September 18, 2015

FOUNDATION INVESTIGATION AND DESIGN REPORT

Retaining Wall (Station 15+350 to 15+460)
Highway 400 Widening From North of King Road to South Canal Bank Road
King City, Ontario
G.W.P. 2835-02-00

Submitted to:
AECOM
30 Leek Crescent, 4th Floor
Richmond Hill, Ontario
L4B 4N4

GEOCRES No. 30M13-213

Report Number: 09-1111-0018-11

Distribution:

4 Copies - MTO Central Region
1 Copy - MTO Foundations Section
1 Copy - AECOM
1 Copy - Golder Associates Ltd.

REPORT

Golder Associates

Table of Contents

PART A

FOUNDATION INVESTIGATION REPORT
RETAINING WALL (STATION 15+350 TO 15+460)
HIGHWAY 400 WIDENING FROM NORTH OF KING ROAD
TO SOUTH CANAL BANK ROAD
G.W.P. 2835-02-00

1.0 INTRODUCTION

Golder Associated Ltd. (Golder) has been retained by AECOM on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services in support of the detail design for the widening of Highway 400 from north of King Road to north of South Canal Bank Road in the Regional Municipality of York, Ontario.

The Terms of Reference for the foundation engineering services are outlined in the Terms of Reference of MTO's Request for Proposal, dated May 2008 that form part of the Consultant's Agreement (Number 2007 E 0002) for this project, and in subsequent change requests. The work has been carried out in accordance with Golder's Supplemental Specialty Quality Control Plan for this project, dated October 2010.

This report addresses the foundation investigation carried out for the detail design of an approximately 110 m long retaining wall along the west side Highway 400 southbound lanes (SBL), between 15th Sideroad and 16th Sideroad in King City, in the Regional Municipality of York.

2.0 SITE DESCRIPTION

The proposed retaining wall is located on the west side of Highway 400 SBL between approximately Station 15+350 and Station 15+460; this is approximately 700 m south of 16th Sideroad.

In general, the topography in the area of the overall project site consists of rolling terrain including agricultural fields and densely treed areas. Commercial facilities are also found adjacent to the Highway 400 corridor. The area around the retaining wall is swampy and flat near the central portion of the wall, and slopes upward toward the north and south limits of the wall, with the existing ground surface at the toe of the slope varying from about Elevation 303 m near the south end, to Elevation 302 m in the central portion, to Elevation 308 m near the north end. The existing Highway 400 grade slopes downward to the south, from about Elevation 315 m to Elevation 311 m within the limits of the proposed retaining wall area.

The existing Highway 400 embankment is approximately 7.5 m in height (relative to the ground surface at the west embankment toe) at the north and south limits of the proposed wall, increasing to a maximum height of approximately 11 m in the central section. The existing west embankment side slope is sloped at approximately 2 horizontal to 1 vertical (2H:1V).

3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out in December 2010, at which time three boreholes, designated as Boreholes RW1-1 to RW1-3, were advanced.

The field investigation was carried out using a Diedrich D-25 track-mounted drill rig, supplied and operated by Walker Drilling Ltd. of Utopia, Ontario. The boreholes were advanced to depths ranging from 11.3 m to 14.3 m below the existing ground surface near the highway embankment toe. The boreholes were advanced using either 108 mm outer diameter continuous flight hollow stem augers, or 108 mm diameter continuous flight solid stem augers. Soil samples were obtained at 0.75 m and 1.5 m intervals of depth, using 50 mm outside diameter split-spoon samplers driven by an automatic hammer, in accordance with the Standard Penetration Test (SPT) procedure. (ASTM D1586-08a – Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of the Soil).

The groundwater conditions in the open boreholes were observed during and immediately following the drilling operations. A standpipe piezometer was installed in Borehole RW1-3 to permit monitoring of the water level at this site. The piezometer consists of a 25 mm diameter PVC pipe, with a slotted screen sealed at a select depth within the borehole. The borehole and annulus surrounding the piezometer pipe above the screen sand pack were backfilled to the surface with bentonite pellets/grout. Piezometer installation details and water level readings are provided on the borehole records in Appendix A. All boreholes in which standpipe piezometers were not installed were backfilled with bentonite upon completion in accordance with Ontario Regulation 903 (as amended).

The field work was observed by a member of Golder's engineering staff, who located the boreholes, arranged for the clearance of underground services, observed the drilling, sampling and in situ testing operations, logged the boreholes, and examined and cared for the soil samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to Golder's Mississauga geotechnical laboratory where the samples underwent further visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected samples. The results of the geotechnical laboratory testing are presented on the borehole records and included in Appendix B.

The borehole locations were surveyed by Callon Dietz, a licensed land surveyor retained by URS. The borehole locations, including MTM NAD 83 northing and easting coordinates, and the ground surface elevations referenced to geodetic datum, are presented on the borehole records in Appendix A and are summarized below.

Borehole	Location (MTM NAD 83)		Ground Elevation (m)	Depth Drilled (m)
	Northing	Easting		
RW1-1	4,867,765.6	298,897.5	303.3	14.3
RW1-2	4,867,809.7	298,885.8	302.1	14.3
RW1-3	4,867,853.6	298,880.7	304.9	11.3

4.0 SUBSURFACE CONDITIONS

4.1 Regional Geology

The 23 km section of Highway 400 included in this overall highway widening project traverses, in a south–north direction, the physiographic regions known as South Slope, Oak Ridges Moraine and Simcoe Lowlands, according to *The Physiography of Southern Ontario (Chapman and Putman, 1984)*[1]. Along Highway 400, the South Slope is present south of King Road, the Oak Ridge Moraines extends from north of King Road to south of Highway 9 and the Simcoe Lowlands occupy a 4 km wide strip extending from south of Highway 9 to Holland River. This retaining wall site is located within the Oak Ridges Moraine physiographic region.

[1] 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.

The surficial soils of the South Slope region are generally cohesive tills. The Oak Ridges Moraine predominately consists of sand and gravel, although in the King Township area these soils are often overlain by till. It is understood that during grading for the initial construction of Highway 400 in this area, deep cuts exposed up to about 10 m of till overlying the sands and gravels.

The Holland River valley, which crosses Highway 400 in the vicinity of Highway 9 and South Canal Road, is located within the Simcoe Lowlands region. This valley extends to the southwest from Cook Bay at the south end of Lake Simcoe, and was once a shallow extension of the lake. The floor of the valley consists of peat, soft clays and loose sands. It is understood that during initial construction of Highway 400, a layer of peat about 2 m to 3 m thick was removed in order to construct the road upon the underlying sand and clay.

4.2 Subsurface Conditions

As part of the subsurface investigation, three boreholes were advanced in the area of the proposed retaining wall location. The detailed subsurface soil and groundwater conditions encountered in the boreholes, and the results of in situ and laboratory testing, are presented on the borehole records contained in Appendix A. The results of geotechnical laboratory testing are also presented in Appendix B.

The stratigraphic boundaries shown on the borehole records 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. The interpreted stratigraphic profile along the retaining wall, shown on Drawing 1, is a simplification of the subsurface conditions. Variation in the stratigraphic boundaries between and beyond boreholes will exist and is to be expected.

In summary, the subsoils encountered along the proposed retaining wall generally consist of a surficial layer of topsoil and/or fill and an upper deposit of silty sand to silt, or clayey silt; in the central portion of the retaining wall, a layer of organic silt was encountered in Borehole RW1-2. These deposits are underlain by clayey silt till, which contains clayey silt interlayers; a lower silt deposit was encountered in one of the boreholes. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following subsections.

4.2.1 Topsoil

Approximately 200 mm of topsoil was encountered immediately below the existing ground surface in Borehole RW1-1.

4.2.2 Fill

Fill material was encountered underlying the topsoil in Borehole RW1-1 and immediately below the ground surface in Borehole RW1-2. The fill is 0.7 m and 1.9 m thick, with the base of the fill extending to about Elevation 302.4 m and Elevation 300.2 m in Boreholes RW1-1 and RW1-2, respectively.

In Borehole RW1-1, the fill is generally non-cohesive and comprised of silty sand, containing zones of clayey silt. In Borehole RW1-2, the upper 0.5 m of the fill is cohesive and consists of silty clay containing trace sand and rootlets; this is underlain by about 1.4 m of non-cohesive sand and silt fill, containing zones of clayey silt.

The natural water content measured on one sample of the non-cohesive fill was about 13 per cent, and the natural water content measured on one sample of the cohesive fill was about 31 per cent.

The Standard Penetration Test (SPT) "N"-values measured within the non-cohesive fill ranged from 7 to 17 blows per 0.3 m of penetration, suggesting that the material has a loose to compact relative density. An SPT "N"-value of 10 blows per 0.3 m of penetration was measured within the cohesive portion of the fill, suggesting that the cohesive fill has a stiff consistency.

4.2.3 Upper Clayey Silt

A 1.6 m thick upper layer of clayey silt was encountered immediately below the existing ground surface at the location of Borehole RW1-3, near the north limit of the retaining wall. The surface of this deposit was encountered at Elevation 304.9 m, and its base at Elevation 303.3 m.

The deposit is comprised of clayey silt with sand, containing rootlets and organic matter. One water content of approximately 22 per cent was measured on this deposit.

SPT "N" values of 4 and 9 blows per 0.3 m were measured in this deposit, suggesting a firm to stiff consistency.

4.2.4 Upper Silty Sand to Silt and Organic Silt

Within the "low point" in the central portion of the proposed retaining wall (refer to the interpreted stratigraphic section on Drawing 1), a 1.5 m thick layer of organic silt was encountered immediately below the fill, extending between Elevation 300.2 m and 298.7 m. This organic silt is underlain by about 2.2 m of low plasticity silt that extends to a total depth of 5.6 m (Elevation 296.5 m). Toward the south limit of the proposed wall, in Borehole RW1-2, an approximately 3.5 m thick layer of silty sand was encountered below the fill, and atop the till deposit; the base of the silty sand was encountered at approximately Elevation 298.9 m in this location.

This upper deposit varies in composition from silty sand to silt containing trace to some sand, to organic silt. A 0.3 m thick layer of clayey silt till was encountered in the lower portion of the silty sand deposit in Borehole RW1-1. The results of grain size distribution tests completed on one sample of silty sand and one sample of organic silt are presented on Figure B1 in Appendix B.

Atterberg limits testing was completed on one sample of the organic silt and measured a plastic limit of about 35 per cent, a liquid limit of about 40 per cent, and a corresponding plasticity index of about 5 per cent; this test result, which is plotted on a plasticity chart on Figure B2 in Appendix B, confirms that this portion of the deposit consists of low plasticity organic silt. Atterberg limits testing was also completed on a sample of the silt from Borehole RW1-2, and measured a plastic limit of about 14 per cent, a liquid limit of about 17 per cent, and a plasticity index of 3 per cent; this result, which is also shown on the plasticity chart on Figure B2, confirms that the tested sample is a silt of slight plasticity. The natural water content measured on samples of the silty sand to silt varied from about 16 to 20 per cent, and the natural water content measured on samples of the organic silt ranged from about 27 to 44 per cent. An organic content test was carried out on one sample of the organic silt and measured an organic content of approximately 4 percent.

The measured SPT "N" values in the organic silt layer are 3 and 4 blows per 0.3 m of penetration, suggesting a soft to firm consistency. The SPT "N"-values recorded within the silty sand to silt deposit range from 8 to 17 blows per 0.3 m of penetration, suggesting that this deposit has a loose to compact relative density.

4.2.5 Clayey Silt Till

A deposit of brown to grey cohesive till was encountered below the upper silty sand to silt in Boreholes RW1-1 and RW1-2, and below the upper clayey silt in Borehole RW1-3. The surface of the till was encountered at approximately Elevation 298.9 m near the south limit of the wall, dipping to about Elevation 296.5 m in the central portion of the wall, and rising to about Elevation 303.3 m near the north limit of the wall (refer to the interpreted stratigraphic profile on Drawing 1).

The cohesive till deposit is comprised of clayey silt, trace to some sand, and trace gravel. The results of grain size distribution tests completed on four samples of the till are presented on Figure B3 in Appendix B.

Atterberg limits testing was conducted on five samples of the till and measured plastic limits of about 12 to 16 per cent, liquid limits of about 22 to 26 per cent, and plasticity indices of about 7 to 10 per cent. These test results, which are plotted on a plasticity chart on Figure B4 in Appendix B, confirm that the deposit consists of clayey silt of low plasticity. The natural water content measured on selected samples of the till ranges from about 14 to 18 per cent, near the plastic limit for the material.

The SPT "N"-values measured within the clayey silt till range from 12 to 55 blows per 0.3 m of penetration. The SPT "N" value of 12 blows per 0.3 m of penetration was measured immediately below the upper clayey silt, at the top of the till deposit in Borehole RW1-3. All other SPT "N" values were generally above 20 blows per 0.3 m of penetration, suggesting that the clayey silt till typically has a very stiff to hard consistency.

4.2.6 Clayey Silt Interlayers

Clayey silt interlayers were encountered within the till deposit in Boreholes RW1-1 and RW1-3 (refer to the stratigraphic profile on Drawing 1). The interlayer is approximately 4.6 m thick in Borehole RW1-1, where it was fully penetrated, and at least 4.1 m thick in Borehole RW1-3; the borehole was terminated within this layer at that location. The interlayers extend from a surface elevation of approximately 297.7 m in both boreholes, to 293.1 m in Borehole RW1-1 and to below 293.6 m in Borehole RW1-3.

The deposit consists of clayey silt containing trace sand; sand seams were noted within the samples in Borehole RW1-1. The results of grain size distribution tests completed on two samples of the clayey silt interlayer are presented on Figure B5 in Appendix B.

Atterberg limits testing was completed on two samples of the interlayers and measured plastic limits of about 17 per cent, liquid limits of 22 and 34 per cent, and plasticity indices of 5 and 17 per cent; these results, which are plotted on a plasticity chart on Figure B6 in Appendix B, confirm that the tested samples of the interlayers consist of clayey silt of low plasticity. The natural water content measured on selected samples varies from about 17 to 30 per cent.

