Foundation Investigation and Design Report

Retaining Walls, Highway 48 – Little Rouge River Culvert (Site No. 37-1195/C), Town of Markham, Regional Municipality of York, Ontario, Ministry of Transportation, Ontario, Agreement No. 2016-E-0029

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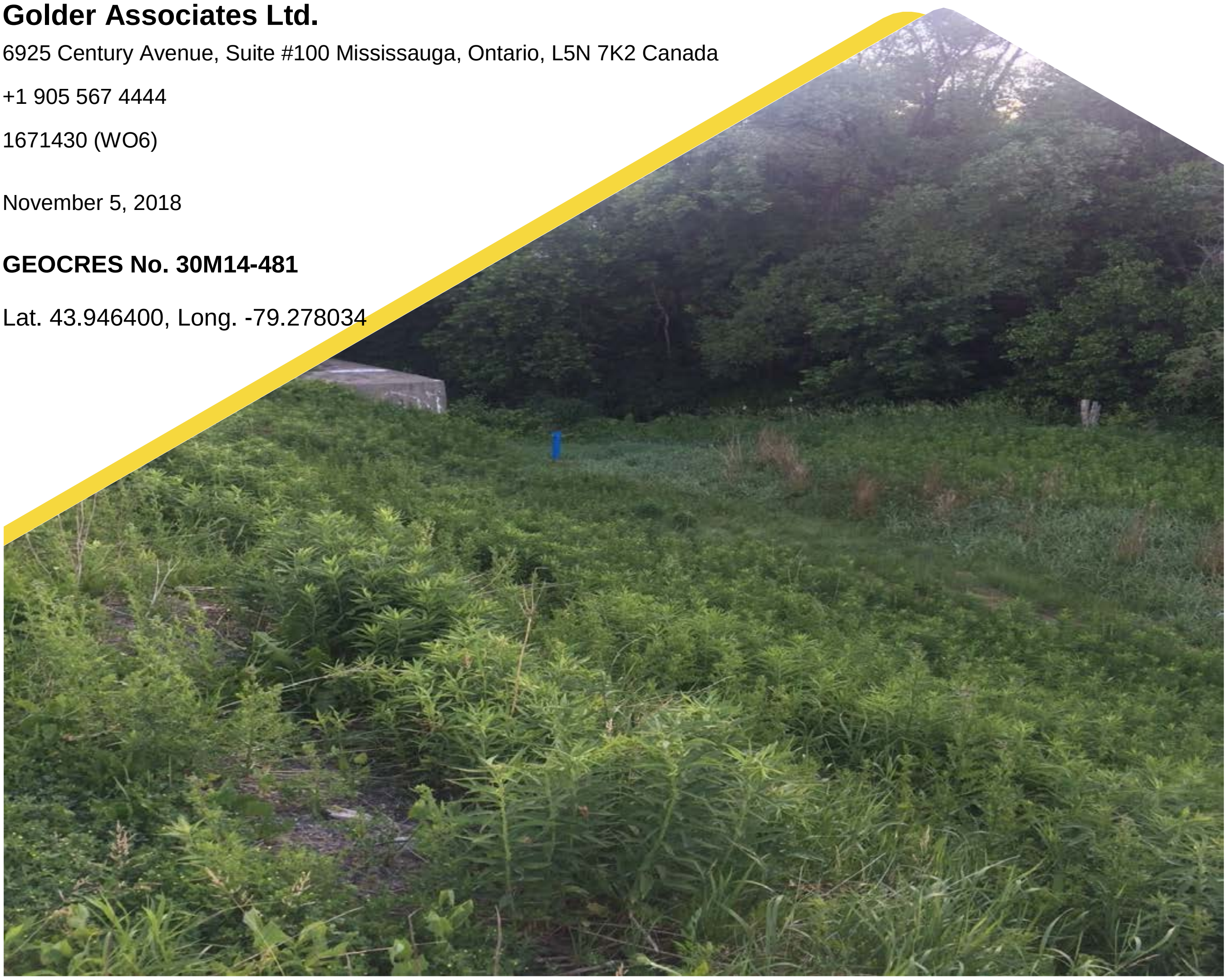

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TOWN OF MARKHAM, REGIONAL MUNICIPALITY OF YORK, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
AGREEMENT NO. 2016-E-0029

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by AECOM Canada Ltd. (AECOM) on behalf of the Ministry of Transportation, Ontario (MTO) to provide detail foundation investigation and engineering services for the proposed retaining walls at the inlet and outlet of Little Rouge River Culvert (Site No. 37-1195/C) to accommodate widening of Highway 48 in the Town of Markham, Regional Municipality of York, Ontario, at the general location shown on the Key Plan on Drawing 1.

The Terms of Reference and scope of work are outlined in MTO's Work Item Order No. 2016-E-0029-006, dated October 24, 2017, which forms part of the Consultant's Assignment for the Central Region Large Value Retainer under Agreement No. 2016-E-0029-006.

2.0 SITE DESCRIPTION

The existing Little Rouge River Culvert is located across Highway 48, approximately 90 m north of the Highway 48 – 19th Avenue intersection in the Town of Markham, Regional Municipality of York, Ontario. The site is surrounded by residential properties, with the topography generally flat-lying to gently rolling. The existing Highway 48 grades down from approximately Elevation 239 m at the 19th Avenue interchange to about Elevation 236 m at the Little Rouge River Culvert crossing, and then rises to the north of the culvert. The storm sewer ditches extend along both sides of Highway 48 with the bottom / invert levels ranging from about Elevation 229.5 m to Elevation 236.5 m near 19th Avenue and at about 50 m north of the culvert. The water level in the culvert, as shown on the 60% drawings provided by AECOM, was at Elevation 229.4 m on July 10, 2018.

The existing Little Rouge River Culvert consists of a concrete box that is approximately 2.8 m high, 6.3 m wide and 40 m long.

Surface erosion was noted along the embankment slopes on both sides of Highway 48, with the most predominant erosion noted in the northeast quadrant of the culvert, as shown on Photograph 1 below. At the time of the site reconnaissance, it appeared that the erosion gullies had been filled in and the embankment slope facing was restored. Minor erosion was noted along the east side of Highway 48 as noted on Photograph 2.

Photograph 1: Surface erosion on east side of Highway 48

Photograph 2: Surface erosion on west side of Highway 48

3.0 INVESTIGATION PROCEDURES

Field work at the Little Rouge River Culvert site was carried out from May 16 to 28, 2018, during which time four boreholes (designated as Boreholes 17-1 to 17-4) were advanced at approximately the borehole locations shown on Drawing 1 and as follows: Boreholes 17-1 and 17-3 were advanced from the roadway platform in the southbound and northbound shoulder of Highway 48 south and north of the culvert, respectively; and Boreholes 17-2 and 17-4 were advanced at the west and east toe of the embankment of Highway 48, to the north and south of the culvert, respectively, near the culvert ends.

Borehole 17-1 was drilled using 210 mm outer diameter hollow-stem augers and Borehole 17-3 was drilled using 102 mm outer diameter solid stem augers both advanced by a D90 truck-mounted drill rig. Boreholes 17-2 and 17-4 were advanced using 89 mm inner diameter casing with a portable tripod drill rig. All drill rigs were supplied and operated by Walker Drilling Ltd. of Utopia, Ontario. Soil samples were 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 Boreholes 17-1 and 17-3 and driven by a full-weight manual hammer in Boreholes 17-2 and 17-4, in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586)[1].

Boreholes 17-1 and 17-3 were advanced through the road embankment to depths of about 12.3 m below existing ground surface. Boreholes 17-2 and 17-4 were advanced at the west and east toe of the embankment of Highway 48, respectively, and terminated upon casing refusal at a depth of about 5.0 m and 3.2 m below existing ground surface.

The groundwater conditions in the open boreholes were observed during and immediately following the drilling operations. A standpipe piezometer was installed in each of Boreholes 17-2 and 17-4 to permit monitoring of the water level. The installed piezometer in Boreholes 17-2 and 17-4 consists of a 50 mm diameter PVC pipe, with a 1.5 m slotted screen sealed within a filter sand pack with the bottom of the piezometer set at the bottom of the borehole. The borehole and annulus surrounding the piezometer pipe above the filter sand pack were backfilled to the ground surface with bentonite pellets. Piezometer installation details and water level readings are described on

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

the respective borehole record in Appendix A. Boreholes 17-1 and 17-3 were backfilled to ground surface with bentonite, in accordance with Ontario Regulation 903, Wells (as amended).

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 Mississauga for further visual review and geotechnical laboratory testing on selected samples, consisting of natural moisture content, Atterberg limits and grain size distribution analyses conducted in accordance with MTO and / or ASTM Standards as applicable.

The borehole locations were marked in the field by Golder personnel relative to the existing culvert and other site features. The locations given in the Record of Borehole sheets and shown on Drawing 1 are positioned relative to MTM NAD 83 (Zone 10) northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum. The borehole locations, including in geographic coordinates of Latitude and Longitude, ground surface elevations and drilled depths are summarized below.

Borehole No.	MTM NAD83		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m) (Latitude)	Easting (m) (Longitude)		
17-1	4867396.4 (43.946341)	322615.0 (-79.278066)	236.0	12.3
17-2	4867430.5 (43.946648)	322605.9 (-79.278178)	233.4	5.0
17-3	4867413.0 (43.946490)	322622.2 (-79.277976)	236.1	12.3
17-4	4867387.2 (43.946257)	322637.2 (-79.277790)	231.8	3.2

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

This section of Highway 48 is located within the South Slope physiographic region, as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984)[2].

The South Slope physiographic region is comprised of calcareous clay till with lacustrine clay and silt reworked by glaciers, with numerous scattered drumlins and deep valley cuts caused by streams flowing towards Lake Ontario.

4.2 General Overview of Subsurface Conditions

The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of the in situ and laboratory tests are provided on the borehole records in Appendix A. The results of the in situ field tests (i.e.,

[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.

SPT "N"-values) as presented on the borehole records, on the stratigraphic profiles and in Section 4 are uncorrected. The results of the laboratory test are presented on the borehole records in Appendix A and in the laboratory test plots in Appendix B. The results of the analytical testing of a soil sample are presented in Maxxam's report included in Appendix C, and summarized in Section 4.4.

The stratigraphic boundaries shown on the borehole records and on the interpreted stratigraphic profile on Drawing 1 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. Variation in the stratigraphic boundaries between and beyond boreholes will exist and is to be expected; however, the factual data presented on the borehole records governs any interpretation of the site conditions.

In general, the native subsurface soils encountered near the proposed retaining walls adjacent to the Little Rouge River Culvert consist of fill underlain by a native soil deposit comprised of clayey silt with sand, underlain in places by a gravelly sand or silt and sand deposit, further underlain by a till deposit comprised of clayey silt with sand to silt and sand. A detailed description of the subsurface conditions encountered in the boreholes is presented in the following sections of this report.

4.2.1 Topsoil

An approximately 1.2 m and 0.1 m thick layer of topsoil (fill) was encountered immediately below ground surface in Boreholes 17-2 and 17-4, respectively.

The Standard Penetration Tests (SPT) "N"-values measured within the thicker topsoil fill layer are 18 blows and 22 blows per 0.3 m of penetration indicating a compact level of compactness.

4.2.2 Fill

Boreholes 17-1 and 17-3 were advanced from the Highway 48 platform and penetrated an approximately 3.0 m and 3.8 m thick layer of fill. At the toe of the roadway embankment an approximately 1.2 m and 1.5 m thick layer of fill was encountered underlying the topsoil in Boreholes 17-2 and 17-4, respectively. The base of the fill layer extends to between Elevations 233.0 m and 230.2 m.

The fill material is generally non-cohesive and the layer is interlayered, with the layer consisting of gravelly sand, sand, silty sand, silt and sand, or silt. A layer of cohesive fill material consisting of clayey silt to sandy clayey silt was encountered interlayered with or underlying the non-cohesive fill material in Boreholes 17-1 and 17-3.

SPT "N"-values measured within the non-cohesive portion of the fill layer range from 6 blows to 78 blows per 0.3 m of penetration, with two discrete values of 114 blows per 0.3 m of penetration and 50 blows for 0.14 m of penetration, indicating that the non-cohesive fill has a loose to very dense level of compactness. SPT "N"-values measured within the cohesive portion of the fill are 33 blows and 36 blows per 0.3 m of penetration, and one value of 100 blows per 0.13 m of penetration, suggesting that the cohesive fill has a hard consistency.

Atterberg limits testing was carried out on one sample of the cohesive fill layer and measured a liquid limit of about 16 per cent, a plastic limit of about 11 per cent, and corresponding plasticity index of about 5 per cent. The result, which is plotted on a plasticity chart on Figure B-1 in Appendix B, indicates that the cohesive fill layer is clayey silt-silt of low plasticity. Atterberg limits testing was also carried out on two samples of the silt portion of the non-cohesive fill and indicate that the material is non-plastic. Natural water contents ranging between about 6 per cent and 25 per cent were measured on selected samples of the non-cohesive fill material, while natural water contents of about 8 per cent and 13 per cent were measured on samples of the cohesive fill.

4.2.3 Clayey Silt with Sand

A 2.6 m and 1.7 m thick deposit of clayey silt with sand was encountered underlying the fill layer in Boreholes 17-1 and 17-3 at Elevations 233.0 m and 232.3 m, respectively.

SPT “N”-values ranging from 16 blows to 28 blows per 0.3 m of penetration and “N”-values of 71 blows per 0.3 m of penetration and 50 blows per 0.13 m of penetration were measured within the clayey silt with sand deposit, suggesting a very stiff to hard consistency.

Grain size distribution tests were carried out on two samples of the clayey silt with sand deposit and the results are shown on Figure B-2 of Appendix B. The deposit consists of clayey silt with sand containing trace to some gravel. Atterberg limits testing was carried out on two samples of the cohesive deposit and measured liquid limits of about 22 per cent and 25 per cent, plastic limits of about 14 per cent and plasticity indices of about 8 per cent and 11 per cent. These results, which are plotted on a plasticity chart on Figure B-3 in Appendix B, indicate that the cohesive deposit is a clayey silt of low plasticity. The natural water content measured on two samples of this deposit are about 15 per cent and 17 per cent.

