culvert inlet, adjacent to creek	Brown organic soil (OL) with sand
GS4	0 - 0.20	Hand Auger	1 m west, 1 m south of culvert inlet, adjacent to creek	Brown organic soil (OL) with silty sand
GS5	0 - 0.10	Shovel	Soil sediment from culvert interior at outlet	Brown poorly graded SAND (SP) with silt and gravel
GS6	0 - 0.30	Shovel	Creek bed at culvert outlet	Brown poorly graded SAND (SP) with frequent gravel and cobbles
GS7	0 - 0.76	Hand Auger	1 m east, 1 m north of culvert outlet, adjacent to creek	Brown poorly graded SAND (SP) with organic soil

The approximate location of the hand auger and shovel samples is provided on Drawing No. 1 and Drawing No. 2 in Appendix A. Moisture content and grain size distribution tests carried out on two representative samples from the above table yielded the following results:

Gravel:	17%
Sand:	78%
Silt:	5%
Clay:	0%
Moisture Content:	8 to 47%

The grain size distribution curve for the sand sediment from the interior of the culvert is provided in Figure No. 2 of Appendix C.

6.0 MISCELLANEOUS

The field work was carried out under the supervision of Zachary Popper, P.Eng. and Jason Hopwood-Jones, under the direction of Christopher McGrath., P.Eng.

USL-1 Underground Service Locators Inc. of Ottawa, Ontario, carried out the public utility locates for the boreholes.

The rubber tire CME 550 drilling equipment was supplied and operated by Landcore Drilling of Chelmsford, Ontario, on May 21, 2015. The truck-mounted CME drill rig and portable drilling equipment was supplied and operated by George Downing Estate Drilling Ltd. on November 22 and 23, 2016.

Elevation and location survey of the borehole locations was carried out by Stantec personnel.

Geotechnical laboratory testing was carried out at Stantec's Ottawa laboratory. Chemical testing for pH, soluble sulphate, and chloride content, and resistivity was carried out by Paracel Laboratories of Ottawa.

This report was prepared by Zachary Popper, and reviewed by Christopher McGrath and by Raymond Haché, MTO Designated Principal Contact.

FOUNDATION INVESTIGATION AND DESIGN REPORT

May 2017

7.0 CLOSURE

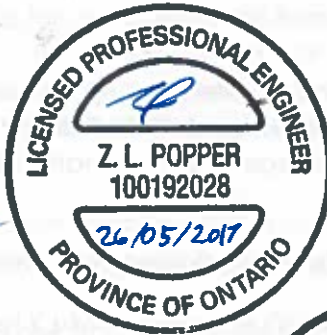
A subsurface investigation is a limited sampling of a site. The subsurface conditions given herein are based on information gathered at the specific borehole locations. Should any conditions at the site be encountered which differ from those at the borehole locations, we request that we be notified immediately in order to assess the additional information.

Respectfully Submitted;

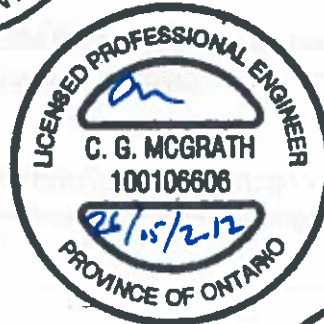
STANTEC CONSULTING LTD.



Zachary Popper, P.Eng.
Geotechnical Engineer



Christopher McGrath, P.Eng.
Associate- Senior Geotechnical Engineer



Raymond Haché, M.Sc., P.Eng.
Designated Principal MTO Foundation Contact



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FOUNDATION INVESTIGATION AND DESIGN REPORT

May 2017

FOUNDATION DESIGN REPORT

For

G.W.P 5143-06-01

Geocres No. 31L-204

Chippewa Creek Culvert Replacement,
Site No. 46-362, Highway 11
North Bay, Ontario

8.0 DISCUSSION AND RECOMMENDATIONS

8.1 PROJECT DESCRIPTION AND BACKGROUND

Project Purpose/Justification

Stantec Consulting Ltd. (Stantec) was retained by the Ministry of Transportation, Ontario, (MTO) to complete a foundation investigation of the Chippewa Creek Culvert on Highway 11 north of North Bay, Ontario. The foundation investigation is required to support the design for the proposed replacement of the Chippewa Creek Culvert. The results of the foundation investigation and the geotechnical engineering recommendations for the replacement of the culvert are presented in this report.

Performance of Existing Culvert

The existing culvert is a combination of precast concrete box culvert, concrete culvert, and concrete slab with stone walls. The 5.0 m middle portion consisting of concrete slab with stone wall is the major poor performing area of the culvert. The concrete slab is resting on stone and mortar foundations which have been partially eroded by the creek flow. The stone and mortar foundations are approximately 0.75 m wide at the top and 1.3 m wide at the base. It is estimated that the footing embedment depth inside the culvert, below the streambed, is 1.8 m. The existing culvert has a width of 3.0 m and a total length of 45.7 m. The culvert is covered with approximately 3.0 m of fill. The approximate alignment of the existing culvert is shown on Drawing No. 1 in Appendix A. The culvert has been identified as requiring replacement.

Minor repairs to the stone wall sections of the culvert were carried out in 2016. The repairs included placement of mass concrete against the interior face of the stone walls.

Proposed Structure

The existing culvert is planned to be replaced. The following options were being considered during the preliminary design for the replacement of the existing Chippewa Creek Culvert: rigid frame box culvert; or, rigid frame open footing culvert or CSP arch with concrete footing. The selected structure design is anticipated to be a precast concrete box culvert with a span of approximately 3.0 m with the height anticipated to be approximately 2.1 m. Segmental concrete block retaining walls are proposed at each of the culvert ends. The preliminary

FOUNDATION INVESTIGATION AND DESIGN REPORT

May 2017

general arrangement drawing for the proposed Chippewa Creek Culvert replacement is presented on Drawing No. P1 in Appendix A.

The analysis and design approach for this report assumes that the replacement structure is classified as follows:

- Buried Structure (CSA S6-14 Section 7)
- Force-based design (FBD) and Elastic Static Analysis (ESA)

The proposed alignment for the culvert replacement is south of the existing culvert at approximate station 14+615.

Key approximate elevations associated with the proposed culvert replacement are as follows:

Pavement Elevation:	312.2 m (near the C/L of Highway 11)
Existing Invert Elevation:	306.5 inlet 306.4 outlet
Existing Obvert Elevation:	308.1 m inlet 307.9 m outlet
Design Streambed Elevation:	306.3 m inlet 306.3 m outlet
Water Level Elevation:	306.7 m (November 23, 2016)
25 Year Design Water Level:	307.9 m
Founding Elevation:	305.7 m proposed replacement box culvert 305.0 m proposed retaining wall; base or granular pad elevation selected to found the wall below the organic soil layer.

Construction Staging & Detours

It is understood that the proposed culvert replacement work is to be carried out without the use of a short term local road detour.

A two stage construction approach is proposed in order to keep one lane in each direction operational to traffic at all times. Highway traffic is anticipated to be controlled using a traffic control setup on one side of the highway at a time. Roadway protection is anticipated to be required during construction.

The use of wellpoints may be required at the site. Partial road closure of Highway 11 would be required to install the anticipated well points on the north and south sides of the culvert. The use of traditional wellpoints would likely require trench excavation in the road to approximately 2 m depth to install the wellpoint header pipe at a lower elevation. Construction staging would likely include closure of one side of the road at a time (northbound lanes or southbound lanes) for the installation of wellpoints. Alternatively, the contractor may choose to install sheet piles driven to bedrock around the perimeter of the excavation and the use of drainage sumps to avoid the need for wellpoints.

8.2 GEOTECHNICAL DESIGN PARAMETERS

The soil conditions at this site generally consisted of asphalt, over sand fill with varying amounts of gravel and silt, over an organic soil layer, over sand and silty sand deposits, over till, underlain by bedrock.

For design purposes, the soil models provided in Table 8.1 will be used. The soil model is based on the soil properties encountered in the boreholes from the 2015 and 2016 field investigations. The measured and calculated soil properties are indicated in Figure 4 of Appendix D.

The “degree of site and prediction model understanding for the native soils” has been assessed as “Typical Understanding” as per Section 6.5 of the Commentary on CSA S6-14, Canadian Highway Bridge Design Code (CHBDC), (S6, 1-14).

Table 8.1: Geotechnical Model at Chippewa Creek Culvert

Approximate Elevation (m)		Soil Type	Design Parameters			
From	To		γ (kN/m ³)	ϕ (°)	S_u (kPa)	E (MPa)
311.9	306.9	FILL: Silty sand with gravel to sandy silt	22.0	33	-	10
306.9	305.9	Organic soil (very loose)	18.0	28	-	5
305.9	301.0	Sand to silty sand (very loose to compact)	20.5	30	-	10
301.0	298.6	Silty sand with gravel till (compact)	20.5	32	-	20
298.6	-	Bedrock	26.5	N/A	N/A	N/A

Notes: (1) γ = total unit weight, ϕ = soil friction angle, S_u = undrained shear strength, E = soil/rock modulus

(2) A design water level of 306.7 m will be used (approximately 5.2 m beneath the top elevation of Highway 11 centre line). Submerged unit weights (γ') should be used below the groundwater level.

(3) The elevations provided in the above table reflect a synthesis of the borehole data to incorporate the most significant aspects of the geotechnical design and are not based on any specific location.

8.3 FROST PENETRATION

OPSD 3090.100 indicates that the frost penetration depth at the Chippewa Creek culvert site is 2.0 m. Therefore, footings should be provided with a minimum of 2.0 m of soil cover or equivalent insulation for protection against frost heave. This frost protection requirement applies to the open footing rigid frame option, and for the retaining walls for the rigid frame box options. Frost protection is not required for the box culvert option.

The depth of frost penetration should also be used in the design of frost tapers for the culvert backfill.

8.4 SEISMIC DESIGN CONSIDERATIONS

8.4.1 Site Class

It is recommended that Site Class D as defined in CHBDC (CHBDC, 2014) Section 4.4.3 be used in the seismic design of this site. The energy-corrected average penetration resistance, \bar{N}_{60} was assessed as 23, values used to assess the seismic site classification for this site are as follows.

<u>Depth Below Footing</u>	<u>Soil</u>	<u>\bar{N}_{60}</u>
0 to 5 m	Silty Sand	5
5 to 7 m	Till	20
7 to 30 m	Bedrock	100

8.4.2 Peak Ground Acceleration (PGA)

Seismic hazard values for this site were obtained from Natural Resources Canada (2015 National Building Code). Table 8.2 summarizes the parameters based on a 2475-year return period to be used in forced based design.

Table 8.2: Peak Ground Acceleration Data

<i>PGA</i>	<i>S_a(0.2)</i>	<i>PGA_{ref}</i>	Site Adjusted <i>PGA</i>	Site Class
0.164g	0.261g	0.1312g	0.2018g	D

The 2015 NBC Seismic Hazard calculation sheet that corresponds to this site is provided in Appendix E.

8.4.3 Vertical Acceleration Ratio (A_v)

CSA S6-14 Section 7.5.5.1 indicates that for the design of buried structures the vertical component of an earthquake, expressed as the vertical acceleration ratio, A_v , effectively increases the unit weight of the soil from γ to $\gamma (1+A_v)$. The vertical acceleration ratio, A_v , is to be two-thirds of the Site Adjusted PGA value for the site. The recommended A_v value for this project is 0.1345 g.

8.4.4 Liquefaction Potential

The potential for soil liquefaction was evaluated by comparing the cyclic stress ratio (CSR) caused by the design earthquake with the soil resistance expressed in terms of the cyclic resistance ratio (CRR). The evaluation follows the analysis methodology suggested by Idriss and Boulanger (2008) and is based on the following:

- The blow count data from boreholes.
- A Site Adjusted PGA of 0.2018 g.
- An earthquake magnitude M_w of 6.6.

The analysis indicates a factor of safety against liquefaction of 1.1 to 2.0, and therefore liquefaction is not a concern at this site.

8.5 STRUCTURE FOUNDATIONS

8.5.1 General

It is understood that the following optional structure types are being considered for the Chippewa Creek culvert replacement.

- Rigid Frame Box Culvert
- Rigid Frame Open Footing Culvert or CSP Arch

Table 8.3 compares the possible replacement options for the original culvert.

Table 8.3: Comparison of the Replacement Options for the Original Culvert

Option	Advantages	Disadvantages	Relative Cost	Risk/Consequences	Rank
Precast Rigid Frame Box	<ul style="list-style-type: none"> • reduces traffic management period • reduces water control period • slightly less dewatering volume • does not require frost cover • low bearing capacity requirement 	<ul style="list-style-type: none"> • needs heavy lifting equipment • it may be difficult to manage flow during construction 	Medium	<ul style="list-style-type: none"> • may be limited precast suppliers/non-competitive bidding environment 	1
Cast-in-Place Rigid Frame Box	<ul style="list-style-type: none"> • suitable if site is not conducive to heavy equipment for installing precast sections • does not require deep excavations to found below frost penetration depth • low bearing capacity requirement 	<ul style="list-style-type: none"> • slower construction process • greater unwatering volume required • additives required to retard the concrete curing process 	Medium	<ul style="list-style-type: none"> • limited concrete plants/non-competitive bidding environment • concrete curing process 	2
Rigid Frame Open Footing/CSP Arch	<ul style="list-style-type: none"> • maintains the natural streambed 	<ul style="list-style-type: none"> • slower construction process • greater unwatering volume required • deeper founding required for frost protection • additives required to retard the concrete curing process • native sands are typically loose with low bearing resistance 	High	<ul style="list-style-type: none"> • may be limited concrete plants/non-competitive bidding environment • concrete curing process 	Not considered feasible

The presence of the saturated very loose silty sand layer will negatively impact design options requiring cast-in-place footings such as Rigid Frame Open Footing culvert or CSP arch culvert with concrete footing options. This would require greater effort in providing groundwater control

and a stable base for footing excavation and construction. For this reason, open footing foundations are not considered appropriate for the proposed culvert replacement at this site.

It is noted that regardless of the option selected, the existing culvert is to be removed.

The foundation soils at the site generally can provide adequate support for rigid frame box options. Precast concrete box culverts would be the preferred options due to the anticipated reduced construction period and reduced excavation depth.

8.6 FOUNDATION RECOMMENDATIONS

8.6.1 Geotechnical Resistances – Box Culvert and Retaining Wall

The following geotechnical resistances are provided for the possible rigid frame concrete box culvert option described earlier. It is recommended that the replacement box culvert be founded on structural backfill consisting of compacted OPSS Granular A placed on the native soil. The excavations should be backfilled with structural backfill consisting of a compacted OPSS Granular A. A 200 mm layer of OPSS Granular A should be placed and compacted beneath the culvert for bedding purposes. For the case of a pre-cast rigid frame box culvert, a 75 mm layer of uncompacted OPSS Granular A should be placed between the compacted Granular A and the underside of the box for bedding purposes. The edges of the pad should extend at least 0.5 m horizontally away from the footing in all directions. The Granular A should be placed within the influence zone of the footing which is defined by a 1:1 line extending down and away from the top of the pad in all directions.

