



August 12, 2015

FOUNDATION INVESTIGATION AND DESIGN REPORT

RETAINING WALLS

HIGHWAY 401 WIDENING FROM HIGHWAY 403/410
INTERCHANGE TO CREDIT RIVER
CITY OF MISSISSAUGA, REGION OF PEEL
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 2150-01-00

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REPORT





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**FOUNDATION REPORT - RETAINING WALLS
HIGHWAY 401 WIDENING, GWP 2150-01-00**

PART A

**FOUNDATION INVESTIGATION REPORT
RETAINING WALLS
HIGHWAY 401 WIDENING FROM HIGHWAY 403 / 410
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CITY OF MISSISSAUGA, REGION OF PEEL
MINISTRY OF TRANSPORTATION, ONTARIO
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by AECOM Canada Ltd. (AECOM) on behalf of the Ministry of Transportation, Ontario (MTO) to provide detail foundation engineering services for the proposed widening of Highway 401 from the Highway 403 / 410 Interchange to Credit River in the City of Mississauga, Region of Peel, Ontario.

This report addresses the foundation investigation carried out for the retaining walls along Highway 401 to accommodate the proposed Highway 401 widening. The purpose of this investigation is to establish the subsurface conditions at the retaining wall locations along the length of the proposed highway widening by borehole drilling, in situ testing and laboratory testing on selected samples.

The Terms of Reference for the foundation engineering services are outlined in MTO's Request for Proposal dated October 2010 and the associated MTO Clarification Packages No. 1 to 3 issued between October and November 2010, which forms part of the Consultant's Agreement Number 2010-E-0003 for this project. The work has been carried out in accordance with Golder's Supplemental Specialty Quality Control Plan for this project, dated April 2011. Foundation engineering services for three of the retaining walls along Highway 401, added to the scope subsequent to MTO's original Request for Proposal, are outlined in Golder's Revised Scope Change Request No. 8 dated November 10, 2014.

2.0 SITE DESCRIPTION

The proposed retaining walls are located along the length of the proposed Highway 401 widening between Mavis Road and Credit River in the City of Mississauga, Ontario. There are five (5) retaining walls, designated as Retaining Wall 1 through 5, which are generally in order of increasing chainage, along the proposed highway, as shown on Drawing 1, following the text of this report. Retaining Walls 1 to 3 are located west of Second Line West and Retaining Walls 4 and 5 are located east of Second Line West.

The topography across the site adjacent to Highway 401 consists of undulating terrain which generally slopes downward to the west towards the Credit River. Vegetation within the project limits generally varies, consisting of grass, shrubs and trees with the Highway cutting through deciduous forest and wetland areas between the Credit River and Second Line West. Fletcher's Creek is located within the flat floodplain and low-lying valley that traverses through the dense forest areas and the creek flows in a north-south direction, perpendicular to Highway 401. Fletcher's Creek has been identified as having areas of "quicksand" (i.e. loose sands with high groundwater pressures) and there are areas of high hydrostatic pressures (artesian conditions) within the floodplain bordering the creek. Residential properties are present along the Highway 401 corridor between Second Line West and Mavis Road, and commercial facilities are located along Highway 401 east of Mavis Road.

A detailed description of each area is presented in Sections 4.3 to 4.7.



3.0 INVESTIGATION PROCEDURES

The field work specifically for the retaining walls was carried out between July 5, 2014 and January 12, 2015 during which time a total of six (6) sampled boreholes (designated as Boreholes RW-1 to RW-4, SC8-3, and SC8-4) were advanced at the locations of the Retaining Walls 2, 4 and 5. In addition, the following boreholes advanced by Golder were utilized to supplement the current investigation: six (6) boreholes (designated as Boreholes FC-1 to FC-4, FC-13 and FC-13A) advanced as part of the field investigation work for the Fletcher's Creek Bridges¹; and seven (7) boreholes (designated as Boreholes DC-7 and DC-9 to DC-14) advanced as part of the field investigation work for the deep cut / high fill areas². The Record of Borehole sheets and the results of the laboratory testing for the boreholes are presented in Appendix A to Appendix E.

The details of each retaining wall and the locations of the boreholes advanced at each site are provided below and the borehole locations are shown on Drawings 2 to 7.

Retaining Wall Designation	Reference Drawing	Approximate Station	Boreholes Advanced	Appendix
Retaining Wall 1	Drawing 2	15+830 to 15+945	3 Boreholes (DC-9 to DC-11)	A
Retaining Wall 2	Drawing 3	15+985 to 16+142	4 Boreholes (SC8-3, SC8-4, FC-2, FC-4)	B
Retaining Wall 3	Drawing 4 and 5	16+179 to 16+250	5 Boreholes (FC-1, FC-3, FC-13, FC-13A, DC-12)	C
Retaining Wall 4	Drawing 6	16+315 to 16+480	4 Boreholes (RW-1, RW-2, DC-13, DC-14)	D
Retaining Wall 5	Drawing 7	16+345 to 16+445	3 Boreholes (RW-3, RW-4, DC-7)	E

The various field investigations were carried out using CME 55 track-mounted and CME 75 truck-mounted drill rigs supplied and operated by Geo-Environmental Drilling Inc. of Halton Hills, Ontario. The boreholes were advanced through the overburden using 57 mm, 76 mm or 108 mm inner diameter (I.D.) hollow-stem augers, 150 mm outer diameter (O.D.) solid-stem augers, and / or 'NW' casing using Tricone and wash boring techniques as required. Soil samples were obtained in the boreholes at 0.75 m and 1.5 m intervals of depth using a 50 mm outer diameter split-spoon sampler driven by an automatic hammer in accordance with Standard

¹ Golder Associates Ltd. March 2013. *Foundation Investigation Report, Fletcher's Creek Bridges, Highway 401 Widening from Highway 403/410 Interchange to the Credit River, City of Mississauga, Region of Peel, GWP 2150-01-00.* GEOCRE No. 30M12-356.

² Golder Associates Ltd. December 2013. *Foundation Investigation Report, Deep Cut / High Fill Areas 1 to 9, Highway 401 Widening from Highway 403/410 Interchange to the Credit River, City of Mississauga, Region of Peel, GWP 2150-01-00.* GEOCRE No. 30M12-364.



Penetration Test (SPT) procedures (ASTM D1586)³. Field vane shear tests were conducted in cohesive soils for determination of undrained shear strengths using a MTO standard 'N' size vane (ASTM D2573)⁴.

The boreholes advanced during the current investigation were extended to a depth of about 8.2 m below existing ground surface, while the boreholes utilized to supplement this investigation were extended to depths between about 5.2 m and 17.8 m below ground surface. Some boreholes were terminated in very dense relative density conditions. Additionally, one borehole was terminated due to refusal to further casing advancement and a dynamic cone penetration test (DCPT) driven from the bottom of the borehole which may be inferred to indicate the potential presence of boulders or proximity to the bedrock surface.

The groundwater conditions in the open boreholes were observed during and immediately following the drilling operations. Standpipe piezometers were installed in selected boreholes to permit monitoring of the water level pertinent to the retaining wall sites. The installed piezometers consist of a 50 mm diameter PVC pipe, with a 1.5 m slotted screen sealed within a filter sand pack at a select depth within the borehole. The borehole and annulus surrounding the piezometer pipe above the filter sand pack were backfilled to the ground surface with bentonite pellets. Piezometer installation details and water level readings are described on the Record of Borehole sheets included in Appendices A to E that correspond to Retaining Walls 1 to 5, respectively. All boreholes in which standpipe piezometers were not installed were backfilled to ground surface with bentonite upon completion, in accordance with Ontario Regulation 903, Wells (as amended). The boreholes advanced through the Second Line West roadway asphalt were sealed at the surface with cold patch asphalt, up to approximately 0.3 m thick.

The field work was monitored on a full-time basis by a member of Golder's technical staff who located the boreholes in the field, directed the sampling and in situ testing operations, logged the boreholes and examined and cared for the soil samples.

The recovered soil samples were identified in the field, placed in labelled containers and transported to Golder's laboratory in Mississauga for visual identification. Selected samples were subjected to a laboratory testing program consisting of natural moisture content, Atterberg limits and grain size distribution analyses conducted in our Whitby and Mississauga Laboratories in accordance with MTO and / or ASTM Standards as applicable. The results of this testing program are shown on the Record of Borehole sheets and the laboratory test figures contained in Appendices A to E.

The borehole locations were staked / marked in the field by Golder personnel relative to the proposed centreline of the new highway and ramp alignments, and the site features. The staked and as-drilled boreholes were surveyed by J.D. Barnes Ltd, a licensed surveying company retained by AECOM. The as-drilled borehole locations (referenced to MTM NAD83 northing and easting coordinates) and ground surface elevations (referenced to Geodetic datum) are provided on the individual Record of Borehole sheets, on Drawings 2 to 7 and are summarized below.

³ ASTM D1586 – Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of Soils.

⁴ ASTM D2573 – Standard Test Method for Field Vane Shear Test in Cohesive Soil.



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Retaining Wall Site	Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)	Borehole Depth (m)
Retaining Wall 1 15+830 to 15+945	DC-9	4,830,722.1	287,015.8	171.1	9.8
	DC-10	4,830,738.9	287,051.3	170.5	12.8
	DC-11	4,830,761.2	287,082.8	173.1	8.2
Retaining Wall 2 15+985 to 16+142	SC8-3	4,830,771.3	287,144.6	164.7	8.1
	SC8-4	4,830,812.3	287,208.7	164.5	10.8
	FC-2	4,830,851.3	287,283.0	163.8	10.4
	FC-4	4,830,837.9	287,276.8	164.4	5.8
Retaining Wall 3 16+179 to 16+250	FC-1	4,830,867.9	287,311.4	163.7	15.9
	FC-3	4,830,871.4	287,325.3	165.9	5.2
	FC-13	4,830,881.8	287,336.5	167.1	10.8
	FC-13A	4,830,881.8	287,335.0	167.1	17.8
	DC-12	4,830,923.2	287,393.6	176.3	12.8
Retaining Wall 4 16+315 to 16+480	RW-1	4,831,002.0	287,527.6	175.2	8.2
	RW-2	4,831,027.1	287,570.9	176.0	8.2
	DC-13	4,830,934.4	287,419.4	174.7	11.3
	DC-14	4,830,962.3	287,473.2	175.4	8.2
Retaining Wall 5 16+345 to 16+445	RW-3	4,830,889.5	287,546.9	173.7	8.2
	RW-4	4,830,913.2	287,595.9	174.4	8.2
	DC-7	4,830,868.9	287,506.4	175.1	11.3

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

This section of Highway 401 is located in the Peel Plain close to the border of the South Slope physiographic region, as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984)⁵.

The Peel Plain physiographic region covers the central portions of the Regional Municipalities of York, Peel and Halton. The general topography of this region consists of level to gently rolling terrain, sloping gradually southward toward Lake Ontario. A surficial till sheet, which generally follows the surface topography, is present throughout much of this area. The till, which is mapped in this area as the Halton Till, typically consists of clayey silt to silty clay, with occasional sand to silt zones. Shallow, localized deposits of loose sand and silt and / or soft clay can overlie this uppermost till sheet, and these represent relatively recent deposits, formed in small glacial melt water ponds scattered throughout the Peel Plain and concentrated near river valleys. The recent sand, silt and clay and uppermost till deposits in this area overlie and are interbedded with stratified deposits of sand, silt and clay. The overburden within the majority of the Peel Plain area is underlain by shale bedrock of the Georgian Bay Formation which contains limestone interlayers.

⁵ Chapman, L.J. and Putnam, D.F. 1984. *The Physiography of Southern Ontario*, Ontario Geological Survey, Special Volume 2, Third Edition. Accompanied by Map P. 2715, Scale 1:600,000.



4.2 General Overview of Subsurface Conditions

The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of the in situ and laboratory tests are provided on the Record of Borehole sheets. The results of the in situ field tests (i.e., SPT “N”-values and field vane undrained shear strength values) as presented on the Record of Borehole sheets, on the stratigraphic profiles and in Section 4 are uncorrected.

The stratigraphic boundaries shown on the borehole records and on the interpreted stratigraphic profiles on Drawings 2, 3 and 5 to 7 are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Variation in the stratigraphic boundaries between and beyond boreholes will exist and is to be expected, however, the factual data presented in the Record of Borehole sheets governs any interpretation of the site conditions. The orientation (i.e. north, south, east, west) stated in the text of the report is typically referenced to project north and/or up-chainage along the proposed highway widening and new ramp alignments.

In general, the stratigraphy encountered at the various borehole locations typically consists of surficial layers of asphalt, topsoil and fill underlain by alternating layers and interlayers of cohesive and / or non-cohesive soils and cohesive and / or non-cohesive glacial till deposits.

Detailed descriptions of the subsurface conditions at each investigated retaining wall are provided in the following sections of this report. Where relatively significant thicknesses of overburden were encountered, the various soil types are described in detail for each main deposit.

4.3 Highway 401 – STA. 15+830 to STA. 15+945 (Retaining Wall 1)

The plan and profile along the proposed retaining wall showing the borehole locations and interpreted stratigraphy between about STA. 15+830 and STA. 15+945 are shown on Drawing 2. The Record of Borehole sheets (Boreholes DC-9-DC-11) and the laboratory test results for this area are presented in Appendix A. The proposed alignment within this section of the highway is in a cut, located along the north side of the existing Highway 401 WBL about 250 m west of Fletcher’s Creek. The topography of this section of the site is an undulating densely treed hill that slopes to the west towards a relatively flat, open area leading to the Credit River and a plateau to the east towards Fletcher’s Creek.

The subsurface soils encountered along the proposed alignment in this area consist of a surficial layer of topsoil and near surface layers of clayey silt and silty sand, underlain by upper and lower deposits of clayey silt till which are separated by a deposit of clayey silt. Interlayers of sandy silt, and sand and silt till were encountered overlying or within the lower clayey silt till deposit.

4.3.1 Topsoil

An approximately 100 mm to 200 mm thick surficial layer of topsoil was encountered at the ground surface in all the boreholes advanced at this site.



4.3.2 Clayey Silt (Near Surface)

A near surface layer of brown clayey silt, trace sand, trace gravel and roots was encountered underlying the topsoil in Borehole DC-11. The surface of the clayey silt layer is at about Elevation 172.9 m and the thickness of the layer is about 0.6 m.

A Standard Penetration Test (SPT) "N"-value of 12 blows per 0.3 m of penetration was recorded within this layer, suggesting that the clayey silt layer has a stiff consistency.

4.3.3 Silty Sand (Near Surface)

An approximately 0.7 m thick layer of brown silty sand, trace clay and trace gravel was encountered underlying the clayey silt layer at about Elevation 172.3 m in Borehole DC-11.

A SPT "N"-value of 50 blows per 0.05 m of penetration was recorded within this layer, indicating a very dense relative density. The natural water content measured on one (1) sample of this deposit is about 7 per cent.

4.3.4 Clayey Silt to Sandy Clayey Silt Till (Upper Deposit)

An upper deposit of brown to grey cohesive till was encountered underlying the topsoil in Boreholes DC-9 and DC-10 and below the near surface layer of silty sand in Borehole DC-11. The top of this deposit ranges from about Elevations 171.6 m to 170.3 m and the thickness of this upper cohesive till deposit ranges from about 3.6 m to 4.4 m. The cohesive till deposit is comprised of clayey silt, some sand to sandy, and trace gravel. The upper 0.6 m and 1.5 m portions of the cohesive deposit in Boreholes DC-9 and DC-10 contain roots.

The SPT "N"-values measured within the upper cohesive till deposit range from 7 blows to 37 blows per 0.3 m of penetration, and typically are greater than 15 blows per 0.3 m of penetration, suggesting a firm to hard (but typically very stiff to hard) consistency.

Grain size distribution tests were carried out on three (3) selected samples of the deposit and the results are shown on Figure A1 in Appendix A. Atterberg limits tests were carried out on three (3) samples of this cohesive till deposit and measured liquid limits ranging from about 25 per cent to 35 per cent, plastic limits ranging from about 15 per cent to 19 per cent, and plasticity indices ranging from about 10 per cent to 16 per cent. The results of the Atterberg limits tests are shown on a plasticity chart on Figure A2 in Appendix A, and indicate that the till deposit is clayey silt of low plasticity. The natural water content measured on seven (7) samples of this cohesive till deposit ranges from about 8 per cent to 16 per cent.

4.3.5 Clayey Silt

A deposit of clayey silt, trace sand and trace gravel was encountered underlying the upper cohesive till deposit in all the boreholes advanced at this site; this layer separated the upper and lower cohesive till deposits. The surface of the clayey silt deposit ranges from about Elevations 167.4 m to 165.9 m and the thickness of this deposit is between about 1.4 m and 2.4 m, generally becoming thinner towards the eastern limit of the investigated area.



The SPT “N”-values recorded within the clayey silt deposit range from 2 blows to 11 blows per 0.3 m of penetration. In situ field vane tests carried out within this deposit measured undrained shear strengths ranging from about greater than 135 kPa to 170 kPa, and the sensitivity is calculated to be about 1 and 2. The field vane tests results together with the SPT “N”-values suggest that the clayey silt deposit has a soft to very stiff (but typically very stiff) consistency.

Atterberg limits tests were carried out on three (3) selected samples of the clayey silt deposit and measured liquid limits ranging from about 28 per cent to 32 per cent, plastic limits ranging from about 16 per cent to 17 per cent and plasticity indices ranging from about 12 per cent to 15 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure A3 in Appendix A, and indicate that the deposit consists of clayey silt of low plasticity. The natural water content measured on three (3) samples of this deposit ranges from about 20 per cent to 23 per cent.

4.3.6 Sandy Silt (Interlayer)

Underlying the clayey silt deposit in Borehole DC-9, an approximately 1.2 m thick interlayer of grey sandy silt, trace to some clay, was encountered at about Elevation 165.0 m.

A SPT “N”-value of 28 blows per 0.3 m of penetration was measured within this layer, indicating a compact relative density.

The result of the grain size distribution test carried out on one (1) sample of the sandy silt layer is presented on Figure A4 in Appendix A. The measured water content on one (1) sample of this layer is about 14 per cent.

4.3.7 Clayey Silt to Sandy Clayey Silt Till (Lower Deposit)

A lower cohesive till deposit was encountered underlying the sandy silt interlayer in Borehole DC-9 and below the clayey silt deposit in Boreholes DC-10 and DC-11. The top of this cohesive till deposit was encountered at depths between about 7.0 m and 7.3 m below ground surface, between about Elevations 165.9 m and 163.5 m. The thickness of the lower cohesive till deposit encountered in the boreholes ranges from about 1.0 m to 4.3 m. All boreholes were terminated within this deposit between about Elevations 164.9 m and 157.7 m.

The cohesive till deposit consists of clayey silt, trace to some sand to sandy, and trace to some gravel, with a silt and sand till interlayer encountered within the cohesive till deposit in Borehole DC-10.

The SPT “N”-values recorded within this lower cohesive till deposit range from 16 blows to 32 blows per 0.3 m of penetration, suggesting that the clayey silt till has a very stiff to hard consistency.

Grain size distribution tests were carried out on two (2) selected samples of the cohesive till deposit and the results are provided on Figure A5 in Appendix A. Atterberg limits tests were performed on two (2) samples of the cohesive till and measured liquid limits of about 19 per cent and 20 per cent, plastic limits of about 13 per cent and 14 per cent, and corresponding plasticity indices of about 6 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure A6 in Appendix A and indicate that this till deposit consists of clayey silt of low plasticity. The measured water content on three (3) selected samples of this till deposit is between about 9 per cent and 10 per cent.



4.3.8 Silt and Sand Till (Interlayer)

In Borehole DC-10, an approximately 1.5 m thick interlayer of silt and sand till, trace to some clay and trace to some gravel, was encountered within the lower clayey silt till deposit between about Elevations 160.1 m and 158.6 m.

A SPT "N"-value of 26 blows per 0.3 m of penetration was recorded within this till interlayer, indicating a compact relative density. The result of a grain size distribution test carried out on a sample of this till interlayer is shown on Figure C7 in Appendix C. The natural water content measured on one (1) sample of the sand and silt till is about 11 per cent.

4.3.9 Groundwater Conditions

The samples taken in the boreholes were generally moist to wet. The groundwater levels in the open boreholes were measured upon completion of drilling operations. A standpipe piezometer was installed in Borehole DC-10 to permit monitoring of the groundwater level at this site. Details of the piezometer installation and the measured groundwater levels are shown on the Record of Borehole sheets in Appendix A. The groundwater levels recorded in the open boreholes and piezometer are summarized below.

Borehole No.	Ground Surface Elevation (m)	Depth to Water Level (m)	Groundwater Elevation (m)	Date	Comments
DC-9	171.1	2.3	168.8	September 07, 2012	Open Borehole
DC-10	170.5	12.0	158.5	September 11, 2012	Piezometer
		1.9	168.6	October 09, 2012	
		2.0	168.5	November 05, 2012	
DC-11	173.1	Dry	--	September 7, 2012	Open Borehole

The groundwater level observations at this site are short term and will be subject to seasonal fluctuations and precipitation events, therefore the water levels should be expected to be higher during the spring season or during any period of heavy precipitation.

4.4 Highway 401 – STA. 15+985 to STA. 16+142 (Retaining Wall 2)

The plan and profile along the proposed retaining wall showing the borehole locations and interpreted stratigraphy between about STA. 15+985 and STA. 16+142 are presented on Drawing 3. The Record of Borehole sheets (Boreholes FC-2, FC-4, SC8-3 and SC8-4) and the laboratory test results on soil samples for this area are presented in Appendix B. The proposed alignment of the retaining wall within this section of the highway is located north of the existing Highway 401 WBL immediately west of Fletcher's Creek. The topography of this section of the site is a relatively flat plateau of the top of a densely treed hill that slopes to the west towards a relatively flat open area leading to the Credit River and to the east towards Fletcher's Creek.



The subsurface conditions generally consist of a surficial layer of topsoil, underlain by fill in places, further underlain by a clayey silt to silty clay deposit. The clayey silt to silty clay deposit is underlain by a non-cohesive to cohesive till deposit.

4.4.1 Topsoil

A 50 mm to 200 mm thick surficial layer of topsoil was encountered at the ground surface in all of the boreholes.

4.4.2 Fill

A 1.3 m thick fill layer was encountered underlying the topsoil in Borehole FC-2 and FC-4 at about Elevation 163.6 m and Elevation 164.2 m, respectively.

The fill is comprised of clayey silt containing some gravel and some sand. Organics and rootlets were also encountered within this layer.

The SPT "N"-values measured within the clayey silt fill range are 5 blows and 6 blows per 0.3 m of penetration, suggesting that the deposit has a firm consistency.

4.4.3 Clayey Silt to Silty Clay

A cohesive deposit of brown to grey clayey silt to clayey silt with sand to silty clay was encountered below the fill in Boreholes FC-2 and FC-4 and below the topsoil in Boreholes SC8-3 and SC8-4. The deposit was encountered between about Elevations 164.6 m and 162.3 m and the thickness of the deposit was between about 2.0 m and 3.8 m. In Borehole SC8-4, the deposit is interlayered with clayey silt with sand till from about Elevation 162.2 m to 160.9 m as described in Section 4.4.4.

The cohesive deposit is generally comprised of clayey silt to silty clay containing some sand to with sand and trace to some gravel. Rootlets were encountered in Boreholes SC8-3, SC8-4 and FC-4 in the upper 2.2 m of the deposit. The presence of cobbles is also inferred from difficulties advancing augers (auger grinding) in Borehole FC-4 at a depth of about 5.2 m (about Elevation 159.2 m) during the drilling operations.

The SPT "N"-values measured within the silty clay to clayey silt deposit range from 0 blows (weight of hammer) to 23 blows per 0.3 m of penetration. In situ field vane tests carried out within this deposit in Boreholes FC-2 and FC-4 measured undrained shear strengths ranging from about 8 kPa to 95 kPa, with the sensitivity calculated to be between about 1 and 9. The field vane tests together with the SPT "N"-value results suggest that the deposit generally has a very soft to stiff consistency.

A grain size distribution test was carried out on one (1) samples of the clayey silt portion of the deposit and the result is provided on Figure B1 in Appendix B.

Atterberg limits tests were carried out on five (5) samples of this deposit. The liquid limits range from about 20 per cent to 46 per cent, the plastic limits range from about 15 per cent to 27 per cent and the plasticity indices range from about 5 per cent to 19 per cent. The results of the Atterberg limits tests are shown on a plasticity chart on Figure B2 in Appendix B, and indicate that this deposit consists of clayey silt of low plasticity to silty clay



of intermediate plasticity. The natural water content measured on seven (7) samples of this silty clay to clayey silt deposit ranges from about 11 per cent to 34 per cent.

4.4.4 Clayey Silt (Till)

A cohesive till deposit was encountered underlying the silty clay deposit in Borehole SC8-3 and interlayered in the silty clay to clayey silt deposit in Borehole SC8-4. The till deposit generally consists of grey silty clay to clayey silt containing trace to with sand and trace to some gravel. The deposit was encountered at about Elevation 162.5 m and 162.2 m in Boreholes SC8-3 and SC8-4, respectively, and the thickness of the deposit is of about 3.4 m and 1.3 m, respectively.

The SPT “N”-values measured within the cohesive till range from 3 blows to 11 blows per 0.3 m of penetration, indicating a soft to stiff relative density.

Grain size distribution tests were carried out on two (2) samples of the till deposit and the results are provided on Figure B3 in Appendix B.

Atterberg limits tests were carried out on three (3) samples of this deposit. The liquid limits range from about 19 per cent to 28 per cent, the plastic limits range from about 13 per cent to 15 per cent and the plasticity indices range from about 6 per cent to 13 per cent. The results of the Atterberg limits tests are shown on a plasticity chart on Figure B4 in Appendix B, and indicate that the till consists of clayey silt of low plasticity. The natural water content measured on five (5) samples of the till ranges from about 12 per cent to 20 per cent.

4.4.5 Gravelly Silty Sand to Sand and Gravel (Till)

A non-cohesive till deposit was encountered underlying the silty clay to clayey silt deposit in Boreholes SC8-4, FC-2 and FC-4 and underlying the cohesive till deposit in Borehole SC8-3. The deposit was encountered at depths ranging from about 5.2 m to 5.6 m below ground surface corresponding to between about Elevations 158.6 m and 159.1 m. These boreholes were terminated within this deposit at depths of about 5.8 m to 10.8 m below the ground surface (Elevations 158.6 m to 153.4 m), penetrating the deposit for a thickness ranging from about 0.5 m to 5.2 m.

The non-cohesive till deposit varies in composition from sand and gravel containing trace to some silt and trace clay; to gravelly sand containing trace to some silt and trace clay; to gravelly silty sand containing trace clay; to sand and silt containing some gravel and trace to some clay; to sandy silt and gravel containing trace clay. The presence of cobbles or boulders is also inferred from difficulties advancing augers (auger grinding) in Borehole SC8-3 at a depth of about 6.6 m (about Elevation 158.1 m) during the drilling operations.

The SPT “N”-values measured within the non-cohesive till generally range from 45 blows to 93 blows per 0.3 m of penetration, indicating a dense to very dense relative density. SPT “N”-values as high as 100 blows per 0.2 m of penetration were recorded within the non-cohesive till.

Grain size distribution tests were carried out on five (5) selected samples of the non-cohesive till deposit and the results are shown on Figures B5 in Appendix B. The natural water content measured on six (6) selected samples of the non-cohesive till deposit ranges from about 5 per cent to 11 per cent.



4.4.6 Groundwater Conditions

In general, the samples taken in the boreholes were moist to wet. The groundwater levels in the open boreholes were measured upon completion of drilling operations. Details of the measured groundwater levels are shown on the Record of Borehole sheets in Appendix B. Artesian groundwater conditions were observed during the drilling operations in Boreholes FC-2 where the groundwater level measured above the ground surface. The groundwater levels recorded in the open boreholes are summarized below.

Borehole No.	Ground Surface Elevation (m)	Depth to Water Level (m)	Groundwater Elevation (m)	Date	Comments
SC8-3	164.7	4.6	160.1	January 11, 2015	Open Borehole
SC8-4	164.5	1.8	162.7	January 12, 2015	Open Borehole
FC-2 ¹	163.8	Above 0.02	Higher than 163.8 ²	May 8, 2012	Inside Casing
FC-4 ³	164.4	2.0	162.4	May 9, 2014	Inside Augers

1. Water level not considered stabilized as water was flowing out of the top of the casing.
2. Artesian Conditions
3. Water level not considered stabilized given that wash boring methods were used and water was introduced into the borehole.

Based on the groundwater levels recorded during this investigation, high hydrostatic pressures are present at this site. The artesian hydrostatic head present in the cohesionless layers at this site is estimated to be higher than Elevation 164 m. Perched water conditions will also be present and the estimated perched groundwater level is inferred to be equivalent to the Fletcher's Creek water level. Based on the 1957 drawing, the creek high water level is at approximately Elevation 164.5 m and the normal water level is at Elevation 162.9 m.

The groundwater level observations at this site are short term and will be subject to seasonal fluctuations and precipitation events, therefore the water levels should be expected to be higher during the spring season or during any period of heavy precipitation.

4.5 Highway 401 – STA. 16+179 to STA. 16+250 (Retaining Wall 3)

The plan and profile along the proposed retaining wall showing the borehole locations and interpreted stratigraphy between about STA. 16+179 and STA. 16+250 are shown on Drawings 4 and 5, respectively. The Record of Borehole sheets (Boreholes FC-1, FC-3, FC-13/13A and DC-12) and the laboratory test results on soil samples from the boreholes for this site are presented in Appendix C. The proposed retaining wall alignment within this section of the highway is located north of the existing Highway 401 WBL, east of Fletcher's Creek and west of the Second Line West bridge underpass.

The subsurface conditions at the site consist of a surficial layer of asphalt or topsoil over a deposit of fill associated with the Highway 401 embankments. The fill is underlain by either a deposit of clayey silt to clayey silt with sand or sandy clayey silt till. The cohesive/cohesive till deposit is underlain by non-cohesive till deposits consisting of sandy silt to gravelly silt and sand, and gravelly sand to sand and gravel.



4.5.1 Asphalt

An approximately 200 mm thick layer of asphalt was encountered in Borehole DC-12, advanced through the roadway pavement on Second Line West.

4.5.2 Topsoil

A 200 mm thick surficial layer of topsoil was encountered at the ground surface in Borehole FC-1.

4.5.3 Fill

Fill materials were encountered underlying the topsoil in Borehole FC-1, immediately below the ground surface in Borehole FC-3, and underlying the asphalt in Borehole DC-12. The thickness of the fill deposit is variable across the site, ranging from about 0.9 m in Borehole FC-3 to 5.6 m in Borehole DC-12 and was encountered between about Elevations 165.9 m to 163.5 m and at Elevation 176.1 m. In Borehole DC-12, the fill is comprised of silty sand, trace clay and trace gravel (inferred to comprise the base / subbase for the pavement structure) below the asphalt layer underlain by cohesive fill consisting of sandy clayey silt, trace gravel, and in Boreholes FC-1 and FC-3 the fill consists of clayey silt some sand, trace to some gravel, and rootlets.

A SPT "N"-value of 12 blows per 0.3 m of penetration was measured within the non-cohesive portion of the fill, indicating a compact relative density. SPT "N"-values measured within the cohesive fill range from 3 blows to 22 blows per 0.3 m of penetration, suggesting a soft to very stiff consistency.

A grain size distribution test was carried out on one (1) selected sample of the clayey silt fill and the result is provided on Figure C1 in Appendix C.

An Atterberg limits test was carried out on one (1) sample of the clayey silt fill and measured a liquid limit of about 27 per cent, a plastic limit of about 16 per cent, and a corresponding plasticity index of about 11 per cent. The result of the Atterberg limits test is shown on the plasticity chart on Figure C2 in Appendix C and indicates that the fill material consists of clayey silt of low plasticity. The natural water content measured on four (4) samples of the clayey silt fill ranges between about 5 per cent and 20 per cent.

4.5.4 Sandy Clayey Silt (Till) (Upper Deposit)

A predominantly cohesive till deposit consisting of brown sandy clayey silt with trace gravel was encountered underlying the fill materials in Borehole DC-12. The surface elevation of the cohesive till deposit is at about Elevation 170.5 m. The deposit is about 3 m thick.

The SPT "N"-values measured within the sandy clayey silt till are 25 blows to 32 blows per 0.3 m of penetration, suggesting that the clayey silt till has a very stiff to hard consistency.

A grain size distribution test was carried out on one (1) selected sample of the sandy clayey silt till and the result is provided on Figure C3 in Appendix C.



An Atterberg limits test carried out on one (1) sample of the sandy clayey silt till and measured a liquid limit of about 29 per cent, a plastic limit of about 18 per cent, and a corresponding plasticity index of about 11 per cent. The result of the Atterberg limits test is shown on the plasticity chart on Figure C4 in Appendix C and indicates that the fill material consists of clayey silt of low plasticity. The natural water content measured on one (1) sample of the clayey silt till was about 14 per cent.

4.5.5 Clayey Silt with Sand to Clayey Silt

A cohesive deposit of brown to grey clayey silt with sand to clayey silt was encountered in all of the boreholes either below the fill and sandy clayey silt deposit or below the ground surface. The cohesive deposit was encountered between Elevation 167.5 m to 162.2 m and the thickness of this deposit ranges from about 1.6 m to 6.6 m. Borehole FC-3 was terminated within this deposit at a depth of about 5.2 m below the ground surface at about Elevation 160.7 m.

The cohesive deposit is comprised of clayey silt with trace to some sand and trace to some gravel to clayey silt with sand containing trace to some gravel. A silt layer (0.8 m thick) was encountered at a depth of 3.7 m below ground surface in FC-3. The upper 0.6 m portion of the cohesive deposit in Borehole FC-13 contains rootlets.

The SPT "N"-values measured within the clayey silt to clayey silt with sand deposit generally range from 3 blows to 22 blows per 0.3 m of penetration, suggesting that the cohesive deposit has a soft to very stiff consistency at these locations. In situ field vane tests were also performed in this layer in Borehole FC-13 and measured undrained shear strengths greater than 136 kPa and 144 kPa, and the sensitivity is calculated to be about 1. The results of the SPT "N"- values and the field vane tests suggest that the cohesive deposit generally has a stiff to very stiff consistency.

Grain size distribution tests were carried out on five (5) samples of the clayey silt to clayey silt with sand deposit and the results are shown on Figure C5 in Appendix C.

Atterberg limits tests were carried out on seven (7) samples of this deposit. The liquid limits range from about 21 per cent to 31 per cent, the plastic limits range from about 13 per cent to 18 per cent and the plasticity indices range from about 5 per cent to 14 per cent. The results of the Atterberg limits tests are shown on a plasticity chart on Figure C6 in Appendix C, and indicate that this deposit consists of clayey silt of low plasticity. The natural water content measured on nine (9) selected samples of the clayey silt to clayey silt with sand deposit ranges from about 10 per cent to 27 per cent.

4.5.6 Silty Sand to Sand and Gravel Till

A non-cohesive till deposit was encountered underlying the clayey silt with sand to clayey silt deposit in Boreholes FC-1, FC-13 / 13A, and DC-12. The top of the deposit was encountered at depths ranging from about 4.5 m to 10.4 m below ground surface (Elevation 165.9 m to 159.2 m). Boreholes FC-1 and FC-13 / 13A were terminated within the non-cohesive till deposit between about Elevation 149.3 m to 147.9 m, after penetrating between about 11 m to 11.2 m into the deposit. In Borehole DC-12, the non-cohesive till deposit was measured to be about 1.5 m thick.



The non-cohesive till deposit varies in composition from silty sand, trace to some gravel; to silt and sand, trace gravel; to gravelly sand, some silt; to silty sand and gravel; to sand and gravel, trace silt; all containing trace to some clay. In Borehole FC-1, refusal to auger or casing advancement was encountered during the drilling operations at a depth of 12.8 m below ground surface, at about Elevation 150.9 m). The borehole was cored between the depths of 12.8 m and 15.5 m (Elevations 150.9 m and 148.2 m) and was terminated within the non-cohesive till due to poor recovery. Cobbles were inferred to be present within the lower portion (below Elevation 155.0 m) of the non-cohesive till in Borehole FC-13A.

The SPT “N”-values measured within the non-cohesive till generally range from 23 blows to 68 blows per 0.3 m of penetration, indicating a dense to very dense relative density. SPT “N”-values ranging from 86 blows per 0.25 m of penetration to 65 blows per 0.03 m of penetration were recorded within the lower portion of the non-cohesive till. A SPT “N”-value of 9 blows per 0.3 m of penetration was recorded within the till deposit in Borehole FC-13A at a depth of about 13.0 m below ground surface, inferred to be a result of soil disturbance due to difficulties advancing augers/casing to this depth.

A Dynamic Cone Penetration Test (DCPT) was advanced from the bottom of the sampled Borehole FC-1 at a depth of about 15.5 m below ground surface; the DCPT was terminated on effective refusal (greater than 163 blows per 0.3 m of penetration) at a depth of about 15.9 m (Elevation 147.9 m).

Grain size distribution tests were carried out on five (5) samples of the non-cohesive till deposits and the results are shown on Figures C7 in Appendix C. The natural water content measured on eight (8) samples of the non-cohesive till deposit ranges from about 6 per cent to 13 per cent.

4.5.7 Clayey Silt (Till) (Lower Deposit)

A lower deposit of cohesive till comprised of grey clayey silt, trace sand, trace gravel was encountered underlying the silty sand till deposit in Borehole DC-12. The surface of the cohesive till deposit is at about Elevations 164.4 m, and the deposit was penetrated for a thickness of about 0.9 m.

A SPT “N”-value of 34 blows per 0.3 m of penetration was measured within the clayey silt till, suggesting that the deposit has a hard consistency.

4.5.8 Groundwater Conditions

In general, the samples taken in the boreholes were moist to wet. Details of the groundwater levels measured in the open boreholes upon completion of drilling are shown on the Record of Borehole sheets in Appendix C. Artesian groundwater conditions were observed during the drilling operations in Boreholes FC-1 and FC13 / 13A located on higher ground (outside the floodplain and low-lying valley) approximately 55 m west of Second Line West Road. The groundwater levels recorded in the open boreholes are summarized below.



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Borehole No.	Ground Surface Elevation (m)	Depth to Water Level (m)	Groundwater Elevation (m)	Date	Comments
FC-1 ¹	163.7	1.8 -1.0	161.9 164.7 ²	May 01, 2012 May 02, 2012	Inside Augers Inside Casing
FC-3	165.9	Dry	--	May 7, 2012	Inside Augers
FC-13 / 13A	167.1	2.7 2.1 -0.6 -2.1 -3.5 -4.8	164.4 165.0 167.7 ² 169.2 ² 170.6 ² 171.9 ²	September 05, 2012 (4:16 pm) September 06, 2012 (7:00 am) September 06, 2012 (12:30 pm) September 06, 2012 (12:36 pm) September 06, 2012 (12:41 pm) September 06, 2012 (1:30 pm)	Inside Augers Inside Augers Inside Casing Inside Casing Inside Casing Inside Casing
DC-12	176.3	Dry	--	September 12, 2012	Inside Augers

1. Water level not considered stabilized as water was flowing out of the top of the casing.
2. Artesian conditions

During the field investigation the ground surface in the areas adjacent to the abandoned Borehole FC-1 was monitored for the presence of groundwater seepage. Groundwater seepage was observed in the area prior to drilling activities; however, additional seepage areas were noticed shortly after the borehole was abandoned by sealing the borehole with cement grout. Although groundwater was not observed emanating from the borehole, localized areas adjacent to the borehole were observed to exhibit water seepage. Although the saturated surficial sand layers (previously referred to as “quick sand” at the MTO start-up meeting) were not encountered in the boreholes advanced near the creek, visual examination and probing of the areas adjacent to the creek using a steel rod confirmed the presence of these surficial saturated layers/zones.

Based on the groundwater levels recorded during this investigation and our observation of the presence of surficial saturated sand layers, high hydrostatic pressures are present at this site. The artesian hydrostatic head present in the cohesionless layers at this site is estimated to be at approximately Elevation 172 m. Perched water conditions will also be present and the estimated perched groundwater level is inferred to be equivalent to the Fletcher’s Creek water level. Based on the 1957 drawing, the creek high water level is at approximately Elevation 164.5 m and the normal water level is at Elevation 162.9 m.

The groundwater level observations at this site are short term and will be subject to seasonal fluctuations and precipitation events, therefore the water levels should be expected to be higher during the spring season or during any period of heavy precipitation.