4.2.4 Gravelly Sand

A 1.6 m thick deposit of gravelly sand was encountered underlying the clayey silt with sand deposit in Borehole 17-1 at Elevation 230.4 m.

An SPT “N”-value of 40 blows per 0.3 m of penetration was measured within the gravelly sand deposit, indicating a dense level of compactness. A grain size distribution test was carried out on one sample of the gravelly sad deposit and the result is shown on Figure B-4, in Appendix B. The natural water content measured on one sample of the gravelly sand deposit is 14 per cent.

4.2.5 Silt and Sand

A 1.6 m thick deposit of silt and sand was encountered underlying the clayey silt with sand deposit in Borehole 17-3 at Elevation 230.6 m.

An SPT “N”-value of 38 blows per 0.3 m of penetration was measured within the silt and sand deposit, indicating a dense level of compactness. A grain size distribution test was carried out on one sample of the silt and sand deposit and the result is shown on Figure B-5, in Appendix B. The natural water content measured on one sample of the gravelly sand deposit is 19 per cent.

4.2.6 Clayey Silt with Sand to Silt and Sand Till

A till deposit comprised of clayey silt with sand grading to a silt and sand was encountered underlying the fill layer in Boreholes 17-2 and 17-4, underlying the gravelly sand deposit in Borehole 17-1 and underlying the silt and sand deposit in Borehole 17-3. The surface of the till deposit was encountered between Elevations 231.0 m and 228.8 m. All borehole terminated within this deposit, penetrating it for a thickness of 1.6 m to 5.2 m.

The SPT “N”-values measures within the till deposit range between 76 blows per 0.3 m of penetration and 300 blows per 0.13 m of penetration, suggesting a hard consistency / very dense level of compactness.

Grain size distribution tests were carried out on four samples of the till deposit and the results are shown on Figure B-6 of Appendix B. Atterberg limits testing was carried out on five samples of the till deposit and one Atterberg limits test indicated non-plastic material, while four tests measured liquid limits between about 12 per cent and 18 per cent, plastic limits between about 9 per cent and 12 per cent and plasticity indices between about 1 per cent and 6 per cent. These results, which are plotted on a plasticity chart on Figure B-7 in Appendix B, indicate that the cohesive portion of the till deposit is a clayey silt-silt of low plasticity and the non-cohesive portion of the till deposit contains a silt of slight plasticity. The natural water content measured on selected samples of the till deposit ranged between about 7 per cent and 12 per cent.

4.3 Groundwater Conditions

The groundwater levels in the open boreholes were measured upon completion of drilling operations. A standpipe piezometer was installed in each of Boreholes 17-2 and 17-4 to permit monitoring of the groundwater level at this site. Details of the piezometer installation and the measured groundwater levels are shown on the borehole records in Appendix A. The groundwater level recorded in the open boreholes and standpipe piezometers are summarized below.

Borehole No.	Ground Surface Elevation (m)	Depth to Water Level (m)	Groundwater Elevation (m)	Date	Comments
17-1	236.0	5.2	230.8	May 16, 2018	Open borehole (borehole caved to 6.6 m depth)
17-2	233.4	0.9 m	232.5	May 28, 2018	Piezometer
		1.0	232.4	June 22, 2018	
17-3	236.1	4.1	232.0	May 23, 2018	Open borehole
17-4	231.8	0.2	231.6	May 24, 2018	Open borehole at completion of drilling
		0.1	231.7	June 22, 2018	Piezometer

The groundwater level observations at this site will be subject to seasonal fluctuations and precipitation events, and the water levels should be expected to be higher during the spring season or during and following periods of heavy precipitation.

4.4 Analytical Testing Results

A soil samples was submitted to MAXXAM Analytical Laboratory for analysis of parameters used to assess the potential corrosivity of the site soil to steel and concrete. Detailed analytical test results are included in Appendix C and the test results are summarized below.

Borehole No. / Sample No.	pH	Resistivity (ohm-cm)	Electrical Conductivity (umho/cm)	Soluble Chloride (ug/g)	Soluble Sulphates (ug/g)
17-4 / 4	8.00	2,900	349	110	78

5.0 CLOSURE

This Foundation Investigation Report was prepared by Ms. Nikol Kochmanová, P.Eng., a geotechnical engineer with Golder. Jorge Costa, P.Eng., a MTO Foundations Designated Contact and Senior Consultant for Golder, conducted a quality control review of the report.

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PART B

FOUNDATION DESIGN REPORT
RETAINING WALLS
HIGHWAY 48 – LITTLE ROUGE RIVER CULVERT (SITE NO. 37-1195/C)
TOWN OF MARKHAM, REGIONAL MUNICIPALITY OF YORK, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
AGREEMENT NO. 2016-E-0029

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides detail foundation engineering design recommendations for the proposed retaining walls at the inlet (west) end of Little Rouge River Culvert (Site No. 37-1195/C) to accommodate widening of Highway 48 in the Town of Markham, Regional Municipality of York, Ontario. These recommendations 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 designer with sufficient information to assess the feasible retaining wall alternative types and carry out the design of the retaining wall foundations.

The foundation investigation report, discussion 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 based on the factual data in Part A (Foundation Investigation) of the 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 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

Based on the design drawings provided to Golder by AECOM on October 10, 2018 and subsequent discussions with AECOM, we understand that a retaining wall, consisting of a concrete cantilever wall extending southerly from the culvert inlet for a length of about 24 m and a toe wall extending southerly from the end of the cantilever wall for a length of about 30.5 m is proposed in the southwest quadrant of the Little Rouge Creek Culvert. Additionally, due to the widening of Highway 48, and the near proximity of private property, a retaining wall consisting of an approximately 28 m long concrete toe wall, to accommodate the Highway 48 widening, is proposed on the northwest quadrant of the Little Rouge Creek Culvert. It is noted that retaining walls were originally proposed to be constructed on the four quadrants of the culvert sire, but we understand that two retaining walls are no longer required.

The water level in the culvert, as shown on the 60% drawings provided by AECOM, was at Elevation 229.4 m on July 10, 2018.

6.2 General Foundation Design Context

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 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 “low degree of site and prediction model understanding” for the retaining wall and toe wall at the southwest quadrant of the site and a “low degree of site and prediction model understanding” for the toe wall at the northwest quadrant of the site given that the alignment and extent of these retaining walls have not been fully defined at this time. 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.

6.2.2 Seismic Design

6.2.2.1 Seismic Site Classification

Subsurface ground conditions for seismic site characterization were established based on the results of the field investigation and laboratory testing. The SPT "N"-values measured in the soil layers and the interpreted shear wave velocity of soils up to 30 m below founding level 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 would be applicable for the design of the retaining wall structures.

6.2.2.2 Spectral Response Values and Seismic Performance Category

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.

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	0.058	0.099
PGV (m/s)	0.029	0.047	0.077
Sa (0.2) (g)	0.061	0.095	0.158
Sa (0.5) (g)	0.040	0.060	0.094
Sa (1.0) (g)	0.023	0.034	0.053
Sa (2.0) (g)	0.011	0.017	0.027
Sa (5.0) (g)	0.0024	0.0040	0.0066
Sa (10.0) (g)	0.0011	0.0017	0.0028

6.3 Retaining Wall and Foundation Options

This section of the report presents a comparison of alternative retaining walls / foundation types based on advantages and disadvantages of the alternative retaining wall types and provides geotechnical recommendations for the various types of walls and foundation alternatives.

It should be noted that the selection of the type of walls and foundation alternative will depend on many factors beyond geotechnical / foundation recommendations. From a geotechnical/foundations perspective the type of retaining wall considered suitable for the replacement of the existing retaining walls given the soil conditions as encountered in the various boreholes drilled at the retaining wall sites include the following:

- **Reinforced Earth Slope:** A reinforced earth slope constructed at an inclination of 1 horizontal to 1 vertical (1H:1V), is geotechnically feasible at this site; however, it is understood that there is insufficient space for all but a vertical retaining wall geometry, and hence reinforced slopes are not discussed further in this report.

- **Reinforced Soil System (RSS) Wall:** RSS walls are geotechnically feasible given the generally competent nature of the shallow soil conditions. These wall types are often advantageous relative to concrete walls on shallow foundations as shallower excavation depths are required for an RSS wall, with associated reduction in groundwater control and/or protection system requirements. However, we understand that an RSS wall within a floodplain or near flowing water, as is the case for this site, would likely require a site-specific design submission and review and approval by the MTO RSS Committee.
- **Concrete Retaining Wall on Shallow Foundation:** A concrete cantilever retaining wall supported on shallow foundation (concrete strip footing) is geotechnically feasible at this site. Temporary excavation support will be required to accommodate the excavation to allow for construction of the strip footing. The excavation is expected to extend below the groundwater level at both locations at this site, particularly during wet periods of the year, and greater groundwater control is expected to be required as compared with an RSS wall.
- **Concrete Retaining Wall on Deep Foundations:** A concrete wall supported on deep foundations (driven piles or caissons) is not required at this site given the competent soil deposits present at shallow depth. Therefore, this option is not discussed further in this report.
- **Soldier Pile and Concrete Panel Wall:** A soldier pile and concrete panel system is generally more advantageous in “top-down” construction applications as part of a cut widening, rather than for an embankment widening, as is the case at this site. Additionally, due to the shallow depth of “100-blow” material at this site, a soldier pile and concrete panel wall is not considered practical and is not discussed further.

A comparison of the various retaining wall options based on advantages, disadvantages and relative cost is presented in Table 1. Based on a comparison of the advantages/disadvantages between the various wall types and supporting foundation alternatives and given the subsurface conditions as encountered at the boreholes, the preferred retaining wall alternative from a foundations perspective is a concrete retaining wall on a shallow foundation (i.e., strip footing) at each wall location.

The following sections of this report provides foundation recommendations for the preferred option and select alternative wall type, and presents the results of the assessment/analyses of settlement and global stability for the preferred / recommended retaining wall and for an alternative type of retaining wall.

6.4 Concrete Cantilever Wall Founded on Shallow Foundations

6.4.1 Founding Elevations

Strip footing (shallow) foundations are feasible for the support of the proposed retaining wall at the northwest and southwest quadrant of the culvert site. The strip footings should be founded below any topsoil, fill or softened/loosened surficial soils, and at a minimum depth of 1.5 m below the adjacent final grade to provide adequate protection against frost penetration, in accordance with OPSD 3090.101 (*Foundation, Frost Penetration Depths for Southern Ontario).*

Based on the AECOM design drawings (Drawing R1-1 and R1-4) provided to us, the lowest final grade in front of the retaining walls will be at approximately Elevation 230.4 m, and as such, the strip footings should be founded no higher than Elevation 228.9 m, on the hard / very dense clayey silt with sand till to silt and sand till deposit and dense gravelly sand deposit. Where the ground surface rises away from the creek channel, the retaining walls foundations may be stepped up, provided that they remain founded at a minimum depth of 1.5 m, and that they are founded on the till deposit. A continuous strip footing constructed of sections on a subgrade at different founding elevations must include a sloping base on native ground (or granular pad) between sections, inclined no steeper than 1H:1V (i.e., not vertical).

The above-noted founding levels will extend below the groundwater level at the site (measured in the piezometers at between Elevations 232.4 m and 231.7 m), and groundwater control will be required. MTO's Non-Standard Special Provision (NSSP – "FOUN0003") for dewatering structure excavations should be included in the Contract Documents, as discussed further in Section 6.9.2.

The clayey silt with sand to silt and sand till 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 following inspection and approval of the subgrade, to protect the subgrade from softening if the footings are not constructed within four hours of such approval for construction. 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 D.

The footing subgrade should be inspected by a 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. Where subexcavation of fill or unsuitable material is required (if the retaining wall foundation will be stepped), the sub-excavated area could be backfilled with granular material meeting OPSS.PROV 1010 (*Aggregates*) Granular 'A' or Granular 'B' Type II that is placed and compacted in accordance with OPSS.PROV 501 (*Compacting*), or the thickness of the footing increased to the full excavation depth. 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, and the top of the granular engineered fill should extend at least 1 m beyond the plan limits of the footing. Temporary protection systems will be required to allow for construction of the strip footings as discussed in Section 6.9.1.

6.4.2 Factored Geotechnical Resistance

Strip footings constructed, at or below the design elevations given in the Section 6.4.1, should be designed based on the factored ultimate geotechnical resistances and the factored serviceability geotechnical resistance (for 25 mm of settlement) given below.

Footing Width (m)	Factored Ultimate Geotechnical Resistance (kPa)	Factored Serviceability Geotechnical Resistance (kPa) (for 25 mm of Settlement)
2.5	200	Does not govern*
3.0**	250	Does not govern*
4.0**	350	Does not govern*
5.0	450	Does not govern*

* The factored serviceability geotechnical resistance at SLS for 25 mm of settlement will be greater than the factored ultimate geotechnical resistance and does not govern.

** Geotechnical resistance may be interpolated between those given for 3.0 m, 4.0 m and 5.0 m wide footings for 3.5 m and 4.5 m wide footings.

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).

6.4.3 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 hard / very dense clayey silt with sand till to silt and sand till deposit or on the Granular 'A' or Granular 'B' Type II engineered fill, the coefficient of friction, tan δ or tan ϕ', can be taken as follows:

- Cast-in-place footing to concrete working slab:tan $\delta = 0.7$
- Cast-in-place concrete footing or working slab to native deposits:tan $\phi' = 0.56$
- Cast-in-place concrete footing or working slab to granular pad:tan $\phi' = 0.56$

6.4.4 Global Stability

Slope stability analyses have been performed for the proposed cantilever retaining walls using the commercially available program SLIDE V2018 produced by Rocscience Inc., employing the Morgenstern-Price method of analysis. For all 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):

Soil Deposit	Bulk Unit Weight (kN/m^3)	Effective Friction Angle
Existing dense to very dense / hard fill	19	32°
Very stiff to hard clayey silt with sand	20	32°
Dense gravelly sand / silt and sand	20	33°
Hard / very dense clayey silt with sand till to silt and sand till	21	35°

A maximum retained wall height of 6.5 m was assumed for both retaining walls. The groundwater level was inferred from the water levels encountered during drilling, as shown on the borehole records.