The SPT "N"-values recorded within these interlayers range from about 8 to 19 blows in Borehole RW1-1, and about 25 to 33 blows in Borehole RW1-3, suggesting a stiff to hard consistency.

4.2.7 Lower Silt

A lower silt layer was encountered below the clayey silt till in Borehole RW1-2, at a depth of 9.2 m (Elevation 292.9 m). The borehole was terminated within the lower silt after penetrating it for a thickness of 5.1 m; the lower silt may represent an interlayer within the till, or a deposit underlying the till.

This deposit consists of silt containing trace clay. The result of a grain size distribution test completed on one sample of the deposit is shown on Figure B7 in Appendix B. An Atterberg limits test was completed on one selected sample of the lower silt, and confirmed that the deposit is non-plastic. The natural water content measured on four samples of the deposit range from about 17 to 19 per cent.

The SPT "N"-values measured within the lower silt range from 18 to 55 blows per 0.3 m of penetration, suggesting a compact to very dense relative density.

4.2.8 Groundwater Conditions

The groundwater levels in the open boreholes were measured upon completion of drilling operations. A standpipe piezometer was installed in Borehole RW1-3, sealed within the clayey silt interlayer in the till deposit, to permit monitoring of the groundwater level at this site. Details of the piezometer installation and measured groundwater levels are shown on the borehole record in Appendix A. The groundwater levels recorded in the open boreholes and piezometers are summarized below.

Borehole No.	Ground Surface Elevation (m)	Depth to Water Level (m)	Groundwater Elevation (m)	Date	Comments
RW1-1	303.3	2.5*	300.8*	December 20, 2010	Open Borehole
RW1-2	302.1	1.1*	301.0*	December 21, 2010	Open Borehole
RW1-3	304.9	0.2* 1.5 0.7	304.7* 303.4 304.2	December 21, 2010 January 2, 2011 July 4, 2011	Open Borehole Piezometer Piezometer

* Water level measurements in open borehole may not represent stabilized groundwater level.

In addition to the above water level observations, samples from the upper soil deposits in Boreholes RW1-1 and RW1-2 were observed to be wet below a depth of approximately 1 m to 1.5 m; this may represent a "perched" water table within the upper silty sand to silt and organic silt soils at these locations.

The groundwater level at the site is expected to fluctuate seasonally in response to changes in precipitation and snow melt, and is expected to be higher during the spring and periods of precipitation.

5.0 CLOSURE

This Foundation Investigation Report was prepared by Ms. Haley Schafer, EIT, and reviewed by Ms. Nikol Kochmanova, P.Eng., a geotechnical engineer with Golder. Ms. Lisa Coyne, P.Eng., a Designated MTO Foundations Contact and Principal with Golder, conducted an independent quality control review of this report.

GOLDER ASSOCIATES LTD.

LICENSED PROFESSIONAL ENGINEER
Sep 18/15
N. KOCHMAN
100117863
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
L. C. COYNE
18 SEP 2015
PROVINCE OF ONTARIO

Nikol Kochmanová, P.Eng.
Geotechnical Engineer

Lisa Coyne, P.Eng.
Designated MTO Foundations Contact, Principal

HS/NK/LCC/sm

\\golder.gds\gal\mississauga\active\2009\1111\09-1111-0018 urs - hwy 400 - york\6 - reports\11 - contract 4 retaining wall\09-1111-0018 rpt 15sep18 highway 400 retaining wall.docx

PART B

FOUNDATION DESIGN REPORT
RETAINING WALL (STATION 15+350 TO 15+460)
HIGHWAY 400 WIDENING FROM NORTH OF KING ROAD
TO SOUTH CANAL BANK ROAD
G.W.P. 2835-02-00

6.0 FOUNDATION ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides discussion and foundation engineering recommendations for the proposed retaining wall located on the west side of Highway 400 southbound lanes (SBL), between approximately Station 15+350 and Station 15+460; this proposed retaining wall is located approximately 700 m south of 16th Sideroad in the Regional Municipality of York, Ontario. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface geotechnical investigation completed at this site. The interpretation and recommendations contained in this report are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out detail design of the foundations for the proposed retaining wall.

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 aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.2 Retaining Wall Options

Based on design cross-sections for this section of Highway 400, the existing embankment is approximately 7.5 m in height (relative to the ground surface at the west embankment toe) at the north and south limits of the proposed wall, increasing to a maximum height of approximately 11 m in the central section. The existing embankment is to be widened by approximately 11 m, as measured at the crest of the embankment. Due to property constraints, the proposed retaining wall will be located approximately 5 m to 9 m west of the existing highway embankment toe. The retaining wall is proposed to have a maximum height of approximately 4.5 m at its highest point, near the central portion of the wall, and to be less than 2 m in height near the north and south limits of the wall. Above the top of the wall, the embankment will be constructed with 2H:1V side slopes.

The key issues for design and construction are summarized as follows, and presented in greater detail in subsequent sections of this report:

- **Presence of relatively weak soils:** Relatively weak (loose/soft to firm) surficial soils are present in the footprint of the proposed retaining wall, including an estimated 1.5 m to 2 m of firm to stiff clayey silt in the northern portion of the retaining wall, and soft to firm organic silt extending to a depth of about 3.4 m in the central portion of the retaining wall; these organic soils are further underlain by loose wet silt that extends to a total depth of 5.6 m in the central portion of the retaining wall.

- **Settlement:** As discussed in Section 6.3, assuming no settlement mitigation measures, the estimated settlement under the proposed embankment widening/retaining wall construction will be up to about 50 mm near the south limit, 250 mm in the central section of the wall, and 75 mm near the north limit of the wall. The wall type must accommodate this settlement, or appropriate settlement mitigation measures must be implemented.

- **Global Stability:** As discussed in Section 6.4, assuming the use of conventional fill materials for the embankment widening and retaining wall construction, it will be necessary to remove the soft to firm organic silt layer that is present in the central portion of the proposed wall (corresponding to the highest wall and embankment height) in order to achieve a factor of safety of 1.5 or greater against global instability under static conditions.

Based on the proposed retaining wall geometry, together with settlement and global stability considerations based on the subsurface conditions at this site, various wall and foundation types have been considered. A summary of the advantages and disadvantages associated with each option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, risks and relative costs is provided in Table 1 following the text of this report.

- **Concrete retaining wall on shallow foundations:** Based on the subsurface conditions, a concrete retaining wall supported on shallow foundations is geotechnically feasible at the south and north ends of the proposed wall area; at the north end, the foundations would be required to extend below the firm to stiff surficial clayey silt deposit. However, in the central section of the wall, significant subexcavation of the organic silt would be required to use shallow foundations. Such subexcavation would extend below the groundwater table to a depth of approximately 3.5 m to 4 m, in close proximity to the toe of the 11 m high embankment, and near the limit of the MTO right-of-way, and this will require special constraints on the excavation and backfilling to maintain the stability of the existing embankments during subexcavation work; protection systems and/or a temporary easement may also be required.
- **Concrete retaining wall on deep foundations:** Concrete walls supported on deep foundations (driven piles or caissons) are considered feasible from a geotechnical/foundations perspective. Pile foundations are not considered necessary for support of the south and north portions of the wall, but they may be advantageous in the centre section if it is desirable to limit subexcavation of the soft to firm organic silt. It is noted that based on the current investigation results, data is available only for the design of friction piles with relatively lower geotechnical resistances; further investigation could be undertaken in later stages of design, to refine the geotechnical resistances for a pile-supported retaining wall if this option is selected in detail design.
- **Reinforced soil system (RSS) walls:** RSS walls are geotechnically feasible for the proposed retaining walls at this site. Similar to the requirements for a concrete wall on shallow foundations, subexcavation of the organic soils in the central portion of the wall area will be required, and such excavation would extend below the groundwater table to a depth of approximately 3.5 m to 4 m, in close proximity to the toe of the 11 m high embankment and near the limit of the MTO right-of-way. As for the shallow foundation option, this will require special constraints on the excavation and backfilling to maintain the stability of the existing embankments during subexcavation work; protection systems and/or a temporary easement may also be required. While the subexcavation requirements are similar to a concrete retaining wall option, the resulting RSS wall is considered to be more settlement-tolerant than a concrete retaining wall, particularly if a two-stage RSS wall system is adopted. An RSS wall would also permit incorporation of other settlement mitigation measures, such as the use of lightweight fill materials in its construction.

Based on the above considerations, from a geotechnical/foundations perspective, an RSS wall is considered to be the most practicable and cost-effective option for the new retaining wall at this site. This option would require subexcavation under a portion of the wall to address global stability, together with settlement mitigation measures. As is presented in Section 6.3, the preferred settlement mitigation measure is the use of subexcavation to remove the soft to firm organic silt layer in the central portion of the wall, and the firm to stiff upper clayey silt in the north portion of the wall. This technique also achieves the minimum factor of safety for global stability of the wall.

Alternatively, to minimize the requirements for preloading or other settlement mitigation options within the retaining wall area, a concrete retaining wall supported on deep foundations (at least in the central section) is considered to be a technically feasible option from a geotechnical/foundations perspective. This option would have a shorter construction timeline – less subexcavation (although some subexcavation under the embankment widening outside of the foundation pile cap would likely still be required to ensure good performance of the overlying embankment), and/or no need to wait for a preloading period to be completed or to accommodate other settlement mitigation measures.

6.3 Settlement Under Embankment Widening/Retaining Wall Construction

Based on the design cross-sections provided by AECOM, the highway embankment is proposed to be widened on the west side by approximately 11 m (horizontal distance between existing and proposed outside shoulders at the crest). This will require placement of a vertical thickness of up to approximately 5.5 m to 6.8 m of fill on top of the existing embankment side slope, or above the ground surface at the existing embankment toe. The new retaining wall is proposed to be constructed approximately 5 m to 9 m west of the existing embankment toe, and will be approximately 2 m in height near the north and south limits, rising to about 4.5 m in height in the central portion of the wall.

6.3.1 Estimated Total Settlement

Settlement analyses for the soils below the proposed retaining wall/embankment widening were carried out using both hand calculations and the commercially available computer program *Settle-3D* from Rocscience. The analyses were completed using estimated elastic deformation moduli and consolidation settlement parameters as given below, based on correlations with the field and laboratory test data and engineering judgement from experience with similar soils in this region of Ontario (Bowles, 1984; Kulhawy and Mayne, 1990; Peck et al., 1974).

Soil Deposit	Bulk Unit Weight	Elastic Modulus	P_c'	e_o	C_c	C_r
Embankment fill (existing and new)	21 kN/m^3	20 MPa	–	–	–	–
Soft to firm organic silt (central portion of wall)	18 kN/m^3	–	20 – 40 kPa	1.2	0.4	0.025
Firm to stiff upper clayey silt (north limit of wall)	18 kN/m^3	–	50 kPa	0.7	0.15	0.025
Loose to compact upper silty sand to silt (south and central portion of wall)	19 kN/m^3	15 MPa	–	–	–	–
Very stiff to hard clayey silt till including very stiff to hard interlayer, and compact to dense lower silt	21 kN/m^3	50 MPa	–	–	–	–
Firm to stiff clayey silt interlayer (south portion of wall)	20 kN/m^3	20 MPa	–	–	–	–

Based on the settlement analyses, the total settlement under the proposed retaining wall is summarized as follows, assuming the use of conventional earth or granular fill for the RSS wall construction or concrete wall backfill:

Area	Borehole No.	Total Settlement (mm)	Immediate/Elastic Settlement (mm)	Consolidation Settlement (mm)	Estimated Time to Complete 90% of Settlement
South End	RW1-1	50	50	0	Immediately
Central Portion	RW1-2	250	125	125	4 months
North End	RW1-3	75	25	50	1 month

The above estimates do not include compression of the fill itself, which would occur during and after the construction of the embankment depending on the type of materials used. The magnitude of fill compression may range from 0.5 per cent to 1 per cent of the height of the embankment, assuming approximately 98 per cent compaction of the embankment fill is achieved, relative to the material's standard Proctor maximum dry density. In the case where granular fill is used for embankment construction, settlement of the fill itself is expected to occur essentially during embankment construction, whereas non-granular earth fill materials are expected to exhibit some additional settlement over time.

6.3.2 Settlement Mitigation Options

The estimated post-construction settlements estimated above, based on the use of conventional fill materials, can be reduced using one or more of the following mitigation options:

- **Subexcavation of weak/organic soils:** This would include excavation to remove approximately 1.5 m to 2 m of soft to firm clayey silt near the northern portion of the proposed retaining wall, and organic silt that extends to a depth of approximately 3.4 m (Elevation 298.7 m) in the central portion of the retaining wall. This technique is considered feasible and further discussion is presented in Section 6.3.2.1.
- **Preloading of the widened embankment and retaining wall area:** This technique may be desirable, in conjunction with subexcavation of the weak/organic soils, and further discussion is presented in Section 6.3.2.2.
- **Use of lightweight fill** such as slag or expanded polystyrene (EPS) for construction of the widened portions of the embankment. With subexcavation of the weak/organic soils and the use of preloading, lightweight fill materials are not expected to be required for the proposed retaining wall construction. However, further discussion is provided in Section 6.3.2.3.
- **Use of ground improvement techniques**, such as aggregate piers or deep soil mixing. However, with the use of such a technique, removal of any organic soils would still be required to improve the long-term performance of the retaining wall; therefore this option does not offer any advantage over "simple" subexcavation of the organic silt layer at this site, and it is not addressed in further detail.
- **Use of wick drains** in conjunction with preloading. This technique is most commonly applied where subexcavation is not practical, due to the thickness or depth to the compressible soil deposits, and where the time required to achieve settlement is considered too long. As the soils requiring subexcavation are relatively shallow and organic in nature, and as the settlement period is relatively rapid, wick drains are not considered to be a practicable settlement mitigation measure for this site.

