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FOUNDATION INVESTIGATION AND DESIGN REPORT - REVISION 1

Proposed Culvert Replacement / Rehabilitation
Site 22-415/C on Highway 7A, Port Perry, Ontario and
Site 21-13/C on Highway 7A near Blackstock, Ontario
GWP 2248-14-00

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REPORT





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NSSP - FOUN0003 Dewatering Structure Excavations
NSSP - Working Slab
NSSP - RSS Wall near Floodplain/Watercourse
SP – 517F01 Amendment to OPSS 517
NSSP - Obstructions



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CULVERT REPLACEMENT / REHABILITATION AT SITE NOS. 22-
415/C AND 21-13/C**

PART A

**FOUNDATION INVESTIGATION REPORT
CULVERT REPLACEMENT / REHABILITATION AT SITE NOS. 22-415C AND
21-13C
PORT PERRY AND BLACKSTOCK, ONTARIO
GWP 2248-14-00**



1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by CIMA+ Canada Inc. (CIMA+) on behalf of the Ministry of Transportation, Ontario (MTO) to provide detailed foundation engineering services for the following:

- Site 22-415C: Replacement of existing culvert and endwall / retaining wall (at culvert outlet); and
- Site 21-13C: Rehabilitation of wingwalls / headwall at existing culvert inlet.

The purpose of this investigation is to establish the subsurface soil and shallow groundwater conditions at the culvert replacement / rehabilitation locations by borehole drilling and laboratory testing on selected soil samples.

The Terms of Reference (TOR) and the scope of work for the foundation investigation are outlined in MTO's Request for Proposal, dated June 3, 2015 and Golder's scope change letter dated April 10, 2017.

2.0 SITE DESCRIPTION

2.1 Site 22-415C

Site 22-415C is located on Highway 7A, about 80 m west of the intersection with Queen Street / Scugog Line 6 in Port Perry, Ontario. The culvert crosses under Highway 7A and the watercourse flows through the culvert from south to north at approximately Station 12+710. The upstream and downstream area consists of a well vegetated confined valley with dense trees. A commercial retail area is located to the northwest of the culvert, while residential land is present at the northeast, southwest and southeast quadrants. The Highway 7A grade at the culvert site is at approximately Elevation 271.5 m, and the road grade increases in elevation in both the east and west directions away from the low point. The toe of the embankment is at about Elevation 266.5 m, resulting in an embankment height up to about 5 m.

The culvert consists of an original non-rigid frame open footing cast-in-place concrete culvert with three extensions added over the years. Each extension consists of a rigid frame open footing cast-in-place concrete culvert. The original culvert has a span of 2.6 m, a rise of 2.1 m and a length of 12.0 m; the extensions have a span of 3.0 m, a rise of 2.1 m, and lengths of 6.7 m, 3.7 m and 5.7 m. The founding elevation of the existing open footing culvert and associated culvert extensions are not known. There is an existing concrete endwall / retaining wall located parallel to Highway 7A at the culvert outlet. The existing retaining wall is about 12 m long and up to about 3.5 m high; existing details for the retaining wall foundation were not available at the time this report was prepared.

Golder visited the site in the Spring / Summer of 2017. The existing embankment slopes were observed to be stable and there were no visual signs of instability or excessive settlement near the existing culvert inlet (see Photograph 1). The existing endwall / retaining wall is shown in Photograph 2 and water seepage was observed to be emanating through cracks in the headwall near the top of the culvert opening.



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Photograph 1 – Site 22-415C (looking north from inlet side)



Photograph 2 – Site 22-415C (looking southeast at endwall from outlet side)

2.2 Site 21-13C

Site 21-13C is located on Highway 7A, about 200 m east of Durham Regional Road 57 along a bend in the highway near Blackstock, Ontario. The culvert crosses under the Highway 7A embankment and Blackstock Creek flows through the culvert from southeast (inlet) to northwest (outlet). Two residences are located approximately 150 m to the southwest of Culvert 21-13C, one along Highway 7A and one along Regional Road 57. The Highway 7A grade at the site is at approximately Elevation 263 m, increasing in elevation away from the low point to the northeast and southwest. The embankment is about 3 m high near the culvert location.



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The existing culvert consists of a rigid frame cast-in-place concrete box culvert with a span of 6.1 m, a rise of 2.2 m and a length of 22.0 m. The base of the culvert is at about Elevation 259.2 m and the inlet consists of flared concrete headwall founded at the same level.

Golder visited the site on several occasions (Summer and Fall 2017) and observed the existing concrete headwall was distressed/delaminating (see Photograph 3). Referring to Photographs 4 and 5, the existing pavement structure above the culvert was observed to be eroding due to the close proximity of the guardrail and edge of pavement to the limit of the headwall, and there was also active erosion of the granular pavement base / subbase on both sides of the culvert inlet (adjacent and above the flared headwalls). Water seepage was observed to be flowing over the flared headwall at about the mid-height of the embankment.



Photograph 3 – Site 21-13 (looking northwest at headwall)



Photograph 4 – Site 21-13C (looking West)



Photograph 5 – Site 21-13C (looking East)



3.0 INVESTIGATION PROCEDURES

The field work for the foundation investigations for the culvert rehabilitation / replacement was carried out between July 5 and August 17, 2017, during which time six foundation boreholes were advanced at Culvert 22-415C (designated as Boreholes 18, 20, 21, 33, 34 and 35) and three foundation boreholes were advanced at Culvert 21-13C (designated as Boreholes 17-18, 17-20 and 17-36), at the locations shown on Drawings 1 and 2, respectively. Three pavement boreholes (designated 13, 15, and 32) advanced by Golder as part of the pavement investigation for this project are referenced and shown on Drawing 1 to supplement the foundation investigation program. The Record of Boreholes are provided in Appendices A and B, along with copies of the supplementary pavement borehole information.

The foundation borehole investigation was carried out using a D-50 truck mounted drill rig and a CME-55 track mounted drill rig, supplied and operated by Tri-phase Group of Mississauga, Ontario. The boreholes were advanced through the overburden using 210 mm outside diameter (O.D.) hollow stem power augers to depths of between about 9.2 m and 15.5 m below ground surface at Culvert 22-415C and to depths between about 2.4 m and 9.6 m below ground surface at Culvert 21-13C. Portable equipment (continuous split-spoon sampling) was required for one borehole (17-36) advanced at Site 21-13C due to wet, soft ground conditions.

Soil samples were typically obtained at 0.75 m and 1.5 m intervals of depth using a 50 mm outer diameter split-spoon sampler driven by an automatic hammer in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586)¹. For borehole 17-36, continuous sampling using the 50 mm split-spoon sampler was performed by driving with a half-weight (32 kg) manual hammer. As a result, the recorded blow count per 0.3 m of penetration was divided in half to represent the equivalent SPT "N"-value (using standard hammer weight) as presented on the Record of Borehole.

The groundwater conditions and water levels in the open boreholes were observed in the boreholes during drilling operations. One standpipe piezometer was installed in Borehole 35 to permit monitoring of the water level pertinent to the culvert replacement at Site 22-415C. The installed piezometer consisted of 50 mm diameter PVC pipe, with a 1.5 m machine slotted screen sealed within a filter sand pack at a selected depth within the borehole. The annulus surrounding the piezometer pipe above the filter sand pack was backfilled to ground surface with bentonite pellets. Piezometer installation details and water level readings are described on the Record of Borehole sheet included in Appendix A. All other boreholes were backfilled to ground surface with bentonite upon completion, in accordance with Ontario Regulation 903, Wells (as amended) and a 0.1 m to 0.2 m thick asphalt cap was placed at roadway surface in the boreholes drilled on roadways/shoulders.

The field work was monitored on a full-time basis by a member of Golder's technical staff who located the boreholes in the field, directed the sampling and in situ testing operations, logged the boreholes and examined the soil samples. The soil samples were identified in the field, placed in labelled containers and transported to Golder's laboratory in Whitby for further visual review and geotechnical laboratory testing on selected samples. The geotechnical laboratory index and classification testing; consisting of natural moisture content, Atterberg limits and grain size distribution was conducted in accordance with MTO and / or ASTM Standards as applicable. One soil sample obtained during the field investigation was submitted for testing of organic content.

¹ ASTM D1586 – Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of Soils.



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The borehole locations were marked in the field by Golder personnel relative to existing road features and pre-selected coordinates using a hand-held GPS. The locations given on the Record of Boreholes and shown on the Drawings are positioned relative to MTM NAD 83 (Zone 10) northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum based on interpolation from the digital terrain model provided by CIMA+. The borehole locations, ground surface elevations and drilled depths are summarized below.

Borehole No.	Location (MTM NAD 83 Zone 10)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (Latitude)	Easting (Longitude)		
18	4,884,173.8 (44.096291)	348,010.4 (-78.960334)	271.3	9.2
20	4,884,180.1 (44.096347)	348,033.0 (-78.960052)	271.8	9.6
21	4,884,178.5 (44.096332)	348,035.0 (-78.960027)	271.7	9.6
33	4,884,160.3 (44.096170)	347,998.7 (-78.960482)	271.9	15.5
34	4,884,153.5 (44.096109)	348,005.6 (-78.960396)	271.9	15.5
35	4,884,143.3 (44.096016)	348,016.8 (-78.960257)	267.2	9.2
17-18	4,888,201.9 (44.131829)	358,670.1 (-78.826786)	263.2	9.6
17-20	4,888,211.3 (44.131921)	358,681.6 (-78.826654)	263.1	9.6
17-36	4,888,204.2 (44.131854)	358,686.5 (-78.826536)	261.0	2.4

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

4.1.1 Site 22-415C

This section of Highway 7A is located near the interface of several physiographic regions including the Schomberg Clay Plains, Oak Ridges Moraine, and Peterborough Drumlin Field, as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984)².

Three large areas, located near Schomberg, Newmarket and Lake Scugog, are included in the Schomberg Clay Plains. They are composed of a number of topographic basins along the northern slopes of the Oak Ridges Moraine that contain deep deposits of stratified clay and silt. This investigation is located in the third area, near Lake Scugog, where the surface under the clay is that of a flat till plain, with few drumlins occurring.

² Chapman, L.J. and Putnam, D.F. 1984. *The Physiography of Southern Ontario*, Ontario Geological Survey, Special Volume 2, Third Edition. Accompanied by Map P. 2715, Scale 1:600,000.



4.1.2 Site 21-13C

This section of Highway 7A is located in the Peterborough Drumlin Field physiographic region, as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984)².

The Peterborough Drumlin Field is a rolling till plain. The drumlins throughout are composed of highly calcareous till with local differences. In some areas deposits of clay lie between the drumlins, in places flooded by the old glacial lakes. In other places such as to the south of Lake Simcoe there are drumlins rising from sand plains. The Peterborough Drumlin Field is also notable for its eskers / gravel ridges.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced during the current investigation and the results of the geotechnical laboratory tests carried out on selected soil samples are presented on the Record of Borehole sheets provided in Appendices A and B for Culverts 22-415C and 21-13C, respectively. The results of the in situ field tests (i.e., SPT "N"-values) as presented on the Record of Borehole sheets and in Section 4.2 are uncorrected. The results of laboratory grain size distribution tests and Atterberg Limits tests are also presented in Appendices A and B, for Culverts 22-415C and 21-13/C, respectively.

The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Furthermore, subsurface conditions will vary between and beyond the borehole locations, however, the factual data presented in the Record of Borehole sheets governs any interpretation of the site conditions.

Detailed descriptions of the subsurface conditions encountered at each culvert site are provided in the following sections of this report.

4.2.1 Site 22-415C

The plan and profile drawings at the existing / proposed culvert replacement and retaining wall location showing the borehole locations and interpreted stratigraphy are shown on Drawings 1 and 2. The Record of Borehole sheets (Boreholes 18, 20, 21, and 33 to 35) and the laboratory test results are presented in Appendix A.

In general, the soil conditions at the area of the proposed culvert and retaining wall replacement consist of surficial asphalt, topsoil or sand and gravel fill, underlain by a silty sand to silt and sand (fill). The fill was underlain by a sandy silt to silt and sand deposit in Boreholes 18, 33 and 34 and by a silt and sand till deposit in Boreholes 20, 21 and 35. In the boreholes where the sandy silt to silt and sand deposit was encountered, it was immediately underlain by the silt and sand till. Interlayers of sand were observed in the silt and sand till deposit in Boreholes 20 and 35.

4.2.1.1 Asphalt

An approximately 200 mm thick layer of asphalt was encountered immediately below ground surface in Borehole 20.

4.2.1.2 Topsoil

An approximately 300 mm thick layer of topsoil was encountered immediately below ground surface in Borehole 35, which was advanced near the inlet on the south side of Highway 7A. The pavement borehole 32



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located near the outlet on the north side of Highway 7A also encountered 300 mm of topsoil and the pavement borehole 15 located near the toe of the embankment west of the culvert encountered about 200 mm of topsoil.

4.2.1.3 *Fill*

A 0.4 m to 4.0 m thick layer of fill was encountered underlying the asphalt pavement in Borehole 20, underlying the topsoil in Borehole 35, and at ground surface in the remaining four boreholes (18, 21, 33, and 34). The surface of the fill layer was encountered between Elevations 271.9 m and 266.9 m, and the base of the fill layer was encountered between Elevations 269.7 m and 266.5 m.

The fill is variable in composition but is mainly non-cohesive and is comprised of silt and sand to sandy silt. The boreholes advanced on Highway 7A typically encountered a gravelly sand to sand and gravel layer near the road / shoulder surface that transitioned into silt and sand at depth. A 0.4 m thick layer of cohesive fill was encountered in Borehole 20 and is comprised of silty clay with gravel containing trace sand. Trace organics were noted within the fill in Boreholes 18, 20, and 35.

The Standard Penetration Test (SPT) "N"-values measured within the non-cohesive fill deposit range from 3 blows to 32 blows per 0.3 m of penetration, indicating a very loose to dense level of compactness. SPT "N"-values were not measured within the cohesive fill.

Atterberg limits testing was carried out on one sample of the non-cohesive fill and indicated that the fill was non-plastic. The natural water content measured on selected samples of the non-cohesive fill ranges from about 5 per cent to 25 per cent.

The supplementary pavement borehole 13, located on the Highway 7A shoulder west of the culvert, indicates that the embankment fill consists of gravelly sand near the surface and transitions to a silty clay with sand at a depth of about 1.1 m and extends to the termination of the pavement hole at about Elevation 269.9 m. An SPT "N"-value measured 5 blows per 0.3 m of penetration at the pavement borehole 13, suggesting the gravelly sand to silty clay with sand fill has a loose to firm consistency.

4.2.1.4 *Sandy Silt to Silt and Sand*

A 1.1 m to 1.6 m thick deposit of sandy silt to silt and sand was encountered underlying the fill in Boreholes 18, 33, and 34. The surface of the sandy silt to silt and sand was encountered between Elevations 269.0 m and 267.9 m. The deposit consisted of varying amounts of sand and silt containing trace to some gravel and clay.

The SPT "N"-values measured within sandy silt to silt and sand deposit range from 2 blows to 21 blows per 0.3 m of penetration, indicating a very loose to compact level of compactness.

Grain size distribution testing was carried out on two selected samples of the sandy silt to silt and sand and the results are shown on Figure A-1 in Appendix A. Atterberg limits testing was carried out on two selected samples of the sandy silt to silt and sand deposit, both tests indicated a non-plastic material. The natural water content measured on one selected sample of the sandy silt to silt and sand indicated a water content of about 18 per cent.

4.2.1.5 *Silt and Sand to Sandy Silt (Till)*

A silt and sand to sandy silt till deposit was encountered underlying the fill or sandy silt to silt and sand deposit in all boreholes (18, 20, 21, 33, 34, and 35). The boreholes were terminated within the silt and sand to sandy silt till deposit, penetrating the deposit between about 5.2 m and 11.5 m. The surface of the silt and sand to sandy silt



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till deposit was encountered between Elevations 269.7 m and 266.3 m. The till deposit consisted of silt and sand to silty sand containing trace to some gravel and clay. Although not encountered during the current investigation, cobbles and boulders are known to be present within glacial till soils in Southern Ontario and should be expected within this soil deposit.

The SPT “N”-values measured within the silt and sand to sandy silt till deposit ranged from 11 blows per 0.3 m of penetration to 50 blows per 0.08 m of penetration, indicating a compact to very dense level of compactness.

Grain size distribution testing was carried out on thirteen selected samples of the silt and sand to sandy silt till and the results are shown on Figures A-2A and A-2B in Appendix A. Atterberg limits testing was carried out on four selected samples of the silt and sand to sandy silt till deposit, all tests indicated a non-plastic material. The natural water content measured on selected samples of the silt and sand to sandy silt till deposit range between about 6 per cent and 23 per cent.

4.2.1.6 Sand

A 1.5 m and 2.9 m thick sand layer was encountered within the silt and sand till deposit in Boreholes 20 and 35, respectively. The sand layer contained trace to some silt and trace gravel and clay.

The SPT “N”-values measured within the sand interlayer ranged from 19 blows per 0.3 m of penetration to 84 blows per 0.28 m of penetration, indicating a compact to very dense level of compactness. A grain size distribution test was conducted on one sample of the sand and is shown on Figure A-3 in Appendix A. The natural water content measured on one selected sample of the sand interlayer is about 21 per cent.

4.2.1.7 Groundwater Conditions

Details of the water levels observed in the open boreholes at the time of drilling are shown on the Record of Borehole sheets for Boreholes 18, 20, 21, and 33 to 35 in Appendix A. A standpipe piezometer was installed in Borehole 35 to monitor the groundwater level at the site. The water levels measured in the open boreholes and the piezometers are summarized below:

Borehole No.	Ground Surface Elevation (m)	Depth to Water Level (m)	Groundwater Elevation (m)	Date	Comments
18	271.3	3.1	268.2	13-Jul-17	Open borehole - on completion of drilling
20	271.8	wet at 5.6 m	wet at 266.2	13-Jul-17	Borehole caved to 5.2 m depth upon completion of drilling
21	271.7	wet at 2.4 m	wet at 269.3	11-Jul-17	Borehole caved to 2.4 m depth upon completion of drilling
33	271.9	-	-	11-Jul-17	Open borehole - on completion of drilling
34	271.9	2.4	269.5	10-Jul-17	Open borehole - on completion of drilling
35	267.2	4.3	262.9	20-Jul-17	Open borehole - on completion of drilling
		(-0.6)*	267.8	24-Aug-17	Piezometer



***Artesian Conditions**

The creek level, as shown on the 60% design drawings, is at about Elevation 266.49 m. It is noted that artesian groundwater conditions were measured in the piezometer installed in Borehole 35 when measured in August 2017, about one month after initial installation. The piezometer was screened within the sand interlayer present within the silt and sand till deposit.

It should be noted that the groundwater level in the area is subject to seasonal fluctuations and precipitation events, and should be expected to be higher during wet periods of the year.

4.2.2 Site 21-13C

The plan and profile of the culvert inlet headwall / wingwall location at Culvert 21-13C showing the borehole locations and interpreted soil stratigraphy are shown on Drawing 3. The Record of Borehole sheets (Boreholes 17-18, 17-20, and 17-36) and the laboratory test results for this site are provided in Appendix B.

In general, the subsurface conditions from the surface of Highway 7A near the culvert inlet and proposed headwall / wingwall rehabilitation consist of roadway granular fill (i.e. gravelly sand) underlain by silt and sand fill that was used to construct the existing Highway 7A embankment. The fill soils are underlain by cohesionless deposits of native gravel to sand and gravel to sand. Adjacent to the road embankment and near the end of the existing wing walls, a surficial layer of topsoil was encountered underlain by silty sand containing variable amounts of organics.

4.2.2.1 Topsoil

An approximately 500 mm thick layer of topsoil was encountered immediately below ground surface in Borehole 17-36, which was advanced near the culvert inlet on the south side of Highway 7A.

4.2.2.2 Fill

A 1.4 m thick layer of gravelly sand fill was encountered at ground surface in Boreholes 17-18 and 17-20, advanced at the shoulder of the Highway 7A eastbound lane. The roadway granular soil was underlain by a 4.2 m thick layer of silt and sand fill at both borehole locations. The bottom of the fill layer was encountered at Elevation 257.6 m and 257.5 m at Boreholes 17-18 and 17-20 respectively.

The gravelly sand fill contained trace to some silt and the silt and sand fill was more variable but typically contained trace to some clay, trace gravel. Trace brick fragments and organic inclusions were encountered near the bottom of the silt and sand fill in Borehole 17-20. A laboratory organic content test was conducted on one selected sample of the bottom of the silt and sand fill and measured an organic content of about 6 per cent.

The SPT "N"-values measured within the non-cohesive fill deposit range from 4 blows to 14 blows per 0.3 m of penetration, indicating a very loose to compact level of compactness.