It is recommended that the segmental concrete block retaining wall be founded on structural backfill consisting of compacted OPSS Granular B Type II placed on undisturbed native soil. The excavations should be backfilled with structural backfill consisting of a compacted OPSS Granular B Type II. A 200 mm layer of OPSS Granular A should be placed and compacted, over the OPSS Granular B Type II pad, beneath the retaining wall for bedding purposes. The edges of the pad should extend at least 0.5 m horizontally away from the footing in all directions. The granular pad (OPSS Granular B Type II and OPSS Granular A) should be placed within the influence zone of the footing which is defined by a 1:1 line extending down and away from the top of the pad in all directions.

The geotechnical resistances provided in Table 8.4 may be used in the design provided the footings are placed on granular bedding over undisturbed native material as described above and all organic soil has been removed.

An approximately 1 m thick organic soil layer containing wood pieces is present below the fill and in the vicinity of the culvert invert elevation. At the location of the culvert outlet the organic soil layer extended to about elevation 305.0 m. All vegetation, organic soils and other deleterious materials must be removed from beneath the proposed culvert and retaining wall foundations. Where deleterious materials are encountered beneath the retaining wall, the material should be excavated, wasted and replaced with Granular B Type II compacted in lifts no greater than 0.3 m. The lateral extent of such excavation should include all deleterious material within the influence zone of the retaining wall foundations. The base of the working

FOUNDATION INVESTIGATION AND DESIGN REPORT

May 2017

surface (culvert and retaining wall) should be examined by a geotechnical inspector to confirm that the soils are consistent with those observed in the boreholes and to ensure that there is no loose or deleterious material present. Any loose, disturbed, or organic material identified during the inspection will require removal to the satisfaction of the geotechnical inspector. Where construction is undertaken during winter conditions, the working surface subgrade should be protected from freezing.

Table 8.4: Recommended Spread Footing Design Parameters

Founding Element	Founding Elev. (m)	Footing Size (m x m)		Factored Geotechnical Resistance at ULS (kPa) $\phi_{gu} = 0.5$	Factored Geotechnical Reaction at SLS (kPa) $\phi_{gs} = 0.8$
		Width (m)	Length (m)		
Rigid Frame Box Culvert on structural backfill over native sand to silty sand	305.7	3.0 to 4.0	39.0	425	110 ⁽⁵⁾
Segmental Concrete Block Retaining Wall on structural backfill over sand to silty sand	306.0	2.0	5.3	140	100 ⁽⁶⁾

Notes:

- (1) The Geotechnical Resistances were estimated assuming a consequence classification of "Typical Consequence" with a consequence factor equal to 1.0. In accordance with Section 6.5 and Table 6.1 of CHBDC, 2014.
- (2) In accordance with Section 6.9 and Table 6.2 of the CHBDC, 2014, a resistance factor of 0.5 has been applied to calculate the factored geotechnical resistance at ULS.
- (3) The geotechnical reaction at SLS typically corresponds to a maximum settlement of 25 mm. In accordance with Section 6.9 and Table 6.2 of CHBDC, 2014, a geotechnical resistance factor of 0.8 has been applied to calculate the factored geotechnical resistance at SLS.
- (4) The use of OPSS Granular A material beneath the culvert foundation is not for the purpose of achieving high bearing resistances or reactions but rather to ensure that the foundations are supported on a consistent engineered structural backfill once the existing fill and soils have been removed from beneath the influence zone of the culvert footings.
- (5) The construction of the box culvert within the existing embankment will result in a net unloading and therefore very little settlements are anticipated as part of the construction, despite the very loose state of the underlying sand. It is estimated that the existing pressure on the underlying sand is in the order of 125 kPa and therefore a pressure at the base of the culvert of greater than 125 kPa would be required in order to produce any significant settlements.
- (6) The low SLS reaction reflects the relatively loose nature of the silty sand encountered at the site.

8.6.2 Sliding Resistance

The unfactored horizontal resistance of spread footings may be calculated using the following unfactored coefficients of friction:

0.55 between OPSS Granular A and pre-cast concrete

0.35 native silty sand and pre-cast concrete

In accordance with Table 6.2 of the CHBDC 2014, a resistance factor against sliding (frictional) of 0.8 should be applied to obtain the resistance at ULS.

8.7 LATERAL EARTH PRESSURES

8.7.1 Lateral Earth Pressures under Static Conditions

Earth pressures will need to be considered in the design of the culvert walls, as well as for roadway protection systems and retaining walls.

Computation of earth pressures should be in accordance with Section 6.12 of the CHBDC, 2014. For walls that are designed to allow rotation, active earth pressure may be used for design. For rigidly tied and unyielding structures, the at-rest earth pressure should be used for design. For a rigid frame box culvert, the walls are considered to be unyielding and the at-rest earth pressure should be used for design. The unfactored soil parameters provided in Table 8.5 may be used for design of walls with a horizontal backfill. The unfactored soil parameters provided in Table 8.6 may be used for design of walls with a 2H:1V backfill. The effects of compaction should be accounted for by applying a compaction surcharge as shown in Figure 6.6 of the CHBDC, 2014.

The total active (P_A) and passive (P_P) thrusts can be calculated using the following equations:

$$P_A = \frac{1}{2} K_a \gamma H^2$$

$$P_O = \frac{1}{2} K_o \gamma H^2$$

$$P_P = \frac{1}{2} K_p \gamma H^2$$

Where H is the height of the wall. Values for K_a , K_p , K_o and γ are provided in Tables 8.5 and 8.6. The thrust acts at a point one third up the height of the wall. The angle of friction between the soil and the wall has been set at 0° to provide a conservative estimate.

Table 8.5: Recommended Non-Seismic Earth Pressure Parameters (Horizontal Backfill)

Parameter	OPSS Gran B Type I	OPSS Gran A and Gran B Type II	Native Sand to Silty Sand	Existing Silty Sand Fill
Bulk Unit Weight, γ (kN/m ³)	21.2	22.0	20.5	22.0
Effective Friction Angle, $^\circ$	32	35	30	33
Coefficient of Earth Pressure at Rest (K_o)	0.47	0.43	0.50	0.46
Coefficient of Active Earth Pressure (K_a)	0.31	0.27	0.33	0.29
Coefficient of Passive Earth Pressure (K_p)	3.25	3.69	3.00	3.39

FOUNDATION INVESTIGATION AND DESIGN REPORT

May 2017

For roadway protection systems the parameters provided in Table 8.5 may be used for calculations. The appropriate pressure distributions will depend on the lateral support methods used; ie. cantilever, dead man anchors, raking, and bracing. The appropriate pressure distribution may be selected from Chapter 26 of the Canadian Foundation Engineering Manual.

Table 8.6: Recommended Non-Seismic Earth Pressure Parameters (2H:1V Backfill)

Parameter	OPSS Gran B Type I	OPSS Gran A and Gran B Type II	Native Sand to Silty Sand	Existing Silty Sand Fill
Bulk Unit Weight, γ (kN/m ³)	21.2	22.0	20.5	22.0
Effective Friction Angle, ϕ	32	35	30	33
Coefficient of Earth Pressure at Rest (K_0)	0.68	0.62	0.72	0.66
Coefficient of Active Earth Pressure (K_a)	0.47	0.39	0.54	0.44
Coefficient of Passive Earth Pressure (K_p)	8.61	10.82	7.46	9.27

For retaining walls the parameters provided in Table 8.6 may be used for calculations. The appropriate pressure distributions will depend on the wall system used. The appropriate pressure distribution may be selected from Chapter 26 of the Canadian Foundation Engineering Manual and the wall manufacturers recommended design procedures.

8.7.2 Lateral Earth Pressures under Seismic Conditions

The culvert walls or RSS walls (if any) should also be designed to resist the earth pressures induced under seismic loading conditions. The seismic earth pressures may be calculated using the parameters detailed in Table 8.8 (2H:1V backfill).

The total active and passive thrusts under seismic loading conditions can be calculated using the following equations:

- $P_{AE} = \frac{1}{2} K_{AE} \gamma H^2 (1 - k_v)$
- $P_{PE} = \frac{1}{2} K_{PE} \gamma H^2 (1 - k_v)$

where:

- K_{AE} = active earth pressure coefficient (combined static and seismic)
- K_{PE} = passive earth pressure coefficient (combined static and seismic)
- H = height of wall
- k_h = horizontal acceleration coefficient
- k_v = vertical acceleration coefficient
- γ = total unit weight

For this site, the following design parameters were used to develop the recommended K_{AE} and K_{PE} values as per CHBDC 2014.

Table 8.7: Seismic Design Parameters to Estimate Lateral Earth Pressures

Site Adjusted PGA	Horizontal Acceleration Coefficient, k_{ho}	Horizontal Acceleration Coefficient, k_h
	Non-Yielding	Yielding > 25 mm
0.2018g	0.2018	0.1009

Note: k_{ho} is the seismic horizontal acceleration coefficient that corresponds to zero wall movement and is equal to the site-adjusted **PGA** estimated at ground surface. The vertical acceleration coefficient (k_v) should be ignored in the calculations as per CHBDC 2014, section C4.6.5.

The angle of friction between the soil and the wall has been set at 0° to provide a conservative estimate.

Table 8.8: Recommended Seismic Earth Pressure Parameters (2H:1V Backfill)

Parameter	OPSS Granular B Type I		OPSS Granular A and Granular B Type II		Existing Silty Sand Fill	
Bulk Unit Weight, γ (kN/m ³)	21.2		22.0		22.0	
Effective Friction Angle (°)	32		35		33	
Wall type: (a) yielding; (b) non-yielding	(a)	(b)	(a)	(b)	(a)	(b)
Active Earth Pressure (K_{AE})	0.77	N/A	0.55	N/A	0.64	N/A
Height of Application of P_{AE} from base as a ratio of wall height (H)	0.447	N/A	0.421	N/A	0.42	N/A

Note: Non-yielding walls with 2H:1V backfill is not anticipated as part of this project.

8.8 EMBANKMENT DESIGN

The roadway profile at the culvert location will not be raised above the existing profile. The existing embankment grade is not anticipated to be changed appreciably. A slight widening of the embankment may be carried out. Any proposed embankment widening can be constructed of the same material as the existing one. Any fill placement associated with the embankment widening/grade raise (on the embankment slopes only) should be carried out in accordance with OPSD 208.010 Benching of Earth Slopes.

8.8.1 Embankment Settlement

The existing roadway profile will be maintained; no grade raises are proposed at the culvert location. The profile and footprint of the existing embankment is not anticipated to be significantly altered. Less than 25 mm of settlement is anticipated for the proposed embankment modification.

It will likely be proposed that the contractor can leave portions of the existing culvert in place. Depending on the construction staging, the top slab of the existing culvert may be removed and filled with compacted granular, or the existing culvert may be filled with unshrinkable fill. Any settlement due to the additional loads of filling the existing culvert would be immediate (underlying soils are non-cohesive) and the embankments could be regraded at the time of construction prior to the placement of pavement structure.

8.8.2 Stability of Slopes

A global stability analysis of a 2H:1V embankment slope with and without a retaining wall was carried out. Both static and conventional pseudo-static limit equilibrium slope stability analysis methods (Geo-Slope, 2012) were used.

The pseudo-static stability analysis of the embankment slope considered seismic loading of 0.1009 which is one-half of the Peak Ground Acceleration (PGA).

The slope stability analysis results are presented in Figures 5a, 5b, 5c and 5d in Appendix D.

The slope stability evaluation results indicate that the failure planes generally tend to be relatively shallow. The estimated factor of safety against failure was 1.5 for static and 1.1 for seismic conditions without a retaining wall. The estimated factor of safety against failure was 1.5 for static and 1.2 for seismic conditions with a retaining wall. Based on this slope stability analysis, the factor of safety against the shallow critical failure plane meets the required target value of 1.5 for static (drained and undrained) and 1.1 (seismic) for highway embankments.

No sign of embankment instability was observed during the foundation drilling. Stantec is not aware of a history of slope instabilities at the culvert location.

8.9 EROSION AND SCOUR PROTECTION

8.9.1 Erosion Protection

Slope protection and drainage measures will be required to ensure the long-term surficial stability of the embankment slopes. All slopes within 3 m of the culvert inlet and outlet should be surfaced with rip-rap at least 300 mm thick placed on a Class II non-woven filter fabric; the rip-rap should extend up the slope to 0.3 m above the design high water level. Where embankment construction includes earth fill, normal slope vegetation should be established as soon as possible after completion of the embankment fills in order to control surficial erosion.

The contractor should provide silt fences and erosion control blankets, as required, throughout the duration of the construction to prevent silt/sediment from running off the site.

8.9.2 Scour Protection

The proposed culvert will be a box culvert; hence, scouring is not anticipated to be of major concern at this site. No special scour protection measures are required for this site.

8.10 CEMENT TYPE AND CORROSION PROTECTION

One sample of the native soil was submitted to Paracel Laboratories in Ottawa, Ontario for analysis of pH, water soluble sulphate and chloride concentrations, and resistivity. The testing was completed to determine the potential for degradation of the concrete in the presence of soluble sulphates and the potential for corrosion of exposed steel used in foundations and buried infrastructure. The analysis results are summarized in Table 4.3.

The concentration of soluble sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with soil and groundwater at the site. The soluble sulphate concentrations for the sample was 61 µg/g. Soluble sulphate concentrations less than 1000 µg/g generally indicate that a low degree of sulphate attack is expected for concrete in contact with soil and groundwater. Type GU (General Use) Portland Cement should therefore be suitable for use in concrete at this site.

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. The soil pH value was 7.2 which is within what is considered the normal range for soil pH of 5.5 to 9.0. The pH level of the tested soil does not indicate a highly corrosive environment. The resistivity result was 10 Ohm-m which suggests a severe degree of corrosiveness for steel. The test results provided in Table 4.3 may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects.

9.0 CONSTRUCTION CONSIDERATIONS

9.1 CONSTRUCTION STAGING

The proposed replacement structure is anticipated to involve a staged construction. Construction staging would likely include closure of one side of the road at a time (northbound lanes or southbound lanes). Due to the looseness and susceptibility of the sands encountered at the site, roadway protection will likely be required. Highway traffic is to be controlled using by setting up traffic control for one lane in each direction on one side of the highway during staged construction.

The use of wellpoints may be required at the site. Partial road closure of Highway 11 would be required to install the well points on the north and south sides of the culvert. The use of traditional wellpoints would likely require trench excavation in the road to approximately 2 m depth to install the wellpoint header pipe at a lower elevation; roadway protection as discussed in Section 9.2 would likely need to be implemented. Construction staging would likely include closure of one side of the road at a time (northbound lanes or southbound lanes) for the installation of wellpoints.

The anticipated requirement to construct a trench for the installation of suction wellpoints reflects the practical suction height of 4.5 m for this type of dewatering system. An alternative approach would consist of using induction wellpoints which can be used to effectively remove water to depths of over 50 m. The use of induction well-points would eliminate the requirement of trenching and the associated roadway protection. Generally, the cost of induction wellpoints is twice that of suction wellpoints.

As an alternative to wellpoints the contractor could choose to install sheet piles driven to bedrock around the perimeter of the excavation and sump and pump from within the excavation.

9.2 TEMPORARY ROADWAY PROTECTION

9.2.1 General

Temporary roadway protection is anticipated to form part of a staged construction approach that will be required to maintain traffic flow during construction. The configuration of the temporary roadway protection has not yet been finalized but will likely consist of a boxed sheet pile configuration with one end placed near the centerline of the road (Highway 11) and internal dewatering.