The stability analysis result indicates that the proposed concrete retaining walls founded on shallow foundations will have a factor of safety greater than 1.5 against global instability. An example of the static global stability results is provided on Figure 1.

Surface run-off can create erosion channels in the embankment shoulders / side slope if it is not properly diverted, as observed in the existing side slopes, and can lead to slope instability if not properly maintained. It is recommended that any erosions channels present within the embankment slopes be maintained and restored on a regular basis.

6.5 Retaining Soil System (RSS) Walls

6.5.1 Founding Elevations

A typical RSS wall has a front facing supported on a strip footing placed at shallow depth below the ground surface in front of the wall. The footing and the RSS mass should be founded below any existing topsoil or unsuitable native or fill soils. The existing topsoil and fill material generally extend to between Elevations 233.0 m and 230.2 m. It is assumed that the RSS Walls will be embedded approximately 0.5 m below the surrounding ground surface, at about Elevation 229.0 m. It is not necessary to embed the RSS wall at frost depth, but the wall will need to be properly / adequately embedded for erosion protection when in close proximity to the existing culvert. The RSS walls would be founded on the dense gravelly sand / silt and sand deposit or the hard / very dense clayey silt with sand till to silt and sand till deposit for the founding elevation noted above (Elevation 229.0 m), or may be founded on granular fill placed on the till to raise the grade to satisfy the minimum embedment depths as set out in MTO's *RSS Wall Design Guidelines* (September 2008), and as discussed further below. The RSS wall may be stepped up, however, the sections constructed at different founding elevations must include a sloping base on native ground (or granular pad) between sections, inclined no steeper than 1H:1V (i.e., not vertical).

The facing footing should be placed on a minimum 300 mm thick layer of compacted OPSS.PROV 1010 (*Aggregates*) Granular 'A', as shown on Figure 5.2 in the MTO *RSS Wall Design Guidelines* (September 2008). The compacted granular pad should extend at least 1.0 m beyond the outside edge of the facing footing, then downward at 1H:1V.

The compacted Granular 'A' pad and the reinforced soil mass should be keyed into the existing embankment fills by benching into the embankment fill, similar to OPSD 208.010 (*Benching of Earth Slopes*).

6.5.2 Geotechnical Resistance and Settlement

Assuming that the RSS wall acts as a unit and uses the full width of the reinforced soil mass (assumed to be a minimum width of 0.67 of the retained height), the proprietary RSS wall design may be based on the factored ultimate geotechnical resistance and factored serviceability geotechnical resistance (25 mm of settlement) given below.

Retained Height (m)	Reinforced Mass Width (m)	Factored Ultimate Geotechnical Resistance (kPa)	Factored Serviceability Geotechnical Resistance (kPa) (for 25 mm of Settlement)
4.5	3.0	150	Does not govern*
5.2	3.5	200	Does not govern*
6.0	4.0	250	Does not govern*
6.5	4.4	300	Does not govern*

* The factored serviceability geotechnical resistance at SLS for 25 mm of settlement will be greater than the factored ultimate geotechnical resistance and does not govern.

6.5.3 Resistance to Lateral Loads/Sliding Resistance

Resistance to lateral forces / sliding resistance between the compacted fill of the RSS wall and the subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, tan δ’, between the compacted granular fills of the RSS wall and the properly prepared native subgrade may be taken as 0.55. Similarly, the coefficient of friction, tan δ’, between cast-in-place concrete facing footing and underlying granular pad may be taken as 0.55. These represent unfactored values.

6.5.4 Global Stability

Slope stability analyses have been performed for the proposed retaining walls using the commercially available program SLIDE V2018 produced by Rocscience Inc., employing the Morgenstern-Price method of analysis. For all 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 static global stability analysis was completed using the parameters outlined in Section 6.4.4. A retained soil height of 6.5 m was assumed. Groundwater levels were inferred from the highest water levels shown on the borehole records.

The stability analysis results indicate that the proposed RSS walls founded on a properly prepared subgrade will have a factor of safety greater than 1.5 against global instability. An example of the static global stability results is provided on Figure 2. It should be noted that the internal stability of a reinforced earth structure is to be designed and assessed by the proprietary product designer/supplier.

6.6 Concrete Toe Wall

Based on discussions with AECOM, a Type II concrete toe wall is proposed for the southern 30.5 m long section of retaining wall at the southwest quadrant and for the approximately 28 m long retaining wall within the northwest quadrant of Little Rouge River Culvert. The concrete toe wall should be founded below any topsoil, fill or softened/loosened surficial soils and should be designed and constructed in accordance with OPSD 3120.100 (*Concrete Toe Wall*), where it is noted that a Type II wall should be founded on undisturbed soil having a bearing capacity at ultimate limits states of 300 kPa. Based on the subsurface conditions, namely the high groundwater levels encountered, the minimum bearing capacity will not be achieved. It is recommended that the toe wall be founded on a minimum 150 mm thick compacted Granular 'A' or Granular 'B' Type II levelling pad that is placed on the native dense gravelly sand / silt and sand deposit and the hard / very dense clayey silt with sand till to silt and sand till deposit, and compacted in accordance with OPSS.PROV 501 (*Compacting*).

6.7 Lateral Earth Pressures for Design

The lateral earth pressures acting on the retaining walls will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the magnitude of the surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls.

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.

- If the walls are to be constructed by temporarily excavating behind the wall, 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. 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 (2014) 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 an unrestrained wall, the granular backfill 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 2014).

6.7.1 Static Lateral Earth Pressures for Design

The following guidelines and recommendations are provided regarding the lateral earth pressures for static (i.e., not earthquake) loading conditions. These lateral earth pressures assume that the ground above the wall will be flat, not sloping. If the inclination of the slope above the wall changes then new lateral earth pressures will need to be calculated.

- The earth pressures acting on the wall will depend on the material behind the wall (i.e., whether there is a zone of granular backfill as described above, or whether the wall is constructed top-down with the existing soils remaining behind the wall. The following parameters (unfactored) may be used:

Material	Granular A	Granular B Type II	Existing Native Materials
Soil Unit Weight:	22 kN/m^3	21 kN/m^3	20 kN/m3
Coefficients of static lateral earth pressure: Active, K_a At rest, K_o	0.27 0.43	0.27 0.43	0.33 0.50

- If the retaining wall structures do not allow lateral yielding, at-rest earth pressures should be assumed for the foundation design. If the 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 wall is 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 consistent with OPSD 3121.150 (*Walls, Retaining, Backfill*).

6.7.2 Seismic Lateral Earth Pressures for Design

Seismic (earthquake) loading must also be taken into account in the design of retaining walls in accordance with Section 4.6.5 of the CHBDC (2014). In this regard, the following should be included in the assessment of lateral earth pressures:

- Seismic loading will result in increased lateral earth pressures acting on the retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure.
- In accordance with Sections 4.6.5 and C.4.6.5 of the CHBDC (2014) and its Commentary, for structures which allow lateral yielding, the horizontal seismic coefficient, k_h, used in the calculation of the seismic active pressure coefficient, is taken as 0.5 times the site-specific PGA. For structures that do not allow lateral yielding, k_h is taken as equal to the site-specific PGA. For both cases the value of the vertical seismic coefficient k_v is taken as zero.
- The following seismic active pressure coefficients (K_{AE}) may be used in design; these coefficients reflect the maximum K_{AE} obtained for each of the earthquake design periods and backfill conditions. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is level. Where sloping backfill is present above the top of the wall, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.

	Design Earthquake	Site PGA	Seismic Active Pressure Coefficients, K_{AE}		
			Granular A	Granular B Type II	SSM
Yielding Wall	475-Yr	0.035	0.26	0.26	0.31
	975-Yr	0.058	0.26	0.26	0.32
	2,475 Yr	0.099	0.27	0.27	0.33
Non-Yielding Wall	475-Yr	0.035	0.27	0.27	0.32
	975-Yr	0.058	0.28	0.28	0.34
	2,475 Yr	0.099	0.30	0.30	0.37

- The K_{AE} value for a yielding wall is applicable provided that the wall can move up to 250k_h mm, where k_h is the site specific PGA as given in the table above. This corresponds to displacements of 9 mm, 15 mm, and 25 mm for the 475-year, 975-year, and 2,475-year design earthquakes at this site.
- The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e. an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined per Section C4.6.5 of the *Commentary to CHBDC* (2014).

6.8 Corrosion Assessment and Protection

Soil corrosivity may affect the concrete foundations and reinforced steel and other concrete elements buried in the soil. The long-term performance and durability of the foundations are directly related to their respective corrosion resistance. Generally, the corrosivity potential to a structure depends on the soil resistivity / electrical conductivity, hydrogen ion concentration, and salts (chloride and sulphate) concentrations. The analytical results for the sample submitted for testing are summarized in Section 4.4 and the analytical laboratory test reports are included in Appendix C.

6.8.1 Potential for Sulphate Attack

The analytical test results were compared to CSA Standard, CAN/CSA-A23.1-14 Table 3 *("Additional requirements for concrete subjected to sulphate attack")* for potential sulphate attack on concrete. The sulphate concentration measured in the sample is less than 0.1 per cent, which is an exposure class of “Moderate”.

6.8.2 Potential for Corrosion

The test results indicate a pH of 8.0 and a resistivity of about 2,900 ohm-cm. According to the Gravity Pipe Design Guidelines (MTO, 2014), the pH is not considered detrimental to concrete durability. However, the resistivity of 2,900 ohm-cm indicates that the soil corrosiveness is “Moderate” (4,500 ohm-cm < R < 2,000 ohm-cm), as per Table 3.2 of the Gravity Pipe Design Guidelines (MTO, 2014), and some level of corrosion protection should be applied to the foundation element / materials. Further, given that the retaining wall foundations are located adjacent to the roadway shoulder and will be exposed to de-icing salt, consideration should be given to selection of a “C” type exposure class as defined by CSA A23.1 Table 1.

It is ultimately up to the structural designer to determine the appropriate exposure class and to ensure that all aspects of CSA A23.1 Section 4.1.1 “Durability Requirements” are followed.

6.9 Construction Considerations

The following subsections identify pertinent construction related issues that should be considered at this stage of the design as they may impact the design. Where applicable, Non-Standard Special Provisions (NSSP) should be included in the Contract Documents.

6.9.1 Excavation and Temporary Roadway Protection

The foundation excavations for strip footings will extend through the fill materials and into / through native clayey silt with sand, gravelly sand and silt and sand deposit and into the clayey silt with sand till to silt and sand till deposit, below the groundwater level. Open-cut excavations into these materials should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act (OHSA) and Regulation 213 for Construction Activities. The existing fill materials are classified as Type 3 soil and the native soils above the water table are classified as Type 2 soil, according to the OHSA. Temporary excavations (i.e., those which are open for a relatively short time period) should be made with side slopes no steeper than 1H:1V through the Type 2 and 3 soils and to within 1.2 m of the bottom of the excavation in Type 2 soils only, provided that appropriate groundwater control is in place to lower the groundwater level to not less than 0.3 m below the excavation base, otherwise the excavations should be made with side slopes no steeper than 3H:1V.

Temporary protection systems will be required to facilitate the construction of the new retaining wall footings or RSS wall mass, in order to safely maintain traffic on Highway 48 and protect the construction zone. The temporary excavation support systems should be designed and constructed in accordance with OPSS.PROV 539 *(Temporary Protection Systems).* The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS.PROV 539.

It is considered that a soldier pile and timber lagging system would be suitable for the temporary excavation support at this site, based on the inferred 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 relatively hard/dense native soils that are present at relatively shallow depth below the natural ground surface, and this 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 to control seepage or include measures to mitigate loss of soil particles through the lagging boards 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.

6.9.2 Surface Water and Groundwater Control

Control of the surface water and groundwater will be necessary to allow excavation and foundation construction to be carried out in dry conditions. MTO’s “FOUN0003” NSSP should be included in the Contract Documents to address the dewatering requirements for retaining wall construction adjacent to the culvert, a copy of which is provided in Appendix D.

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 clayey silt with sand to silt and sand till subgrade or granular backfill/bedding material.

The groundwater level at the site was generally encountered between Elevations 232.5 m (in standpipe piezometer) and 230.8 m (in open borehole), which is higher than the surrounding ground surface adjacent to the toes of the embankment slope at the locations of the wall alignments. Excavations for construction of the retaining walls will extend below the water level, however, it is expected that water inflow from granular zones of fill or present within the native material, or through the till, can be handled by pumping from well filtered sumps located outside the foundation footprint. It is recommended that the groundwater level be lowered to not less than 0.3 m below the base of any excavation and below the footing founding level prior to excavating to those depths to minimize the potential for loosening of the subgrade material.

Based on the subsurface conditions, the construction water dewatering volumes are anticipated to be greater than 50,000 L/day but less than 400,000 L/day; therefore, it is recommended that registry on the Ministry of the Environment Conservation and Parks (MECP) Environmental Activity and Sector Registry (EASR) be prepared ahead of construction to avoid delays. The construction dewatering volumes will be dependent on the contractor’s chosen methods of construction and the groundwater / river water levels at the time of construction; the dewatering recommendations for this project, including the requirement for an EASR or a Permit to Take Water (PTTW) are addressed in a separate report by AECOM’s hydrogeology team.

Surface water seepage into the excavations should be expected and will be heavier during periods of sustained precipitation and all surface water should be directed away from the excavations.

6.9.3 Subgrade Protection

The native 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 if the footing is not constructed within this time period. This requirement can be addressed with a note on the General Arrangement drawing and/or with an NSSP. An NSSP is included in Appendix D.

7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Nikol Kochmanová, P.Eng., a geotechnical engineer with Golder. Jorge Costa, P.Eng., a MTO Foundations Designated Contact and Senior Consultant for Golder, conducted a quality control review of the report.

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NK/JMAC/rb

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Canadian Highway Bridge Design Code (CHBDC) 2014 and Commentary on CAN/CSA-S6-14. Canadian Standard Association (CSA) Group.