Grain size distribution testing was carried out on two selected samples of the non-cohesive silt and sand fill and the results are shown on Figure B-1 in Appendix B. Atterberg limits testing was carried out on one sample of the silt and sand fill and indicated that the soil was non-plastic. The natural water content measured on selected samples of the non-cohesive fill ranges from about 10 per cent to about 39 per cent, but is generally between about 10 per cent and 13 per cent. The high water content value of 39 per cent was measured on a sample of the fill that contained organics in Borehole 17-20, Sample 5.



4.2.2.3 Gravel

A 1.5 m thick gravel deposit was encountered underlying the fill in Borehole 17-18. The surface of the gravel was encountered at Elevation 257.6 m. The gravel contained trace to some sand and trace amounts of silt.

The SPT “N”-value measured within the gravel deposit was 12 blows per 0.3 m of penetration, indicating a compact level of compactness. The natural water content measured on a selected sample of the gravel deposit is about 3 per cent.

4.2.2.4 Sand

A sand deposit was encountered underlying the gravel in Borehole 17-18. Borehole 17-18 was terminated within the sand, penetrating the sand deposit for 2.5 m. The surface of the sand deposit was encountered at Elevation 256.1 m and the sand deposit contained trace to some silt and trace amounts of gravel.

The SPT “N”- values measured within the sand deposit were 23 blows and 36 blows per 0.3 m of penetration, indicating a compact to dense level of compactness.

Grain size distribution testing was carried out on one selected sample of the sand deposit and the result is shown on Figure B-2 in Appendix B. The natural water content measured on a selected sample of the sand deposit is about 14 per cent.

4.2.2.5 Sand and Gravel

A sand and gravel deposit was encountered underlying the fill in Borehole 17-20. Borehole 17-20 was terminated within the sand and gravel, penetrating the deposit for about 4.0 m. The surface of the sand and gravel deposit was encountered at Elevation 257.5 m. The sand and gravel deposit contained trace to some silt and trace amounts of clay.

The SPT “N”-values measured within the sand and gravel deposit range from 10 blows to 35 blows per 0.3 m of penetration, indicating a compact to dense level of compactness.

Grain size distribution testing was carried out on one selected sample of the sand and gravel deposit and the result is shown on Figure B-3 in Appendix B. The natural water content measured on selected samples of the sand and gravel deposit were 8 per cent and 18 per cent.

4.2.2.6 Silty Sand

A silty sand deposit was encountered underlying the topsoil in Borehole 17-36. Borehole 17-36 was terminated within the silty sand, penetrating the deposit for about 1.9 m. The surface of the silty sand deposit was encountered at Elevation 260.5 m. The silty sand contained trace clay and trace to some organics.

The SPT “N”-values measured within the silty sand deposit range from 6 blows to 21 blows per 0.3 m of penetration, indicating a loose to compact level of compactness.

Grain size distribution testing was carried out on one selected sample of the silty sand deposit and the result is shown on Figure B-4 in Appendix B. The natural water content measured on selected samples of the silty sand deposit range from about 25 per cent and 33 per cent.



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4.2.2.7 Groundwater Conditions

The groundwater levels in the open boreholes were measured upon completion of drilling operations. Upon completion of Boreholes 17-18 and 17-20, the water level was measured to be at a depth of about 1.2 m below ground surface (corresponding to Elevation 262.0 m and 261.9 m, respectively); however, it should be noted that these boreholes progressively caved soon after the augers were removed and may have influenced the observed water level. Borehole 17-36 was observed to be dry upon completion of drilling, although wet soil samples were noted below a depth of 1.8 m (Elevation 259.2 m).

The creek water level was measured at Elevation 260.0 m in December 2015, as shown on the 60% design drawings.

It should be noted that the groundwater level in the area is subject to seasonal fluctuations and precipitation events, and should be expected to be higher during wet periods of the year.

5.0 CLOSURE

This Foundation Investigation Report was prepared by Mr. Kevin Wallin, and was reviewed by Ms. Nikol Kochmanová, P.Eng., a geotechnical engineer with Golder. Mr. Kevin Bentley, P.Eng., an Associate and MTO Foundations Designated Contact for Golder, conducted an independent technical and quality control review of this report.

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PART B

**FOUNDATION DESIGN REPORT
CULVERT REPLACEMENT / REHABILITATION OF SITE NOS. 22-415C AND
21-13C
PORT PERRY AND BLACKSTOCK, ONTARIO
GWP 2248-14-00**



6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides discussion and foundation engineering recommendations for the design of the proposed culvert replacement at Site 22-415C and new headwall / wingwall rehabilitation at Culvert 21-13C in Durham Region, Ontario.

The recommendations presented are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation. The discussion and recommendations presented are intended to provide the design engineers with sufficient information to assess the feasible foundation alternatives and carry out the detail design of the culvert and associated wall foundations. The foundation investigation and design report with the interpretation and recommendations are intended for the use of the Ministry of Transportation, Ontario (MTO) and shall not be used or relied upon for any other purpose or by any other parties, including the construction or design-build contractor. The contractor must make their own interpretation of the factual information provided in Part A (Foundation Investigation Report). Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project, and for which special provisions or operational constraints may be required in the Contract Documents. Those requiring information on the aspects of construction must make their own interpretation of the factual information provided, as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.1 General

6.1.1 Site 22-415C (Highway 7A, Port Perry)

Based on the General Arrangement (GA) drawing provided to us by CIMA+ (dated May 10, 2018), the existing culvert is to be replaced by a single concrete box culvert with a span of 3 m, rise of 2.4 m, and length of 30 m. A new 19 m long endwall / retaining wall is proposed at the outlet (north) side of the culvert to retain the existing embankment and replace the existing concrete endwall. Based on the GA drawings, the preferred alternative for the new endwall / retaining wall is a Retained Soil System (RSS) wall. The proposed new culvert will have an invert elevation of about 266.4 m at the upstream (south) end and about 266.1 m at the downstream (north) end. The proposed culvert is to be constructed using open cut methods (i.e., trenchless methods are not proposed).

The new culvert will be installed directly adjacent (within 1.5 m) to the existing culvert on the west side. Although the existing open footing culvert span and rise is known (varies along length), the design drawings for the original structure and extensions are not available and details about the open footings (i.e. size and founding level) are not known. In addition, there are no design drawings / details available for the existing concrete endwall / retaining wall foundation that is to be removed and replaced with the new endwall.

The construction is to be completed in stages with temporary protection systems installed to maintain traffic along Highway 7A. The end sections of the existing culvert and the endwall will be removed while the mid-section will be left in place and filled with concrete to facilitate staging. A temporary protection system will be required near the centreline of existing Hwy 7A to facilitate the conventional "half and half" construction/removal of the culverts.

As part of the temporary diversion of the eastbound traffic around the culvert replacement work area to maintain two lanes of traffic during construction, the highway embankment is proposed to be temporarily widened to the south by about 5 m. The proposed embankment widening side slopes would extend onto private property and impede stormwater drainage flow paths south of the highway; as a result, a temporary protection system is needed



to accommodate the temporary widening for a length of about 150 m on the south side of Highway 7A, east and west of the culvert.

6.1.2 Site 21-13C (Highway 7A, near Blackstock, Ontario)

Based on the GA drawing provided to us by CIMA+ (dated May 10, 2018), the existing concrete headwall (about 0.5 m high above top of culvert) is to be repaired and raised on the upstream (south) end of the existing culvert to accommodate a localized embankment widening / grade levelling (less than 0.5 m high for an area about 1.8 m wide by about 20 m long) near the edge of the existing pavement structure along Highway 7A eastbound lanes to level the ground surface to control erosion and stabilize the crest of the existing slope. Referring the GA drawing, the new headwall will be about 1 m high above the top of the existing culvert and will be extended about 6.3 m east and 6.3 m west beyond the edge of the culvert. The new wall extensions are shown to be an RSS wall founded near the bottom of the existing culvert as the preferred structural option.

The existing flared concrete wing walls are to be repaired and remain in place to continue to control erosion and direct the creek into the existing culvert. There are no drawings or design details available regarding the existing wing wall foundations and observations made during site visits indicates active erosion near the bottom of the wing walls.

6.2 Site Classification

6.2.1 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the 2014 Canadian Highway Bridge Design Code and its Commentary (CHBDC, 2014), the culvert replacement, new headwall and associated retaining walls and their foundation systems are considered to be classified as having a “typical consequence level” associated with exceeding limits states design. In addition, given the level of foundation investigation completed to date in comparison to the degree of site understanding in Section 6.5 of CHBDC (2014), the level of confidence for design is considered to be a “typical degree of site and prediction model understanding.” Accordingly, the appropriate corresponding ULS and SLS consequence factor, Ψ , from Table 6.1 and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs} , from Table 6.2 of the CHBDC (2014) have been used for design, as indicated in the sections below.

6.3 Culvert Replacement Options (Site 22-415C)

Although it is recognized that the preferred structural culvert replacement type at Site 22-415C will consist of a precast box culvert, this section of the report presents advantages, disadvantages and geotechnical recommendations for a box culvert replacement, as well as for open footing culvert replacement alternative.

Based on the soil and groundwater condition, either a box culvert or open footing (shallow foundation) concrete culvert is feasible for the replacement of the existing culvert. Deep foundations are not practical nor required at this site, as competent founding stratum is present for shallow foundations for an open footing culvert or for a concrete base slab (i.e. box culvert). Both pre-cast and cast-in-place concrete elements are feasible from a foundations perspective; however, a pre-cast option is preferred over a cast-in-place option as it would likely be more efficient to install, require less construction time and, therefore, less disruptive to traffic.

The advantages, disadvantages, relative costs, and risks/consequences associated with both the pre-cast box culvert and cast-in-place open footing culvert replacement options are summarized in Table 1 following the text of



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this report. From a foundations perspective, a pre-cast box culvert replacement is preferred over cast-in-place open footing culvert replacement based on the following:

- Pre-cast box culvert replacement reduces the depth of excavation and associated groundwater control requirements as compared with open footings. It is noted that artesian groundwater pressures were measured within sand deposits at depth and reduced excavation depths will reduce the risk of base heave, upward seepage, instability of subgrade, etc.
- Pre-cast box culvert segments can often be installed more expeditiously than cast-in-place open footing culverts, resulting in shorter durations for dewatering and surface water pumping and for temporary traffic detours.
- Pre-cast box culvert segments are more tolerant of total and differential settlement that may be experienced during the temporary embankment widening for traffic diversion and if the highway embankment is widened in the future.

It is noted, however, that a box culvert replacement may be subject to specific fisheries requirements related to channel substrate, in which case an open footing culvert is feasible (though not preferred from a foundations perspective). Alternatively, a deeper box culvert section with appropriate substrate material may be used for replacement to satisfy fisheries requirements.

Recommendations for both a box culvert replacement and a shallow foundation (open footing) culvert replacement are provided in the following sections.

6.3.1 Concrete Box Culvert

6.3.1.1 Founding Elevations

Following typical MTO practice, it is not considered necessary to found box culverts at the standard depth for frost protection purposes, as the majority of the box structure is fully buried below the embankment and the concrete sections are typically tolerant of small magnitudes of movement related to freeze-thaw cycles, should these occur. The box culvert should, however, be founded below any existing and surficial organic or disturbed (loosened / softened) materials.

Based on the 60% design drawings provided by CIMA+, the proposed culvert will have an invert elevation of about 266.4 m at the upstream (south) end and 266.1 m at the downstream (north) end. Based on the borehole information, the new culvert will be founded on the compact to very dense silt and sand till, below any embankment fill, and soft or loose soils.

Given that the new culvert will be located directly adjacent to the existing culvert, care must be exercised when removing / abandoning the existing culvert so as not to disturb the founding soils of the new box culvert. Preferably, the existing culvert open footings should remain in place and the existing culvert above the stream bed (C-shaped concrete section) could be cut and removed to reduce disturbance to the founding soils. During construction, if the existing footings are located above or near the founding subgrade of the new box culvert, consideration could be given to removing the existing foundations.

Depending on the proximity to and level of the existing culvert footings, and construction practices used during installation of the existing culvert, there is the potential for previous backfill soils and/or loosened / softened soils



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to be present within the proposed new box culvert foundation footprint and these soils are not considered capable of supporting the box culvert. If such soils are encountered, subexcavation down to the native compact to very dense silt and sand till will be required and the subgrade will need to be raised to founding level using suitable engineered granular fill such as Ontario Provincial Standard Specification (OPSS.PROV) 1010 (*Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material*) Granular A or Granular B Type II. The granular engineered fill will need to be placed and compacted in accordance with the requirements OPSS.PROV 501 (*Construction Specification for Compacting*).

The silt and sand till subgrade will be susceptible to disturbance and degradation on exposure to water and construction traffic. To protect the subgrade from such degradation, a 100 mm thick 20 MPa concrete working slab could be placed on the subgrade within the culvert footprint or alternatively a minimum 150 mm thick layer of compacted Granular A (i.e. bedding), as further discussed in Section 6.9. The concrete working slab should be placed within four hours after inspection and approval of the subgrade. In this case, a 75 mm thick layer of OPSS.PROV 1010 Granular A or concrete fine aggregate meeting the gradation requirements set out in OPSS.PROV 1002 (*Material Specification for Aggregates - Concrete*) should be placed on top of the concrete working slab or incorporated into the upper portion of the bedding to provide a “levelling pad” for the box culvert.

Groundwater and surface water control will be required during the excavation operations and construction of box culvert replacement. Groundwater seepage from the fill, silty sand to sand and silt and sand till deposits into the excavation is expected and will need to be controlled using adequate dewatering procedures that could include ditching and gravity drainage in combination with pumping from filtered sumps, adequate diversion of the creek and use of cofferdams to create a sufficient barrier to control seepage as necessary. As discussed further in Section 6.9, it is recommended that a Non-Standard Special Provision (NSSP) be included in the Contract Documents to address groundwater control requirements for the culvert replacement.

The box culvert subgrade should be inspected by a qualified geotechnical personnel following subexcavation to ensure that the founding subgrade is consistent with the foundation investigation results and that all existing fill and surficial organic soils or other unsuitable material have been removed and replaced with suitable engineered fill, in accordance with OPSS 422 (*Box Culverts and Box Sewers in Open Cut*) and OPSS 902 (*Excavating and Backfilling Structures*).

6.3.1.2 Factored Geotechnical Resistances

For a box culvert (3.0 m wide) founded at the elevations provided in Section 6.3.1.1, on the compact to very dense silt and sand till or properly placed and compacted engineered granular fill, the following factored ultimate and serviceability resistance values may be used for design:

Founding Soil	Factored Ultimate Geotechnical Resistance (kPa)	Factored Serviceability Geotechnical Resistance for 25 mm of settlement (kPa)
Compact to very dense silt and sand (Till)	500	450
Compacted Granular A or B Type II engineered fill over compact to very dense silt and sand (Till)	500	350



The factored ultimate and serviceability geotechnical resistances are dependent on the culvert dimensions and founding elevation and as such, the geotechnical resistance values should be reviewed if the culvert dimensions or founding elevation changes from those provided and outlined in Section 6.3.1.1.

The factored geotechnical resistance values provided above are based on loading applied perpendicular to the top surface of the culvert. Where the load is not applied perpendicular to the top surface of the culvert, inclination of the load should be taken into account in accordance with Section 6.10.4 of the CHBDC (2014) and its Commentary.

6.3.1.3 Resistance to Lateral Loads/Sliding Resistance

Resistance to lateral forces/sliding resistance between the pre-cast concrete base slab and the subgrade should be calculated in accordance with Section 6.10.5 of the CHBDC (2014). For pre-cast concrete box culvert segments placed on compacted granular bedding, the factored coefficient of friction, $\tan \delta$, can be taken as 0.4.

6.3.2 Open Footing Culvert / Strip Footing

6.3.2.1 Founding Elevations

Strip footings for an open footing culvert replacement, and for any associated concrete toe wall/retaining wall, should be founded at a minimum depth of 1.6 m below the lowest surrounding grade to provide adequate protection against frost penetration, as per OPSD 3090.101 (*Foundation Frost Depths for Southern Ontario*). In addition, the footings should extend below any existing fill and organic materials, and any disturbed/loosened/softened material where present.

Based on the GA drawings, the proposed culvert will have an invert elevation of about 266.4 m at the upstream (south) end and 266.1 m at the downstream (north) end. As a result, strip footings would need to be founded below Elevation 264.8 m at the upstream (south) end and Elevation 264.5 m at the downstream (north) end. Based on the borehole information, the open footings would be founded within the very dense silt and sand till which is considered competent to support the shallow foundations.

Given that the new culvert open footing on the east side will be located directly adjacent to the existing culvert footings, care must be exercised when removing / abandoning the existing culvert so as not to disturb the founding soils supporting the new culvert. Preferably, the existing culvert open footings should remain in place to reduce disturbance to the founding soils. During construction, if the existing footings are located above or near the founding subgrade of the new culvert open footings, consideration could be given to carefully removing the existing foundations. However, this is not recommended given that artesian groundwater conditions were also encountered within the sand layers present at depth and removing the existing footings could disturb and/or promote artesian groundwater flow to the ground surface. Given that the founding elevation of the new open footings is as low as Elevation 264.5 m, and the top of the sandy aquifer exhibiting artesian water pressures was encountered between about Elevation 263 m and 264 m, there is also a risk that subexcavation for the new footings could puncture into the aquifer and/or lead to artesian groundwater flows could compromise the founding subgrade of the strip footings unless advanced depressurization / dewatering prior to excavation is executed.

Groundwater and/or surface water control will be required during the excavation operations and construction of the open footing culvert replacement option. Groundwater seepage from the fill, silty sand to sand and silt and sand till deposits into the excavation is expected, and there is a higher risk that artesian groundwater flows and/or instability of the foundation base could occur if advanced dewatering / depressurization is not performed. As



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discussed further in Section 6.9, it is recommended that a NSSP be included in the Contract Documents to address groundwater control requirements for the culvert replacement, especially if open footings are to be considered.

The footing subgrade would need to be inspected by qualified geotechnical personnel following excavation, in accordance with OPSS 902 (*Excavating and Backfilling Structures*) to check that founding soils are consistent with the foundation investigation report and that all existing fill, disturbed, loosened/softened and/or other unsuitable materials have been removed and replaced with suitable engineered fill up to the founding level.

6.3.2.2 Factored Geotechnical Resistance

Strip footings placed on the native dense to very dense silt and sand till subgrade should be designed based on the following factored geotechnical ultimate and serviceability resistance values:

Footing Width (m)	Founding Soil	Factored Ultimate Geotechnical Resistance (kPa)	Factored Serviceability Geotechnical Resistance for 25 mm of settlement (kPa)
0.6 to 1.2	Dense to very dense silt and sand (Till)	450	400
	Compacted Granular A or B Type II engineered fill over compact to very dense silt and sand (Till)	450	350

The factored geotechnical resistances are dependent on the culvert footing width and founding elevation and as such, the geotechnical resistances should be reviewed if the footing width is different than the width specified above or the founding elevation differs from that given in Section 6.3.2.1.

The factored geotechnical resistances provided are based on loading applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footings, inclination of the load should be taken into account in accordance with Section 6.10.4 of the CHBDC (2014) and its Commentary.

6.3.2.3 Resistance to Lateral Loads / Sliding Resistance

Resistance to lateral forces / sliding resistance between the base of the concrete footings and the subgrade should be calculated in accordance with Section 6.10.5 of the CHBDC (2014). For cast-in-place concrete (either the footing itself, or a concrete working slab) and potential interface materials, the factored coefficient of friction, $\tan \delta$, can be taken as follows:

- Cast-in-place footing or working slab to sand and silt till: $\tan \delta = 0.5$
- Cast-in-place footing to compact granular pad: $\tan \delta = 0.5$

6.4 Culvert Bedding, Backfill, Cover and Erosion Protection

For a box culvert replacement, the bedding, levelling pad and backfill requirements should be in accordance with OPSS 422 (*Box Culverts and Box Sewers in Open Cut*). The box culvert should be provided with at least 150 mm of OPSS.PROV 1010 (*Aggregates*) Granular 'A' material for bedding / levelling pad purposes.

Backfill and cover for the concrete culvert (box culvert or open footing) should be completed similar to OPSD 803.010 (*Backfill and Cover for Concrete Culverts*). Backfill and cover to culverts should consist of free-draining



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granular fill meeting the requirements of OPSS.PROV 1010 (*Aggregates*) Granular A or Granular B Type II. The backfill should be placed and compacted in accordance with OPSS.PROV 501 (*Compacting*). The culvert replacement should be designed for the full overburden pressure and live load, assuming that the embankment fill has a unit weight of 22 kN/m³ for Granular A, and 21 kN/m³ for Granular B Type II or select earth fill above and/or surrounding the culvert.

To prevent surface water from flowing either beneath the culvert (potentially causing undermining and scouring) or around the culvert (creating seepage through the embankment fill, and potentially causing erosion and loss of fine soil particles), a concrete cut-off wall should be provided at the upstream end of the box culvert replacement. Alternatively, a clay seal should be provided at the upstream end of an open footing or box culvert replacement. Clay seals should also be placed adjacent to the culvert inlet opening for both box culvert and open footing structure types. The clay material should meet the requirements of OPSS.PROV 1205 (*Material Specification for Clay Seal*). The clay seal should have a thickness of 1 m, and the seal should extend from a depth of 1 m below the scour level to a minimum horizontal distance of 2 m on either side of the culvert inlet opening, and a minimum vertical height equivalent to the high water level including treatment of the adjacent side slopes.