The following table compares the available roadway protection options considered for the culvert replacement:

Table 9.1: Comparison of Roadway Protection Systems

Option	Advantages	Disadvantages	Relative Cost	Risk & Consequences
H-Piles with timber lagging; struts/rakers		<ul style="list-style-type: none"> • Would require a cut-off to allow lagging installation below water level • More difficult to control unwatering 	Low	<ul style="list-style-type: none"> • Major water control • Likely require a well point system or other dewatering method to lower the groundwater level during construction
Steel sheet piles (SSP) driven to form a boxed cofferdam	<ul style="list-style-type: none"> • No unwatering required during roadway protection installation • The sheet piles could be driven sufficiently beyond the groundwater level such that less dewatering would be required to lower the groundwater table to avoid base instability 	<ul style="list-style-type: none"> • Potentially difficult to drive/install in the gravel and sand fill 	High	<ul style="list-style-type: none"> • Damage of sheet pile walls during driving

If the precast rigid box option is carried forward and provided an appropriate groundwater control method is put in place, coffered sheet piling would be the preferred option. Lateral resistance could be provided by internal bracing within the coffered sheet piles. The use of soldier pile and lagging system will likely be not feasible due to the potential difficulty with unwatering. It should be noted that high SPT "N" values were measured in the embankment fill material in boreholes BH16-2, Borehole No. 2 and Borehole No. 3; The high SPT "N" values suggest obstruction during driving of the sheet piles could be encountered. A Notice to Contractor is provided in Appendix F which alerts the contractor of the possible presence of cobbles and boulders.

Within the silty sand to sand layer, SPT "N" values were less than 4 blows per 0.3 m suggesting very loose zones. Densification of the very loose zones may occur during the vibratory

FOUNDATION INVESTIGATION AND DESIGN REPORT

May 2017

installation of sheet piles; the densification of the very loose zones could cause localized settlements around the perimeter of the sheet piles. Surface distortion should be leveled with compacted structural backfill.

Computation of earth pressures should be in accordance with Section 6.12 of the Canadian Highway Bridge Design Code (CHBDC, 2014). For the second stage of excavation the effects of backfill compaction should be accounted for by estimating the compaction-induced load in accordance with the methodology described in Figure 6.6 of the CHBDC.

For temporary excavation support the total active (P_A), at rest (P_O) and passive (P_P) thrusts will depend on the excavation support method selected by the contractor. The appropriate earth pressure distributions can be estimated using Chapter 26 of the Canadian Foundation Engineering Manual.

A triangular earth pressure distribution can be used to calculate the active, at rest, and passive earth pressure thrusts which allow deformations for the general loading condition for cantilevered walls and for walls laterally supported by anchors or rakers. The horizontal earth pressure distribution for such a sheet pile wall is calculated as follows.

$$p = K (\gamma z + q)$$

where:

p (kPa)	earth pressure at depth z
K	applicable coefficient of earth pressure K_a , K_o or K_p
γ (kN/m ³)	soil unit weight
z (m)	height of soil above any point, measured from the top of the excavation for the active thrust or bottom of the excavation for the passive resistance below the excavation level
q (kPa)	surcharge from traffic or other sources

Table 9.2 provides geotechnical design parameters applicable to this site.

Table 9.2: Soil Parameters - Temporary Roadway Protection

Approximate Elevation (m)		Soil Type	Design Parameters				
From	To		γ (kN/m ³)	ϕ (°)	Active K_a	At Rest K_o	Passive K_p
312.0	306.9	FILL: Silty sand with gravel to sandy silt	22.0	33	0.29	0.46	3.39
306.9	305.9	Organic soil (very loose)	18.5	28	0.36	0.53	2.77
305.9	301.0	Sand to silty sand (very loose to compact)	20.5	30	0.33	0.50	3.00

Notes: (1) k_a , k_o , k_p are the coefficients of earth pressure calculated assuming a horizontal ground surface behind the support system.

(2) Roadway protection types and materials will dictate the actual pressure distributions. For the case of fixed-deformation supports, such as provided by internal bracing, the "apparent earth pressure distribution" method should be used in lieu of the triangular distribution.

(3) The elevations provided in the above table are assumed to represent inferred stratigraphic boundaries near the centreline of the existing Highway 11 at the culvert location.

(4) Groundwater is at approximate elevation 306.7 m.

The contractor will ultimately be responsible to develop and implement a roadway protection system meeting the requirements of OPSS 539, including establishing appropriate geotechnical design parameters.

Shoring design should meet the requirements of Performance Level 2 as per OPSS 539 and should consider traffic loading. Performance Level 2 specifies a Maximum Angular Distortion of 1:200 and a Maximum Horizontal Displacement of 25 mm. The sheet pile and bracing system must be designed not to exceed these limits. Horizontal movement should be monitored throughout the culvert replacement process as described in OPSS 539. The monitoring requirements outlined in OPSS 539 are considered to be appropriate for this project.

9.2.2 Geotechnical Lateral Resistance

Should the contractor propose to use a system with raked wales or raked sheet piles, the effects of load inclination should be incorporated into the design, in accordance with section 6.10 of the CHBDC, 2014.

The toe resistance of a soldier pile may be calculated using a passive (K_p) triangular pressure distribution acting over an equivalent width of three times the pile width or pile socket diameter (if applicable) using the soil parameters provided in Table 8.2. Greater resistances could be developed by socketing and concreting the soldier piles into bedrock. For the case of sheet piles, actual wall dimensions are used to calculate the passive resistance.

Alternatively, ground anchors could provide lateral support to a soldier pile system or a sheet pile system. The ground anchors should be designed in accordance with Section 6.13 of the CHBDC, 2014. A preliminary capacity of a pressure grouted anchor can be estimated based on Table 26.6 of the Canadian Foundation Engineering Manual (CFEM) (2006). Anchors installed in bedrock and soil can be designed with an ultimate load transfer of 730 kN/m and 100 kN/m respectively. This value assumes:

- bond length does not exceed 8 m;
- nominal diameter of the anchor is between 150 mm and 200 mm;
- grout is injected using a pressure of 1 MPa;
- and the center-to-center anchor spacing at the bonds is more than four times the anchor diameter or 20% of the bond length.

Using a resistance factor ϕ_{gu} of 0.4, the estimated anchor capacity at ULS would be calculated as 292 kN/m of anchor bonded length up to a maximum value of 2336 kN for rock anchors; the respective values for soils anchors will be 40 kN/m and 320 kN.

9.3 EXCAVATION AND BACKFILLING

Excavation and backfill for the replacement structure should be carried out in accordance with OPSS 902 Construction Specification for Excavation and Backfilling – Structures and SP422501.

All vegetation, organic soils and other deleterious materials must be removed from beneath the proposed structure foundation. Where deleterious materials are encountered, the material should be excavated, wasted and replaced. The lateral extent of such excavation should include all deleterious material within the influence zone of the foundations.

Side slopes for open cut excavations should conform to the Occupational Health and Safety Act (OHSA) regulations for Construction Projects. The soils encountered at the site may be classified in accordance to the OHSA as follows:

Existing Fill	Type 3 Soil
Native Silty Sand	Type 4 Soil

Grading work for reinstatement of the highway and embankments along the existing culvert alignment should be carried out in accordance with OPSS 206 Construction Specification for Grading and SP 206S03. Backfilling of precast concrete box culverts should be carried out in accordance with MTOD 803.021 or OPSD 803.010 for culverts with spans less than or equal to 3 m.

Bedding, leveling and cover material for the culverts should consist of OPSS Granular A.

9.4 REUSE OF EXCAVATED MATERIAL

The native material in the vicinity of the project site consists of organic soil, sand, and silty sand. This material will not be suitable as backfill within and behind the structures for the proposed replacement structure. The silty sand and existing embankment fill material may be used for embankments if properly processed and compacted.

9.5 TEMPORARY CONSTRUCTION UNWATERING AND DEWATERING

Replacement of the Chippewa Creek Culvert will require excavations below the water level of 306.7 m observed on November 23, 2016. In the existing culvert the creek water flows from west to east and groundwater levels observed were 0.1 m to 0.3 m above the inverts of the culvert.

A rising head permeability test was completed in the borehole BH16-2 monitoring well on November 23, 2016 to determine the hydraulic conductivity of the soil. The test results were analyzed using the Bouwer-Rice solution method for an unconfined aquifer model. The analysis yielded a hydraulic conductivity 'K' value of 2.8×10^{-4} cm/s, which falls within the range of values typically found for poorly graded sand. The results of grain size distribution testing from samples obtained in BH15-2 shows that locally the deposit contains less fines and would be more permeable than observed in BH16-2; based on Hazen's formula the cleaner sand in BH15-2 would be estimated at 8×10^{-3} cm/sec. From the test results and the subsurface information

FOUNDATION INVESTIGATION AND DESIGN REPORT

May 2017

gathered at the site, it is evident that groundwater control will be a necessary requirement during construction.

The use of well points may be required at the site. The use of traditional suction well points would likely require trench excavation in the road to approximately 2 m depth to install the well point header pipe at a lower elevation, to reduce the suction height to the maximum practical limit of 4.5 m. Induction well-points could also be considered which have significantly greater application depths, but are generally considered to be double the cost of suction well-points. The use of well-points requires partial road closure for the duration of the construction.

For stream flow control at the site, it is anticipated that a sheet-piled cutoff wall will be used and that the water will be pumped across Highway 11. Given the highly permeable nature of the sands encountered on site, the use of an Aquadam type system would need to consider the potential for erosion piping beneath the temporary dam.

For reference, the results of the grain size distribution tests (and Unified Soil Classifications) completed on the predominant soil strata encountered in the boreholes have been compared to the grain size curves and soil types referenced in Supplementary Standard SB-6 of the 2012 Ontario Building Code (OBC). The OBC has been used as a guideline to estimate the likely range in the coefficient of permeability of the soils encountered in the investigation. It is noted that the industry typically refers to "hydraulic conductivity" rather than "coefficient of permeability" in this respect. The terms are often considered interchangeable, but for purposes of this report the values provided are in the form of "length/time" (cm/sec) and are therefore considered strictly applicable to "hydraulic conductivity", and hence "hydraulic conductivity" is used herein.

Based on the comparison conducted, the following values are provided:

<u>Soil Type</u>	<u>Estimated Hydraulic Conductivity</u>	<u>Comment</u>
Silty Sand (SM)	10^{-3} to 10^{-5} cm/sec	Medium to Low Permeability
Poorly Graded Sand (SP)	10^{-2} to 10^{-3} cm/sec	Permeable

The OBC states, in part, that "it must be emphasized that, particularly for fine grained soils, there is no consistent relationship (between coefficient of permeability and soils of various types) due to the many factors involved". Such factors as structure, mineralogy, density (compactness or consistency), plasticity, and organic contents of the soil can have a large influence on the hydraulic conductivity; variations in excess of an "order of magnitude" are common place in this respect.

The estimated hydraulic conductivity for the native soil at the site is expected to range from 1×10^{-6} m/s to 1×10^{-4} m/s. Unwatering of the structure excavations using conventional sump and pump techniques will not likely be adequate. It is recommended that the contract documents for this site include a special provision to address issues related to groundwater control during construction. A Notice to Contractor is provided in Appendix F which alerts the contractor to the presence of high permeability soils at the site.

10.0 SPECIFICATIONS

The following specifications are referenced in this report:

Table 10.1: Specifications Referenced in Report

Document	Title
OPSS 206	Construction Specification for Grading
OPSS 539	Construction Specification for Temporary Protection System
OPSD 803.010	Backfill and Cover for Concrete Culverts with Spans Less Than or Equal to 3.0 m
OPSS 902	Construction Specification for Excavation and Backfilling - Structures
OPSD 3090.100	Foundation Frost Depths for Northern Ontario
OPSD 3101.150	Walls, Abutment, Backfill Minimum Granular Requirements
SP 206S03	Earth Excavation, Grading
MTOD 803.021	Bedding and Backfill for Precast Concrete Box Culverts
OPSD 208.010	Benching of Earth Slopes
SP 422501	Precast Concrete Box Culvert
OPSS 422	Construction Specifications for Precast Reinforced Concrete Box Culverts and Box Sewers in Open Cuts

11.0 REFERENCES

- ASTM. 1999. Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils (ASTM D1586). ASTM International, West Conshohocken, PA.
- ASTM. 2000. Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System) (ASTM D2487). ASTM International, West Conshohocken, PA.
- Barnet, P.J., Henry, A.P. and Babuin, D. 1991. Quaternary geology of Ontario, Ontario Geological Survey, scale 1:1 000 00.
- Canadian Foundation Engineering Manual (CFEM). 2006. Fourth Edition. Canadian Geotechnical Society, 488 p.
- Center for Earthworks Engineering Research (CEER). 2012. Modified Sheet Pile Abutments for Low-Volume Road Bridges. IHRB Project TR-568. Iowa Highway Research Board.
- CHBDC. 2014. Canadian Highway Bridge Design Code. Canadian Standards Association, Mississauga, Ontario.
- Geo-Slope International, Ltd. 2010. GeoStudio 2010 (Slope/W 2010), Calgary, Alberta.
- Meyerhof, G.G. 1976. Bearing Capacity and Settlement of Pile Foundations. Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, New York, NY, vol. 102, No. GT3, pp. 196-228.
- Ontario Building Code. 2012. Ministry of Municipal Affairs and Housing, Building and Development Branch.

12.0 CLOSURE

A soil investigation is a limited sampling of a site. The conclusions given herein are based on information gathered at the specific borehole locations. Should any conditions at the site be encountered which differ from those at the borehole locations, we request that we be notified immediately in order to assess the additional information and its effects on the above recommendations.

We trust the information presented herein meets your present requirements. Should you have any questions or require additional information, please do not hesitate to contact us.

This report has been prepared by Zachary Popper and reviewed by Christopher McGrath and Raymond Haché, Designated Principal MTO Foundation Contact.

Respectfully submitted,

STANTEC CONSULTING LTD.