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.

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National Resources Canada, 2017. *Earthquake Hazard*. http://www.earthquakescanada.nrcan.gc.ca/hazard-alea/interpolat/index_2015-en.php. Accessed on August 7, 2018.

Ontario Provincial Standard Specifications (OPSS)

OPSS.PROV 501 Construction Specification for Compacting

OPSS.PROV 539 Construction Specification for Temporary Protection Systems

OPSS 902 Construction Specification for Excavating and Backfilling Structures

OPSS.PROV 1010 Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

Ontario Provincial Standard Drawings (OPSD)

OPSD 3090.101 Foundation, Frost Penetration Depths for Southern Ontario

OPSD 3120.100 Walls, Retaining, Concrete Toe Wall

OPSD 208.010 Benching of Earth Slopes

OPSD 3212.150 Walls, Retaining, Backfill, Minimum Granular Requirements

Special Provisions

SP109S12 Amendment to OPSS902

ASTM International

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

ASTM D7012 Standard Test Methods for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures

Commercial Software:

Slide (Version 2018) by Rocscience Inc.

Ontario Water Resources Act

Ontario Regulation 903Wells (as amended)

Ontario Occupational Health and Safety Act

Ontario Regulation 213Construction Projects (as amended)

Ministry of Transportation, Ontario

RSS Design Guidelines, Ministry of Transportation Engineering Standards Branch, September 2008

Gravity Pipe Design Guideline, Ministry of Transportation Ontario. Drainage and Hydrology Design and Contract Standards Office. 2014

TABLE 1 – COMPARISON OF RETAINING WALL AND FOUNDATION ALTERNATIVES

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Reinforced earth embankment (1H:1V slope)	Not feasible due to space restrictions at this site.	■ Relative ease of construction but proprietary product required. ■ Vegetated surfaces could be used to improve aesthetics.	■ Special treatment of reinforced earth slopes required to allow vegetation to grow and minimize erosion; there can be challenges in establishing vegetation. ■ Likely concerns over risk of constructing such systems in proximity to floodplain and flowing water.	■ Lower cost than all vertical wall options.	■ Requires wider footprint than cantilever retaining wall ■ Very tolerant of settlement (although this is not an issue at this site) ■ Risk of wall system susceptibility to erosion in flood conditions.
RSS wall	■ Potentially feasible but the RSS wall would be within the floodplain and near flowing water. ■ Requires a relatively large footprint at the back of the wall, potentially encroaching onto Highway 48 southbound lanes.	■ More tolerable to post-construction settlements than other types of walls, although settlement is not a significant issue at this site.	■ Potentially large excavation required to install reinforcing strips, (0.67H). ■ Temporary protection systems required. ■ Potential for loss of soil particles in flood conditions, although additional measures can be adopted and incorporated into the design to mitigate those impacts on the wall.	■ Lower cost than concrete retaining wall or walls supported on deep foundations.	■ Risk of loss of soil particles in flood conditions; special site-specific design and approval through MTO RSS Committee may be required.

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Concrete retaining wall on shallow foundations (strip footings).	Feasible provided sufficient space is available during construction and/or for temporary shoring if used.	▪ Conventional excavation and construction techniques ▪ Suitable founding stratum below depth of frost penetration at this site.	▪ Less tolerable to post construction settlements ▪ Temporary protection systems will be required during construction. ▪ Footings must be founded below depth of frost penetration.	▪ Higher cost relative to RSS wall.	▪ Deeper excavation as compared with RSS walls, adjacent to existing culvert and creek channel ▪ Deeper protection systems and greater potential for groundwater control.
Concrete retaining wall on deep foundations (piles or caissons).	Not required at this site due to competent soil present at shallow depth.	▪ Potentially reduced excavation, protection system and backfill requirements compared to RSS wall. ▪ Greater geotechnical resistance available so few elements would be required.	▪ Temporary/permanent liners would be required to allow for construction of caissons. ▪ If refusal (100-blow) stratum or obstructions are encountered, can get piles to hang-up, requiring pre-drilling ▪ Requires pile cap to be constructed below depth of frost penetration, likely below groundwater level. ▪ Requires use of large construction equipment to install piles/caissons	▪ Higher cost relative to RSS wall.	▪ Least demanding on right-of-way space if tie-backs not required.

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Soldier pile and concrete panel wall.	■ Not practical at this site given that competent soil is present at shallow depth and generally used in “top-down” construction applications as part of a cut widening, rather than for an embankment widening, as is the case at this site.	■ Most advantageous in “top-down” construction applications, minimizes excavation and requirement for temporary excavation support.	■ Likely requires large heavy equipment to work in the floodplain. ■ 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 / sockets. ■ Potential need for tie-back/anchor installation, with associated testing requirements.	■ Anticipated to be of comparable costs to concrete retaining wall, but higher than RSS wall. ■ Cost of temporary protection system comparable with RSS wall.	■ Lesser excavation than other options, and may eliminate or reduce requirements for temporary protection system. ■ Risks associated with heavier drilling equipment accessing and working in floodplain. ■ Risks associated with loss of soil particles at gaps in panels during flood conditions (similar to, albeit not as high a risk as, RSS walls).

MINISTRY OF TRANSPORTATION, ONTARIO

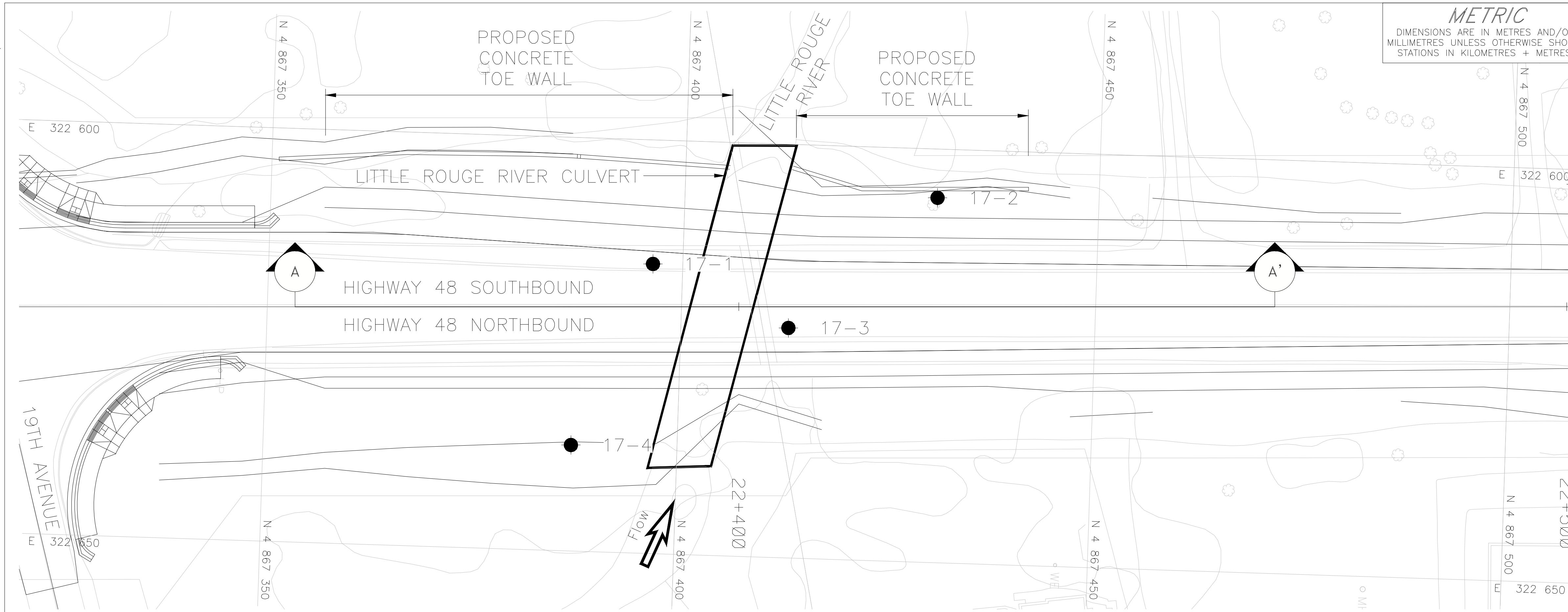
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No. 2019–2005
WP No. 2087–17–00

SHEET

RETAINING WALLS – LITTLE ROUGE RIVER CULVERT
HIGHWAY 48
BOREHOLE LOCATIONS AND SOIL STRATA

GOLDER

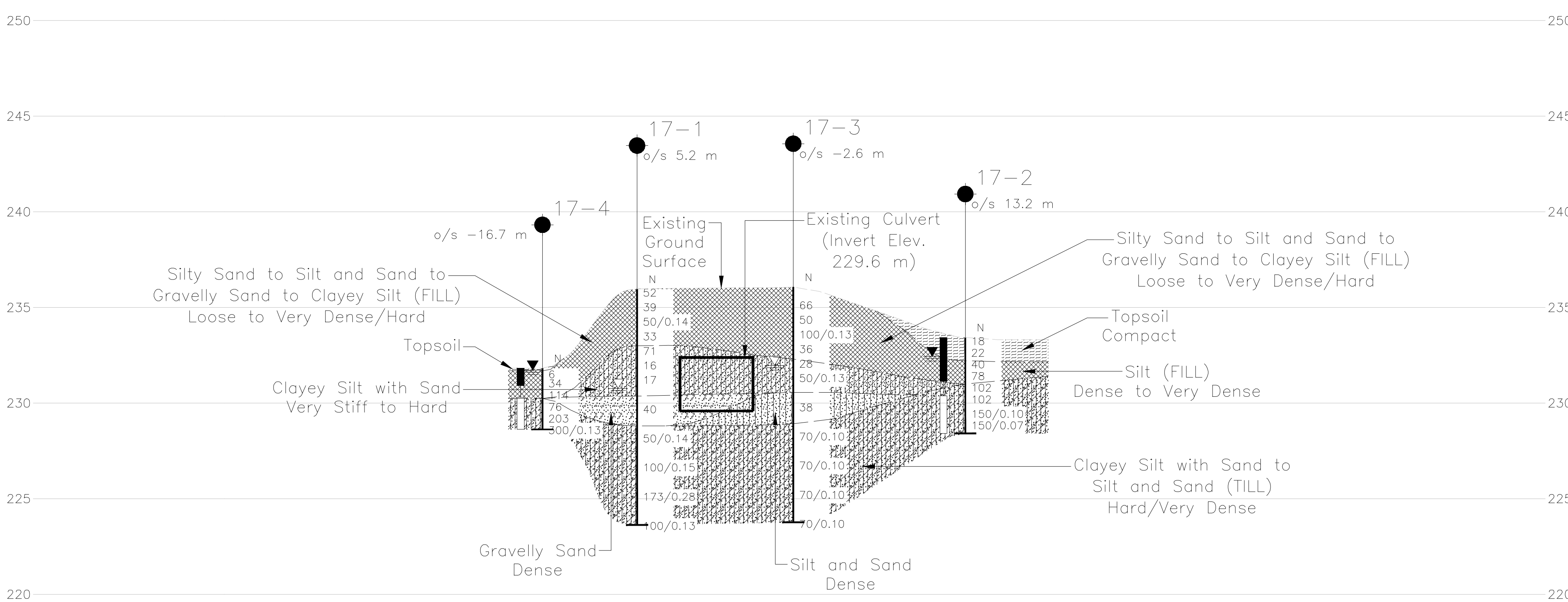


PLAN
SCALE
6 0 6 12 m

LEGEND

- Borehole
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL upon completion of drilling
- WL in piezometer, measured on Jun. 22, 2018

BOREHOLE CO-ORDINATES (MTM NAD 83 ZONE 10)			
No.	ELEVATION	NORTHING	EASTING
17–1	236.0	4867396.4	322615.0
17–2	233.4	4867430.5	322605.9
17–3	236.1	4867413.0	322622.2
17–4	231.8	4867387.2	322637.2



PROFILE A – A'

SCALE HORIZONTAL
6 0 6 12 m

SCALE VERTICAL
3 0 3 6 m

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 Aecom, drawing file nos. Hwy48 19th_bgd_ph.dwg, xs_Hwy48 at 19th.dwg, Hwy48 19th_plan.dwg, Hwy48 and 19th_alignment.dwg, received June 25, 2018, 60572848 R1–3 South Toe Wall.dwg and 60572848 R1–4 North Toe Wall.dwg, received October 12, 2018. Retaining Walls plan provided in digital format by Aecom, drawing file no. X–60572848 Structural Retaining Walls.dwg, received January 11, 2019.