If the creek flow velocities are sufficiently high, provision should be made for scour and erosion protection (suitable non-woven geotextiles and rip-rap) at the culvert inlet and outlet, including in front of any wing walls/retaining walls adjacent to the creek channel. The requirements for and design of erosion protection measures for the culvert inlet should be assessed by the hydraulic design engineer. As a minimum, rip-rap treatment for the culvert outlet should be consistent with the standard Treatment Type A presented in OPSS 810.010 (*Rip-Rap Treatment for Sewer and Culvert Outlets*), with the rip-rap placed up to the toe of slope level, in combination with the cut-off measures noted above.

6.5 Retaining Wall Options (Site 22-415C and Site 21-13C)

A proposed new retaining wall (endwall) is proposed at the downstream (north) end of Site 22-415C to replace the existing concrete wall. In addition, new retaining walls (headwall extensions) are proposed at the upstream (south) end of the existing culvert at Site 21-13C.

At Site 22-415C, based on the design drawings, the new retaining wall (endwall) at Site 22-415C will be at the same location as the existing concrete wall but extended for a total length of about 20 m. No details on the existing wall foundation is available. The new wall foundations are shown to be “stepped” to follow the valley slope grade of about 2H:1V. The corresponding grade at the toe of the wall follows the 2H:1V slope and is lowest adjacent to the culvert where the toe of wall is at about Elevation 266.5. The top of the new wall is about Elevation 270.5 m, about 0.5 m higher than the existing wall. The ground surface at the top of the wall slopes upward at about 2H:1V to meet the shoulder of Highway 7A that is at about Elevation 271.5. In summary, the maximum height of the exposed face of the new wall is about 4 m, and the total height of the embankment is about 5 m. The preferred wall design from a structural perspective is an RSS wall.

At Site 21-13C, referring to the design drawings, the existing concrete headwall (about 0.5 m high above top of culvert) is to be repaired and raised on the upstream (south) end of the existing culvert to accommodate a localized embankment widening / grade levelling (less than 0.5 m high for an area about 1.8 m wide by about 20 m long) near the edge of the existing pavement structure along Highway 7A eastbound lanes to control erosion and stabilize the crest of the existing slope. Referring the GA drawing, the new headwall will be about 1 m high above the top of the existing culvert and will be extended about 6.3 m east and 6.3 m west beyond the edge of the culvert.



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The new wall extensions are shown to be an RSS walls founded near the bottom of the existing culvert as the preferred structural option and these new wall extensions are discussed in this section.

Considering the proposed new retaining walls (i.e. endwall at Site 22-415C and headwall at Site 22-14C) are located where minor grade raise is expected (typically less than 0.5 m), the foundation soils have essentially been “preloaded” by the existing highway embankment at both locations. Given the generally compact to very dense silt and sand soils at Site 22-415C and “preloaded” loose to compact sand and silt fill soils at Site 21-13C, the following wall types and foundation options are considered appropriate for the new retaining walls:

- **Reinforced Earth Slope:** A reinforced earth slope constructed at an inclination of 1 horizontal to 1 vertical (1H:1V), or even steeper, is geotechnically feasible at both sites; however, it is understood that there is insufficient space and/or logistical barriers (drainage features, etc.) for all but vertical retaining solutions, and hence reinforced slopes are not discussed further in this report.
- **Reinforced Soil System (RSS) Wall:** An RSS wall is geotechnically feasible given the competent nature of the shallow soil conditions and given the fact that there is limited grade raise; hence, there is limited net additional load at the wall locations. RSS wall types are often advantageous relative to concrete walls on shallow foundations as shallower excavation depths are required (frost depth typically not required), with associated reduction in groundwater control, cofferdam and/or protection system requirements. In order to install the reinforcing strips, larger amounts of excavation may be required compared to other wall types. RSS walls are typically more tolerable to post construction settlements, which is especially advantageous at Site 21-13C where founding in the existing fill soils may be considered an option. It should be noted that an RSS wall within a floodplain or near flowing water, which may be applicable to these sites, would require a site-specific design to reduce the potential for migration of soil through the facing.
- **Concrete Retaining Wall on Shallow Foundations:** A concrete retaining wall supported on shallow strip footing foundations is geotechnically feasible for both sites. Temporary protection systems would be required parallel to Highway 7A to permit excavation through the existing embankment side slopes to reach the footing founding level on native soils. The excavation is expected to extend below the groundwater level at both sites and greater groundwater control (possible depressurization and advanced dewatering operations prior to excavation) would be required at both site.
- **Concrete Retaining Wall on Deep Foundations:** A concrete wall supported on deep foundations (driven piles or caissons) is not required at both sites due to the competent materials being present at shallow depth for the other options. Extension excavation is required for the culvert replacement at Site 22-415C and given the relatively small footprint of improvements needed at Site 21-13C, this option is not discussed further in this report.
- **Soldier Pile and Concrete Panel Wall:** A soldier pile and concrete panel system is considered feasible from a geotechnical/foundation perspective. However, it is anticipated that either a pile driving rig or caisson rig would have to access and work within the staging areas and possibly within the floodplain, and such access and work zone preparation may prove to be challenging, as experienced during the foundation field investigation program. If soldier piles are to be installed in pre-augered holes rather than driven, temporary liners would be required, in conjunction with the use of drilling fluids, to control the ground and groundwater for socket formation prior to the placement of soldier piles. For the permanent wall, special measures may be required to minimize the potential for loss of soils from behind the concrete panels, due to the location



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adjacent to the creek channel and floodplain. At Site 22-415C, the excavation for installation of the new culvert makes this option less desirable, as it is typically used for top-down construction. Based on these risks, geotechnical/foundation recommendations are not provided for this type of retaining wall.

- **Cantilevered Concrete Walls from Concrete Box Culvert:** At both sites, consideration could be given to using cantilevered concrete walls extending each side of the box culvert with the base slab of the box culvert acting as the supporting foundation. Based on conversations with the structural designer, this option was considered; however, the length of the endwall at Site No. 22-415C makes this option not feasible. At site 21-13C, the additional vertical and lateral loading on the existing box culvert end section (albeit relatively low) will create localized forces and moments that will be distributed to the existing wing walls (for which the foundations are not known) and founding fill materials (known to be present a significant depth below the bottom of the culvert and contain organics and construction debris). Based on these facts, there is a high risk that the existing box culvert end section could be compromised and not perform satisfactorily and this option is not considered further.

A comparison of the various retaining wall options based on advantages, disadvantages, relative costs and risks is presented in Table 2. Based on this comparison and given the subsurface conditions as encountered in the boreholes, the preferred alternative from a geotechnical perspective may be summarized as:

- Site 24-415C: RSS wall
- Site 21-13C: RSS wall

The following sections of this report present foundation recommendations for shallow concrete footings and RSS wall options.

6.5.1 Concrete Cantilever Wall Founded on Shallow Foundations

6.5.1.1 Founding Elevations

Strip footing foundations are feasible for the support of the proposed retaining walls and should be founded below the existing fill materials on native subgrade (or engineered fill placed on native subgrade) and at a minimum depth of 1.6 m below the adjacent final grade to provide adequate protection against frost penetration as per OPSD 3090.101 (*Foundation, Frost Penetration Depths for Southern Ontario*).

At Site 22-415C, the fill layer extends to about Elevation 268.6 m in Borehole 18; however, given the toe of the existing wall is at Elevation 266.5 m adjacent to the existing culvert, the fill is expected be locally deeper near the culvert. Below the fill, the native compact sand and silt and compact to very dense silt and sand (Till) are present and are considered suitable to support the foundations and/or engineered fill to support foundations.

At Site 21-13C, the existing fill layer extends to about Elevation 257.5 m. Subexcavation of up to about 6 m of existing fill will be required to expose the native gravel to sand and gravel that is considered capable of supporting the foundations and/or engineered fill to support foundations.

Temporary excavations to reach founding subgrade for support of foundations and/or engineered fill to support foundations will extend below the groundwater level at both sites and groundwater control / dewatering will be required. A Non-Standard Special Provision for dewatering structure excavations should be included in the Contract Documents, as discussed further in Section 6.9. Given the artesian groundwater conditions encountered



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at Site 22-415C (Borehole 35) and permeable gravelly soils at Site 21-13C, advanced dewatering / depressurization will be required prior to excavating.

The silt and sand to sand and gravel subgrade will be susceptible to disturbance and degradation on exposure to water and construction traffic. It is recommended that a 100 mm thick, 20 MPa concrete working slab be placed within four hours following inspection and approval of the subgrade, to protect the subgrade from softening / loosening. This requirement should be illustrated on the Contract Drawings, and an NSSP should be included in the Contract Documents. An NSSP is included for this item in Appendix C.

The footing subgrade should be inspected by qualified geotechnical personnel following excavation, in accordance with OPSS 902 (*Excavating and Backfilling Structures*) and SP109S12 (*Amendment to OPSS 902*) to check that all existing fill and/or other unsuitable material have been removed to the founding elevation. In areas where deeper fill, loosened, softened materials are encountered (possibly during removal of existing culvert and retaining wall), further subexcavation of fill or unsuitable material will be required. The sub-excavated area and any areas where competent subgrade is below the proposed new founding level could be backfilled with engineered granular fill meeting OPSS.PROV 1010 (*Aggregates*) Granular 'A' or Granular 'B' Type II that is placed and compacted in accordance with OPSS.PROV 501 (*Compacting*). Alternatively, additional concrete or unshrinkable fill could be placed up to founding level. If replacement of unsuitable materials with engineered fill is being considered, the area to be subexcavated should be defined by a line extending from the top of the engineering fill pad outward and downward at 1H:1V. The top of the granular engineered fill should extend at least 1 m beyond the plan limits of the footing, and constructed in accordance with OPSS.PROV 501 (*Compacting*).

Additional information regarding excavation and temporary protection systems (as required) are discussed further in Section 6.9.

6.5.1.2 Factored Geotechnical Resistance

Concrete strip footings (2 m to 3 m wide) founded on the properly prepared subgrade (as discussed in the previous section) may be designed based on the factored ultimate geotechnical resistance and the factored serviceability geotechnical resistance (for 25 mm of settlement) given below for both sites.

Site	Founding Soil	Factored Ultimate Geotechnical Resistance (kPa)	Factored Serviceability Geotechnical Resistance for 25 mm of settlement (kPa)
22-415C	Compact silt and sand / compact to very dense silt and sand (Till)	425	375
	Compacted Granular A or B Type II engineered fill over compact silt and sand / compact to very dense silt and sand (Till)	425	350
21-13C	Compact to dense sand and gravel to gravel	500	400
	Compacted Granular A or B Type II engineered fill over compact to dense sand and gravel to gravel	500	350



The geotechnical resistances and settlement are dependent on the footing size, configuration and applied loads; the geotechnical resistances should, therefore, be reviewed if the selected footing width or founding elevation differs from those given above.

The geotechnical resistances provided above are given for loads applied perpendicular to the surface of the footing. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Sections 6.10.4 of the CHBDC (2014).

6.5.1.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding between the concrete footings and the subgrade should be calculated in accordance with Section 6.10.5 of the CHBDC (2014). For cast-in-place concrete footings constructed on a concrete working slab that is cast on top of the compact to dense silt and sand to silty sand and sand and gravel to gravel or on Granular A or Granular B Type II engineered fill, the coefficient of friction, $\tan \delta$ or $\tan \phi'$, can be taken as follows:

- Cast-in-place footing to concrete working slab: $\tan \delta = 0.7$
- Cast-in-place concrete to silt and sand to silty sand deposit: $\tan \phi' = 0.5$
- Cast-in-place concrete to sand and gravel to gravel deposit: $\tan \phi' = 0.5$

6.5.2 Retaining Soil System (RSS) Walls

It should be noted that an RSS wall design within a floodplain or near a watercourse, which is applicable to these sites, is not currently approved in the Designated Sources for Materials (DSM) Index and would require the Contractor to submit a site-specific design to the MTO RSS Committee for approval, and would require a minimum 8 week review window. The submission shall include working drawings, supporting design documentation and commentary which specifically address the proposed RSS design with respect to the following:

- a. RSS embedment depth and scour protection;
- b. Backfill material and the control of migration of fines;
- c. Performance in differential hydrostatic pressures;
- d. Pullout capacity and frictional resistance between reinforcements and select backfill under fully saturated conditions;
- e. CHBDC structure design requirement for a 75 year service life – stability, durability, long-term performance.

An NSSP to notify the Contactor of special design and submission requirements for this item is to be included in the Contract documents and is included in Appendix C.

6.5.2.1 Founding Elevations

A typical RSS wall consists of a reinforced earth mass with a front facing supported on a strip footing (placed on a levelling pad) founded at shallow depth below the ground surface. The footing and the RSS reinforced earth mass should be founded below any existing topsoil, unsuitable fill and/or loosened/disturbed soils.



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Based on the design drawings provided, the retaining walls at both sites are proposed to be stepped, with the base of the RSS wall levelling pad at Site 22-415C ranging from about Elevation 265.5 m to 268.5 m and at Site 21-13C ranging from about Elevation 259 m to 261 m.

At Site 22-415C, the existing fill layer extends to about Elevation 268.6 m and 269.0 m in Borehole 18 and 33, below which native compact silt and sand soils are present. However, given that the toe of the existing wall is at Elevation 266.5 m adjacent to the existing culvert, existing fill is expected to be locally deeper near the culvert. Below the fill, the native compact sand and silt and compact to very dense silt and sand (Till) are present and are considered suitable to support the foundations and/or engineered fill to support the RSS wall.

At Site 21-13C, the existing fill layer extends to about Elevation 257.5 m. Subexcavation of at least 6 m of existing fill would be required to expose the native gravel to sand and gravel that is considered capable of supporting the foundations and/or engineered fill to support foundations. Considering the grade raise in this area is localized and less than 0.5 m, consideration can be given to founding an RSS wall on the existing cohesionless fill that is compact in the upper zone and transitions to a loose state of compactness below a depth of about 1.5 m below road surface. Based on the boreholes, the fill contains organics and construction debris below a depth of about 3.7 m (Elevation 259.4 m) and perched groundwater was encountered in the boreholes at a depth of about 1.2 m (Elevation 262 m), therefore, a founding subgrade as high as possible in the fill is preferred (above Elevation 261 m) to reduce excavation and dewatering efforts. Consideration could be given to founding the RSS walls at lower elevations; however, unsuitable fill containing organics may be encountered and will need to be subexcavated and replaced with suitable compacted engineered granular fill such as OPSS.PROV 1010 (*Aggregates*) Granular B Type II material.

Temporary excavations to reach founding subgrade for support of foundations and/or engineered fill to support foundations will extend below the groundwater level at both sites and groundwater control / dewatering will be required. A Non-Standard Special Provision for dewatering structure excavations should be included in the Contract Documents, as discussed further in Section 6.9. Given the artesian groundwater conditions encountered at Site 22-415C (Borehole 35) and permeable gravelly soils at depth at Site 21-13C, the founding elevation should be kept as high as possible to reduce dewatering efforts and avoid advanced dewatering / depressurization prior to excavating.

The footing subgrade should be inspected by qualified geotechnical personnel following excavation, in accordance with OPSS 902 (*Excavating and Backfilling Structures*) to check that all existing unsuitable fill and any disturbed / loosened or softened materials have been removed. The RSS reinforced soil mass should consist of Granular 'A' or Granular 'B' Type II engineered fill founded on competent fill materials or native soils. The facing footing should be placed on a minimum 500 mm thick levelling pad of compacted OPSS.PROV 1010 (*Aggregates*) Granular 'A' or Granular 'B' Type II extending at least 1.0 m beyond the outside edge of the facing footing, then downward at 1H:1V as per Figure 5.2 in MTO's RSS Design Guidelines (MTO, 2008). It is recommended that the bottom of the levelling pad be below frost depth (1.6 m); however, a minimum 800 mm is considered acceptable with the bottom of the front facing buried at least 500 mm below ground surface (MTO, 2008).

The RSS retaining walls are to be designed for low performance rating in accordance with the RSS Design Guidelines (MTO, 2008) and MTO Special Provision (SP) 599S22 (Retained Soil System).



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The reinforced soil mass should be keyed or stepped into the existing embankment fill side-slope, if applicable, by benching into the embankment fill, similar to OPSD 208.010 (*Benching of Earth Slopes*) in order to reduce abrupt differential settlement.

6.5.2.2 Geotechnical Resistance and Settlement

For the reinforced soil mass founded at the depths / levels discussed in Section 6.5.2.1, the factored ultimate geotechnical resistances and the factored serviceability geotechnical resistance (for 25 mm of settlement) given below may be used for design. These values assume that the reinforced soil mass acts as a unit and uses the full width of the reinforced soil mass, which can be taken as approximately 0.8 times the wall height for design purposes.

Retaining Wall Site	Subexcavation Elevation (m)	Maximum Exposed Wall Height Above Finished Grade (m)	Estimated Minimum Reinforced Width (m)	Factored Ultimate Geotechnical Resistance (kPa)	Factored Serviceability Geotechnical Resistance (kPa) (for 25 mm of Settlement)
22-415C	268.5	4.0	3.7	350	275
21-13C	*above 257.5	3.5	2.8	200	100
	**below 257.5	3.5	2.8	500	400

*founded on existing fill

**founded on native sand and gravel to sand and gravel

At Site 21-13C, based on the GA drawing, the existing Highway 7A side-slope will be lowered (slope at 2H:1V towards existing wing walls) in front of the RSS wall to reduce loading on the existing wing walls. As a result, there will be net unloading direction in front of the RSS wall facing, and slight increase in net loading (less than 0.5 m grade raise) directly behind the RSS wall facing. Actual settlements are challenging to predict for RSS walls founded within the existing fills, however, assuming the fills were placed in a relatively controlled manner during construction of the Highway 7A embankment, differential settlements across the length of the walls are estimated to be less than 25 mm.

Depending on the facing material / appearance, vertical slip joints may be incorporated into the RSS walls at appropriate intervals to accommodate the differential settlements without adversely affecting the aesthetic appearance of the RSS facing. Consideration should be given to extending the upper geogrid reinforcement across the full width of Highway 7A and beyond the length of the RSS walls to reduce the potential for abrupt differential movements / cracking that could lead to reduced performance of the pavement structure in the area.

6.5.2.3 Resistance to Lateral Loads/Sliding Resistance

Resistance to lateral forces / sliding resistance between the reinforced earth fill of the RSS wall and the subgrade should be calculated in accordance with Section 6.10.5 of the *CHBDC (2014)*. The coefficient of friction, $\tan \delta'$, between the compacted granular fills (Granular A or Granular B Type II engineered fill) of the RSS wall and the subgrade may be taken as:

- RSS Granular engineered fill and existing silt and sand fill: $\tan \phi' = 0.5$



- RSS Granular engineered fill and native sand and gravel deposit: $\tan \phi' = 0.6$

6.6 Global Stability

Slope stability analyses have been performed for the proposed retaining walls using the commercially available program SLIDE V7 produced by Rocscience Inc., employing the Morgenstern-Price method of analysis. For the analyses, the Factor of Safety (FoS) of numerous potential failure surfaces was computed in order to establish the minimum FoS; in general, circular slip surfaces were analyzed. The FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum factored FoS of 1.5 is adopted for the design of retaining walls under static conditions at the end of construction as per the CHBDC (2014). This FoS is considered adequate for the retaining walls at this site considering the design requirements and the field data available.

The following parameters have been used in the analysis for the long-term (drained, effective stress) condition, based on field and laboratory test data as well as accepted correlations (Bowles, 1984 and Kulhawy and Mayne, 1990):

Site 22-415C

Soil Deposit	Bulk Unit Weight (kN/m ³)	Effective Friction Angle
Existing granular fill	20	30°
Compact silt to sand	20	32°
Compact to dense silt and sand till	21	34°

Site 21-13C

Soil Deposit	Bulk Unit Weight (kN/m ³)	Effective Friction Angle
Existing granular fill	20	30°
Loose to compact silty sand	19	32°
Compact to dense sand and gravel to gravel	20	34°

A maximum exposed wall height of 4 m and 3.5 m was assumed for the retaining walls at Sites 22-415C and 21-13C, respectively. The groundwater level was inferred from the highest water levels encountered during drilling, as shown on the borehole records and graphically shown on Figures 1 and 2.

Global stability analyses were carried out assuming both RSS walls and concrete cantilever walls on shallow foundations. The results indicate the RSS wall have a factor of safety greater than or equal to 1.5 against global instability as shown on Figures 1 and 2 for Site 21-415C and 22-13C respectively. The results indicate the FoS improves for the concrete cantilever wall on shallow foundation option at both sites.