Zachary Popper, P.Eng.
Geotechnical Engineer



Christopher McGrath, P.Eng.
Associate- Senior Geotechnical Engineer



Raymond Haché, M.Sc., P.Eng.
Designated Principal MTO Foundation Contact



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APPENDIX A

Drawing No. 1 – Borehole Location Plan and Soil Strata Plot

Drawing No. 2 - Borehole Location Plan and Soil Strata Plot

Drawing No. P1 – Preliminary General Arrangement

Site Photos

165000836_Hwy 11_Chippewa Creek_May2017.dwg
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CREATED BY: GBB
MODIFIED: 2017-05-03
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Printed: May 03, 2017
MINISTRY OF TRANSPORTATION, ONTARIO
PR-D-707
86-05

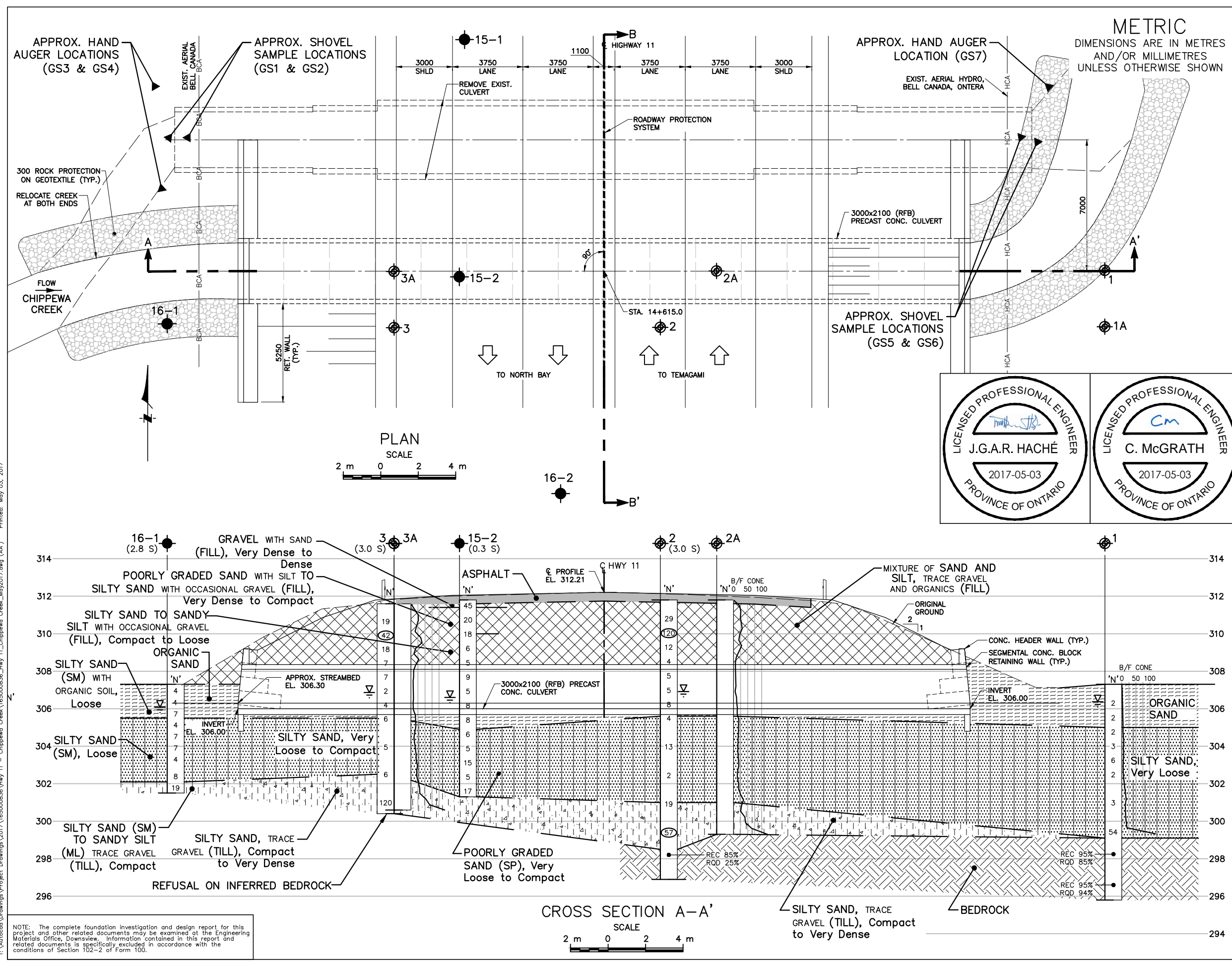


PLATE No
CONT
WP 5143-06-01

HWY 11
CHIPPEWA CREEK CULVERT
BOREHOLE LOCATIONS & SOIL STRATA

Stantec

DISTRICT OF NIPISSING
TOWNSHIP OF WIDDIFIELD

KEY PLAN
1 km 0 1 2 km

LEGEND

No	ELEVATION	MTM ZONE 10 COORDINATES NORTH	MTM ZONE 10 COORDINATES EAST
16-1	307.3	5 136 636.0	307 386.0
16-2	311.9	5 136 627.0	307 407.0
15-1	311.9	5 136 651.2	307 401.8
15-2	311.8	5 136 638.5	307 401.6
1	307.3	5 136 640.6	307 436.0
1A	307.3	5 136 637.6	307 436.0
2	311.8	5 136 637.6	307 412.3
2A	311.8	5 136 640.6	307 415.3
3	311.6	5 136 637.6	307 398.2
3A	311.6	5 136 640.6	307 398.2

NOTES

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

REVISIONS

DATE	BY	DESCRIPTION

GEORES No 31L-204

HWY No	HWY	DIST
11	11	

SUBM'D	CM	CHECKED	DATE	2017-05-03	SITE
DRAWN	GBB	CHECKED	APPROVED		DWG 1

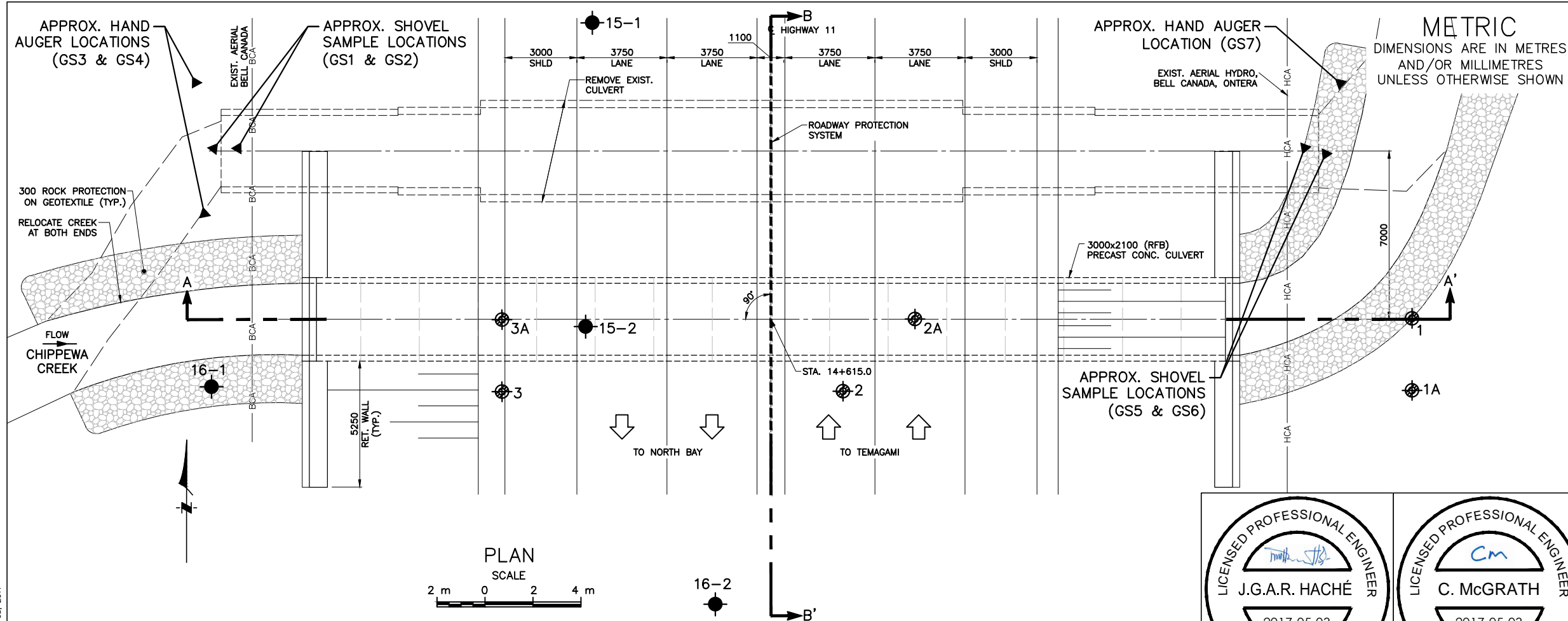


PLATE No
CONT
WP 5143-06-01

HWY 11
CHIPPEWA CREEK CULVERT
BOREHOLE LOCATIONS & SOIL STRATA

Stantec

DISTRICT OF NIPISSING
TOWNSHIP OF WIDDIFIELD

11

LAPORTE ROAD

NORTH BAY

CHIPPEWA CREEK

SITE

KEY PLAN
1 km 0 1 2 km

LEGEND

Borehole

Existing Borehole
(Locations are approximate based on Memorandum File No. 3162-2-4-113 issued By MTO, Soils and Aggregates Section, Engineering Materials office, Central Building, Room 311, dated 96/07/04.)

N Blows/0.3m (Std Pen Test, 475 J/blow)

Inferred cobbles and boulders

CONE Blows/0.3m (60' Cone, 475 J/blow)

WL at time of investigation Nov 2016, May 2015 & June 1995

WL measured in standpipe on Nov 23, 2016

(x.x E/W) Offset in metres, East/West of Cross Section Line B-B'

No	ELEVATION	MTM. ZONE 10 NORTH	COORDINATES EAST
16-1	307.3	5 136 636.0	307 386.0
16-2	311.9	5 136 627.0	307 407.0
15-1	311.9	5 136 651.2	307 401.8
15-2	311.8	5 136 638.5	307 401.6
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1A	307.3	5 136 637.6	307 436.0
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2A	311.8	5 136 640.6	307 415.3
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3A	311.6	5 136 640.6	307 398.2

NOTES

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

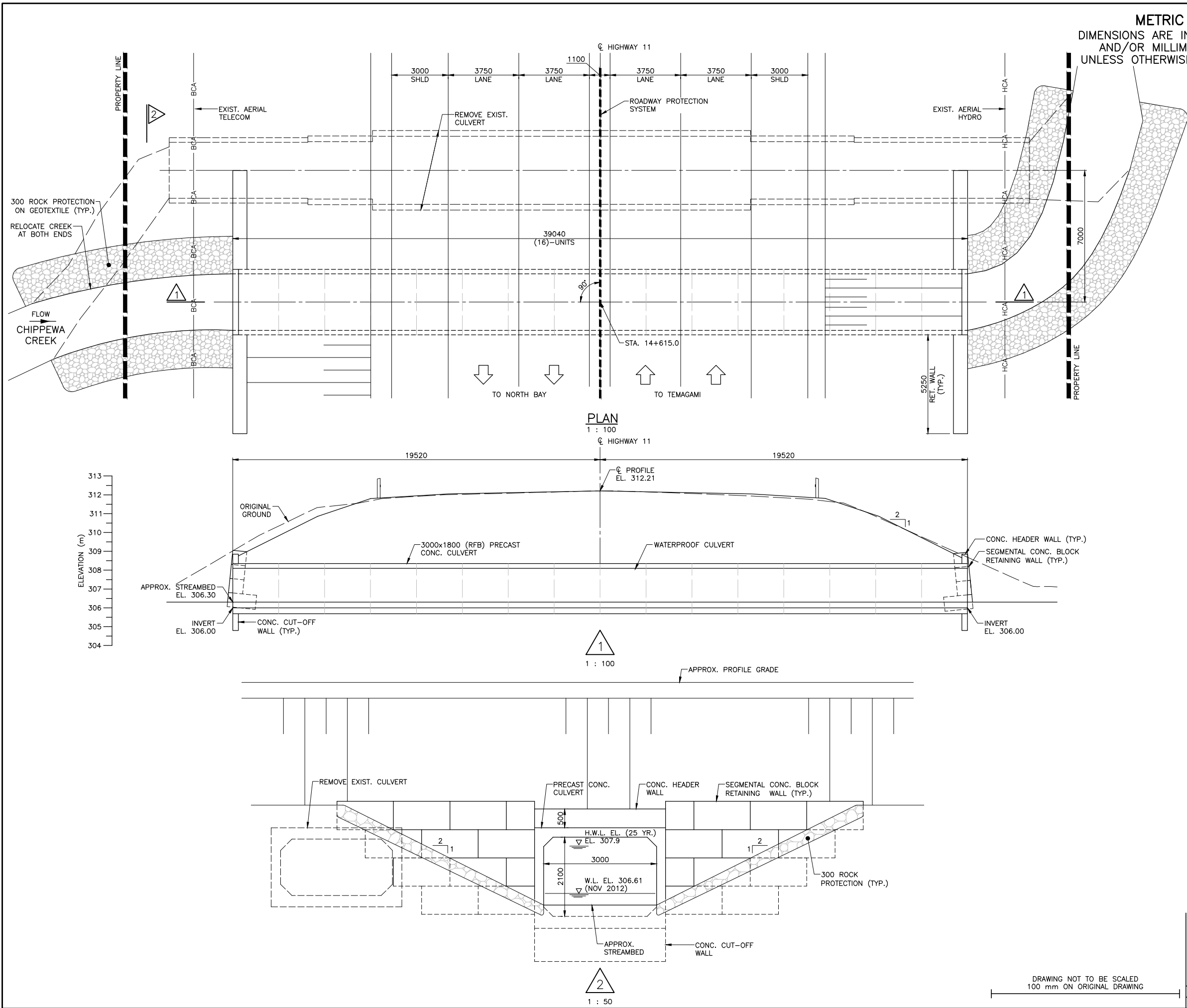
REVISIONS

DATE	BY	DESCRIPTION

GEOCREs No 31L-204

HWY No	HWY 11	DIST
SUBM'D	CM	CHECKED
DATE	2017-05-03	SITE
DRAWN	GBB	CHECKED
APPROVED		DWG 2

165001021-43-362-P1.DWG Jan 10 2017



DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS		DATE	BY	DESCRIPTION
DESIGN	CHK			
DRAWN	A.P.	CHK	M.T.	

CODE CHBDC-2014	LOAD CL-625-ONT	DATE	JAN 2017
SITE 43-362/C	STRUCT	SCHEME	DWG. P1

HWY 11
CONT
WP 5143-06-01

CHIPPEWA CREEK CULVERT
REPLACEMENT
PRELIMINARY
GENERAL ARRANGEMENT

SHEET

- GENERAL NOTES**
- CLASS OF CONCRETE:**
PRE-CAST CONCRETE 35 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
 - CULVERT DESIGN**
CULVERT DESIGN IS THE RESPONSIBILITY OF THE CONTRACTOR, AND SHALL BE IN ACCORDANCE WITH CHBDC 2014, LIVE LOAD SHALL BE CL-625-ONT. MINIMUM WALL/SLAB THICKNESS SHALL BE 250.
 - REINFORCING STEEL:**
REINFORCING STEEL SHALL BE GRADE 400W UNLESS OTHERWISE SPECIFIED.
TENSION LAP LENGTHS NOT INDICATED ON THE CONTRACT DRAWINGS 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:**
- THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS, DETAILS AND ELEVATIONS OF THE EXISTING STRUCTURE THAT ARE RELEVANT TO THE WORK SHOWN ON THE DRAWINGS PRIOR TO COMMENCEMENT OF THE WORK. ANY DISCREPANCIES SHALL BE REPORTED TO THE CONTRACT ADMINISTRATOR AND THE PROPOSED ADJUSTMENT OF THE WORK REQUIRED TO MATCH THE EXISTING STRUCTURE SHALL BE SUBMITTED FOR APPROVAL.
 - THE LOCATION OF EXISTING UTILITIES IS APPROXIMATE. THE CONTRACTOR SHALL PROTECT THE UTILITIES FROM DAMAGE DURING WORK OPERATIONS.
 - DEBRIS FROM STRUCTURE REMOVALS SHALL BE PREVENTED FROM ENTERING THE WATERCOURSE.
 - 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.
 - ROADWAY PROTECTION SYSTEMS SHALL BE DESIGNED FOR PERFORMANCE LEVEL 2.
 - THE CONTRACTOR SHALL ISOLATE WORK AREAS FROM THE WATERCOURSE FLOW FOR CULVERT REPLACEMENT AS REQUIRED TO COMPLETE ALL WORK IN THE DRY.