LICENSED PROFESSIONAL ENGINEER N. KOCHMAN 100117863 Jan. 23, 2019 PROVINCE OF ONTARIO

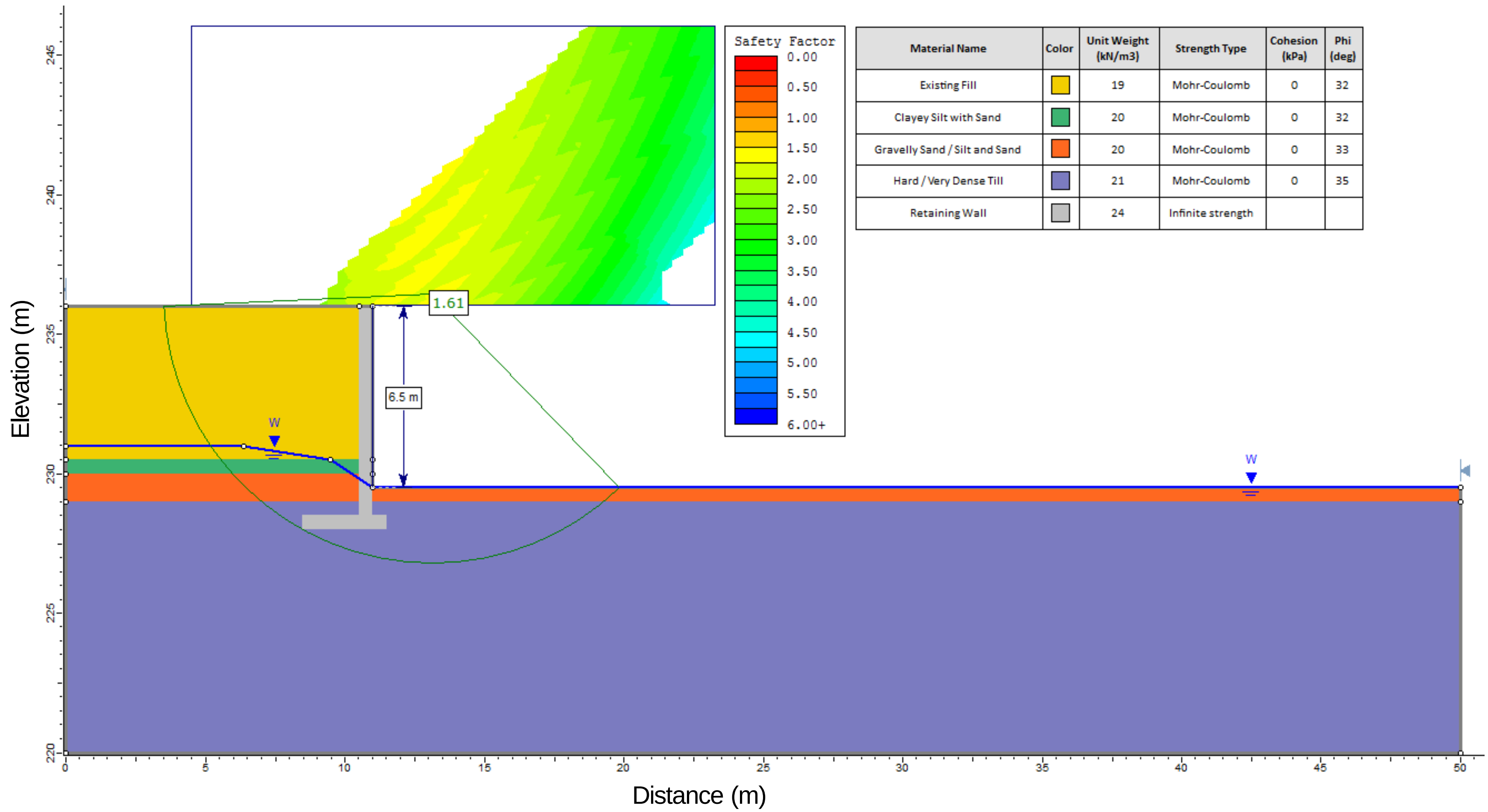
LICENSED PROFESSIONAL ENGINEER J.M.A.COSTA Jan. 23, 2019 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 30M14–481

HWY. 48		PROJECT NO. 1671430	DIST.
SUBM'D. NK	CHKD. NK	DATE: 01/23/2019	SITE: 37–1195/C
DRAWN: DD	CHKD. NK	APPD. JMAC	DWG. 1

PLOT DATE: January 23, 2019
FILENAME: S:\Clients\MTO\Hwy_48\99_PROJ\1671430_Aecom\40_PROD\0004_Little_Rouge_River_Retaining_Walls\1671430-0004-BD-0001.dwg



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Existing Fill		19	Mohr-Coulomb	0	32
Clayey Silt with Sand		20	Mohr-Coulomb	0	32
Gravelly Sand / Silt and Sand		20	Mohr-Coulomb	0	33
Hard / Very Dense Till		21	Mohr-Coulomb	0	35
Retaining Wall		24	Infinite strength		

CLIENT
AECOM / Ministry of Transportation Ontario (MTO)

CONSULTANT

GOLDER

YYYY-MM-DD	2018-11-05
PREPARED	NK
DESIGN	
REVIEW	JMAC
APPROVED	JMAC

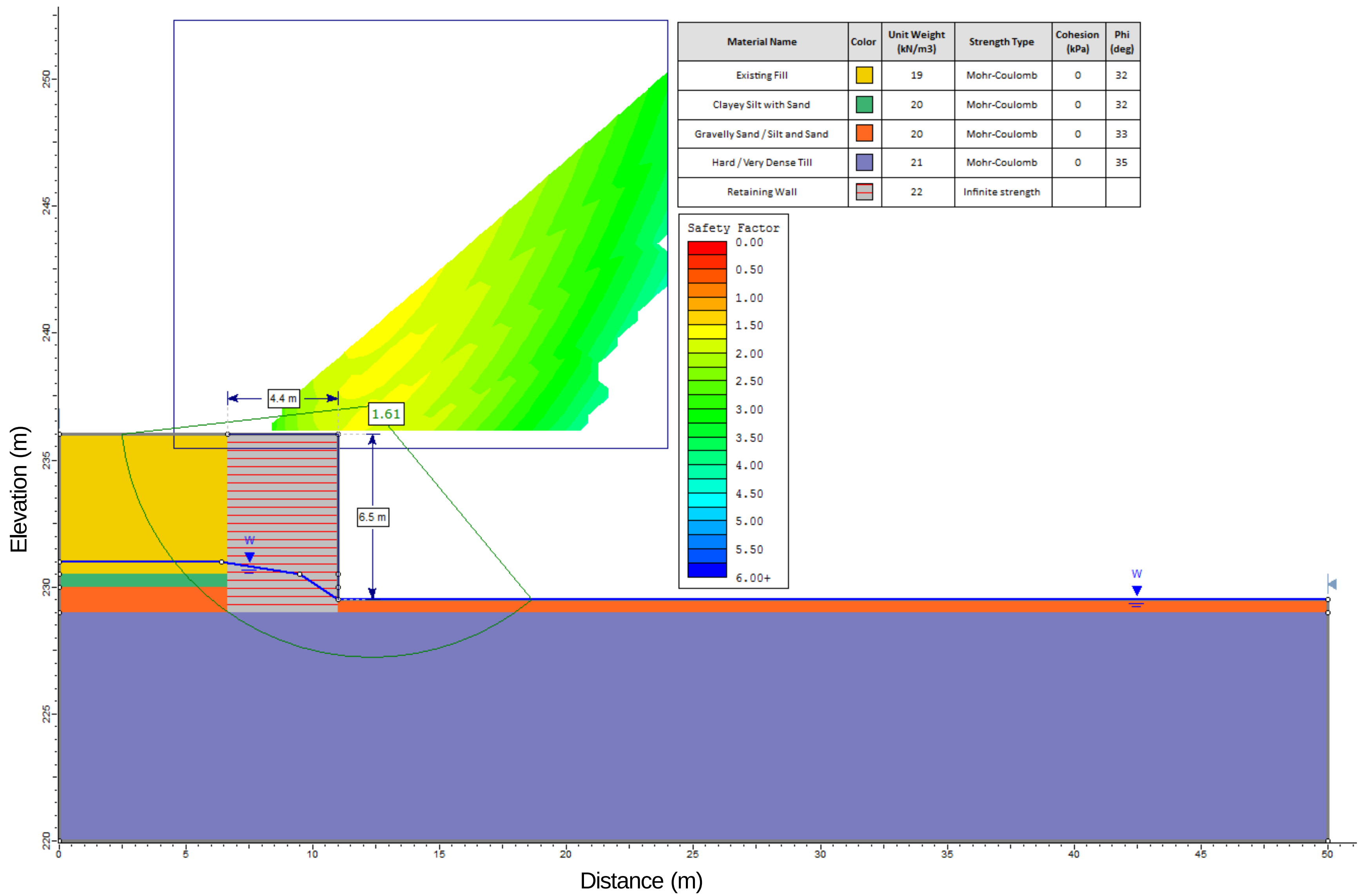
PROJECT
HIGHWAY 48 – LITTLE ROUGE RIVER CULVERT (SITE NO. 37-1195/C), TOWN OF MARKHAM, REGIONAL MUNICIPALITY OF YORK, ONTARIO, AGREEMENT NO. 2016-E-0029

TITLE
STATIC GLOBAL STABILITY ANALYSIS
CONCRETE CANTILEVER WALL ON SHALLOW FOUNDATIONS

PROJECT No.
1671430 WO6

FIGURE 1

1669995_1007_CPRailOverhead_SlopeStability



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Existing Fill		19	Mohr-Coulomb	0	32
Clayey Silt with Sand		20	Mohr-Coulomb	0	32
Gravelly Sand / Silt and Sand		20	Mohr-Coulomb	0	33
Hard / Very Dense Till		21	Mohr-Coulomb	0	35
Retaining Wall		22	Infinite strength		

CLIENT
AECOM / Ministry of Transportation Ontario (MTO)

PROJECT
HIGHWAY 48 – LITTLE ROUGE RIVER CULVERT (SITE NO. 37-1195/C), TOWN OF MARKHAM, REGIONAL MUNICIPALITY OF YORK, ONTARIO, AGREEMENT NO. 2016-E-0029

CONSULTANT
GOLDER

YYYY-MM-DD	2018-11-05
PREPARED	NK
DESIGN	
REVIEW	JMAC
APPROVED	JMAC

TITLE
STATIC GLOBAL STABILITY ANALYSIS
RETAINED SOIL SYSTEM (RSS) WALL

PROJECT No.
1671430 WO6

2

APPENDIX A

Borehole Records

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
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_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

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

(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 $= \sigma'_p / \sigma'_{vo}$

(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

Notes: 1 $\tau = c' + \sigma' \tan\phi'$
2 shear strength = (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

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.)

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, 60º 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 cm^2 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.

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

Compactness 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

(b) Cohesive Soils

Consistency	**c_u, s_u kPa**	**c_u, s_u psf**
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

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**	**Example**
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (non-cohesive (cohesionless)) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand

RECORD OF BOREHOLE No 17-1

SHEET 1 OF 1 **METRIC**

PROJECT 1671430 W06

G.W.P. 2087-17-00

DIST Central HWY 48

DATUM Geodetic

LOCATION N 4867396.4; E 322615.0 MTM NAD 83 ZONE 10 (LAT. 43.946341; LONG. -79.278066)

BOREHOLE TYPE 210 mm O.D., Hollow Stem Augers, Truck-mounted Drill Rig

DATE May 16, 2018

ORIGINATED BY CC

COMPILED BY AM

CHECKED BY NK

ELEV / DEPTH (m)	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
236.0	GROUND SURFACE							
0.0	Gravelly sand, trace organics (FILL) Very dense Brown Moist	1	SS	52				
235.3 / 0.7	Silty sand. some gravel (FILL) Dense Brown Moist	2	SS	39				
234.5 / 1.5	Gravelly sand, some silt (FILL) Very dense Brown Moist	3	SS	50 / 0.14				
234.0 / 2.0	Clayey silt, some sand, trace gravel (FILL) Hard Brown Moist	4A	SS	33				
233.3	Silt and sand, some gravel (FILL) Dense Brown Moist	4B						
233.0 / 3.0	CLAYEY SILT with SAND, trace gravel Very stiff to hard Grey Moist	5	SS	71				
		6	SS	16	4	39	45	12
		7	SS	17				
230.4 / 5.6	Gravelly SAND, trace to some silt, trace clay Dense Grey Moist	8	SS	40	24	61	12	3
228.8 / 7.2	CLAYEY SILT with SAND to SILT and SAND, trace to some clay, trace to some gravel (TILL) Hard/Very dense Grey Moist	9	SS	50/0.14				
		10	SS	100 / 0.15	6	51	36	7
		11	SS	173 / 0.28				
223.7		12	SS	100/0.13	7	51	33	9
12.3	END OF BOREHOLE							

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT W_L, WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Elevation scale: 235, 234, 233, 232, 231, 230, 229, 228, 227, 226, 225, 224

NOTES:

1. Borehole caved to 6.6 m below ground surface upon completion of drilling.

2. Water level measured in open borehole at a depth of 5.2 m below ground surface (Elev. 230.8 m) upon completion of drilling.

GTA-MTO 001 S:\CLIENTS\MTO\HWY_48\02_DATA\GINT\HWY_48.GPJ GAL-GTA.GDT 18-8-21

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

GOLDER

RECORD OF BOREHOLE No 17-2

SHEET 1 OF 1 **METRIC**

PROJECT 1671430 W06
G.W.P. 2087-17-00
LOCATION N 4867430.5; E 322605.9 MTM NAD 83 ZONE 10 (LAT. 43.946648; LONG. -79.278178)
ORIGINATED BY AM
DIST Central HWY 48
BOREHOLE TYPE 89 mm I.D., Casing, Portable Drill Rig
COMPILED BY AM
DATUM Geodetic
DATE May 28, 2018
CHECKED BY NK

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	PLASTIC LIMIT / NATURAL MOISTURE CONTENT / LIQUID LIMIT	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
233.4	GROUND SURFACE							
0.0	TOPSOIL - ORGANIC SILT, some gravel, trace to some sand Compact Dark brown Moist to wet	1	SS	18	233			
		2	SS	22				
232.2 1.2	Silt, some sand, some gravel (FILL) Dense to very dense Brown Moist	3	SS	40	232			
		4A 4B	SS	78			NP	
231.0 2.4	CLAYEY SILT with SAND to SILT and SAND, trace to some clay, some gravel (TILL) Hard/Very dense Brown to grey Moist to wet	5	SS	102	231			
		6	SS	102	230			
		7	SS	150 / 0.10				18 37 33 12
	- Casing refusal on inferred cobbles at 4.5 m depth	8A 8B	SS	150 / 0.07	229			
228.4 5.0	CASING REFUSAL END OF BOREHOLE							

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100
WATER CONTENT (%): 10 20 30 (W_P, W, W_L)

NOTES:

1. Water level measured at a depth of 0.9 m below ground surface (Elev. 232.5 m) during piezometer installation.

2. Water level measured in standpipe piezometer at a depth of 1.0 m bellow ground surface (Elev. 232.4 m) on June 22, 2018.