Based on the design drawings, the highway embankment is to be re-established to pre-construction conditions after the culvert replacement and wall rehabilitation. Any disturbed side-slopes should be re-established with suitable topsoil and vegetated as per OPSS.PROV 804 "Construction Specification for Seed and Cover".



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6.7 Seismic Design

6.7.1.1 Seismic Site Classification

Subsurface ground conditions for seismic site characterization were established based on the results of the field investigation, laboratory testing, and understanding of the geology of the area. The SPT “N”-values measured in the soil layers and the interpreted shear wave velocity of soils extrapolated to 30 m below founding level (based on local experience in the area and regional geology) were used to define the seismic site classification in accordance with Table 4.1 of the CHBDC (2014). Based on this methodology it is considered that a Site Class C and D would be applicable for the design of the culvert replacement / rehabilitation work at Sites 22-415C and 21-13C respectively.

6.7.1.2 Spectral Response Values and Seismic Performance Category

If required and in accordance with Section 4.4.3.4 of the CHBDC (2014), the peak ground acceleration (PGA) values and design spectral acceleration (Sa) values for Site Class C based on the National Resource Canada (NRC) website are presented below.

6.7.1.2.1 Site 22-415C, Highway 7A, Port Perry

Seismic Hazard Values	10% Exceedance in 50 years (475-year return period)	5% Exceedance in 50 years (975-year return period)	2% Exceedance in 50 years (2,475 return period)
PGA (g)	0.035	0.055	0.091
PGV (m/s)	0.03	0.047	0.076
Sa (0.2) (g)	0.06	0.092	0.147
Sa (0.5) (g)	0.041	0.06	0.092
Sa (1.0) (g)	0.024	0.035	0.053
Sa (2.0) (g)	0.011	0.017	0.027
Sa (5.0) (g)	0.0025	0.0041	0.0068
Sa (10.0) (g)	0.0011	0.0017	0.003

6.7.1.2.2 Site 21-13C, Highway 7A, near Blackstock, Ontario

Seismic Hazard Values	10% Exceedance in 50 years (475-year return period)	5% Exceedance in 50 years (975-year return period)	2% Exceedance in 50 years (2,475 return period)
PGA (g)	0.034	0.053	0.087
PGV (m/s)	0.03	0.047	0.075
Sa (0.2) (g)	0.06	0.090	0.142
Sa (0.5) (g)	0.041	0.06	0.091
Sa (1.0) (g)	0.024	0.035	0.053
Sa (2.0) (g)	0.011	0.018	0.027
Sa (5.0) (g)	0.0026	0.0041	0.0068
Sa (10.0) (g)	0.0011	0.0018	0.003



6.8 Lateral Earth Pressures for Design

The lateral earth pressures acting on the culvert walls and on associated retaining walls will depend on the type and method of placement of the backfill material, the nature of the soils/embankment fill behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls. Seismic (earthquake) loading may also be taken into account in the design, if applicable.

The following recommendations are made concerning the design of the walls. These design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select, free draining granular fill meeting the specifications of OPSS.PROV 1010 (*Aggregates*) Granular 'A' or Granular 'B' Type II should be used as backfill behind the walls, and on top of the culvert for a thickness of not less than 300 mm. Granular 'B' Type III can be used if the excavation is dry. Backfill should be placed in a maximum of 200 mm loose lift thickness and nominally compacted. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 (*Compacting*).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with CHBDC Section 6.12.3 and Figure 6.6. Care must be taken during the compaction operation not to overstress the wall. Heavy construction equipment should be maintained at a distance of at least 1 m away from the walls while the backfill soils are being placed. Hand-operated compaction equipment should be used to compact the backfill soils within a 1 m wide zone adjacent to the walls. Other surcharge loadings should be accounted for in the design, as required.
- For restrained walls, granular fill should be placed in a zone with the width equal to at least 1.6 m behind the back of the wall (Figure C6.20(a) of the Commentary to the CHBDC). For unrestrained walls, fill should be placed within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the base of the walls (Figure C6.20(b) of the *Commentary* to the CHBDC).

6.8.1 Static Lateral Earth Pressures for Design

The following recommendations are provided regarding the lateral earth pressures for static (i.e., not earthquake) loading conditions.

- For restrained walls, the pressures are based on the soil strata adjacent to the structure and the following parameters (unfactored) may be used assuming the use of earth fill or existing native materials:

Material	Earth Fill or Existing Native Materials
Soil Unit Weight:	20 kN/m ³
Coefficients of static lateral earth pressure:	
Active, K_a	0.33
At rest, K_o	0.50



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- For unrestrained walls, the pressures are based on using engineered granular fill behind the walls and the following parameters (unfactored) may be used:

Material	Granular A	Granular B Type II	Granular B Type III
Soil Unit Weight:	22 kN/m ³	21 kN/m ³	21 kN/m ³
Coefficients of static lateral earth pressure:			
Active, K_a	0.27	0.27	0.33
At rest, K_o	0.43	0.43	0.50
Passive, K_p	3.7	3.7	3.0

If the culvert or retaining wall structures does not allow lateral yielding, at-rest earth pressures should be assumed for the foundation design. If the culvert or retaining wall structure allows for lateral yielding, active earth pressures should be used in the foundation design. The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.12.1 and Table C6.6 of the Commentary to the CHBDC (2014).

Where space is restricted and the retaining walls are constructed in a top-down fashion, with a thinner or absent zone of granular backfill behind the wall, it is recommended that drainage measures (e.g., pre-fabricated sheets) be incorporated on the back of the walls, before or concurrent with the panel installation, to promote drainage and minimize the risk of frost action during freezing temperatures. The wall system and facing should also incorporate subdrains and weep holes at intervals through the wall face.

6.9 Construction Considerations

6.9.1 Surface Water and Groundwater Control

Control of the surface water and groundwater will be necessary for the construction of the culvert replacement and retaining walls, to allow excavation and foundation construction to be carried out in dry conditions. Dewatering must be in accordance with OPSS.PROV517 and SP517F01 for Site No. 21-13/C. MTO's "FOUN0003" NSSP shall be included in the Contract Documents to address the dewatering requirements for structure replacement and retaining wall construction at Site No.22-415/C. The "filled in" versions of SP517F01 and FOUN0003 were obtained from CIMA+ and are included in Appendix C.

Depending on the time of year and water flow at the time of construction and staging requirements, the surface water being conveyed through the existing culvert could bypass the culvert construction area by means of a temporary pipe, diverted by pumping from behind a temporary barrier (cofferdam) placed/constructed within the existing culvert or channel. Precipitation runoff in the construction area should be directed away from the excavation areas, to prevent ponding of water that could result in disturbance and weakening of the subgrade or granular backfill/bedding material.

At Site 22-415C, the creek water level was measured to be at about Elevation 266.5 m. In borehole 35, advanced near the inlet of the culvert, artesian groundwater conditions were measured at 0.6 m above ground surface (Elevation 267.8 m). Groundwater was observed seeping from the existing endwall and the investigation indicates higher groundwater levels east and west of the culvert along Highway 7A (either side of the existing culvert) ranging from about Elevation 268.5 m to 269.5 m.



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At Site 21-13C, the creek level was measured to be at about Elevation 260 m. Perched groundwater levels were measured at about Elevation 262 m (July 2017) within the existing Highway 7A embankment on either side of the existing culvert.

At Site 22-415C, due to the proximity of the proposed culvert location and the endwall to the edge of the creek, it is recommended that a groundwater cut-off (cofferdam or similar measure) be incorporated to minimize dewatering requirements and potential environmental impacts for excavation of the strip footings and/or RSS segments, as applicable. A cut-off/cofferdam could consist of interlocking steel sheetpiles driven below the proposed base of excavation, although this not recommended due to the artesian groundwater conditions and associated risks. Alternatively, an inflatable bladder system and/or barrier may be adopted to separate the foundation excavation from the creek.

At both sites, depending on the effectiveness of the flow diversion / bypass system, depth of subexcavation to the founding level and proposed staging / sequence of operations, dewatering will be required and the effort may range from conventional pumping from sump pits and drainage trenches (above creek level) to advanced dewatering / depressurization below the creek level. Specialized dewatering techniques would be required if excavations will penetrate into the artesian sandy layers at Site 22-415C and/or the permeable gravel soils at Site 21-13C.

Water taking activities for highway construction projects must meet the latest legislative requirements of the Ministry of the Environment and Climate Change (MOECC), now referred to as the Ministry of the Environment, Conservation and Parks (MECP). Therefore, if groundwater taking limits are less than 50 m³/day, no requirements are needed by the Contractor. If groundwater taking limits range between 50 m³/day and 400m³/day (as anticipated for staged construction / excavation at Site 22-415C and Site 21-13C), an Environmental Activity and Sector Registry (EASR) will be required to be prepared and submitted by the Contractor for each site. If groundwater takings are greater than 400m³/day, a Permit to Take Water will need to be prepared and submitted by the Contractor for acceptance.

6.9.2 Excavation and Temporary Protection Systems

Temporary excavations for the culvert replacement and new retaining walls will be made through the existing fill, sandy silt to silt and sand, gravel, sand and gravel and silt and sand till. Excavation works must be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) and Regulations for Construction Projects in Ontario. At both sites, the existing fill would be classified as Type 3 soil, while the native deposits would be classified as a Type 2 soil, according to the OHSA. Temporary excavations (i.e. those that are open for a relatively short time period) should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V) in accordance with OHSA. If the sites are not dewatered, the saturated granular fill soils and native loose granular soils below the creek level would be classified as Type 4 soil, and temporary excavations should be made with side slopes no steeper than 3H:1V.

It is expected that temporary protection systems will be required for the culvert replacement and retaining wall construction works, installed parallel to Highway 7A to facilitate staging and foundation excavation and removals. The temporary excavation support systems for the culvert replacement and wall rehabilitation works should be designed and constructed in accordance with OPSS.PROV 539 (*Construction Specification for Temporary Protection Systems*). The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS.PROV 539, provided that any adjacent utilities can tolerate this magnitude of deformation.



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The following soil parameters may be used in the design of temporary protection systems, based on field and laboratory test data as well as accepted correlations (Bowles, 1984 and Kulhawy and Mayne, 1990), and taking into account the groundwater level at the time of construction.

Site 22-415C

Soil Deposit	Bulk Unit Weight (kN/m ³)	Effective Friction Angle	Coefficients of Static Lateral Earth Pressure		
			Active, K _a	At rest, K _o	Passive, K _p
Existing granular fill	20	30°	0.33	0.50	3.00
Compact silt to sand	20	32°	0.31	0.47	3.25
Compact to dense silt and sand till	21	34°	0.28	0.44	3.54

Site 21-13C

Soil Deposit	Bulk Unit Weight (kN/m ³)	Effective Friction Angle	Coefficients of Static Lateral Earth Pressure		
			Active, K _a	At rest, K _o	Passive, K _p
Existing granular fill	20	30°	0.33	0.50	3.00
Loose to compact silty sand	19	32°	0.31	0.47	3.25
Compact to dense sand and gravel to gravel	20	34°	0.28	0.44	3.54

It is considered that either a driven, interlocking sheetpile system or a soldier pile and timber lagging system would be suitable for the temporary excavation support at both sites, based on the subsurface soil conditions and groundwater conditions. An interlocking sheetpile system would contribute to both ground and groundwater control; however, there is some risk associated with driving the sheetpiles to sufficient depth within the very dense native till soils (possibly containing cobbles/boulders) that are present at Site 22-415C. In the same regard, obstructions (construction debris such as brick fragments) and organic material (possibly tree branches) was encountered below the bottom of the existing culvert at Site 21-13C. At both sites, these challenges must be considered by the contractor's temporary works designer. For a soldier pile and lagging system, it is anticipated that soldier piles could be either driven or installed in pre-augered holes. Where pre-augered holes are used, they would need to be advanced with temporary liners and drilling fluids to avoid disturbance of the ground, and water seepage must be controlled or include measures to mitigate loss of soil particles through the lagging boards below the groundwater levels, especially near the base of the excavations and adjacent to the creek channel. Lateral support to the sheetpiles or soldier piles could be provided in the form of rakers, temporary anchors or cross-bracing. The selection and design of the protection system will be the responsibility of the Contractor.

To facilitate the culvert replacement at Site 22-415C, temporary southward widening of Highway 7A is proposed as part of the temporary diversion of the eastbound traffic around the culvert replacement work area to maintain two lanes of traffic during construction. To maintain the widening within MTO's right-of-way, a temporary retaining



FOUNDATION REPORT - REVISION 1

CULVERT REPLACEMENT / REHABILITATION AT SITE NOS. 22-415/C AND 21-13/C

wall, possibly consisting of an RSS wall, sheetpile wall, or soldier pile and lagging could be considered. The proposed retaining wall is a temporary structure and its design and construction is the responsibility of the Contractor. Based on the competent materials encountered on site, global stability is not anticipated to be a concern although stability will need to be verified by the Contractor's temporary works designer.

6.9.3 Subgrade Protection

The native and fill soils that will be exposed at the foundation subgrade level will be susceptible to disturbance from construction traffic, groundwater infiltration and/or ponded water. To limit this degradation, it is recommended that a concrete working slab be placed on the subgrade within four hours after preparation, inspection and approval of the footing subgrade. This requirement can be addressed with a note on the General Arrangement drawing and/or with an NSSP, an example is included in Appendix C.

6.9.4 Obstructions

Obstructions (construction debris such as brick fragments) and organic material (possibly tree branches / logs) was encountered below the bottom of the existing culvert at Site 21-13C and very dense native till soils (possibly containing cobbles/boulders) are present at Site 22-415C. At both sites, these challenges must be considered for proper selection of equipment and procedures to allow for excavation, installation of any dewatering systems and retaining walls, and for installation of temporary protection systems. It is recommended that an NSSP be included in the Contract Document to alert the Contractor of the presence of these obstructions in the overburden soils and an example NSSP is present in Appendix C.



FOUNDATION REPORT - REVISION 1 CULVERT REPLACEMENT / REHABILITATION AT SITE NOS. 22-415/C AND 21-13/C

7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Nikol Kochmanová, P.Eng., a geotechnical engineer with Golder. Mr. Kevin Bentley, P.Eng., an Associate and MTO Foundations Designated Contact for Golder, conducted an independent technical and quality control review of this report.

GOLDER ASSOCIATES LTD.



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Geotechnical Engineer



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MTO Foundations Designated Contact, Associate

KAW/NK/KJB/sm

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FOUNDATION REPORT - REVISION 1 CULVERT REPLACEMENT / REHABILITATION AT SITE NOS. 22-415/C AND 21-13/C

REFERENCES

- Bowles, J.E., 1984. *Physical and Geotechnical Properties of Soils*, Second Edition. McGraw Hill Book Company, New York.
- Canadian Standards Association (CSA), 2014. *Canadian Highway Bridge Design Code (CHBDC) and Commentary on CAN/CSA-S6-14*. CSA Group.
- Chapman, L.J., and Putnam, D.F., 1984. *The Physiography of Southern Ontario*, 3rd Edition. Ontario Geological Survey, Special Volume 2. Ontario Ministry of Natural Resources.
- Kulhawy, F.H. and Mayne, P.W. 1990. *Manual on Estimating Soil Properties for Foundation Design*. EL 6800, Research Project 1493 6. Prepared for Electric Power Research Institute, Palo Alto, California.
- Ministry of Transportation Engineering Standards Branch. RSS Design Guidelines. September 2008
- National Resources Canada, 2018. *Earthquake Hazard*. http://www.earthquakescanada.nrcan.gc.ca/hazard-alea/interpolat/index_2015-en.php. Accessed on May 24, 2018.

Ontario Provincial Standard Specifications (OPSS)

OPSS 422	Construction Specification for Precast Reinforced Concrete Box Culverts and Box Sewers in Open Cut
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 517	Construction Specification for Dewatering
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS 902	Construction Specification for Excavating and Backfilling Structures
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material
OPSS.PROV 1205	Material Specification for Clay Seal

Special Provisions

SP109S12	Amendment to OPSS902
SP517F01	Amendment to OPSS517
SP599S22	Retained Soil System, Wall/Slope, Low Performance

Ontario Provincial Standard Drawings (OPSD)

OPSD 208.010	Benching of Earth Slopes
OPSD 803.010	Backfill and Cover for Concrete Culverts with Spans Less Than or Equal to 3.0 m
OPSD 810.010	Granular Rip-Rap Layout for Sewer and Culvert Outlets



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CULVERT REPLACEMENT / REHABILITATION AT SITE NOS. 22-415/C AND 21-13/C

OPSD 3090.101 Foundation Frost Depths for Southern Ontario

ASTM International

ASTM D1586 Standard Test Method for Standard Penetration Test (SPT) and Split Barrel Sampling of Soils

Ontario Water Resources Act

Ontario Regulation 903 Wells (as amended)



LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π	3.1416
$\ln x$,	natural logarithm of x
\log_{10}	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
FoS	factor of safety

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta \sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a)	Index Properties
$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

(a) Index Properties (continued)

w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity index = $(w_l - w_p)$
w_s	shrinkage limit
I_L	liquidity index = $(w - w_p) / I_p$
I_c	consistency index = $(w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_{α}	secondary compression index
m_v	coefficient of volume change
C_v	coefficient of consolidation (vertical direction)
C_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio = σ'_p / σ'_{vo}

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = $\tan \delta$
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

* Density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)

Notes: 1
2

$$\tau = c' + \sigma' \tan \phi'$$

$$\text{shear strength} = (\text{compressive strength})/2$$



LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Split-spoon
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

Condition	N Blows/300 mm or Blows/ft
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

(b) Cohesive Soils Consistency

	<u>kPa</u>	<u>C_u, S_u</u>	<u>psf</u>
Very soft	0 to 12		0 to 250
Soft	12 to 25		250 to 500
Firm	25 to 50		500 to 1,000
Stiff	50 to 100		1,000 to 2,000
Very stiff	100 to 200		2,000 to 4,000
Hard	over 200		over 4,000

Dynamic Cone Penetration Resistance; N_d:

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

IV. SOIL TESTS

w	water content
w _p	plastic limit
w _l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D _R	relative density (specific gravity, G _s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO ₄	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

Note: 1 Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

V. MINOR SOIL CONSTITUENTS

Per cent by Weight

Modifier

0 to 5	Trace
5 to 12	Trace to Some (or Little)
12 to 20	Some
20 to 30	(ey) or (y)
over 30	And (non-cohesive (cohesionless)) or With (cohesive)

Example

Trace sand
Trace to some sand
Some sand
Sandy
Sand and Gravel
Silty Clay with sand / Clayey Silt with sand



FOUNDATION REPORT - REVISION 1 CULVERT REPLACEMENT / REHABILITATION AT SITE NOS. 22-415/C AND 21-13/C

TABLE 1 – COMPARISON OF CULVERT ALTERNATIVES – CULVERT REPLACEMENT

Site 22-415C

G.W.P. 2248-14-00

Option	Advantages	Disadvantages	Risks/Consequences including Relative Costs
Box culvert replacement	<ul style="list-style-type: none"> Minimizes depth of excavation, temporary excavation support and dewatering requirements, shorter duration for dewatering and surface water pumping compared to open footing option. Pre-cast box sections expected to allow for faster construction than cast-in-place open footings. More tolerant of differential settlement than open footing culvert. 	<ul style="list-style-type: none"> Where excavation extends below the groundwater level, dewatering would still be required. Transportation of numerous large box segments to site, including via local roads, is more onerous than local construction of open footing structure. Fisheries requirement may not allow closed box design. 	<ul style="list-style-type: none"> Some risk of disturbance of the subgrade during construction; can be mitigated with appropriate groundwater control and use of a concrete working slab. Limited risk related to settlement performance. Overall faster construction potentially resulting in cost savings. Close proximity to existing open culvert footings and risk of encountering previous “unsuitable” backfill within new foundation footprint Risk of disturbance to founding soils if existing culvert footings are removed. Lower cost relative an open footing culvert replacement option if pre-cast box units used.
Open footing culvert replacement	<ul style="list-style-type: none"> Would satisfy any fisheries requirements related to natural channel substrate, if applicable. Likely founded near existing open footing culvert foundation level; although no details of existing foundations are available. 	<ul style="list-style-type: none"> Excavation depths are greater than for box culvert option, resulting in increased excavation support and dewatering / depressurization requirements. Excavation may penetrate aquifer and artesian groundwater condition Cast-in-place footings likely require a longer duration for construction, and surface water pumping, as compared with pre-cast culvert segments. 	<ul style="list-style-type: none"> Some risk of disturbance of the subgrade during construction; can be mitigated with appropriate groundwater control and use of a concrete working slab. Overall longer construction period potentially resulting in higher costs Risk of disturbance of the footing subgrade due to artesian groundwater conditions. Risk of disturbance to founding soils if existing culvert footings are removed Higher cost than box culvert replacement option due to deeper excavation, additional dewatering, and cast-in-place foundations. Additional costs can be reduced if pre-cast C-sections are used.