- LIST OF DRAWINGS:**
- GENERAL ARRANGEMENT
 - BOREHOLE LOCATIONS & SOIL STRATA
 - CONSTRUCTION STAGING
 - CULVERT DETAILS
 - RETAINING WALLS
 - EXCAVATION AND BACKFILL
- LEGEND:**
RFB -DENOTES RIGID FRAME BOX

APPLICABLE STANDARD DRAWINGS
OPSD-3941.200 FIGURES IN CONCRETE SITE NUMBER AND DATE LAYOUT



Photo No. 1: Highway 11 looking south. Approximate borehole locations are painted on the asphalt.



Photo No. 2: Highway 11 looking south at culvert location.



Photo No. 3: Culvert inlet, west side of Highway 11.



Photo No. 4: Culvert outlet, east side of Highway 11 showing rock fill on slope.



Photo No. 5: Culvert inlet, west side of Highway 11.



Photo No. 6: Interior of culvert looking east.



Photo No. 7: Interior of culvert looking west.



Photo No. 8: Culvert outlet, showing creek bed and sediment in culvert.



Photo No. 9: Deterioration of stone wall and concrete in middle-west section of culvert.



Photo No. 10: Deterioration of stone wall and concrete in middle-west section of culvert.



Photo No. 11: Deterioration of stone wall and concrete in middle-west section of culvert.

APPENDIX B

Symbols and Terms Used on Borehole Records

Stantec Borehole Records

Borehole Records from 1996 Memorandum 'WP 25-84-01, Chippewa Culvert
Replacement, Hwy. 11, District 54, North Bay'

SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

SOIL DESCRIPTION

Terminology describing common soil genesis:

<i>Rootmat</i>	- vegetation, roots and moss with organic matter and topsoil typically forming a mattress at the ground surface
<i>Topsoil</i>	- mixture of soil and humus capable of supporting vegetative growth
<i>Peat</i>	- mixture of visible and invisible fragments of decayed organic matter
<i>Till</i>	- unstratified glacial deposit which may range from clay to boulders
<i>Fill</i>	- material below the surface identified as placed by humans (excluding buried services)

Terminology describing soil structure:

<i>Desiccated</i>	- having visible signs of weathering by oxidization of clay minerals, shrinkage cracks, etc.
<i>Fissured</i>	- having cracks, and hence a blocky structure
<i>Varved</i>	- composed of regular alternating layers of silt and clay
<i>Stratified</i>	- composed of alternating successions of different soil types, e.g. silt and sand
<i>Layer</i>	- > 75 mm in thickness
<i>Seam</i>	- 2 mm to 75 mm in thickness
<i>Parting</i>	- < 2 mm in thickness

Terminology describing soil types:

The classification of soil types are made on the basis of grain size and plasticity in accordance with the Unified Soil Classification System (USCS) (ASTM D 2487 or D 2488) which excludes particles larger than 75 mm. For particles larger than 75 mm, and for defining percent clay fraction in hydrometer results, definitions proposed by Canadian Foundation Engineering Manual, 4th Edition are used. The USCS provides a group symbol (e.g. SM) and group name (e.g. silty sand) for identification.

Terminology describing cobbles, boulders, and non-matrix materials (organic matter or debris):

Terminology describing materials outside the USCS, (e.g. particles larger than 75 mm, visible organic matter, and construction debris) is based upon the proportion of these materials present:

<i>Trace, or occasional</i>	Less than 10%
<i>Some</i>	10-20%
<i>Frequent</i>	> 20%

Terminology describing compactness of cohesionless soils:

The standard terminology to describe cohesionless soils includes compactness (formerly "relative density"), as determined by the Standard Penetration Test (SPT) N-Value - also known as N-Index. The SPT N-Value is described further on page 3. A relationship between compactness condition and N-Value is shown in the following table.

Compactness Condition	SPT N-Value
<i>Very Loose</i>	<4
<i>Loose</i>	4-10
<i>Compact</i>	10-30
<i>Dense</i>	30-50
<i>Very Dense</i>	>50

Terminology describing consistency of cohesive soils:

The standard terminology to describe cohesive soils includes the consistency, which is based on undrained shear strength as measured by *in situ* vane tests, penetrometer tests, or unconfined compression tests. Consistency may be crudely estimated from SPT N-Value based on the correlation shown in the following table (Terzaghi and Peck, 1967). The correlation to SPT N-Value is used with caution as it is only very approximate.

Consistency	Undrained Shear Strength		Approximate SPT N-Value
	kips/sq.ft.	kPa	
<i>Very Soft</i>	<0.25	<12.5	<2
<i>Soft</i>	0.25 - 0.5	12.5 - 25	2-4
<i>Firm</i>	0.5 - 1.0	25 - 50	4-8
<i>Stiff</i>	1.0 - 2.0	50 - 100	8-15
<i>Very Stiff</i>	2.0 - 4.0	100 - 200	15-30
<i>Hard</i>	>4.0	>200	>30

ROCK DESCRIPTION

Except where specified below, terminology for describing rock is as defined by the International Society for Rock Mechanics (ISRM) 2007 publication "The Complete ISRM Suggested Methods for Rock Characterization, Testing and Monitoring: 1974-2006"

Terminology describing rock quality:

RQD	Rock Mass Quality
0-25	<i>Very Poor Quality</i>
25-50	<i>Poor Quality</i>
50-75	<i>Fair Quality</i>
75-90	<i>Good Quality</i>
90-100	<i>Excellent Quality</i>

Alternate (Colloquial) Rock Mass Quality	
<i>Very Severely Fractured</i>	<i>Crushed</i>
<i>Severely Fractured</i>	<i>Shattered or Very Blocky</i>
<i>Fractured</i>	<i>Blocky</i>
<i>Moderately Jointed</i>	<i>Sound</i>
<i>Intact</i>	<i>Very Sound</i>

RQD (Rock Quality Designation) denotes the percentage of intact and sound rock retrieved from a borehole of any orientation. All pieces of intact and sound rock core equal to or greater than 100 mm (4 in.) long are summed and divided by the total length of the core run. RQD is determined in accordance with ASTM D6032.

SCR (Solid Core Recovery) denotes the percentage of solid core (cylindrical) retrieved from a borehole of any orientation. All pieces of solid (cylindrical) core are summed and divided by the total length of the core run (It excludes all portions of core pieces that are not fully cylindrical as well as crushed or rubble zones).

Fracture Index (FI) is defined as the number of naturally occurring fractures within a given length of core. The Fracture Index is reported as a simple count of natural occurring fractures.

Terminology describing rock with respect to discontinuity and bedding spacing:

Spacing (mm)	Discontinuities	Bedding
>6000	<i>Extremely Wide</i>	-
2000-6000	<i>Very Wide</i>	<i>Very Thick</i>
600-2000	<i>Wide</i>	<i>Thick</i>
200-600	<i>Moderate</i>	<i>Medium</i>
60-200	<i>Close</i>	<i>Thin</i>
20-60	<i>Very Close</i>	<i>Very Thin</i>
<20	<i>Extremely Close</i>	<i>Laminated</i>
<6	-	<i>Thinly Laminated</i>

Terminology describing rock strength:

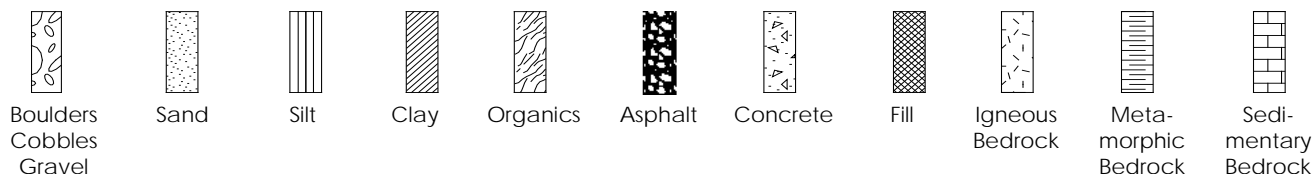
Strength Classification	Grade	Unconfined Compressive Strength (MPa)
<i>Extremely Weak</i>	R0	<1
<i>Very Weak</i>	R1	1 – 5
<i>Weak</i>	R2	5 – 25
<i>Medium Strong</i>	R3	25 – 50
<i>Strong</i>	R4	50 – 100
<i>Very Strong</i>	R5	100 – 250
<i>Extremely Strong</i>	R6	>250

Terminology describing rock weathering:

Term	Symbol	Description
<i>Fresh</i>	W1	No visible signs of rock weathering. Slight discoloration along major discontinuities
<i>Slightly</i>	W2	Discoloration indicates weathering of rock on discontinuity surfaces. All the rock material may be discolored.
<i>Moderately</i>	W3	Less than half the rock is decomposed and/or disintegrated into soil.
<i>Highly</i>	W4	More than half the rock is decomposed and/or disintegrated into soil.
<i>Completely</i>	W5	All the rock material is decomposed and/or disintegrated into soil. The original mass structure is still largely intact.
<i>Residual Soil</i>	W6	All the rock converted to soil. Structure and fabric destroyed.

STRATA PLOT

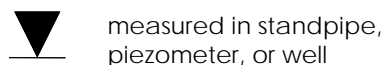
Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.



SAMPLE TYPE

SS	Split spoon sample (obtained by performing the Standard Penetration Test)
ST	Shelby tube or thin wall tube
DP	Direct-Push sample (small diameter tube sampler hydraulically advanced)
PS	Piston sample
BS	Bulk sample
HQ, NQ, BQ, etc.	Rock core samples obtained with the use of standard size diamond coring bits.

WATER LEVEL MEASUREMENT



RECOVERY

For soil samples, the recovery is recorded as the length of the soil sample recovered. For rock core, recovery is defined as the total cumulative length of all core recovered in the core barrel divided by the length drilled and is recorded as a percentage on a per run basis.

N-VALUE

Numbers in this column are the field results of the Standard Penetration Test: the number of blows of a 140 pound (63.5 kg) hammer falling 30 inches (760 mm), required to drive a 2 inch (50.8 mm) O.D. split spoon sampler one foot (300 mm) into the soil. In accordance with ASTM D1586, the N-Value equals the sum of the number of blows (N) required to drive the sampler over the interval of 6 to 18 in. (150 to 450 mm). However, when a 24 in. (610 mm) sampler is used, the number of blows (N) required to drive the sampler over the interval of 12 to 24 in. (300 to 610 mm) may be reported if this value is lower. For split spoon samples where insufficient penetration was achieved and N-Values cannot be presented, the number of blows are reported over sampler penetration in millimetres (e.g. 50/75). Some design methods make use of N-values corrected for various factors such as overburden pressure, energy ratio, borehole diameter, etc. No corrections have been applied to the N-values presented on the log.

DYNAMIC CONE PENETRATION TEST (DCPT)

Dynamic cone penetration tests are performed using a standard 60 degree apex cone connected to 'A' size drill rods with the same standard fall height and weight as the Standard Penetration Test. The DCPT value is the number of blows of the hammer required to drive the cone one foot (300 mm) into the soil. The DCPT is used as a probe to assess soil variability.

OTHER TESTS

S	Sieve analysis
H	Hydrometer analysis
k	Laboratory permeability
γ	Unit weight
G_s	Specific gravity of soil particles
CD	Consolidated drained triaxial
CU	Consolidated undrained triaxial with pore pressure measurements
UU	Unconsolidated undrained triaxial
DS	Direct Shear
C	Consolidation
Q_u	Unconfined compression
I_p	Point Load Index (I_p on Borehole Record equals $I_p(50)$ in which the index is corrected to a reference diameter of 50 mm)

	Single packer permeability test; test interval from depth shown to bottom of borehole
	Double packer permeability test; test interval as indicated
	Falling head permeability test using casing
	Falling head permeability test using well point or piezometer



RECORD OF BOREHOLE No BH15-1

1 OF 1

METRIC

W.P. GWP 5143-06-01 LOCATION Hwy 11 Chippewa Creek Culvert, North Bay, ON N: 5 136 651 E: 307 402 ORIGINATED BY ZP
DIST HWY 11 BOREHOLE TYPE NW Casing, Splitspoon Sampler COMPILED BY ZP
DATUM Geodetic DATE 2015 05 21 - 2015 05 21 CHECKED BY CM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED	✕ FIELD VANE	● QUICK TRIAXIAL	✕ LAB VANE									
311.9	Asphalt						20	40	60	80	100									
310.9	200 mm ASPHALT						20	40	60	80	100									
310.8	FILL: brown gravel with sand		1	SS	87															
0.4	Very dense FILL: brown poorly graded sand to silty sand with gravel																			
	Very dense to compact		2	SS	22												19 56 (25)			
310.4	FILL: brown silty sand to sandy silt																			
1.5	Compact to loose		3	SS	11												7 47 45 1			
			4	SS	6															
			5	SS	6												0 41 (59)			
308.1	FILL: brown poorly graded sand to silty sand																			
3.8	Compact		6	SS	13															
			7	SS	17												3 70 (27)			
			8	SS	22															
306.1	Organic soil (OL) with sand and wood pieces																			
5.8	Very loose																			
305.2	Brown																			
6.7	Silty SAND (SM)																			
	Very loose to compact		9	SS	18															
	Brown		10	SS	7															
			11	SS	9															
			12	SS	3												0 67 33 0			
			13	SS	1															
301.4	End of Borehole																			
10.5																				

\times^3, \times^3 : Numbers refer to Sensitivity \circ 3% STRAIN AT FAILURE

STN13-ONTARIO MTO STANTEC 165000836 HWY 17B NORTH BAY.GPJ ONTARIO MTO.GDT 2/16/17



RECORD OF BOREHOLE No BH15-2

1 OF 1

METRIC

W.P. GWP 5143-06-01 LOCATION Hwy 11 Chippewa Creek Culvert, North Bay, ON N: 5 136 639 E: 307 402 ORIGINATED BY ZP
DIST HWY 11 BOREHOLE TYPE NW Casing, Splitspoon Sampler COMPILED BY ZP
DATUM Geodetic DATE 2015 05 21 - 2015 05 21 CHECKED BY CM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						W _P W W _L WATER CONTENT (%)			GR	SA	SI	CL
								20 40 60 80 100												
								○ UNCONFINED × FIELD VANE ● QUICK TRIAXIAL × LAB VANE												
311.8	Asphalt					▽														
311.8	200 mm ASPHALT																			
310.8	FILL: brown gravel with sand		1	SS	45														3 89 (8)	
0.4	Dense FILL: brown poorly graded sand with silt to silty sand																			
	Dense to compact		2	SS	20				311										4 50 45 1	
310.0	FILL: brown silty sand to sandy silt		3	SS	18				310											
1.8	Compact to loose																			
			4	SS	6				309										0 47 52 1	
			5	SS	5				308											
			6	SS	9				307											
306.5	Organic soil (OL) with sand																			
5.3	Very loose	8	SS	8			306													
305.9	Brown Poorly graded SAND (SP) with organic soil and wood pieces																			
5.9	Loose	9	SS	8			305													
304.9	Brown Poorly graded SAND (SP)																			
6.9	Loose to compact	10	SS	6			304													
	Brown																			
		11	SS	5			303										4 92 (4)			
		12	SS	15			302													
		13	SS	5																
		14	SS	17																
301.3	End of Borehole																			
10.5																				