4.6 Highway 401 – STA. 16+315 to STA. 16+480 (Retaining Wall 4)

The plan and profile along the centerline of the proposed widened embankment showing the borehole locations and interpreted stratigraphy between about STA. 16+315 and STA. 16+480 on the north side of Highway 401



are presented on Drawing 6. The Record of Borehole sheets (Boreholes RW-1, RW-2, DC-13 and DC-14) and the laboratory test results for this area are presented in Appendix D. The proposed retaining wall alignment within this section of the highway is located north of the existing Highway 401 WBL and east of the Second Line West bridge underpass. Second Line West crosses Highway 401 in a north-south direction via a 3-span bridge on the west end of the site. Residential properties are present at the northeast quadrant of Second Line West.

The subsurface conditions generally consist of a surficial layer of topsoil, underlain by fill in places, underlain by upper and lower units of a deposit of clayey silt till interlayered with a deposit of sand.

4.6.1 Topsoil

A 200 mm thick surficial layer of topsoil was encountered at the ground surface in all four boreholes.

4.6.2 Fill

Fill materials were encountered underlying the topsoil along the eastern extent of the site in Boreholes RW-1 and RW-2 at about Elevation 175.0 m and Elevation 175.8 m, respectively, and the fill deposit is about 0.6 m to 1.2 m thick. The fill is comprised of sandy clayey silt, trace gravel, trace organics and rootlets.

A SPT "N"-value of 19 blows per 0.3 m of penetration was measured within the sandy clayey silt fill, indicating a very stiff relative density.

A grain size distribution test was carried out on one (1) selected sample of the sandy clayey silt fill and the result is provided on Figure D1 in Appendix D. The natural water content measured on one (1) sample of the sandy clayey silt fill was about 12 per cent.

4.6.3 Sandy Clayey Silt to Clayey Silt (Till) (Upper Deposit)

A predominantly cohesive till deposit was encountered underlying the fill materials in Boreholes RW-1 and RW-2 and directly below the topsoil in Boreholes DC-13 and DC-14. The surface of the cohesive till deposit ranges from about Elevations 175.2 m to 174.4 m and the thickness of the deposit ranges from about 4.2 m to 7.1 m. The cohesive till deposit generally consists of brown to grey sandy clayey silt to clayey silt, some sand, trace to some gravel to gravelly in some places. The upper 0.6 m to 1.1 m portion of the cohesive till deposit in Boreholes RW-1, DC-13 and DC-14 contains roots.

The SPT "N"-values measured within the sandy clayey silt to clayey silt till range from 5 blows to 45 blows per 0.3 m of penetration, but are typically greater than 10 blows per 0.3 m of penetration, suggesting that the deposit has a firm to hard (but generally stiff to hard) consistency. A SPT "N"-value of 50 blows per 0.15 m of penetration was measured within the cohesive till deposit, inferred to be on cobbles.

Grain size distribution tests were carried out on five (5) samples of the cohesive till deposit and the results are provided on Figure D2 in Appendix D. Atterberg limits tests were carried out on seven (7) selected samples of this cohesive till deposit and measured liquid limits ranging from about 24 per cent to 31 per cent, plastic limits ranging from about 13 per cent to 18 per cent and plasticity indices ranging from about 9 per cent to 14 per cent.



These results, which are plotted on a plasticity chart on Figure D3 in Appendix D, indicate that the till deposit generally consists of clayey silt of low plasticity. The natural water content measured on 12 selected samples of the till ranges from about 9 per cent to 18 per cent.

4.6.4 Clayey Silt

An interlayer of grey clayey silt, trace sand was encountered underlying the clayey silt till deposit in Borehole DC-13. The thickness of the clayey silt interlayer is about 0.6 m, extending from a depth of about 7.3 m below ground surface (Elevation 167.4 m).

A SPT “N”-value of 9 blows per 0.3 m of penetration was measured across the boundary of the clayey silt interlayer and the underlying lower till unit, indicating a stiff relative density.

An Atterberg limits test was carried out on one (1) sample of the clayey silt interlayer and measured a liquid limit of about 22 per cent, a plastic limit of about 15 per cent, and a corresponding plasticity index of about 7 per cent. The result of the Atterberg limits test is shown on a plasticity chart on Figure D4 in Appendix D and indicates that this interlayer consists of clayey silt of low plasticity. The natural water content measured on one (1) selected sample of the clayey silt is about 36 per cent.

4.6.5 Silt to Sandy Silt to Silt and Sand

Underlying the upper clayey silt till deposit in Boreholes RW-1, RW-2 and DC-14 and underlying the clayey silt interlayer in Borehole DC-13, the boreholes penetrated a deposit of silt to sandy silt to silt and sand, trace to some clay and trace gravel. The deposit was encountered at depths between about 5.6 m and 7.3 m below ground surface, corresponding to between about Elevations 170.4 m and 168.1 m. The thickness of the silt to silt and sand deposit ranges from about 0.9 m to about 1.6 m but was not fully penetrated in Borehole DC-14 advanced at the eastern limit of the site which was terminated within the deposit at a depth of about 8.2 m below ground surface (Elevation 167.2 m).

The SPT “N”-values measured within the deposit range between 9 blows and 28 blows per 0.3 m of penetration, indicating a loose to compact relative density.

Grain size distribution tests were carried out on three (3) samples of the silt to silt and sand deposit and the results are shown on Figure D5 in Appendix D. The natural water content measured on four (4) samples of this deposit is about 18 per cent and 29 per cent.

4.6.6 Sandy Clayey Silt to Clayey Silt (Till) (Lower Deposit)

A lower deposit of cohesive till comprised of grey clayey silt with sand to clayey silt, trace sand, trace gravel, was encountered underlying the silt to silt and sand deposit in Boreholes RW-1, RW-2 and DC-13. The surface of the cohesive till deposit was encountered at depths between about 7.2 m and 8.8 m below ground surface corresponding to between about Elevations 168.8 m and 165.9 m, and the thickness of the deposit ranges from about 1.0 m to 2.5 m. The boreholes were terminated within this deposit at depths of about 8.2 m and 11.3 m below the ground surface, between Elevations 167.8 m and 163.4 m.



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The SPT “N”-values recorded within the lower unit of the clayey silt till deposit range between 18 blows and 66 blows per 0.3 m of penetration, suggesting that the clayey silt till has a very stiff to hard consistency.

Atterberg limits tests were carried out on two (2) samples of the till deposit and measured liquid limits of about 16 per cent and 19 per cent, plastic limits of about 10 per cent and 12 per cent and corresponding plasticity indices of about 6 per cent to 7 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure D6 in Appendix D and indicates that the till deposit consists of clayey silt of low plasticity. The natural water content measured on two (2) samples of the cohesive till deposit is about 11 per cent.

4.6.7 Groundwater Conditions

In general, the samples taken in the boreholes were moist to wet. The groundwater levels in the open boreholes were measured upon completion of drilling operations and a standpipe piezometer was installed in each of Borehole RW-1 and DC-13 to permit monitoring of the groundwater level at this site. Details of the piezometer installation and measured groundwater levels are shown on the Record of Borehole sheets in Appendix D and are summarized below.

Borehole No.	Ground Surface Elevation (m)	Depth to Water Level (m)	Groundwater Elevation (m)	Date	Comments
RW-1	175.2	7.0	168.2	July 7, 2014	Open Borehole
		3.1	172.1	August 8, 2014	Piezometer
		3.1	172.1	August 27, 2014	Piezometer
		3.1	172.1	September 15, 2014	Piezometer
RW-2	176.0	6.9	169.1	July 7, 2014	Open Borehole
DC-13	174.7	Dry	--	September 13, 2012	Piezometer
		6.2	168.5	October 09, 2012	
		6.3	168.4	November 05, 2012	
DC-14	175.4	7.0	168.4	September 12, 2012	Inside Augers

Based on the observed groundwater levels in the open boreholes and standpipe piezometers, the groundwater level fluctuates from about Elevations 168 m to 172 m. The groundwater level observations at this site are short term and will be subject to seasonal fluctuations and precipitation events, therefore the water levels should be expected to be higher during the spring season or during any period of heavy precipitation.

4.7 Highway 401 – STA. 16+345 to STA. 16+445 (Retaining Wall 5)

The plan and profile along the centerline of the proposed widened embankment showing the borehole locations and interpreted stratigraphy between about STA. 16+345 and STA. 16+445 are shown on Drawings 7. The Record of Borehole sheets (Boreholes RW-3, RW-4 and DC-7) and the laboratory test results for this area are presented in Appendix E. The proposed retaining wall alignment within this section of the highway is located south of the existing Highway 401 EBL and east of the Second Line West bridge underpass. Second Line West



crosses Highway 401 in a north-south direction via a 3-span bridge on the west end of the site. Residential properties are present at the southeast quadrant of the Second Line West.

The subsurface soils encountered along the proposed alignment in this area consist of a surficial layer of topsoil (where present) underlain either by a deposit of fill or clayey silt till or clayey silt to a silty clay deposit, underlain by a silt to silt and sand deposit. The silt to silt and sand deposit is in turn underlain by a cohesive till deposit comprised of clayey silt.

4.7.1 Topsoil

A 100 mm thick surficial layer of topsoil was encountered at the ground surface in Boreholes DC-7, RW-3 and RW-4.

4.7.2 Fill

A deposit of fill material was encountered underlying the topsoil in Boreholes RW-3 and RW-4 at about Elevation 173.6 m and Elevation 174.3 m, respectively, and the deposit is about 1.3 m thick. The fill is comprised of sandy clayey silt, trace gravel, trace rootlets and is interlayered with sand lenses.

The SPT "N"-values measured within the sandy clayey silt fill are from 12 blows and 15 blows per 0.3 m of penetration, suggesting a stiff relative density.

A grain size distribution test was carried out on one (1) selected sample of the sandy clayey silt fill and the result is provided on Figure E1 in Appendix E. An Atterberg limits test was performed on one (1) selected sample of the fill and measured a liquid limit of about 28 per cent, a plastic limit of about 15 per cent and a corresponding plasticity index of about 13 per cent. The result of the Atterberg limits test is shown on the plasticity chart on Figure E2 in Appendix E, and indicates the fill material to be a clayey silt of low plasticity. The natural water content measured on three (3) samples of the sandy clayey silt fill ranges between about 13 per cent and 18 per cent.

4.7.3 Clayey Silt (Till) (Upper Deposit)

A cohesive till deposit was encountered underlying the fill material in Boreholes RW-3 and RW-4 and below the topsoil in Borehole DC-7. The surface of the till deposit ranges from about Elevations 175.0 m to 172.3 m at the boreholes and the thickness of the till deposit ranges from about 4.2 m to 6.0 m. The till generally consists of brown to grey clayey silt with sand to sandy clayey silt, trace to some gravel

The SPT "N"-values recorded within the till deposit range from 6 blows to 29 blows per 0.3 m of penetration, suggesting a firm to very stiff consistency.

The results of grain size distribution tests carried out on two (2) samples of the sandy clayey silt till deposit are presented on Figure E3 in Appendix E. Atterberg limits tests were carried out on four (4) samples of this cohesive till deposit and measured liquid limits between about 23 per cent and 31 per cent, plastic limits between about 14 per cent to 16 per cent, and corresponding plasticity indices between about 9 per cent and 15 per cent.



The results of the Atterberg limits tests are shown on a plasticity chart on Figure E4 in Appendix E, and indicate that the till deposit is clayey silt of low plasticity. The natural water content measured on seven (7) samples of this cohesive till deposit ranges between about 9 per cent and 17 per cent.

4.7.4 Silty Clay

A cohesive deposit of grey silty clay was encountered below the upper deposit of sandy clayey silt till in Borehole DC-7. The surface of the silty clay deposit is at about Elevation 169.0 m and the thickness of this deposit is about 2.0 m. The silty clay deposit generally contains trace sand and trace gravel.

A SPT “N”-value of 7 blows per 0.3 m of penetration was measured within the silty clay deposit, suggesting a firm consistency. An in situ field vane test carried out within this deposit measured an undrained shear strength of about 170 kPa and the sensitivity is calculated to be about 1. The field vane test together with the SPT “N” values results suggest that the deposit generally has a firm to very stiff consistency.

4.7.5 Silt to Silt and Sand

Underlying the upper clayey silt till deposit in Boreholes RW-3 and RW-4 and underlying the silty clay deposit in Borehole DC-7, a deposit of silt to silt and sand was encountered at depths of between about 5.6 m and 8.1 m below the ground surface, corresponding to between Elevations 168.8 m and 167.0 m. The thickness of the silt to silt and sand deposit ranges from about 0.7 m to 1.0 m.

The SPT “N”-values recorded within this deposit range from 7 blows to 25 blows per 0.3 m of penetration, suggesting that the deposit has a loose to compact relative density.

The results of grain size distribution tests carried out on two (2) samples of the silt to silt and sand layer is shown on Figure E5 in Appendix E. The natural water content measured on seven (7) samples of this cohesive till deposit ranges between about 9 per cent and 17 per cent.

4.7.6 Clayey Silt with Sand to Clayey Silt (Till) (Lower Deposit)

A lower deposit of cohesive till comprised of clayey silt with sand to sandy clayey silt to clayey silt, trace sand, trace to some gravel, was encountered underlying the silt to silt and sand deposit in Boreholes DC-7, RW-3, and RW-4. The surface of the cohesive till deposit ranges from about Elevations 168.0 m to 166.3 m at the boreholes, and the thickness of the deposit ranged from about 1.6 m to 2.5 m. These borehole were terminated within the till deposit at depths of between about 8.2 m and 11.3 m below ground surface, corresponding to between Elevations 166.2 m and 163.8 m.

The SPT “N”-values measured within the lower clayey silt till deposit range between 13 blows and 28 blows per 0.3 m of penetration, suggesting that the cohesive till has a stiff to very stiff consistency.

A grain size distribution test was carried out on one (1) selected sample of the clayey silt till deposit and the result is provided on Figure E6 in Appendix E. An Atterberg limits test conducted on one (1) sample of the clayey silt till deposit measured a liquid limit of about 21 per cent, a plastic limit of about 14 per cent, and a



corresponding plasticity index of about 7 per cent. The result of the Atterberg limits test is shown on the plasticity chart on Figure E7 in Appendix E and indicates that the till deposit consist of clayey silt of low plasticity. The natural water content measured on three (3) samples of the cohesive till deposit is about 11 per cent.

4.7.7 Groundwater Conditions

The samples taken in the boreholes were generally moist to wet. The groundwater levels in the open boreholes were measured upon completion of drilling operations and a standpipe piezometer was installed in each of Boreholes RW-3 and DC-7 to permit monitoring of the groundwater level at this site. Details of the piezometer installation and measured groundwater levels are shown on the Record of Borehole sheets in Appendix E. 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
DC-7	175.1	Dry	--	August 30, 2012	Piezometer
		5.6	169.5	October 09, 2012	
		5.5	169.6	November 05, 2012	
RW-3	173.7	2.5	171.2	August 8, 2014	Piezometer
		2.5	171.2	August 27, 2014	Piezometer
		2.2	171.2	September 15, 2014	Piezometer
RW-4	174.4	Dry	--	July 5, 2014	Open Borehole

Based on the observed groundwater levels in the open boreholes and standpipe piezometers, the groundwater level fluctuates from about Elevations 169.5 m to 171 m. The groundwater level observations at this site are short term and have not stabilized. The groundwater levels will be subject to seasonal fluctuations and precipitation events and therefore should be expected to be higher during the spring season or during any period of heavy precipitation.

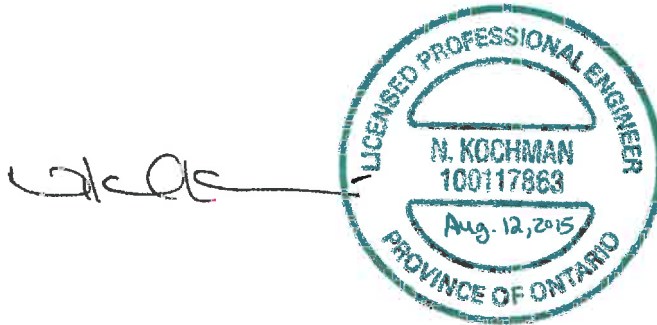


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5.0 CLOSURE

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

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PART B

FOUNDATION DESIGN REPORT
RETAINING WALLS
HIGHWAY 401 WIDENING FROM HIGHWAY 403 / 410
INTERCHANGE TO CREDIT RIVER
CITY OF MISSISSAUGA, REGION OF PEEL
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 2150-01-00



6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides recommendations on the geotechnical aspects of design for the proposed retaining walls along the proposed Highway 401 widening from Highway 403 / 410 Interchange to the Credit River, in the City of Mississauga, Region of Peel, Ontario. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during various subsurface investigations both for the retaining walls specifically and for other structures in the immediate vicinity. The discussion and recommendations presented are intended to provide the design engineers with sufficient information to carry out the design of the retaining walls.

Where comments are made on construction, they are provided to highlight those aspects that could affect the preliminary design of the project, and for which special provisions or operational constraints may be required in the Contract Documents. Those requiring information on the aspects of construction 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 and Foundation Options

It is understood, based on information provided by AECOM that the widening of Highway 401 from the Highway 403/410 Interchange to the Credit River may require the construction of retaining walls at various locations along the highway section between Credit River and the Mavis Road Interchange as detailed below. Retaining Walls 1 to 3 are discussed herein however, it is understood that Retaining Walls 4 and 5 (east of Second Line West) are no longer required.

It should be noted that the following types of walls are alternatives for consideration only and the selected wall type at each location will depend on many factors.

Retaining Wall No.	Location	Approximate Station	Approximate Length of Wall	Maximum Height of Wall Above Existing Adjacent Ground Surface	Proposed New Structure
1	North of Highway 401 between Credit River and Fletcher's Creek	15+830 to 15+945	115 m	4.8 m	Soldier Pile and Concrete Panel Wall with Tie-Backs; Alternatively RSS, Concrete Cantilever or Gabion Wall
2	North of Highway 401 west of Fletcher's Creek	15+985 to 16+142	157 m	4.05 m	Concrete Retaining Wall on Deep Foundations; Alternatively an RSS Wall



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Retaining Wall No.	Location	Approximate Station	Approximate Length of Wall	Maximum Height of Wall Above Existing Adjacent Ground Surface	Proposed New Structure
3	North of Highway 401, East of Fletcher's Creek	16+179 to 16+250	71 m	4.5 m	Concrete Retaining Wall on Deep Foundations; Alternatively RSS Wall
4	North of Highway 401, East of Second Line West	16+315 to 16+480	165 m	N/A	N/A
5	South of Highway 401, East of Second Line West	16+345 to 16+445	100 m	N/A	N/A

In consideration of the proposed type of retaining wall to accommodate the proposed highway widening and given the generally stiff to hard/compact to very dense soil conditions as encountered in the various boreholes drilled at the retaining wall sites, the following wall types and foundation options are considered appropriate for the retaining walls:

- **Soldier Pile and Concrete Panel Walls:** A soldier pile and concrete panel system is proposed for and is considered appropriate as Retaining Wall 1, as this type of wall is generally more advantageous in “top-down” construction applications as part of a cut widening as is the case at this site, rather than for an embankment widening. This type of wall system would decrease the excavation zone and potentially decrease the need for temporary excavation support along the retaining wall. For Retaining Wall 1, it is anticipated that lateral restraint may be required in the form of soil anchors. Easements may be required to accommodate the soil anchors depending on the distance from the wall to the property limits. It is considered that construction of a soldier pile and concrete panel wall would be more time-consuming than the construction of an RSS wall or a precast concrete wall due to the various steps involved (i.e., auger holes; place and concrete soldier piles; place backfill in lifts and install concrete panels; and install and pre-stress tie-backs, including testing of selected tie-backs).

Soldier Pile and Concrete Panel Walls are not considered feasible for Retaining Walls 2 and 3 (see below).

- **Concrete Retaining Wall on Deep Foundations:** A concrete wall supported on deep foundations (driven piles or caissons) is considered feasible from a geotechnical/foundations perspective for Retaining Walls 1, 2 and 3, provided sufficient space is available. A concrete retaining wall supported on deep foundations may potentially reduce the excavation zone, require less of a protection system and backfill requirements compared to a Reinforced Soil System (RSS) Wall. Temporary or permanent liners may be required to construct the deep foundations (i.e. caissons) depending on the soil conditions encountered during conditions.



- **Concrete Retaining Wall on Shallow Foundations:** A concrete retaining wall supported on shallow foundations (concrete strip footing) is geotechnically feasible for the proposed Retaining Wall 1. Concrete retaining walls supported on shallow foundations are not considered feasible at Retaining Walls 2 and 3 due to the presence of the near-surface very soft to firm clayey soil deposits in the upper 5 m to 7 m. Temporary excavations to allow for construction of the spread footings would be required and depending on the proximity of the excavation to adjacent property/structures/highway limits, temporary excavation support may be required. Concrete retaining walls supported on shallow foundations are typically less tolerable to post construction settlements.
- **Reinforced Soil System (RSS) Wall:** An RSS wall is geotechnically feasible for the proposed Retaining Wall 1. RSS walls are not considered practical at Retaining Walls 2 and 3 due to the presence of the very soft to firm clayey soil deposits in the upper 5 m to 7 m, temporary steepening of the existing Highway 401 embankment side slopes and the use of temporary protection systems along the outside of the Highway 401 lanes and north property limits (near the woodland area), and extensive dewatering efforts required to construct the reinforced soil mass. In order to install the reinforcing strips, larger amounts of excavation may be required compared to other wall types. RSS walls are typically more tolerable to post construction settlements.
- **Conventional Earth Embankment or Reinforced Earth Slope:** A conventional earth slope constructed at an inclination of 2 Horizontal to 1 Vertical (2H:1V) is geotechnically feasible for Retaining Walls 1, 2 and 3 provided sufficient space is available. If property limits do not allow for conventional construction at 2H:1V slopes, a reinforced earth slope could be considered with slopes steeper than 2H:1V (for example, 1.5H:1V or 1H:1V).

The feasibility, advantages and disadvantages for the various retaining wall options are summarized in Table 1 for Retaining Walls 1 to 3. Based on a comparison of the advantages/disadvantages between the various wall types and supporting foundation alternatives and given the subsurface conditions as encountered at the boreholes, the preferred alternative from a geotechnical perspective for the three retaining walls may be summarized as:

- Retaining Wall 1 – Soldier Pile and Concrete Panels
- Retaining Wall 2 – Concrete Wall supported on Deep Foundations
- Retaining Wall 3 – Concrete Wall supported on Deep Foundations

The following sections of this report present the results of the assessment/analyses of settlement and global stability for Retaining Walls 1, 2 and 3, comparison of the wall/foundation alternatives and provide geotechnical recommendations for the preferred options.

6.3 Embankment Settlement – Retaining Walls 2 and 3

Settlement of the high fill area between Station 16+150 and 16+270 (in the area of proposed Retaining Wall 3), and the approach embankments associated with the Fletcher's Creek bridge structure were addressed in the following reports:



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- MTO GEOCREs No. 30M12-364: Report titled "Foundation Investigation and Design Report, Deep Cut / High Fill Areas 1 to 9, Highway 401 Widening from Highway 403 / 410 Interchange to the Credit River, City of Mississauga, Region of Peel, GWP 2150-01-00," Golder Associates Ltd., dated December 20, 2013.
- MTO GEOCREs No. 30M12-356: Report titled "Foundation Investigation and Design Report, Fletcher's Creek Bridges, Highway 401 Widening from Highway 403 / 410 Interchange to the Credit River, City of Mississauga, Region of Peel, GWP 2150-01-00," Golder Associates Ltd., dated March 2013.

Settlement analyses for the soils below the widened embankments in the area of Retaining Walls 2 and 3 were carried out using both hand calculations and the commercially available computer program *Settle-3D* from Rocscience, 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). The coefficient of consolidation, c_v (cm^2/s), required in the time-rate analysis was established using the results of the laboratory tests and/or estimated from the U.S. Navy (1986) correlation with liquid limits assuming normally-consolidated soils.

Soil Deposit	Bulk Unit Weight	Estimated Deformation Properties
Existing soft to firm clayey silt fill	20 kN/m^3	$E = 15 \text{ MPa} - 40 \text{ MPa}$
Compact silty sand	20 kN/m^3	$E = 27 \text{ MPa}$
Very soft to very stiff clayey silt to clayey silt with sand	19 $\text{kN/m}^3 - 20 \text{ kN/m}^3$	$C_c = 0.138 - 0.221$ $C_r = 0.024 - 0.040$
Very Stiff to Hard clayey silt till	21 kN/m^3	$E = 40 \text{ MPa}$
Dense to very dense silty sand to sand and gravel till	21 kN/m^3	$E = 150 \text{ MPa}$

Based on the settlement analyses (details included in the reports outlined above), the following settlements are estimated:

Retaining Wall Site	Total Estimated Settlement (mm)	Immediate Settlement Component (mm)	Primary Consolidation Settlement Component (mm)
Retaining Wall 2 15+985 to 16+142	90 - 125	5 - 10	85 - 115
Retaining Wall 3 16+179 to 16+250	60 - 160	15 - 45	45 - 115

It is estimated that the time to complete ninety per cent of the primary consolidation settlement will be approximately 60 days following completion of placement of the fill for the embankment widening.

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.

6.3.1 Settlement Mitigation Options

The estimated post-construction settlement due to the embankment widening/earth retaining wall construction can be mitigated or reduced using the following mitigation options:

- sub-excavation of weak or compressible soils (i.e. soft to firm clayey silt);
- preloading of the widened embankment areas;
- preloading of the widening embankment areas in conjunction with a surcharge, to accelerate the rate of settlement over the use of preloading alone;

Other ground improvement measures such as rammed aggregate piers, deep soil mixing and dynamic compaction are not considered suitable or cost effective due to the composition of the site soils (i.e. cohesive soils), thickness of the deposit and the high groundwater conditions, and these are not addressed further in this report.

6.3.1.1 Subexcavation

Sub-excavation of the weak/soft and compressible cohesive (i.e. soft to firm clayey silt) deposits underlying the footprint of the proposed embankment widening/retaining walls is an option, but is not considered to be practical at Retaining Walls 2 and 3. The soft to firm clayey silt deposit extends to about Elevation 159.0 m at both Retaining Wall 2 and 3, and to effectively mitigate settlement, the clayey silt should be removed to Elevation 159.0 m. This would require subexcavations up to approximately 8.75 m and 9.5m below the Highway 401 grade for Retaining Walls 2 and 3, respectively, or 5 m and 7.25 m below the surrounding ground surface grade.

6.3.1.2 Preloading

Preloading may be considered for reducing post-construction settlements of the subsoils under the proposed embankment widening and new retaining wall areas. Preloading refers to the placement of fill either up to the proposed profile grade of the highway or a portion thereof (i.e. partial preload), in one or more stages, in advance of the embankment completion and final pavement construction, 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.

Because of right-of-way limits and to minimize impacts to the woodland along the north side of the highway, embankment widening in the vicinity of Fletcher's Creek is to be supported by Retaining Walls 2 and 3 rather than being constructed with standard 2H:1V side slopes. As the walls are to be constructed at the outset, either



supported on deep foundations or as a reinforced soil mass, they will either settle as the soil mass of the embankment settles or have to be designed to resist downdrag forces/load.

A Non-Standard Special Provision (NSSP) has been provided in Appendix F for inclusion in the Contract Documents to address preloading, which may be incorporated as an Operational Constraint. Monitoring of the settlement during the preloading period is recommended, as discussed further in Section 6.3.2.

The preload fill for the widening areas/retaining walls should be constructed up to the top of the highway granular sub-base level. After the preload period, it is recommended that additional sub-base granular fill be placed to achieve the final subgrade level prior to placement of the pavement structure.

6.3.1.3 *Preloading and Surcharging*

Similar to preloading, surcharging refers to the placement of additional embankment fill in advance of final pavement construction to reduce long-term, post-construction settlements. The difference between preloading and surcharging is the amount of fill placed and the time required for consolidation to be achieved. With surcharging, the fill is placed to the full embankment height (i.e. preloading), followed by an additional lift of fill (the surcharge) above that required to construct the final embankment geometry. The additional lift of fill applies greater stress to the underlying cohesive deposits and reduces the time for consolidation over that achieved by preloading alone, resulting in “overconsolidation” of the underlying compressible foundation soils relative to the final design embankment height. At the end of consolidation, the portion of the surcharge fill remaining above the required embankment height is removed. Surcharging would be most suitable if the construction schedule did not allow for sufficient time for the consolidation settlements to occur under preload fill loads alone.

As discussed further in Section 6.3.2, monitoring of the settlement during the preloading and surcharging period is recommended.

6.3.2 *Monitoring of Settlement*

It is recommended that settlement monitoring be carried out for the widened portions of the Highway 401 embankments that are preloaded/surcharged, to monitor the magnitude and rate of settlement during the preloading/surcharging period. The monitoring program should consist of the installation of a series of settlement plates (SPs) at the base of the fill platform in the embankment widening areas, which would be surveyed at regular intervals of time over the duration of the monitoring period. It is also suggested that vibrating wire piezometers (VWPs) be installed in the surficial deposits under the fill widening area to monitor for pore water pressures and excess pore pressure dissipation, although the relatively short (i.e. about 2 months) monitoring period and suggested frequency of readings (bi-weekly) may not make VWPs effective. A NSSP has been provided in Appendix F for inclusion in the Contract Documents to address the Supply and Installation of Monitoring Equipment.



6.4 Soldier Pile and Concrete Panel Wall (Retaining Wall 1)

A soldier pile and concrete panel wall could be considered, and from a foundations perspective is the preferred alternative, to support the cut section (up to 5 m high) of Retaining Wall 1. The soldier pile and concrete panel wall system is advantageous in this area, since it would minimize temporary excavation into the cut slope compared to the other wall types (i.e., for construction of spread footings for concrete cantilever or reinforced soil masses).

This wall system would consist of soldier piles socketted to sufficient depth to provide the necessary axial and passive (lateral) resistance for the maximum retained soil height. Axial geotechnical resistance recommendations for the soldier piles (i.e. steel H-piles installed in concrete caissons) are provided in Section 6.5.2 (Caisson Foundations) of this report. Additional lateral support to the soldier pile and concrete panel wall system could be provided in the form of permanent soil anchors located at strategic locations along the retaining wall.

The concrete lagging panels should be installed as the excavation for the cut progresses such that the unsupported height does not exceed 1.2 m at any time, and the space behind the lagging should be immediately packed with granular material to ensure intimate contact of the soil with the back of the wall and to aid in achieving proper drainage. If sufficient thickness of free-draining granular soil is not provided behind the concrete panels to provide adequate drainage and frost protection, consideration should be given to using a drainage sheet. An insulation layer could also be provided immediately behind the wall to provide frost protection, if required.

6.4.1 Passive Resistance for Soldier Pile Sockets

The factored passive resistance at ULS in front of the soldier piles below the base of the wall may be assessed using the equation and the design parameters provided below:

$$P_p = 1.5 K_p \gamma' z B$$

- where P_p is the factored lateral resistance at ULS (kN/m);
- K_p is the coefficient of passive earth pressure, which may be taken as 3.0;
- γ' is the effective unit weight of the soil in front of the soldier pile socket, which may be taken as 10 kN/m³ below the groundwater level;
- z is the depth from the ground surface in front of the pile to the base of the pile socket (m); and
- B is the diameter of the soldier pile socket (m).

The equation above assumes that the lateral resistance acts over a width equal to three times the socket diameter. The upper 1.2 m of soil in front of the soldier piles should be ignored in the calculation of the passive resistance, to account for frost effects. It is recommended that the soldier piles extend to a depth equal to at least the height of the wall plus the top 1.2 m of soil which is ignored in the calculation, or deeper if necessary to satisfy limiting equilibrium requirements.



6.4.2 Permanent Soil Anchors

If required, a soil anchor support system can be designed to accommodate the loads applied from lateral earth pressures and surcharge pressures from area, line or point loads and take into account any sloping ground behind the retaining wall system. For design, the soil anchors may be sized based on the following unfactored bond stresses acting between the grout and native soil.

Soil Deposit	Estimated Ultimate Load Transfer (kPa)
Stiff clayey silt till	30
Very stiff to hard clayey silt till	65

In accordance with the CHBDC, a factor of 0.4 should be applied to the unfactored bond stress value for ULS conditions. The SLS value for 25 mm of displacement will not govern and may be greater than the ULS value. For design purposes an SLS value equal to the ULS value should be used.

The sustained working load should not be greater than 60 per cent of the ultimate tensile strength of the anchor tendons or bars. Soil tie-back anchors should have their fixed length (bond zone) formed within the native very stiff to hard clayey silt till deposit, and should be installed at a downwards angle of 20 degrees or steeper. The first row of anchors should be installed not less than 1.5 m below the top of the wall face. A minimum of 4.5 m of overburden is required above the center of the fixed length (bond zone) to provide the necessary overburden pressure to develop anchor capacity in gravity-grouted anchors; to prevent grout leakage during installation of pressure grouted anchors and to prevent heaving of the ground surface for higher grout pressure operations (FHWA, 1999). The fixed length (bond zone) of the anchors should be at least 3 m (and may be up to 8 m) and should be maintained behind a line drawn upward at 45 degrees from the toe of the proposed wall. The horizontal spacing between anchors will be dependent of the spacing of the soldier piles but should be greater than four times the diameter of the anchor diameter (grouted section) or 1.2 m. The permanent soil anchors should be provided with suitable corrosion protection.

Lateral earth pressures for design are discussed in Section 6.9.

Anchor installation, grouting and testing should be carried out in accordance with OPSS 942 (*Pre-Stressed Soil and Rock Anchors*).

6.4.3 Global Stability

Slope stability analyses have been performed for the proposed Retaining Wall 1 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 retaining wall height and geometries under static conditions. This minimum factor of safety is considered appropriate for the proposed walls on this project, considering the design requirements and the available field and laboratory testing data.

The following parameters have been used in the analyses, based on field and laboratory test data as well as accepted correlations (Bowles, 1984 and Kulhawy and Mayne, 1990):



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Soil Deposit	Bulk Unit Weight (kN/m ³)	Undrained Shear Strength (kPa)	Cohesion (c') kPa	Effective Friction Angle
Stiff clayey silt	20	50	-	-
Very dense silty sand	20	-	0	32°
Firm to hard clayey silt till	21	150	3	32°
Soft to very stiff clayey silt (lower)	20	80	-	-
Compact sand and silt till	21	-	-	32°

A maximum retained wall height of 4.8 m was assumed for Retaining Wall 1. The groundwater level was inferred from the highest water levels shown on the borehole records.

The stability analysis results indicate that the proposed soldier pile and concrete panel wall at Retaining Wall 1 will have a factor of safety greater than 1.5 against global instability. An example of the static global stability results is provided on Figure 1 for the preferred alternative for Retaining Wall 1.

6.5 Concrete Retaining Wall Founded on Deep Foundations

6.5.1 Driven Steel H-Pile or Steel Pipe (Tube) Pile Foundations

6.5.1.1 Founding Elevations

Driven steel H-Pile or steel tube (pipe) pile foundations are feasible for the support of Retaining Walls 1, 2 and 3 with H-Pile foundations as the preferred alternative for Retaining Walls 2 and 3. For Retaining Wall 2, artesian groundwater conditions were encountered in Borehole FC-2, located near the east end of the proposed wall, closest to the proposed Fletcher's Creek North Bridge west abutment. For Retaining Wall 3, artesian conditions were encountered in the boreholes located to the east of Fletcher's Creek. Artesian conditions are potentially present at depth along the entire length of the proposed retaining walls.

Given the presence of artesian groundwater conditions, steel H-Piles are recommended rather than tube piles as the H-Piles will create a thinner pathway along the pile than the tube piles. Based on the results of the geotechnical investigation, the design of pile foundations for Retaining Wall 1 should be based on friction piles, whereas for Retaining Walls 2 and 3 the design should be based on friction and end bearing for piles driven to found within the "100-blow" very dense cohesionless till. For Retaining Wall 2 and for Retaining Wall 3 appropriate measures should be included in the design to mitigate the risks associated with penetration close to or into the artesian groundwater zone, such as the inclusion of a sand drainage blanket constructed immediately below the retaining wall structure (i.e., below the pile cap/footing) to dissipate groundwater seepage that may propagate along the piles as discussed below. The drainage blanket should extend at least 0.5 m beyond the bottom of the pile cap and should be placed beneath the entire length of the retaining wall(s).

For steel HP 310 x 110 piles or steel tube piles (324 mm diameter x 6.4 mm thickness) driven to found within the lower very stiff to hard clayey silt till at Retaining Wall 1 and within the "100-blow" very dense sandy silt and gravel to sand and gravel till at Retaining Walls 2 and 3, the design pile tip elevations provided below may be used for design of the pile foundations.



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Retaining Wall Site	Founding Stratum	Estimated Design Pile Tip Highest Elevation / Minimum Pile Length
Retaining Wall 1 15+830 to 15+945 (Ground Surface Elev. 167.0 m at Road Level – assumed)	Compact silt and sand till / Very stiff clayey silt till	158.5 m / 8.5 m
Retaining Wall 2 15+985 to 16+142 (Base of Pile Cap Elev. 162.5 m assumed)	“100-blow” very dense cohesionless till	153.5 m / 9.0 m
Retaining Wall 3 16+179 to 16+250 (Base of Pile Cap Elev. 162.5 m assumed)	“100-blow” very dense cohesionless till	153.5 m / 9.0 m

Note: Does not include the length of pile extruding above roadway level for Retaining Wall 1, estimated to be about 4.6 m

Alternatively, shorter pile foundations could be considered at Retaining Walls 2 and 3 by terminating the steel HP 310 x 110 piles, or steel tube piles (324 mm diameter x 6.4 mm thickness), at relatively higher elevation within the dense to very dense cohesionless till as presented below. The higher pile tip elevations would reduce the risks associated with artesian groundwater control (i.e. migration of fine soil particles along the pile shafts), as well as reduce the risk associated with encountering obstructions (i.e. cobbles and boulders) within the overburden soils that were typically encountered at or below these higher founding elevations. However, higher founding elevations will result in relatively low geotechnical resistance values.

Retaining Wall Site	Founding Stratum	Estimated Design Pile Tip Elevation / Pile Length
Retaining Wall 2 15+985 to 16+142 (Base of Pile Cap Elev. 162.5 m assumed)	Dense to very dense cohesionless till	157.0 m / 5.5 m
Retaining Wall 3 16+179 to 16+250 (Base of Pile Cap Elev. 162.5 m assumed)	Dense to very dense cohesionless till	157.0 m / 5.5 m

For the installation of piles, consideration must be given to the presence of cobbles and boulders within the cohesionless till deposits, as encountered in the boreholes (below about Elevation 157 m) advanced at the east end of Retaining Wall 2. Auger grinding observed at variable depths during the borehole investigation suggests obstructions may be encountered above the proposed founding tip elevations. In this regard, steel H-piles are preferred over steel tube piles given that steel tubes are considered to pose a higher risk of “hanging up” or being deflected from their vertical or battered orientation during installation, due to their larger end area. The piles should be reinforced at the tip for protection during driving to reduce the potential for damage to the piles in the event that cobbles/boulders and/or very dense layers are encountered within the till deposits. The steel H-



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piles should be reinforced with flange plates as per OPSD 3000.100 (Foundation Piles Steel H-Pile Driving Shoe) or driving shoes such as Titus Standard "H" Bearing Pile Point design for protection during driving. Similarly, if steel tube piles are being considered, driving shoes should be in accordance with OPSD 3001.100 Type II (Steel Tube Pile Driving Shoe). The requirement for driving shoes should be included in the Contract Drawings.

The pile caps for the new retaining walls should be provided with a minimum of 1.2 m of soil cover to provide adequate protection against frost penetration (as per OPSD 3090.101 – *Foundation Frost Depths for Southern Ontario*).

Given the presence of high hydrostatic pressures in the very dense cohesionless till deposit and the groundwater seepage issues observed during the field investigation at the site, suitable construction techniques will be required to mitigate and/or control the possible loss of fine size carried by upward flow of water along the pile shaft, especially at the proposed Retaining Wall 3. It is recommended that a sand drainage/filter, in combination with a geotextile and adjacent drainage ditches, be placed beneath the pile caps to minimize the migration of fines that may be transported along the piles during and after construction and to control any water seepage. The drainage/filter layer should consist of concrete fine aggregate, meeting the gradation requirements of OPSS.PROV 1002 (Aggregates Concrete) and should be included in the Design Drawings. Further details on the use of sand drainage/filter blankets are provided in Section 6.11.4 Groundwater and Surface Water Control.

6.5.1.2 Geotechnical Axial Resistances

For steel HP 310 x 110 piles (or steel tube piles) (324 mm diameter x 6.4 mm thickness) driven to the design pile tip elevations provided in Section 6.5.1.1, the factored geotechnical axial resistance at ULS and the geotechnical reaction at SLS for 25 mm of settlement are given below. Given the high artesian pressures within the cohesionless till soils near the estimated pile tip elevations, the resistances are lower than would typically be given for driven piles in similar soil conditions with similar pile lengths.