FOUNDATION REPORT - REVISION 1 **CULVERT REPLACEMENT / REHABILITATION AT SITE NOS. 22-415/C AND 21-13/C**

TABLE 2 – COMPARISON OF RETAINING WALL ALTERNATIVES
G.W.P. 2248-14-00

Foundation Option	Feasibility		Advantages	Disadvantages	Relative Costs	Risks/Consequences
	Site 22-415C	Site 21-13C				
RSS Walls	Feasible provided sufficient space is available during construction and/or for temporary shoring, if required.	Feasible provided sufficient space is available during construction and/or for temporary shoring if used.	<ul style="list-style-type: none"> • More tolerable to post construction settlements • Lowest cost alternative where feasible • Can found within the competent fill material at both sites given that founding soils have essentially been “preloaded” 	<ul style="list-style-type: none"> • Potentially larger amount of excavation required to install reinforcing strips; temporary protection systems will be required • Potential for deeper excavation if existing wall foundations are lower than proposed new founding level at Site 22-415C • Requires special approval from MTO RSS Committee 	<ul style="list-style-type: none"> • Lower cost than concrete retaining wall or walls supported on deep foundations 	<ul style="list-style-type: none"> • Requires larger excavation footprint to accommodate reinforced earth zone • Can better accommodate some degree of differential settlement, although reinforced soil mass may settle differentially compared to adjacent embankment • Increased risk of soil migration through facing adjacent to active watercourse (if applicable) and will require MTO acceptance for specialized design near watercourse
Concrete Cantilever Wall on Shallow Foundations	Feasible provided sufficient space is available during construction and/or for temporary shoring if used.	Feasible provided sufficient space is available during construction and/or for	<ul style="list-style-type: none"> • Conventional excavation and construction techniques • Allows for watertight tie-in to 	<ul style="list-style-type: none"> • Footings must be founded below depth of frost penetration • Site 21-13C would require deeper excavation (up to 6 m below road surface) 	<ul style="list-style-type: none"> • Higher cost relative to RSS wall 	<ul style="list-style-type: none"> • More susceptible for visible distortion if differential settlement occurs • Presence of artesian groundwater



FOUNDATION REPORT - REVISION 1

CULVERT REPLACEMENT / REHABILITATION AT SITE NOS. 22-415/C AND 21-13/C

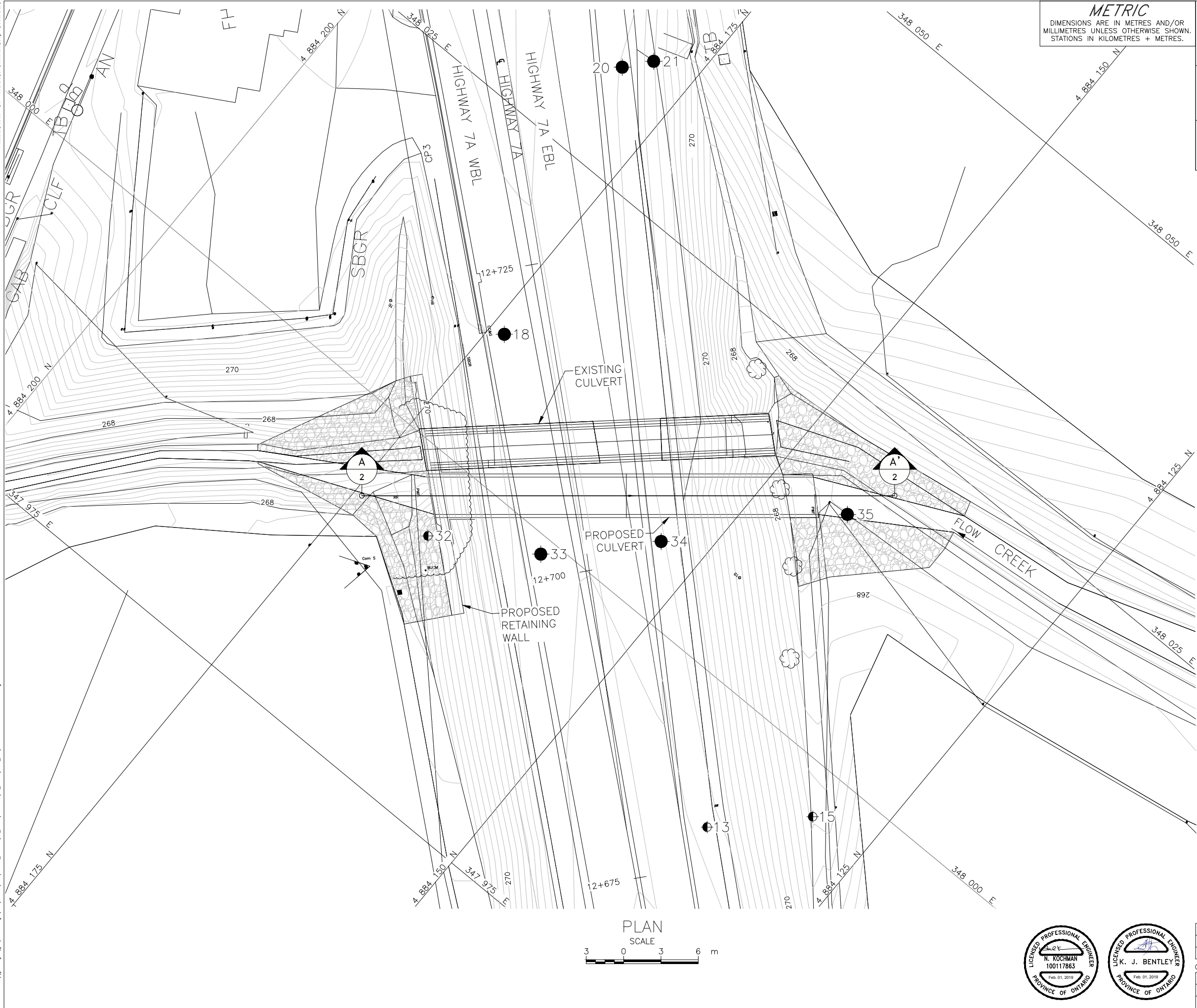
Foundation Option	Feasibility		Advantages	Disadvantages	Relative Costs	Risks/Consequences
	Site 22-415C	Site 21-13C				
		temporary shoring if used.	existing concrete headwall at Site 22-13C and new culvert at Site 21-415C, if preferred	to reach competent native soils to support rigid concrete foundations. <ul style="list-style-type: none"> • Dewatering prior to excavation required • Deeper temporary protection system required • Potential for deeper excavation if existing wall foundations are lower than proposed new founding level at Site 22-415C 		conditions at Site 22-415 may require depressurization and increased dewatering efforts <ul style="list-style-type: none"> • Deeper excavation combined with permeable gravelly founding soils at Site 21-13 may require specialized dewatering measures and higher risk of disturbance to foundations
Soldier Pile and Concrete Panel Wall or similar (e.g. secant caisson wall)	Feasible from a geotechnical perspective; however, greater challenges with constructability in terms of staging, access and space using large caisson rig.	Not considered feasible at this site	<ul style="list-style-type: none"> • Most advantageous in “top-down” construction applications, minimizes excavation and requirement for temporary excavation support • Can be designed to accommodate “unknown” foundation level of existing wall • Temporary protection system not required 	<ul style="list-style-type: none"> • Potential risk of loss of soil particles at gaps between panels/soldier piles • Requires use of liners and fluid control to minimize disturbance and ground loss during formation of soldier pile holes • Potential need for tie-back/anchor installation, with associated testing requirements • Presence of artesian groundwater conditions may create upward groundwater seepage concerns 	<ul style="list-style-type: none"> • Anticipated to be comparable costs to concrete retaining wall, but higher than RSS wall • Cost of temporary protection system combined with RSS wall is comparable 	<ul style="list-style-type: none"> • Lesser excavation than other options, and may eliminate or reduce requirements for temporary protection system • Risks associated with heavier rig equipment accessing and working near floodplain • Risks associated with loss of soil particles at gaps in panels during flood conditions (similar to RSS walls)



FOUNDATION REPORT - REVISION 1

CULVERT REPLACEMENT / REHABILITATION AT SITE NOS. 22-415/C AND 21-13/C

Foundation Option	Feasibility		Advantages	Disadvantages	Relative Costs	Risks/Consequences
	Site 22-415C	Site 21-13C				
Concrete Retaining Wall on Deep Foundations	Not required at this site due to competent soil present at shallow depth	Feasible	<ul style="list-style-type: none"> • Driven piles or caissons could be considered • Reduced excavation, protection system and backfill requirements compared to concrete wall on shallow foundations 	<ul style="list-style-type: none"> • Temporary/permanent liners required to allow for construction of caissons (if considered) • If refusal (100-blow) stratum or obstructions are encountered, can get piles to hang-up, requiring pre-drilling • Low lateral resistance of piles / caissons in existing fill soils may require soil anchors / tie-backs 	<ul style="list-style-type: none"> • Higher cost relative to RSS wall 	<ul style="list-style-type: none"> • If tie-backs are needed, adequate right-of-way and space for tie back anchors installation • Additional staging / widening may be required to accommodate large construction equipment / drill rigs
Reinforced Earth Slope Embankment	Not feasible due to space restrictions.	Not feasible due to space restrictions.	<ul style="list-style-type: none"> • Relative ease of construction but proprietary product required • Vegetated surfaces could be used to improve aesthetics 	<ul style="list-style-type: none"> • Proprietary product design • Special treatment of reinforced earth slope surfaces required to allow vegetation to grow and minimize erosion 	<ul style="list-style-type: none"> • Lowest cost 	<ul style="list-style-type: none"> • Requires wider right-of-way footprint • Can accommodate some degree of differential settlement but susceptible to surface erosion

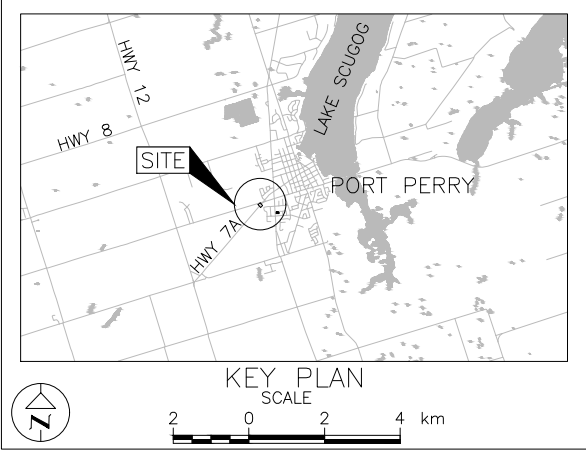


METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No.2248-14-00

HIGHWAY 7A
PORT PERRY CULVERT (SITE NO. 22-415C)
BOREHOLE LOCATIONS

SHEET



LEGEND

Borehole - Current Investigation

Borehole - Pavement Investigation

BOREHOLE CO-ORDINATES (MTM NAD83 ZONE 10)			
No.	ELEVATION	NORTHING	EASTING
13	271.9	4884136.0	347990.3
15	268.8	4884130.0	347996.3
18	271.3	4884173.8	348010.4
20	271.8	4884180.1	348033.0
21	271.7	4884178.5	348035.0
32	269.0	4884168.2	347994.0
33	271.9	4884160.3	347998.7
34	271.9	4884153.5	348005.6
35	267.2	4884143.3	348016.8

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

REFERENCE

Base plans provided in digital format by CIMA+, drawing file nos. site 22-415C highway 7A.DWG, received July 25, 2017.

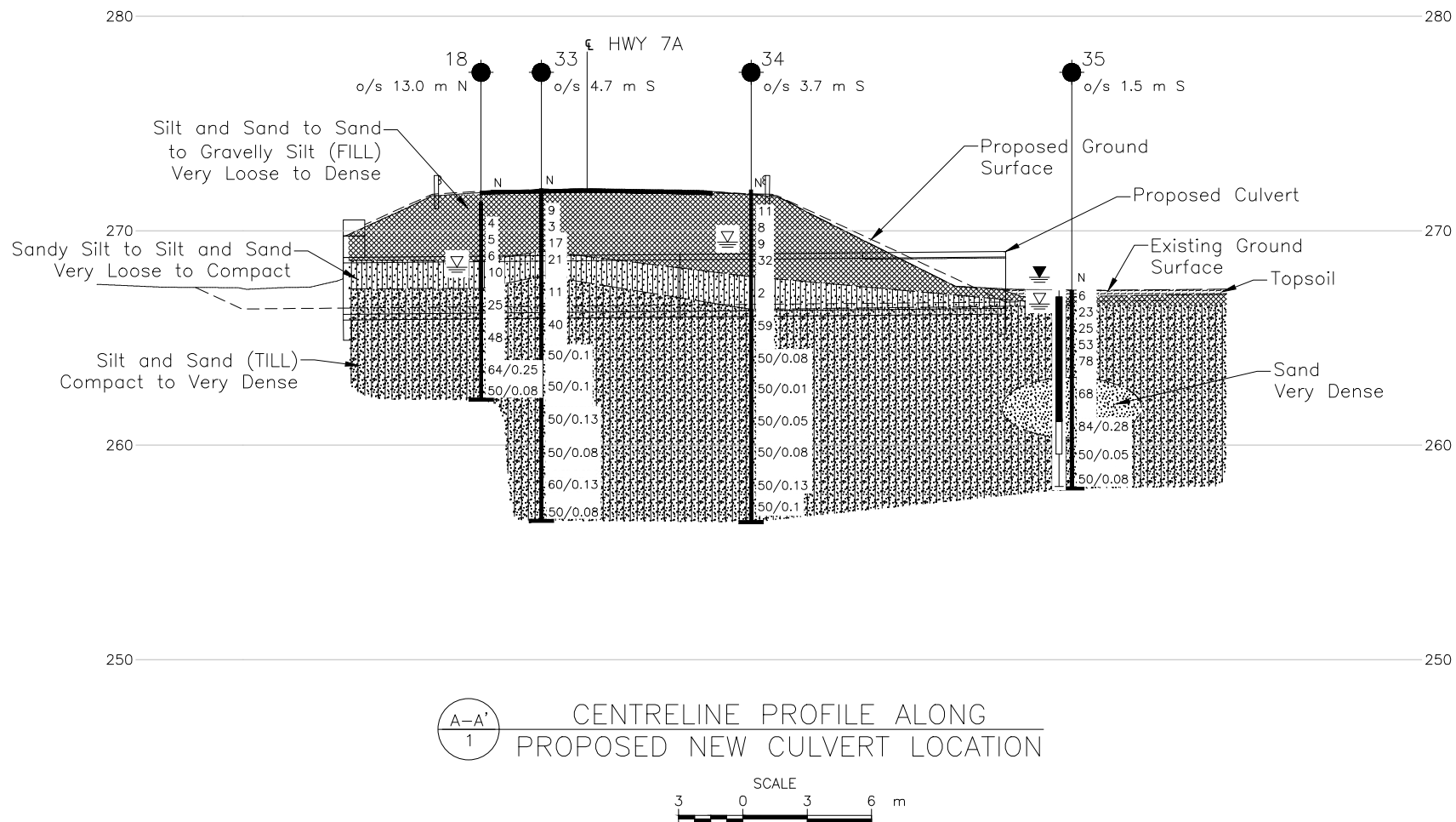


NO.	DATE	BY	REVISION
Geocres No. 31D-706			
HWY. 7A	PROJECT NO. 1533653		DIST. .
SUBM'D.	CHKD.	DATE: 2/1/2019	SITE: 22-415C
DRAWN: TR	CHKD. NK	APPD. KJB	DWG. 1








CONT No.
GWP No.2248-14-00

HIGHWAY 7A
PORT PERRY CULVERT (SITE NO. 22-415C)
SOIL STRATA

SHEET



LEGEND

-  Borehole — Current Investigation
 Seal
 Piezometer
 Standard Penetration Test Value
 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
 WL in piezometer, measured on AUG 24, 2017
 WL upon completion of drilling

BOREHOLE CO-ORDINATES (MTM NAD83 ZONE 10)

No.	ELEVATION	NORTHING	EASTING
18	271.3	4884173.8	348010.4
33	271.9	4884160.3	347998.7
34	271.9	4884153.5	348005.6
35	267.2	4884143.3	348016.8

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

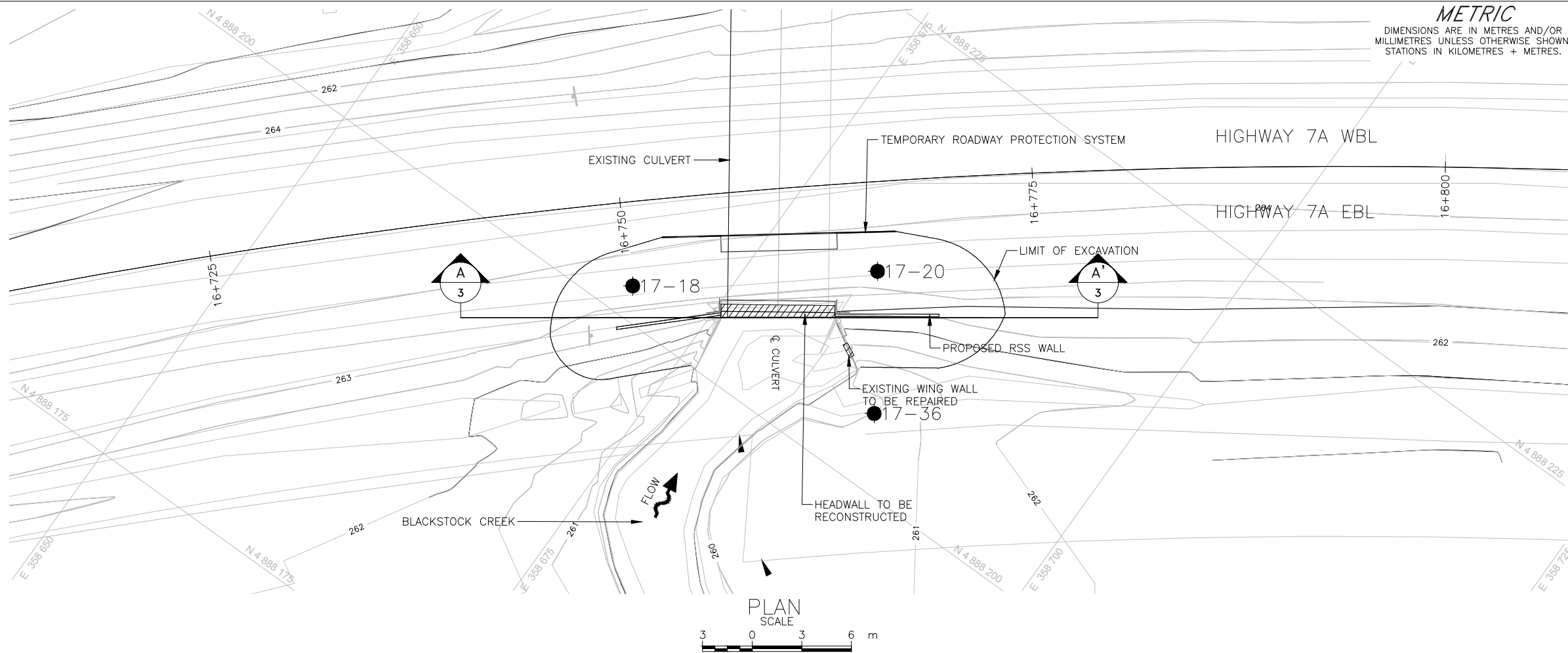
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

REFERENCE

Base plans provided in digital format by CIMA+, drawing file nos. site 22-415C highway 7A.DWG, received July 25, 2017.