6.3.2.1 Subexcavation of Weak/Organic Soils

Subexcavation is recommended to a depth of 1.5 m to 2 m in the north portion of the wall, and about 3.5 m to 4 m in the central portion of the wall, to remove the firm to stiff clayey silt and the soft to firm organic silt, respectively. The following table summarizes the subexcavation requirements and the estimated settlements under the proposed retaining wall, assuming the use of conventional earth or granular fill for the wall construction:

Area	Subexcavation Depth (m)	Total Settlement (mm)	Immediate/Elastic Settlement (mm)	Consolidation Settlement (mm)
South End	N/A	50	50	0
Central Portion	3.5 – 4	50	50	0
North End	1.5 – 2	<25	<25	0

Staged subexcavation, in strips of limited width, will be required to maintain the stability of the temporary subexcavation in the central portion of the proposed retaining wall, to protect the existing Highway 400 embankment. It is envisaged that this subexcavation will be completed in "wet conditions" (i.e., without dewatering), as follows:

- Removal of the soft to firm organic silt and the overlying fill materials within the wall footprint is to be carried out in short "strip" sections perpendicular to the Highway 400, with the base of the excavation (as measured parallel to the toe of the Highway 400 embankment) not wider than 3 m.
- Temporary excavation side slopes or back slopes shall be no steeper than 1 horizontal to 1 vertical (1H:1V). Depending on the footprint for the wall foundation (footing or reinforced soil mass), excavation into the toe of the existing Highway 400 embankment may be necessary, and a temporary protection system may be required. The design of such a system will be the responsibility of the contractor; however, it is envisaged that driven steel sheetpiles may be used in conjunction with the special excavation techniques.
- Excavation and backfilling operations are to be carried out simultaneously in a manner that the excavation is not left open for more than the 3 m "strip" width at any given time.

An Operational Constraint is provided in Appendix C to address this requirement, for inclusion in the Contract Documents. The subexcavation areas should be backfilled with Granular B Type II, which will minimize segregation of the soil particles during placement assuming wet conditions in the strip excavations.

6.3.2.2 Preloading

Preloading may be considered for reducing post-construction settlements of the subsoils under the proposed embankment widening and new retaining wall area. Preloading refers to the placement of fill either up to the proposed profile grade of the highway embankment or a portion thereof (i.e. partial preload), in one or more stages, in advance of the embankment completion, in order to preconsolidate the underlying compressible soils. Preloading reduces the magnitude of long-term, post-construction settlements by promoting such settlements to occur under the fill loads in advance of final grading of the embankment. Given the relatively short period for primary consolidation to occur (i.e., on the order of four months), it is anticipated that surcharging would not be required to accelerate the preloading settlement, and consequently detailed assessment and global stability analyses for higher sections have not been completed related to surcharging.

Assuming that the organic silt soils are subexcavated (as outlined in Section 6.3.2.1) to achieve the required factor of safety against global instability of the retaining wall system, no longer-term consolidation settlement will remain under the wall area. Therefore, preloading is not required at this site provided that subexcavation is adopted.

6.3.2.3 Lightweight Fill

Lightweight fill, such as lightweight slag, ultra-lightweight slag, or cellular concrete, could be used for the construction of the new retaining wall to reduce the additional loading imposed on the underlying soils, which in turn would reduce the magnitude of post-construction settlement. In the north section of the retaining wall, the shallow depth of subexcavation (as outlined in Section 6.3.2.1) is more cost-effective than using lightweight fill materials, and so the use of lightweight materials is only considered in the central portion of the wall. Assuming no subexcavation of the organic silt layer in the central portion of the proposed retaining wall, the lighter fill loading would reduce the predicted magnitude of the primary consolidation settlement as outlined below.

Fill Option	Unit Weight (kN/m^3)	Total Settlement (mm)	Immediate/Elastic Settlement (mm)	Consolidation Settlement (mm)
Lightweight Slag	14	175	75	100
Ultra-Lightweight Slag	11	150	90	70
Cellular Concrete	1	<25	<25	<25

As there would still be 75 mm to 90 mm of consolidation settlement with the use of lightweight or ultra-lightweight slag fill, other settlement mitigation measures (such as subexcavation or preloading) would be required to meet settlement performance requirements. In addition, as noted above, it is not advisable to leave organic soils under the proposed retaining wall, due to the potential for ongoing organic degradation and settlement over time. Given that subexcavation alone will aid in minimizing the settlement, lightweight fill materials are not considered necessary for the proposed retaining wall construction.

6.4 Global Stability

Global stability analyses were performed for the critical geometry, corresponding to the highest wall section (approximately 4.5 m, with a 2H:1V embankment side slope above the top of the wall and a total embankment height of approximately 11 m), using subsurface conditions consistent with those encountered in Borehole RW1-2 in the central portion of the proposed retaining wall. The global stability analyses were completed using the commercially available program *SLIDE,* produced by Rocscience Inc., to check that a minimum factor of safety of 1.5 is achieved for the proposed maximum retaining wall height and geometry under static conditions. This minimum factor of safety is considered appropriate for the proposed walls on this site, considering the design requirements and the available field and laboratory testing data.

The following parameters have been used in the global stability analyses for the 4.5 m high wall section, for the soil conditions encountered in Borehole RW1-2, based on field and laboratory test data as well as accepted correlations (Bowles, 1984 and Kulhawy and Mayne, 1990):

Soil Deposit	Short-Term (Undrained) Analysis			Long-Term (Drained) Analysis		
	Bulk Unit Weight (kN/m^3)	Effective Friction Angle, φ' (degrees)	Undrained Shear Strength (kPa)	Unit Weight (kN/m^3)	Effective Friction Angle, φ' (degrees)	Cohesion (kPa)
Fill (existing and new)	21	32	-	21	32	0
Soft to firm organic silt	18	0	30	18	24	0
Loose upper silt	19	30	-	19	30	0
Very stiff to hard clayey silt till including interlayers	21	0	150	21	34	10

The results of the static global stability analyses indicate that a minimum factor of safety of 1.5 is achieved for both short-term (undrained) and long-term (effective stress) conditions for a RSS walls up to approximately 4.5 m in height, ***provided that the soft to firm organic silt layer is subexcavated from below the footprint of the retaining wall*** within the central portion of the proposed retaining wall. These results are presented on Figures 1 and 2 for short-term and long-term conditions, respectively. It is noted that for the purposes of the global stability analyses, the retaining wall has been modelled to represent either a concrete cantilever wall supported on shallow foundations, or a reinforced soil system (RSS) wall. If an RSS wall is adopted, the internal stability of such a system is to be assessed by the proprietary product designer/supplier.

Pseudo-static seismic stability analyses were completed for the same wall/slope geometry and subsurface conditions, to demonstrate that the wall will have a factor of safety greater than 1.1 against global instability instability, using a peak ground acceleration of 0.06g (see Section 6.7.1).

6.5 Retained Soil System (RSS) Walls

As discussed above, from a geotechnical/foundations perspective, an RSS wall is the preferred option for the proposed retaining wall at this site, in conjunction with subexcavation of weak/organic soils as outlined in Section 6.3.2.1. The RSS wall should be designed for high performance and appearance in accordance with MTO Special Provision (SP) 599S22 and the MTO *RSS Design Guidelines*.

6.5.1 Founding Elevation

A typical RSS wall has a front facing supported on compacted granular fill or a strip footing 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. In the central and northern portions of the wall, where the weak/organic soils are recommended to be subexcavated and backfilled with Granular B Type II, no additional subexcavation will be required for the proposed RSS wall or facing footing. However, a minimum 0.3 m thick compacted Granular A pad should be used for levelling purposes below the facing panels, and this pad should extend at least 0.5 m beyond the outside edge of both sides of the facing panels, then outward/downward at 1H:1V.

In the south portion of the wall, to account for the presence of topsoil or softened/loosened surficial soils, it is recommended that the levelling pad be placed following subexcavation to a depth of 1 m below the existing ground surface (which varies along the length of the wall).

6.5.2 Geotechnical Resistance and Settlement

For the RSS facing panels bearing on compacted granular fill as described above, a factored geotechnical resistance at ULS of 150 kPa and a factored geotechnical resistance at SLS of 125 kPa should be used for design.

For the reinforced soil mass founded following subexcavation and replacement with compacted Granular B Type II fill, as discussed in Section 6.3.2.1, the factored geotechnical resistances at ULS given below may be used for design of the reinforced soil mass. These values assume that the reinforced soil mass acts as a unit and uses the full width of the reinforced soil mass (which can be taken as approximately 0.8 times the wall height for design purposes).

Retaining Wall Section	Maximum Wall Height Above Finished Grade	Assumed Reinforced Width	Factored Geotechnical Resistance at ULS
South End Station 15+350 to Station 15+375	4.0 m	3.2 m	300 kPa
Middle Station 15+375 to Station 15+430	4.5 m	3.6 m	400 kPa
North Side Station 15+430 to Station 15+460	4.0 m	3.2 m	400 kPa

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 φ', between the compacted granular fills of the RSS wall and the properly prepared native subgrade or Granular B Type II fill may be taken as 0.6. The coefficient of friction, tan ϕ', between a cast-in-place concrete facing footing and underlying granular pad may be taken as 0.55. This represents an unfactored value; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance. The actual values used should be reviewed and revised, if necessary, by the proprietary RSS wall designer during detailed design.

6.6 Concrete Retaining Wall Supported on Pile Foundations

6.6.1 Founding Elevation

Driven steel H-pile or steel pipe (tube) pile foundations are feasible for support of the retaining wall. Based on the results of the geotechnical investigation (i.e., boreholes terminated in materials having SPT “N” values on the order of 30 to 50 blows per 0.3 m of penetration), recommendations are provided for friction piles only. Further borehole investigation is recommended if this foundation option is adopted. However, for preliminary purposes, driven steel piles may be designed based on a pile tip elevation of 286 m.

The pile cap should be founded at a minimum depth of 1.5 m below the lowest surrounding grade to provide adequate protection against frost penetration (per OPSD 3090.101 – *Foundation Frost Penetration Depths for Southern Ontario)*.

For the installation of steel H-piles or steel pipe piles, consideration must be given to the potential presence of cobbles and boulders within the glacially-derived soils at this site. In this regard, steel H-piles are preferred over steel pipe piles as pipe piles are considered to pose a higher risk of experiencing refusal on boulders or being deflected away from the vertical/battered orientation during installation due to their larger end area. Piles should be reinforced at the tip with driving shoes and/or flange plates in accordance with OPSD 3000.100 (*Steel H-Pile Driving Shoe*) or OPSD 3001.100 (*Steel Tube Pile Driving Shoe*) Type II, as appropriate, to reduce the potential for damage to the piles during driving. In very dense strata containing cobbles and/or boulders, driving shoes (such as Titus Standard 'H' Bearing Pile Points) are preferred over flange plates.

6.6.2 Geotechnical Resistance

For HP 310x110 piles driven to the design tip elevation given above, the factored axial geotechnical resistance at ULS may be taken as 500 kN. The axial geotechnical reaction at Serviceability Limit States (SLS, for 25 mm of settlement) may be taken as 425 kN. The same axial resistances may be used in the design of closed-end, concrete-filled, 324 mm (12 ¾ in.) diameter steel pipe piles having a minimum wall thickness of 9.5 mm (3/8 in.) founded at the same design pile tip elevations.

The following note, or similar, should be shown on the Contract Drawings, assuming that a resistance factor of 0.5 is applied to the use of the Hiley calculation based on MTO experience in the Southern Ontario region (Note 2 from the *Structural Manual*, Section 3.3.3 (MTO, 2008)):

> *"Piles to be driven in accordance with Standard SS-103-11 using an ultimate geotechnical resistance of 1,000 kN per pile."*

Pile installation should be in accordance with OPSS 903 (Deep *Foundations*). The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile; the criteria must therefore be established at the time of construction after the piling equipment is known. The pile capacity should then be verified in the field by the use of the Hiley formula (MTO's Standard Drawing SS103-11, *Pile Driving Control)* during the final stages of driving to verify that the required ultimate capacity has been achieved. Relaxation of soil surrounding the pile tips and/or heaving of the pile tips as a result of driving of adjacent piles could lead to reduced pile capacities. In this regard, it is recommended that a minimum of 10 per cent of piles be re-tapped at each foundation element to confirm that relaxation/heave is not occurring. If a significant reduction in the pile driving resistance is noted during re-tapping, all of the piles may need to be re-tapped and/or re-driven.

6.6.3 Downdrag Loads

Assuming the use of conventional earth or granular fill for the embankment construction, the widened embankment/retaining wall loading will cause consolidation settlement of the soft organic silt (as discussed in Section 6.3.1). Downdrag loads will develop along the portion of the pile shaft that is embedded within the organic silt deposit, unless measures to eliminate downdrag loads are adopted as discussed below.

In calculating the magnitude of the downdrag force, the methods described in both the Canadian Foundation Engineering Manual (CFEM, 2006) as well as the US Transportation Research Board's report, "Design and Construction Manual For Downdrag on Uncoated and Bitumen-Coated Piles" (Briaud and Tucker, 1994) were considered. Given the larger predicted settlement of the organic silt and silt deposit in comparison to the elastic shortening of the pile, the neutral plane used in the analysis of downdrag was assumed to be at the underside of the silt deposit. For preliminary design, the unfactored downdrag load on a single HP 310x110, 310x132 or 310x152 pile may be taken as 150 kN.

There is no space at this site to preload the retaining wall area to eliminate downdrag loading on the piles. Alternatively, downdrag loads could be eliminated with the use of lightweight expanded polystyrene (EPS) or cellular concrete fill as backfill behind the wall.

6.6.4 Resistance to Lateral Loads

The design of piles subjected to lateral loads should take into account such factors as the batter of the pile (if any), the relative rigidity of the pile to the surrounding soil, the fixity condition at the head of the pile (pile cap level), the structural capacity of the pile to withstand bending moments, the soil resistance that can be mobilized, the tolerable lateral deflections at the head of the pile, and pile group effects. For a longer, more flexible pile, the maximum yield moment of the pile may be reached prior to mobilization of the lateral geotechnical resistance. For design purposes, both the structural and geotechnical resistances should be evaluated to establish the governing case. Lateral loading could also be resisted fully or partially by the use of battered piles.

The resistance to lateral loading in front of the piles may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction is determined based on the equation given below (CFEM, 1992, as noted in Section C6.8.7.1 (Table C6.5) and in Section C6.8.7.3 of the *Commentary to CHBDC*).

For non-cohesive soils:

$$k_h = \frac{n_h z}{B}$$

where
k_h is the coefficient of horizontal subgrade reaction (kPa/m);
n_h is the constant of subgrade reaction (kPa/m);
z is the depth (m); and
B is the pile diameter / width (m).