Retaining Wall Site	Founding Stratum	Factored Geotechnical Resistance at ULS	Geotechnical Reaction at SLS (for 25 mm of Settlement)	Downdrag Load
Retaining Wall 1 15+830 to 15+945	Compact silt and sand till / Very stiff clayey silt till	300 kN	250 kN	N/A
Retaining Wall 2 15+985 to 16+142	"100-blow" very dense cohesionless till	800 kN	675 kN	130 kN
	Dense to very dense cohesionless till	500 kN	400 kN	130 kN
Retaining Wall 3 16+179 to 16+250	"100-blow" very dense cohesionless till	800 kN	675 kN	130 kN
	Dense to very dense cohesionless till	500 kN	400 kN	130 kN



The following note, or similar notation, 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 (refer to the Structural Manual Section 3.3.3 (MTO, 2008)) for the Retaining Walls:

For Retaining Wall 1:

"Piles to be driven in accordance with Standard SS-103-11 using an ultimate geotechnical resistance of 600 kN per pile but must be driven below Elevation 159.0 m and not below Elevation 158.0 m without approval of the Engineer."

Similarly, for Retaining Walls 2 and 3 driven to the lower "100-blow" very dense cohesionless till:

"Piles to be driven in accordance with Standard SS-103-11 using an ultimate geotechnical resistance of 1,600 kN per pile but must be driven below Elevation 155.5 m and not below Elevation 153.5 m without approval of the Engineer."

Alternatively, for Retaining Walls 2 and 3 driven to the higher dense to very dense cohesionless till:

"Piles to be driven in accordance with Standard SS-103-11 using an ultimate geotechnical resistance of 1,000 kN per pile but must be driven below Elevation 158.0 m and not below Elevation 157.0 m without approval of the Engineer."

For both options at Retaining Walls 2 and 3, similar axial resistances and drawing notes may be used in the design for closed-end, concrete filled 324 mm (12 ¾ in.) diameter steel tube piles having a minimum wall thickness of 6.4 mm (¼ in.).

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 Standard Drawing SS103-11) during the final stages of driving to achieve an ultimate capacity, as indicated in the Contract Drawing Notes above.

6.5.1.3 Resistance to Lateral Loads

Resistance to lateral loading can be derived using vertical piles, with enhanced support offered by battered piles, if required. If vertical piles are used, the resistance to lateral loading will have to be derived solely from the soil in front of the piles, whereas battered piles derive lateral resistance from the soil in front of the piles as well as the horizontal component of the axial load present in the inclined pile.

The resistance to lateral loading in front of the pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction (k_h in kPa/m) is determined based on the equations given below (CFEM 1992 as noted in Section 6.8.7.1 of the *Commentary to the CHBDC, 2006*):

For cohesionless soils:

$$k_h = \frac{n_h z}{B} \quad \text{Where} \quad \begin{array}{l} k_h \text{ is the coefficient of horizontal subgrade reaction (kPa/m);} \\ n_h \text{ is the constant of subgrade reaction (kPa/m);} \end{array}$$



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z is the depth (m); and

B is the pile diameter or width (m).

For cohesive soils:

$$k_h = \frac{67s_u}{B} \quad \text{Where}$$

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 or 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 1.2 m below the lowest surrounding grade in front of the piles).

Soil Unit	n_h (kPa/m)	s_u (kPa)
Soft to hard clayey silt to clayey silt with sand fill	-	40
Compact silty sand	4,400	-
Firm to stiff clayey silt to silty clay	-	40
Very soft to firm clayey silt to silty clay	-	20
Stiff to very stiff clayey silt to silty clay till	-	150
Compact to very dense sandy silt to sand and gravel till	11,000	-

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 are 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 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-7.02, 1986) as follows:

Pile Spacing in direction of Loading (d = Pile Diameter)	Subgrade Reaction Reduction Factor (R)
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 above.

6.5.2 Caisson Foundations

6.5.2.1 Caisson Founding Elevations

Caisson foundations are feasible for the support of Retaining Wall 1, 2 and 3. For Retaining Wall 2, artesian groundwater conditions were encountered in Borehole FC-2, located on the east side of the proposed wall, closest to the Fletcher's Creek abutment. For Retaining Wall 3, artesian conditions were encountered along the length of the proposed retaining wall.

If caisson foundations are adopted at Retaining Walls 1, 2 and 3, a temporary or permanent liner would be required to support the soils during construction, to minimize disturbance and loss of ground in the water-bearing cohesionless soils present at the site. Specialized construction techniques would be required during advancement of the caisson in order to maintain a sufficient head of water and / or use of drilling fluid within the liner to prevent basal heave and disturbance of the water-bearing cohesionless till, specifically at Retaining Wall 3.

Given the artesian conditions and requirement to balance the hydrostatic head, concrete must be placed using tremie techniques. After initial placement of concrete at the bottom of the caisson, the tremie discharge point should be maintained below the surface of the wet concrete during placement. It is recommended that a Non-Standard Special Provision (NSSP) be included in the Contract Documents to address the need for control of the ground and groundwater during caisson construction as discussed further under Construction Considerations in Section 6.11.

The performance of caissons will depend upon the final cleaning and verification of the subgrade at the base of the caissons. Each caisson excavation should be carefully cleaned to remove all loosened debris to ensure that the concrete is in intimate contact with the competent bearing stratum. Difficulties verifying the base of the caisson are anticipated and base verification will likely not be feasible given the high artesian groundwater conditions.

The caisson caps should be provided with a minimum of 1.2 m of soil cover to provide adequate protection against frost penetration (per OPSD 3090.101 – *Foundation, Frost Penetration Depths for Southern Ontario*) unless the caissons are continuous to form the retaining walls, in which case caisson caps are not required.

For Retaining Wall 1 caissons may be founded within the lower very stiff clayey silt till; for Retaining Walls 2 and 3 caissons may be founded within the cohesionless till deposit and socketted at least 1.5 m into the very dense silty sand and gravel till to sand and gravel till at the following design base elevations.

Retaining Wall Site	Founding Stratum	Estimated Founding Elevation / Caisson Length
Retaining Wall 1 15+830 to 15+945	Compaction silty and sand till / Very stiff clayey silt till	158.5 m / 8.5 m



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Retaining Wall Site	Founding Stratum	Estimated Founding Elevation / Caisson Length
Retaining Wall 2 15+985 to 16+142	"100-blow" very dense cohesionless till	154.5 m / 8.0 m
Retaining Wall 3 16+179 to 16+250	100-blow" very dense cohesionless till	154.5 m / 8.0 m

Similar to the driven pile foundation options, shorter caissons founded at a higher founding elevation within the dense to very dense till deposit may be considered at Retaining Walls 2 and 3 to reduce the risks associated with the artesian groundwater conditions and potential difficulties augering through obstructions (i.e. cobbles and boulders) as shown below.

Retaining Wall Site	Founding Stratum	Estimated Founding Elevation / Caisson Length
Retaining Wall 2 15+985 to 16+142	Dense to very dense cohesionless till	157.0 m / 5.5 m
Retaining Wall 3 16+179 to 16+250	Dense to very dense cohesionless till	157.0 m / 5.5 m

6.5.2.2 Geotechnical Resistance/Reaction

The recommended design values for factored geotechnical axial resistance at ULS and geotechnical reaction at SLS (for 25 mm of settlement) for caissons founded at the elevations given in Section 6.5.2.1 are provided below.

Retaining Wall Site	Founding Stratum	Caisson Diameter	Factored Geotechnical Resistance at ULS	Geotechnical Reaction at SLS (for 25 mm of Settlement)
Retaining Wall 1 15+830 to 15+945	Compact silt and sand till / Very stiff clayey silt till	0.9 m	1,000 kN	800 kN
		1.2 m	1,800 kN	1,500 kN
		1.5 m	2,800 kN	2,300 kN
Retaining Wall 2 15+985 to 16+142	"100-blow" very dense cohesionless till	0.9 m	1,100 kN	950 kN
		1.2 m	2,000 kN	1,650 kN



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Retaining Wall Site	Founding Stratum	Caisson Diameter	Factored Geotechnical Resistance at ULS	Geotechnical Reaction at SLS (for 25 mm of Settlement)
	Dense to very dense cohesionless till	1.5 m	3,100 kN	2,650 kN
		0.9 m	700 kN	550 kN
		1.2 m	1,200 kN	1,000 kN
		1.5 m	2,000 kN	1,600 kN
Retaining Wall 3 16+179 to 16+250	"100-blow" very dense cohesionless till	0.9 m	1,100 kN	950 kN
		1.2 m	2,000 kN	1,650 kN
		1.5 m	3,100 kN	2,650 kN
	Dense to very dense cohesionless till	0.9 m	700 kN	550 kN
		1.2 m	1,200 kN	1,000 kN
		1.5 m	2,000 kN	1,600 kN

6.5.2.3 Resistance to Lateral Loads

Resistance to lateral loading will be derived from the soil in front of the caissons. The resistance to lateral loading in front of the caisson may be calculated using subgrade reaction theory and the equations and soil parameters provided in Section 6.5.1.3 may be used for design.

6.5.3 Global Stability

Slope stability analyses have been performed for the proposed Retaining Walls 1, 2 and 3 for the alternative of cast-in-place concrete walls supported on deep foundations, 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 retaining wall heights and geometries under static conditions. This minimum factor of safety is considered appropriate for the proposed walls on this project, considering the design requirements and the available field and laboratory testing data.

The following parameters have been used in the analyses, based on field and laboratory test data as well as accepted correlations (Bowles, 1984 and Kulhawy and Mayne, 1990):

Soil Deposit	Bulk Unit Weight (kN/m ³)	Undrained Shear Strength (kPa)	Cohesion (c') kPa	Effective Friction Angle
New embankment fill	21	-	0	32°
Excising embankment fill	20	40	-	-



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Soil Deposit	Bulk Unit Weight (kN/m ³)	Undrained Shear Strength (kPa)	Cohesion (c') kPa	Effective Friction Angle
Very soft to firm clayey silt to silty clay	19	20	-	-
Stiff to very stiff clayey silt to silty clay	20	50 – 80	-	-
Compact to very dense silty sand, sand and silt to silt	20	-	0	32°
Stiff to very stiff clayey silt to silty clay till	21	150	3	32°
Compact to very dense silty and sand/sandy silt to sand and gravel till	21	-	-	32°

The maximum height of retained soil assumed for Retaining Walls 1, 2 and 3 is 4.8 m, 4.05 m and 4.5 m, respectively. Groundwater levels were inferred from the highest water levels shown on the borehole records.

The stability analysis results indicate that the proposed concrete retaining wall founded on deep foundations at Retaining Walls 1, 2 and 3 will have a factor of safety greater than 1.5 against global instability. An example of the static global stability result for Retaining Walls 2 and 3 is provided on Figures 2 and 3, respectively.

6.6 Concrete Cantilever Wall Founded on Shallow Foundations

6.6.1 Founding Elevations

Strip footing (shallow) foundations are feasible for the support of Retaining Walls 1 to 3. For Retaining Wall 1, strip footings founded on the stiff to very stiff lower clayey silt till can be considered. For Retaining Walls 2 and 3, strip footings founded on the generally dense to very dense sandy silt to sand and gravel till, or dense sandy silt to silty sand till or very stiff clayey silt to silty clay till deposits can be considered. As a result of having to construct the foundations below the topsoil, fill, loose surficial soils and soft to firm clayey silt to clayey silt with sand deposit, sub-excavation depths between about 5 m and 7 m will be required at Retaining Walls 2 and 3 which is not considered feasible.

All footings should be founded at a minimum depth of 1.2 m below the adjacent final grade to provide adequate protection against frost penetration, in accordance with Ontario Provincial Standard Drawing (OPSD) 3090.101 (*Foundation, Frost Penetration Depths for Southern Ontario*).

The following founding elevations for the retaining walls are recommended for strip footings founded on competent native soil or for engineered fill placed to support the footings at a higher founding elevation.

Retaining Wall Site	Founding Stratum	Highest Founding Elevation
Retaining Wall 1 15+830 to 15+945	Very stiff to hard clayey silt till	164.0 m



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Retaining Wall Site	Founding Stratum	Highest Founding Elevation
Retaining Wall 2 15+985 to 16+142	Dense to very dense sandy silt and gravel to sand and gravel till	159.0 m
Retaining Wall 3 16+179 to 16+250	Very dense sandy silt to gravelly silt and sand till	159.0 m

Considering the recommended founding depth for Retaining Walls 2 and 3, the subexcavated area (for depths between about 5 m and 7 m) could be replaced with properly placed and compacted Granular 'A' or Granular 'B' Type II (OPSS PROV 1010) to found the footings at a higher elevation (a minimum of 1.2 m depth below the lowest surrounding grade to provide adequate protection against frost penetration). If replacement of unsuitable materials with engineered fill is being considered, the area to be subexcavated should be defined by a line extending from the top of the engineering fill pad outward and downward at 1H:1V. The top of the granular engineered fill should extend at least 1 m beyond the plan limits of the footing, and constructed in accordance with OPSS.PROV 501 (Compacting). Temporary shoring would be required and is discussed in Section 6.11.

In addition, given the high groundwater pressures encountered at depth in the cohesionless till deposit and the generally high water level, as well as the potential for running/flowing of the silty sand to silt and sand materials into the excavation(s), temporary dewatering would be required in advance of excavation and during backfilling and construction of the retaining wall foundations at the retaining wall locations.

6.6.2 Geotechnical Resistance/Reaction

Strip footings constructed about 3 m wide on the properly prepared subgrade, at or below the design elevations given in the Section 6.6.1, should be designed based on the factored geotechnical resistances at ULS and geotechnical reaction at SLS (for 25 mm of settlement) given below.

Retaining Wall Site	Factored Geotechnical Resistance at ULS	Geotechnical Reaction at SLS (for 25 mm of Settlement)
Retaining Wall 1 15+830 to 15+945	500 kPa	325 kPa
Retaining Wall 2 15+985 to 16+142	850 kPa	575 kPa
Retaining Wall 3 16+179 to 16+250	750 kPa	500 kPa

If the strip footings are founded on compacted Granular 'A' or Granular 'B' Type II at higher elevations, a factored geotechnical resistance at ULS of 750 kPa and a geotechnical reaction at SLS (for 25 mm of



settlement) of 350 kPa could be employed for the design of the retaining wall foundations, assuming the granular pad is at least 2 m thick.

The ULS resistance and settlement are dependent on the footing size (assumed to be at least 3 m wide), configuration and applied loads; the geotechnical resistances should, therefore, be reviewed if the selected footing width or founding elevation differs from those given above.

The geotechnical resistances provided above are given for loads applied perpendicular to the surface of the footing. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Sections 6.7.4 and C6.7.4 in the *Canadian Highway Bridge Design Code (CHBDC)* and its *Commentary*, using the curves for cohesive soils and non-cohesive soil.

The base of each footing excavation should be cleaned of softened / loosened soil and it is recommended that the founding level for the footings be inspected by a Quality Verification Engineer (QVE) following excavation, in accordance with OPSS 902 (*Excavating and Backfilling Structures*) to verify that all existing fill and other unsuitable material have been removed. If the concrete for the footings cannot be poured immediately after excavation and inspection, it is recommended that a concrete working slab (100 mm thickness of 20 MPa compressive strength concrete) be placed on the subgrade to protect the integrity of the bearing stratum. This requirement can either be added as a note on the Contract Drawings or included as a Non-Standard Special Provision (NSSP) in the Contract Documents. A sample NSSP is included for this item in Appendix F.

6.6.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding between the concrete footings and the subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC*. For cast-in-place concrete footings constructed on the dense to very dense cohesionless till or on the Granular 'A' or Granular 'B' Type II engineered fill, the coefficient of friction, $\tan \delta$, can be taken as 0.5; if constructed on the stiff to very stiff clayey silt till, the coefficient of friction, $\tan \delta$, can be taken as 0.45. These values are unfactored.

6.6.4 Global Stability

Slope stability analyses have been performed for the proposed concrete Retaining Walls 1 to 3 supported on shallow foundations, 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 retaining wall heights and geometries under static conditions. This minimum factor of safety is considered appropriate for the proposed walls on this project, considering the design requirements and the available field and laboratory testing data.

The static global stability analyses for Retaining Walls 1, 2 and 3 were completed using the parameters outlined in Section 6.4.3. Retained soil heights of 4.8 m, 4.05 m and 4.5 m were assumed for Retaining Walls 1, 2 and 3, respectively. Groundwater levels were inferred from the highest water levels shown on the borehole records.

The stability analysis results indicate that the proposed concrete retaining wall founded on shallow foundations at Retaining Walls 1 to 3 will have a factor of safety greater than 1.5 against global instability.



6.7 Retaining Soil System (RSS) Walls

6.7.1 Founding Elevations

A typical RSS wall has a front facing supported on a strip footing (placed on a levelling pad) founded 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. A compacted granular pad consisting of OPSS.PROV 1010 Granular A material should be used for levelling purposes and should extend at least 1.0 m beyond the outside edge of both sides of the facing footing, then outward/downward at a 1H:1V.

In the area of Retaining Walls 2 and 3 a soft to firm clayey silt / silty clay deposit is present to approximately Elevation 159.0 m. Due to the presence of these soft to firm cohesive soil, it is estimated that between approximately 125 mm and 160 mm of settlement will occur as a result of the new embankment loadings if these deposits are not removed and replaced with competent fill. Additionally, the geotechnical resistances of the subgrade will be greatly reduced if these materials are not removed and replaced. Therefore, to mitigate the occurrence of future settlement and avoid the need for preloading / surcharging these embankments through these two areas, it is recommended that this material be removed, requiring subexcavations between approximately 5 m and 7 m deep, extending below the groundwater level. The subexcavated area should be backfilled with compacted OPSS.PROV 1010 Granular B Type II material. Given the constraints on site, subexcavation to these depths is not considered practical.

Based on the subsurface information available, the following maximum (highest) founding elevations are recommended for design of RSS walls for Retaining Walls 1, 2 and 3.

Retaining Wall Site	Founding Stratum	Maximum Founding Elevation
Retaining Wall 1 15+830 to 15+945	Very stiff to hard clayey silt till	164.0 m
Retaining Wall 2 15+985 to 16+142	Dense to very dense sandy silt and gravel to sand and gravel till	159.0 m
Retaining Wall 3 16+179 to 16+250	Very dense sandy silt to gravelly silt and sand till	159.0 m

The footing subgrade should be inspected by a Quality Verification Engineer following excavation, in accordance with OPSS 902 (*Excavating and Backfilling Structures*) to check that all existing fill or other unsuitable material have been removed.

The RSS soil mass and facing footing may be supported on the Granular 'A' or Granular 'B' Type II engineered fill pad founded on competent native soils. The facing footing should be placed on a minimum 500 mm thick pad of compacted OPSS.PROV 1010 Granular 'A' extending at least 1.0 m beyond the outside edge of the facing footing, then downward at 1H:1V as per Figure 5.2 in MTO's RSS Design Guidelines (September 2008).



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The RSS retaining walls are to be designed for high performance and appearance in accordance with MTO Special Provision (SP) 599S22 (Retained Soil System).

The reinforced soil mass should be keyed into the existing embankment fill side-slope, if applicable, by benching into the embankment fill, as per OPSD 208.010 (Benching of Earth Slopes).

6.7.2 Geotechnical Resistance and Settlement

For the reinforced soil mass founded at the depths / levels discussed in Section 6.7.1, 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 Site	Subexcavation Elevation	Maximum Wall Height Above Finished Grade	Estimated Minimum Reinforced Width	Factored Geotechnical Resistance at ULS	Geotechnical Reaction at SLS (for 25 mm of Settlement)
Retaining Wall 1 15+830 to 15+945	164.0 m	4.45 m	3.6 m	525 kPa	300 kPa
Retaining Wall 2 15+985 to 16+142	159.0 m	3.75 m	3.0 m	850 kPa	575 kPa
	2 m Granular Pad to Elev. 161.0 m	3.75 m	3.0 m	900 kPa	600 kPa
Retaining Wall 3 16+179 to 16+250	159.0 m	4.5 m	3.6 m	800 kPa	525 kPa
	2 m Granular Pad to Elev. 161.0 m	3.75 m	3.0 m	900 kPa	600 kPa

Based on discussions with designers and suppliers of RSS walls, it is our understanding that the proprietary RSS systems are flexible and can tolerate on the order of 1 m of differential settlement over a 100 m length. The total settlement of the RSS walls will be governed by the settlement that will occur under the placement of additional fill for the widening of the Highway 401 embankment.

If the soft to firm clayey silt at Retaining Walls 2 and 3 is not removed it is estimated that the period of time to complete ninety per cent of the primary consolidation settlement is approximately 60 days following completion of placement of the fill for the embankment widening/retaining wall. Section 6.3 outlines settlement mitigations options for Retaining Walls 2 and 3. As discussed in Section 6.3.2, settlement monitoring is recommended at these sites.

As an alternative to preloading or surcharging of the retaining wall area to permit use of conventional RSS systems, consideration could be given to incorporating EPS “sandwiched” between layers of granular fill to reduce the overall loading on the subsoil by the embankment widening. Such a system would involve a proprietary design by the RSS designer/supplier, but this could be pursued if there is insufficient time in the schedule or insufficient space at the site to allow for preloading.



As an alternative to preloading the retaining wall area to permit use of conventional RSS systems, consideration could be given to the use of cellular concrete in place of the granular backfill that is typically used in the reinforced soil zone. Cellular concrete is a lightweight material that is typically used in applications where its reduced weight improves settlement performance and/or minimizes impacts on existing adjacent structures, and where it can accelerate the construction schedule by reducing preloading time. For highway applications, a higher density (stronger) foaming agent is typically recommended, with placement of the cellular concrete in approximately 0.6 m thick lifts and with the anchor points for the permanent wall facing panels embedded into the cellular concrete during placement of the lifts. Although an initial set is usually attained within 90 minutes, a subsequent lift cannot be placed until a further curing period of about six to ten hours has elapsed. As cellular concrete is also a proprietary construction material, the supplier should be consulted to the design requirement for density as well as placement application of the material.

It is recommended that vertical slip joints be incorporated into the RSS walls at appropriate intervals to accommodate the differential settlements without adversely affecting the aesthetic appearance of the RSS facing panels.

6.7.3 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, $\tan \delta'$, between the compacted granular fills of the RSS wall and the properly prepared native subgrade may be taken as 0.6. Similarly, the coefficient of friction, $\tan \delta'$, between cast-in-place concrete facing footing and underlying granular pad may be taken as 0.55. These represent unfactored values.

6.7.4 Global Stability

Slope stability analyses have been performed for the proposed Retained Soil System (RSS) walls along Retaining Walls 1, 2 and 3 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 retaining wall heights and geometries under static conditions. This minimum factor of safety is considered appropriate for the proposed walls on this project, considering the design requirements and the available field and laboratory testing data.

The static global stability analyses for Retaining Walls 1, 2 and 3 were completed using the parameters outlined in Section 6.4.3, and assume that all existing topsoil and fill soils are completely removed prior to constructing the RSS walls. Retained soil thicknesses of 4.8 m, 4.05 m and 4.5 m were assumed for Retaining Walls 1, 2 and 3, respectively. Groundwater levels were inferred from the highest water levels shown on the borehole records.

The stability analysis results indicate that the proposed RSS walls founded on a properly prepared subgrade for Retaining Walls 1, 2 and 3 will have a factor of safety greater than 1.5 against global instability. It should be noted that the internal stability of a reinforced earth structure is to be designed and assessed by the proprietary product designer/supplier.



6.8 Earth Embankment

Generally, property restrictions limit the use of conventional earth embankments cut or fill sections sloped at 2H:1V at the retaining wall sites. The conventional slope could be constructed at 2H:1V if space permits or a steeper reinforced earth embankment could be considered at these locations given the relatively competent foundation soils.

In accordance with MTO's standard practice for earth slopes (cut or fill section), vegetation cover should be established on all fill slope faces to protect against surficial erosion, as per OPSS.PROV 804 (*Seeding and Cover*). Consideration should be given to including an interceptor ditch along the crest of the slope to minimize surface water flow over the crest and slope face, and to reduce surface erosion potential.

6.8.1 Global Stability

Slope stability analyses have been performed for the proposed conventional earth embankment option in lieu of a retaining wall at the proposed Retaining Walls 1, 2, and 3 locations 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 embankment heights and geometries under static conditions. This minimum factor of safety is considered appropriate for design considering the available field and laboratory testing data.

The static global stability analyses for Retaining Walls 1, 2 and 3 were completed using the parameters outlined in Section 6.4.3 for maximum vertical heights of 4.8 m, 4.05 m and 4.5 m for the areas of Retaining Walls 1, 2 and 3. Groundwater levels were inferred from the highest water levels shown on the borehole records.

The analysis results indicate that the roadway embankment constructed at 2H:1V inclination will have a factor of safety greater than 1.5 against side slope instability.

If the distance from the existing top of embankment to the MTO property limits is not sufficient to construct side-slopes at 2H:1V, a reinforced earth slope at a steeper angle (e.g. 1.5H:1V) could be designed based on the subgrade / founding soils at the Retaining Wall locations. The reinforced earth slope would need to be designed by a proprietary product designer and the global stability would need to be checked / confirmed during design.

6.9 Lateral Earth Pressures for Design

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

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

- Select, free-draining granular fill meeting the specifications for OPSS.PROV 1010 (*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 walls. Longitudinal drains and weep holes should be installed to provide positive



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drainage of the granular backfill. In the case of soldier pile and lagging wall for the cut at Retaining Wall 1, the drainage medium could consist of a drainage fabric / net constructed at the ground surface (roadway level) to the longitudinal granular drains and weep holes.

- 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 (including type of equipment, target densities, etc.) should be used in accordance with OPSS.PROV 501). Other surcharge loadings should be accounted for in the design as required.
- For restrained structures, the granular fill should be placed in a zone with the width equal to at least 1.2 m behind the back of the walls (in accordance with Figure C6.20(a) of the *Commentary* to the *CHBDC*). For unrestrained structures, granular fill should be placed within the wedge shaped zone defined by a line drawn at no steeper than 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the base of the footing (in accordance with Figure C6.20(b) of the *Commentary* to the *CHBDC*).
- For restrained structures, the pressures are based on the existing embankment fill materials and the existing overburden soils and the following parameters (unfactored) may be used:

	Earth Fill	Native Till
Soil unit weight:	20 kN/m ³	21 kN/m ³
Coefficients of static lateral earth pressure:		
Active, K_a	0.33	0.31
At rest, K_o	0.50	0.47

- For Retaining Wall 1 where an upward 2H:1V slope exists above the top of the rear of the wall, the active and at rest coefficients of static lateral earth pressure $K_a = 0.48$ and $K_o = 0.7$ may be assumed.
- For unrestrained structures 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

If the wall support allows lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure and if the wall support does not allow lateral yielding, at-rest earth pressures should be assumed for the geotechnical design. The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.9.1 and Table C6.6 of the *Commentary* to the *CHBDC*.



6.10 Seismic Site Coefficient

For seismic design purposes, the Site Coefficient (S) for this site may be taken as 1.2, consistent with Soil Profile Type II. The soil profile is based on the guidelines in Section 4.4.6 and Table 4.4 of the *CHBDC (2006)* and local experience.

6.10.1 Seismic Analysis Coefficient

The potential for seismic (earthquake) loading may also need to be considered for the design of the retaining wall stems/reinforced soil mass and for the assessment of liquefaction potential of foundation soils in accordance with Section 4.6 of the *CHBDC*, as significant seismic loading will result in increased lateral earth pressures acting on the wall stem and reinforced soil mass systems.

According to Table A3.1.1 of the *CHBDC*, the overall site of Retaining Walls 1, 2 and 3 is located in Seismic Zone 1. The site-specific zonal acceleration ratio for the City of Mississauga is 0.05. Based on experience, for the subsurface conditions at this site, a 20 per cent amplification of the ground motion may occur (i.e. Site Coefficient, $S=1.2$ for Soil Profile II from Table 4.4 of *CHBDC*), resulting in an increase in the peak horizontal ground acceleration (PGA) from 0.05 g to 0.06 g at the ground surface. Based on Section 4.4.4 of the *CHBDC*, this bridge structure is assigned Seismic Performance Zone 1. Given this, and in accordance with Section 4.4.5.1 of the *CHBDC*, no seismic analysis is required for structures located in Seismic Performance Zone 1.

6.11 Construction Considerations

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

6.11.1 Embankment Construction

It is recommended that all topsoil and organic material be stripped for the proposed embankment footprint at all retaining wall/embankment widening locations. After stripping of organics, the exposed subgrade should be proof-rolled to identify any loose/softened areas requiring sub-excavation or additional compaction prior to fill placement.

Embankment fill should be placed and compacted in accordance with OPSS.PROV 206 (*Earth Excavation, Grading*) and OPSS.PROV 501 (*Compacting*). New embankment fill placed against existing embankment slopes or on sloping ground should be benched into the existing slope in accordance with OPSD208.010 (Benching of Earth Slope).

6.11.2 Excavation and Temporary Roadway Protection

The foundation excavations for strip footings and RSS walls at Retaining Walls 1, 2 and 3 will extend through topsoil, fills, near surface deposit of clayey silt to silty clay, silty sand and cohesive and non-cohesive till soils of



varying consistency and relative density. Open-cut excavations into these materials should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act (OHSA) and Regulation 213 for Construction Activities. The existing fill and the silt and sand materials are classified as Type 3 soil; the native very soft to firm clayey silt deposits and cohesionless till (containing silt, sand, and gravel zones under high artesian pressures) are classified as Type 4 soil; and the clayey silt till is classified as Type 2 soil, according to the OHSA. Temporary excavations (i.e. those which are open for a relatively short time period) should be made with side slopes no steeper than 1H:1V through the Type 2 and 3 soils and to within 1.2 m of the bottom of the excavation in Type 2 soils only. In Type 4 soils the sides of the excavation should be sloped at no steeper than 3H:1V from ground surface down to the bottom of the Type 4 soil stratum.

6.11.3 Temporary Excavation Support

Temporary excavation support is likely required to facilitate the construction of the new retaining walls in order to maintain traffic on Highway 401, limit environmental impact to the area north of the Highway, and to reduce the quantity of sub-excavation required for the project. The temporary excavation support systems should be designed and constructed in accordance with OPSS.PROV 539 (*Temporary Protection Systems*). The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS 539, provided that the existing structure, as well as any adjacent utilities, can tolerate this magnitude of deformation.

It is considered that either a driven, interlocking sheetpile system or a soldier pile and timber lagging system would be suitable for the temporary excavation support at this site, based on the inferred subsurface soil conditions and groundwater conditions. An interlocking sheetpile system would contribute to both ground and, where applicable, groundwater control of seepage from cohesionless zones or interlayers/lenses within the cohesive deposits will be required. For a soldier pile and lagging system, it would be necessary to control seepage or include measures to mitigate loss of soil particles through the lagging boards in the water-bearing cohesionless soils encountered, and with the presence of high groundwater level at this site.

If deep excavation for the shallow foundation options is considered (up to 9.5 m below the Highway 401 road structure), a more elaborate excavation support system will be required. Lateral support to the sheetpiles or soldier piles could be provided in the form of rakers, temporary anchors or cross-bracing. The selection and design of the protection system will be the responsibility of the Contractor.

6.11.4 Groundwater and Surface Water Control

Artesian groundwater conditions were encountered in most of the boreholes at Retaining Wall 3 and in the boreholes closest to Fletcher's Creek at Retaining Wall 2. The artesian pressures (measured to be as high as Elevation 172 m in September 2012) were typically encountered within the cohesionless till deposit. The groundwater levels recorded in the augers and /or casing in the open boreholes where artesian conditions were not encountered typically range from about 0.3 m to 2.2 m below the existing ground surface/Highway 401 grade, corresponding to about Elevation 164.4 m. In addition, it was noted during the course of the fieldwork that the ground surface in the area of the abandoned Borehole FC-1 (observed a day after abandoning the borehole) advanced at the Retaining Wall 3 footprint, exhibited signs of groundwater seepage. Although groundwater was not observed emanating from the abandoned borehole, groundwater was observed to be



seeping from localized areas adjacent to the borehole. The seepage rates were periodically monitored and appeared to dissipate over time. No erosion or visual signs of instability were observed throughout the duration of the fieldwork.

Based on the water level measurements and visual observations of soil colour/moisture changes, the estimated shallow groundwater level is at about Elevation 164.4 m, corresponding to the existing ground surface and approximate water level at Fletcher's Creek.

Due to the proximity of Retaining Wall 3 to the edge of the Fletcher's Creek, a groundwater cut-off (cofferdam or similar measure) is recommended to minimize dewatering requirements and potential environmental impacts for excavation of the pile caps or other foundation options such as strip footings. A cut-off/cofferdam could consist of interlocking steel sheetpiles driven below the proposed base of excavation. In addition, measures would be required to control groundwater seepage and prevent loss of soil through the "gaps" that may exist at the base of some of the sheetpile sections during excavation.

If spread footings or sub-excavation and replacement options are considered, such as removal of the clayey silt deposit to the top of the cohesionless till (approximately Elevation 158 m, and about 6 m below the creek level), a well point dewatering system would be required to allow placement of concrete or compaction of engineered fill in the dry. Considering the high artesian pressures (up to Elevation 172 m) encountered in the cohesionless till soils, dewatering may not be feasible for the deep sub-excavations.

Caissons constructed with temporary or permanent liners within the cohesionless till subjected to unbalanced hydrostatic head at Retaining Wall 3 and possibly Retaining Wall 2 will require special measures to prevent 'boiling' or basal heave of the base materials. If caisson foundations are adopted, it is recommended that a constant head of water or drilling mud be maintained inside the caisson liners to counterbalance the natural groundwater or artesian conditions. Concrete placement by tremie methods would be required.

Due to the artesian conditions encountered in the areas of Retaining Walls 2 and 3, driven steel H-pile/tube pile or caisson liner installations within the cohesionless till will create a flow path along the pile to ground surface. Therefore a sand filter, in combination with a geotextile, should be placed beneath the pile caps and around the caisson liners if these are permanent installations, to prevent the migration of fines that may be transported along the piles or caisson liner during and after construction. The filter/drainage blanket should consist of a minimum 0.5 m thick layer of concrete fine aggregate meeting the gradation requirements of OPSS.PROV 1002 (*Aggregates – Concrete*). The concrete fine aggregate should extend a minimum of 0.5 m horizontally beyond the limit of the pile caps. Appropriate drainage from under the pile cap should be provided for the granular blanket by using a 100 mm perforated subdrain as per OPSS 405 (*Pipe Subdrains*) wrapped in knitted sock geotextile and draining to a temporary ditch or sump during construction. The geotextile should consist of non-woven, Class 2 fabric with filtration opening size (FOS) of not greater than 212 µm in accordance with OPSS 1860 (*Geotextiles*).

It is recommended that an NSSP be included in the Contract Documents to warn the contractor of the artesian groundwater levels during foundation construction; an example NSSP is presented in Appendix F.



6.11.4.1 Permit to Take Water

The Ontario Ministry of Environment (MOE) requires a Permit To Take Water (PTTW) for any groundwater pumping in excess of 50 m³/day. Based on the result of the drawdown/seepage analysis carried out as part of the Fletcher's Creek Bridge Report (GEOCRE 30M12-356), the proximity to Fletcher's Creek and the high groundwater pressure observed locally, it is expected that an MOE Permit To Take Water will be required to support the construction of the retaining walls at this site.

6.11.5 Subgrade Protection

The native soils that will be exposed at the foundation subgrade level will be susceptible to disturbance from construction traffic, groundwater infiltration and/or ponded water. To limit this degradation, it is recommended that a concrete working slab be placed on the subgrade immediately after preparation, inspection and approval of the footing subgrade as noted in Section 6.6.2.

6.11.6 Obstructions During Pile Driving / Caisson Installation

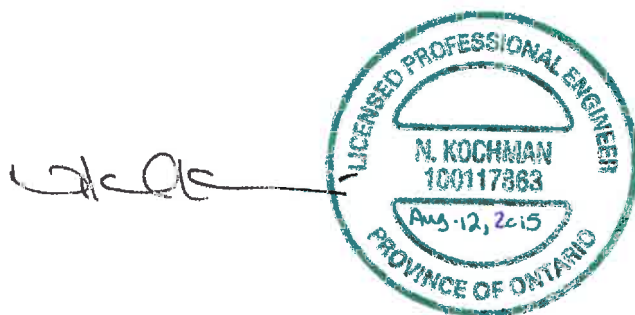
Cobbles and/or boulders were inferred due to difficulty in advancing the augers and auger grinding at varying depths (typically below Elevation 157 m) in the boreholes drilled during the current subsurface investigation, which may affect the installation of steel H-piles/tube piles or caissons. It is recommended that driving shoes be used on all steel H-piles or tube piles to facilitate driving into the overburden soils. In addition it is recommended that an NSSP be included in the Contract Documents to warn the Contractor of the possible presence of cobbles and/or boulders within the overburden soils and an example NSSP is presented in Appendix F.



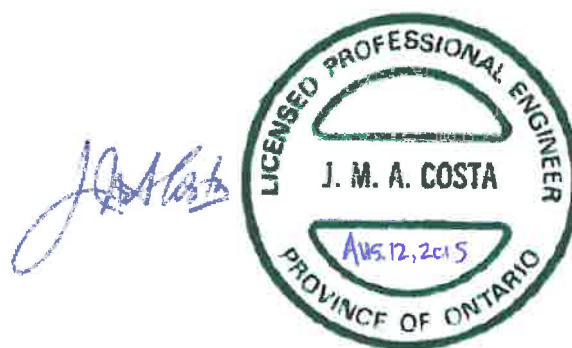
7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Nikol Kochmanová, P.Eng., a geotechnical engineer with Golder, and was reviewed by Mr. Kevin Bentley, P.Eng., a senior geotechnical engineer and Associate of Golder. Mr. Jorge Costa, P.Eng., the Designated MTO Contact for this project and Principal of Golder, conducted a technical review and quality control review of the report.

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Ontario Provincial Standard Specifications (OPSS)

OPSS.PROV 206	Construction Specification for Grading
OPSS 405	Construction Specifications for Pipe Subdrains
OPSS.PROV 501	Construction Specifications for Compacting
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS 902	Construction Specification for Excavating and Backfilling Structures
OPSS 903	Construction Specification for Deep Foundations
OPSS 942	Construction Specification for Prestressed Soil and Rock Anchors
OPSS.PROV 1002	Material Specification for Aggregates – Concrete



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OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material
OPSS 1860	Material Specification for Geotextiles

Ontario Provincial Standard Drawings (OPSD)

OPSD 208.010	Benching of Earth Slopes
OPSD 3000.100	Foundation, Piles, Steel H-Pile Driving Shoe
OPSD 3001.100	Foundation, Piles, Steel Tube Pile Driving Shoe
OPSD 3090.101	Foundation, Frost Penetration Depths for Southern Ontario

ASTM International

ASTM D1586	Standard Test Method for Standard Penetration Test (SPT) and Split Barrel Sampling of Soils
ASTM 2573	Standard Test Method for Filed Vane Sear 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.



LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

Notes: 1
2

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

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



LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

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, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

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



FOUNDATION REPORT - RETAINING WALLS

HIGHWAY 401 WIDENING, GWP 2150-01-00

TABLE 1 – COMPARISON OF RETAINING WALL AND FOUNDATION ALTERNATIVES
RETAINING WALL NOS. 1 TO 3

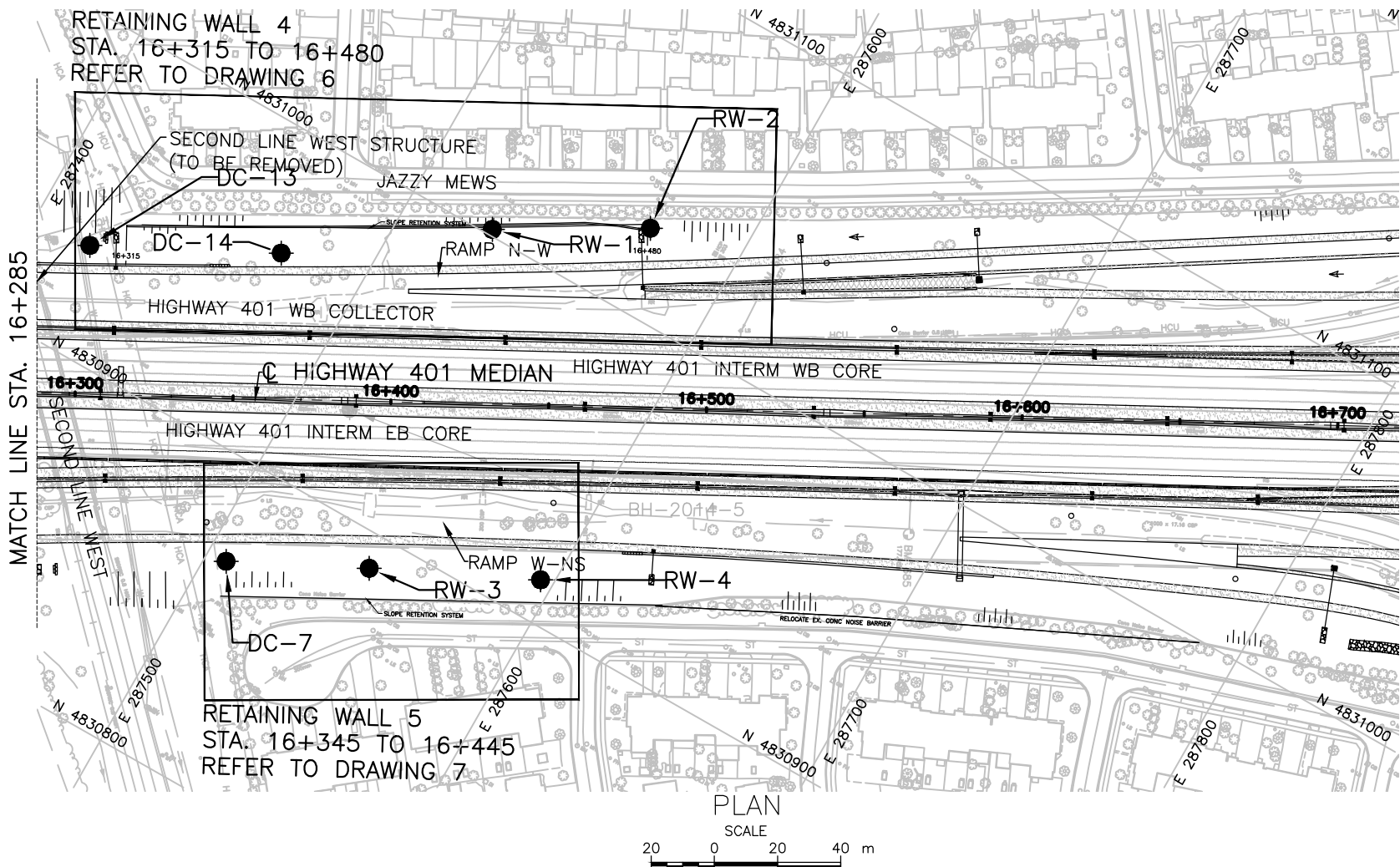
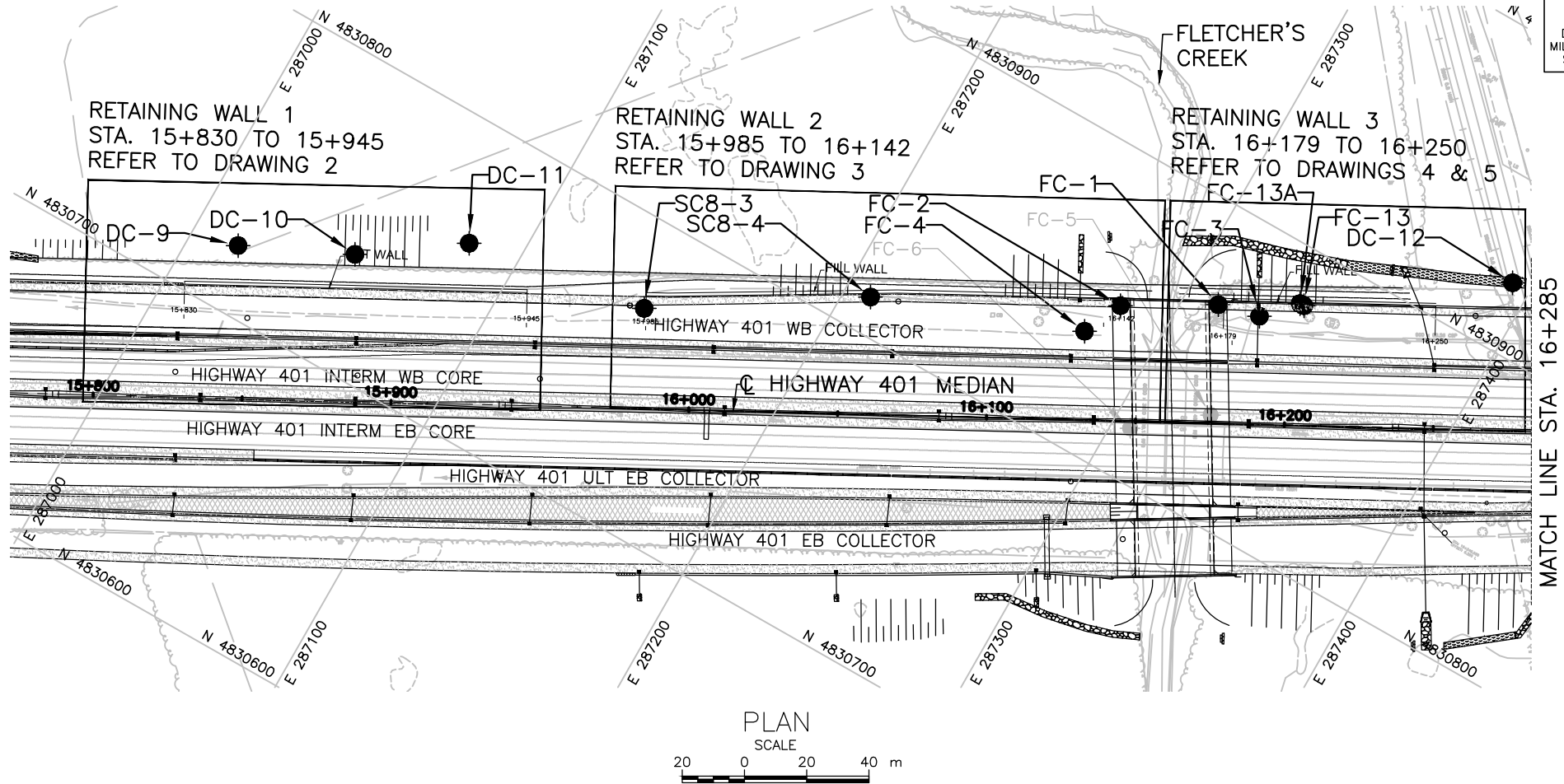
Foundation Option	Feasibility			Advantages	Disadvantages	Relative Costs
	Retaining Wall 1	Retaining Wall 2	Retaining Wall 3			
Soldier Pile and Concrete Panel Wall with Tie-Backs	Feasible – requires sufficient right-of-way space for tie-backs • Preferred Alternative	Not appropriate as this is a fill wall site	Not appropriate as this is a fill wall site	<ul style="list-style-type: none"> Most advantageous in “top-down” construction applications, such as the cut section at Retaining Wall 1 Retaining Wall 1 - minimizes excavation and requirement for temporary excavation support 	<ul style="list-style-type: none"> Easement for soil anchors may be required at Retaining Wall 1, depending on distance from wall to property limits Likely more time-consuming to install 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) 	<ul style="list-style-type: none"> Comparable costs to concrete retaining wall, but higher than RSS wall Cost of temporary protection system combined with RSS wall is comparable
Concrete Retaining Wall on Deep Foundations	Feasible – may require tie-backs; requires pile cap below frost penetration depth	Feasible – may require tie-backs installed towards / under existing highway or battered piles • Preferred Alternative	Feasible – may require tie-backs installed towards / under existing highway or battered piles • Preferred Alternative	<ul style="list-style-type: none"> Potentially reduced excavation, protection system and backfill requirements compared to RSS wall 	<ul style="list-style-type: none"> Temporary/permanent liners may be required to allow for construction of caissons If refusal (100-blow) stratum or obstructions are encountered, can get piles to hang-up, requiring pre-drilling If tie-backs are required, significant length required to anchor into competent till soils and design will need to account for settlement of embankment 	<ul style="list-style-type: none"> Higher cost relative to RSS wall
Concrete Wall on Shallow	Feasible provided sufficient	Not Feasible – requires up to 5 m of	Not Feasible – requires up to 7 m of	<ul style="list-style-type: none"> Conventional excavation and construction techniques 	<ul style="list-style-type: none"> Less tolerable to post construction settlements Temporary excavation 	<ul style="list-style-type: none"> Higher cost relative to RSS wall



FOUNDATION REPORT - RETAINING WALLS

HIGHWAY 401 WIDENING, GWP 2150-01-00

Foundation Option	Feasibility			Advantages	Disadvantages	Relative Costs
	Retaining Wall 1	Retaining Wall 2	Retaining Wall 3			
Foundations	space is available during construction and/or for temporary shoring if used. Construction must not interfere with existing fence.	excavation to found footings below very soft to firm clayey soils	excavation to found footings below very soft to firm clayey soils	<ul style="list-style-type: none"> Suitable founding stratum below depth of frost penetration at Retaining Wall 1 	support will be required <ul style="list-style-type: none"> A temporary construction easement may be required Footings must be founded below depth of frost penetration Retaining Wall 1 requires additional width of excavation for footing construction and new backfill behind wall Significant dewatering effort at Retaining Wall 2 and 3 	
RSS Walls	Feasible but not applicable for a cut embankment	Not Practical – requires up to 5 m of excavation to found front facing footing and wall panels below very soft to firm clayey soil stratum	Not Practical – requires up to 7 m of excavation to found front facing and wall mass below existing soft/firm fill and soft clayey silt stratum	<ul style="list-style-type: none"> More tolerable to post construction settlements Lowest cost alternative where feasible 	<ul style="list-style-type: none"> Potentially larger amount of excavation required to install reinforcing strips; temporary protection systems required Significant dewatering effort required at Retaining Wall 2 and 3 	<ul style="list-style-type: none"> Lower cost than concrete retaining wall or walls supported on deep foundations
Reinforced Earth Slope Embankment	Feasible - but not applicable for a cut embankment	Feasible provided sufficient space is available	Feasible provided sufficient space is available	<ul style="list-style-type: none"> Relative ease of construction but proprietary product required Vegetated surfaces could be used to improve aesthetics 	<ul style="list-style-type: none"> Proprietary product design Special treatment of reinforced earth slope surfaces required to allow vegetation to grow and minimize erosion 	<ul style="list-style-type: none"> Lower cost than RSS wall



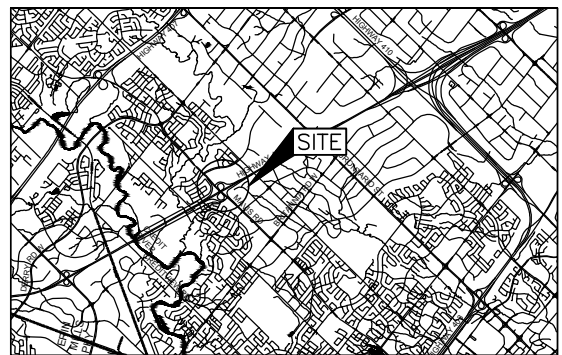
METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 2150-01-00



HIGHWAY 401
RETAINING WALLS
INDEX PLAN

SHEET



KEY PLAN
SCALE
1.5 0 1.5 3 km

NOTES

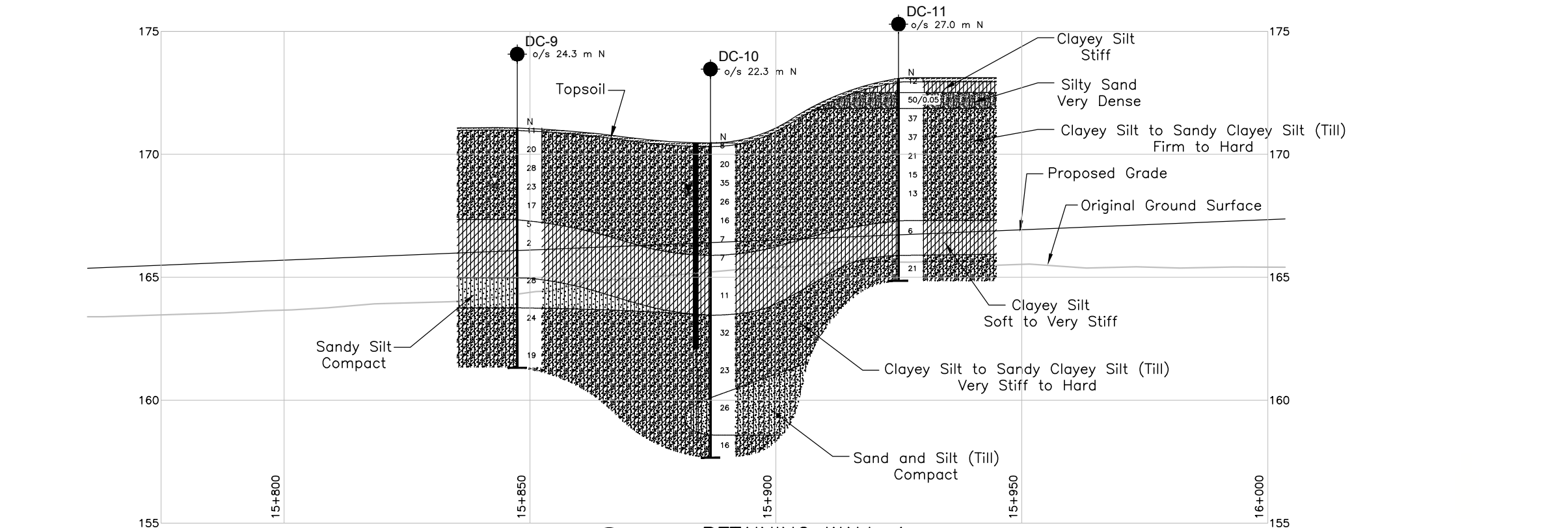
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REFERENCE

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X-60213979-C-DE-NB & SLOPE.dwg, received February 25, 2015 and
X-60213979-C-DE-HWY401_MAVIS.dwg, received September 24, 2014.

NO.	DATE	BY	REVISION
Geocres No. 30M12-388			
HWY. 401		PROJECT NO. 10-1111-0211	DIST. .
SUBM'D. KJB	CHKD. NK	DATE: July 2015	SITE: .
DRAWN: JFC	CHKD. KJB	APPD. JMAC	DWG. 1










SHEET



KEY PLAN
SCALE



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 November 05, 2012
 WL upon completion of drilling

BOREHOLE CO—ORDINATES			
No.	ELEVATION	NORTHING	EASTING
DC—9	171.1	4830722.1	287015.8
DC—10	170.5	4830738.9	287051.3
DC—11	173.1	4830761.2	287082.8

NOTES

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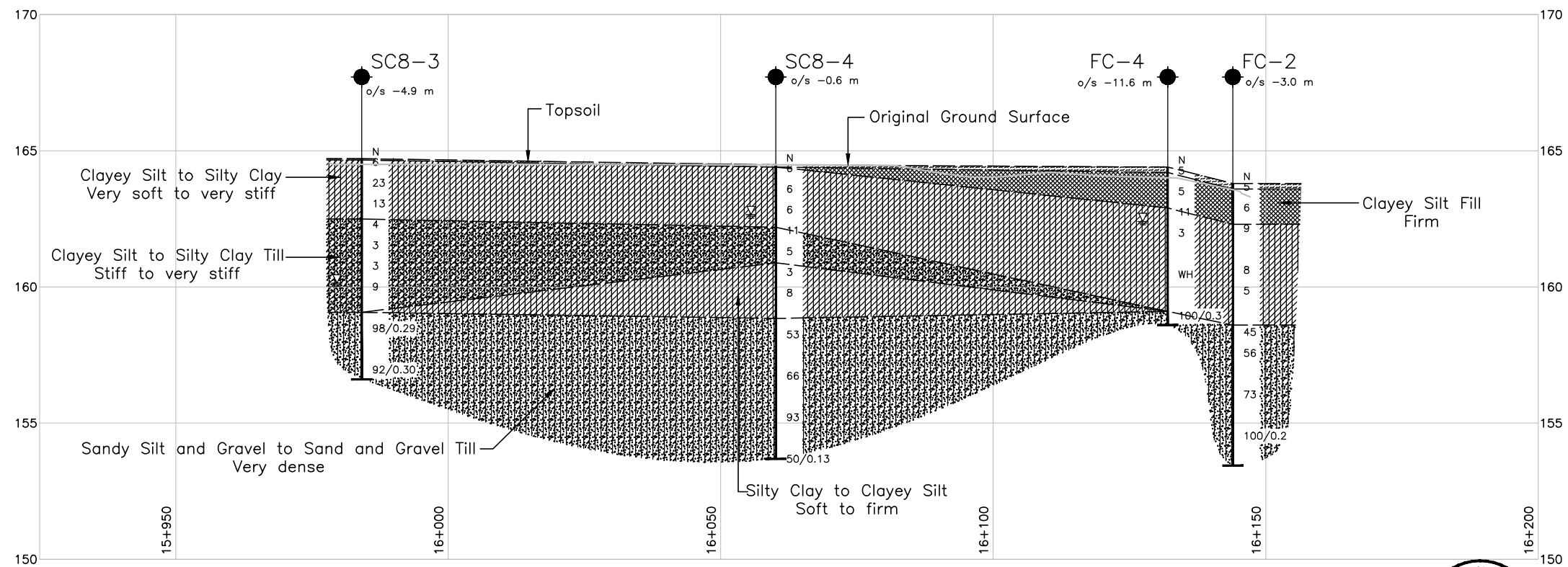
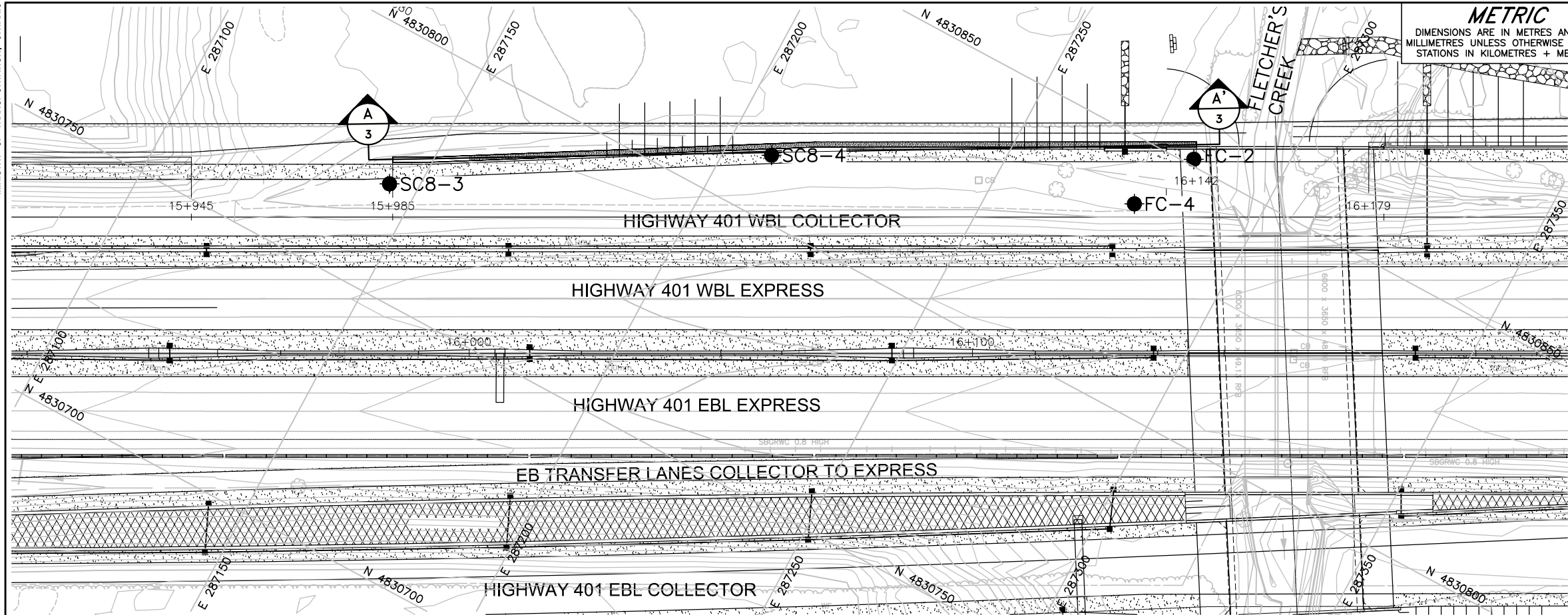
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-	-	-	-	-
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Geocres No. 30M12-388				
HWY. 401		PROJECT NO. 10-1111-0211		DIST. .
SUBM'D. KJB	CHKD. NK	DATE: July 2015		SITE:
DRAWN: JFC	CHKD. KJB	APPD. JMCA		DWG. 2



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

CONT No. .
GWP No. 2150-01-00



HIGHWAY 401
RETAINING WALL 2 STA. 15+985 TO 16+142
BOREHOLE LOCATIONS AND
SOIL STRATA

SHEET



KEY PLAN
SCALE
10 0 10 20 m

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 November 05, 2012
- ≡ WL upon completion of drilling

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
SC8-3	164.7	4830771.3	287144.6
SC8-4	164.5	4830812.3	287208.7
FC-2	163.8	4830851.3	287283.0
FC-4	164.4	4830837.9	287276.8

NOTES

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NO.	DATE	BY	REVISION
Geocres No. 30M12-388			
HWY. 401	PROJECT NO. 10-1111-0211	DIST. .	
SUBM'D. KJB	CHKD. NK	DATE: July 2015	SITE: .
DRAWN: JFC	CHKD. KJB	APPD. JMAC	DWG. 3

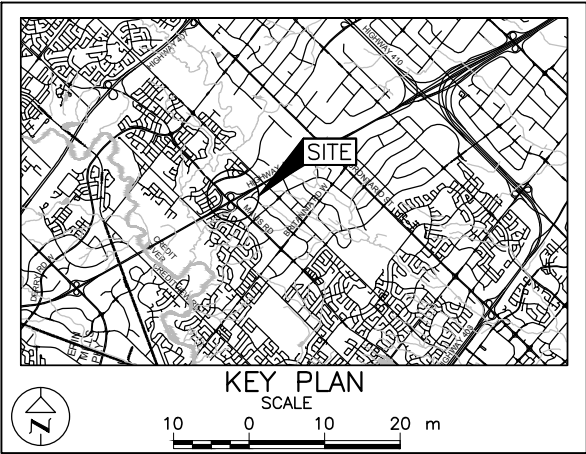
METRIC
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STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 2150-01-00

HIGHWAY 401
RETAINING WALL 3 STA. 16+179 TO 16+250

BOREHOLE LOCATIONS


SHEET



LEGEND

 Borehole – Current Investigation

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
DC-12	176.3	4830923.2	287393.6
FC-1	163.7	4830867.9	287311.4
FC-3	165.9	4830871.4	287325.3
FC-13	167.1	4830881.8	287336.5
FC-13A	167.1	4830881.8	287335.0

NOTES

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REFERENCE

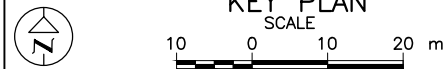
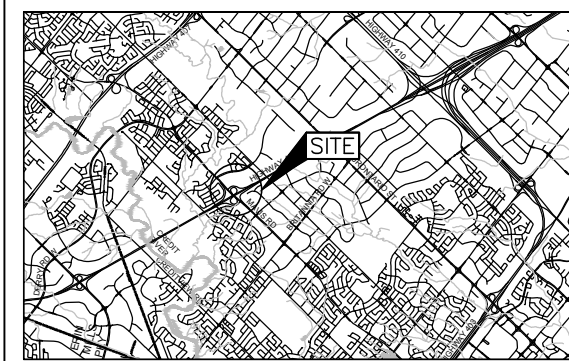
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Geocres No. 30M12-388			
HWY. 401		PROJECT NO. 10-1111-0211	DIST. .
SUBM'D. KJB	CHKD. NK	DATE: July 2015	SITE: .
DRAWN: JFC	CHKD. KJB	APPD. JMAC	DWG. 4

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

CONT No. GWP No. 2150-01-00		SHEET
HIGHWAY 401 RETAINING WALL 3 STA. 16+179 TO 16+250		
SOIL STRATA		



LEGEND

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

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
DC-12	176.3	4830923.2	287393.6
FC-1	163.7	4830867.9	287311.4
FC-3	165.9	4830871.4	287325.3
FC-13	167.1	4830881.8	287336.5
FC-13A	167.1	4830881.8	287335.0

NOTES

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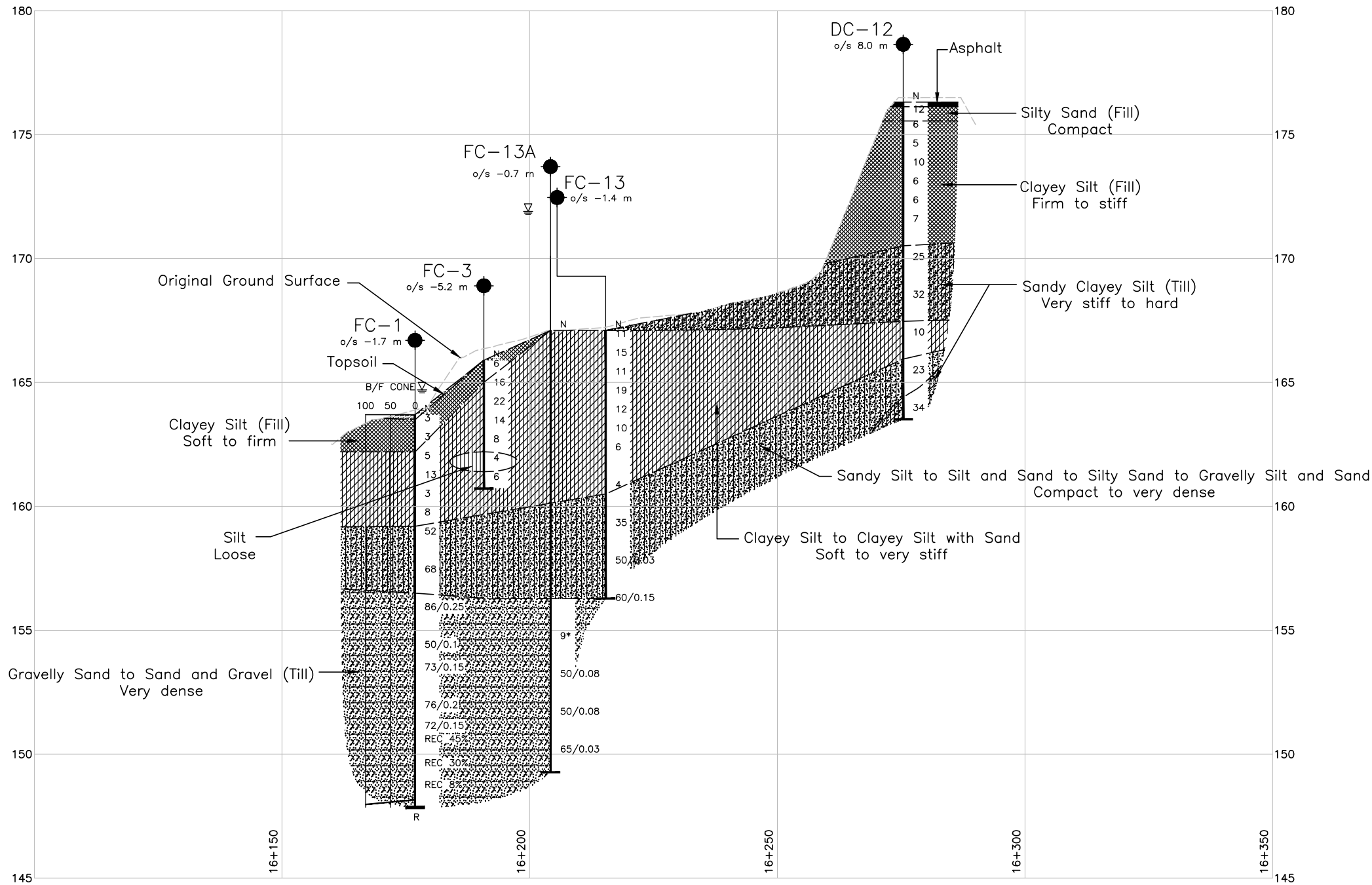
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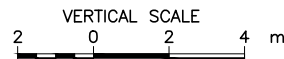
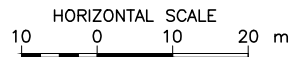
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Base plans provided in digital format by AECOM, drawing file nos.
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X-60213979-C-DE-NB & SLOPE.dwg, received February 25, 2015 and
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NO.	DATE	BY	REVISION
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HWY. 401	PROJECT NO. 10-1111-0211		DIST. .
SUBM'D. KJB	CHKD. NK	DATE: July 2015	SITE: .
DRAWN: JFC	CHKD. KJB	APPD. JMAC	DWG. 5



RETAINING WALL 3
STA. 16+182 TO 16+250





VERTICAL SCALE

5 0 5 10 m

HORIZONTAL SCALE

10 0 10 20 m

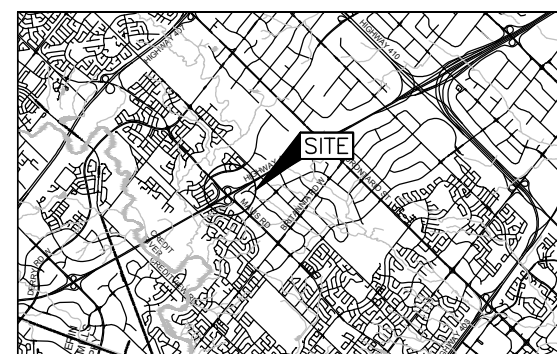


SHEET

HIGHWAY 401

RETAINING WALL 4 STA. 16+315 TO 16+480






BOREHOLE LOCATIONS AND SOIL STRATA



KEY PLAN
SCALE



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
November 05, 2012 (DC-13) and
September 15, 2014 (RW-1) |
|  | WL upon completion of drilling |

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
DC-13	174.7	4830934.4	287419.4
DC-14	175.4	4830962.3	287473.2
RW-1	175.2	4831002.0	287527.6
RW-2	176.0	4831027.1	287570.9

NOTES

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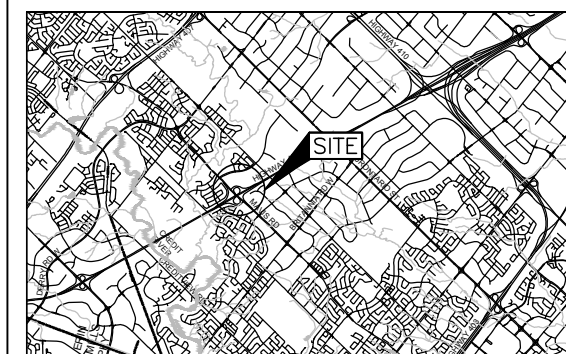
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NO.		DATE		BY	
				REVISION	
Geocres No. 30M12-388					
HWY. 401			PROJECT NO. 10-1111-0211		DIST.
SUBM'D. KJB		CHKD. NK		DATE: July 2015	
DRAWN: JFC		CHKD. KJB		APPD. JMAG	
				DWG. 6	










SHEET



KEY PLAN
SCALE



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
November 05, 2012 (DC-7) and
September 15, 2014 (RW-3) |
|  | WL upon completion of drilling |

BOREHOLE CO—ORDINATES			
No.	ELEVATION	NORTHING	EASTING
DC—7	175.1	4830868.9	287506.4
RW—3	173.7	4830889.5	287546.9
RW—4	174.4	4830913.2	287595.9

NOTES

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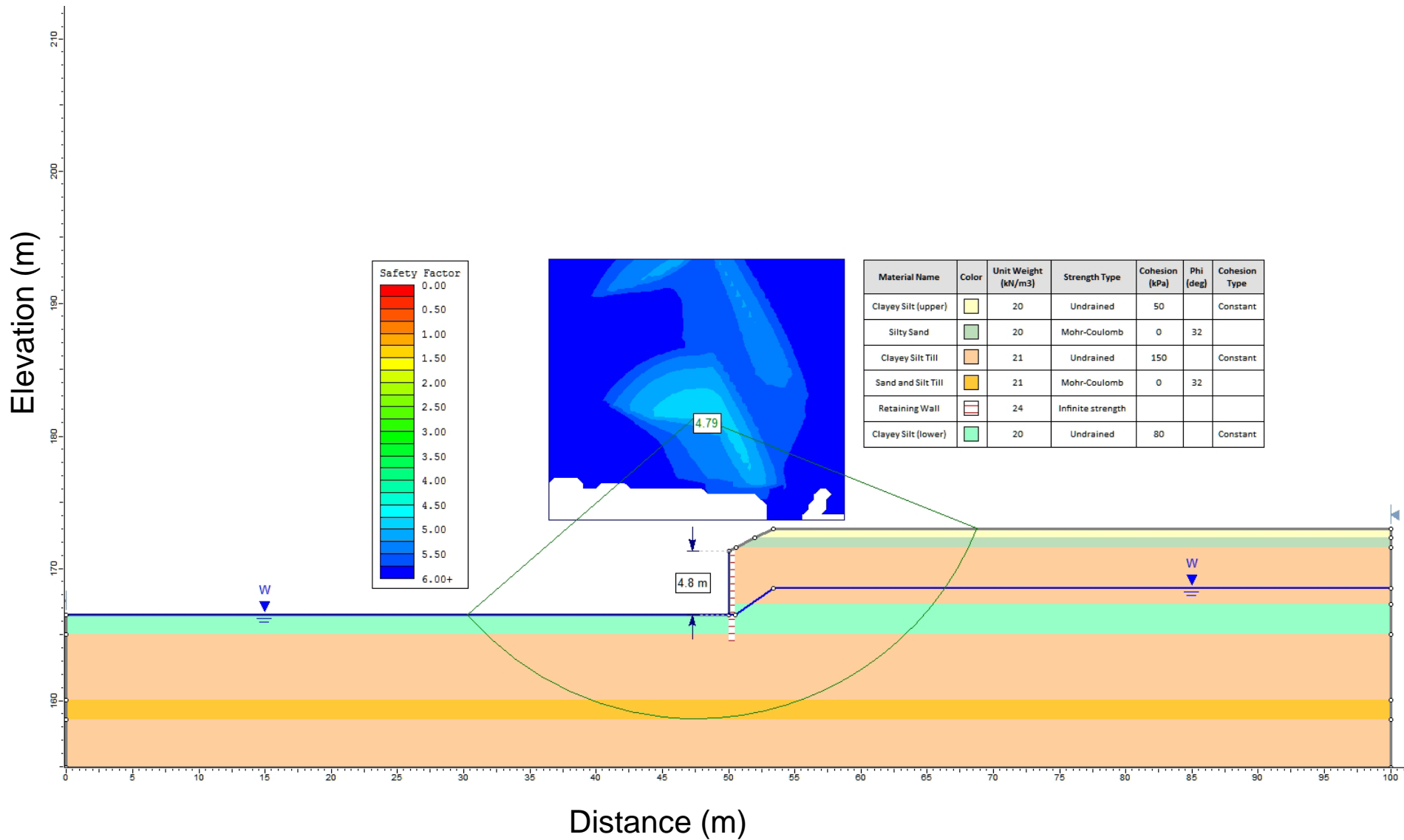
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X-60213979-C-DE-HWY401_FLTCH_CRK - RW Plan.dwg,
X-60213979-C-DE-NB & SLOPE.dwg, received February 25, 2015 and
X-60213979-C-DE-HWY401_MAVIS.dwg, received September 24, 2014.

NO.		DATE		BY	
				REVISION	
Geocres No. 30M12-388					
HWY. 401			PROJECT NO. 10-1111-0211		DIST. ,
SUBM'D. KJB		CHKD. NK		DATE: July 2015	
DRAWN: JFC		CHKD. KJB		APPD. JMAM	
				DWG. 7	



STATIC GLOBAL STABILITY
RETAINING WALL 1 – SOLDIER PILE AND CONCRETE PANEL WALL
(PREFERRED ALTERNATIVE)

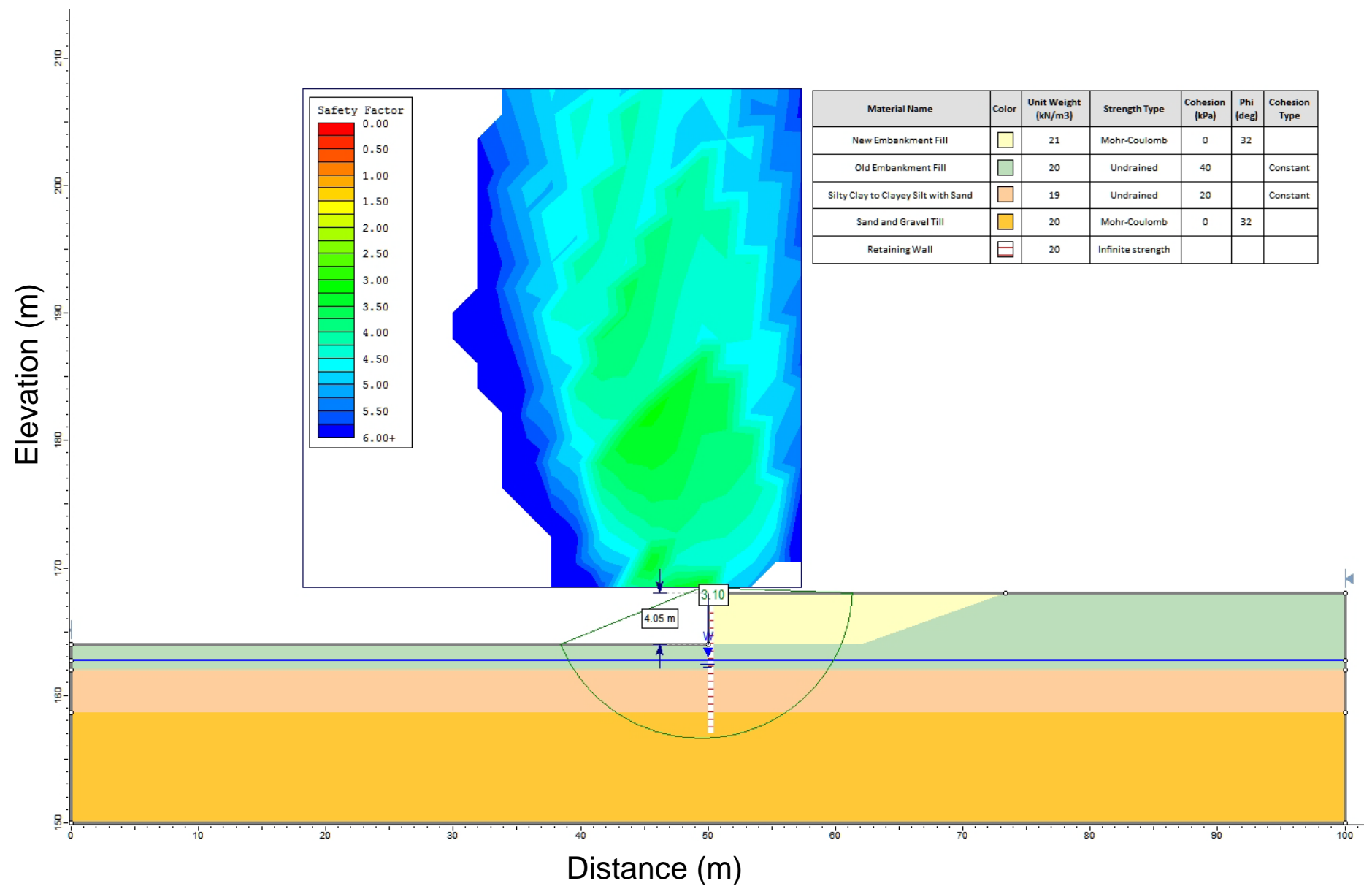
Figure 1





**STATIC GLOBAL STABILITY
RETAINING WALL 2 – CONCRETE WALL ON DEEP FOUNDATIONS
(PREFERRED ALTERNATIVE)**

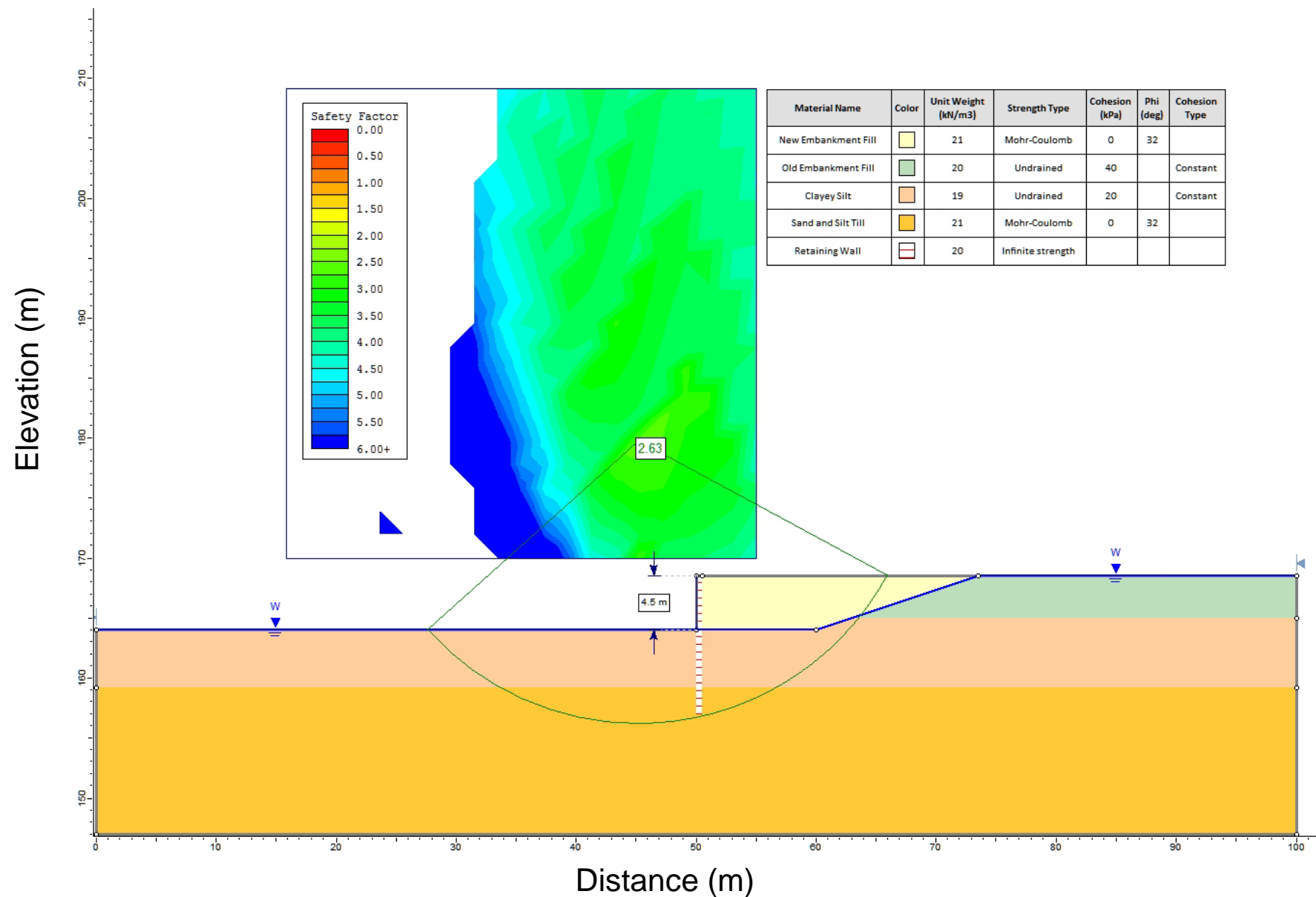
Figure 2





STATIC GLOBAL STABILITY RETAINING WALL 3 – CONCRETE WALL ON DEEP FOUNDATIONS (PREFERRED ALTERNATIVE)

Figure 3





APPENDIX A

Retaining Wall 1, Highway 401 - STA. 15+830 to STA 15+945 **Record of Borehole Sheets and Laboratory Test Results**

PROJECT		10-1111-0211		RECORD OF BOREHOLE No DC-9		SHEET 1 OF 1		METRIC																
2150-01-00		LOCATION		N 4830722.1 ; E 287015.8		ORIGINATED BY		SB																
DIST		HWY		401- 403 W Ramp		BOREHOLE TYPE		57 mm I.D. Hollow Stem Augers																
DATE		Geodetic		DATE		September 7, 2012		CHECKED BY																
								TVA																
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			ELEVATION SCALE			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES																			
171.1	GROUND SURFACE																							
0.9	TOPSOIL		1	SS	11																			
	CLAYEY SILT, some sand, trace gravel, containing roots to a depth of 0.6 m (TILL) Stiff to very stiff Brown Moist		2	SS	20																			
			3	SS	28																			
			4	SS	23																			
			5	SS	17																			
167.4																								
3.7	CLAYEY SILT, trace sand, trace gravel Soft to very stiff Grey Moist to wet		6	SS	5																			
			7	SS	2																			
165.0																								
6.1	Sandy SILT, trace to some clay Compact Grey Moist to wet		8	SS	28																			
163.8																								
7.3	CLAYEY SILT, trace sand, trace gravel (TILL) Very stiff Grey Moist		9	SS	24																			
161.3			10	SS	19																			
9.8	END OF BOREHOLE																							
	NOTES: 1. Borehole open to a depth of 8.4 m below ground surface (Elev. 162.7 m). 2. Water level in open borehole at a depth of 2.3 m below ground surface (Elev. 168.8 m) upon completion of drilling.																							