\times^3, \times^3 : Numbers refer to Sensitivity \circ 3% STRAIN AT FAILURE

STN13-ONTARIO MTO STANTEC 165000836 HWY 17B NORTH BAY.GPJ ONTARIO MTO.GDT 2/16/17



RECORD OF BOREHOLE No BH16-1

1 OF 1

METRIC

W.P. GWP 5143-06-01 LOCATION Hwy 11 Chippewa Creek Culvert, North Bay, ON N: 5 136 636 E: 307 386 ORIGINATED BY JHJ
 DIST HWY 11 BOREHOLE TYPE Portable drilling with HQ casing, Splitspoon Sampler COMPILED BY ZP
 DATUM Geodetic DATE 2016 11 22 - 2016 11 22 CHECKED BY CM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)		
								○ UNCONFINED	✕ FIELD VANE								
307.3	Silty Sand						20	40	60	80	100						
0.0	Silty SAND (SM) with organic soil																
	Loose		1	SS	4												
	Brown		2	SS	4												
	Wet at 1.22 m		3	SS	7												
305.5																	
1.8	Silty SAND (SM)		4	SS	4												
	Loose		5	SS	7												
	Brown to grey		6	SS	7												
	Wet		7	SS	4												
			8	SS	8												
302.1																	
5.2	Silty sand (SM) to sandy silt (ML) trace gravel TILL		9	SS	19												
301.5	Compact																
5.8	Grey																
	Inferred cobbles and boulders																
	End of Borehole																

\times^3, \times^3 : Numbers refer to Sensitivity \circ 3% STRAIN AT FAILURE



RECORD OF BOREHOLE No BH16-2

1 OF 1

METRIC

W.P. GWP 5143-06-01 LOCATION Hwy 11 Chippewa Creek Culvert, North Bay, ON N: 5 136 627 E: 307 407 ORIGINATED BY JHJ
DIST HWY 11 BOREHOLE TYPE 8" Augers, Splitspoon Sampler COMPILED BY ZP
DATUM Geodetic DATE 2016 11 23 - 2016 11 23 CHECKED BY CM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
311.9	Asphalt												
310.6	150 mm ASPHALT												
0.2	FILL: brown gravel with sand		1	GS									
311.3	Dense												
0.6	FILL: brown silty sand to sandy silt, trace gravel												
	Loose to very dense		2	SS	32								
	- Inferred cobble or boulder		3	SS	61								
			4	SS	28								
			5	SS	5								
	- Inferred cobble or boulder		6	SS	65								
307.0	Organic soil (OL) with sand and wood pieces		7	SS	19								
306.4	Loose												
5.5	Dark brown		8	SS	11								
	Wet												
	Well graded SAND with silt (SW-SM) to Silty SAND (SM)		9	SS	6								
	-large wood piece at 6.4 m												
	Very loose to loose												
	Brown to grey		10	SS	9								
	Wet												
			11	SS	3								
			12	SS	5								
			13	SS	2								

\times^3, \times^3 : Numbers refer to Sensitivity \circ 3% STRAIN AT FAILURE

STN13-ONTARIO MTO STANTEC 165000836 HWY 17B NORTH BAY.GPJ ONTARIO.MOT.GDT 2/16/17

RECORD OF BOREHOLE No 1

1 OF 1 METRIC

W.P. 25-84-01 LOCATION Sta. 14+570 e/s 26.7m RT of Centreline Hwy. 11 ORIGINATED BY L.V.
 DIST 54 HWY 11 BOREHOLE TYPE H.S. Auger, Rock Coring COMPILED BY L.V.
 DATUM Geodetic DATE 1995 06 27 CHECKED BY T.K.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID UNIT WEIGHT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20 40 60 80 100	20 40 60 80 100	W _p W W _L	WATER CONTENT (%) 25 50 75		
307.3	Natural Ground Surface												
0.0	Organic Sand Wood Particles		1	SS	2								
			2	SS	2								
305.0													
2.3	Silty Sand Very Loose		3	SS	2								
			4	SS	3								
			5	SS	6								
			6	SS	2								
			7	SS	3								
			8	SS	54								
299.1													
8.2	Bedrock		9	RC	REC 95%								RCO 85%
			10	RC	REC 94%								RCO 94%
295.8													
11.5	End of Borehole												
	*NOTE: BLOW UP CONDITIONS WERE ENCOUNTERED WITHIN THE SILTY SAND DEPOSIT 'N' VALUES MAY BE DISTURBED												

RECORD OF BOREHOLE No 1A

1 OF 1

METRIC

W.P. 25-84-01 LOCATION Sta. 14+567 o/s 26.7m RT of Centreline Hwy.11 ORIGINATED BY L.V.
DIST 54 HWY 11 BOREHOLE TYPE HS Auger, Rock Coring COMPILED BY L.V.
DATUM Geodetic DATE 1996 08 25 CHECKED BY T.K.

[illegible]

RECORD OF BOREHOLE No 2

1 OF 1 METRIC

W.P. 25-84-01 LOCATION Sta. 14+567 o/s 3m RT. of Centreline Hwy. 11 ORIGINATED BY L.V.
 DIST 54 HWY 11 BOREHOLE TYPE HS Auger, Rock Coring COMPILED BY L.V.
 DATUM Geodetic DATE 1998 08 26 CHECKED BY T.K.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100	W _p	W	W _L		
311.8																	
0.0	Mixture of Sand and Silt Trace Gravel Trace Organics [FH]		1	SS	29												
			2	SS	120												
			3	SS	12												
			4	SS	4												
			5	SS	5												
307.0			6	SS	5												
4.9	Organic Sand Wood Particles		7	SS	8												
305.7																	
6.2	Silty Sand Very Loose to Compact		8	SS	4												
			9	SS	13												
			10	SS	2												
301.0																	
10.8	Silty Sand Trace Gravel [TH] Very Dense		11	SS	19												
			12	SS	57												
298.8																	
13.3	Bedrock		13	RC	REC 85%												
296.9																	
14.9	End of Borehole																

RECORD OF BOREHOLE No 2A

1 OF 1 METRIC

W.P. 25-84-01 LOCATION Sta. 14+570 e/s 6m RT. of Centreline on Hwy. 11 ORIGINATED BY L.V.
 DIST 54 HWY 11 BOREHOLE TYPE HS Auger, Rock Coring COMPILED BY L.V.
 DATUM Geodetic DATE 1996 06 28 CHECKED BY T.K.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES								
311.8	0.0	Probable Mixture of Sand and Silt Trace Gravel Trace Organics [Fm]											
308.9	4.9	Probable Organic Sand Wood Particles											
305.6	6.2	Probable Silty Sand											
301.0	10.8	Probable Silty Sand Trace Gravel [m]											
299.3		REFUSAL Probable Bedrock											
12.5		End of Cone Test											

RECORD OF BOREHOLE No 3

1 OF 1

METRIC

W.P. 25-84-01 LOCATION Sta. 14+567 s/s 11.1m LT. of Centreline Hwy. 11 ORIGINATED BY L.V.
 DIST 54 HWY 11 BOREHOLE TYPE HS Auger, Rock Coring COMPILED BY L.V.
 DATUM Geodetic DATE 1996 08 26 CHECKED BY T.K.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100					
311.8	Highway Grade															
0.0	Mixture of Sand and Silt Trace Gravel Trace Organics [Fill]		1	SS	19											
			2	SS	42											
			3	SS	18											
			4	SS	7											
			5	SS	7											
306.9			6	SS	2											
4.7	Organic Sand Wood Particles		7	SS	4											
305.5			8	SS	6											
6.1	Silty Sand Loose		9	SS	5											
302.5			10	SS	6											
9.1	Silty sand Trace Gravel [Tail] Compact to Very Dense		11	SS	120											
300.4	•REFUSAL Probable Bedrock Surface															
11.2	End of Borehole															

RECORD OF BOREHOLE No 3A

1 OF 1 METRIC

W.P. 25-84-01 LOCATION Sta. 14+570 o/e 11.1m LT of Centaline Hwy.11 ORIGINATED BY L.V.
 DIST 54 HWY 11 BOREHOLE TYPE HS Auger, Rock Coring COMPILED BY L.V.
 DATUM Geodetic DATE 1996 08 27 CHECKED BY T.K.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40						60	80	100	20	40
311.6	Highway Grade																		
0.0	Probable Mixture of Sand and Silt Trace Gravel Trace Organics [FIH]																		
308.9																			
4.7	Probable Organic Sand Wood Particles																		
305.5																			
6.1	Probable Silty Sand																		
302.5																			
9.1	Probable Silty Sand Trace Gravel [TIIH]																		
300.8	• REFUSAL Probable Bedrock																		
11.0	End of Cone Test																		

ROCK CORE DESCRIPTION **WP 25-84-01**

Page 1 of 1

Page 1 of 1

CORE RECOVERY					CORE DESCRIPTION	
BH#	RC#	DEPTH (m)	% CR*	% RQD*	DEPTH (m)	DESCRIPTION
1	9	8.18-9.85	95	85	8.18-11.48	BIOTITE GNEISS, greyish orange pink to greyish red to dark grey; medium to coarse grained; strong; unweathered to slightly weathered; fractures wide to close spaced, flat to near vertical, undulating to planar, smooth to rough.
	10	9.85-11.48	94	94		
2	13	13.28-14.91	89	25	13.28-14.91	BIOTITE GNEISS, greyish orange pink to greyish red to dark grey; medium to coarse grained; strong; unweathered to slightly weathered; fractures moderate to very close spaced, flat, planar to undulating, smooth to rough.

*CR = CORE RECOVERY

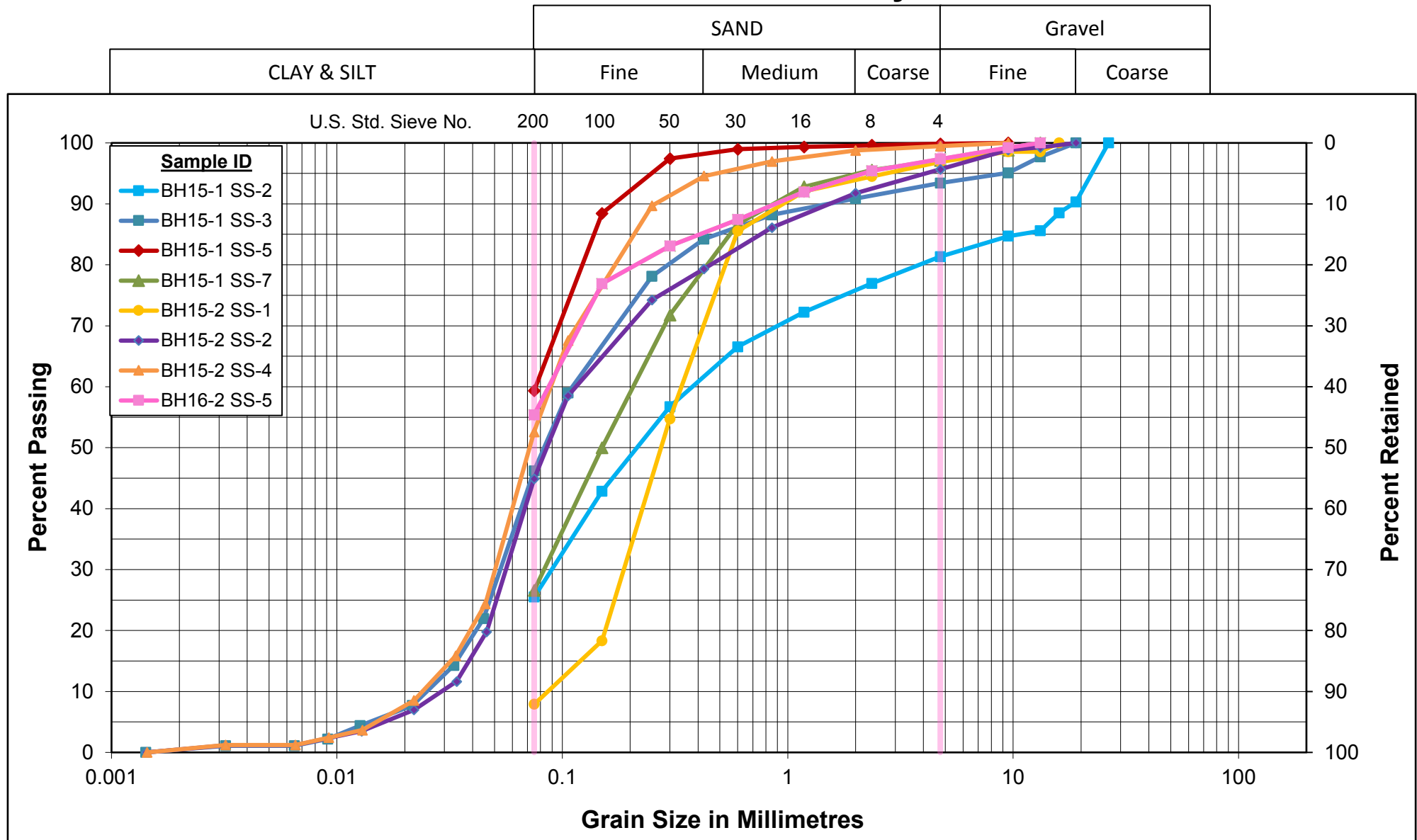
*RQD = ROCK QUALITY DESIGNATION

Note: Depths are approximated where core recovery is less than 100%
 Logged by: DAW, Soils and Aggregates Section