For cohesive soils:

$$k_h = \frac{67 s_u}{B}$$

where
k_h is the coefficient of horizontal subgrade reaction (kPa/m);
s_u is the undrained shear strength of the soil (kPa); and
B is the pile diameter / width (m).

The values of n_h and s_u (Terzaghi, 1955 and Reese, 1975) to be incorporated into the calculations of the coefficient of horizontal subgrade reaction (k_h) within the native subsoils to be utilized for the structural analysis of the piles and casings at this site are summarized below. The resistance to lateral loading should be neglected within the zone of frost penetration (i.e., within 1.5 m below the lowest surrounding grade in front of the piles).

Soil Unit	n_h (kPa/m)	s_u (kPa)
Embankment fill (assuming engineered earth fill)	5,000	-
Soft to firm organic silt	-	30
Loose to compact silty sand to silt	4,000	
Very stiff to hard clayey silt till including very stiff to hard interlayer	-	150
Stiff to very stiff clayey silt interlayer	-	100
Compact to very dense lower silt	10,000	-

A maximum factored lateral resistance of 120 kN at ULS, and a maximum lateral resistance of 35 kN at SLS (for 10 mm of horizontal deflection at pile cap level) is recommended for HP 310x110 piles. These values are based on the “Assessed Horizontal Passive Resistance Values for Various Pile Types” provided in Table C6.8.7.1(a) of the *Commentary* to the *CHBDC*. The above recommendations based on subgrade reaction theory and assessed values can be refined based on soil-structure interaction modelling using a software program such as L-Pile, if necessary, as the detail design of the deep foundations proceeds.

Group action for lateral loading should be considered where the pile spacing in the direction of the loading is less than approximately six to eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor, R (NAVFAC DM-7.2, 1982) as follows:

Pile Spacing in direction of Loading (d = Pile Diameter)	Subgrade Reaction Reduction Factor
8d	1.00
6d	0.70
4d	0.40
3d	0.25

The subgrade reaction reduction factor should be interpolated for pile spacings in between those provided in the above table.

6.7 Lateral Earth Pressures for Design of Concrete Walls

If a concrete retaining wall is adopted, the lateral earth pressures acting on the retaining wall 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. Seismic (earthquake) loading must also be taken into account in the design.

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

- Select, free-draining granular fill meeting the specifications of SP105S13 *(Aggregates)* Granular A or Granular B Type II (but with less than 5 percent passing the 200 sieve) should be used as backfill behind the wall.
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with *CHBDC* Section 6.9.3 and Figure 6.6. Compaction equipment should be used in accordance with OPSS.PROV 501 *(Compacting).* Other surcharge loadings should be accounted for in the design as required.
- The granular fill may be placed either in a zone with the width equal to at least 1.5 m behind the back of the walls (see Case A in Figure C6.20(a) of the Commentary to the CHBDC), or within the wedge-shaped zone defined by a line drawn at 1.5H:1V extending up and back from the rear face of the footing (see Case B in Figure C6.20(b) of the Commentary to the CHBDC).

- For Case A, the pressures are based on the existing embankment fill materials and the existing overburden soils and the following parameters (unfactored) may be used:

	Existing Fill
Soil unit weight:	20 kN/m^3
Coefficients of static lateral earth pressure:	
Active, K_a	0.33
At rest, K_o	0.50

- For Case B, where the pressures are based on OPSS.PROV 1010 Granular A or Granular B Type II fill behind the wall, the following parameters (unfactored) may be assumed:

	Granular A	Granular B Type II
Soil unit weight:	22 kN/m^3	21 kN/m^3
Coefficients of static lateral earth pressure:		
Active, K_a	0.27	0.27
At rest, K_o	0.43	0.43

For a concrete retaining wall, where the wall support allows lateral yielding of the stem, active earth pressures should be used in the geotechnical design of the wall structure. 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 C6.9.1 and Table C6.6 of the *Commentary* to the *CHBDC*.

6.7.1 Seismic Considerations

Seismic loading must be taken into account in accordance with Section 4.6.4 of *CHBDC*, as it can result in increased lateral earth pressures acting on the retaining wall.

The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the applicable earthquake-induced dynamic earth pressure. The earthquake-induced dynamic 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 as follows:

$$P = K\gamma' d + (K_{AE} - K)\gamma' (H-d)$$

where

K is either the static active earth pressure coefficient (K_a) or the static at rest earth pressure coefficient (K_o);

K_{AE} is the seismic active earth pressure coefficient;

γ' is the effective unit weight of the soil (kN/m^3)

- taken as soil unit weights given above for fill materials
- taken as 21 kN/m^3 for the native materials

d is the depth below the top of the wall (m); and

H is the height of the wall above the toe (m).

According to Table C4.2 of the *Commentary* to the *CHBDC*, this site is located in Seismic Zone 1, and the site-specific zonal acceleration ratio (A) for the Aurora-Newmarket area is 0.05. The site-specific peak ground acceleration (PGA) is 0.023g based on the NRC (2010) website; however, the more conservative *CHBDC* value has been used in the assessments presented below. The Site Coefficient (S) may be taken as 1.2, consistent with Soil Profile Type II in accordance with Section 4.4.6 and Table 4.4 of *CHBDC* (2006). Based on the subsurface conditions at the site, a 20 per cent amplification of the ground motion is recommended for design, resulting in an increase in the ground surface acceleration to approximately 0.06g.

The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of A = 0.06. These coefficients have been determined in accordance with Sections 4.6.4 and C4.6.4 of the *CHBDC* and its *Commentary.*

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Case A	Case B	
	Existing Fill	**Granular A**	**Granular B Type II**
Yielding Wall	0.32	0.26	0.26
Non-Yielding Wall	0.36	0.30	0.30

Notes:

1. These seismic K_{AE} values include the effect of wall friction, and assume that the back of the wall is vertical and the ground surface behind the wall is flat.
2. The above K_{AE} values for yielding walls are applicable provided that the wall can move up to 250A (mm), where A is the design zonal acceleration ratio of 0.06. This corresponds to displacements of up to approximately 15 mm at this site.

It is noted that for the very low zonal acceleration ratio for this site, the seismic K_{AE} values are similar to or less than the static values of K_a and K_o reported above.

6.8 Construction Considerations

6.8.1 Subexcavation of Weak/Organic Materials

As discussed, subexcavation is recommended as outlined below, in order to achieve the required minimum factor of safety for global stability of the retaining wall, and mitigate settlement:

- Central portion, Station 15+375 to 15+430: Subexcavate existing fill and soft to firm organic silt, to a depth of 3.5 m to 4 m.
- North portion, Station 15+430 to 15+460: Subexcavate firm to stiff upper clayey silt, to a depth of 1.5 m to 2 m.

Staged subexcavation, in strips of limited width, will be required to maintain the stability of the temporary subexcavation in the central portion of the proposed retaining wall, to protect the existing Highway 400 embankment. It is envisaged that this subexcavation will be completed in “wet conditions” (i.e., without dewatering), as follows:

- Removal of the soft to firm organic silt and the overlying fill materials within the wall footprint is to be carried out in short “strip” sections perpendicular to the Highway 400, with the base of the excavation (as measured parallel to the toe of the Highway 400 embankment) not wider than 3 m.

- Temporary excavation side slopes or back slopes shall be no steeper than 1 horizontal to 1 vertical (1H:1V). Depending on the footprint for the wall foundation (footing or reinforced soil mass), excavation into the toe of the existing Highway 400 embankment may be necessary, and a temporary protection system may be required. The design of such a system will be the responsibility of the contractor; however, it is envisaged that driven steel sheetpiles may be used in conjunction with the special excavation techniques.

- Excavation and backfilling operations are to be carried out simultaneously in a manner that the excavation is not left open for more than the 3 m “strip” width at any given time.

An Operational Constraint is provided in Appendix C to address this requirement, for inclusion in the Contract Documents. The subexcavation areas should be backfilled with Granular B Type II, which will minimize segregation of the soil particles during placement assuming wet conditions in the strip excavations.

6.8.2 Groundwater Control

The groundwater level is expected to be relatively shallow, within about 1 m below the ground surface at the toe of the existing Highway 400 embankment. It is anticipated that the excavations to remove the organic silt and firm to stiff clayey silt will extend to or below the groundwater table at the site. The strip excavation work outlined in Section 6.8.1 may be carried out in wet conditions, without dewatering, provided that Granular B Type II backfill is used both below and above the water table to minimize segregation and to form a base for the subsequent construction of the embankment widening and RSS wall.

6.8.3 Temporary Protection Systems

Where temporary protection systems are required along Highway 400 in conjunction with the subexcavation works, they should be designed and constructed in accordance with OPSS 539 (*Temporary Protection Systems).* The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS 539, provided that any existing adjacent structures or utilities can tolerate this magnitude of deformation. It is considered that a driven, interlocking sheet pile system would be most suitable for the temporary excavation support associated with the strip excavation work at this site, based on the subsurface soil and groundwater conditions.

The sheet piles would have to be driven or socketted to sufficient depth to provide the necessary passive resistance for the retained soil height under the temporary subexcavation works, including any surcharge loads behind the protection system within at least a 1H:1V zone relative to the base of the excavation.

The selection and design of the protection system will be the responsibility of the Contractor.

7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Nikol Kochmanová, P.Eng., a geotechnical engineer with Golder. Ms. Lisa Coyne, P.Eng., a Designated MTO Foundations Contact and a Principal with Golder, conducted an independent quality control review of this report.

GOLDER ASSOCIATES LTD.

LICENSED PROFESSIONAL ENGINEER
Sep 18/15
N. KOCHMAN
100117863
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
L. C. COYNE
18SEP2015
PROVINCE OF ONTARIO

Nikol Kochmanová, P.Eng.
Geotechnical Engineer

Lisa Coyne, P.Eng.
Designated MTO Foundations Contact, Principal

HS/NK/LCC/sm

\\golder.gds\gal\mississauga\active\2009\1111\09-1111-0018 urs - hwy 400 - york\6 - reports\11 - contract 4 retaining wall\09-1111-0018 rpt 15sep18 highway 400 retaining wall.docx

REFERENCES

Canadian Geotechnical Society. 2006. *Canadian Foundation Engineering Manual*, 4th Edition. The Canadian Geotechnical Society, BiTech Publisher Ltd., British Columbia.

Canadian Standards Association (CSA). 2006. *Canadian Highway Bridge Design Code* and *Commentary on CAN/CSA-S6-06*. CSA Special Publication, S6.1 06.

Chapman, L.J., and Putnam, D.F. 1984. *The Physiography of Southern Ontario*. Ontario Geological Survey, Special Volume 2, 3rd Edition. Ontario Ministry of Natural Resources.

Kulhawy, F.H. and Mayne, P.W. 1990. *Manual on Estimating Soil Properties for Foundation Design*. EL 6800, Research Project 1493 6. Prepared for Electric Power Research Institute, Palo Alto, California.

Hynes-Griffin M.E, Franklin A.G. 1984. *Rationalizing the Seismic Coefficient Method*. U.S. Army Corps of Engineers Waterways Experiment Station, Vicksburg, Mississippi, 1984, Miscellaneous Paper GL-84-13.

Unified Facilities Criteria, U.S. Navy. 1986. *NAVFAC Design Manual DM 7.02: Soil Mechanics, Foundation and Earth Structures*. Alexandria, Virginia.

Ministry of Transportation. Foundation Guideline. *Embankment Settlement Criteria for Design*, March 2010.

Ontario Provincial Standard Specifications (OPSS)

OPSS 180 General Specification for the Management of Excess Materials

OPSS 212 Construction Specifications for Borrow

OPSS 501 Construction Specifications for Compacting

OPSS 511 Construction Specification for Rip-Rap, Rock Protection, and Granular Sheeting Temporary Protection Systems

OPSS 802 Construction Specification for Topsoil

OPSS 803 Construction Specification for Sodding

OPSS 804 Construction Specification for Seed and Cover

Ontario Provincial Standard Drawings (OPSD)

OPSD 208.010 Benching of Earth Slopes

OPSD 202.010 Slope Flattening Using Surplus Excavated Material on Earth or Rock Embankment

OPSD 200.020 Earth / Shale Grading

Construction Design Estimating and Documentation (CDED) Special Provisions (SP)

SP 110S13 Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

SP 206S03 Earth Excavation and Grading; Rock Excavation and Grading, Excavation for Pavement Widening

SP 105S21 Amendment to OPSS 501, Construction Specification for Compaction

ASTM International

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

ASTM D2573 Standard Test Method for Field Vane Shear Test in Cohesive Soil.

Ontario Water Resources Act

Ontario Regulation 903 Wells (as amended)

Ontario Occupational Health and Safety Act

Ontario Regulation 213 Construction Projects (as amended)

Commercial Software

Slide (Version 6.0) by Rocscience Inc.

Settle3D (Version 2.0) by Rocscience Inc.