PROJECT 10-1111-0211		RECORD OF BOREHOLE No DC-10		SHEET 1 OF 1		METRIC	
2150-01-00		LOCATION N 4830738.9 ; E 287051.3		ORIGINATED BY SB			
DIST _____ HWY 401- 403 W Ramp		BOREHOLE TYPE 57 mm I.D. Hollow Stem Augers		COMPILED BY BM			
DATUM Geodetic		DATE September 11, 2012		CHECKED BY TVA			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)		
								20 40 60 80 100	20 40 60 80 100	W _p W W _L							
170.5	GROUND SURFACE																
0.0	TOPSOIL																
0.2	CLAYEY SILT, some sand, trace gravel, containing roots to a depth of 1.5 m (TILL) Firm to hard Brown Moist		1	SS	8							○					
			2	SS	20												
			3	SS	35								○	┌───┐	4 20 49 27		
			4	SS	26												
			5	SS	16								○				
			6	SS	7												
165.9	CLAYEY SILT, trace sand, trace gravel Firm to stiff Grey Moist to wet		7	SS	7							┌─○─┐					
			8	SS	11												
163.5	CLAYEY SILT, sandy, trace gravel (TILL) Very stiff to hard Grey Moist																
7.0			9	SS	32								○	┌──┐	5 30 48 17		
			10	SS	23												
160.1	SAND and SILT, trace to some clay, trace to some gravel (TILL) Compact Grey Moist																
10.4			11	SS	26								○		6 35 50 9		
158.6	CLAYEY SILT, trace sand, trace gravel (TILL) Very stiff Grey Moist																
11.9			12	SS	16												
157.7	END OF BOREHOLE NOTE: 1. Piezometer installation consists of 50 mm diameter PVC pipe with a 3.0 m slotted screen. Water Level Readings Date Depth (m) Elev. (m) 09/11/12 12.0 158.5 10/09//12 1.9 168.6 11/05//12 2.0 168.5																
12.8																	

GTA-MTO 001 1011110211.GPJ GAL-GTA.GDT 2/17/15 PR

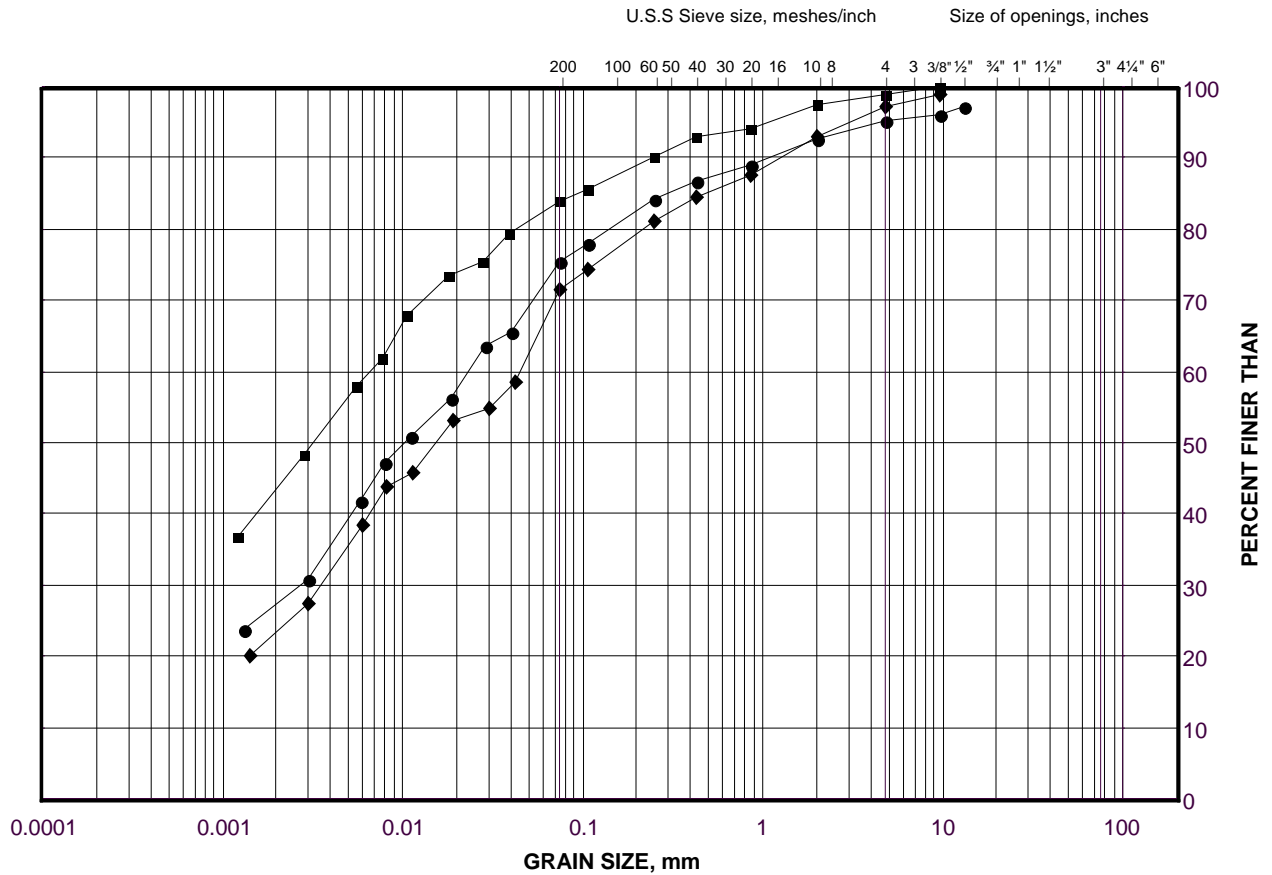
PROJECT <u>10-1111-0211</u>		RECORD OF BOREHOLE No DC-11		SHEET 1 OF 1		METRIC	
<u>2150-01-00</u>		LOCATION <u>N 4830761.2 ; E 287082.8</u>		ORIGINATED BY <u>SB</u>			
DIST <u> </u> HWY <u>401- 403 W Ramp</u>		BOREHOLE TYPE <u>57 mm I.D. Hollow Stem Augers</u>		COMPILED BY <u>BM</u>			
DATUM <u>Geodetic</u>		DATE <u>September 7, 2012</u>		CHECKED BY <u>TVA</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
								20 40 60 80 100	20 40 60 80 100	10 20 30				
						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED								
173.1	GROUND SURFACE						173							
0.0	TOPSOIL													
0.2	CLAYEY SILT, trace sand, trace gravel, containing roots		1	SS	12									
172.3	Stiff Brown Moist						172							
0.8	Silty SAND, trace clay, trace gravel		2	SS	50/0.05									
171.6	Very dense Brown Moist						171							
1.5	CLAYEY SILT, sandy, trace gravel (TILL)		3	SS	37									
	Stiff to hard Brown to grey Moist						170							
			4	SS	37									
			5	SS	21									
			6	SS	15		169							
			7	SS	13		168							
167.3	CLAYEY SILT, trace sand, trace gravel						167							
5.8	Firm Grey Moist		8	SS	6									
165.9	CLAYEY SILT, some sand, trace gravel (TILL)						166							
7.2	Very stiff Grey Moist													
164.9	END OF BOREHOLE		9	SS	21		165							
8.2	NOTES: 1. Borehole open to a depth of 7.0 m below ground surface (Elev. 166.1 m). 2. Open borehole dry upon completion of drilling.													

GRAIN SIZE DISTRIBUTION

Sandy Clayey Silt to Clayey Silt (Till) (Upper Deposit)
Retaining Wall 1

FIGURE A1



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

LEGEND

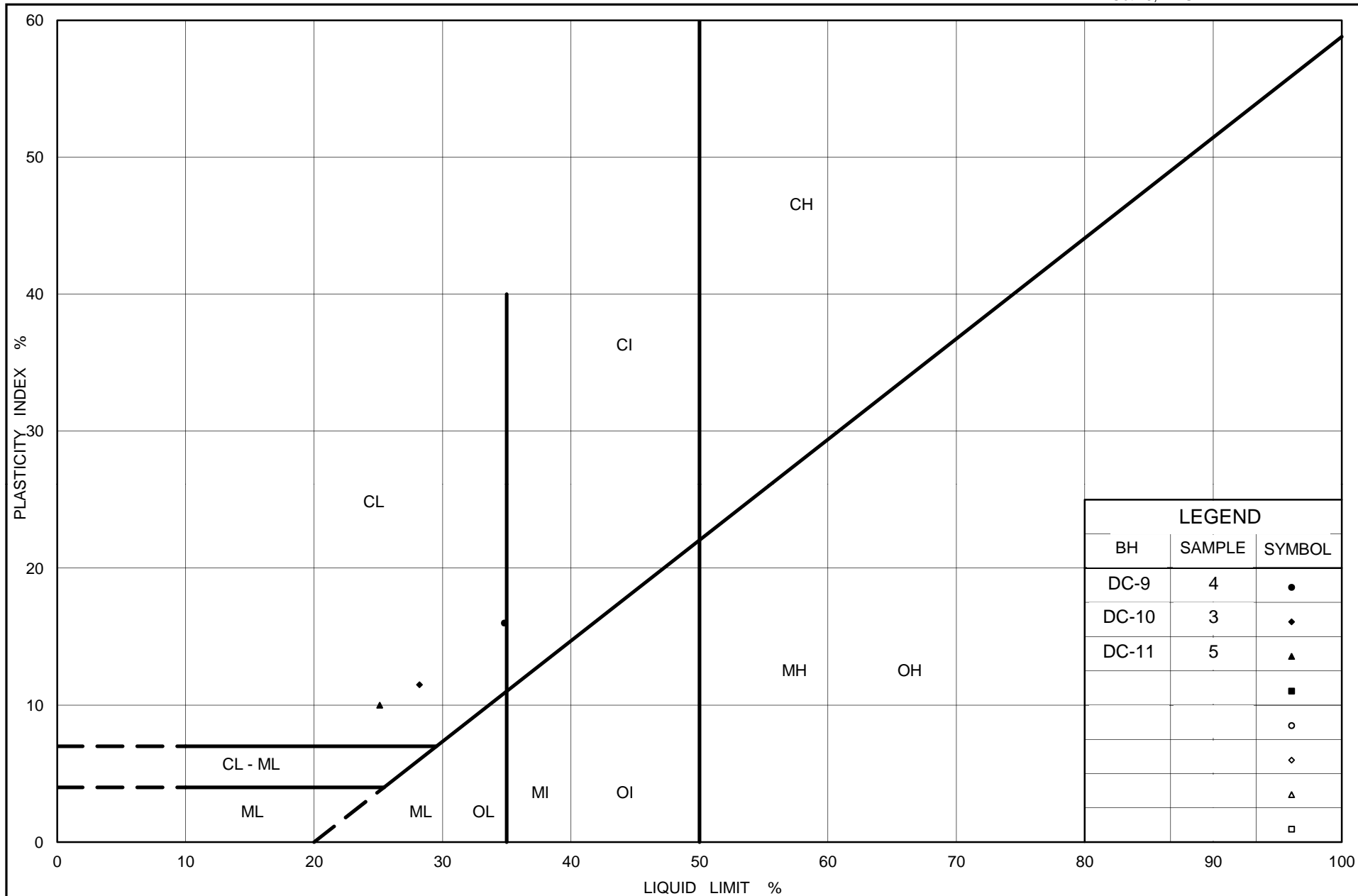
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	DC-10	3	168.6
■	DC-9	4	168.5
◆	DC-11	5	169.7

Project Number: 10-1111-0211

Checked By: NK

Golder Associates

Date: 11-Mar-15



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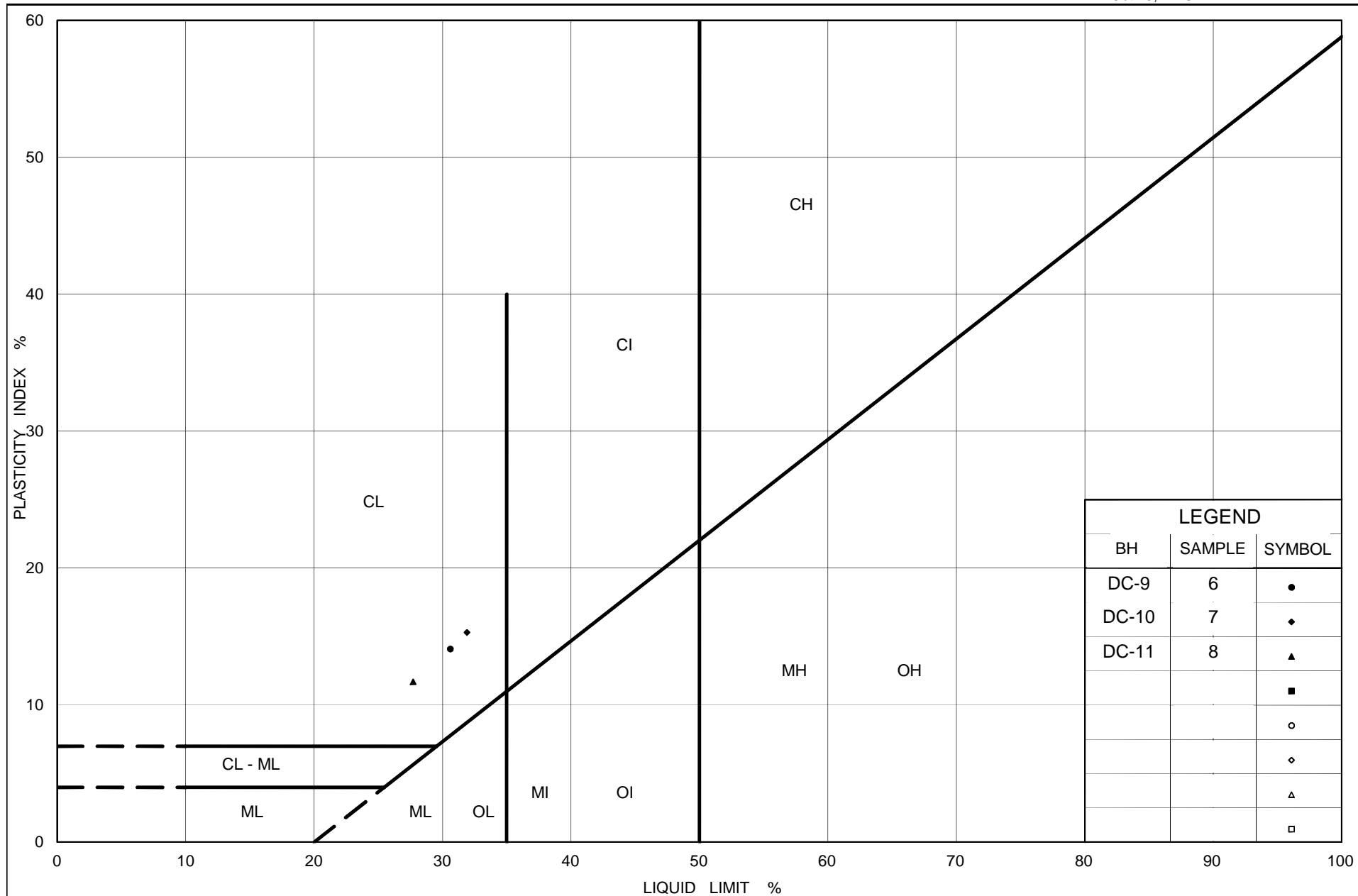
Ontario

PLASTICITY CHART **Sandy Clayey Silt to Clayey Silt (Till) (Upper Deposit)** **Retaining Wall 1**

Figure No. A2

Project No. 10-1111-0211

Checked By: NK



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PLASTICITY CHART Clayey Silt Retaining Wall 1

Figure No. A3

Project No. 10-1111-0211

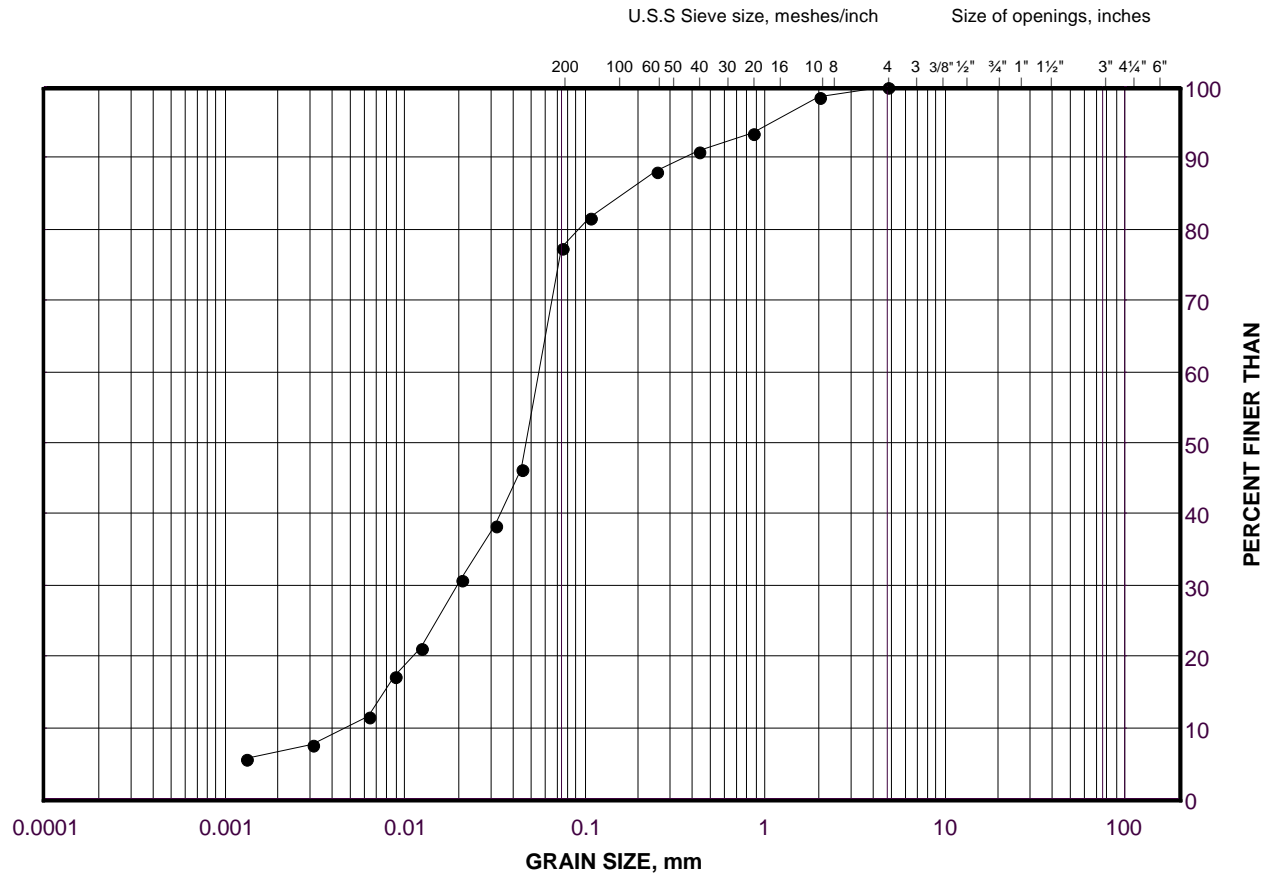
Checked By: NK

GRAIN SIZE DISTRIBUTION

Sandy Silt (Interlayer)

Retaining Wall 1

FIGURE A4



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	DC-9	8	164.7

Project Number: 10-1111-0211

Checked By: NK

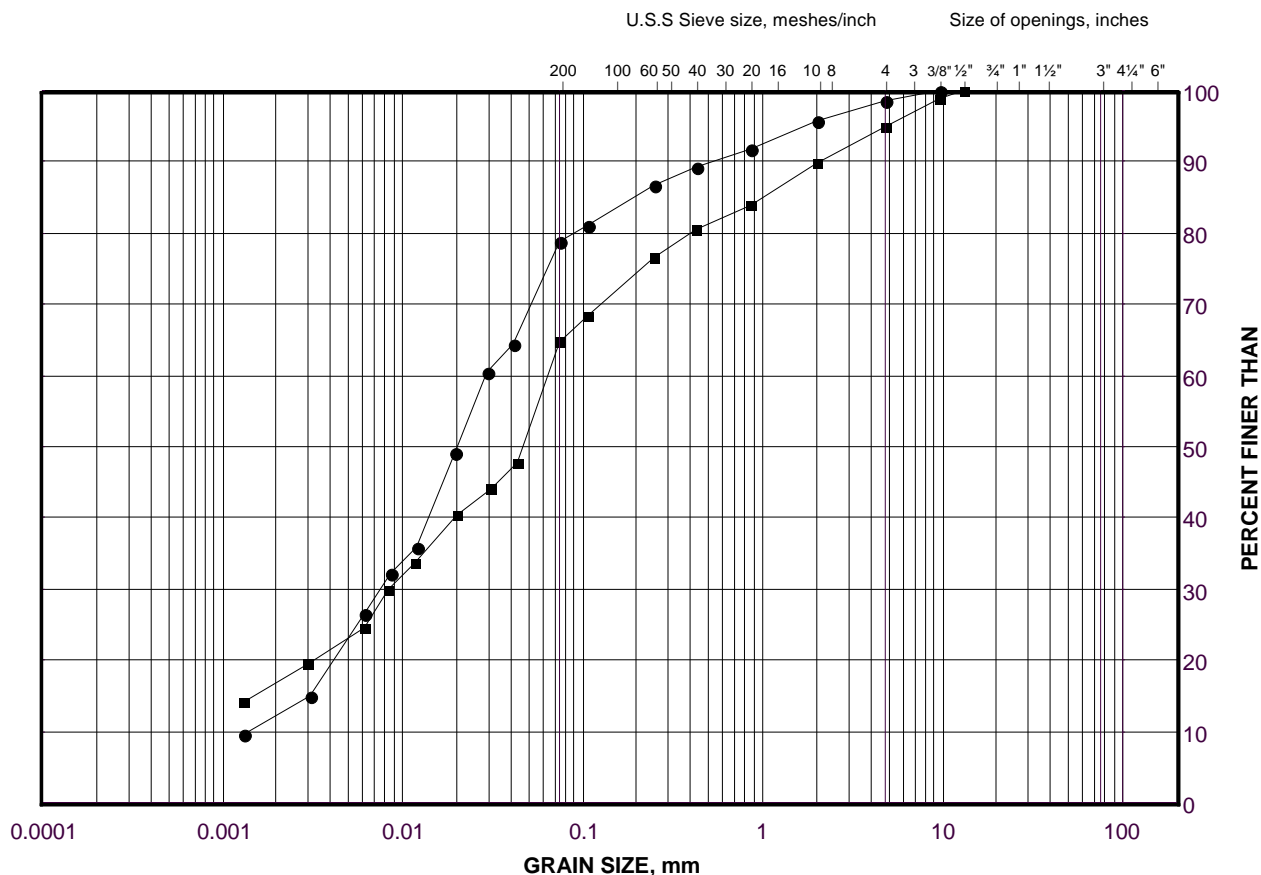
Golder Associates

Date: 11-Mar-15

GRAIN SIZE DISTRIBUTION

Sandy Clayey Silt to Clayey Silt (Till) (Lower Deposit)
Retaining Wall 1

FIGURE A5



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

LEGEND

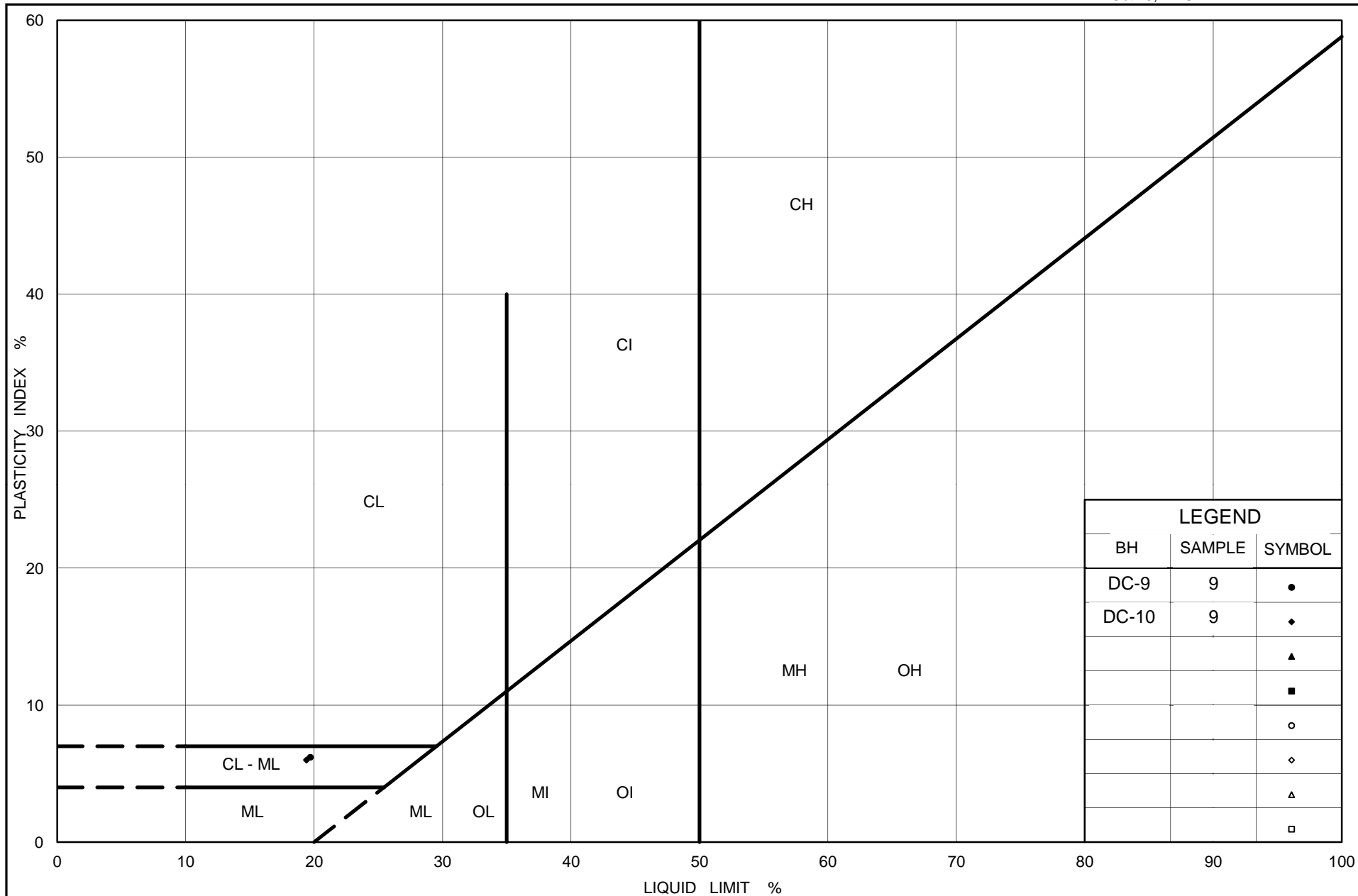
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	DC-11	9	165.2
■	DC-10	9	162.5

Project Number: 10-1111-0211

Checked By: NK

Golder Associates

Date: 11-Mar-15



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PLASTICITY CHART Sandy Clayey Silt to Clayey Silt (Till) (Lower Deposit) Retaining Wall 1

Figure No. A6

Project No. 10-1111-0211

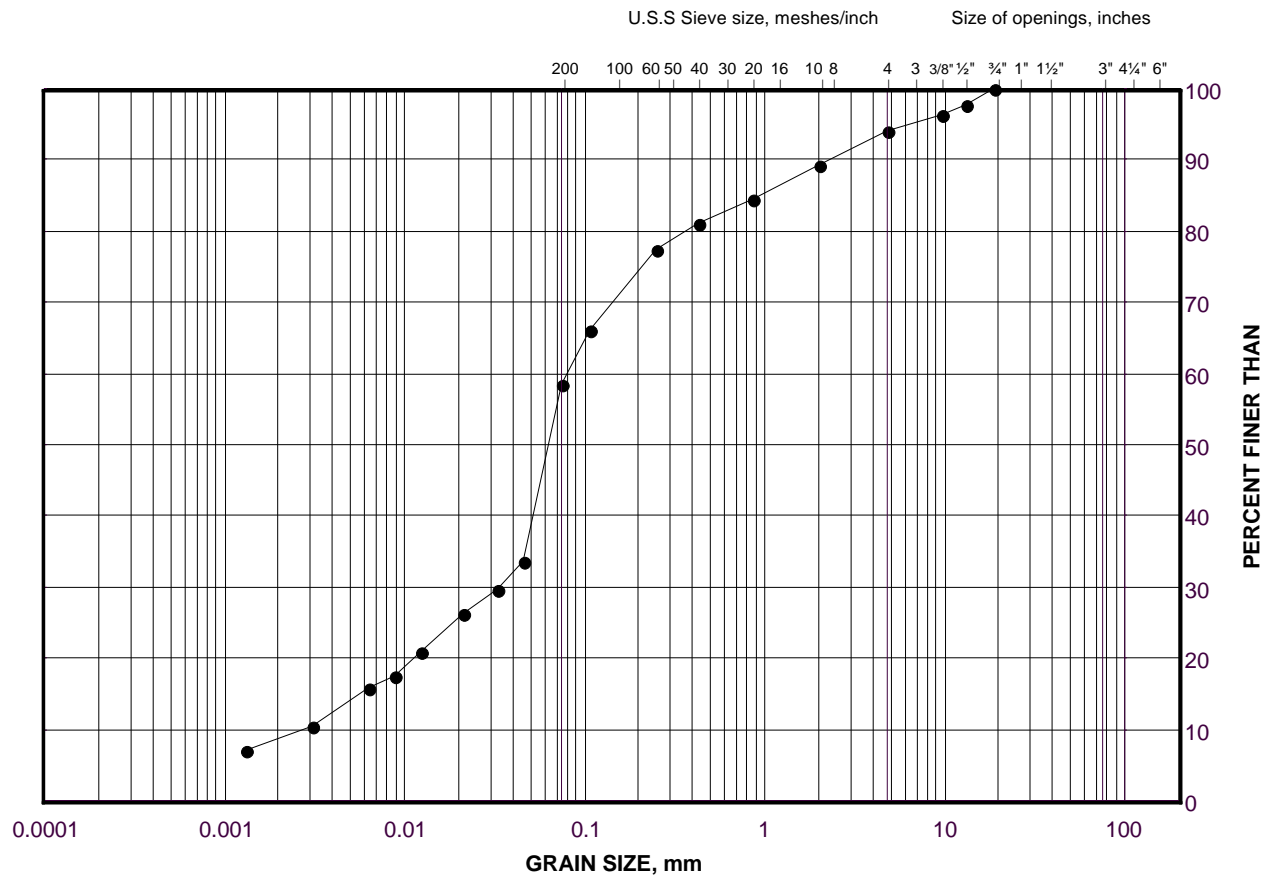
Checked By: NK

GRAIN SIZE DISTRIBUTION

Silt and Sand (Till) (Interlayer)

Retaining Wall 1

FIGURE A7



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	DC-10	11	159.5

Project Number: 10-1111-0211

Checked By: NK

Golder Associates

Date: 11-Mar-15



APPENDIX B

Retaining Wall 2, Highway 401 - STA. 15+985 to STA. 16+142 **Record of Borehole Sheets and Laboratory Test Results**

PROJECT 10-1111-0211			RECORD OF BOREHOLE No SC8-3			SHEET 1 OF 1			METRIC								
G.W.P. 2152-01-00			LOCATION N 4830771.3 ; E 287144.6			ORIGINATED BY QC											
DIST Central HWY 401			BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers			COMPILED BY MP											
DATUM GEODETIC			DATE January 11, 2015			CHECKED BY KJB											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
164.7	GROUND SURFACE							20	40	60	80	100					
164.7	TOPSOIL		1	SS	6												
	SILTY CLAY, trace to some sand, trace to some gravel, trace rootlets to 0.6 m Firm to very stiff dark to light brown Moist		2	SS	23												
			3	SS	13												
162.5	CLAYEY SILT, trace to some sand, trace to some gravel (TILL) Soft to stiff Grey Moist		4	SS	4												
			5	SS	3												
			6	SS	3												
			7	SS	9												
159.1	Gravelly SILTY SAND to SAND and GRAVEL, trace clay, grinding of augers on inferred cobble/boulder at 6.6 m depth (TILL) Very dense Grey Moist to Wet		8	SS	98/0.29												
			9	SS	92												
156.6	END OF BOREHOLE																
8.1	NOTES: 1. Water level in open borehole measured at a depth of 4.6 m below ground surface (Elev. 160.1 m) upon completion of drilling.																

PROJECT		10-1111-0211		RECORD OF BOREHOLE No SC8-4		SHEET 1 OF 1		METRIC						
G.W.P.		2152-01-00		LOCATION		N 4830812.3 ; E 287208.7		ORIGINATED BY						
DIST		Central HWY 401		BOREHOLE TYPE		108 mm I.D. Hollow Stem Augers		COMPILED BY						
DATUM		GEODETIC		DATE		January 12, 2015		CHECKED BY						
								KJB						
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
164.5	GROUND SURFACE							20 40 60 80 100	20 40 60 80 100	10 20 30				
0.9	TOPSOIL													
	SILTY CLAY to CLAYEY SILT, trace to some sand, trace to some gravel, trace organics / rootlets throughout, oxidation staining Firm Brown Moist to Wet		1	SS	6									
			2	SS	6									
			3	SS	6									
162.2	CLAYEY SILT with SAND, trace to some gravel (TILL) Firm to stiff Grey Moist		4	SS	11									
			5	SS	5									
160.9	CLAYEY SILT with SAND, trace gravel Soft to firm Grey Moist		6	SS	3									
			7	SS	8									
158.9	SAND and GRAVEL to gravelly SAND, trace to some silt, trace clay (TILL) Very dense Grey Moist to wet		8	SS	53									
			9	SS	66									
			10	SS	93									
153.7	END OF BOREHOLE		11	SS	50/0.13									
10.8	NOTES :													
	1. Water level in open borehole measured at a depth of 1.8 m below ground surface (Elev. 162.7 m) upon completion of drilling.													

PROJECT		2150-01-00		LOCATION		N 4830851.3 ; E 287283.0		ORIGINATED BY		CS							
DIST		HWY 401- 403 W Ramp		BOREHOLE TYPE		108 mm I.D. Hollow Stem Augers, NW Casing with Tricone, Wash Boring		COMPILED BY		CC/TVA							
DATUM		Geodetic		DATE		May 8, 2012		CHECKED BY		KJB							
SOIL PROFILE				SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	SHEAR STRENGTH kPa											
163.8	GROUND SURFACE					20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED				WATER CONTENT (%)							
0.0	TOPSOIL					20 40 60 80 100				10 20 30							
0.2	Clayey silt, some gravel, some sand, containing organics and rootlets to a depth of 0.8 m (FILL) Firm Brown Moist		1	SS	5												
			2	SS	6												
162.3																	
1.5	CLAYEY SILT with SAND, trace to some gravel Firm to stiff Grey Moist		3	SS	9											8 28 47 17	
			4	SS	8												
			5	SS	5												
158.6																	
5.2	Sandy SILT and GRAVEL, trace clay (TILL) Dense to very dense Grey Moist		6	SS	45											33 27 36 4	
			7	SS	56												
156.6																	
7.2	SAND and GRAVEL, some silt, trace clay, containing cobbles (TILL) Very dense Grey Wet		8	SS	73											43 38 15 4	
			9	SS	100/0.2												
153.4																	
10.4	END OF BOREHOLE SPOON AND CASING REFUSAL ** Artesian Conditions - see Note 2. NOTES: 1. Unable to advance casing below a depth of 10.4 m below ground surface (Elev. 153.4 m). 2. Water flowing from top of casing at the end of work day on May 8, 2012, when bottom of casing at a depth of 10.4 m below ground surface (Elev. 153.4 m).																