NO.	DATE	BY	REVISION
Geocres No. 31D-706			
HWY. 7A		PROJECT NO.	DIST.
SUBM'D.	CHKD.	DATE: 2/1/2019	SITE: 22-415C
DRAWN: TR	CHKD. NK	APPD. KJB	DWG. 2



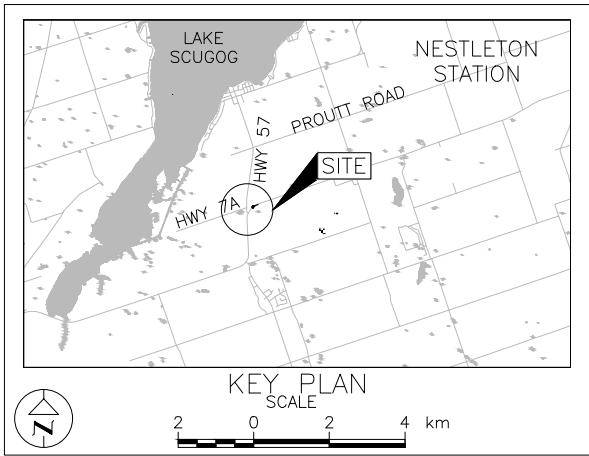


METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 2248-14-00

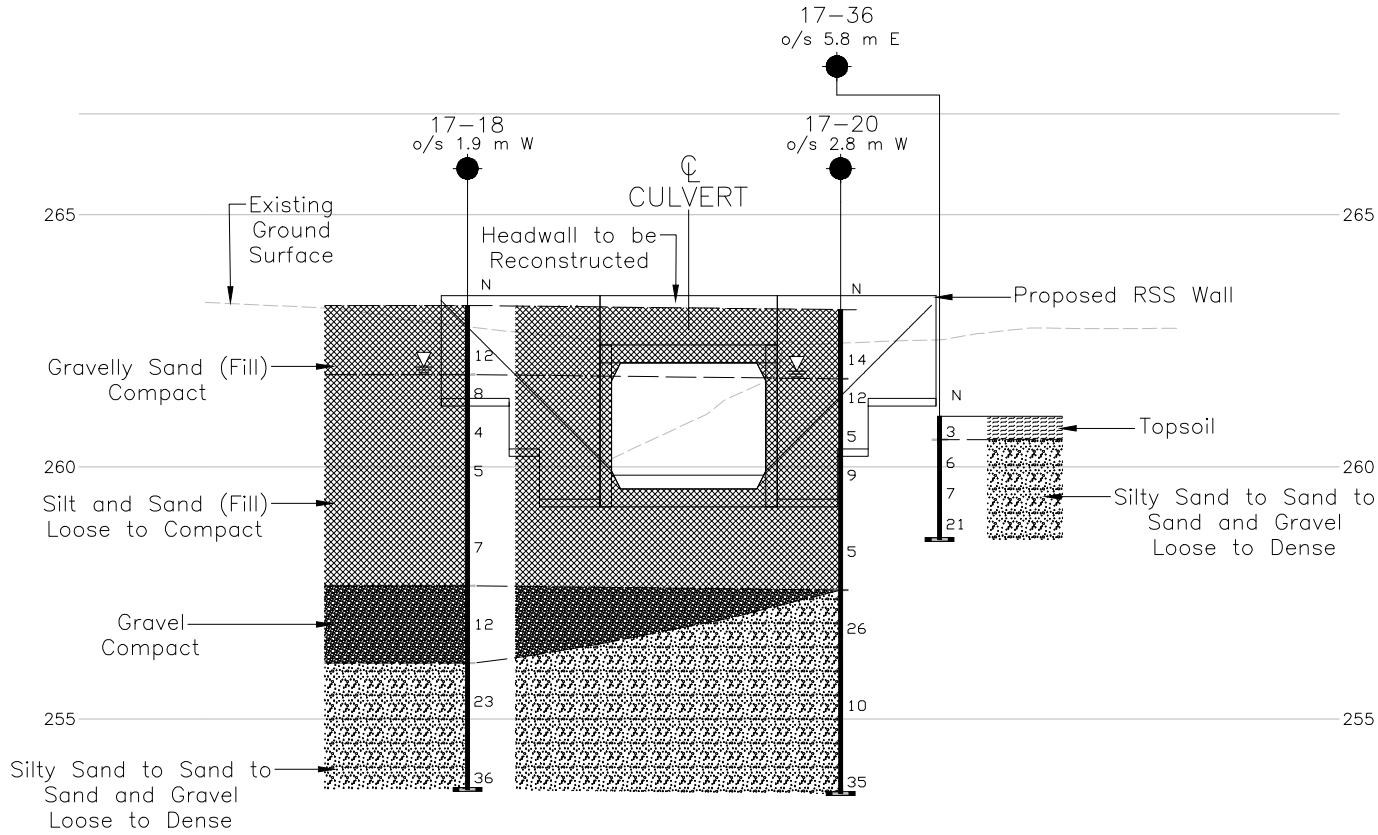
HIGHWAY 7A
BLACKSTOCK CULVERT (SITE NO. 21-13C)
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



LEGEND	
	Borehole - Current Investigation
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
	WL upon completion of drilling

BOREHOLE CO-ORDINATES (MTM NAD83 ZONE 10)			
No.	ELEVATION	NORTHING	EASTING
17-18	263.2	4888201.9	358670.1
17-20	263.1	4888211.3	358681.6
17-36	261.0	4888204.2	358686.5



A-A
3
PROFILE ALONG PROPOSED HEADWALL / RETAINING WALL

NOTES
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

REFERENCE
Base plans provided in digital format by client, drawing file no. "GA_X-21-13c S1.dwg, and GA_X-21-13c S2.dwg, and GA_X-21-13c S3.dwg", dated Feb. 27, 2017, received Feb. 2018.

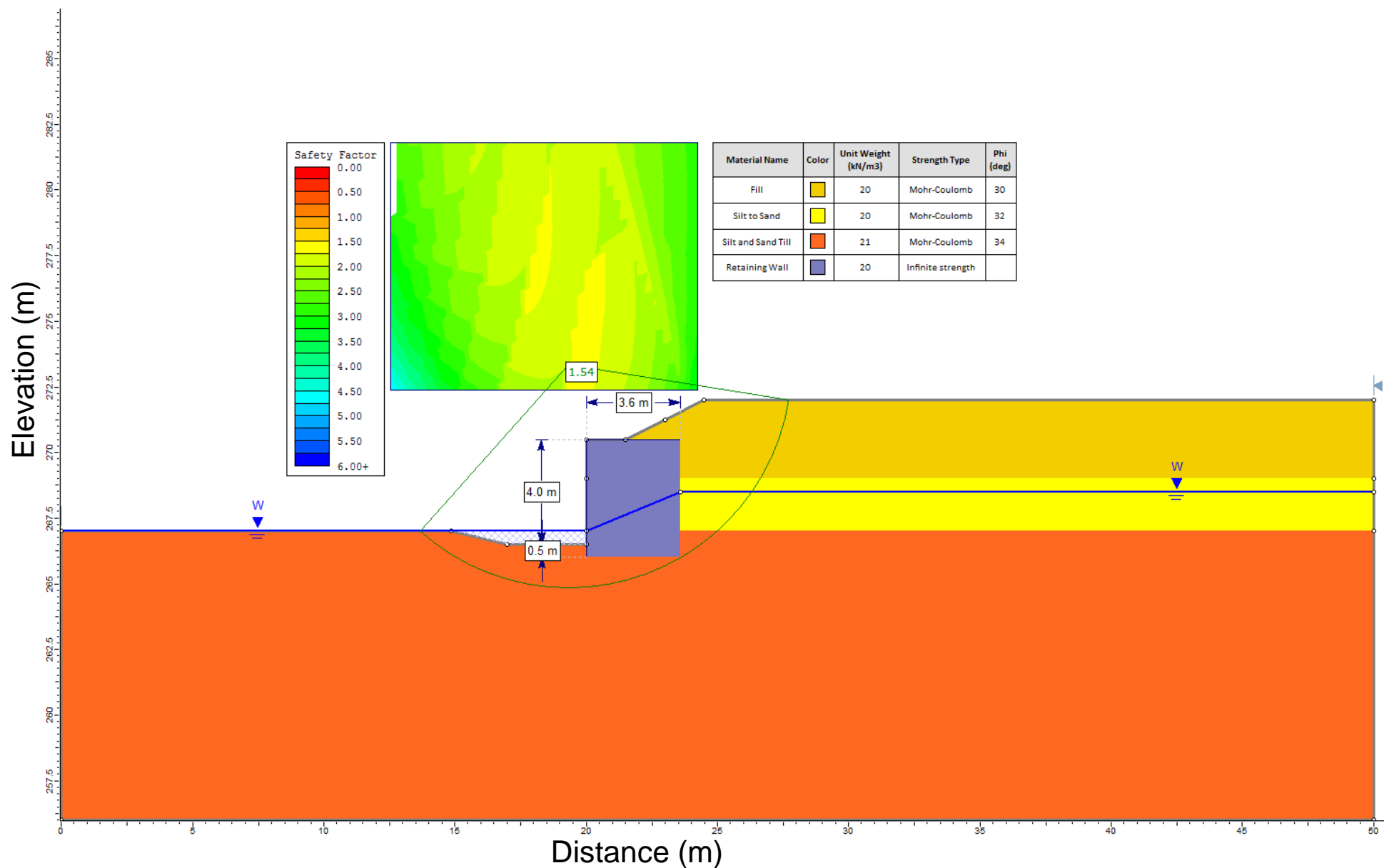


NO.	DATE	BY	REVISION
Geocres No. 31D-706			
HWY. 7A	PROJECT NO. 1533653		DIST. .
SUBM'D. NK	CHKD. NK	DATE: 2/1/2019	SITE: 21-13C
DRAWN: SMD/DD	CHKD. KJB	APPD. KJB	DWG. 3



**STATIC GLOBAL STABILITY
RETAINED SOIL SYSTEM WALL (END WALL)
SITE 22-415C**

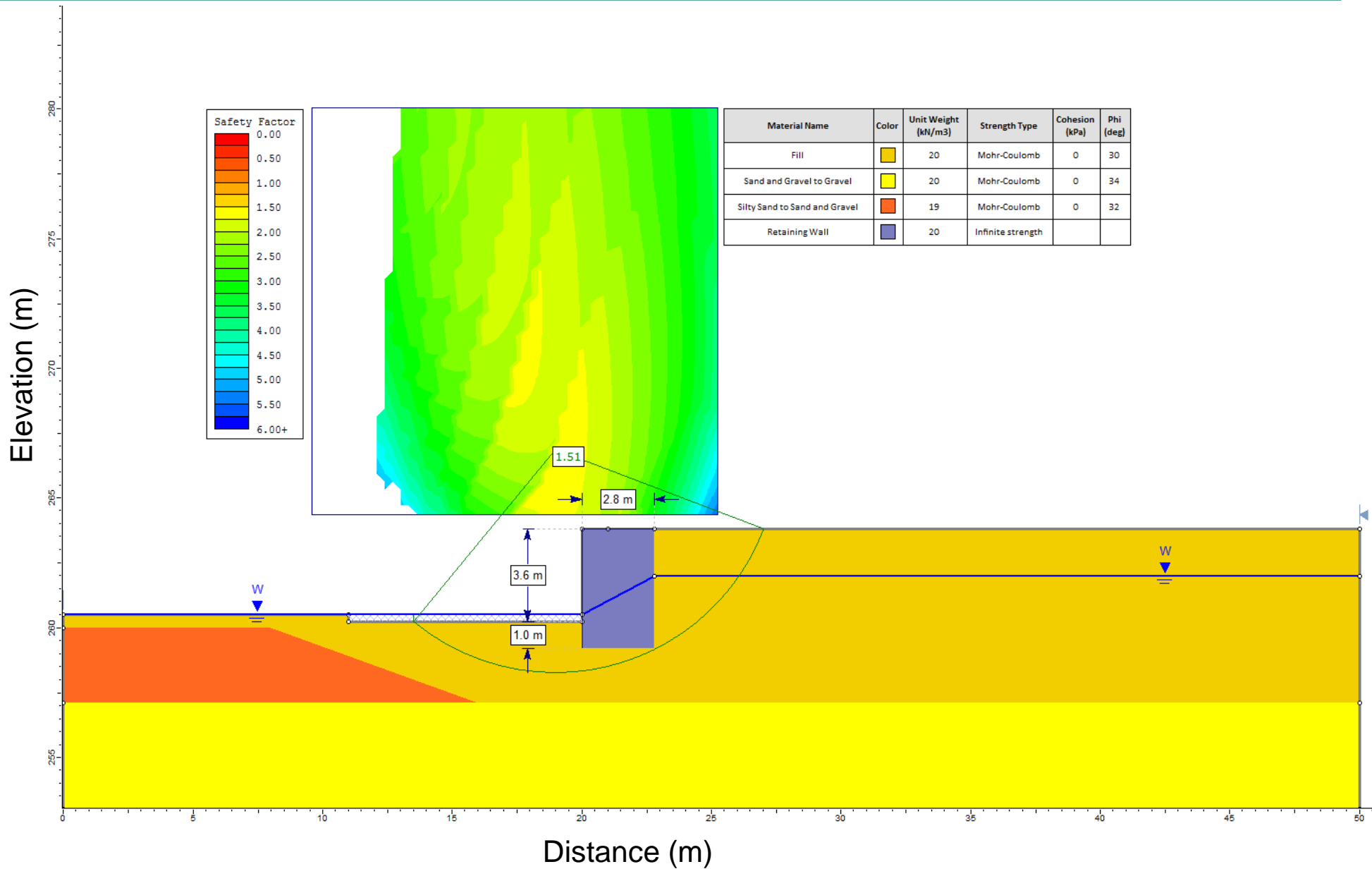
Figure 1





STATIC GLOBAL STABILITY
RETAINED SOIL SYSTEM WALL
SITE 21-13C

Figure 2





APPENDIX A

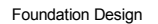
Site 22-415/C

Record of Borehole Sheets
Laboratory Test Results

PROJECT 1533653		RECORD OF BOREHOLE No 18				SHEET 1 OF 1		METRIC								
G.W.P. 2248-14-00		LOCATION N 4884173.8; E 348010.4 MTM NAD 83 ZONE 10 (LAT. 44.096291; LONG. -78.960334)				ORIGINATED BY LP										
DIST Central HWY 7A		BOREHOLE TYPE 210 mm Diameter Hollow Stem Power Augers				COMPILED BY AMP										
DATUM Geodetic		DATE July 13, 2017				CHECKED BY KJB										
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								
271.3	GROUND SURFACE															
0.0	Crushed granular (FILL) Brown															
270.9	Sand, fine to medium, some gravel, trace silt (FILL) Brown Moist		1	SS	4											
270.5	Silt and sand, trace to some clay, trace gravel, trace organic inclusions (FILL) Loose Brown and black Moist to wet		2	SS	5											
268.6			3	SS	6											
2.7	Sandy SILT, trace to some clay, trace gravel, oxidation staining Compact Brown Wet		4	SS	10											
267.3																
4.0	SILT and SAND, trace gravel, trace clay (TILL) Compact to very dense Brown to grey Wet to moist		5	SS	25											
			6	SS	48											
			7	SS	64/0.25											
262.1			8	SS	50/0.08											
9.2	END OF BOREHOLE															
Notes: 1. Water level measured at a depth of about 3.05 m below ground surface upon completion of drilling.																

PROJECT 1533653		RECORD OF BOREHOLE No 20		SHEET 1 OF 1		METRIC																									
G.W.P. 2248-14-00		LOCATION N 4884180.1; E 348033.0 MTM NAD 83 ZONE 10 (LAT. 44.096347; LONG. -78.960052)		ORIGINATED BY LP																											
DIST Central HWY 7A		BOREHOLE TYPE 210 mm Diameter Hollow Stem Power Augers		COMPILED BY AMP																											
DATUM Geodetic		DATE July 13, 2017		CHECKED BY KJB																											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)																
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%) W _p W W _L			γ			GR SA SI CL												
271.8	GROUND SURFACE							20 40 60 80 100																							
0.0	ASPHALT																														
0.4	Crushed granular (FILL) Brown																														
270.7	Sand, fine to medium, with gravel, trace silt (FILL) Brown																														
1.1	Moist																														
270.3	Silty clay, with gravel, trace sand (FILL) Grey		1	SS	6																										
1.5	Moist																														
269.7	Silt and sand, trace to some gravel, trace to some clay, trace rootlets (FILL) Loose Dark brown Moist		2	SS	32																										
2.1	SILT and SAND, trace to some gravel, trace to some clay, oxidation staining (TILL) Dense to very dense Brown Moist		3	SS	40																										
			4	SS	59																										
266.2	SAND, trace to some silt, trace gravel, trace clay Compact Brown Wet		5	SS	19																										
5.6																															
264.7	SILT and SAND, trace to some gravel, trace clay (TILL) Very dense Brown to grey Moist		6	SS	79																										
7.1																															
262.2	END OF BOREHOLE		7	SS	76																										
9.6	Notes: 1. Borehole caved to a depth of about 5.18 m below ground surface upon completion of drilling.																														

PROJECT 1533653		RECORD OF BOREHOLE No 21				SHEET 1 OF 1		METRIC									
G.W.P. 2248-14-00		LOCATION N 4884178.5; E 348035.0 MTM NAD 83 ZONE 10 (LAT. 44.096332; LONG. -78.960027)				ORIGINATED BY LP											
DIST Central HWY 7A		BOREHOLE TYPE 210 mm Diameter Hollow Stem Power Augers				COMPILED BY AMP											
DATUM Geodetic		DATE July 11, 2017				CHECKED BY KJB											
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									
271.7	GROUND SURFACE																
0.0	Gravelly sand (FILL) Compact Brown Moist																
270.6			1A	SS	16												
1.1	Silt and sand, trace to some gravel, trace to some clay (FILL) Very loose to compact Black to brown Moist to wet		1B														
			2	SS	3												
			3	SS	20												
268.8																	
2.9	SILT and SAND, trace to some gravel, trace to some clay (TILL) Dense to very dense Brown to grey Moist		4	SS	49												
			5	SS	44												
			6	SS	41												
			7	SS	96/0.23												
	- Becoming grey at a depth of about 8.6 m																
262.1	- 50 mm sand seam at a depth of about 9.2 m		8	SS	53												
9.6	END OF BOREHOLE																
	Note: 1. Borehole caved to a depth of about 2.4 m below ground surface upon completion of drilling.																



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+³, ×³: Numbers refer to Sensitivity ○^{3%} STRAIN AT FAILURE

PROJECT		1533653		RECORD OF BOREHOLE No 33				SHEET 2 OF 2		METRIC							
G.W.P.		2248-14-00		LOCATION		N 4884160.3; E 347998.7 MTM NAD 83 ZONE 10 (LAT. 44.096170; LONG. -78.960482)				ORIGINATED BY LP							
DIST		Central HWY 7A		BOREHOLE TYPE		210 mm Diameter Hollow Stem Power Augers				COMPILED BY AMP							
DATUM		Geodetic		DATE		July 11, 2017				CHECKED BY KJB							
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									
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100					
256.4	END OF BOREHOLE		12	SS	50/0.08												2 33 59 6
15.5																	

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Continued Next Page

+³, ×³: Numbers refer to Sensitivity ○^{3%} STRAIN AT FAILURE

PROJECT		1533653		RECORD OF BOREHOLE No 34				SHEET 2 OF 2		METRIC							
G.W.P.		2248-14-00		LOCATION		N 4884153.5; E 348005.6 MTM NAD 83 ZONE 10 (LAT. 44.096109; LONG. -78.960396)				ORIGINATED BY LP							
DIST		Central HWY 7A		BOREHOLE TYPE		210 mm Diameter Hollow Stem Power Augers				COMPILED BY AMP							
DATUM		Geodetic		DATE		July 10, 2017				CHECKED BY KJB							
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									
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100					
256.4	END OF BOREHOLE		12	SS	50/0.1												
15.5	Notes: 1. Water level measured at a depth of 2.44 m below ground surface upon completion of drilling.																