TABLE 1 – COMPARISON OF RETAINING WALL TYPES AND FOUNDATION ALTERNATIVES

Wall Type and Foundation Option	Feasibility	Advantages	Disadvantages	Constructability/ Risks	Estimated Costs
Retained soil system (RRS) walls	• Feasible, though subexcavation required for global stability	• More tolerable to post construction settlements • Lowest cost alternative	• Excavation of organic silt and firm clayey silt required • Groundwater control required	• Special procedures for subexcavation • Conventional construction techniques	• Lower cost than concrete retaining wall
Concrete retaining walls on deep foundations	• Feasible	• Potentially reduced excavation, protection system and backfill requirements compared to RSS wall	• Could still have settlement of soils behind wall under embankment widening area unless organic silt/firm clay is excavated, in which case better to adopt an RSS wall or shallow foundation with subexcavation	• Conventional excavation and construction techniques • Relatively long construction time compared to most wall alternatives	• Higher cost relative to RSS wall
Concrete retaining walls on shallow foundations	• Feasible, though subexcavation required and more costly than RSS		• Excavation of organic silt and firm clayey silt required • Groundwater control required; foundations would extend below groundwater level	• Special procedures for subexcavation • Likely greater dewatering than for RSS wall • Conventional construction techniques	• Higher cost relative to RSS wall; must also consider subexcavation costs
Soldier pile and concrete panel walls	• Not considered appropriate		• Most advantageous in “top-down” construction applications, i.e. as part of a cut-widening, rather than for an embankment widening • Likely more time-consuming than other wall types due to steps involved (pre-augering for socket holes, placing soldier piles, placing backfill in lifts, installing concrete panels, installing, pre-stressing and testing tie-backs)	• More specialized equipment and skilled labour required • Construction costs and time may escalate if cobbles and boulders are encountered in soldier pile installation	• Comparable costs to concrete retaining wall, but higher than RSS wall

MINISTRY OF TRANSPORTATION, ONTARIO

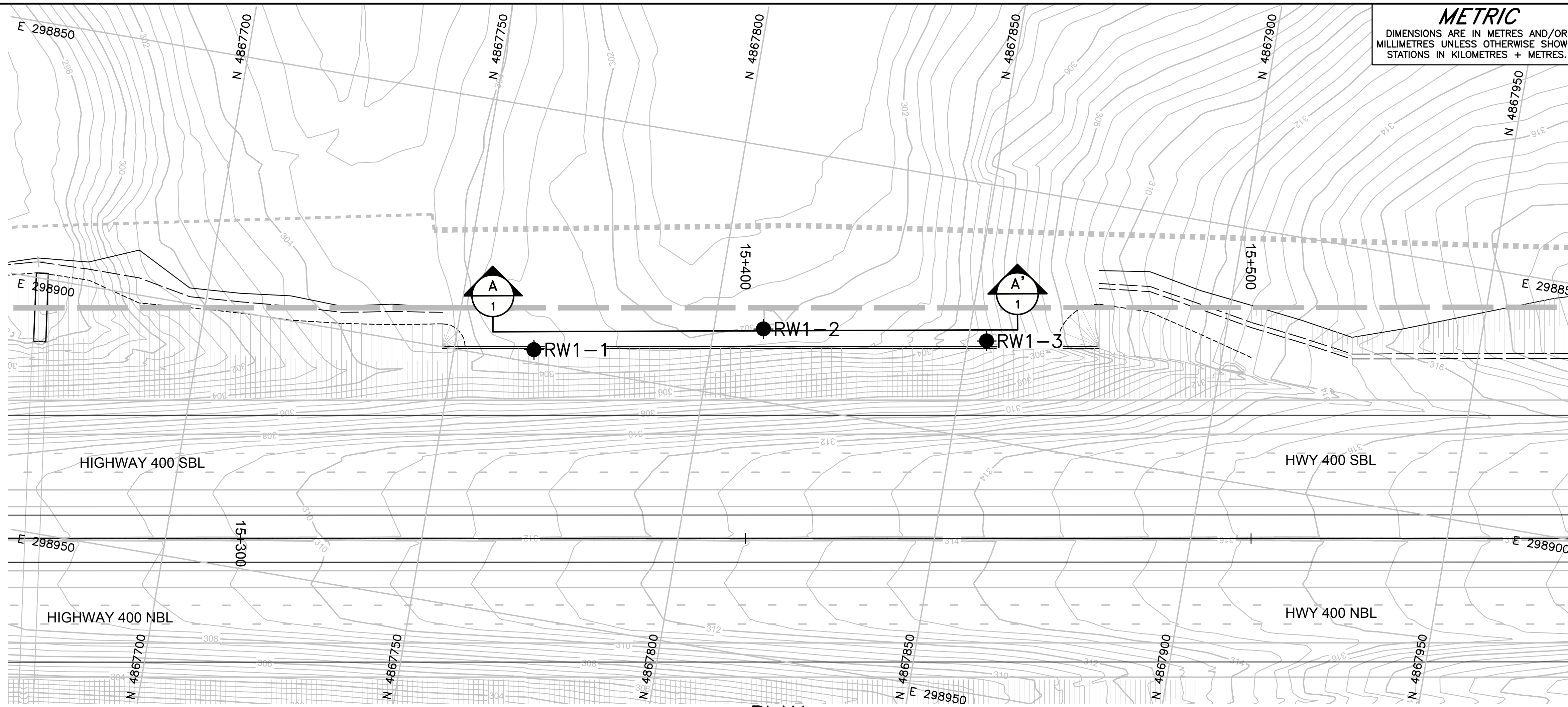
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 2835-02-00

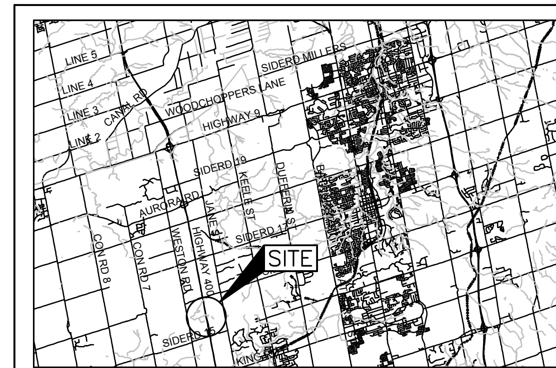
HIGHWAY 400 WIDENING
RETAINING WALL
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

Golder Associates



PLAN
SCALE
10 0 10 20 m

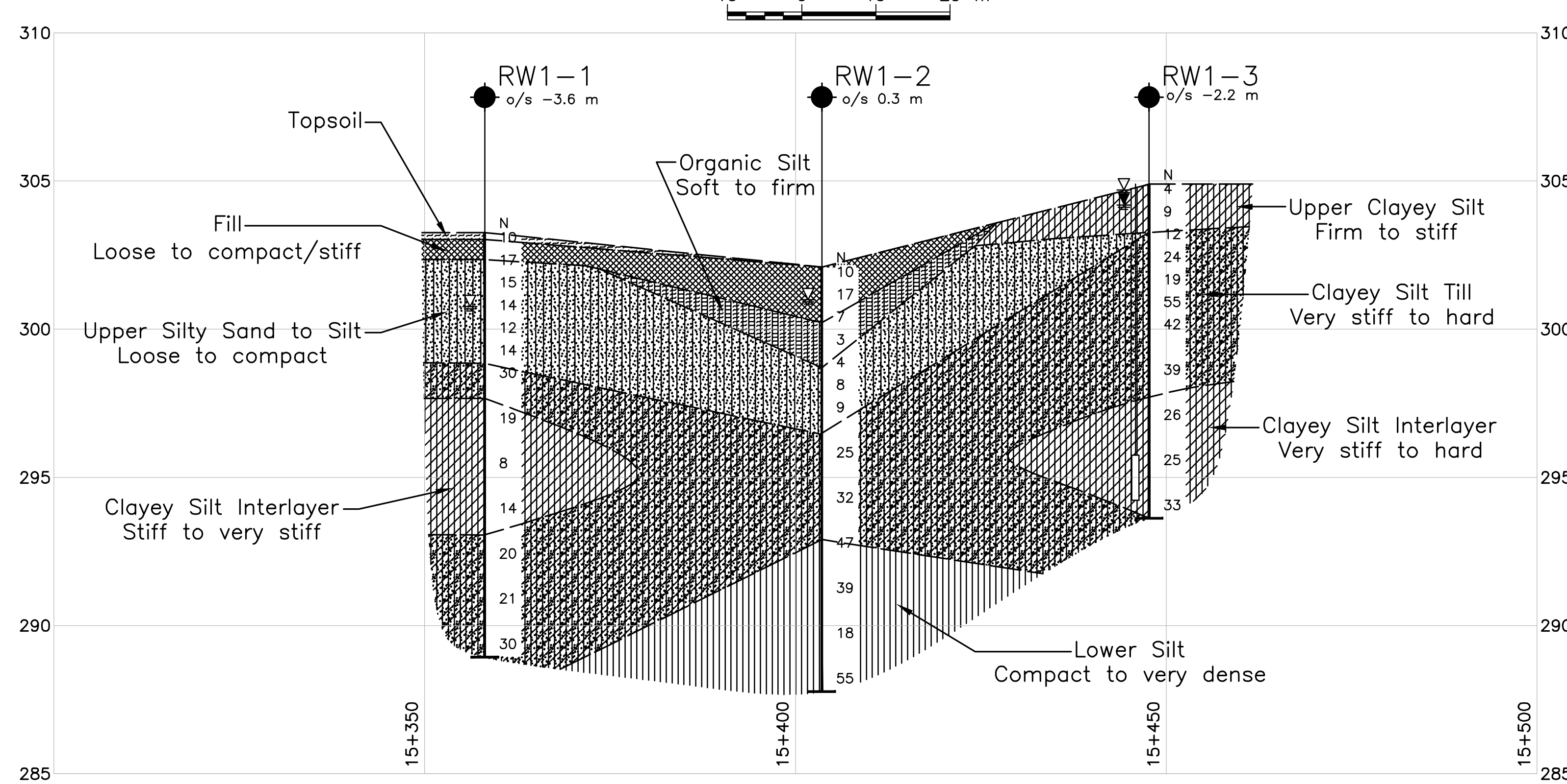


KEY PLAN
SCALE
4 0 4 8 km

LEGEND

●	Borehole – Current Investigation
	Seal
	Piezometer
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
▼	WL in piezometer, measured on April 7, 2011
▽	WL upon completion of drilling

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
RW1-1	303.3	4867765.6	298897.5
RW1-2	302.1	4867809.7	298885.8
RW1-3	304.9	4867853.6	298880.7



PROFILE (A-A' / 1)
HORIZONTAL SCALE
10 0 10 20 m
VERTICAL SCALE
2.5 0 2.5 5 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.

The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

Base plans provided in digital format by URS, drawing file Hwy400_plan.dwg, received July 28, 2014 and Hwy400_contours.dwg, received July 12, 2011.

LICENSED PROFESSIONAL ENGINEER L.C. COYNE Sept. 18, 2015 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER N. KOCHMAN 100117863 Sept. 18, 2015 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 30M13-213

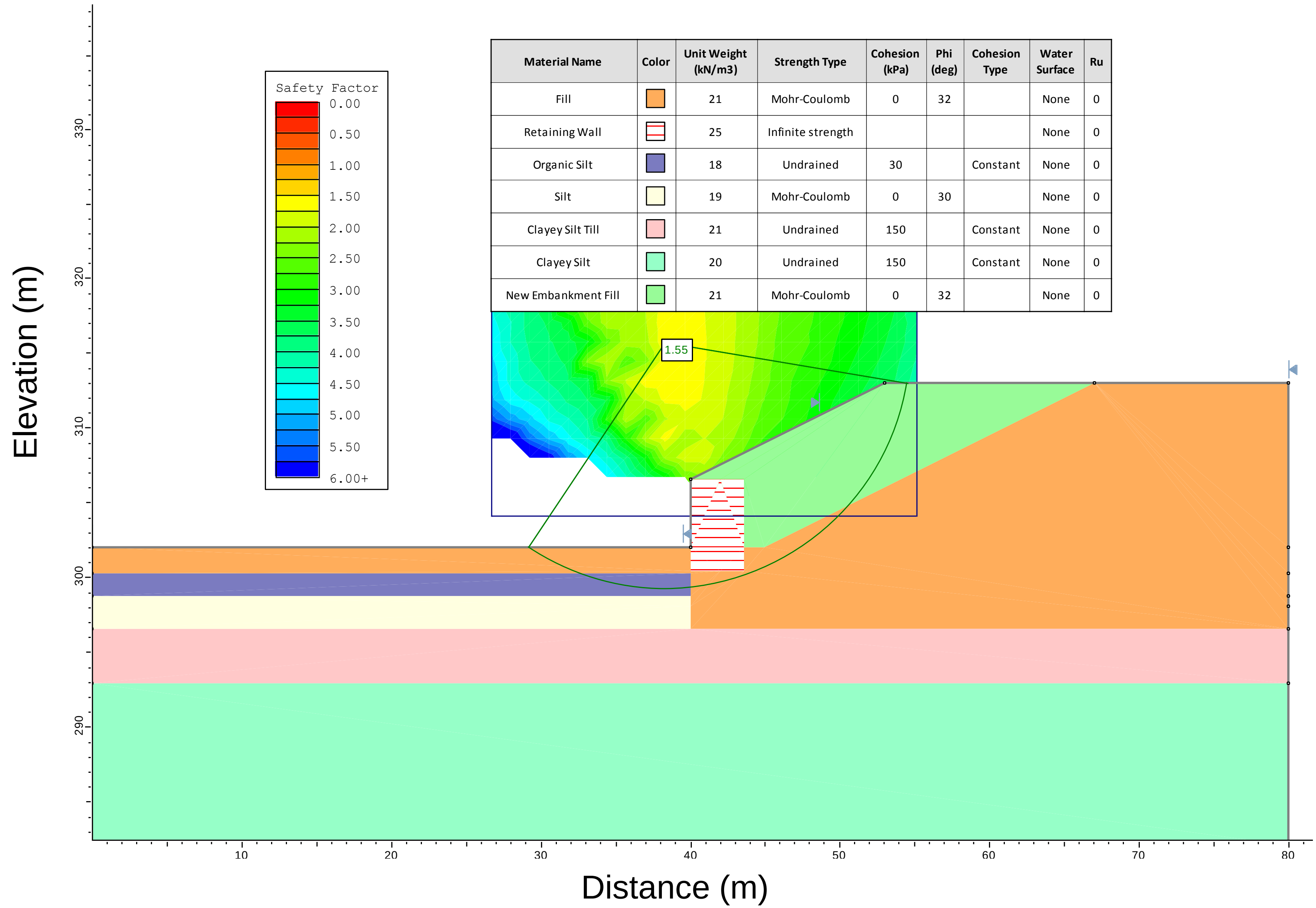
HWY. 400	PROJECT NO. 09-1111-0018		DIST. CENTRAL
SUBM'D. HLS	CHKD. NK	DATE: 8/21/2015	SITE:
DRAWN: JFC	CHKD. NK	APPD. LCC	DWG. 1

PLOT DATE: September 23, 2015
FILENAME: T:\Projects\2009\09-1111-0018 (URS, York Region)\-PB- Retaining_Wall\0911110018PB001.dwg