APPENDIX C

Laboratory Test Results

Unified Soil Classification System



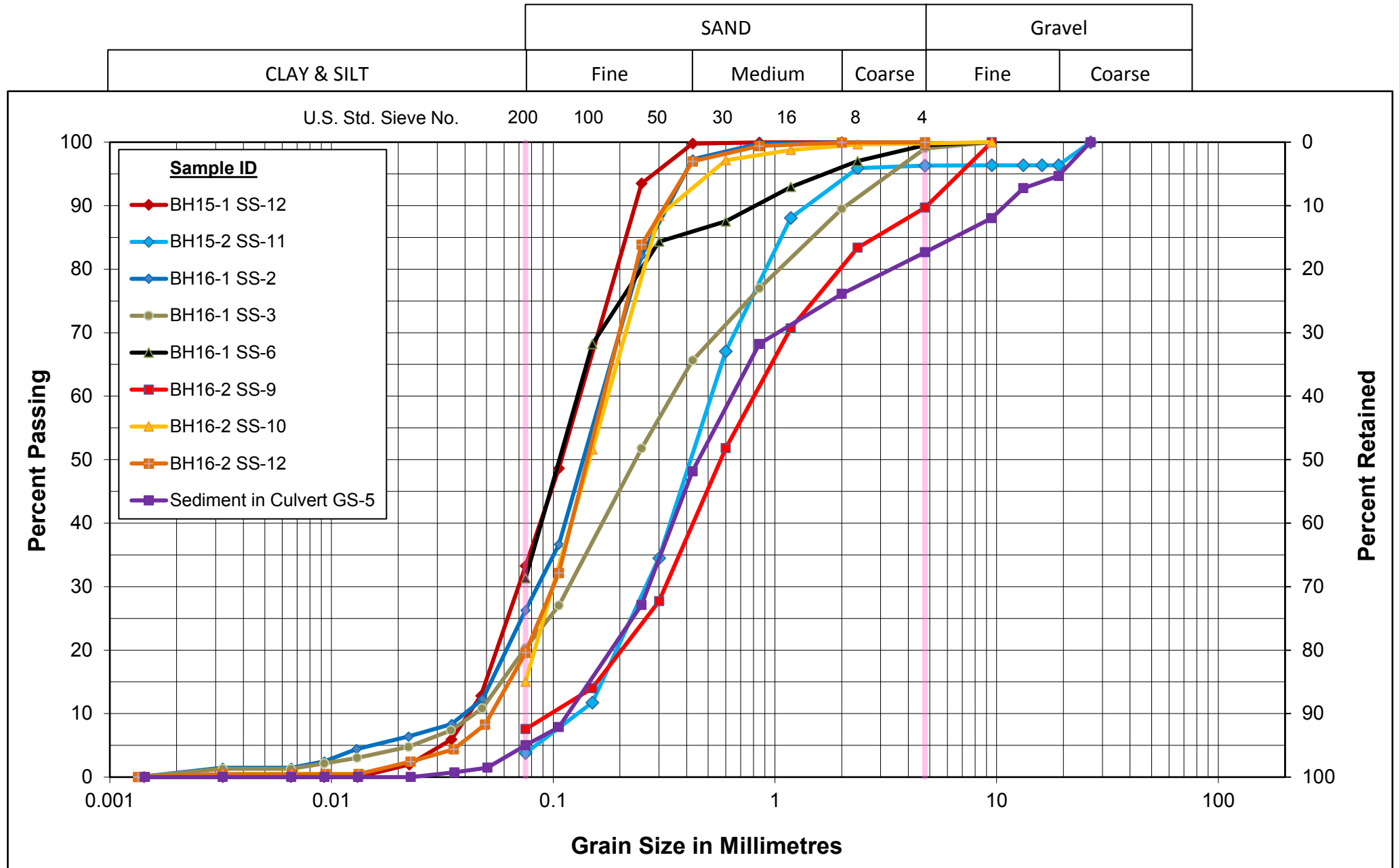
GRAIN SIZE DISTRIBUTION

FILL: poorly graded sand with silt, silty sand
with/without gravel, sandy silt

Figure No. 1

Project No. 165000836
GWP 5143-06-01

Unified Soil Classification System



GRAIN SIZE DISTRIBUTION

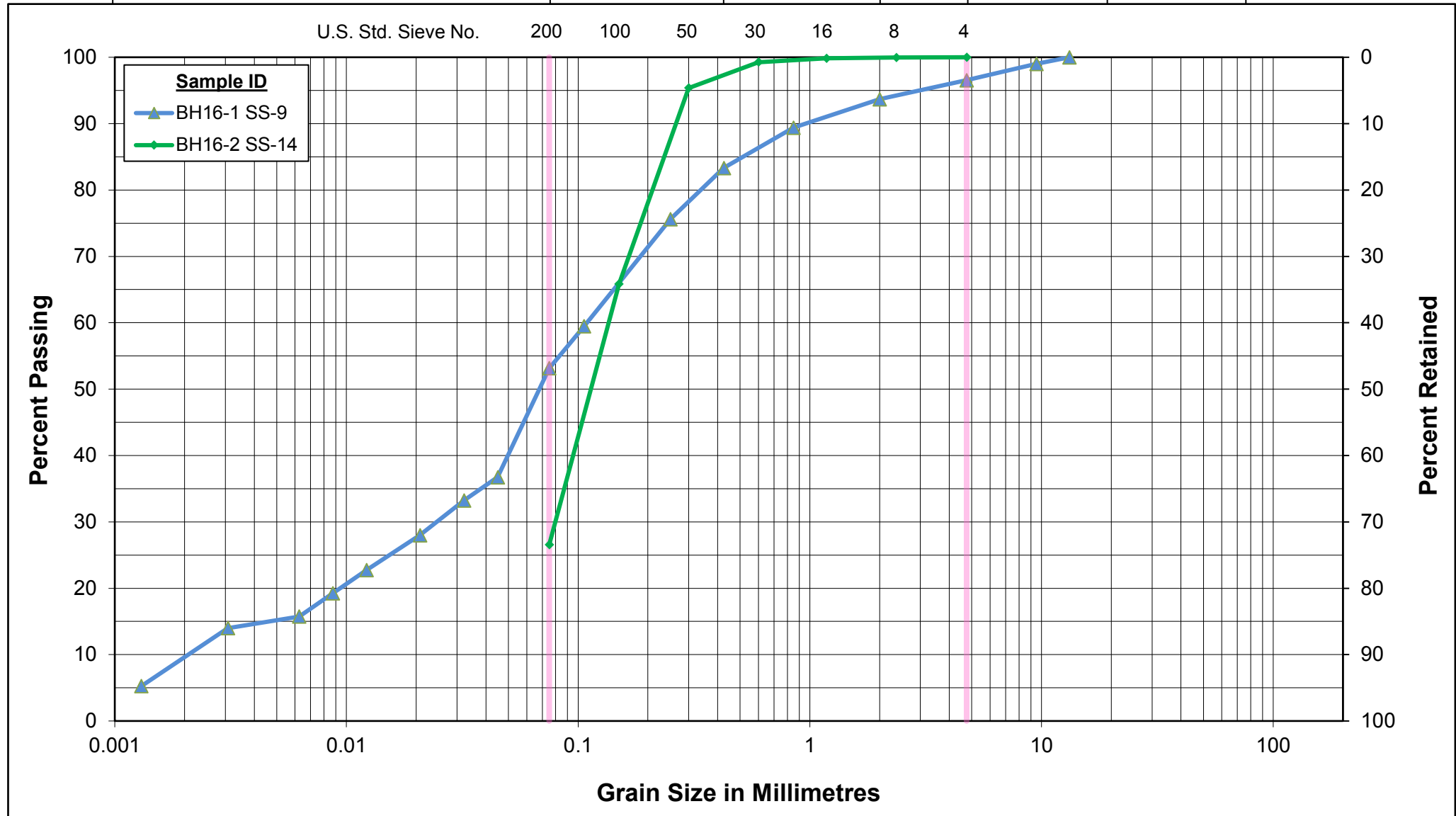
SAND (SP and SW-SM) with/without Gravel
and Silty SAND (SM)