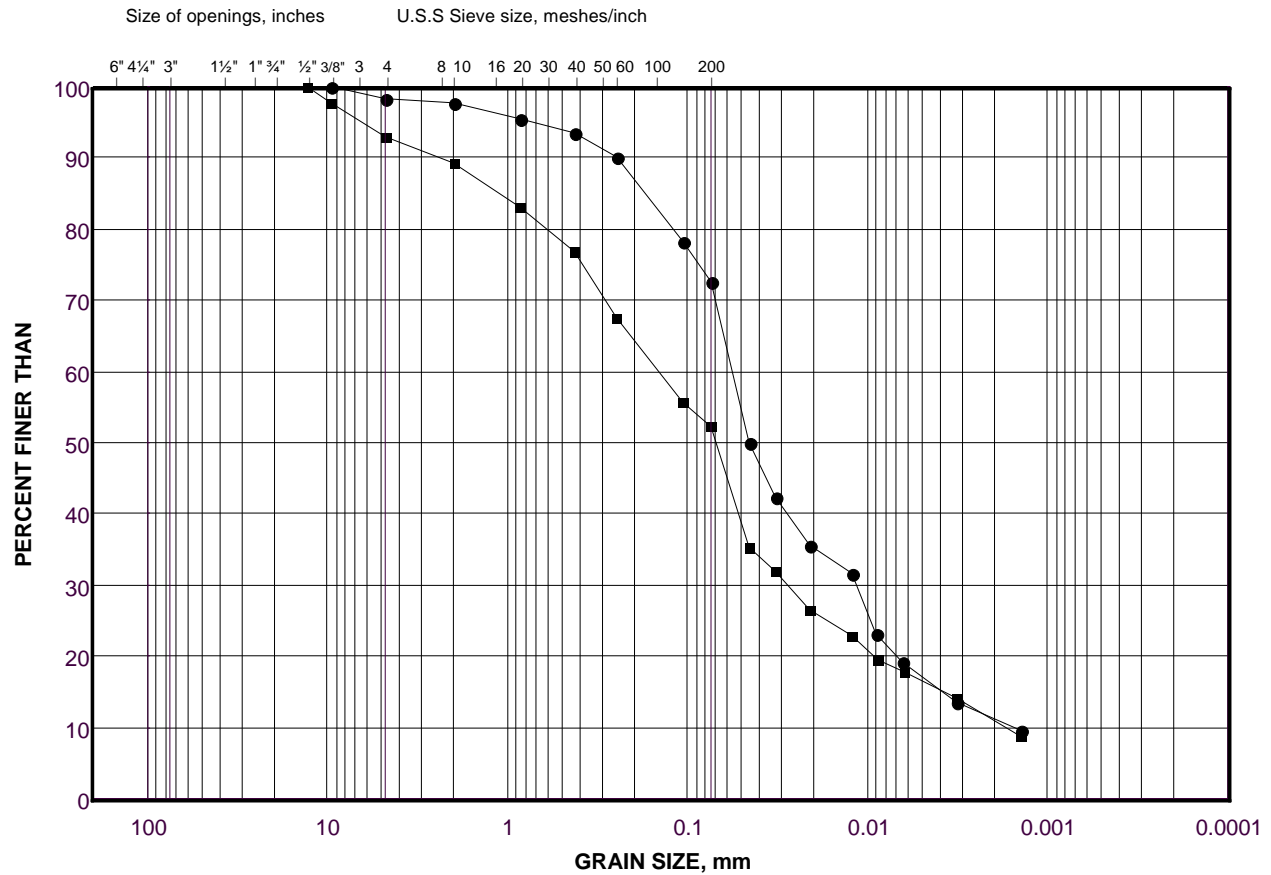
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PROJECT 1533653		RECORD OF BOREHOLE No 35		SHEET 1 OF 1		METRIC															
G.W.P. 2248-14-00		LOCATION N 4884143.3; E 348016.8 MTM NAD 83 ZONE 10 (LAT. 44.096016; LONG. -78.960257)		ORIGINATED BY JS																	
DIST Central HWY 7A		BOREHOLE TYPE 210 mm Diameter Hollow Stem Power Augers		COMPILED BY AMP																	
DATUM Geodetic		DATE July 20, 2017		CHECKED BY KJB																	
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ			GR SA SI CL		
267.2	GROUND SURFACE							20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p — W — W _L 10 20 30			kN/m ³					
0.0	TOPSOIL		1	SS	6		267														
0.3	Silt and sand, trace to some clay, trace gravel, organic inclusions (FILL)		2	SS	23		266														
0.7	Loose Dark brown Moist		3	SS	25		265														
	SILT and SAND, trace gravel, trace clay, some oxidation staining (TILL)		4	SS	53		264														
	Compact to very dense Brown Moist		5	SS	78		263														
263.2	SAND, trace to some silt, trace gravel		6	SS	68		262														
4.0	Very dense Brown Wet		7	SS	84/0.28		261														
260.3	SILT and SAND, trace to some clay, trace gravel (TILL)		8	SS	50/0.06		260														
6.9	Very dense Grey Moist		9	SS	50/0.08		259														
258.0	END OF BOREHOLE						258														
9.2	Notes: 1. Water level measured at a depth of about 4.27 m below ground surface upon completion of drilling. 2. Water level measured in piezometer at 0.63 m above ground surface on August 24, 2017.																				

GRAIN SIZE DISTRIBUTION

Sandy Silt to Silt and Sand

FIGURE A-1



COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
	GRAVEL SIZE		SAND SIZE			FINE GRAINED

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	18	4	268.3
■	34	5	267.3

Project Number: 1533653

Checked By: KJB

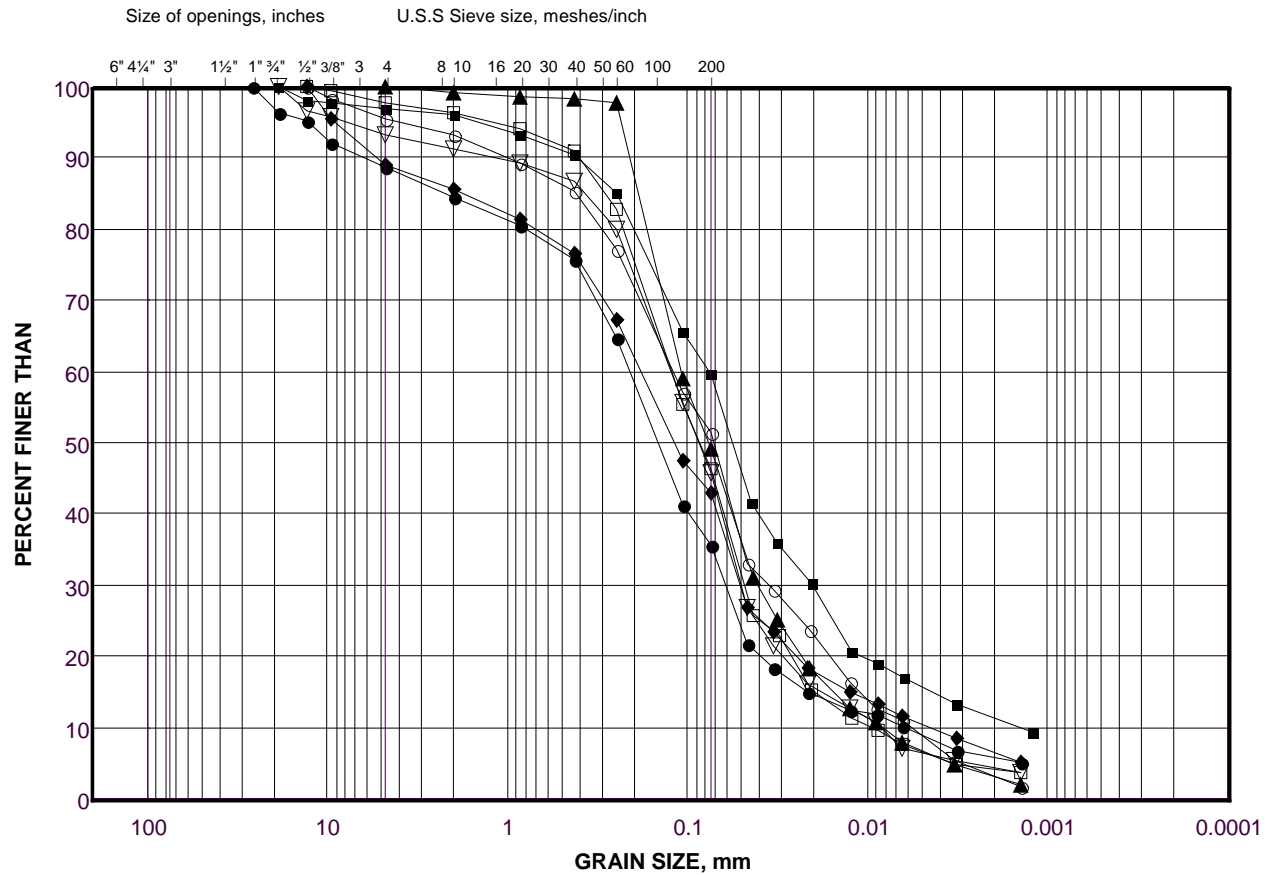
Golder Associates

Date: 01-Jun-18

GRAIN SIZE DISTRIBUTION

Silt and Sand Till

FIGURE A-2A



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	20	2	269.5
■	20	4	267.2
◆	21	4	268.7
▲	18	6	265.2
▽	20	6	264.2
○	18	8	262.2
□	21	8	262.6

Project Number: 1533653

Checked By: KJB

Golder Associates

Date: 01-Jun-18

Silt and Sand Till

FIGURE A-2B



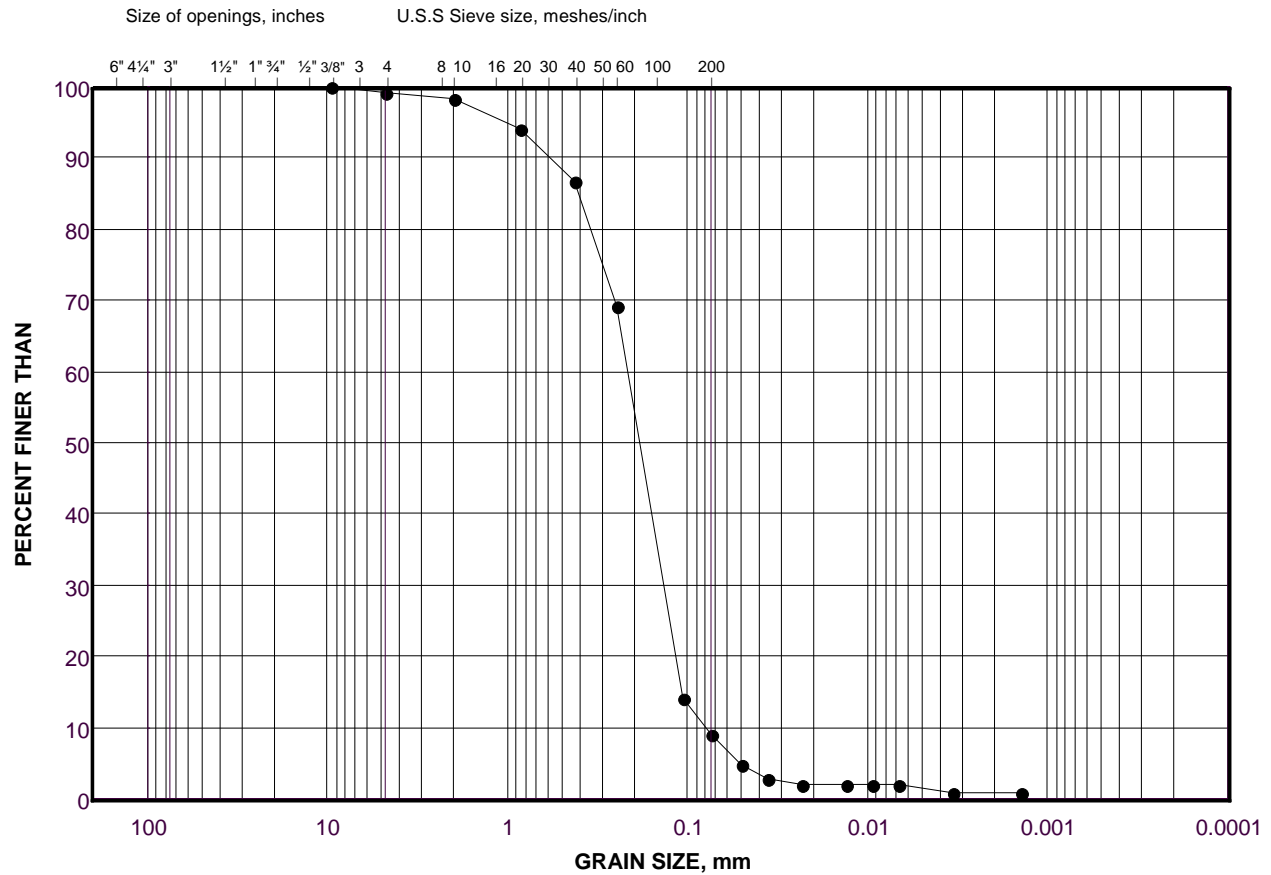
SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	34	11	258.2
■	33	12	256.7
◆	35	4	264.9
▲	33	7	264.3
▽	35	8	259.6
○	34	8	262.8

Date: 01-Jun-18

GRAIN SIZE DISTRIBUTION

Sand

FIGURE A-3



COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
	GRAVEL SIZE		SAND SIZE			FINE GRAINED

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	17-20	5	265.7

Project Number: 1533653

Checked By: KJB

Golder Associates

Date: 01-Jun-18



APPENDIX B

Site 21-13/C

Record of Borehole Sheets
Laboratory Test Results

PROJECT 1533653		RECORD OF BOREHOLE No 17-18		SHEET 1 OF 1		METRIC														
G.W.P. 2248-14-00		LOCATION N 4888201.9; E 358670.1 MTM NAD 83 ZONE 10 (LAT. 44.131838; LONG. -78.826799)		ORIGINATED BY JS																
DIST Central HWY 7A		BOREHOLE TYPE 210 mm Diameter Hollow Stem Power Augers		COMPILED BY AMP																
DATUM Geodetic		DATE July 5, 2017		CHECKED BY KJB																
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ	GR SA SI CL			
							20 40 60 80 100	20 40 60 80 100	W _p	W	W _L	10 20 30								
263.2	GROUND SURFACE						263													
0.0	Gravelly sand, trace silt (FILL) Compact Brown Moist to wet		1	SS	12		262													
261.8	Silt and sand, trace to some clay, trace gravel (FILL) Brown to grey Loose Moist to wet - Wet at 2.3 m depth		2	SS	8		261													
1.4			3	SS	4		260													
			4	SS	5		259													
			5	SS	7		258													
257.6	- Auger grinding at 5.2 m depth						257													
5.6	Gravel, trace to some sand, trace to some silt Compact Brown Wet		6	SS	12		256													
256.1	Sand, trace to some silt, trace gravel Compact to dense Brown Wet		7	SS	23		255													
7.1							254													
253.6	END OF BOREHOLE		8	SS	36															
9.6	Notes: 1. Borehole caved to a depth of about 0.61 m below ground surface upon completion of drilling. 2. Water level measured at a depth of about 1.22 m below ground surface upon completion of drilling.																			

PROJECT 1533653		RECORD OF BOREHOLE No 17-20		SHEET 1 OF 1		METRIC														
G.W.P. 2248-14-00		LOCATION N 4888211.3; E 358681.6 MTM NAD 83 ZONE 10 (LAT. 44.131921; LONG. -78.826654)		ORIGINATED BY JS																
DIST Central HWY 7A		BOREHOLE TYPE 210 mm Diameter Hollow Stem Power Augers		COMPILED BY AMP																
DATUM Geodetic		DATE July 5, 2017		CHECKED BY KJB																
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ	OC=6.1%	GR SA SI CL		
							20 40 60 80 100	○ UNCONFINED + FIELD VANE	● QUICK TRIAXIAL × REMOULDED	W _p	W	W _L	20 40 60 80 100	10 20 30						
263.1	GROUND SURFACE						263													
0.0	Gravelly sand, trace to some silt (FILL) Compact Brown Moist		1	SS	14		262													
261.7	Silt and sand, trace to some clay, trace gravel (FILL) Loose to compact Brown Moist to wet		2	SS	12		261													
1.4			3	SS	5		260													
			4	SS	9		259													
	- Auger grinding at 3.7 m depth						258													
	- Trace brick fragments, organic inclusions		5	SS	5		257													
257.5	SAND and GRAVEL, trace to some silt, trace clay Compact to dense Grey Wet		6	SS	26		256													
5.6			7	SS	10		255													
			8	SS	35		254													
253.5	END OF BOREHOLE																			
9.6	Notes: 1. Borehole caved to a depth of about 4.27 m below ground surface upon completion of drilling. 2. Water level measured at a depth of about 1.22 m below ground surface upon completion of drilling.																			

PROJECT <u>1533653</u>			RECORD OF BOREHOLE No 17-36			SHEET 1 OF 1			METRIC		
G.W.P. <u>2248-14-00</u>			LOCATION <u>N 4888204.2; E 358686.5 MTM NAD 83 ZONE 10 (LAT. 44.131857; LONG. -78.826594)</u>			ORIGINATED BY <u>JS</u>					
DIST <u>Central</u> HWY <u>7A</u>			BOREHOLE TYPE <u>Portable Equipment Using Continuous SPT</u>			COMPILED BY <u>NK</u>					
DATUM <u>Geodetic</u>			DATE <u>August 17, 2017</u>			CHECKED BY <u>KJB</u>					

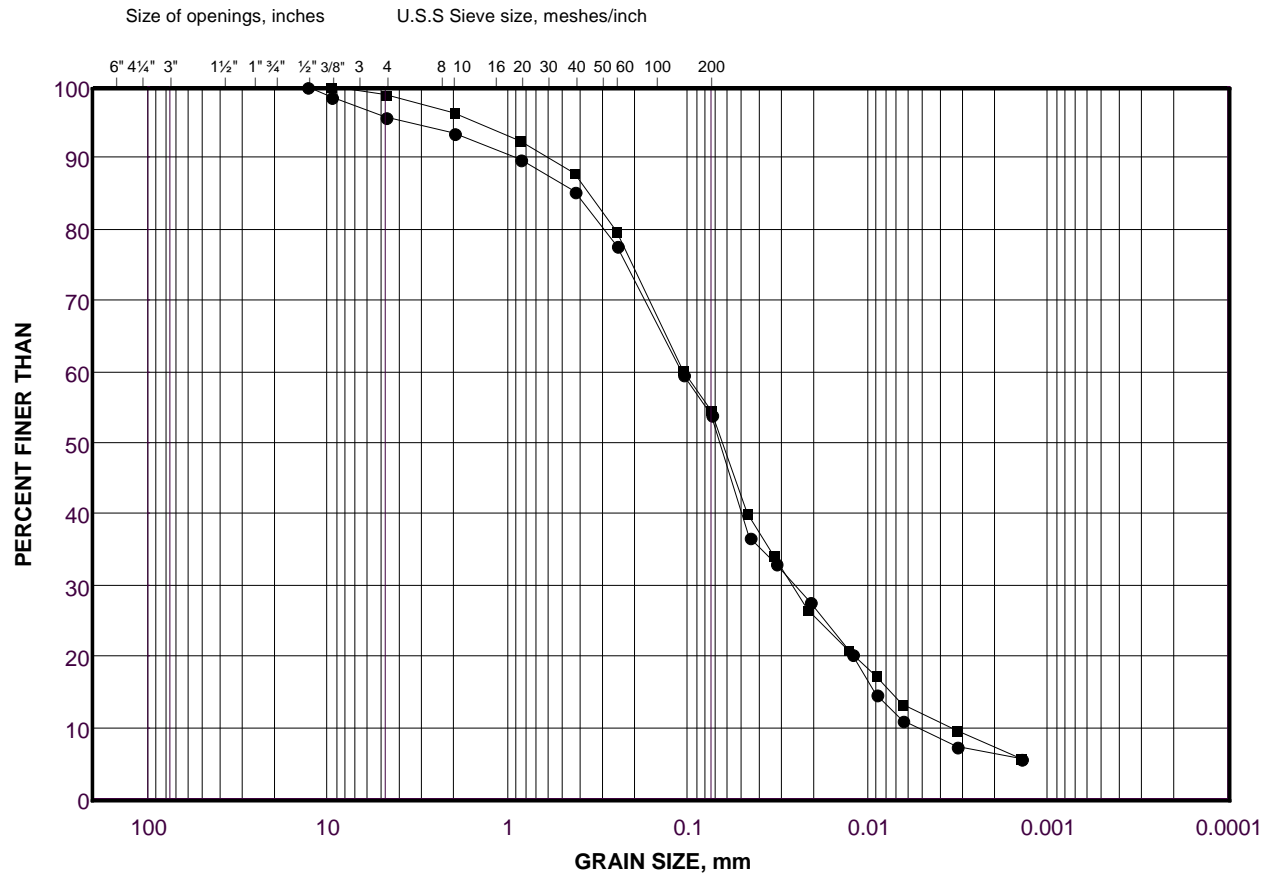
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W _p	W	W _L		
							20	40	60	80	100						
261.0	GROUND SURFACE																
0.0	TOPSOIL		1	SS	3												
260.5	Silty SAND, trace clay, trace to some organics Loose to compact below 1.83 m Blackish brown Moist to wet below 1.83 m		2	SS	6												
0.5			3	SS	7												
			4	SS	21												
258.6																	
2.4	END OF BOREHOLE																
	Notes: 1. Borehole dry upon completion of drilling. 2. SPT "N" values have been corrected to account for half-weight hammer used to drive split-spoon sampler.																