STATIC GLOBAL STABILITY OF RSS WALL
SHORT-TERM (UNDRAINED) CONDITIONS

Figure 1

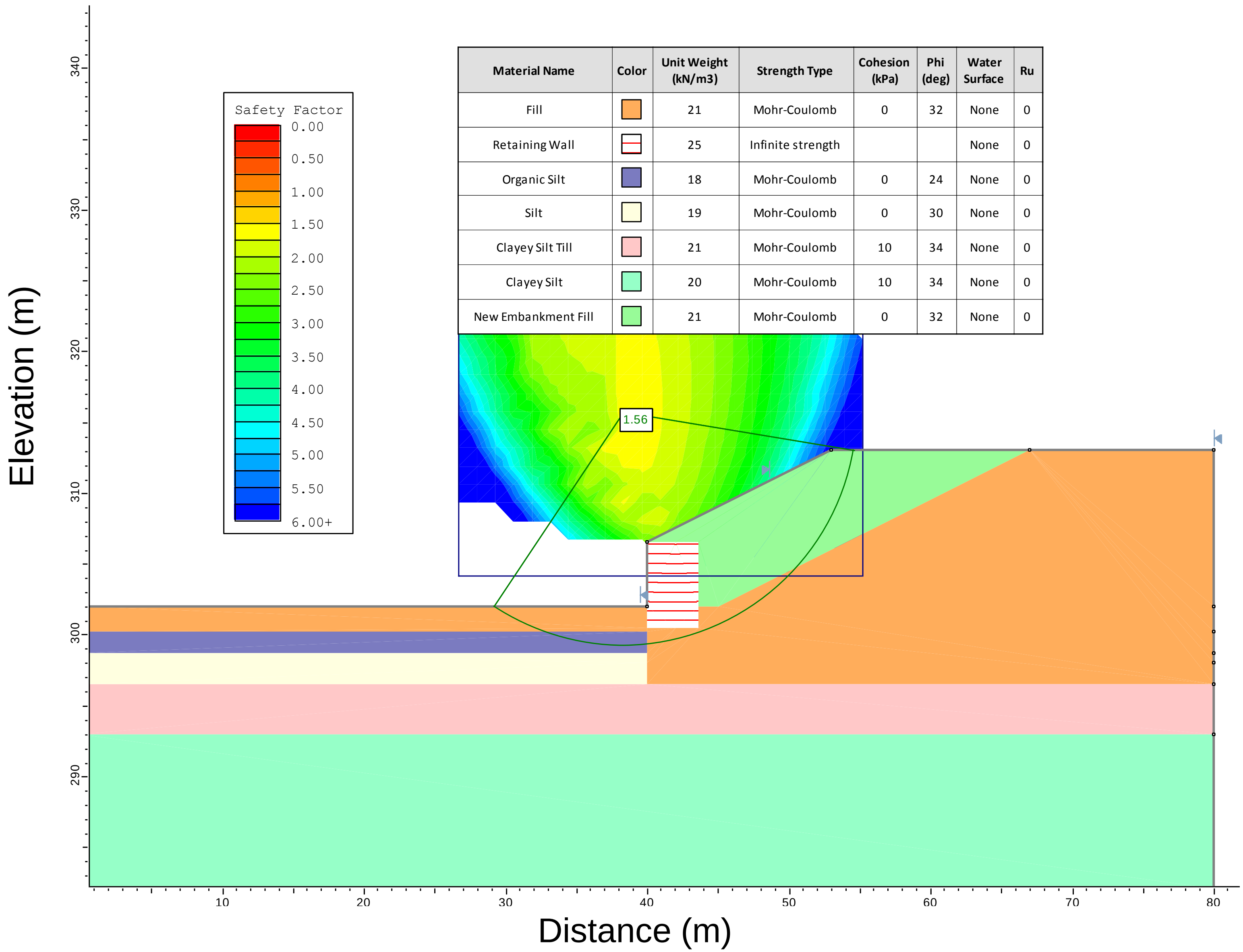
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Water Surface	Ru
Fill		21	Mohr-Coulomb	0	32		None	0
Retaining Wall		25	Infinite strength				None	0
Organic Silt		18	Undrained	30		Constant	None	0
Silt		19	Mohr-Coulomb	0	30		None	0
Clayey Silt Till		21	Undrained	150		Constant	None	0
Clayey Silt		20	Undrained	150		Constant	None	0
New Embankment Fill		21	Mohr-Coulomb	0	32		None	0



Date: July 2015
Project No: 09-1111-0018
Analysis By: HLS
Reviewed By: LCC

STATIC GLOBAL STABILITY OF RSS WALL
LONG-TERM (EFFECTIVE STRESS) CONDITIONS

Figure 2



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Water Surface	Ru
Fill		21	Mohr-Coulomb	0	32	None	0
Retaining Wall		25	Infinite strength			None	0
Organic Silt		18	Mohr-Coulomb	0	24	None	0
Silt		19	Mohr-Coulomb	0	30	None	0
Clayey Silt Till		21	Mohr-Coulomb	10	34	None	0
Clayey Silt		20	Mohr-Coulomb	10	34	None	0
New Embankment Fill		21	Mohr-Coulomb	0	32	None	0

Date: July 2015
Project No: 09-1111-0018

Analysis By: HLS

Reviewed By: LCC

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
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

* 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 δ
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

Density Index Relative Density	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	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	Sand and Gravel
	With (cohesive)	Silty Clay with sand / Clayey Silt with sand

Golder Associates

RECORD OF BOREHOLE No RW1-1

SHEET 1 OF 2

METRIC

PROJECT 09-1111-0018

W.P. 2835-02-00

DIST Central HWY 400

DATUM Geodetic

LOCATION N 4867765.6 ;E 298897.5

BOREHOLE TYPE D-25 Track Mount, 108 mm Outside Diameter Solid and Hollow Stem Auger

DATE December 17 & 20, 2010

ORIGINATED BY TT

COMPILED BY ARM

CHECKED BY LCC

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
303.3	GROUND SURFACE
0.0	TOPSOIL
0.2	Silty sand, containing zones of clayey silt (FILL) Compact Brown Moist
302.4 / 0.9	Silty SAND, trace clay Compact Brown Wet
299.9	CLAYEY SILT, trace to some sand, trace gravel (TILL) Stiff Brown and grey Moist
299.6 / 3.7	
298.9 / 4.4	Silty SAND Compact Grey Wet
	CLAYEY SILT, trace to some sand, trace gravel (TILL) Hard Grey Moist
297.7 / 5.6	CLAYEY SILT, containing sand seams Stiff to very stiff Grey Moist
293.1 / 10.2	CLAYEY SILT, trace to some sand, trace gravel (TILL) Very stiff to hard Grey Moist
289.0 / 14.3	END OF BOREHOLE

SAMPLES

NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
1	SS	10				
2	SS	17				
3	SS	15				
4	SS	14	0	69	28	3
5	SS	12				
6	SS	14				
7	SS	30	3	13	63	21
8	SS	19				
9	SS	8	0	2	38	60
10	SS	14				
11	SS	20				
12	SS	21	3	8	64	25
13	SS	30				

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (303 to 289); 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/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

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

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

Golder Associates

RECORD OF BOREHOLE No RW1-1

SHEET 2 OF 2

METRIC

PROJECT 09-1111-0018

W.P. 2835-02-00

LOCATION N 4867765.6 ;E 298897.5

ORIGINATED BY TT

DIST Central HWY 400

BOREHOLE TYPE D-25 Track Mount, 108 mm Outside Diameter Solid and Hollow Stem Auger

COMPILED BY ARM

DATUM Geodetic

DATE December 17 & 20, 2010

CHECKED BY LCC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	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	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	NOTES: 1. Blowing sands encountered in open borehole at a depth of 4.4 m; changed boring method to hollow stem augers at a depth of 4.6 m. 2. Water level in open borehole at a depth of 2.5 m (Elev. 300.8 m) upon completion of drilling.										

GTA-MTO 001 T:\PROJECTS\2009\09-1111-0018 (URS, YORK REGION)\LOG\0911110018.GPJ GAL-GTA.GDT 08/21/15 SIB

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

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

Foundation Design

RECORD OF BOREHOLE No RW1-2

SHEET 1 OF 2 — METRIC

PROJECT 09-1111-0018
W.P. 2835-02-00
LOCATION N 4867809.7 ;E 298885.8
ORIGINATED BY TT
DIST Central HWY 400
BOREHOLE TYPE D-25 Track Mount, 108 mm Outside Diameter Hollow Stem Auger
COMPILED BY MAS
DATUM Geodetic
DATE December 20 & 21, 2010
CHECKED BY LCC

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
302.1	GROUND SURFACE					
0.0	Silty clay, trace sand, containing rootlets (FILL) Stiff Brown Moist	1	SS	10		
301.6 / 0.5	Sand and silt, containing zones of clayey silt (FILL) Loose to compact Brown Moist becoming wet at a depth of 1.4 m	2	SS	17	▽	
		3	SS	7		
300.2 / 1.9	Organic SILT, some sand, trace to some clay, containing rootlets, wood fragments and organics shell fragments Soft to firm Black Wet	4	SS	3		O.C. = 4.1; 0 13 73 14
	Containing peat at a depth of 3.4 m	5	SS	4		
298.7 / 3.4	SILT, trace to some sand, trace clay Loose Grey Wet	6	SS	8		
		7	SS	9		
296.5 / 5.6	CLAYEY SILT, trace to some sand, trace gravel (TILL) Very stiff to hard Brown Moist	8	SS	25		
	Containing sand seams at a depth of 7.6 m Becoming grey at a depth of 7.8 m	9	SS	32		1 9 60 30
292.9 / 9.2	SILT, trace clay Compact to very dense Grey Moist	10	SS	47		
		11	SS	39		N.P.; 0 0 93 7
	Becoming wet at a depth of 11.7 m	12	SS	18		
	Becoming moist at a depth of 13.3 m	13	SS	55		
287.8 / 14.3	END OF BOREHOLE					

Elevation scale: 302, 301, 300, 299, 298, 297, 296, 295, 294, 293, 292, 291, 290, 289, 288

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

44.2

UNIT WEIGHT γ kN/m³ — NP

Continued Next Page

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

Golder Associates

PROJECT 09-1111-0018

RECORD OF BOREHOLE No RW1-2

SHEET 2 OF 2

METRIC

W.P. 2835-02-00

LOCATION N 4867809.7 ;E 298885.8

ORIGINATED BY TT

DIST Central HWY 400

BOREHOLE TYPE D-25 Track Mount, 108 mm Outside Diameter Hollow Stem Auger

COMPILED BY MAS

DATUM Geodetic

DATE December 20 & 21, 2010

CHECKED BY LCC

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
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	NOTES: 1. Water level in open borehole at a depth of 1.1 m (Elev. 301.0 m) upon completion of drilling. 2. Borehole caved at a depth of 6.5 m (Elev. 295.6 m) upon completion of drilling.										

GTA-MTO 001 T:\PROJECTS\2009\09-1111-0018 (URS, YORK REGION)\LOG\0911110018.GPJ GAL-GTA.GDT 08/21/15 SIB

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

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

Golder Associates

Foundation Design

RECORD OF BOREHOLE No RW1-3

SHEET 1 OF 1 | **METRIC**

PROJECT 09-1111-0018

W.P. 2835-02-00 | LOCATION N 4867853.6 ;E 298880.7 | ORIGINATED BY TT

DIST Central HWY 400 | BOREHOLE TYPE D-25 Track Mount, 108 mm Outside Diameter Hollow Stem Auger | COMPILED BY ARM

DATUM Geodetic | DATE December 21, 2010 | CHECKED BY LCC

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
304.9	GROUND SURFACE
0.0	CLAYEY SILT with sand, containing rootlets and organics Firm to stiff Brown Moist becoming wet at a depth of 0.8 m
303.3 / 1.6	CLAYEY SILT, some sand, trace gravel (TILL) Stiff to hard Brown and grey Moist Containing zones of oxidation at a depth of 4.6 m Containing a sand seam at a depth of 6.2 m
297.7 / 7.2	CLAYEY SILT Very stiff to hard Grey Wet
293.6 / 11.3	END OF BOREHOLE

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	SS	4				
2	SS	9				
3	SS	12				
4	SS	24	2	13	58	27
5	SS	19				
6	SS	55				
7	SS	42				
8	SS	39				
9	SS	26				
10	SS	25	0	0	90	10
11	SS	33				

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (304, 303, 302, 301, 300, 299, 298, 297, 296, 295, 294); 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

NOTES:

1. Water level in open borehole at a depth of 0.2 m (Elev. 304.7 m) upon completion of drilling.

2. Borehole caved at a depth of 9.8 m (Elev. 296.4 m) upon completion of drilling.

3. Water level measurements in piezometer:

Date	Depth (m)	Elev. (m)
02/01/11	1.5	303.4
04/07/11	0.7	304.2

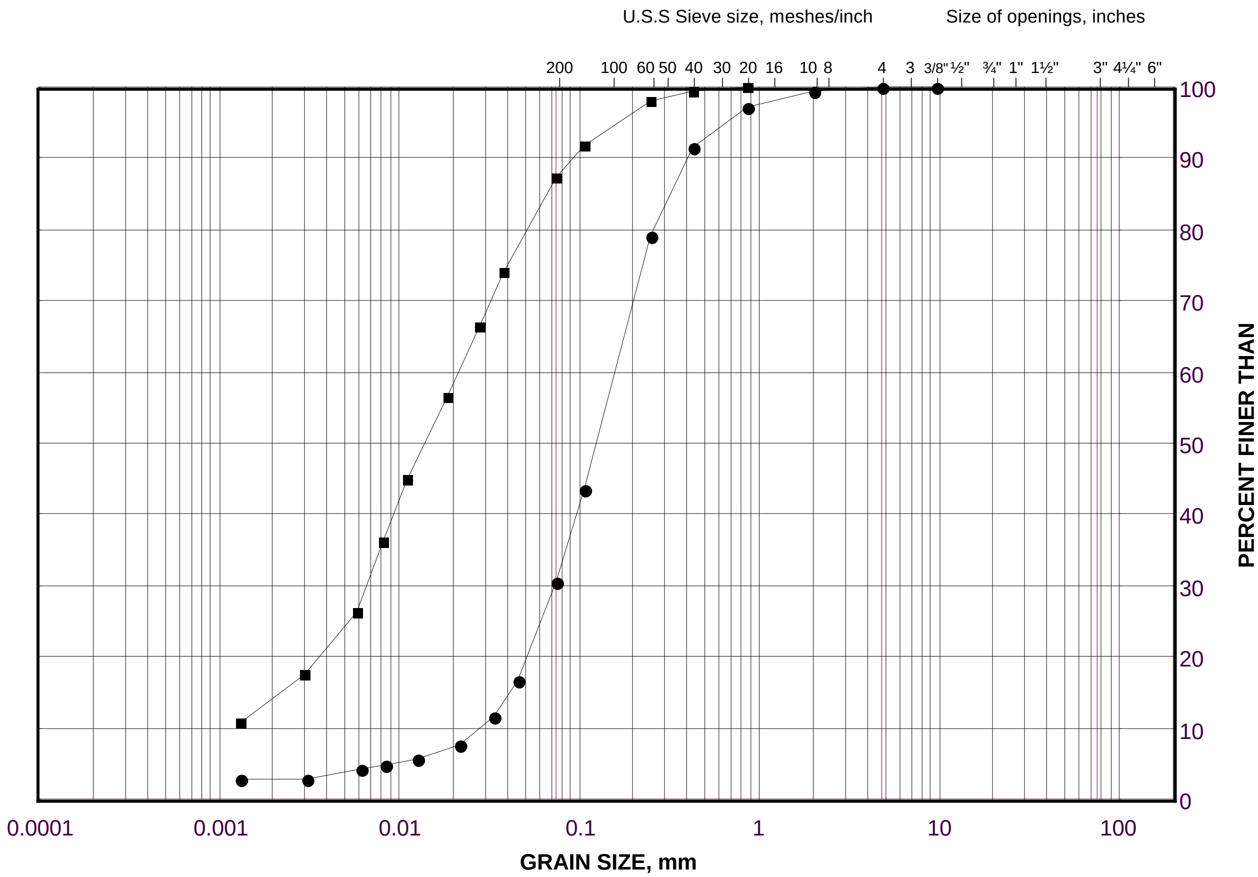
$+^3, \times^3$: Numbers refer to Sensitivity ○ $^{3\%}$ STRAIN AT FAILURE

APPENDIX B

Geotechnical Laboratory Test Results

GRAIN SIZE DISTRIBUTION
Upper Silty Sand and Organic Silt

FIGURE B1



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

LEGEND

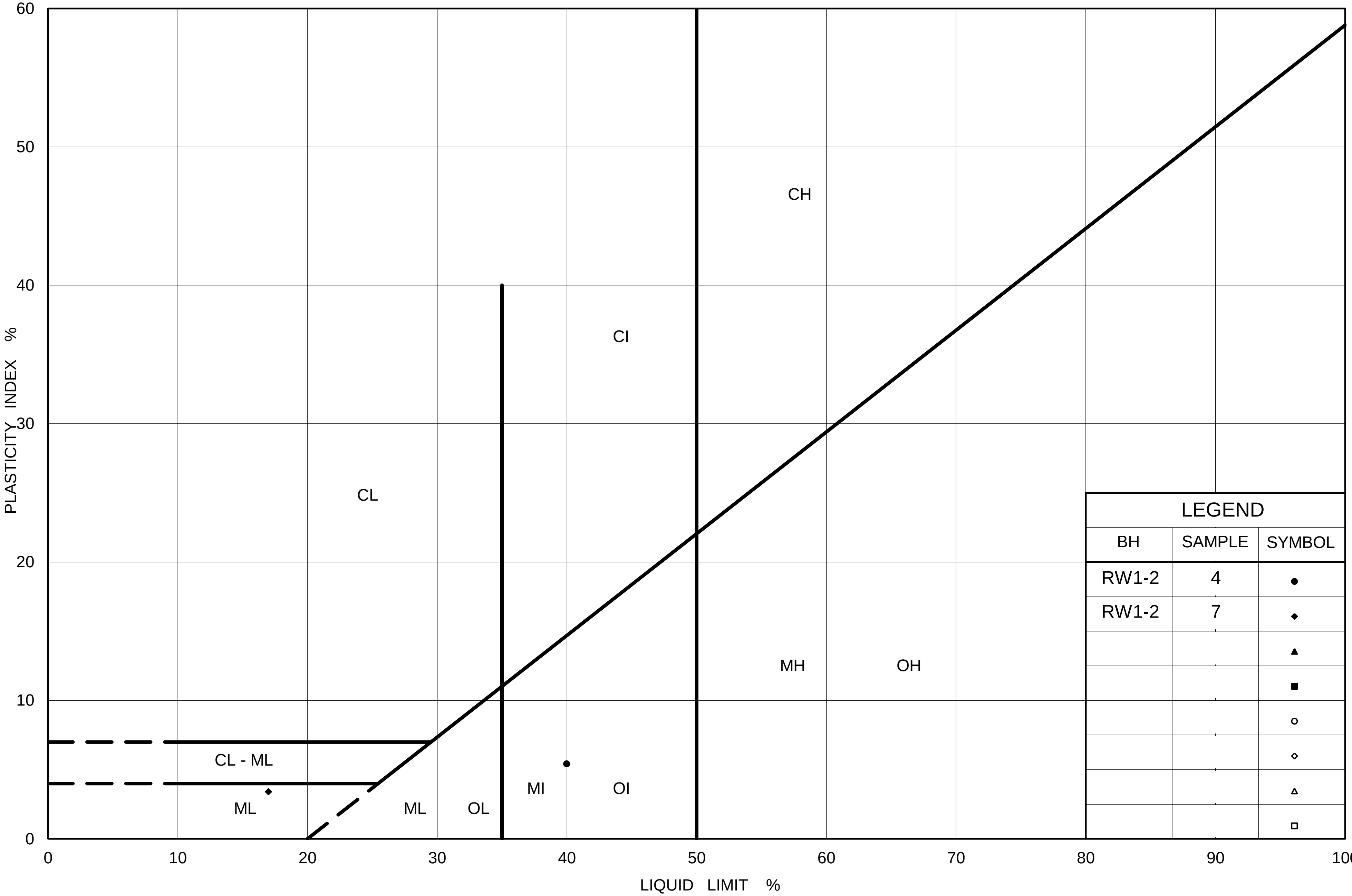
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	RW1-1	4	300.7
■	RW1-2	4	299.5

Project Number: 09-1111-0018

Checked By: LCC

Golder Associates

Date: 21-Aug-15



Ministry of Transportation

Ontario

PLASTICITY CHART
Upper Silt and Organic Silt

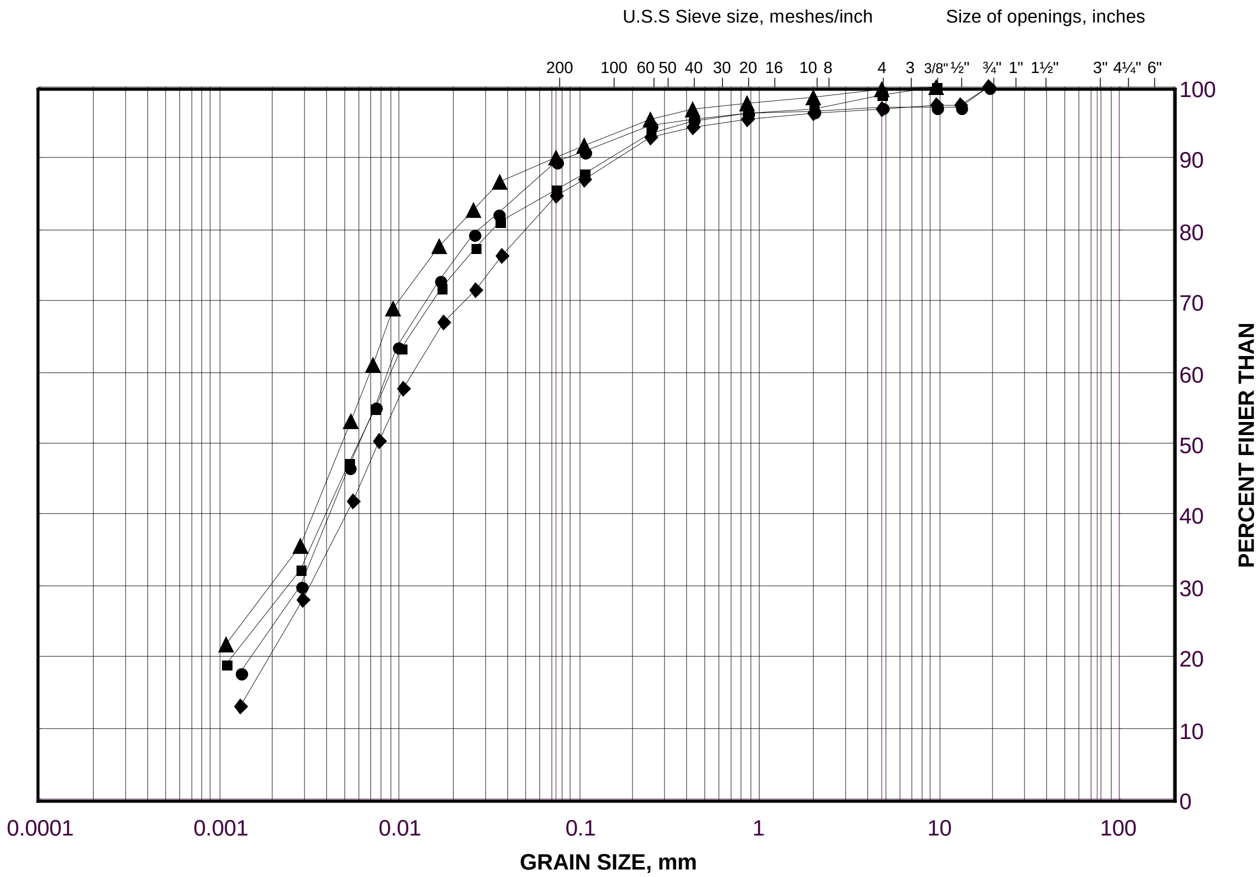
Figure No. B2

Project No. 09-1111-0018

Checked By: LCC

GRAIN SIZE DISTRIBUTION
Clayey Silt Till

FIGURE B3



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	RW1-1	12	290.8
■	RW1-3	4	302.3
◆	RW1-1	7	298.4
▲	RW1-2	9	294.2