$+^3, \times^3$: Numbers refer to Sensitivity $O^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No 17-3

SHEET 1 OF 1 **METRIC**

PROJECT 1671430 W06
G.W.P. 2087-17-00
LOCATION N 4867413.0; E 322622.2 MTM NAD 83 ZONE 10 (LAT. 43.946490; LONG. -79.277976)
ORIGINATED BY AM
DIST Central HWY 48
BOREHOLE TYPE 102 mm O.D., Solid Stem Auger, Truck-mounted Drill Rig
COMPILED BY AM
DATUM Geodetic
DATE May 23, 2018
CHECKED BY NK

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
236.1	GROUND SURFACE						
0.0	Sand, trace gravel to gravelly, trace to some silt (FILL) Very dense Brown Moist	1	SS	66		235	
	- 152 mm layer of gravelly clayey silt encountered at 1.6 m	2A 2B 2C	SS	50		234	
233.9 / 2.2	Sandy clayey silt to silt, some gravel (FILL) Hard Brown Moist	3	SS	100 / 0.13			
		4	SS	36		233	
232.3 / 3.8	CLAYEY SILT with SAND, trace to some gravel Very stiff to hard Grey-brown Moist	5	SS	28	∇	232	
		6	SS	50/0.13		231	9 34 38 19
230.6 / 5.5	SILT and SAND, trace clay Dense Brown Wet	7	SS	38		230	0 46 53 1
229.0 / 7.1	CLAYEY SILT with SAND to SILT and SAND, trace to some clay, trace to some gravel (TILL) Hard/Very dense Brown to grey below 11.5 m Moist - 0.1 m sand layer encountered at 7.8 m depth	8	SS	70/0.10		229, 228	
		9	SS	70/0.10		227, 226	
	- Augers grinding on inferred cobble/ boulder at 11.0 m depth	10	SS	70/0.10		225, 224	
223.8 / 12.3	END OF BOREHOLE	11	SS	70/0.10			

NOTE:

1. Water level in open borehole measured at a depth of 4.1 m below ground surface (Elev. 232.0 m) upon completion of drilling.

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED: 20 40 60 80 100
PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L
WATER CONTENT (%): 10 20 30
UNIT WEIGHT γ kN/m³

$+^3, \times^3$: Numbers refer to Sensitivity
O $^{3\%}$ STRAIN AT FAILURE

GOLDER

RECORD OF BOREHOLE No 17-4

SHEET 1 OF 1

METRIC

PROJECT 1671430 W06

G.W.P. 2087-17-00

LOCATION N 4867387.2; E 322637.2 MTM NAD 83 ZONE 10 (LAT. 43.946257; LONG. -79.277790)

ORIGINATED BY AM

DIST Central HWY 48

BOREHOLE TYPE 89 mm I.D., Casing, Portable Drill Rig

COMPILED BY AM

DATUM Geodetic

DATE May 24, 2018

CHECKED BY NK

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
231.8	GROUND SURFACE										
0.0	Topsoil (FILL)										
0.1	Silt and sand, trace to some gravel, trace clay, trace organics (FILL) Loose to dense Grey Moist - Split spoon bouncing at 1.4 m - 50 mm gravel layer encountered at 1.5 m	1	SS	6							
		2	SS	34	231	~17	NP				
		3A / 3B	SS	114		~12					
230.2 / 1.6	SILT and SAND, trace to some clay, trace gravel (TILL) Very dense Grey Wet	4	SS	76	230						
		5	SS	203	229	~10	NP	5	52	36	7
228.6 / 3.2	CASING REFUSAL END OF BOREHOLE	6	SS	300/0.13							

NOTES:

1. Water level in open borehole measured at a depth of 0.2 m below ground surface (Elev. 231.6 m) upon drilling completion.

2. Water level measured in standpipe piezometer at a depth of 0.1 m bellow ground surface (Elev. 231.7 m) on June 22, 2018.

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; REMARKS & GRAIN SIZE DISTRIBUTION (%)

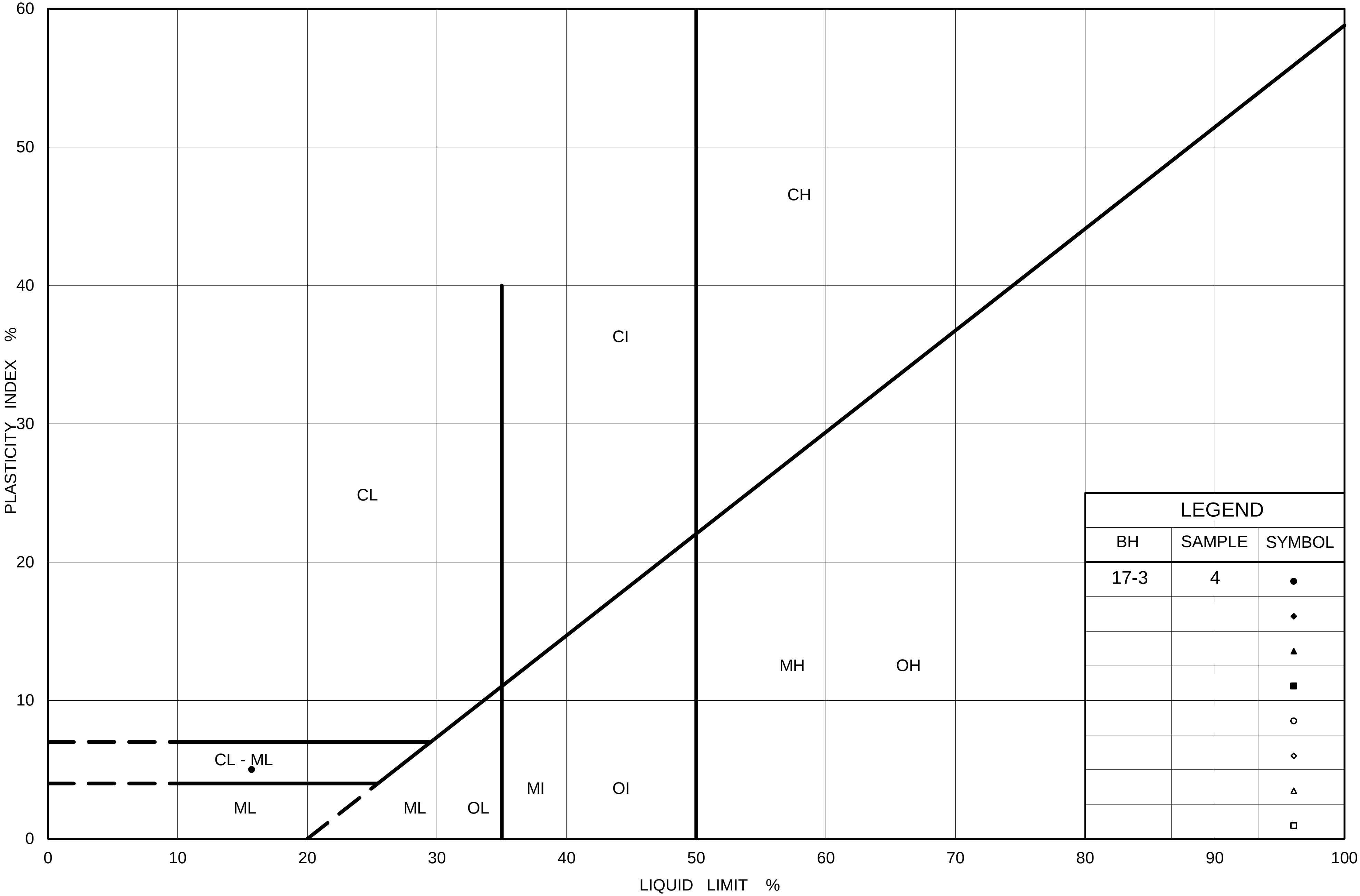
$+^3, \times^3$: Numbers refer to Sensitivity

$\circ^{3\%}$ STRAIN AT FAILURE

APPENDIX B

Geotechnical Laboratory Test Results

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Sandy Clayey Silt - Silt (Fill)

Figure No. B1

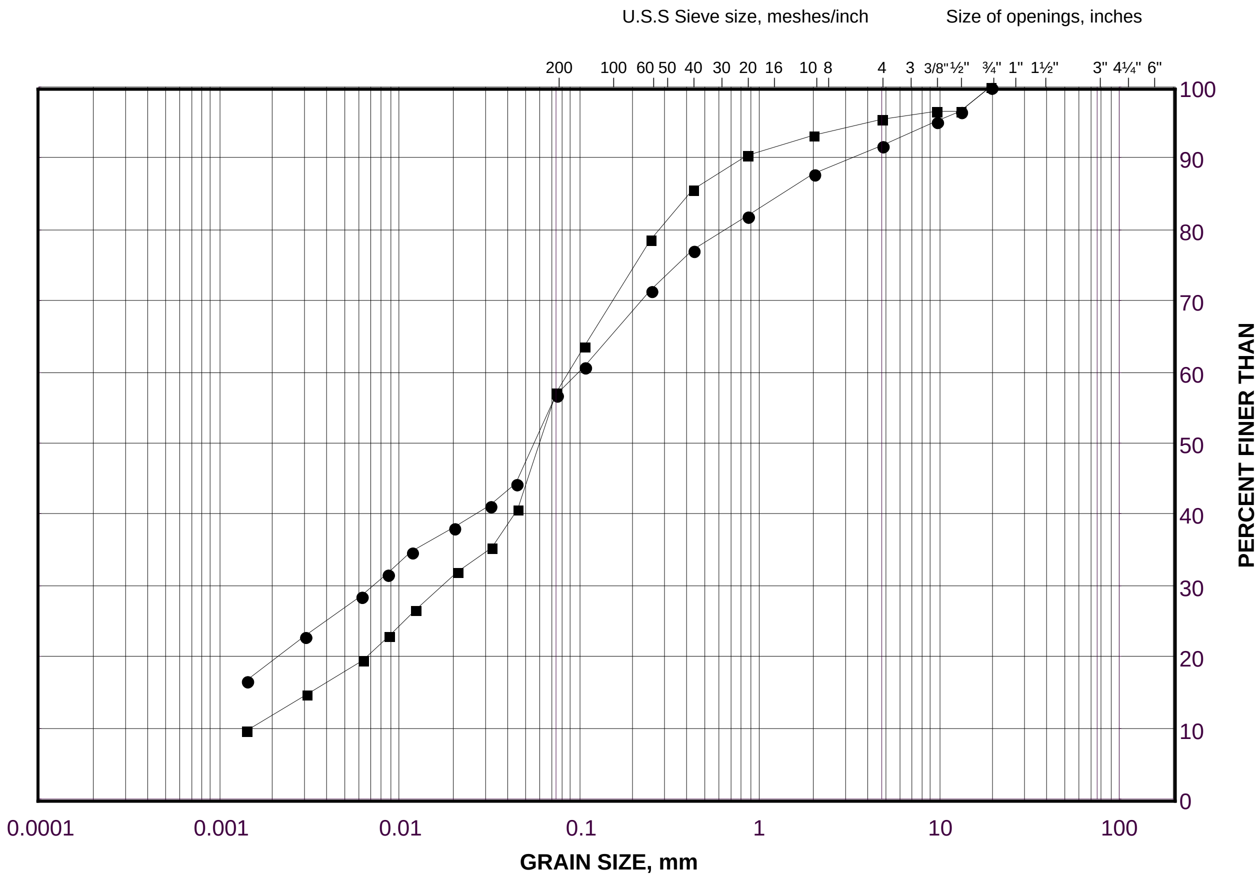
Project No. 1671430

Checked By: NK

GRAIN SIZE DISTRIBUTION

Clayey Silt with Sand

FIGURE B2



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	17-3	6	231.3
■	17-1	6	231.8