Figure No. 2

Project No. 165000836
GWP 5143-06-01

Unified Soil Classification System

CLAY & SILT	SAND			Gravel	
	Fine	Medium	Coarse	Fine	Coarse



GRAIN SIZE DISTRIBUTION

Sandy Silt (ML) to Silty Sand (SM) TILL

Figure No. 3

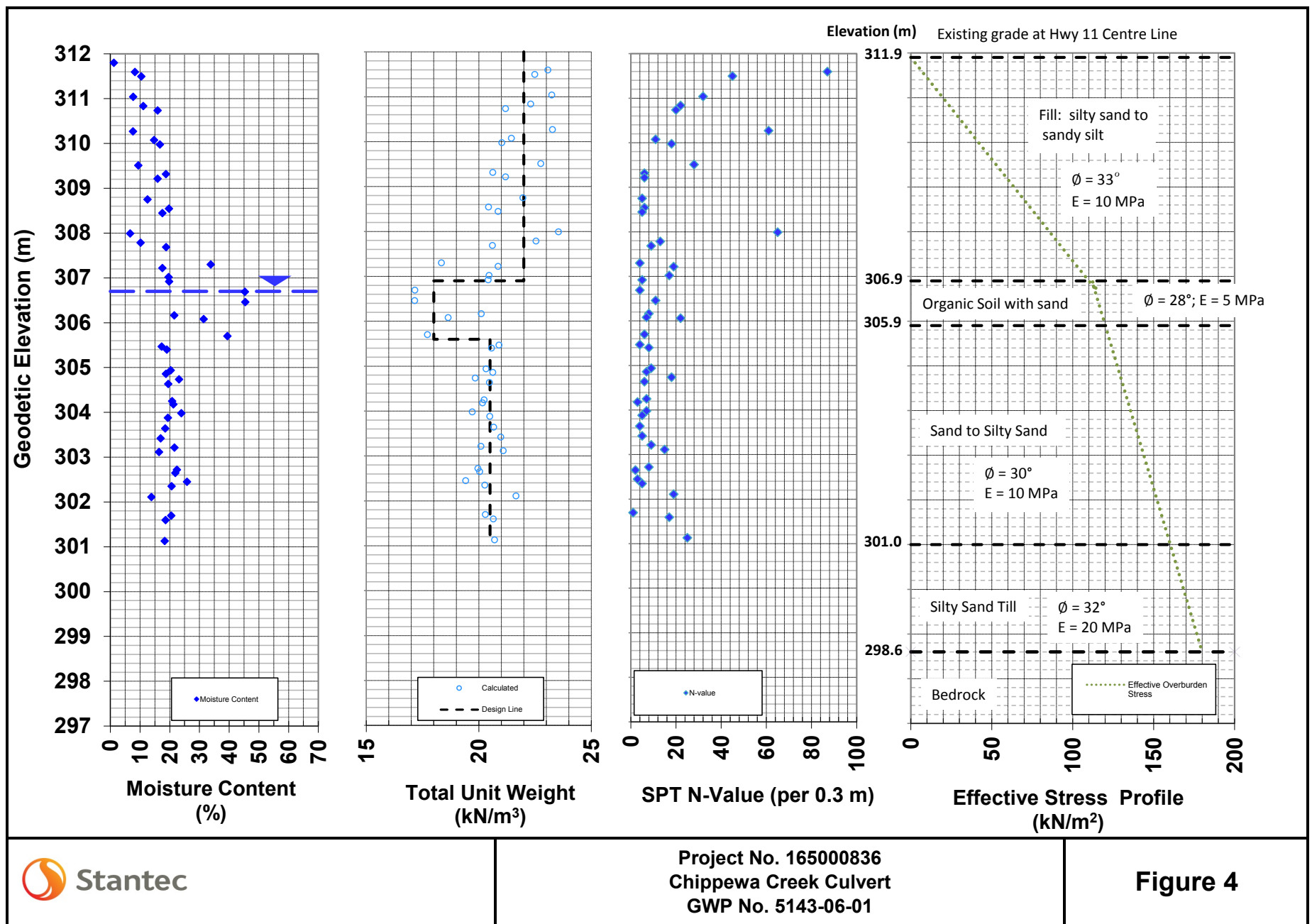
Project No. 165000836

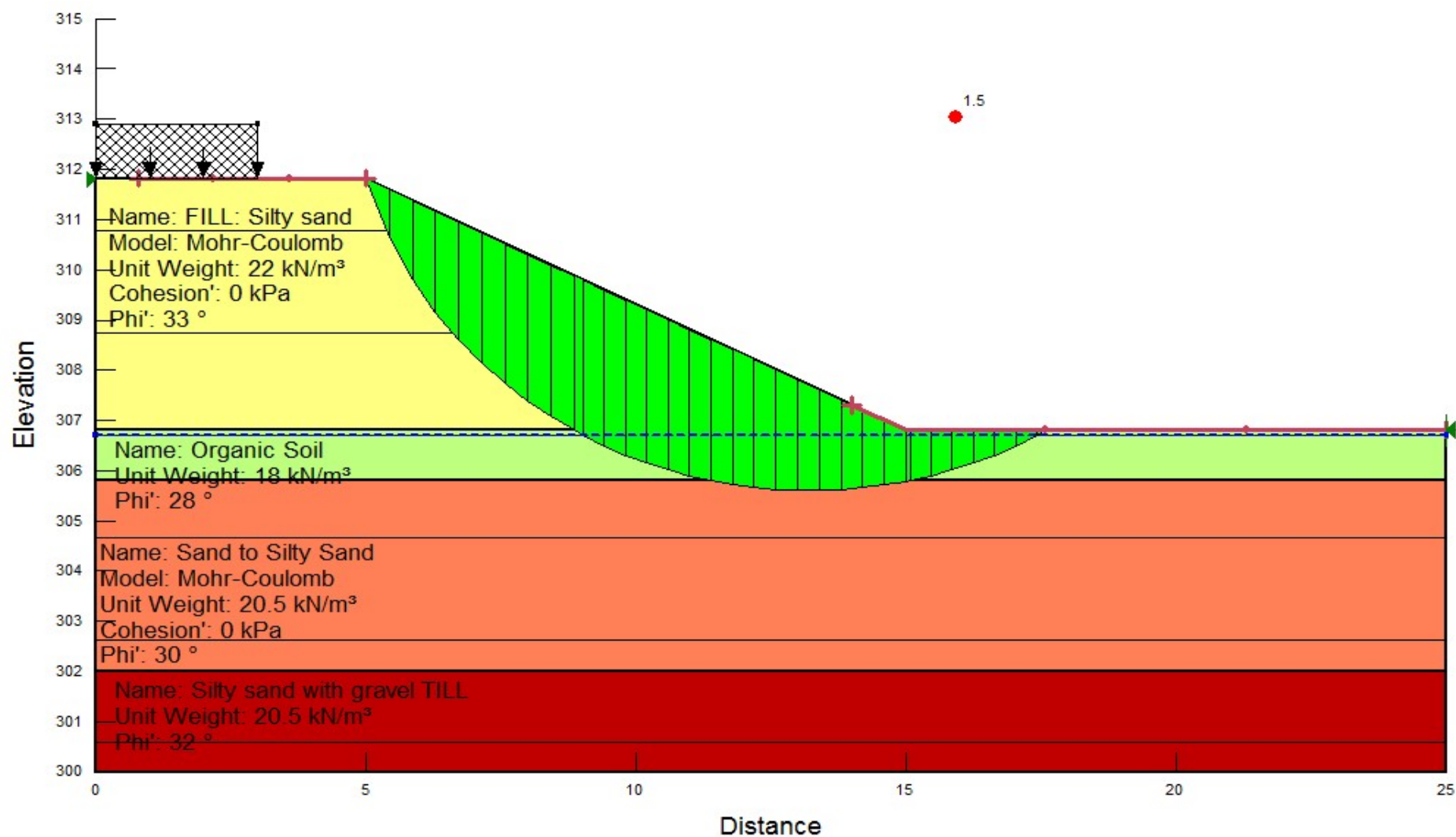
GWP 5143-06-01

APPENDIX D

Design Parameters

Slope Stability Analysis Results





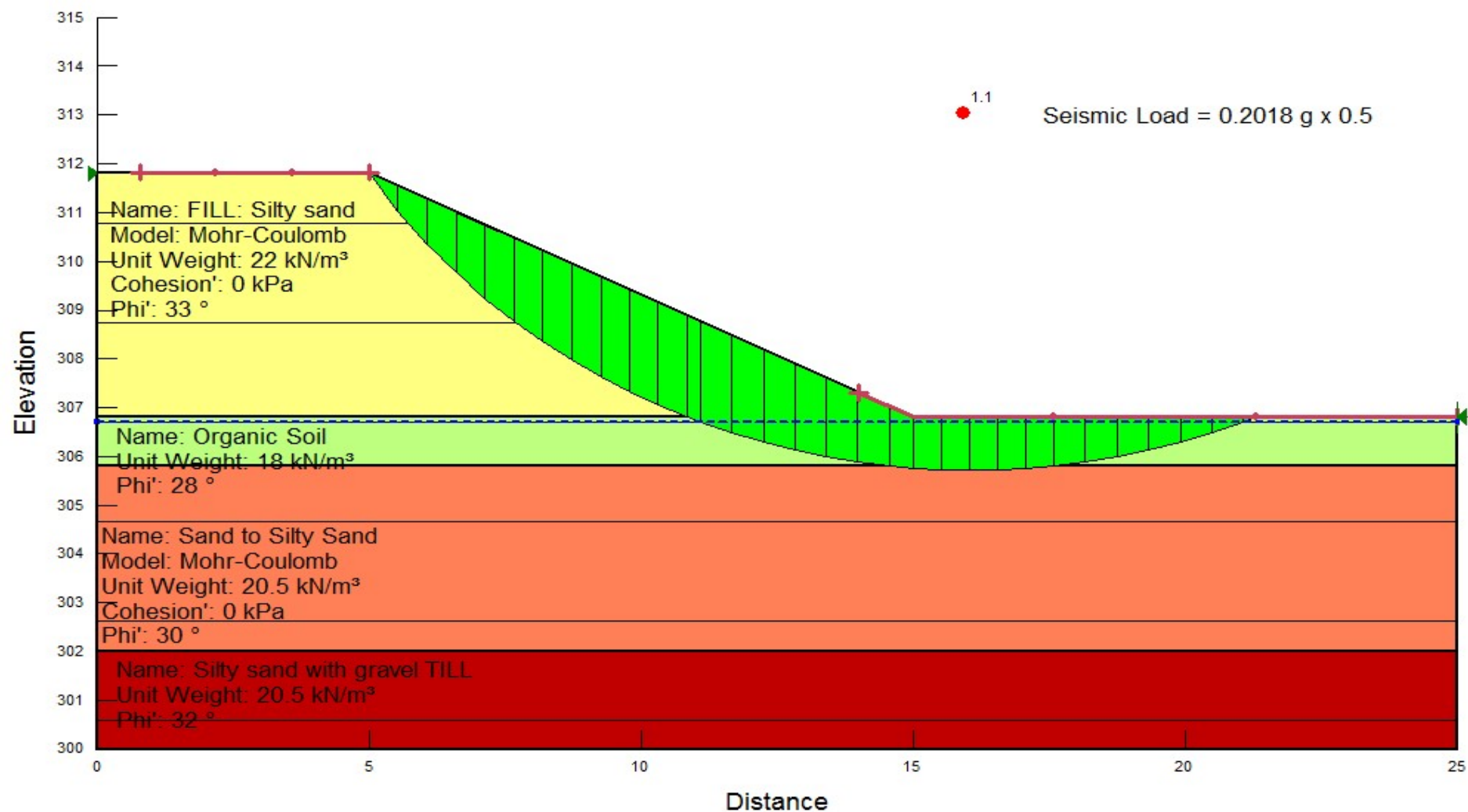
Static Slope Stability Analysis

Chippewa Creek Culvert Replacement

Figure 5a

Project No. 165000836

GWP No. 5143-06-01



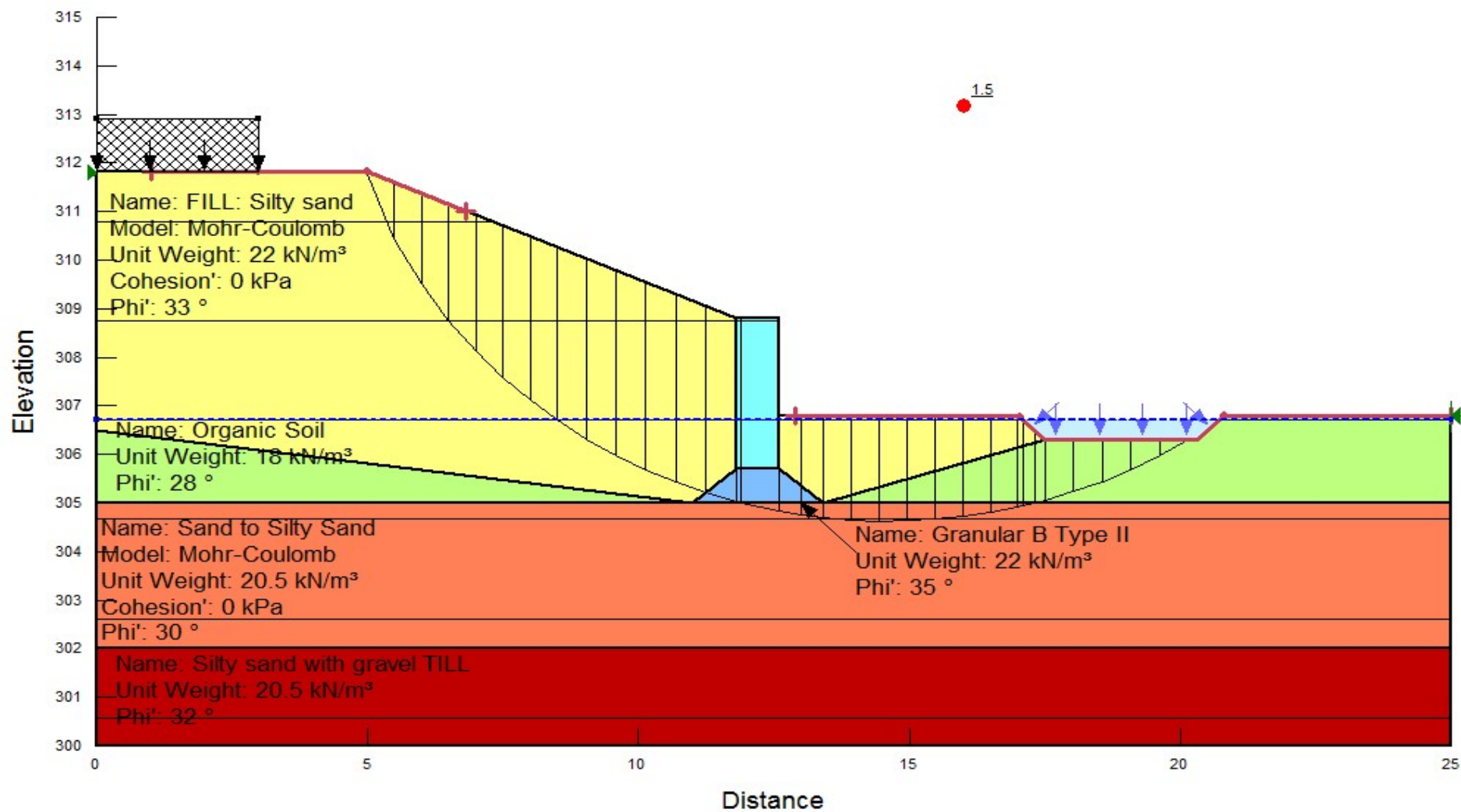
Seismic Slope Stability Analysis

Chippewa Creek Culvert Replacement

Figure 5b

Project No. 165000836

GWP No. 5143-06-01



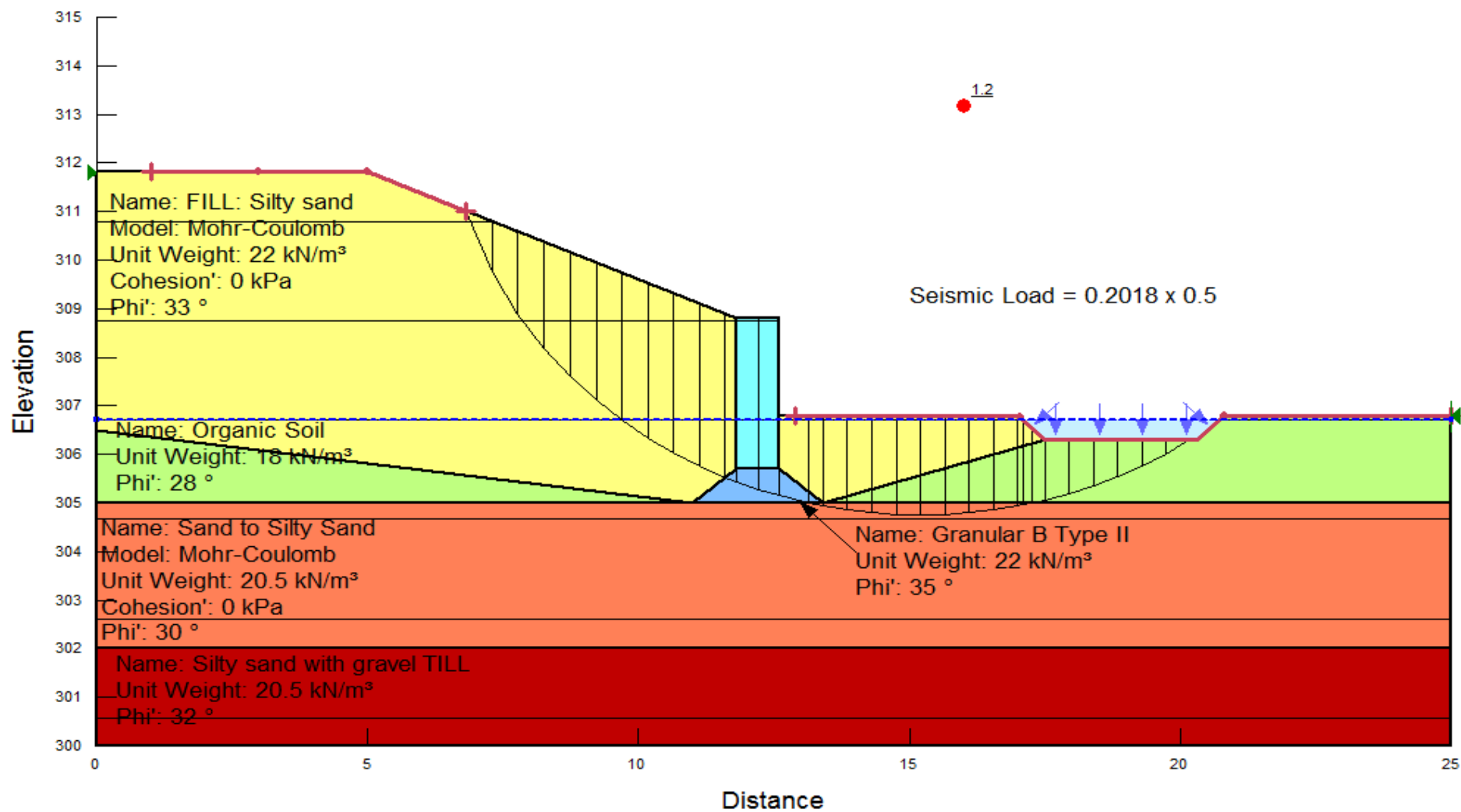
Static Slope Stability Analysis with Retaining Wall

Chippewa Creek Culvert Replacement

Figure 5c

Project No. 165000836

GWP No. 5143-06-01



Seismic Slope Stability Analysis with Retaining Wall

Chippewa Creek Culvert Replacement

Figure 5d

Project No. 165000836

GWP No. 5143-06-01

APPENDIX E

NBC Seismic Hazard Calculation Sheet

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

December 01, 2015

Site: 46.2209 N, 79.2758 W User File Reference: Chippewa Creek Culvert, North Bay

Requested by: , Stantec Consulting Ltd.

National Building Code ground motions: 2% probability of exceedance in 50 years (0.000404 per annum)

Sa(0.05)	Sa(0.1)	Sa(0.2)	Sa(0.3)	Sa(0.5)	Sa(1.0)	Sa(2.0)	Sa(5.0)	Sa(10.0)	PGA (g)	PGV (m/s)
0.237	0.294	0.261	0.206	0.151	0.079	0.039	0.010	0.0038	0.164	0.122

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" (NBCC 2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are specified in **bold** font. 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.*

Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.05)	0.022	0.087	0.142
Sa(0.1)	0.032	0.114	0.181
Sa(0.2)	0.031	0.101	0.160
Sa(0.3)	0.026	0.079	0.126
Sa(0.5)	0.019	0.057	0.091
Sa(1.0)	0.0093	0.030	0.047
Sa(2.0)	0.0038	0.014	0.023
Sa(5.0)	0.0008	0.0032	0.0054
Sa(10.0)	0.0004	0.0013	0.0022
PGA	0.017	0.062	0.099
PGV	0.012	0.042	0.070

References

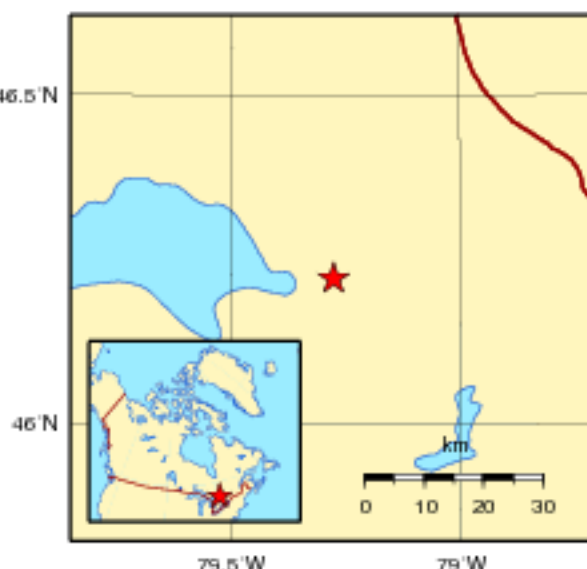
National Building Code of Canada 2015 NRCC no. 58190;
Appendix C: Table C-3, Seismic Design Data for Selected Locations in 46.5°N
Canada

User's Guide - NBC 2015, Structural Commentaries NRCC no.
xxxxxx (in preparation)
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.nationalbuildingcode.ca for more information

Aussi disponible en français



Natural Resources
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APPENDIX F

Notice to Contractor – Cobbles and Boulders

Notice to Contractor – Groundwater Control

NOTICE TO CONTRACTOR – Cobbles and Boulders

Special Provision

Cobbles and Boulders within Soil

Cobbles and boulders were inferred during drilling of the boreholes at several of the culvert replacement locations. Cobbles and boulders were inferred during drilling and the observations are documented in the Foundation Investigations Report. It is recommended that the bidder review the Foundation Investigation Report and borehole records provided in the Reports with respect to the presence of cobbles and boulders.

Cobbles and boulders may obstruct excavation and the installation of temporary roadway protection systems. Impacts associated with these obstructions could include, but are not limited to, the following:

- Additional effort will be required to excavated material containing cobbles and boulders.
- The installation of sheet piles could be obstructed or damaged. Multiple attempts at driving sheet piles should be anticipated.
- The installation of drilled soldier piles could be obstructed. Multiple attempts at installing drilled soldier piles should be anticipated.
- Driven soldier piles could be obstructed or damaged. Pile tips should be reinforced and multiple attempts at driving piles should be anticipated.
- The contractor will require appropriate equipment and construction methods to penetrate or remove cobbles and boulders.

NOTICE TO CONTRACTOR – GROUNDWATER CONTROL

Special Provision

PRESENCE OF HIGH PERMEABILITY SOILS

The work required for the above tender item shall include consideration of dewatering to provide a stable working platform during construction and removal of the existing culvert.

The contractor is advised of the following:

- Excavation is required to remove the existing culvert and for the construction of a new culvert.
- The groundwater level was measured at elevation 306.7 m in Borehole BH16-2 on November 23, 2016.
- The water level at the existing Chippewa Creek Culvert was measured at elevation 306.6 m on November 23, 2016.
- The contractor shall consider that seasonal groundwater fluctuations may result in higher groundwater levels than observed and that higher groundwater levels may result in an unstable working earth platform.
- The estimated hydraulic conductivity for the native soil at the site is expected to range from 1×10^{-6} m/s to 1×10^{-4} m/s.
- The anticipated excavation level is below the groundwater level noted above.
- The presence of cohesionless sand can render the soils susceptible to unbalanced hydrostatic head, soil sloughing and cave-in.
- The contractor shall consider the site conditions, sequence of work and schedule when assessing requirements for dewatering.

Requirements for dewatering are contained in OPSS 517.

Requirements for the control of water during construction are contained in OPSS 518.

If the designer finds that it is necessary to use well points for dewatering, a specialized dewatering designer will be required to design the dewatering system.