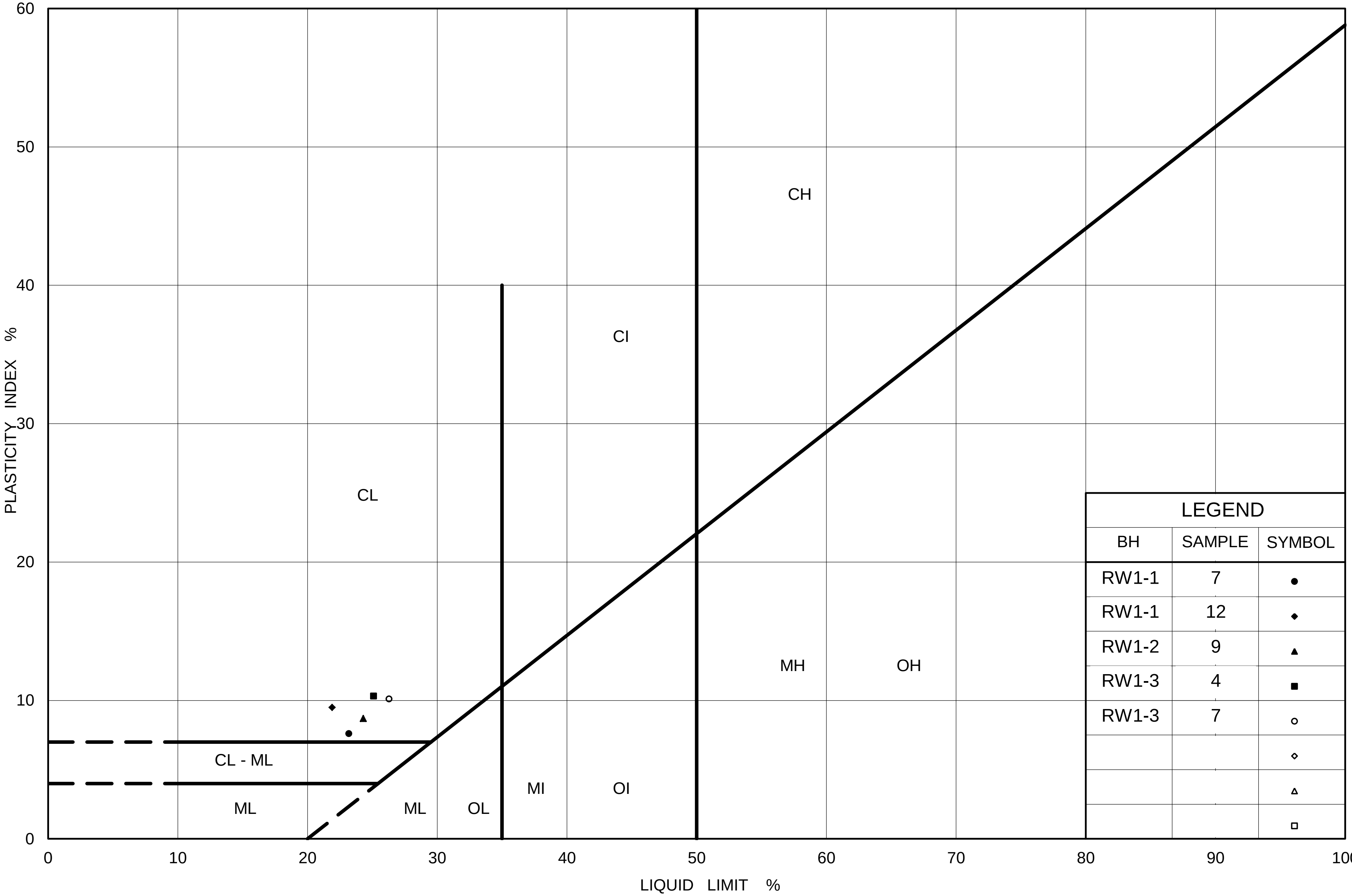
Project Number: 09-1111-0018

Checked By: LCC

Golder Associates

Date: 21-Aug-15

Oct 75, FF-S-21



LEGEND

BH	SAMPLE	SYMBOL
RW1-1	7	●
RW1-1	12	◆
RW1-2	9	▲
RW1-3	4	■
RW1-3	7	○
		◇
		△
		□

Ministry of Transportation

Ontario

PLASTICITY CHART
Clayey Silt Till

Figure No. B4

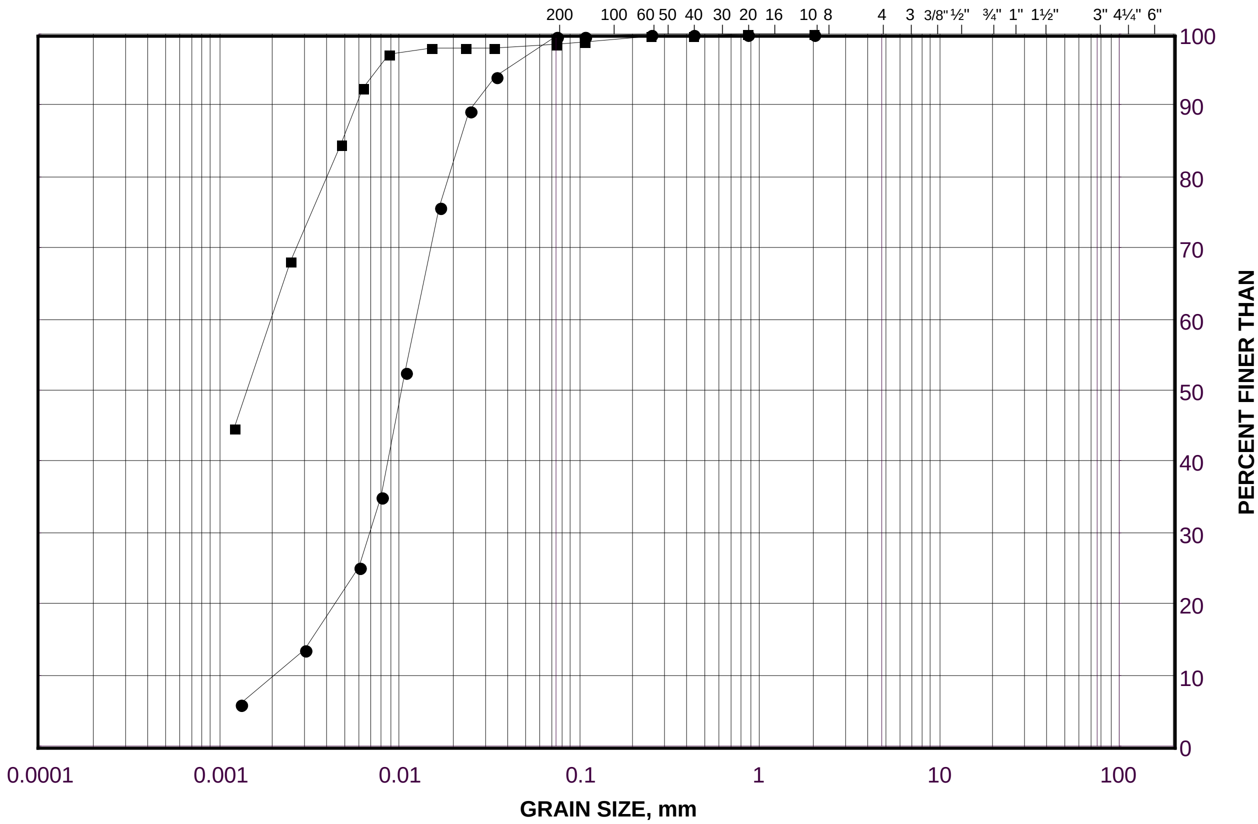
Project No. 09-1111-0018

Checked By: LCC

GRAIN SIZE DISTRIBUTION

Clayey Silt Interlayers

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)
●	RW1-3	10	295.5
■	RW1-1	9	295.4

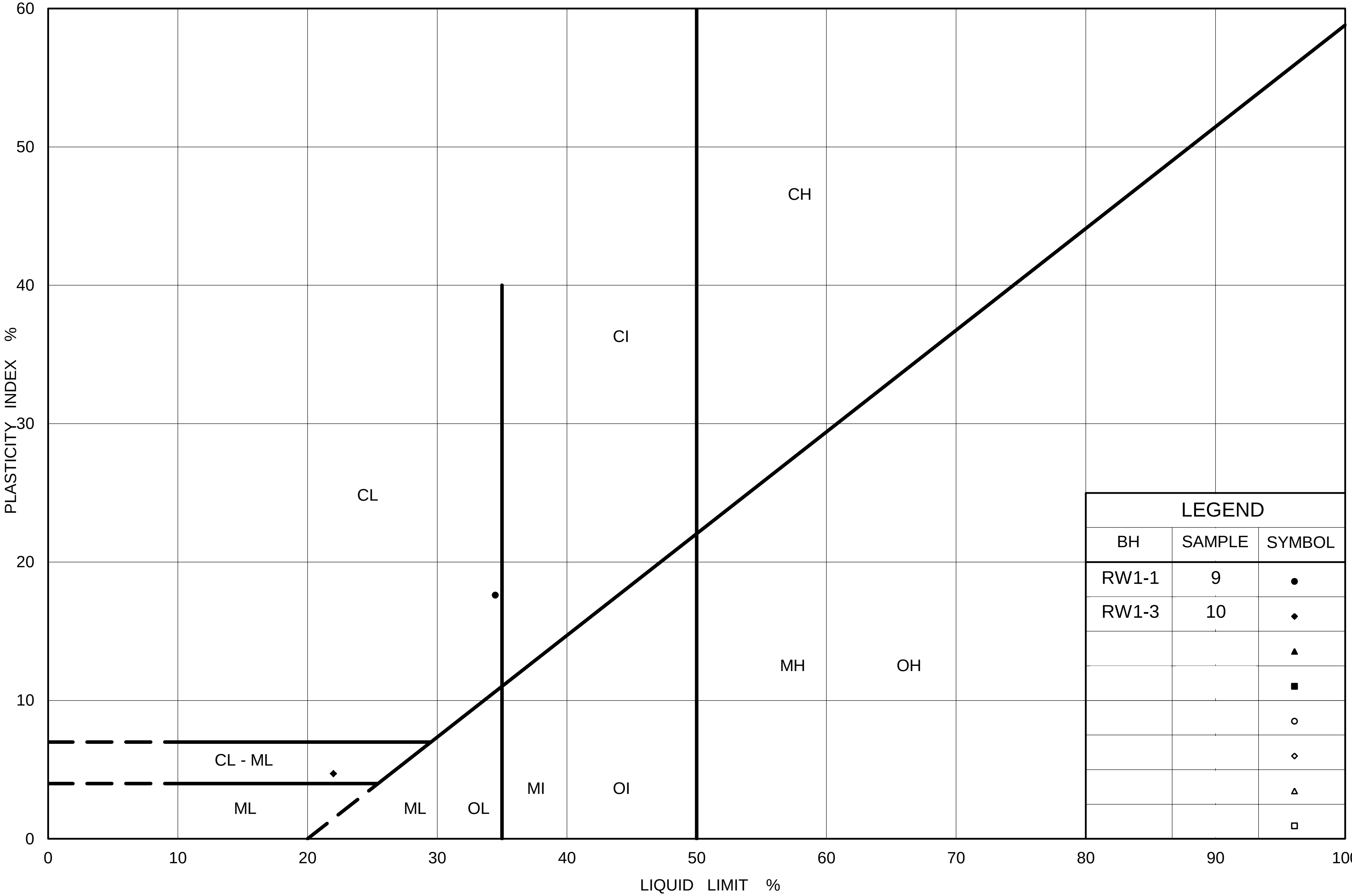
Project Number: 09-1111-0018

Checked By: LCC

Golder Associates

Date: 21-Aug-15

Oct 75, FF-S-21



LEGEND

BH	SAMPLE	SYMBOL
RW1-1	9	●
RW1-3	10	◆
		▲
		■
		○
		◇
		△
		□

Ministry of Transportation

Ontario

PLASTICITY CHART
Clayey Silt Interlayers

Figure No. B6

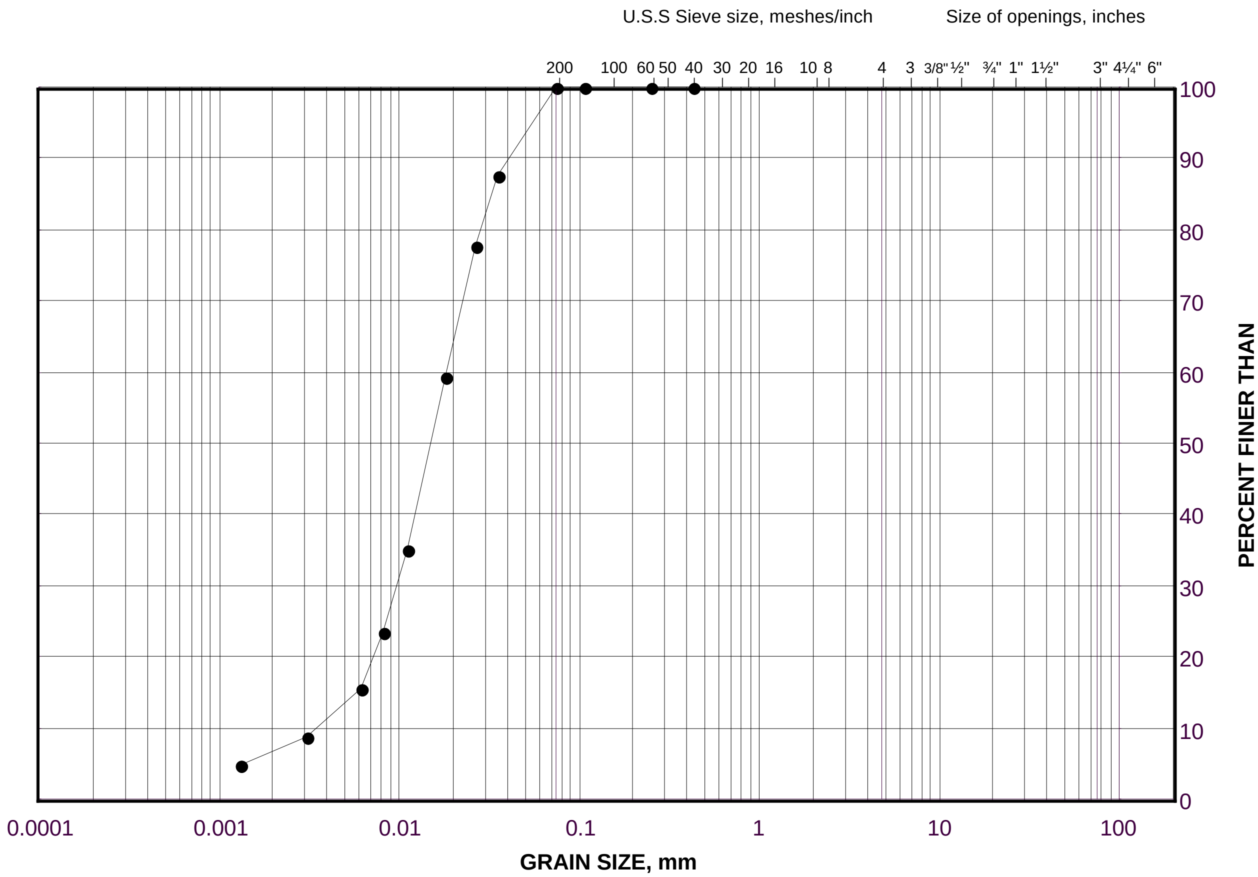
Project No. 09-1111-0018

Checked By: LCC

GRAIN SIZE DISTRIBUTION

Lower Silt

FIGURE B7



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	RW1-2	11	291.1

Project Number: 09-1111-0018

Checked By: LCC

Golder Associates

Date: 21-Aug-15

APPENDIX C

Non-Standard Special Provisions

OPERATIONAL CONSTRAINT – Subexcavation of Organic Silt

Special Provision

This special provision outlines the procedure to be used for subexcavation of the organic silt between approximately Station 15+375 and 15+430; the depth/elevation of subexcavation is shown on the Contract Drawings.

Staged excavation in strips of limited width shall be carried out to maintain the stability of the excavation and protection system along Highway 400 during the subexcavation and backfilling operations. The staged excavations procedures are outlined as follows:

a) The work may be carried out simultaneously from both ends of the area to be subexcavated, working towards the centre.

b) Removal of the soft to firm organic silt and overlying fill materials within the embankment widening and retaining wall footprint shall be carried out in short “strip” sections perpendicular to the Highway 400 alignment, with the base of the excavation (as measured parallel to Highway 400) not wider than 3 m.

c) Temporary excavation side slopes or back slopes through the organic silt and overlying fill materials shall be no steeper than 1 horizontal to 1 vertical (1H:1V).

d) Excavation and backfilling operations shall be carried out simultaneously in a manner that the excavation is not left open for more than the 3 m “strip” width at any given time.

Payment for the Contractor to provide the above requirements, including all equipment, labour and materials shall be deemed to be included in the contract bid price for the various tender items.

As a global, employee-owned organisation with over 50 years of experience, Golder Associates is driven by our purpose to engineer earth's development while preserving earth's integrity. We deliver solutions that help our clients achieve their sustainable development goals by providing a wide range of independent consulting, design and construction services in our specialist areas of earth, environment and energy.

For more information, visit golder.com

Africa	+ 27 11 254 4800
Asia	+ 86 21 6258 5522
Australasia	+ 61 3 8862 3500
Europe	+ 44 1628 851851
North America	+ 1 800 275 3281
South America	+ 56 2 2616 2000

solutions@golder.com
www.golder.com

Golder Associates Ltd.
6925 Century Avenue, Suite #100
Mississauga, Ontario, L5N 7K2
Canada
T: +1 (905) 567 4444