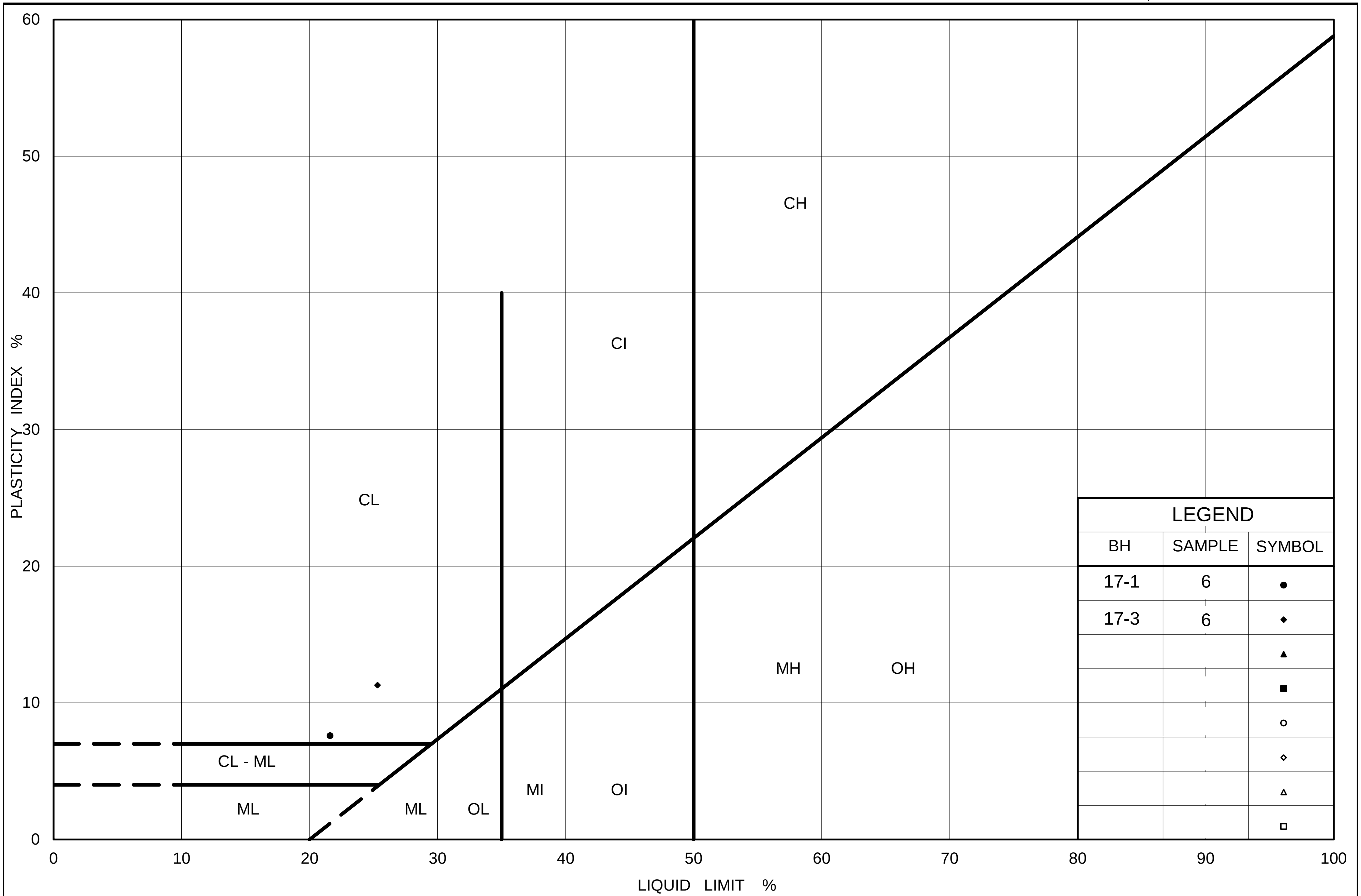
Project Number: 1671430

Checked By: NK

Golder Associates

Date: 07-Aug-18

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Clayey Silt with Sand

Figure No. B3

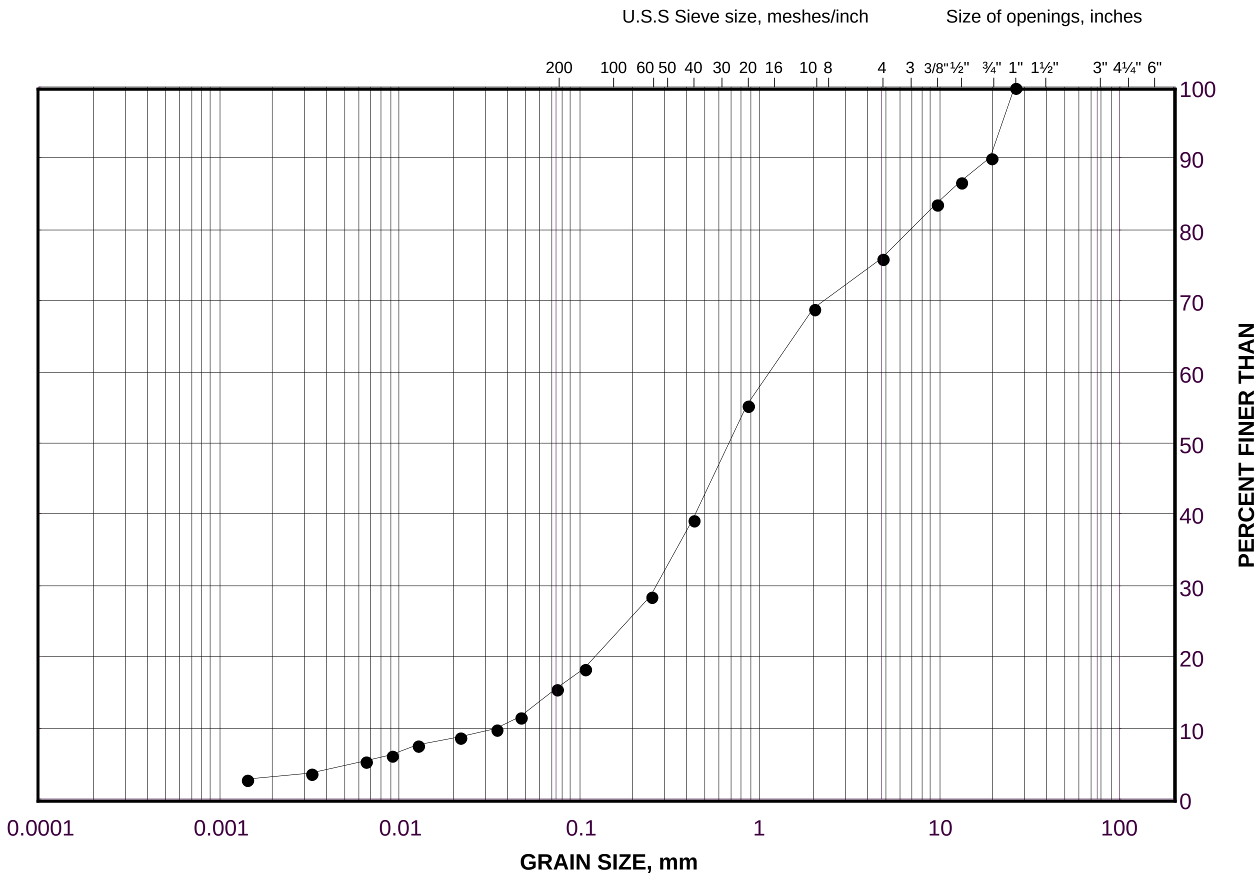
Project No. 1671430

Checked By: NK

GRAIN SIZE DISTRIBUTION

Gravelly Sand

FIGURE B4



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	17-1	8	229.6

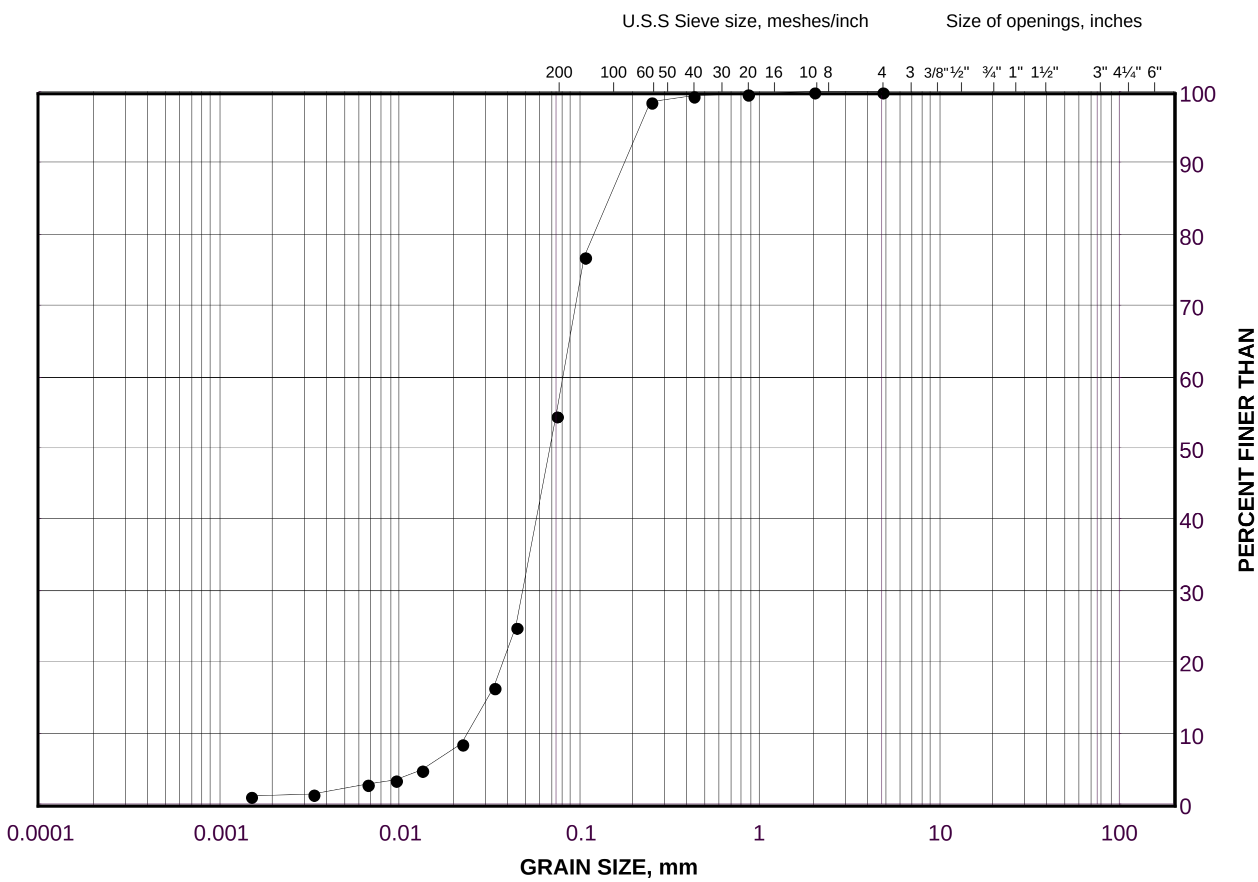
Project Number: 1671430

Checked By: NK

Golder Associates

Date: 07-Aug-18

GRAIN SIZE DISTRIBUTION Silt and Sand	FIGURE B5



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	17-3	7	229.7

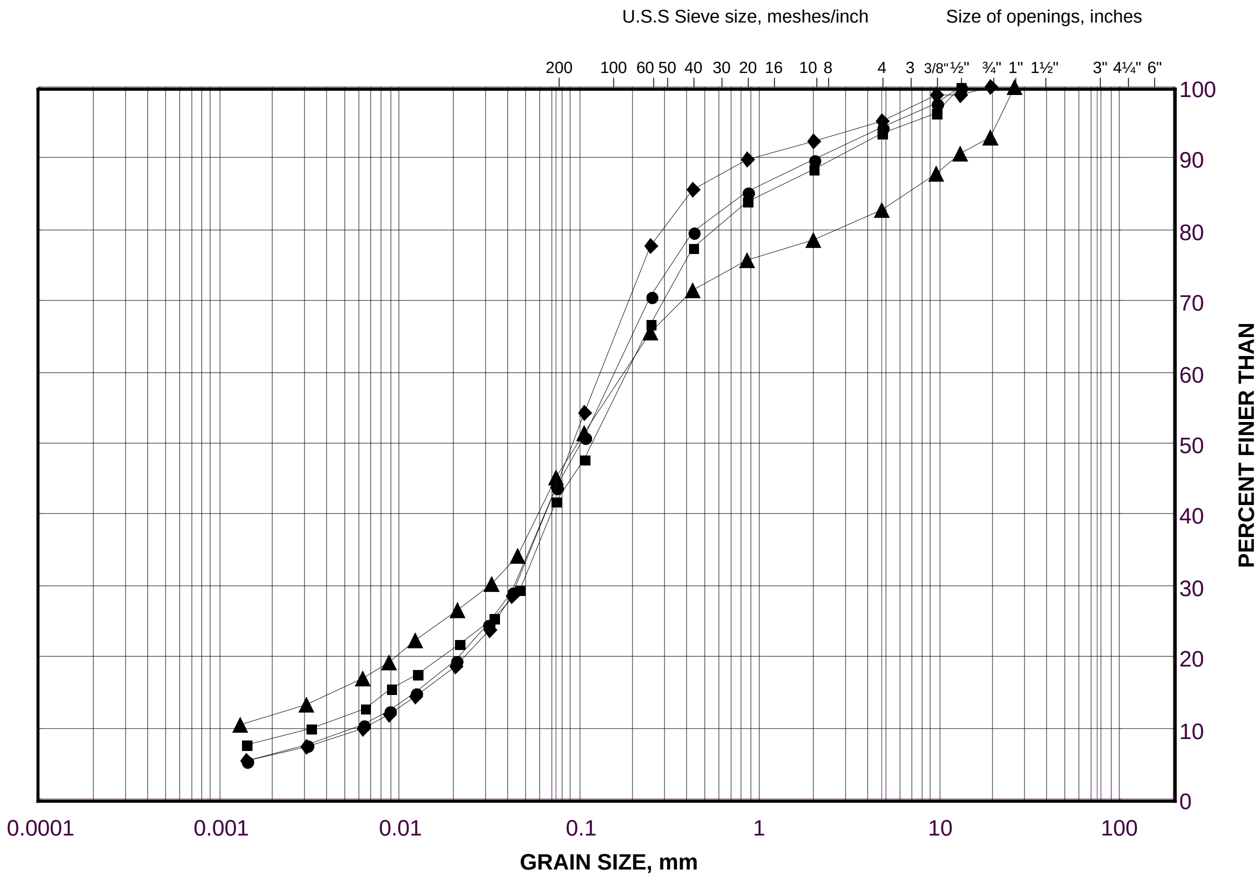
Project Number: 1671430

Checked By: NK

Golder Associates

Date: 07-Aug-18

GRAIN SIZE DISTRIBUTION Clayey Silt with Sand to Silt and Sand (Till)	FIGURE B6



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	17-1	10	226.6
■	17-1	12	223.7
◆	17-4	5	229.1
▲	17-2	7	229.3