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GRAIN SIZE DISTRIBUTION

Silt and Sand Fill

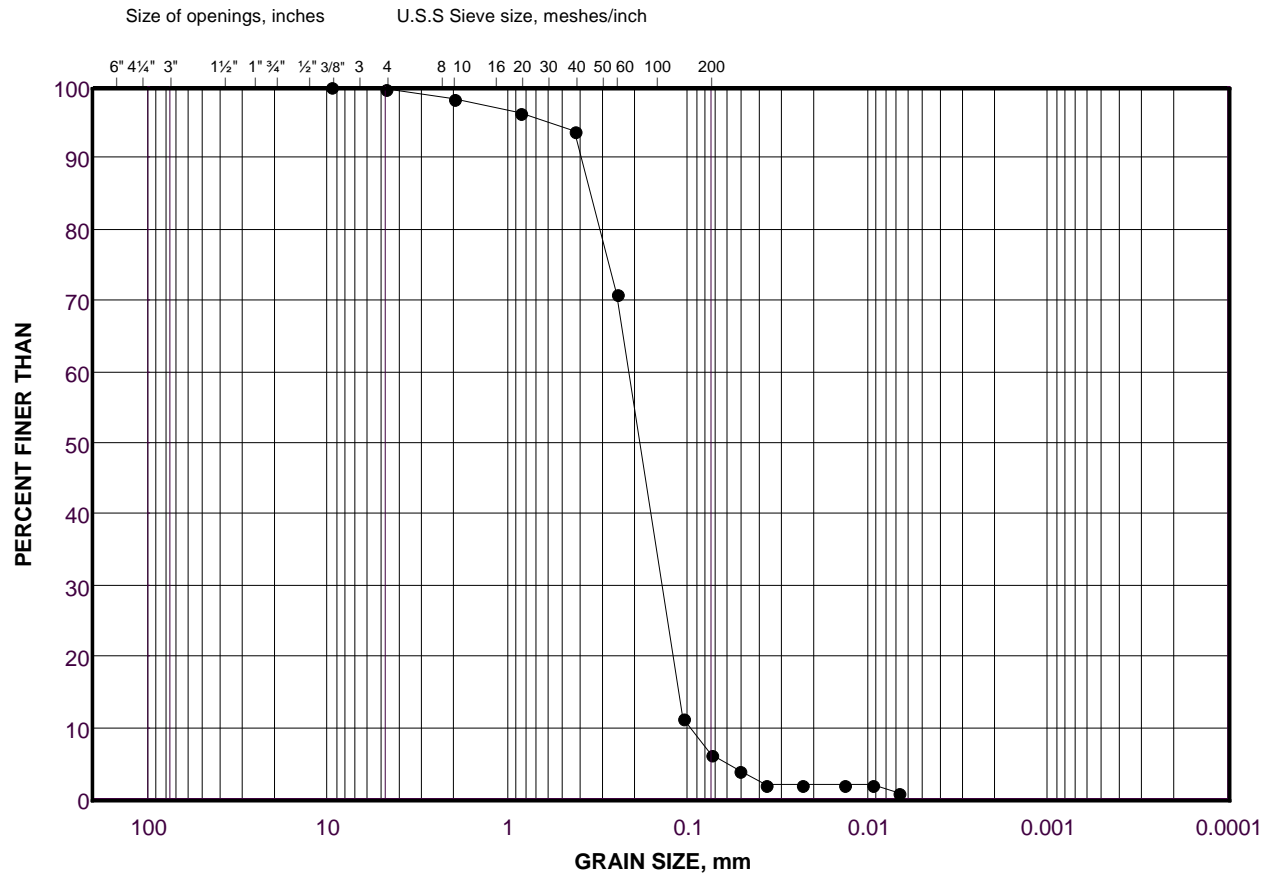
FIGURE B-1



GRAIN SIZE DISTRIBUTION

Sand

FIGURE B-2



COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
	GRAVEL SIZE		SAND SIZE			FINE GRAINED

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	17-18	7	255.5

Project Number: 1533653

Checked By: KJB

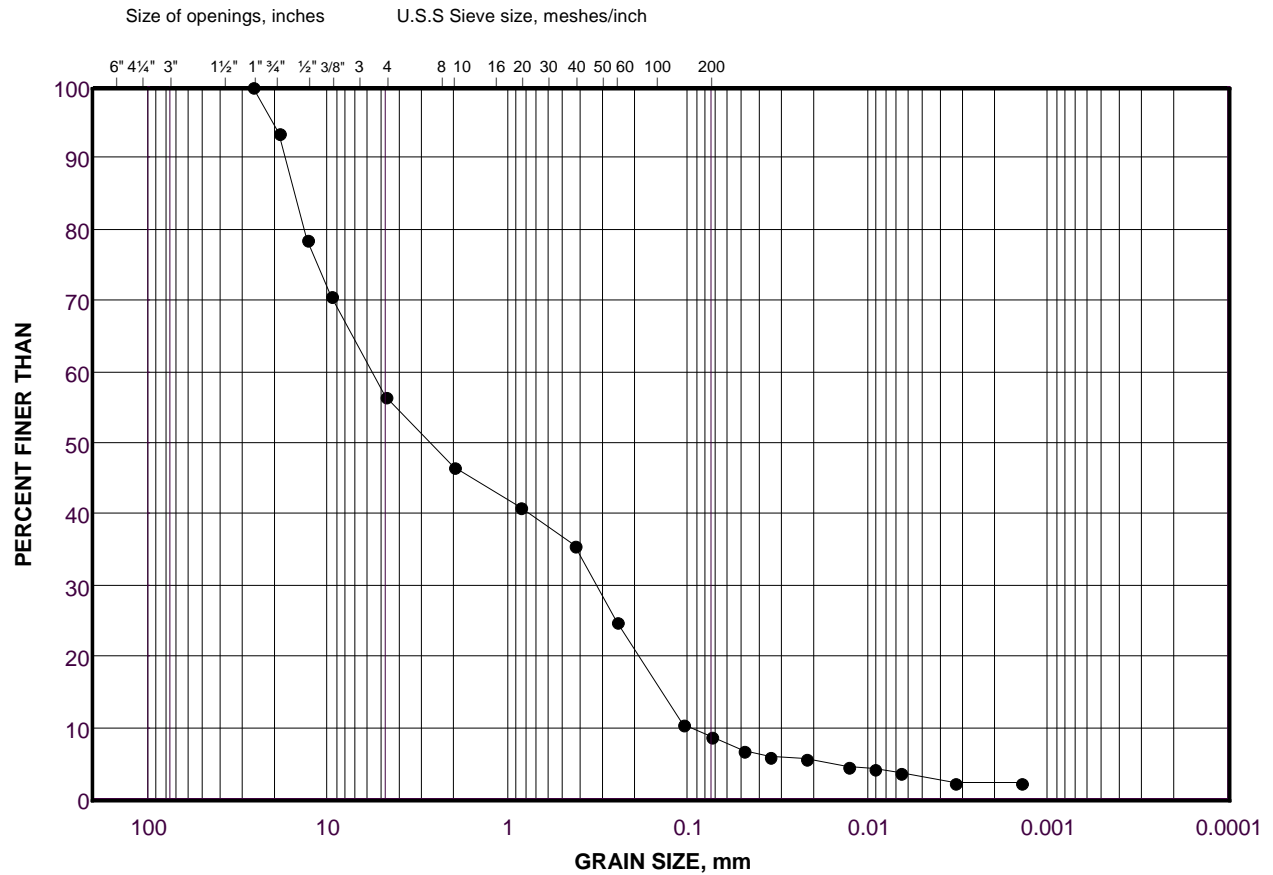
Golder Associates

Date: 01-Jun-18

GRAIN SIZE DISTRIBUTION

Sand and Gravel

FIGURE B-3



COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
	GRAVEL SIZE		SAND SIZE			FINE GRAINED

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	17-20	6	257.0

Project Number: 1533653

Checked By: KJB

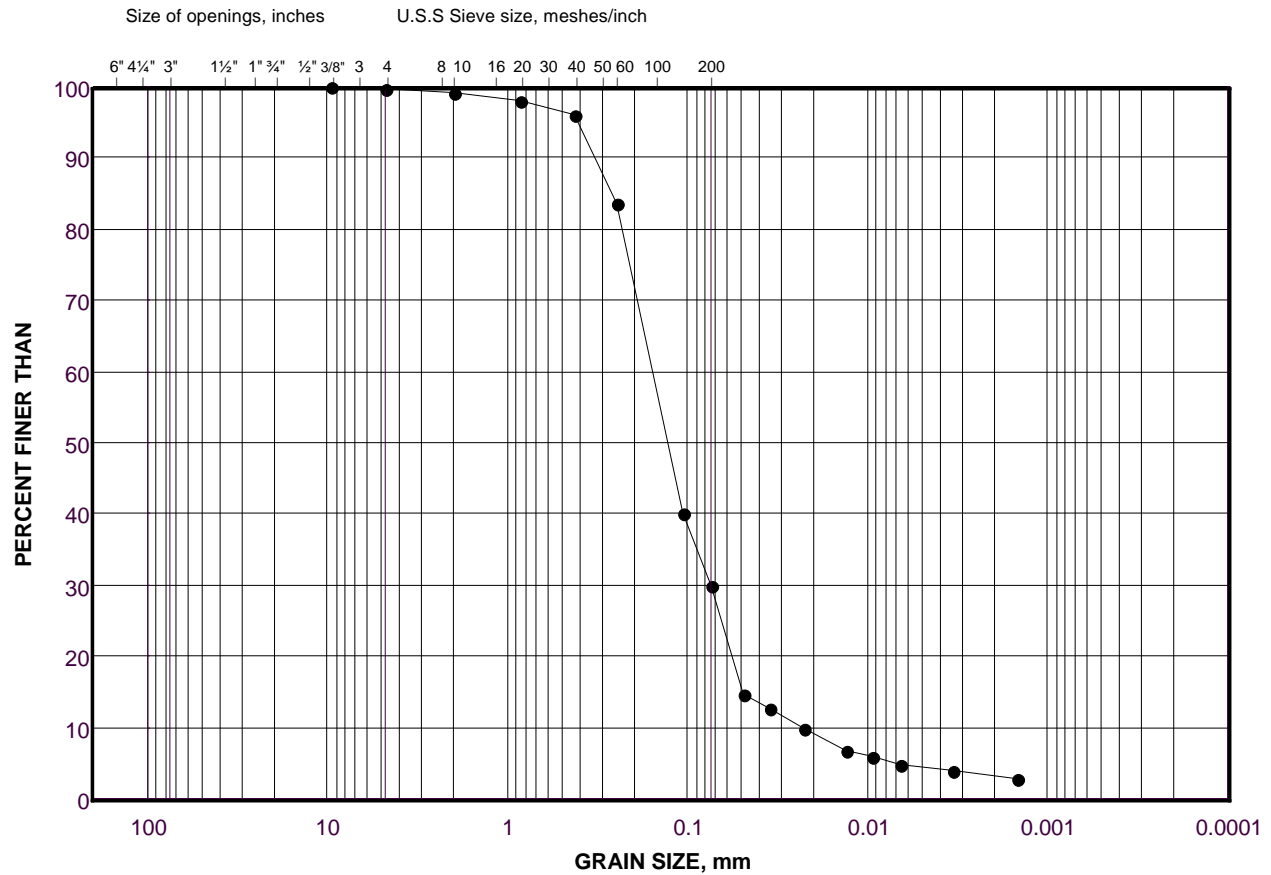
Golder Associates

Date: 01-Jun-18

GRAIN SIZE DISTRIBUTION

Silty Sand

FIGURE B-4



COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
	GRAVEL SIZE		SAND SIZE			FINE GRAINED

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	17-36	3	257.9

Project Number: 1533653

Checked By: KJB

Golder Associates

Date: 01-Jun-18



APPENDIX C

Non-Standard Special Provisions

DEWATERING STRUCTURE EXCAVATIONS – Item No.

Special Provision No. FOUN0003

Amendment to OPSS 902, November 2010

902.02 REFERENCES

Section 902.02 of OPSS 902 is amended by the addition of the following:

Ontario Provincial Standard Specifications, Construction

OPSS 517 Dewatering
OPSS 805 Temporary Erosion and Sediment Control Measures

902.03 DEFINITIONS

Section 903.03 of OPSS 902 is amended by the addition of the following:

Automatic Transfer Switch means as defined in OPSS 517.

Cofferdam means as defined in OPSS 539.

Cut-Off Wall means as defined in OPSS 517.

Design Storm Return Period means as defined in OPSS 517.

Groundwater Control System means as defined in OPSS 517.

Plug means as defined in OPSS 517.

Sediment means as defined in OPSS 517.

Sediment Control Measure means as defined in OPSS 517.

Temporary Flow Passage System means as defined in OPSS 517.

Unwatering means as defined in OPSS 517.

Vegetated Discharge Area means as defined in OPSS 517.

Waterbody means as defined in OPSS 517.

Watercourse means as defined in OPSS 517.

902.04 DESIGN AND SUBMISSION REQUIREMENTS

902.04.01 Design Requirements

902.04.01.01 Dewatering

Clause 902.04.01.01 of OPSS 902 is deleted in its entirety and replaced with the following:

A dewatering system shall be designed to control water and the flow of water into the excavation, prevent disturbance of the foundation, permit the placing of concrete in the dry, and complete the excavating and backfilling for structures work.

When the system includes temporary flow passage system, the system shall be designed, as a minimum, for a 5 year design storm return period, and groundwater discharge. A longer return period shall be used when determined appropriate for the work.

The dewatering system shall be according to the design requirements specified in OPSS 517.

902.04.02 Submission Requirements

Subsection 902.04.02 of OPSS 902 is deleted in its entirety and replaced with the following:

902.04.02.01 Preconstruction Survey

When a groundwater control system by wells or a well point system will be used, a condition survey of property and structures that may be affected by the work shall be carried out. The condition survey shall include the location and condition of adjacent properties, buildings, underground structures, water wells, Utilities, and structures, within a distance of 100 metres from the groundwater control system. In addition, all water wells used as a supply of drinking water and located within this distance shall be tested for compliance with Ontario Drinking Water Quality Standards.

Water wells within the preconstruction survey distance can be located using the website <https://www.ontario.ca/environment-and-energy/map-well-records> or its successor site.

Copies of the condition survey and water quality test results shall be submitted to the Contract Administrator prior to the operation of the groundwater control system.

902.04.02.02 Working Drawings

Working Drawings for the dewatering system shall be according to OPSS 517.

902.07 CONSTRUCTION

902.07.04 Dewatering Structure Excavation

Subsection 902.07.04 of OPSS 902 is amended by the addition of the following clauses:

902.07.04.01 General

The dewatering systems shall be constructed and operated according to the Working Drawings.

Activation and deactivation of a temporary flow passage system, if applicable, shall be according to OPSS 517.

The dewatering system shall be continuously operational to control buoyancy forces until such forces can be resisted by backfill and structure self-weight, to keep excavations stable, to avoid erosion impacts from the release of accumulated water, and to keep the work area in the condition required to complete the associated work as specified in the Contract Documents.

When a temporary flow passage system is to remain operational through a seasonal shutdown period, the Contractor shall be responsible for any maintenance or repair costs due to the system during the seasonal shutdown period.

Temporary erosion and sediment control measures, including controlling the discharge of water, shall be according to OPSS 805. Measures not specified in OPSS 805 shall be according to the Working Drawings. Temporary erosion and sediment control measures and cover material to protect exposed soils, as required by the Working Drawings, shall be installed as soon as is practical.

Stranded fish shall be managed as specified in the Contract Documents.

Unwatering shall be carried out as necessary.

Water suspected of being contaminated as indicated by visual or olfactory observations shall be reported to the Contract Administrator.

Dewatering and temporary flow passage systems shall be discontinued in a manner that does not disturb any structure, pipeline, or flow channel. Operation of the dewatering system shall be shut down according to the procedures specified in the Working Drawings, where applicable.

902.07.04.02 Discharge of Water

The discharge of water shall be according to OPSS 517.

902.07.04.03 Monitoring

Monitoring shall be according to OPSS 517.

902.07.04.04 System Amendments

Amendments to stop any displacement, damage, soil loss or erosion due to the operation of the dewatering system shall be according to OPSS 517.

902.07.04.05 Removal

Removal of dewatering system and temporary flow passage system components shall be according to OPSS 517.

WORKING SLAB - Item No.

Non-Standard Special Provision

1.0 Scope

This Special Provision covers the requirements for the supply and placement of a concrete working slab under foundations for the Highway 7A culvert replacement and retaining wall structures.

1.1 References

This Special Provision refers to the following standards, specifications or publications:

Ontario Provincial Standard Specifications, Construction
OPSS 902 Excavating and Backfilling - Structures

2.0 Definitions - Not Used

3.0 Design and Submission Requirements - Not Used

4.0 Materials

Concrete for working slabs shall have a minimum 28 day strength of 20 MPa.

5.0 EQUIPMENT - Not Used

7.0 CONSTRUCTION

7.01 Excavation

Excavation for the working slab shall be according to OPSS 902.

7.02 Protection of Founding Soil

Following inspection and approval of the prepared subgrade, a working slab with a minimum thickness of 100 mm shall be placed on the foundation subgrade as specified in the Contract Documents.

7.04 Dewatering

Dewatering shall be carried out according to OPSS 902.

6.0 Quality Assurance - Not Used

9.0 Measurement for Payment - Not Used

10.0 Basis of Payment

10.01 Working Slab - Item

Payment at the Contract price for the above tender item shall be full compensation for all labour, Equipment and Material to do the work.

END OF SECTION

RETAINED SOIL SYSTEM - Item No.

Non-Standard Special Provision

Amendment to SP 599S22 (March 2018), Section 4.0 Design and Submission Requirements

The following Section 4.01.06 is to be added:

Section 4.01.06 RSS Walls Adjacent to Watercourse or Floodplain

The Retained Soil System (RSS) wall types listed on the Ministry of Transportation, Ontario (MTO) Designated Sources of Material (DSM) are not pre-approved for use within or adjacent to watercourses or floodplains. If consideration is given to the use of an RSS wall at Site No. 22-415C and/or Site No 21-13C.

The Contractor shall submit the proposed RSS wall system design to the Contract Administrator who will provide the MTO RSS committee for review and approval.

The proposed wall system shall be suitable for use near/adjacent to water. The submission shall include working drawings, supporting design documentation and commentary which specifically address the proposed design with respect to the following:

- a. Differential hydrostatic pressures – water level in front of RSS wall vs. water level within/behind RSS wall and potential for loss of fines (piping) from the granular backfill.
- b. Potential use of coarser backfill with little or no fines at/below High Water Level (HWL).
- c. Pullout capacity and frictional resistance between reinforcements and select backfill under submerged conditions (buoyant unit weight).
- d. Adequate RSS embedment depth.
- e. Adequate reinforcement length.
- f. Scour protection / rip-rap – properly sized and filter graded.
- g. CHBDC structure design requirement for a 75 year service life – stability, durability, long-term performance.

The submission shall be made to the Contract Administrator prior to [insert date submission must be provided for – based on project schedule] for approval. The Contractor shall assume a minimum of 8-weeks of review time by the MTO RSS Committee.

DEWATERING SYSTEM - Item No.
TEMPORARY FLOW PASSAGE SYSTEM - Item No.

Special Provision No. 517F01

July 2017

Amendment to OPSS 517, November 2016

Design Storm Return Period and Preconstruction Survey Distance

517.01 SCOPE

Section 517.01 of OPSS 517 is deleted in its entirety and replaced with the following:

This specification covers the requirements for the design, operation, and removal of a dewatering or temporary flow passage system or both to control water during construction, and the control of the water prior to discharge to the natural environment and sewer systems.

517.04 DESIGN AND SUBMISSION REQUIREMENTS

517.04.01 Design Requirements

Subsection 517.04.01 of OPSS 517 is amended by deleting the first paragraph in its entirety and replacing it with the following:

A dewatering or temporary flow passage system or both shall be designed to control water at the locations specified in the Contract Documents and at any other location where a system is necessary to complete the work. The design of the system shall be sufficient to permit the work at each location to be carried out as specified in the Contract Documents.

Subsection 517.04.01 of OPSS 517 is further amended by deleting the second last paragraph in its entirety and replacing it with the following:

Temporary flow passage systems shall be designed, as a minimum, for a 2 year design storm return period and groundwater discharge, except for the work specified in Table A. For the work specified in Table A, the temporary flow passage system shall be designed, as a minimum, for the design storm return period specified in Table A and groundwater discharge. A longer return period shall be used when determined appropriate for the work.

Intensity-Duration Factor (IDF) curve location, site specific minimum return period, return period flow estimates, and other information is provided in Table A. The IDF information can be accessed through the MTO IDF Curve Look up Tool on the Drainage and Hydrology page of MTO's website. The return period flow estimates do not include flow volumes from groundwater discharge. The Owner specifically excludes these flow estimates from the warranty in the Reliance on Contract Documents subsection of OPSS 100, MTO General Conditions of Contract.

Table A

IDF Curve Location	Latitude: 44.131275			Longitude: -78.82883		
Temporary Flow Passage Systems						
Site Name / Station Reference	Minimum Return Period (Years)	Return Period Flow Estimates (m³/s)				Design Engineer Requirements (Note 1)
		2 Year	5 Year	10 Year	25 Year	
Site 21-13/C- Culvert Rehabilitation	2	4.63	10.32	14.33	19.03	Yes
Dewatering Systems						
Site Name / Station Reference	Preconstruction Survey Distance (Note 2) (m)					Design Engineer Requirements (Note 1)
Site 21-13/C- Culvert Rehabilitation	100					Yes
Note:						
1. “Yes” means the design Engineer and design-checking Engineer shall have a minimum of 5 years of experience in designing systems of similar nature and scope to the required work. “No” means a minimum experience level is not required for the design Engineer and design-checking Engineer.						
2. “N/A” indicates a preconstruction survey is not required.						

OBSTRUCTIONS

Notice to Contractor

The Contractor shall be alerted to the potential presence of cobbles, boulders, brick fragments and organic inclusions (possibly wood / logs) in the fill and near the interface with the native soils. There is also the presence of cobbles and boulders and gravel nests present in the native soils and especially within the glacially derived (till) soils. Consideration of the presence of these obstructions must be made in the selection of appropriate equipment and procedures for advancing excavations for both culverts and associated retaining wall structures at Site Nos. 22-415C and 21-13C, and installation of any dewatering systems, temporary protection systems and/or cofferdams that may be required.

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