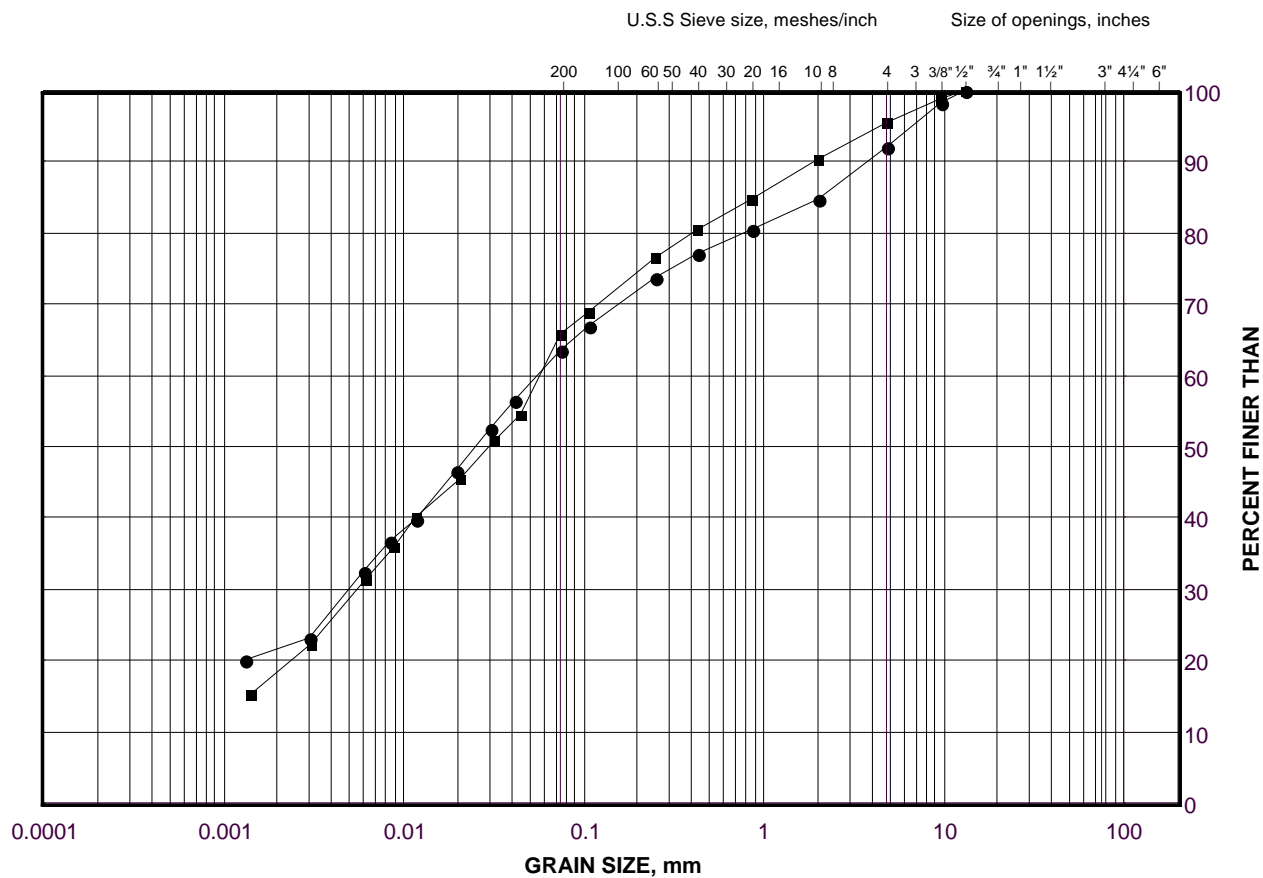
+³, ×³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

PROJECT		10-1111-0211		RECORD OF BOREHOLE No FC-4		SHEET 1 OF 1		METRIC	
2150-01-00		LOCATION		N 4830837.9 ; E 287276.8		ORIGINATED BY		CS	
DIST		HWY 401- 403 W Ramp		BOREHOLE TYPE		108 mm I.D. Hollow Stem Augers		COMPILED BY	
CC/TVA		DATE		May 9, 2012		CHECKED BY		KJB	
DYNAMIC CONE PENETRATION RESISTANCE PLOT		SHEAR STRENGTH kPa		WATER CONTENT (%)		UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)	
<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 20 40 60 80 100 </div>		<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 10 20 30 </div>		<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 10 20 30 </div>		<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 10 20 30 </div>		<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 10 20 30 </div>	
<div style="display: flex; justify-content: space-between;"> ○ UNCONFINED + FIELD VANE </div>		<div style="display: flex; justify-content: space-between;"> ● QUICK TRIAXIAL × REMOULDED </div>		<div style="display: flex; justify-content: space-between;"> ○ UNCONFINED + FIELD VANE </div>		<div style="display: flex; justify-content: space-between;"> ● QUICK TRIAXIAL × REMOULDED </div>		<div style="display: flex; justify-content: space-between;"> ○ UNCONFINED + FIELD VANE </div>	
<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 10 20 30 </div>		<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 10 20 30 </div>		<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 10 20 30 </div>		<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 10 20 30 </div>		<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 10 20 30 </div>	
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS		ELEVATION SCALE			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES				
164.4	GROUND SURFACE								
0.0	TOPSOIL								
0.2	Clayey silt, some sand, containing rootlets (FILL) Firm Brown Moist		1	SS	5	164			
			2	SS	5	163			
162.9						163			
1.5	CLAYEY SILT, trace to some gravel, trace to some sand, containing rootlets to a depth of 2.2 m Very soft to stiff Brown Moist to wet		3	SS	11	162			
			4	SS	3	161			
						161			
			5	SS	WH	160			
						159			
159.1	Auger grinding at a depth of 5.2 m								
5.3	SAND and SILT, some gravel, trace to some clay (TILL)		6	SS	100/0.3	159			
158.6	Very dense Grey Wet								
5.8	END OF BOREHOLE								
NOTES: 1. Water level inside augers at a depth of 0.3 m below ground surface (Elev. 164.1 m) when advanced augers to a depth of 4.6 m below ground surface (Elev. 159.8 m). 2. Water level inside augers at a depth of 2.2 m below ground surface (Elev. 162.2 m) upon completion of sampling. 3. Water level in open borehole at a depth of 2.0 m below ground surface (Elev. 162.4 m), measured one hour upon completion of drilling.									

GRAIN SIZE DISTRIBUTION

Clayey Silt with Sand to Clayey Silt
Retaining Wall 2

FIGURE B1



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

LEGEND

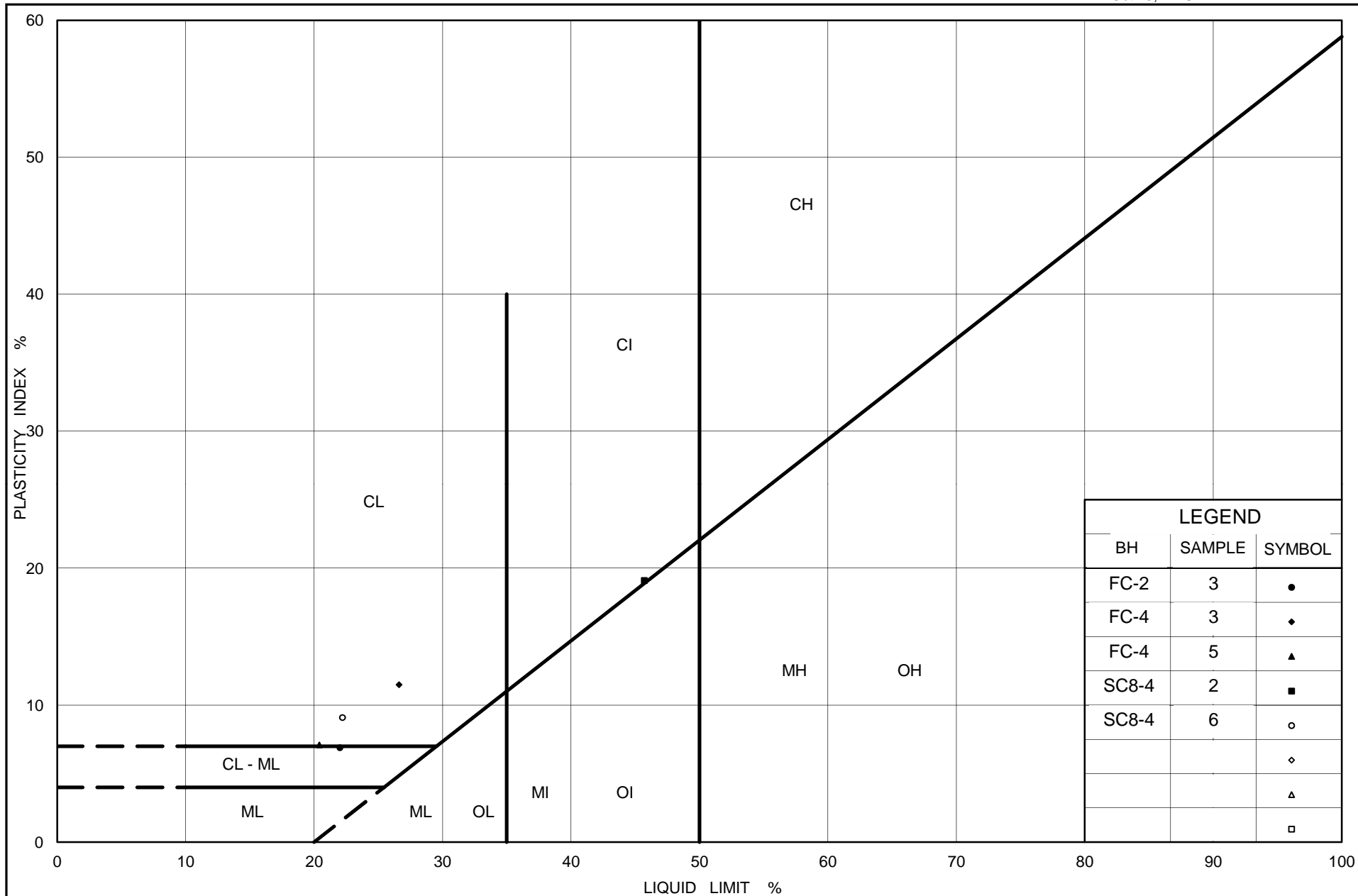
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	FC-2	3	162.0
■	SC8-4	6	160.4

Project Number: 10-1111-0211

Checked By: NK

Golder Associates

Date: 12-Jun-15



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PLASTICITY CHART Clayey Silt to Silty Clay Retaining Wall 2

Figure No. B2

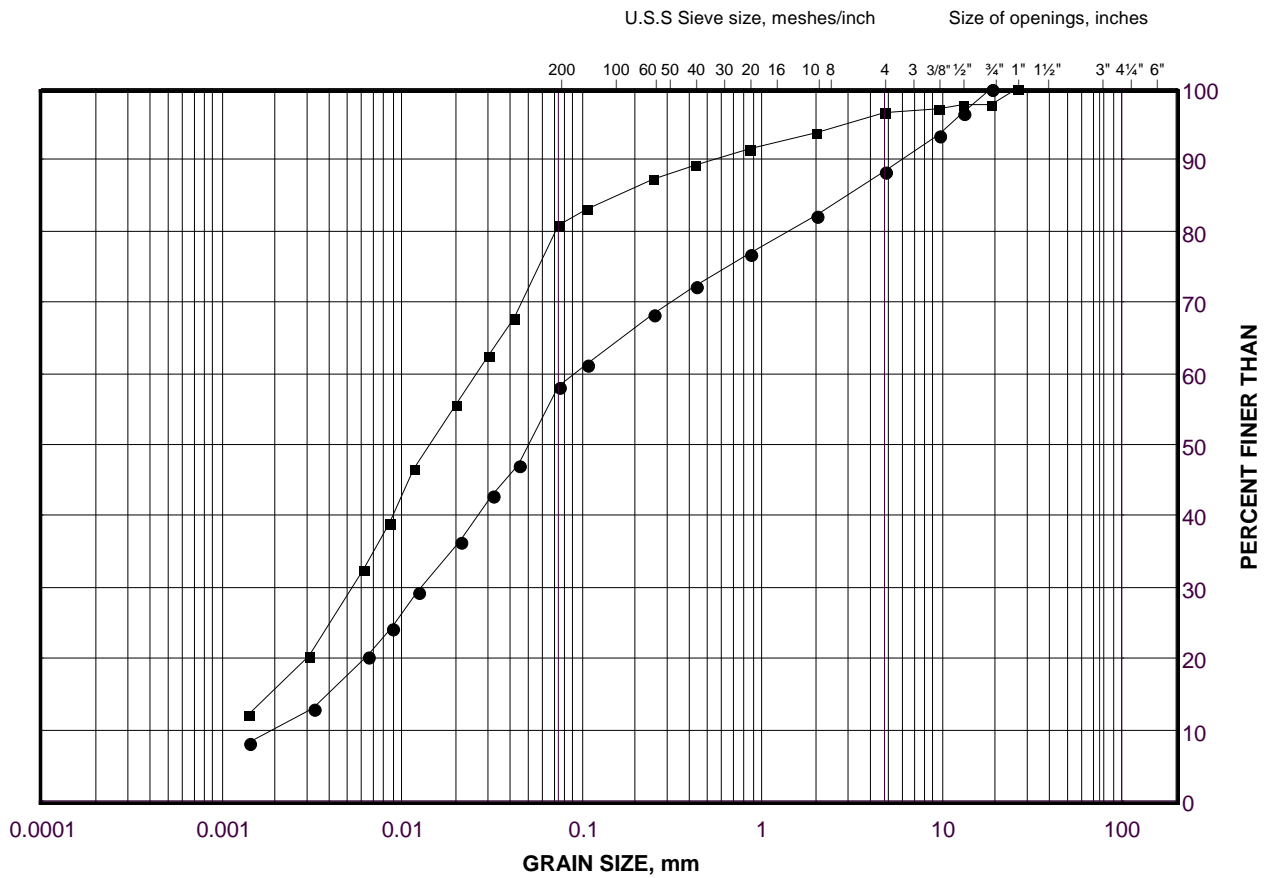
Project No. 10-1111-0211

Checked By: NK

GRAIN SIZE DISTRIBUTION

Clayey Silt (Till)
Retaining Wall 2

FIGURE B3



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

LEGEND

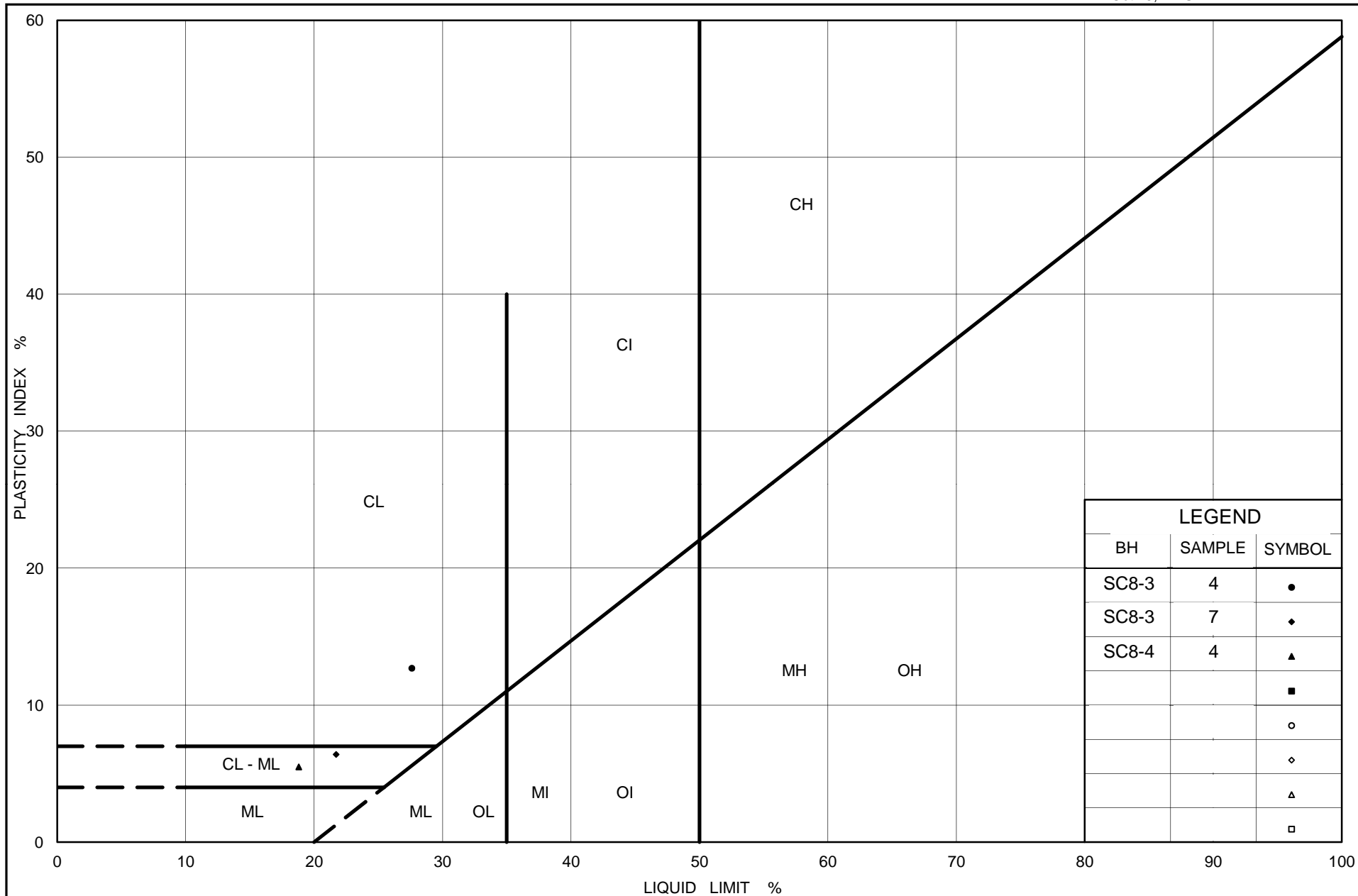
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	SC8-4	4	161.9
■	SC8-3	7	159.8

Project Number: 10-1111-0211

Checked By: NK

Golder Associates

Date: 12-Jun-15



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PLASTICITY CHART Clayey Silt (Till) Retaining Wall 2

Figure No. B4

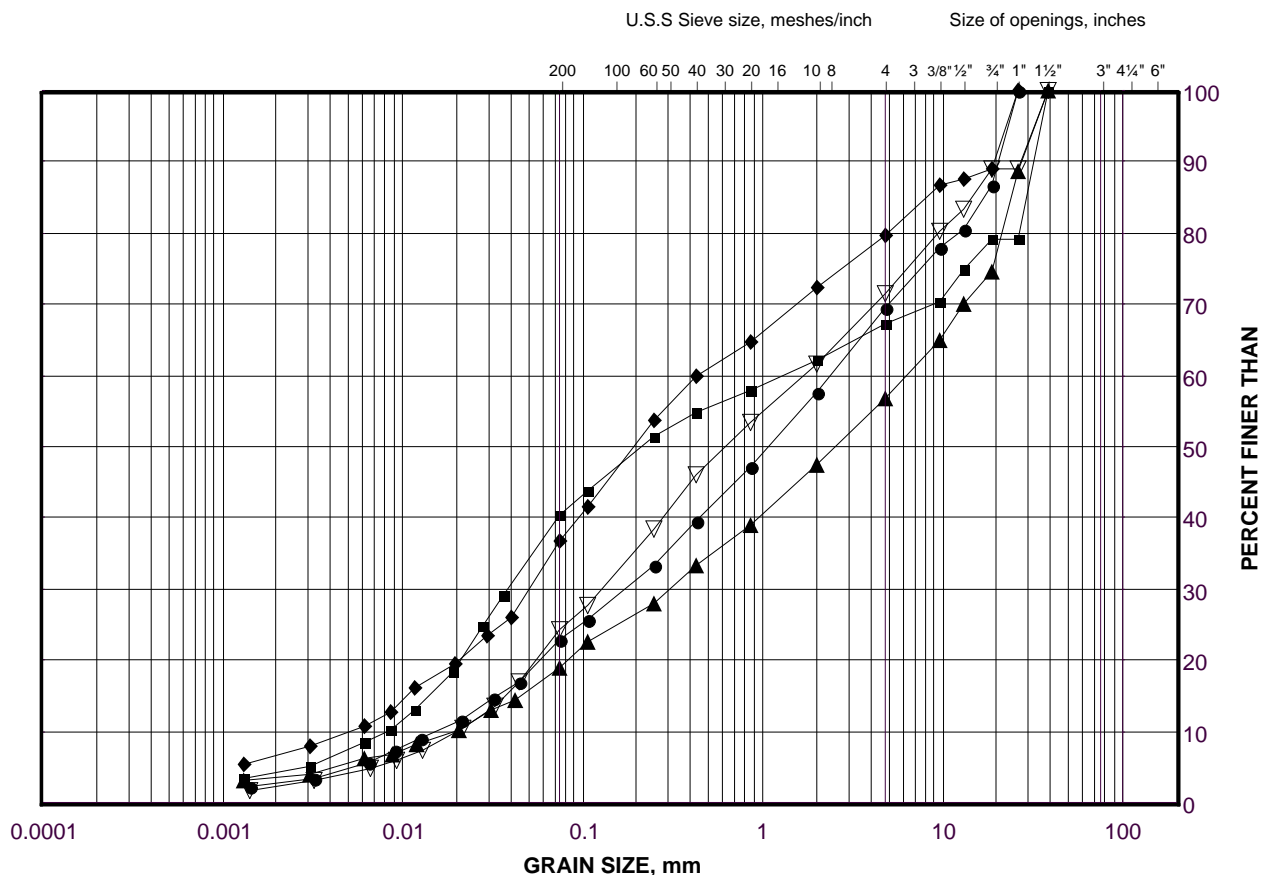
Project No. 10-1111-0211

Checked By: NK

GRAIN SIZE DISTRIBUTION

Gravelly Silty Sand to Sand and Gravel (Till)
Retaining Wall 2

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)
●	SC8-4	10	155.1
■	FC-2	6	158.2
◆	FC-4	6	158.8
▲	FC-2	8	155.9
▽	SC8-3	8	158.3

Project Number: 10-1111-0211

Checked By: NK

Golder Associates

Date: 26-May-15



APPENDIX C

Retaining Wall 3, Highway 401 – STA. 16+179 to STA. 16+250 Record of Borehole Sheets and Laboratory Test Results

PROJECT 10-1111-0211		RECORD OF BOREHOLE No FC-1		SHEET 1 OF 2		METRIC	
2150-01-00		LOCATION N 4830867.9 ; E 287311.4		ORIGINATED BY CS			
DIST _____ HWY 401- 403 W Ramp		BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, HW Casing and HQ Coring		COMPILED BY CC/TVA			
DATUM Geodetic		DATE May 1 and 3, 2012		CHECKED BY KJB			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS ▽**	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
								20 40 60 80 100	20 40 60 80 100	W _p	W	W _L		
163.7	GROUND SURFACE													
0.0	TOPSOIL													
0.2	Clayey silt, some gravel, some sand, containing rootlets (FILL) Soft Brown and grey Moist		1	SS	3									
			2	SS	3									
162.2														
1.5	CLAYEY SILT with SAND, trace to some gravel Soft to stiff Brown Moist		3A	SS	5									
			3B											
			4	SS	13									
			5	SS	3									
			6	SS	8									
159.2														
4.5	SAND and SILT, some gravel, trace to some clay (TILL) Very dense Grey Wet		7	SS	52									
			8	SS	68									
156.5														
7.2	Silty SAND and GRAVEL, trace to some clay (TILL) Very dense Grey Wet		9	SS	86/0.25									
			10	SS	50/0.1									
			11	SS	73/0.15									
			12	SS	76/0.2									
150.9														
12.8	SAND and GRAVEL, trace silt, containing cobbles and shale fragments (TILL) Very dense Grey Wet		13	SS	72/0.15									
			1	RC	REC 45%									
			2	RC	REC 30%									
			3	RC	REC 8%									

Continued Next Page

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

GTA-MTO 001 1011110211.GPJ GAL-GTA.GDT 2/17/15 PR

PROJECT		RECORD OF BOREHOLE No FC-1				SHEET 2 OF 2		METRIC						
10-1111-0211		LOCATION N 4830867.9 ; E 287311.4				ORIGINATED BY CS								
2150-01-00		BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, HW Casing and HQ Coring				COMPILED BY CC/TVA								
DIST _____ HWY 401- 403 W Ramp		DATE May 1 and 3, 2012				CHECKED BY KJB								
DATUM Geodetic														
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20	40	60	80			100
148.2	END OF BOREHOLE													
147.9	END OF DCPT Refusal to Further Penetration (163 Blows/0.3 m)													
15.9	<p>** Artesian Conditions - see Note 3.</p> <p>NOTES:</p> <p>1. Auger refusal at a depth of 12.9 m, cored through overburden soil and cobbles using HQ size core barrel to a depth of 15.5 m. Advanced Dynamic Cone Penetration Test (DCPT) from depths of 15.5 m to 15.9 m.</p> <p>2. Water level inside augers at a depth of 1.8 m below ground surface (Elev. 161.9 m), measured at the end of work day on May 1, 2012, when augers advanced to a depth of 12.8 m below ground surface (Elev. 150.9 m).</p> <p>3. Water level inside casing at 1.0 m above ground surface (Elev. 164.7 m), measured at start of work day on May 2, 2012, when bottom of casing at a depth of 13.0 m below ground surface (Elev. 150.7 m).</p>													

PROJECT		2150-01-00		LOCATION		N 4830871.4 ; E 287325.3		ORIGINATED BY		CS					
DIST		HWY 401- 403 W Ramp		BOREHOLE TYPE		108 mm I.D. Hollow Stem Augers		COMPILED BY		CC/TVA					
DATUM		Geodetic		DATE		May 7, 2012		CHECKED BY		KJB					
SOIL PROFILE				SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	SHEAR STRENGTH kPa									
165.9	GROUND SURFACE														
0.0	Clayey silt, some sand, trace gravel, containing rootlets (FILL) Firm Brown Moist		1	SS	6										
165.0															
0.9	CLAYEY SILT, some sand, some gravel Stiff to very stiff Brown to grey Moist		2	SS	16										
			3	SS	22										
			4	SS	14										
			5	SS	8										
162.2															
3.7	SILT, trace to some sand, trace to some clay, trace gravel Loose Grey Wet		6	SS	4										
161.4															
4.5	CLAYEY SILT, some gravel, some sand Firm Grey Moist		7	SS	6										
160.7															
5.2	END OF BOREHOLE														
NOTE: 1. Open borehole dry upon completion of drilling.															

PROJECT 10-1111-0211		RECORD OF BOREHOLE No FC-13		SHEET 1 OF 1		METRIC															
2150-01-00		LOCATION N 4830881.8 ; E 287336.5		ORIGINATED BY SB																	
DIST _____ HWY 401- 403 W Ramp		BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, NW Casing with Tricone, Wash Boring		COMPILED BY BM/TVA																	
DATUM Geodetic		DATE September 4, 2012		CHECKED BY KJB																	
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ			GR SA SI CL		
167.1 0.0	GROUND SURFACE CLAYEY SILT with SAND, trace to some gravel, containing rootlets to a depth of 0.6 m Stiff to very stiff Brown to grey Moist		1	SS	11		167						○								
			2	SS	15		166														
			3	SS	11		165						○			5 30 44 21					
			4	SS	19		164						○								
			5	SS	12		163						○								
			6	SS	10		162						○			13 29 35 23					
			7	SS	6		161						○								
160.5 6.6	SAND and SILT, trace to some clay, trace gravel (TILL) Dense Grey Moist to wet		8A	SS	4		160														
			8B				159														
			9	SS	35		158						○			2 37 54 7					
158.4 8.7	Silty SAND, some gravel, trace clay (TILL) Very dense Wet		10	SS	50/0.03		157														
156.3 10.8	END OF BOREHOLE BROKEN CASING NOTE: 1. Unable to advance borehole beyond a depth of 10.8 m as part of the casing broke while penetrating through the very dense overburden. Backfilled borehole, moved drilling 1.5 m west and advanced Borehole FC-13A, and continued sampling below 10.8 m depth.		11	SS	60/0.15								○								

GTA-MTO 001 1011110211.GPJ GAL-GTA.GDT 2/17/15 PR

Continued Next Page

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

PROJECT		RECORD OF BOREHOLE				No FC-13A		SHEET 2 OF 2		METRIC																						
10-1111-0211		2150-01-00		LOCATION		N 4830881.8 ; E 287335.0		ORIGINATED BY		SB																						
DIST		HWY 401- 403 W Ramp		BOREHOLE TYPE		108 mm I.D. Hollow Stem Augers, NW Casing with Tricone, Wash Boring		COMPILED BY		BM/TVA																						
DATUM		Geodetic		DATE		September 4 to 6, 2012		CHECKED BY		KJB																						
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)															
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)														
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100																				
	Gravelly SAND, some silt, trace clay (TILL) Very dense Wet		14	SS	50/0.08		152																									
							151																									
			15	SS	65/0.03		150																									
149.3 17.8	END OF BOREHOLE																															
<p>* SPT "N" value may have been influenced by difficulties advancing augers / wash boring at this depth.</p> <p>** Artesian Conditions - see Note 4.</p> <p>NOTES:</p> <p>1. Borehole FC-13A augered from ground surface to a depth of 7.6 m; then switched to 'NW' casing method. Lost casing shoe at depth of 11.9 m while penetrating through the very dense overburden, then switched back to auger with difficulties advancing auger to 12.2 m depth, then completed borehole using NW Casing.</p> <p>2. Water level inside augers at a depth of 2.7 m below ground surface (Elevation 164.4 m), measured at end of work day (at 4:16 pm) on Sept. 05, 2012, when augers advanced to a depth of 12.2 m below ground surface (Elev. 154.9 m).</p> <p>3. Water level inside augers at a depth of 2.1 m below ground surface (Elevation 165.0 m), measured at start of work day (at 7:00 am) on Sept. 06, 2012, when augers advanced to a depth of 12.8 m below ground surface (Elev. 154.3 m).</p> <p>4. Water flowing from top of casing when advanced to a depth of 17.8 m (Elev. 149.3 m). Stacked up casing to about 5.0 m above ground surface to monitor the hydrostatic head upon completion of drilling operations on September 6, 2012. The recorded water level readings are:</p> <table border="1" style="width:100%; border-collapse: collapse;"> <thead> <tr> <th>Time</th> <th>Depth (m) to W.L.</th> <th>W.L. Elev(m)</th> </tr> </thead> <tbody> <tr> <td>12:30 pm</td> <td>-0.6</td> <td>167.7</td> </tr> <tr> <td>12:36 pm</td> <td>-2.1</td> <td>169.2</td> </tr> <tr> <td>12:41 pm</td> <td>-3.5</td> <td>170.6</td> </tr> <tr> <td>1:30 pm</td> <td>-4.8</td> <td>171.9</td> </tr> </tbody> </table>																		Time	Depth (m) to W.L.	W.L. Elev(m)	12:30 pm	-0.6	167.7	12:36 pm	-2.1	169.2	12:41 pm	-3.5	170.6	1:30 pm	-4.8	171.9
Time	Depth (m) to W.L.	W.L. Elev(m)																														
12:30 pm	-0.6	167.7																														
12:36 pm	-2.1	169.2																														
12:41 pm	-3.5	170.6																														
1:30 pm	-4.8	171.9																														

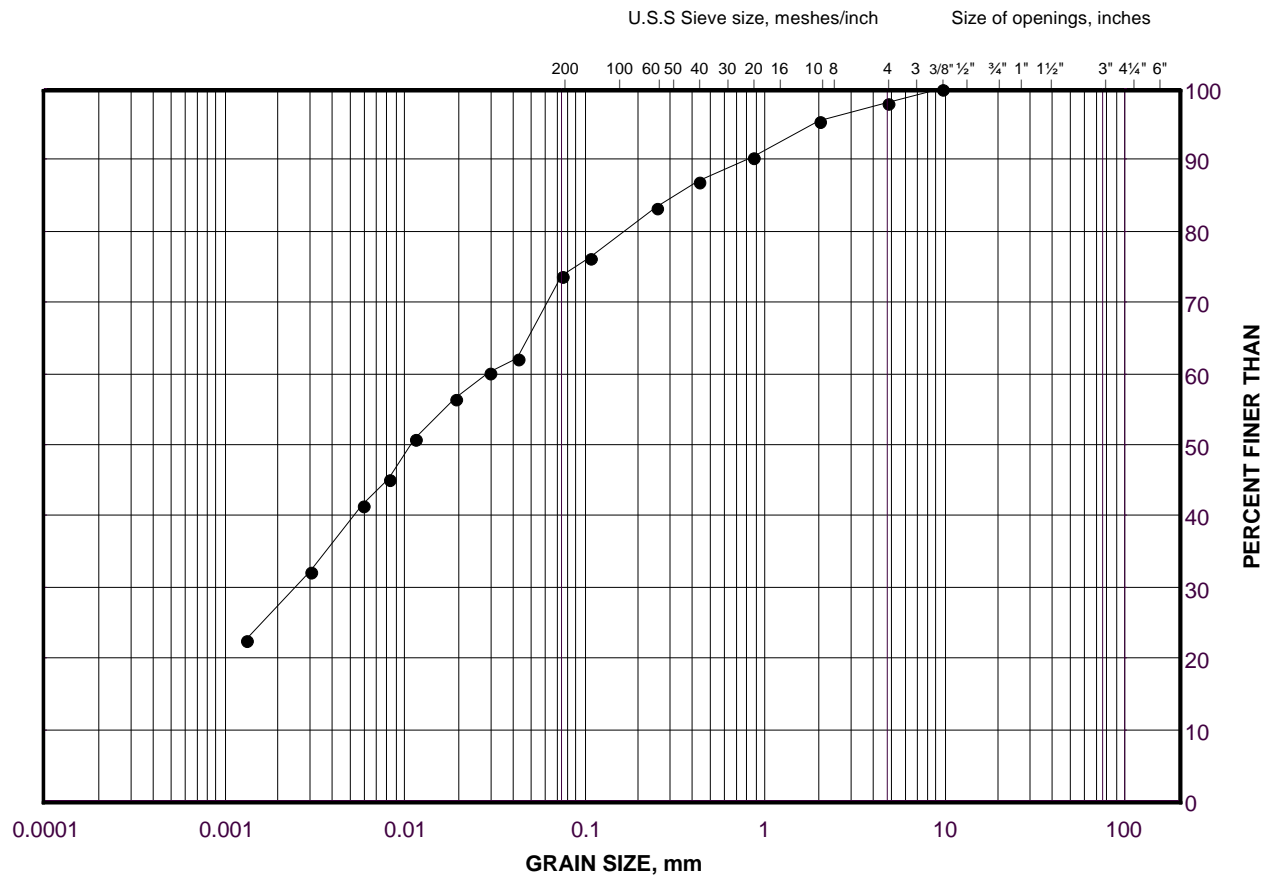
PROJECT		10-1111-0211		RECORD OF BOREHOLE No DC-12		SHEET 1 OF 1		METRIC							
2150-01-00		LOCATION		N 4830923.2 ; E 287393.6		ORIGINATED BY		SB							
DIST		HWY 401- 403 W Ramp		BOREHOLE TYPE		57 mm I.D. Hollow Stem Augers		COMPILED BY							
BM		DATE		September 12, 2012		CHECKED BY		TVA							
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS		ELEVATION SCALE		DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES										
176.3	GROUND SURFACE														
0.0	ASPHALT														
0.2	Silty sand, trace clay, trace gravel (FILL)		1	SS	12										
175.5	Compact Brown Moist		2	SS	6										
0.8	Clayey silt, sandy, trace gravel (FILL)		3	SS	5										
	Firm to stiff Grey Moist		4	SS	10										
			5	SS	6										
			6	SS	6										
			7	SS	7										
170.5	CLAYEY SILT, sandy, trace gravel (TILL)		8	SS	25										
5.8	Very stiff to hard Brown Moist		9	SS	32										
167.5	CLAYEY SILT, trace sand, trace gravel		10	SS	10										
8.8	Stiff Grey Moist														
165.9	Silty SAND, trace clay, trace gravel (TILL)		11	SS	23										
10.4	Compact Grey Moist														
164.4	CLAYEY SILT, trace sand, trace gravel (TILL)		12	SS	34										
11.9	Hard Grey Moist														
163.5	END OF BOREHOLE														
12.8	NOTES:														
	1. Borehole open to a depth of 10.0 m below ground surface (Elev. 166.3 m).														
	2. Open borehole dry upon completion of drilling.														