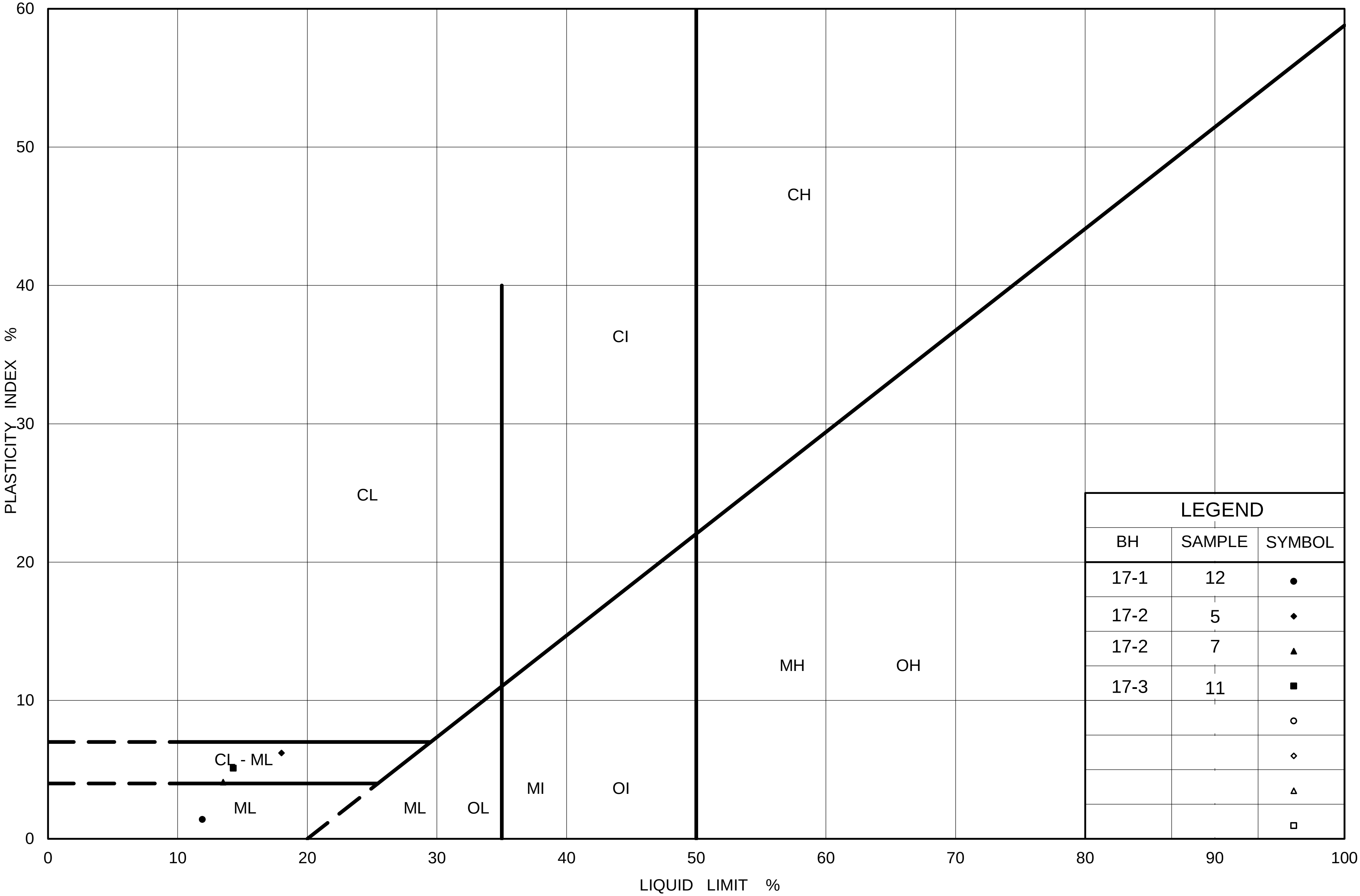
Project Number: 1671430

Checked By: NK

Golder Associates

Date: 07-Aug-18

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Clayey Silt with Sand to Silt and Sand (Till)

Figure No. B7

Project No. 1671430

Checked By: NK

APPENDIX C

Analytical Chemical Test Results

Maxxam
A Bureau Veritas Group Company

Your Project #: 1671430 W0006
Site Location: HWY 48 AND 19TH AVE
Your C.O.C. #: n/a

Attention: Nikol Kochmanova

Golder Associates Ltd
6925 Century Ave
Suite 100
Mississauga, ON
CANADA L5N 7K2

Report Date: 2018/06/15
Report #: R5242750
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B8E2845
Received: 2018/06/12, 15:59

Sample Matrix: Soil
Samples Received: 1

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Reference
Chloride (20:1 extract)	1	N/A	2018/06/15	CAM SOP-00463	EPA 325.2 m
Conductivity	1	N/A	2018/06/15	CAM SOP-00414	OMOE E3530 v1 m
pH CaCl2 EXTRACT	1	2018/06/14	2018/06/14	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	1	2018/06/13	2018/06/15	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	1	N/A	2018/06/15	CAM SOP-00464	EPA 375.4 m

Remarks:

Maxxam Analytics' laboratories are accredited to ISO/IEC 17025:2005 for specific parameters on scopes of accreditation. Unless otherwise noted, procedures used by Maxxam are based upon recognized Provincial, Federal or US method compendia such as CCME, MDDELCC, EPA, APHA.

All work recorded herein has been done in accordance with procedures and practices ordinarily exercised by professionals in Maxxam’s profession using accepted testing methodologies, quality assurance and quality control procedures (except where otherwise agreed by the client and Maxxam in writing). All data is in statistical control and has met quality control and method performance criteria unless otherwise noted. All method blanks are reported; unless indicated otherwise, associated sample data are not blank corrected.

Maxxam Analytics' liability is limited to the actual cost of the requested analyses, unless otherwise agreed in writing. There is no other warranty expressed or implied. Maxxam has been retained to provide analysis of samples provided by the Client using the testing methodology referenced in this report. Interpretation and use of test results are the sole responsibility of the Client and are not within the scope of services provided by Maxxam, unless otherwise agreed in writing.

Solid sample results, except biota, are based on dry weight unless otherwise indicated. Organic analyses are not recovery corrected except for isotope dilution methods.
Results relate to samples tested.
This Certificate shall not be reproduced except in full, without the written approval of the laboratory.
Reference Method suffix “m” indicates test methods incorporate validated modifications from specific reference methods to improve performance.
* RPDs calculated using raw data. The rounding of final results may result in the apparent difference.

Maxxam Analytics International Corporation o/a Maxxam Analytics 6740 Campobello Road, Mississauga, Ontario, L5N 2L8 Tel: (905) 817-5700 Toll-Free: 800-563-6266 Fax: (905) 817-5777 www.maxxam.ca

Your Project #: 1671430 W0006
Site Location: HWY 48 AND 19TH AVE
Your C.O.C. #: n/a

Attention: Nikol Kochmanova

Golder Associates Ltd
6925 Century Ave
Suite 100
Mississauga, ON
CANADA L5N 7K2

Report Date: 2018/06/15
Report #: R5242750
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B8E2845
Received: 2018/06/12, 15:59

Encryption Key

Please direct all questions regarding this Certificate of Analysis to your Project Manager.
Ema Gitej, Senior Project Manager
Email: EGitej@maxxam.ca
Phone# (905)817-5829
==
Maxxam has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per section 5.10.2 of ISO/IEC 17025:2005(E), signing the reports. For Service Group specific validation please refer to the Validation Signature Page.

Maxxam
A Bureau Veritas Group Company

Maxxam Job #: B8E2845
Report Date: 2018/06/15

Golder Associates Ltd
Client Project #: 1671430 W0006
Site Location: HWY 48 AND 19TH AVE
Sampler Initials: AM

SOIL CORROSIVITY PACKAGE (SOIL)

Maxxam ID		GYB555			GYB555		
Sampling Date		2018/05/24			2018/05/24		
COC Number		n/a			n/a		
	UNITS	**17-4 SA#4**	**RDL**	**QC Batch**	**17-4 SA#4 Lab-Dup**	**RDL**	**QC Batch**
Calculated Parameters							
Resistivity	ohm-cm	2900		5578503			
Inorganics							
Soluble (20:1) Chloride (Cl)	ug/g	110	20	5582214	120	20	5582214
Conductivity	umho/cm	349	2	5582341			
Available (CaCl2) pH	pH	8.00		5580453			
Soluble (20:1) Sulphate (SO4)	ug/g	78	20	5582215	72	20	5582215

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch
Lab-Dup = Laboratory Initiated Duplicate

Maxxam Job #: B8E2845
Report Date: 2018/06/15

Golder Associates Ltd
Client Project #: 1671430 W0006
Site Location: HWY 48 AND 19TH AVE
Sampler Initials: AM

TEST SUMMARY

Maxxam ID: GYB555
Sample ID: 17-4 SA#4
Matrix: Soil

Collected: 2018/05/24
Shipped:
Received: 2018/06/12

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5582214	N/A	2018/06/15	Deonarine Ramnarine
Conductivity	AT	5582341	N/A	2018/06/15	Tahir Anwar
pH CaCl2 EXTRACT	AT	5580453	2018/06/14	2018/06/14	Tahir Anwar
Resistivity of Soil		5578503	2018/06/15	2018/06/15	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5582215	N/A	2018/06/15	Deonarine Ramnarine

Maxxam ID: GYB555 Dup
Sample ID: 17-4 SA#4
Matrix: Soil

Collected: 2018/05/24
Shipped:
Received: 2018/06/12

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5582214	N/A	2018/06/15	Deonarine Ramnarine
Sulphate (20:1 Extract)	KONE/EC	5582215	N/A	2018/06/15	Deonarine Ramnarine

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GENERAL COMMENTS

Each temperature is the average of up to three cooler temperatures taken at receipt

Package 1	22.3°C

Results relate only to the items tested.

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Maxxam Job #: B8E2845
Report Date: 2018/06/15

QUALITY ASSURANCE REPORT

Golder Associates Ltd
Client Project #: 1671430 W0006
Site Location: HWY 48 AND 19TH AVE
Sampler Initials: AM

			Matrix Spike		SPIKED BLANK		Method Blank		RPD	
QC Batch	**Parameter**	**Date**	**% Recovery**	**QC Limits**	**% Recovery**	**QC Limits**	**Value**	**UNITS**	**Value (%)**	**QC Limits**
5580453	Available (CaCl2) pH	2018/06/14			101	97 - 103			0.21	N/A
5582214	Soluble (20:1) Chloride (Cl)	2018/06/15	NC	70 - 130	102	70 - 130	<20	ug/g	2.3	35
5582215	Soluble (20:1) Sulphate (SO4)	2018/06/15	NC	70 - 130	109	70 - 130	<20	ug/g	8.1	35
5582341	Conductivity	2018/06/15			100	90 - 110	<2	umho/cm	0.50	10

N/A = Not Applicable

Duplicate: Paired analysis of a separate portion of the same sample. Used to evaluate the variance in the measurement.

Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate sample matrix interference.

Spiked Blank: A blank matrix sample to which a known amount of the analyte, usually from a second source, has been added. Used to evaluate method accuracy.

Method Blank: A blank matrix containing all reagents used in the analytical procedure. Used to identify laboratory contamination.

NC (Matrix Spike): The recovery in the matrix spike was not calculated. The relative difference between the concentration in the parent sample and the spike amount was too small to permit a reliable recovery calculation (matrix spike concentration was less than the native sample concentration)

VALIDATION SIGNATURE PAGE

The analytical data and all QC contained in this report were reviewed and validated by the following individual(s).

Brad Newman, Scientific Service Specialist

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CAM FCD-01191/3

CHAIN OF CUSTODY RECORD

Page ____ of ____

Invoice Information		Report Information (if differs from invoice)		Project Information (where applicable)	
Company Name:	Golder Associates Ltd.	Company Name:		Quotation #:	
Contact Name:	Nikol Kochmanová	Contact Name:		P.O. #/ AFE#:	
Address:	6925 Century Ave Suite 100 Mississauga ON L5N 7K2	Address:		Project #:	1671430-W0006
				Site Location:	Hwy 48 and 14th Ave
Phone:	905-567-4444 Fax: 905-567-6561	Phone:	Fax:	Site #:	
Email:	Nikol_Kochmanova@golder.com	Email:		Sampled By:	AM

Turnaround Time (TAT) Required

[X] Regular TAT (5-7 days) Most analyses

PLEASE PROVIDE ADVANCE NOTICE FOR RUSH PROJECTS

Rush TAT (Surcharges will be applied)

[ ] 1 Day [ ] 2 Days [ ] 3-4 Days

Date Required:

Rush Confirmation #:

MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE MAXXAM DRINKING WATER CHAIN OF CUSTODY

Regulation 153

[ ] Table 1 [ ] Res/Park [ ] Med/ Fine
[ ] Table 2 [ ] Ind/Comm [ ] Coarse
[ ] Table 3 [ ] Agri/ Other
[ ] Table ____

FOR RSC (PLEASE CIRCLE) Y / N

Other Regulations

[ ] CCME [ ] Sanitary Sewer Bylaw
[ ] MISA [ ] Storm Sewer Bylaw
[ ] PWQO Region
[ ] Other (Specify)
[ ] REG 558 (MIN. 3 DAY TAT REQUIRED)

Include Criteria on Certificate of Analysis: Y / N

SAMPLES MUST BE KEPT COOL (< 10 °C) FROM TIME OF SAMPLING UNTIL DELIVERY TO MAXXAM

Analysis Requested

	SAMPLE IDENTIFICATION	DATE SAMPLED (YYYY/MM/DD)	TIME SAMPLED (HH:MM)	MATRIX	# OF CONTAINERS SUBMITTED	FIELD FILTERED (CIRCLE) Metals / Hg / CrVI	BTEX/ PHC F1	PHCs F2 - F4	VOCs	REG 153 METALS & INORGANICS	REG 153 ICPMS METALS	REG 153 METALS (Hg, Cr VI, ICPMS Metals, HWS - B)	Corrosivity Package (Cl, SO4, pH, resistivity)	no sulphide and redox potential						HOLD- DO NOT ANALYZE
1	17-4 SA#4	2018/05/24	AM	SOIL									X							
2																				
3																				
4																				
5																				
6																				
7																				
8																				
9																				
10																				

LABORATORY USE ONLY

CUSTODY SEAL Y / N		COOLER TEMPERATURES
Present	Intact	
		22 / 22 / 23

COOLING MEDIA PRESENT: Y / N

COMMENTS

RELINQUISHED BY: (Signature/Print)	DATE: (YYYY/MM/DD)	TIME: (HH:MM)	RECEIVED BY: (Signature/Print)	DATE: (YYYY/MM/DD)	TIME: (HH:MM)
Alex MacMillan	2018/06/12	16:00	Junaid Usman	18/06/12	15:59

12-Jun-18 15:59
Ema Gitej
B8E2845
VMK ENV-1084

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APPENDIX D

Non-Standard Special Provisions

<u>DEWATERING STRUCTURE EXCAVATIONS</u> - Item No.

Special Provision No. FOUN0003 March 8, 2018

Amendment to OPSS 902, November 2010

OPSS 902, November 2010, Construction Specification for Excavating and Backfilling - Structures is amended as follows:

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.

Dewatering System 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 two 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 Working Drawings

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

902.04.02.02 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 300 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.

902.04.02.03 Milestone Inspections

Clause 902.04.02.03 of OPSS 902 is deleted in its entirety.

902.07 CONSTRUCTION

Subsection 902.07.04 of OPSS 902 is deleted in its entirety and replaced with the following:

902.07.04 Dewatering Structure Excavation

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 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

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