GRAIN SIZE DISTRIBUTION

Clayey Silt (Fill)

Retaining Wall 3

FIGURE C1



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

LEGEND

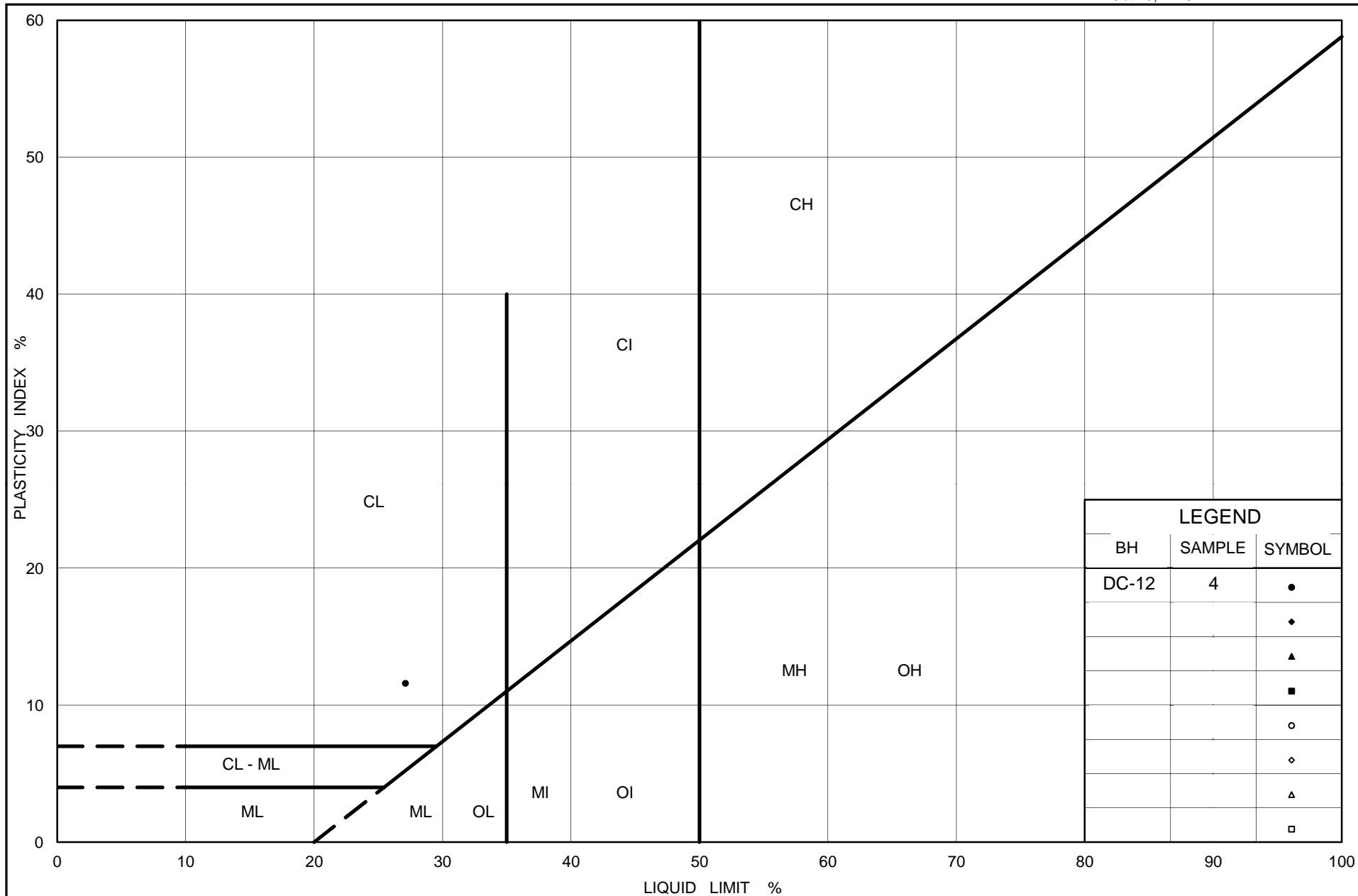
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	DC-12	4	173.7

Project Number: 10-1111-0211

Checked By: NK

Golder Associates

Date: 11-Mar-15



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PLASTICITY CHART Clayey Silt (Fill) Retaining Wall 3

Figure No. C2

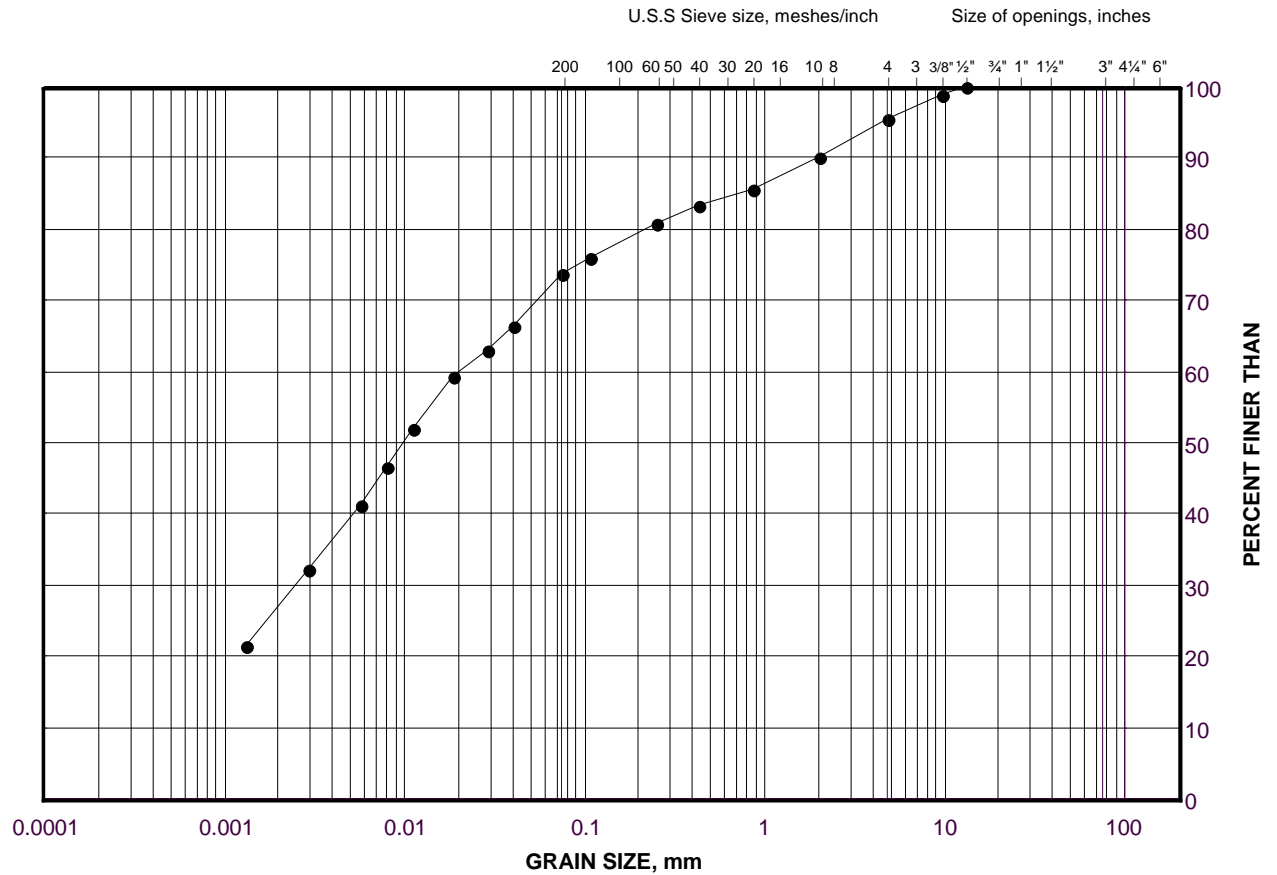
Project No. 10-1111-0211

Checked By: NK

GRAIN SIZE DISTRIBUTION

Sandy Clayey Silt (Till) (Upper Deposit)
Retaining Wall 3

FIGURE C3



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

LEGEND

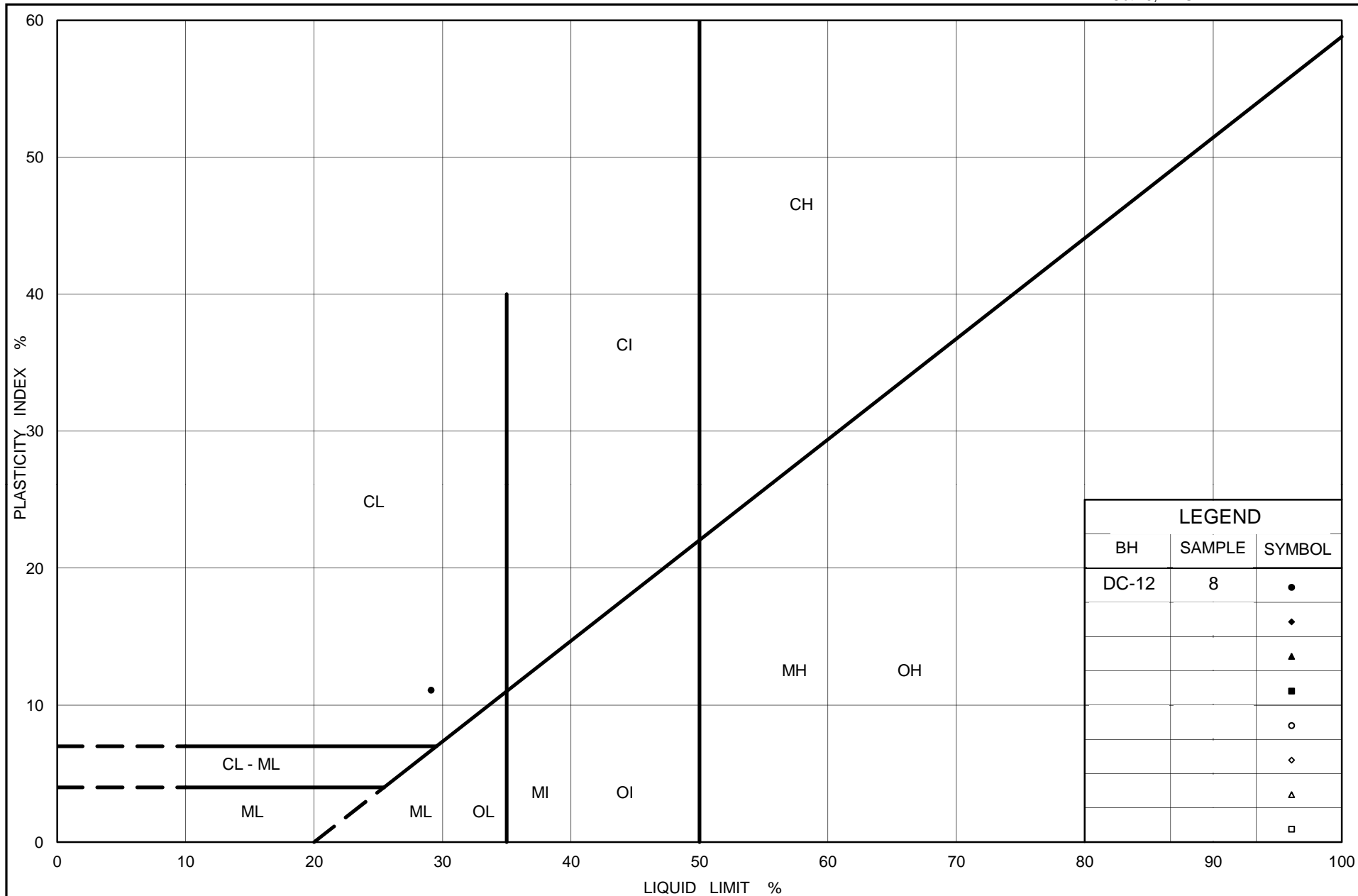
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	DC-12	8	169.9

Project Number: 10-1111-0211

Checked By: NK

Golder Associates

Date: 11-Mar-15



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PLASTICITY CHART Sandy Clayey Silt (Till) (Upper Deposit) Retaining Wall 3

Figure No. C4

Project No. 10-1111-0211

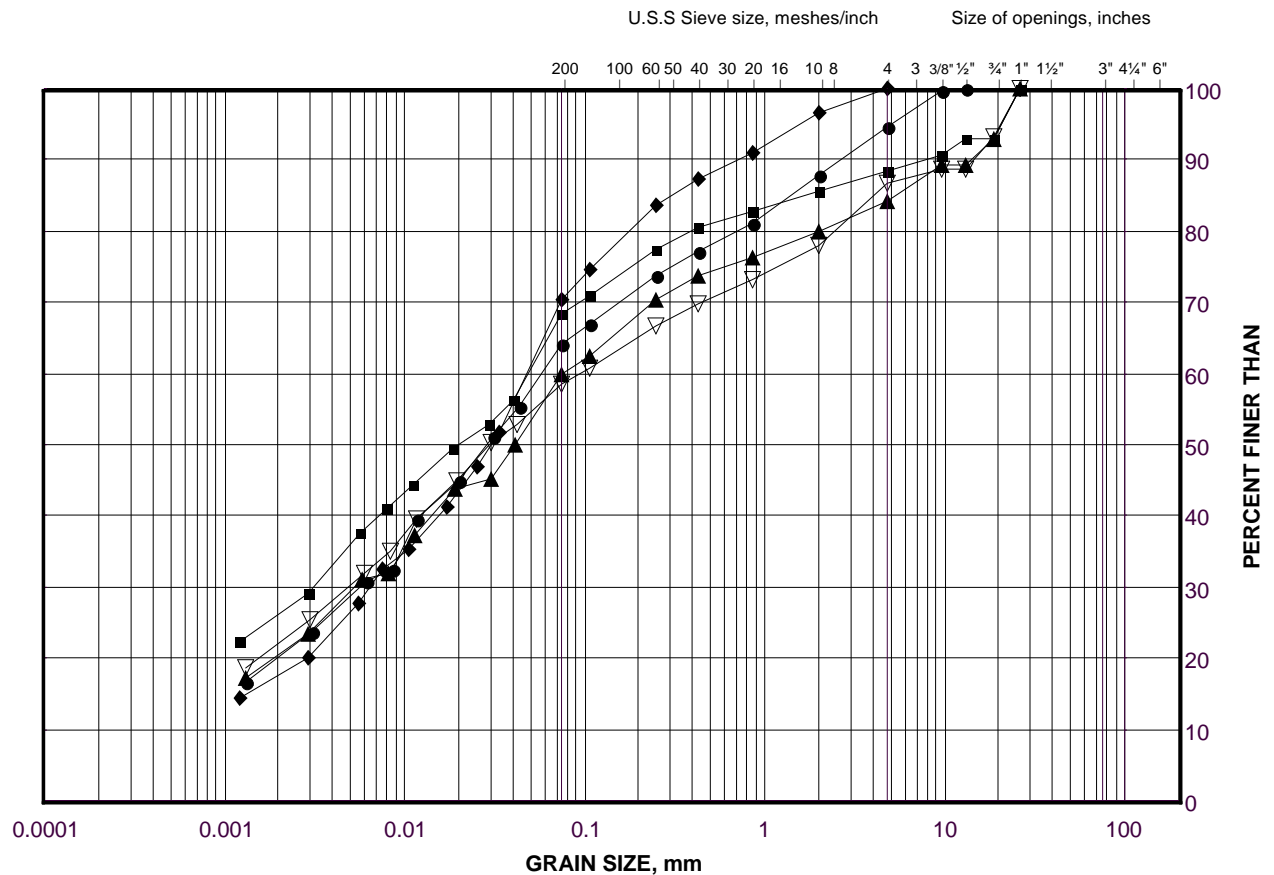
Checked By: NK

GRAIN SIZE DISTRIBUTION

Clayey Silt with Sand to Clayey Silt

Retaining Wall 3

FIGURE C5



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

LEGEND

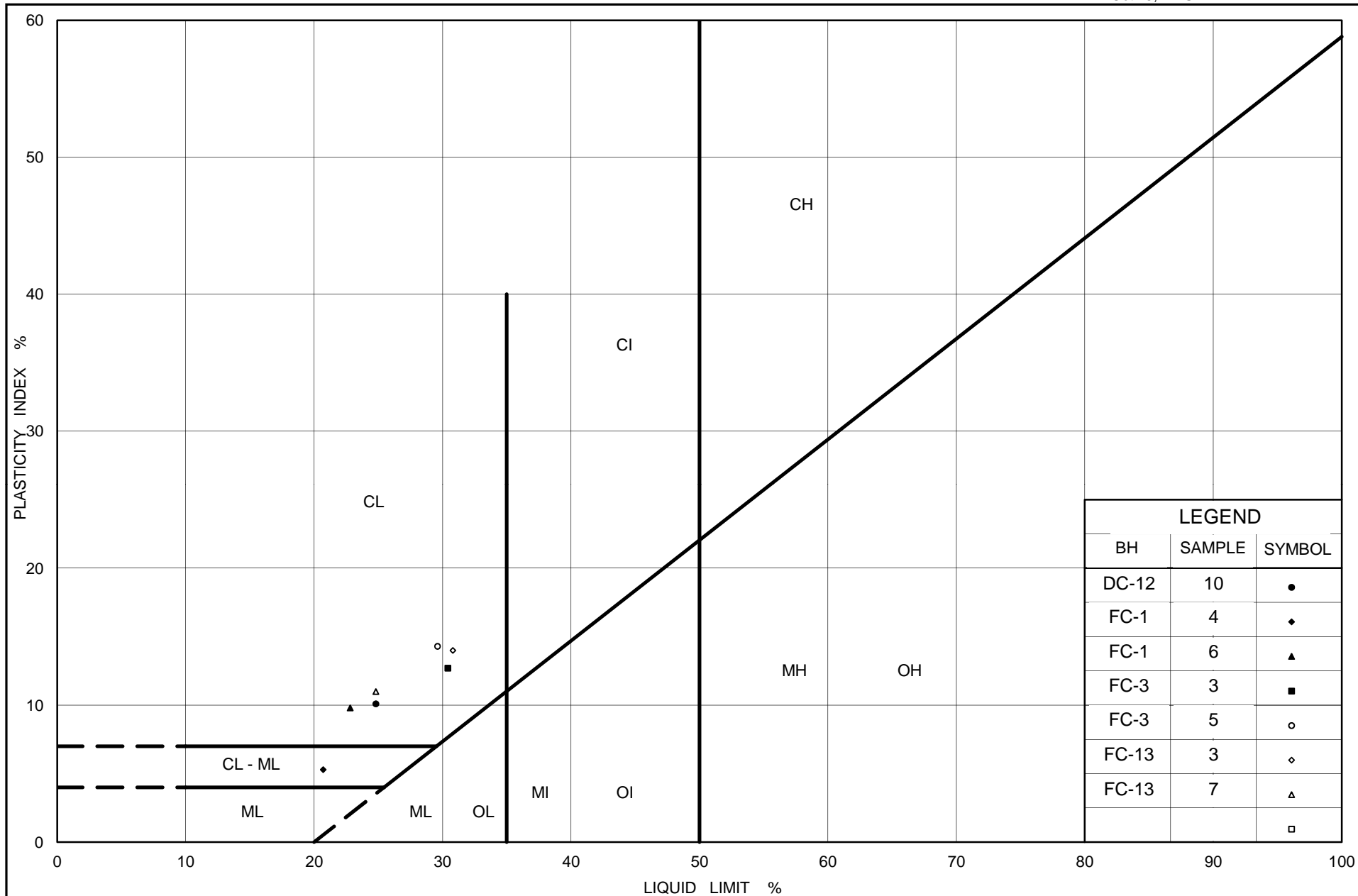
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	FC-13	3	165.3
■	FC-3	3	164.1
◆	FC-1	4	161.1
▲	FC-1	6	159.6
▽	FC-13	7	162.2

Project Number: 10-1111-0211

Checked By: NK

Golder Associates

Date: 24-Mar-15



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PLASTICITY CHART Clayey Silt with Sand to Clayey Silt Retaining Wall 3

Figure No. C6

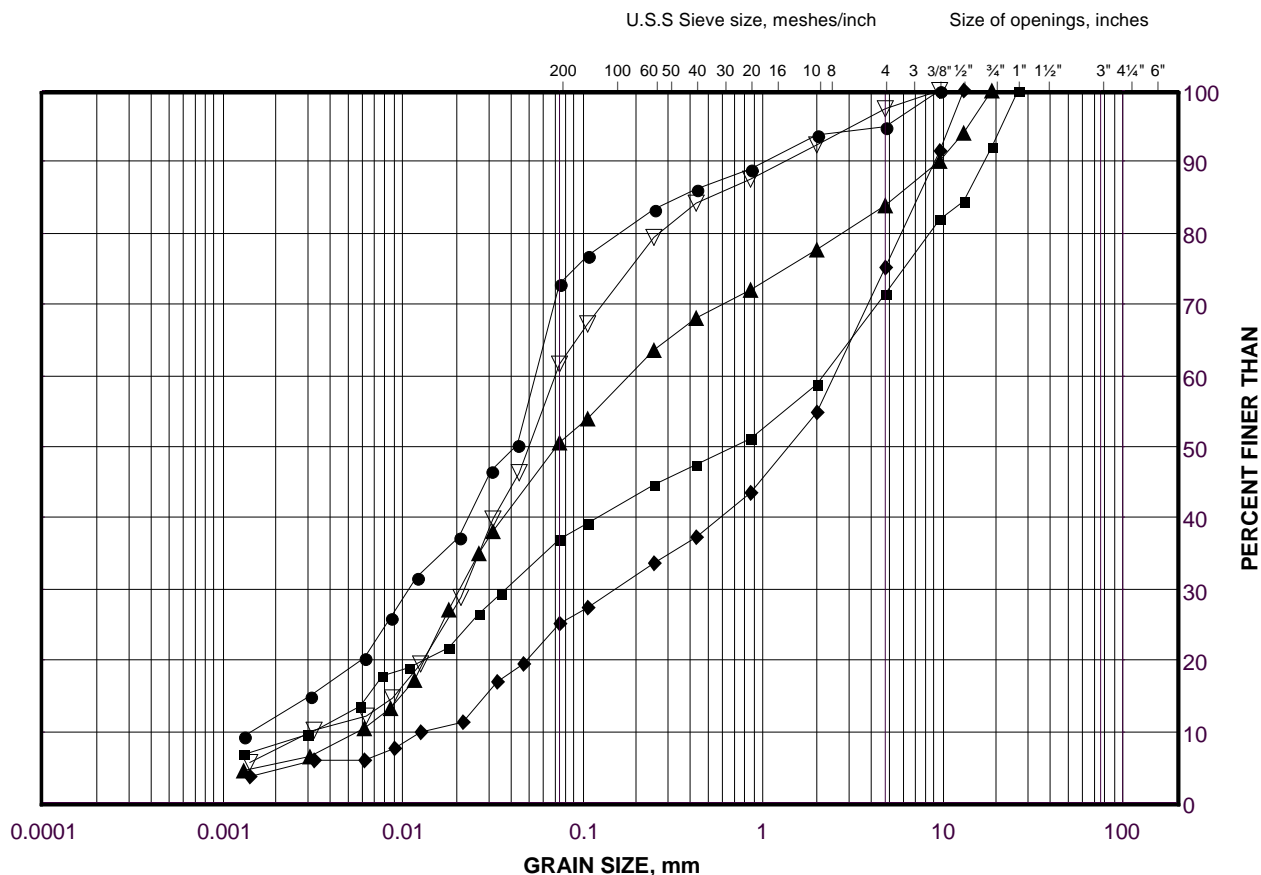
Project No. 10-1111-0211

Checked By: NK

GRAIN SIZE DISTRIBUTION

Sandy Silt to Silt and Sand to Gravelly Sand (Till)
Retaining Wall 3

FIGURE C7



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	DC-12	11	165.4
■	FC-1	12	151.9
◆	FC-13A	13	153.3
▲	FC-1	7	158.8
▽	FC-13	9	159.2

Project Number: 10-1111-0211

Checked By: NK

Golder Associates

Date: 24-Mar-15



APPENDIX D

Retaining Wall 4, Highway 401 – STA. 16+315 to STA. 16+480 Record of Borehole Sheets and Laboratory Test Results

PROJECT		10-1111-0211		RECORD OF BOREHOLE No RW-1		SHEET 1 OF 1		METRIC					
G.W.P.		2152-01-00		LOCATION		N 4831002.0 ; E 287527.6		ORIGINATED BY					
DIST		Central HWY 401		BOREHOLE TYPE		150 mm O.D. Solid Stem Augers		COMPILED BY					
DATUM		GEODETIC		DATE		July 7, 2014		CHECKED BY					
								NK					
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID UNIT REMARKS				
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	W _p W W _L	WATER CONTENT (%)	γ	GR SA SI CL
175.2	0.0	GROUND SURFACE											
	0.2	TOPSOIL											
	0.2	Clayey silt, sandy, trace gravel, trace rootlets (FILL)		1	AS	-		175					
	174.4	Brown Moist											
	0.8	Sandy CLAYEY SILT, trace to some gravel, trace rootlets to a depth of 1.1 m (TILL)		2	SS	21		174					
		Stiff to hard											
		Brown Moist		3	SS	30		173					5 28 45 22
		Auger grinding		4	SS	35							
				5A	SS	32		172					
		Becoming grey		5B	SS								
				6	SS	11		171					
				7	SS	10		170					
	169.6	SILT and SAND, trace to some clay											
	5.6	Compact Grey Wet		8	SS	28		169					0 34 60 6
	168.0	CLAYEY SILT with SAND, some gravel (TILL)						168					
	7.2	Very stiff Moist		9	SS	24		167					
	167.0	END OF BOREHOLE											
	8.2	NOTES:											
		1. Borehole open to a depth of 7.0 m below ground surface (Elev. 168.2 m).											
		2. Water level at a depth of 5.0 m below ground surface (Elev. 170.2 m) upon completion of drilling.											
		3. Water level readings in piezometer:											
		Date Depth (m) Elev. (m)											
		08/08/14 3.1 172.1											
		08/27/14 3.1 172.1											
		09/15/14 3.1 172.1											

PROJECT		10-1111-0211		RECORD OF BOREHOLE No RW-2		SHEET 1 OF 1		METRIC						
G.W.P.		2152-01-00		LOCATION		N 4831027.1 ; E 287570.9		ORIGINATED BY						
DIST		Central HWY 401		BOREHOLE TYPE		150 mm O.D. Solid Stem Augers		COMPILED BY						
DATUM		GEODETIC		DATE		July 7, 2014		CHECKED BY						
								NK						
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
176.0	GROUND SURFACE													
0.0	TOPSOIL													
0.2	Sandy clayey silt, sandy, trace gravel, trace organics and rootlets (FILL). Very stiff Brown Moist		1	AS	-									
174.6			2	SS	19									4 27 45 24
1.4	Sandy CLAYEY SILT, trace to some gravel (TILL) Stiff to hard Brown Moist		3	SS	26									
			4	SS	34									
			5	SS	45									
			6	SS	22									
	----- Becoming grey Auger grinding at 4.6 m depth		7	SS	11									10 28 40 22
170.4														
5.6	SILT, some sand, dilutant Compact Grey Wet		8	SS	18									
168.8														
7.2	CLAYEY SILT, sandy, some gravel (TILL) Hard Grey Moist		9	SS	31									
167.8														
8.2	END OF BOREHOLE													
NOTES: 1. Borehole open to a depth of 7.0 m below ground surface (Elev. 169.0 m). 2. Water level at a depth of 6.9 m below ground surface (Elev. 169.1 m) upon completion of drilling.														

PROJECT		10-1111-0211		RECORD OF BOREHOLE No DC-13		SHEET 1 OF 1		METRIC																			
2150-01-00		LOCATION		N 4830934.4 ; E 287419.4		ORIGINATED BY		SB																			
DIST		HWY 401- 403 W Ramp		BOREHOLE TYPE		57 mm I.D. Hollow Stem Augers		COMPILED BY																			
BM		DATE		September 13, 2012		CHECKED BY		TVA																			
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS		ELEVATION SCALE		DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)													
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES																					
174.7	0.0	GROUND SURFACE																									
0.2		TOPSOIL																									
		CLAYEY SILT, some sand, some gravel to gravelly, containing roots to a depth of 0.6 m (TILL) Firm to hard Brown Moist		1	SS	5																					
				2	SS	24																					
				3	SS	22																					
				4	SS	22																					
				5	SS	50/0.15																					
				6	SS	24																					
				7	SS	14																					
				8	SS	24																					
167.4	7.3	CLAYEY SILT, trace sand Stiff Grey Wet		9A	SS	9																					
166.8	7.9	SILT, trace to some clay, trace sand Loose Grey Wet		9B																							
165.9	8.8	CLAYEY SILT, trace sand, trace gravel (TILL) Very stiff to hard Grey Moist to wet		10	SS	18																					
				11	SS	66																					
163.4	11.3	END OF BOREHOLE																									
NOTE: 1. Piezometer installation consists of 50 mm diameter PVC pipe with a 3.0 m slotted screen. Water Level Readings <table border="1" style="width:100%; border-collapse: collapse;"> <thead> <tr> <th>Date</th> <th>Depth (m)</th> <th>Elev. (m)</th> </tr> </thead> <tbody> <tr> <td>09/13/12</td> <td>Dry</td> <td>-</td> </tr> <tr> <td>10/09/12</td> <td>6.2</td> <td>168.5</td> </tr> <tr> <td>11/05/12</td> <td>6.3</td> <td>168.4</td> </tr> </tbody> </table>																Date	Depth (m)	Elev. (m)	09/13/12	Dry	-	10/09/12	6.2	168.5	11/05/12	6.3	168.4
Date	Depth (m)	Elev. (m)																									
09/13/12	Dry	-																									
10/09/12	6.2	168.5																									
11/05/12	6.3	168.4																									

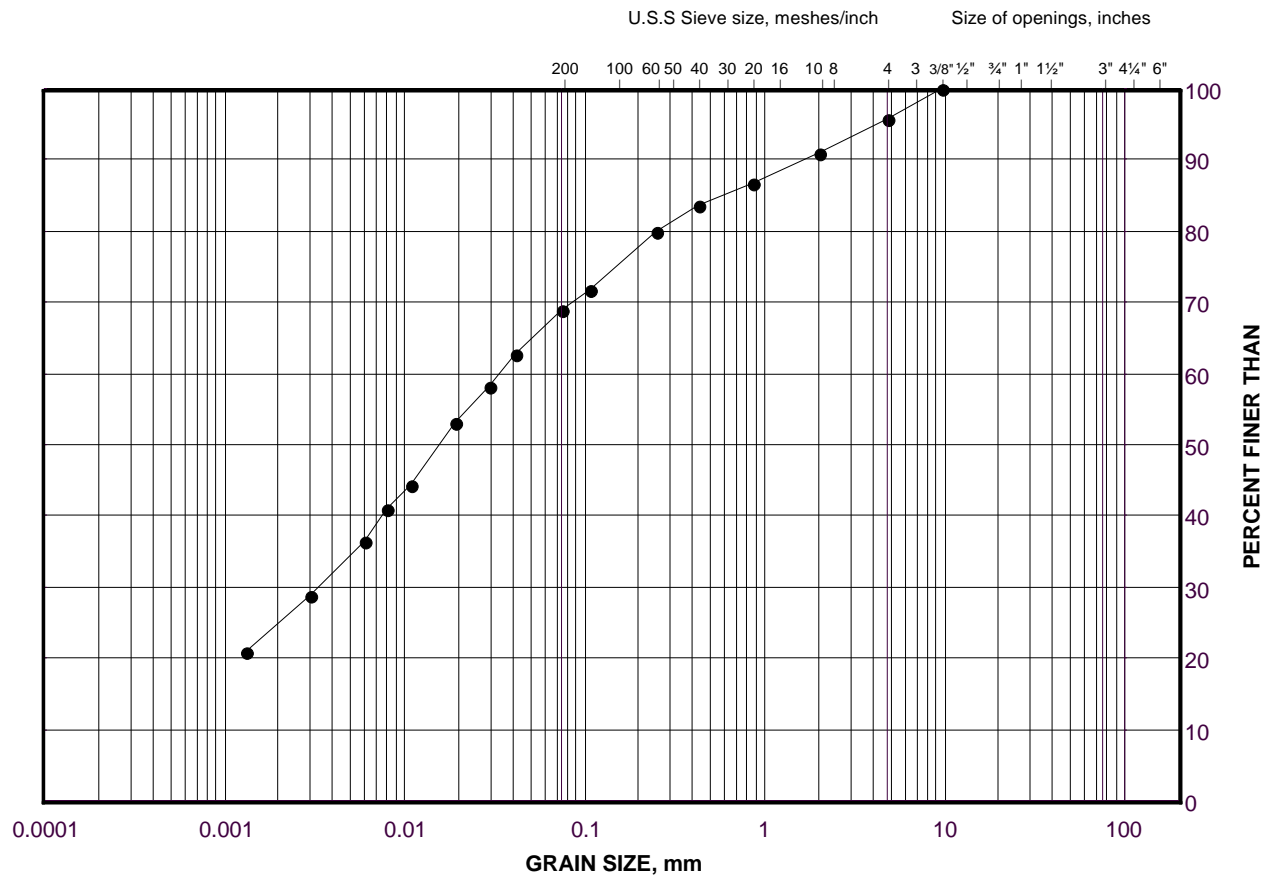
PROJECT		2150-01-00		LOCATION		N 4830962.3 ; E 287473.2		ORIGINATED BY		SB								
DIST		HWY 401- 403 W Ramp		BOREHOLE TYPE		57 mm I.D. Hollow Stem Augers		COMPILED BY		BM								
DATUM		Geodetic		DATE		September 12, 2012		CHECKED BY		TVA								
SOIL PROFILE				SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	SHEAR STRENGTH kPa												
175.4	GROUND SURFACE																	
0.0	TOPSOIL																	
0.2	CLAYEY SILT, some sand, trace to some gravel, containing roots to a depth of 0.6 m (TILL) Firm to very stiff Brown Moist		1	SS	5	175												
			2	SS	19	174												
			3	SS	20	173												
			4	SS	28	172												
			5	SS	21	171												
			6	SS	15	170												
			7	SS	13	169												
			8	SS	12	168												
168.1	Sandy SILT, trace to some clay, trace gravel Compact Grey Wet		9	SS	11													
167.2	END OF BOREHOLE																	
8.2	NOTES: 1. Borehole open to a depth of 7.3 m below ground surface (Elev. 168.1 m). 2. Water level inside augers at a depth of 7.0 m below ground surface (Elev. 168.4 m) upon completion of drilling.																	

GRAIN SIZE DISTRIBUTION

Sandy Clayey Silt (Fill)

Retaining Wall 4

FIGURE D1



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	RW-2	2	175.0

Project Number: 10-1111-0211

Checked By: NK

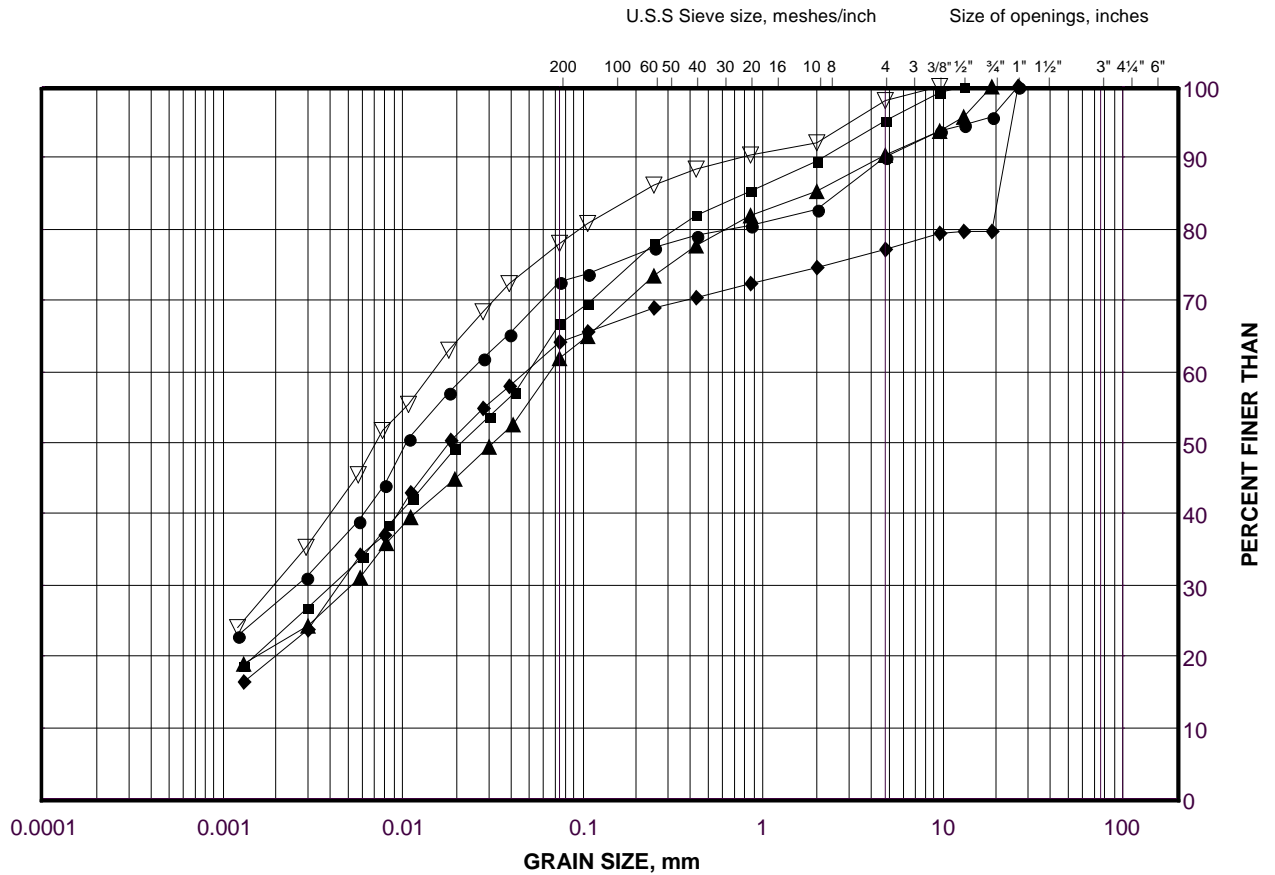
Golder Associates

Date: 11-Mar-15

GRAIN SIZE DISTRIBUTION

Sandy Clayey Silt to Clayey Silt (Till) (Upper Deposit)
Retaining Wall 4

FIGURE D2



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

LEGEND

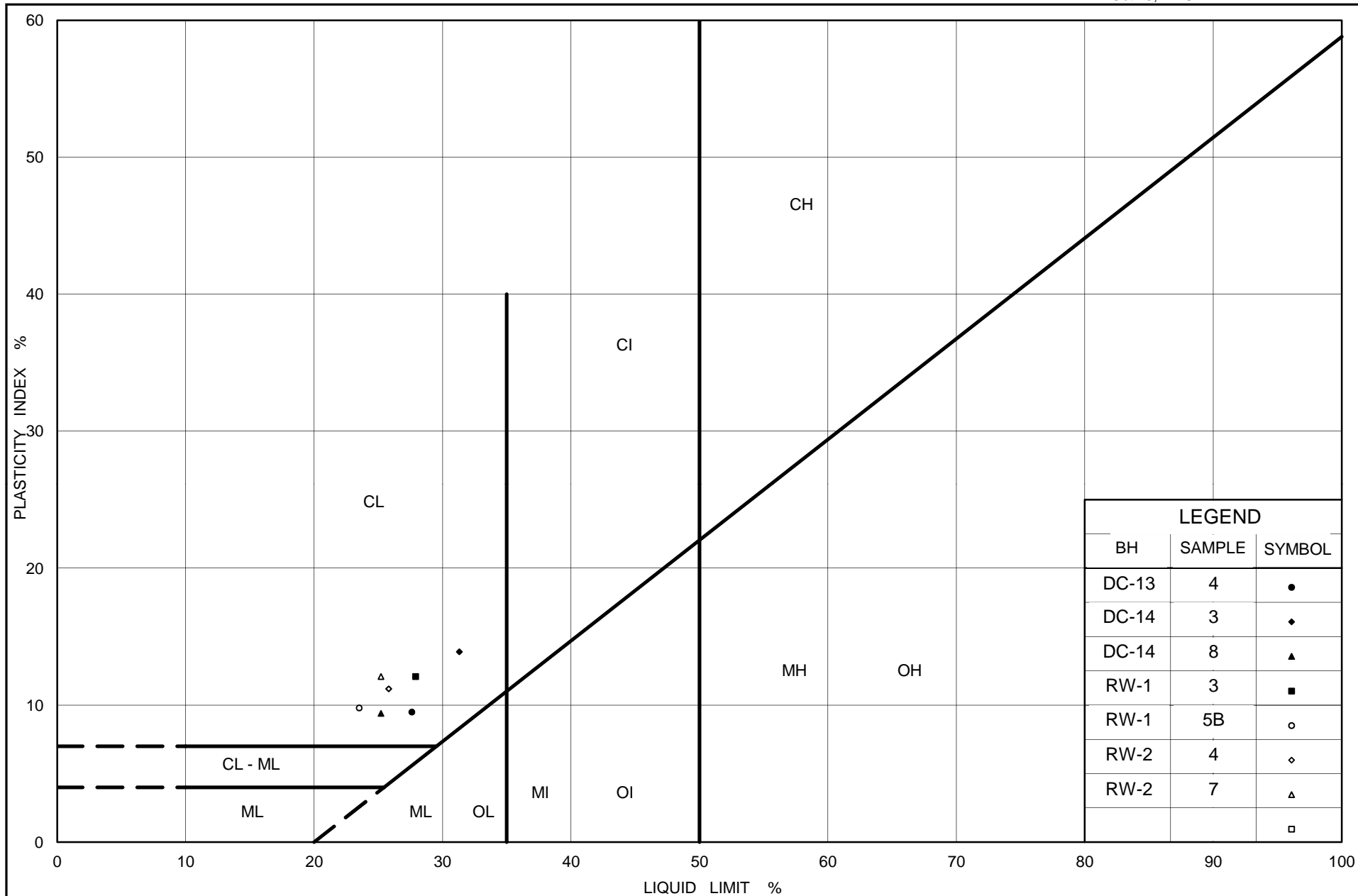
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	DC-14	3	173.7
■	RW-1	3	173.4
◆	DC-13	4	172.1
▲	RW-2	4	173.4
▽	DC-14	8	168.9

Project Number: 10-1111-0211

Checked By: NK

Golder Associates

Date: 24-Mar-15



Ministry of Transportation

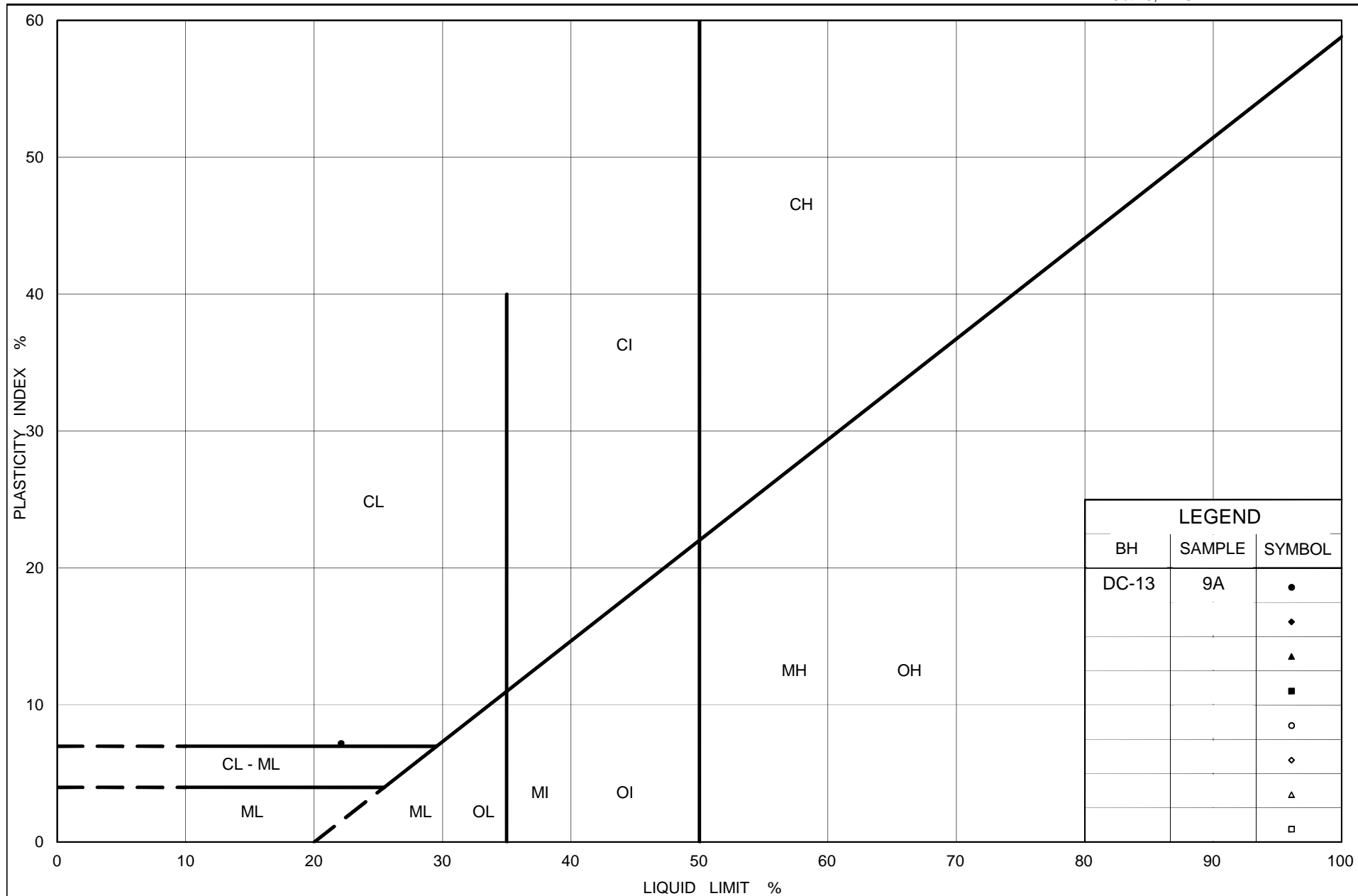
Ontario

PLASTICITY CHART **Sandy Clayey Silt to Clayey Silt (Till) (Upper Deposit)** **Retaining Wall 4**

Figure No. D3

Project No. 10-1111-0211

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Ontario

PLASTICITY CHART Clayey Silt Retaining Wall 4

Figure No. D4

Project No. 10-1111-0211

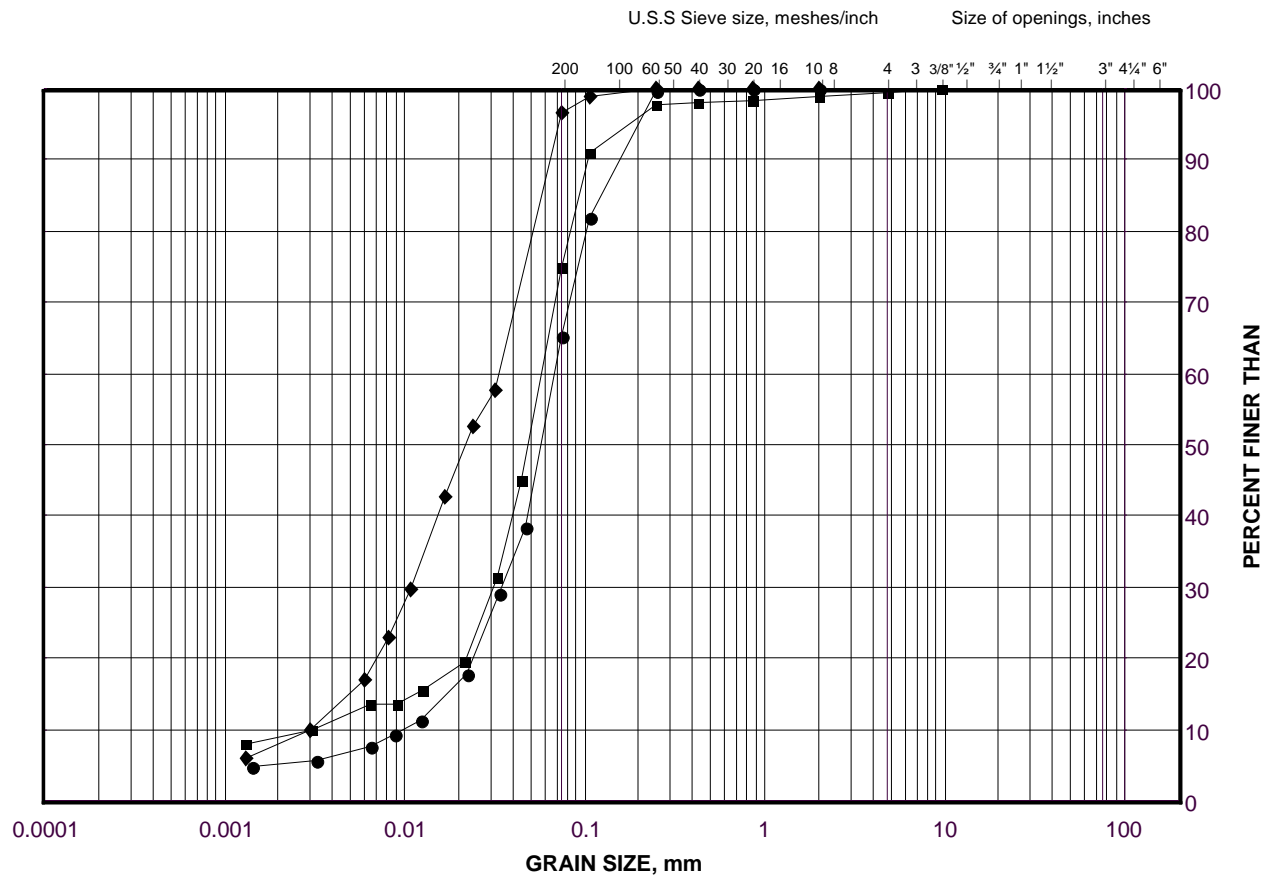
Checked By: NK

GRAIN SIZE DISTRIBUTION

Silt to Sandy Silt to Silt and Sand

Retaining Wall 4

FIGURE D5



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

LEGEND

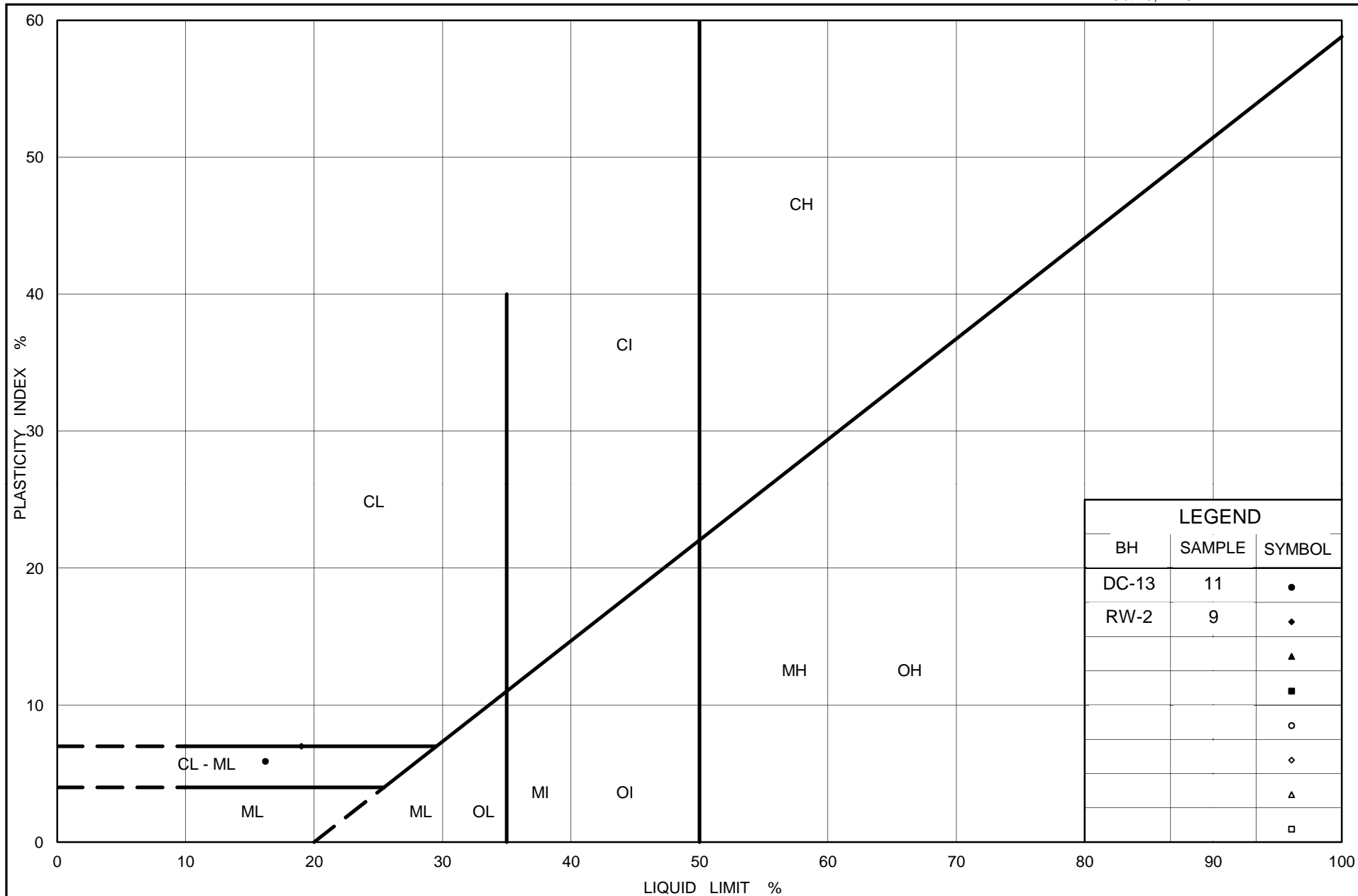
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	RW-1	8	168.8
■	DC-14	9	167.5
◆	DC-13	9B	166.7

Project Number: 10-1111-0211

Checked By: NK

Golder Associates

Date: 24-Mar-15



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PLASTICITY CHART Clayey Silt (Till) (Lower Deposit) Retaining Wall 4

Figure No. D6

Project No. 10-1111-0211

Checked By: NK



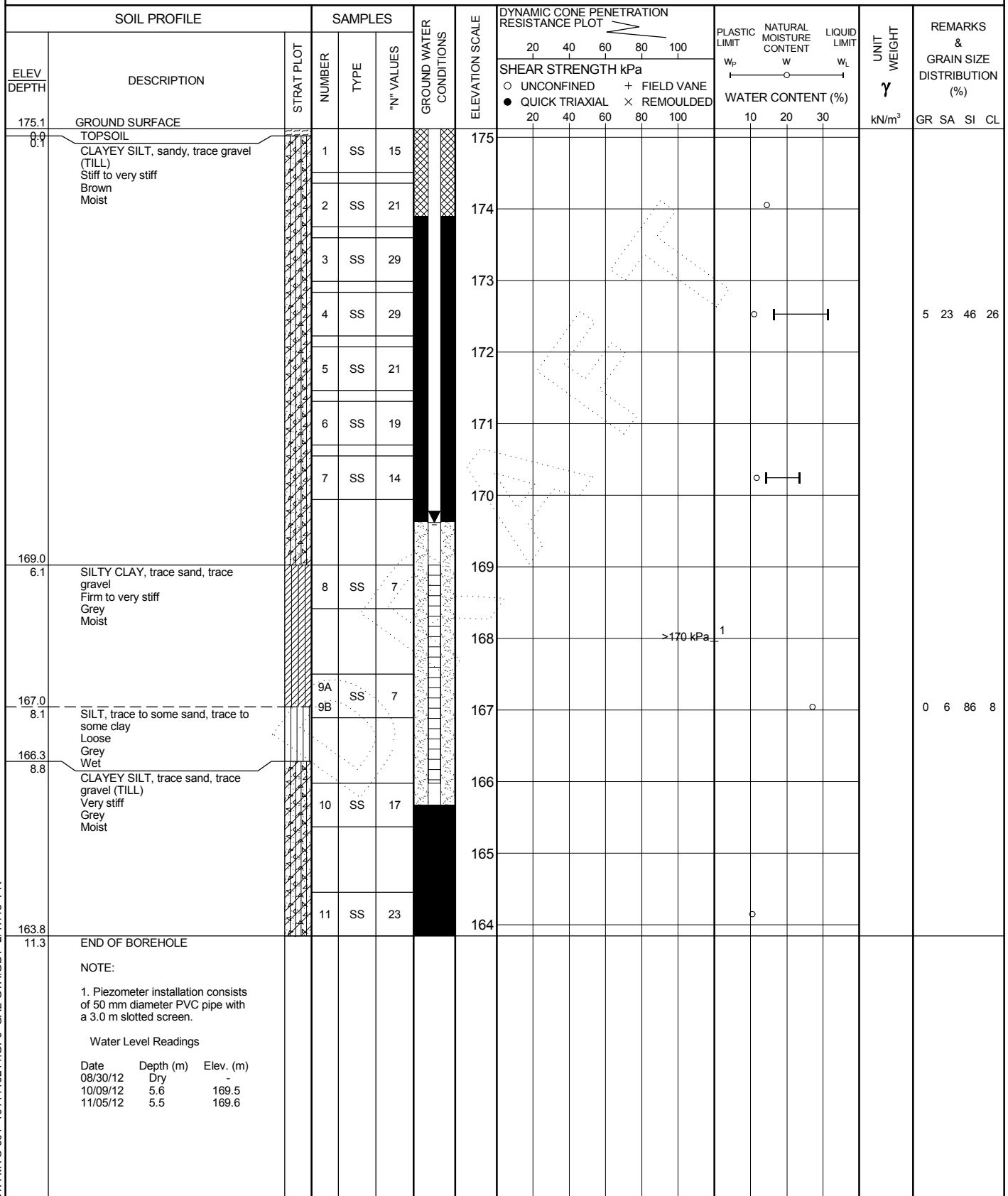
APPENDIX E

Retaining Wall 5, Highway 401 – STA. 16+345 to STA. 16+445 Record of Borehole Sheets and Laboratory Test Results

PROJECT		10-1111-0211		RECORD OF BOREHOLE No RW-3		SHEET 1 OF 1		METRIC					
G.W.P.		2152-01-00		LOCATION		N 4830889.5 ; E 287546.9		ORIGINATED BY					
DIST		Central HWY 401		BOREHOLE TYPE		150 mm O.D. Solid Stem Augers		COMPILED BY					
DATUM		GEODETIC		DATE		July 8, 2014		CHECKED BY					
								NK					
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID UNIT REMARKS				
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	W _p W W _L	WATER CONTENT (%)	γ	GR SA SI CL
173.7	0.0	GROUND SURFACE											
	0.1	TOPSOIL											
		Sandy clayey silt, trace gravel (FILL)		1	AS	-							
		Stiff		2	SS	15							
		Brown											
		Moist											
172.3	1.4	CLAYEY SILT with SAND, trace to some gravel (TILL)		3	SS	24							
		Firm to very stiff		4	SS	26							
		Brown		5	SS	15							
		Moist		6	SS	14							
		Auger grinding											
		Becoming grey											
169.2	4.5	CLAYEY SILT		7	SS	6							
		Firm											
		Brown											
		Moist											
168.1	5.6	SILT and SAND		8A	SS	25							
		Compact		8B									
		Grey											
		Wet											
167.1	6.6	CLAYEY SILT with SAND, trace to some gravel (TILL)											
		Very stiff											
		Grey											
		Moist											
165.5	8.2	END OF BOREHOLE		9	SS	25							
		NOTES:											
		1. Water level at a depth of 4.70 m below ground surface (Elev. 169.0 m) upon completion of drilling.											
		2. Water level readings in piezometer:											
		Date Depth (m) Elev. (m)											
		08/08/14 2.5 171.2											
		08/27/14 2.5 171.2											
		09/15/14 2.5 171.2											

PROJECT		10-1111-0211		RECORD OF BOREHOLE No RW-4		SHEET 1 OF 1		METRIC								
G.W.P.		2152-01-00		LOCATION		N 4830913.2 ; E 287595.9		ORIGINATED BY								
DIST		Central HWY 401		BOREHOLE TYPE		150 mm O.D. Solid Stem Augers		COMPILED BY								
DATUM		GEODETIC		DATE		July 5, 2014		CHECKED BY								
								NK								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
174.4	GROUND SURFACE															
0.0	TOPSOIL															
	Sandy clayey silt, trace gravel, trace rootlets, with sand lenses (FILL)		1	AS	-											
	Stiff															
	Brown		2	SS	12											2 25 48 25
	Moist															
173.0																
1.4	Sandy CLAYEY SILT, some gravel (TILL)															
	Firm to very stiff		3	SS	21											
	Brown															
	Moist		4	SS	23											
	Becoming grey															
			5	SS	19											
			6	SS	15											
169.9																
4.5	CLAYEY SILT															
	Firm		7	SS	7											
	Brown															
	Moist															
168.8																
5.6	SILT, some sand, trace to some gravel, trace to some clay															
	Compact		8A	SS	13											10 18 64 8
	Grey		8B													
	Wet															
168.0																
6.4	Sandy CLAYEY SILT, some gravel (TILL)															
	Stiff to very stiff															
	Grey															
	Wet		9	SS	28											
166.2																
8.2	END OF BOREHOLE															
	NOTE:															
	1. Open borehole dry upon completion of drilling.															

PROJECT <u>10-1111-0211</u>		RECORD OF BOREHOLE No DC-7		SHEET 1 OF 1		METRIC	
2150-01-00		LOCATION <u>N 4830868.9 ; E 287506.4</u>		ORIGINATED BY <u>SB</u>			
DIST <u> </u> HWY <u>401- 403 W Ramp</u>		BOREHOLE TYPE <u>108 mm I.D. Hollow Stem Augers</u>		COMPILED BY <u>BM</u>			
DATUM <u>Geodetic</u>		DATE <u>August 30, 2012</u>		CHECKED BY <u>TVA</u>			



GTA-MTO 001 1011110211.GPJ GAL-GTA.GDT 2/17/15 PR

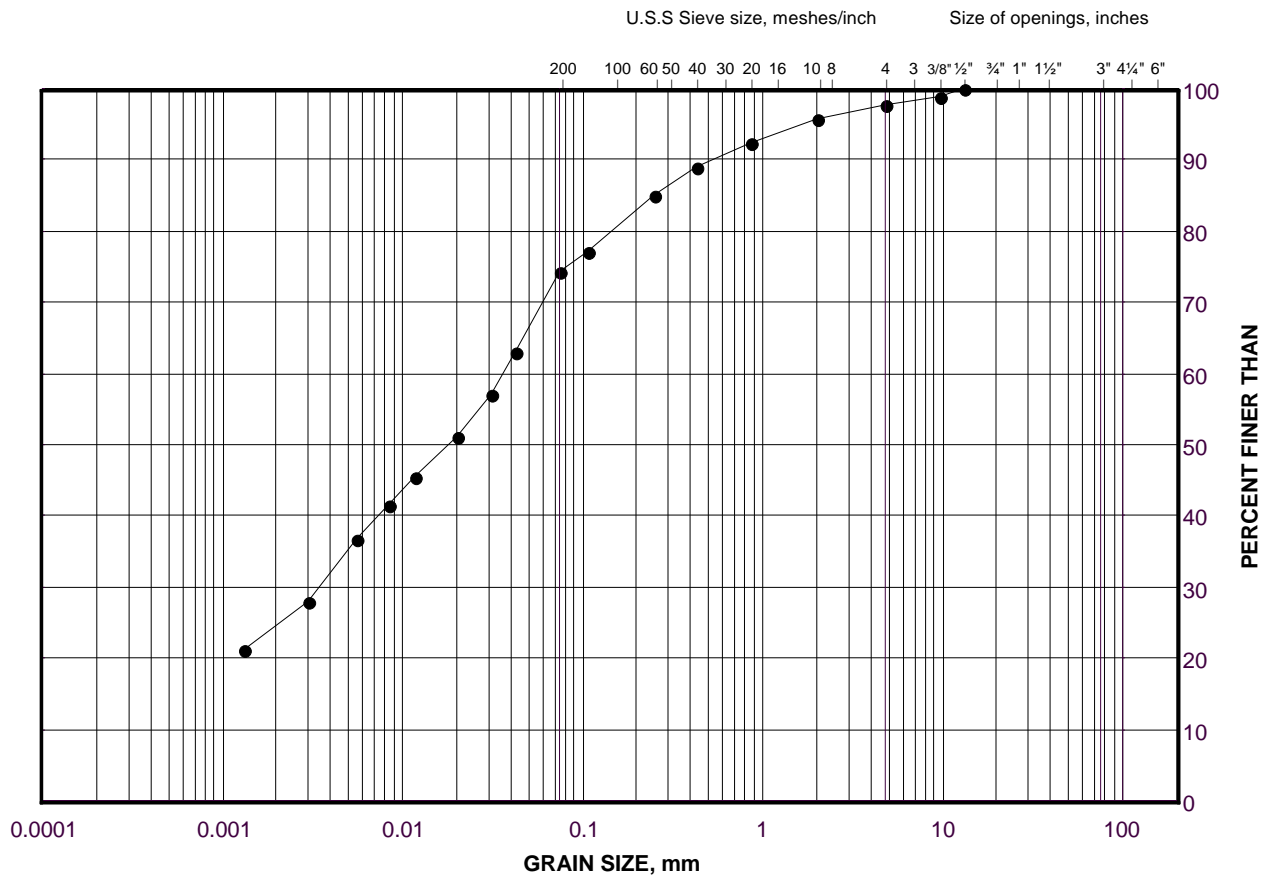
+ ³, × ³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

GRAIN SIZE DISTRIBUTION

Sandy Clayey Silt (Fill)

Retaining Wall 5

FIGURE E1



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

LEGEND

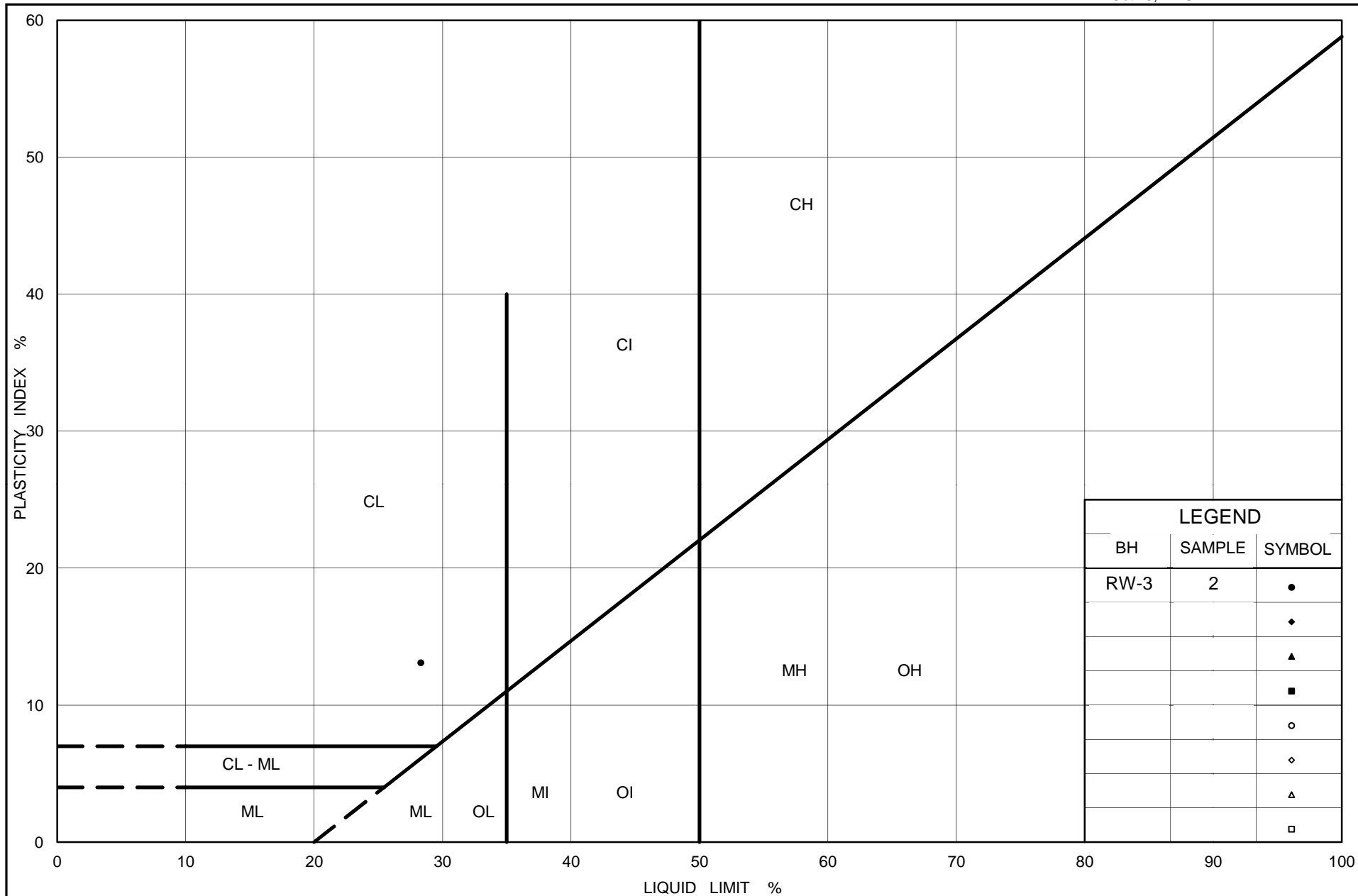
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	RW-4	2	173.3

Project Number: 10-1111-0211

Checked By: NK

Golder Associates

Date: 11-Mar-15



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PLASTICITY CHART Sandy Clayey Silt (Fill) Retaining Wall 5

Figure No. E2

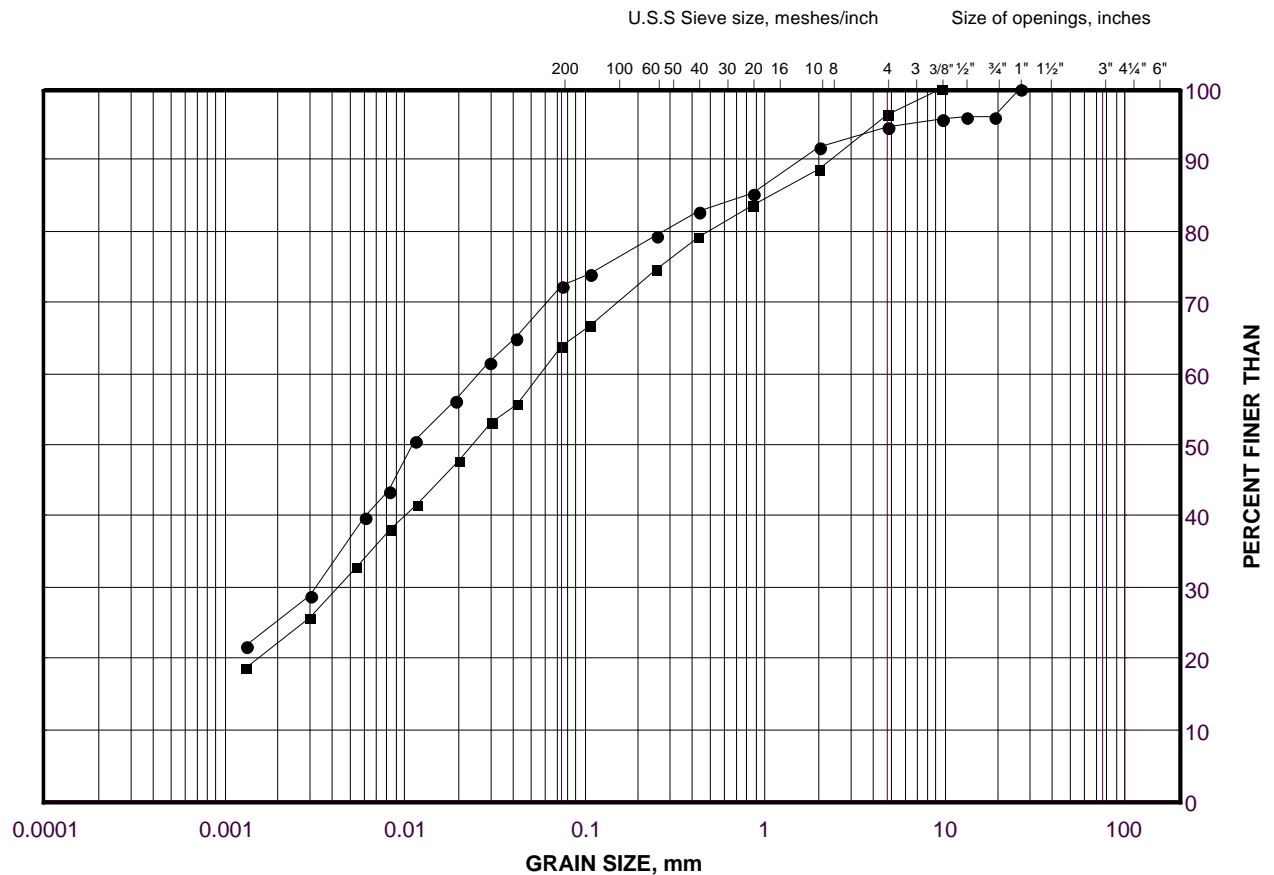
Project No. 10-1111-0211

Checked By: NK

GRAIN SIZE DISTRIBUTION

Sandy Clayey Silt (Till) (Upper Deposit)
Retaining Wall 5

FIGURE E3



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

LEGEND

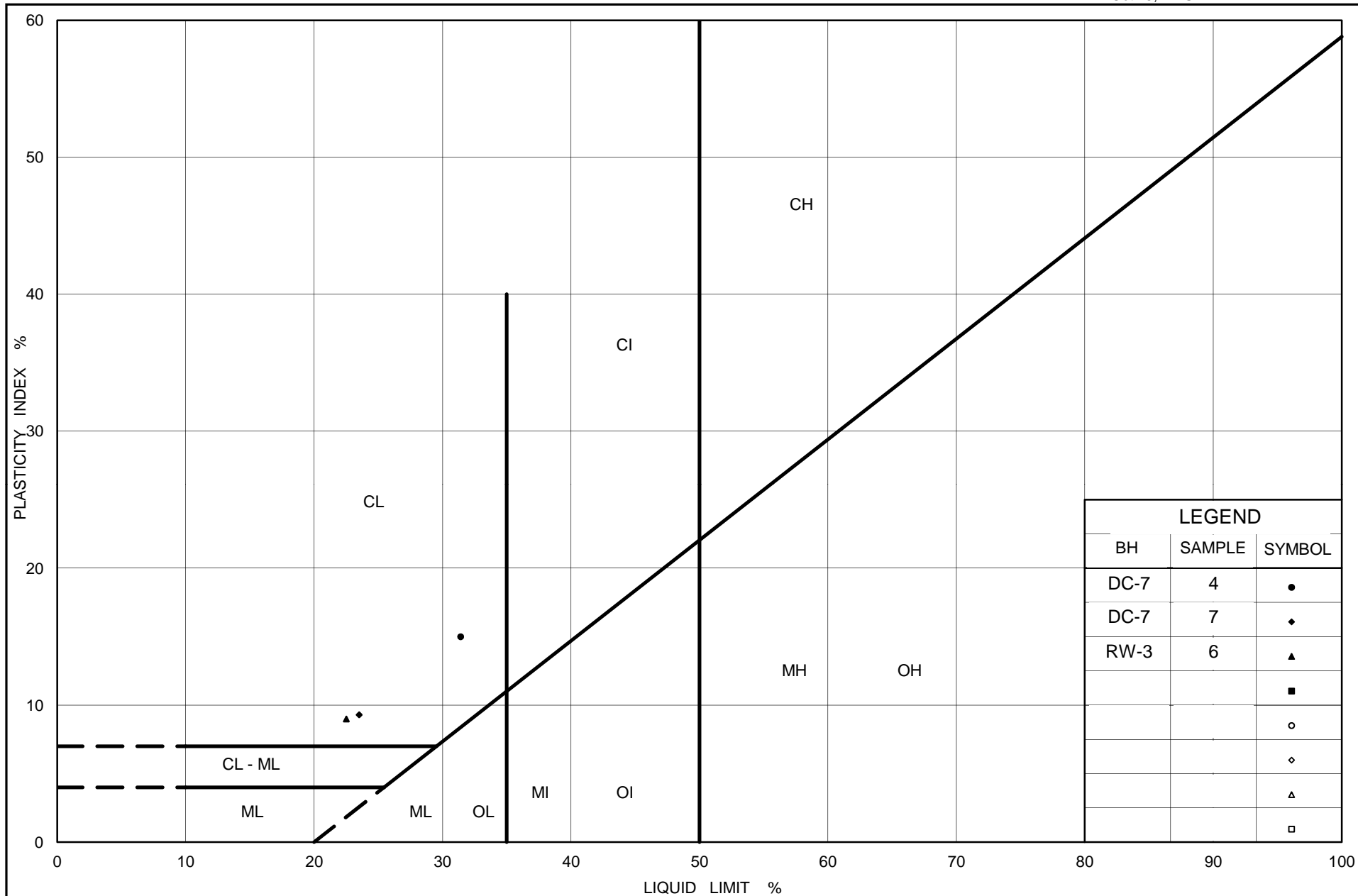
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	DC-7	4	172.5
■	RW-3	4	171.1

Project Number: 10-1111-0211

Checked By: NK

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Date: 11-Mar-15



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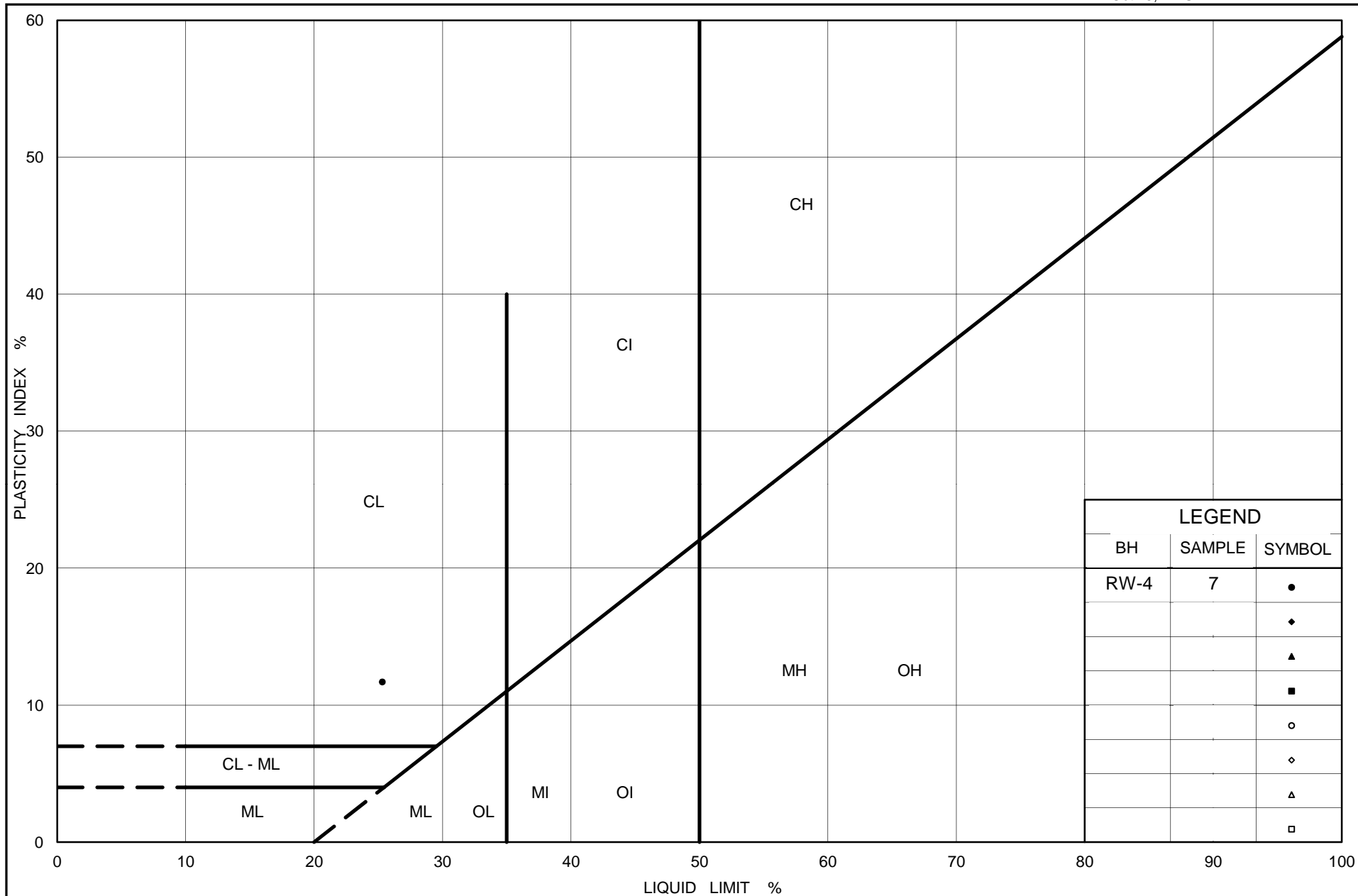
Ontario

PLASTICITY CHART
 Sandy Clayey Silt (Till) (Upper Deposit)
 Retaining Wall 5

Figure No. E4

Project No. 10-1111-0211

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PLASTICITY CHART Clayey Silt Retaining Wall 5

Figure No. E5

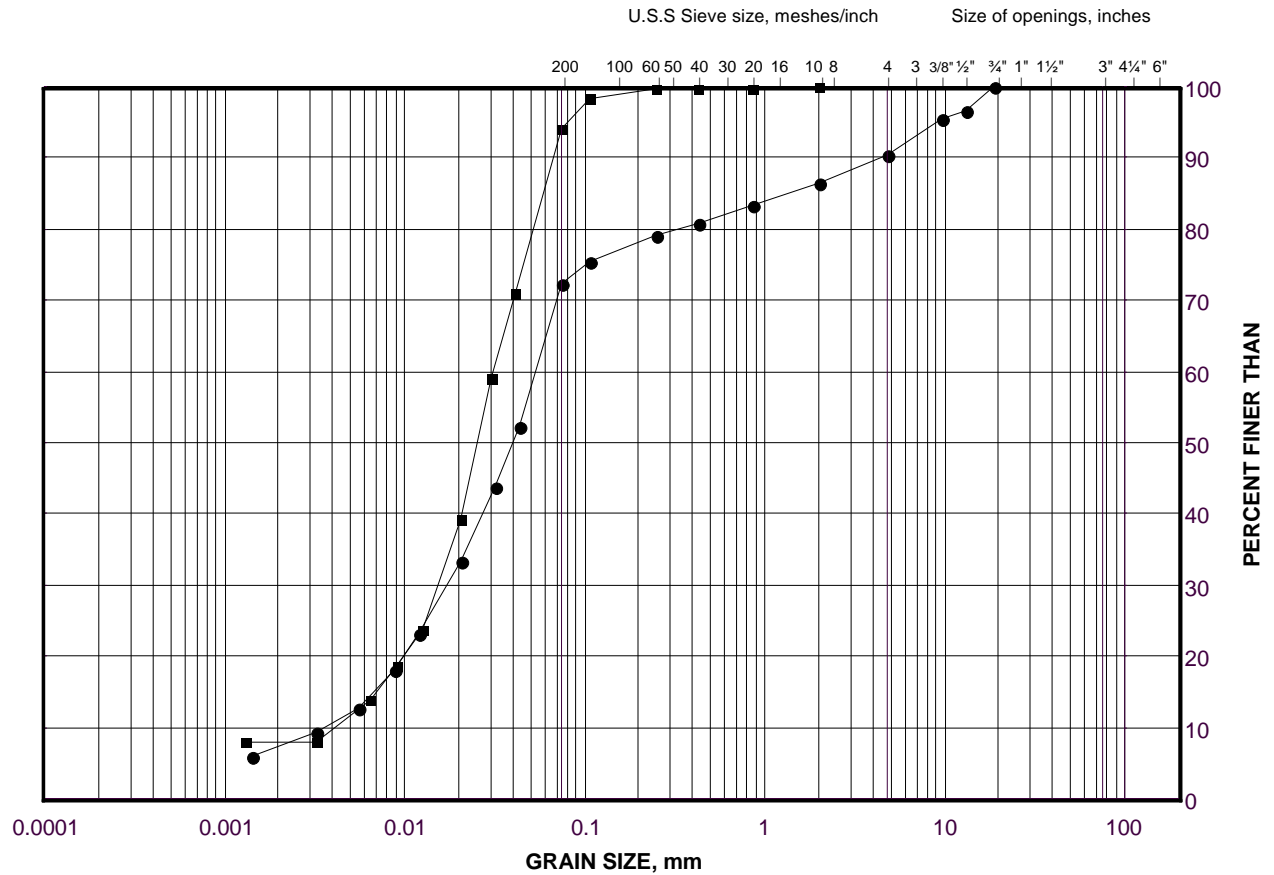
Project No. 10-1111-0211

Checked By: NK

GRAIN SIZE DISTRIBUTION

Silt
Retaining Wall 5

FIGURE E6



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	RW-4	8A	168.2
■	DC-7	9B	167.0

Project Number: 10-1111-0211

Checked By: NK

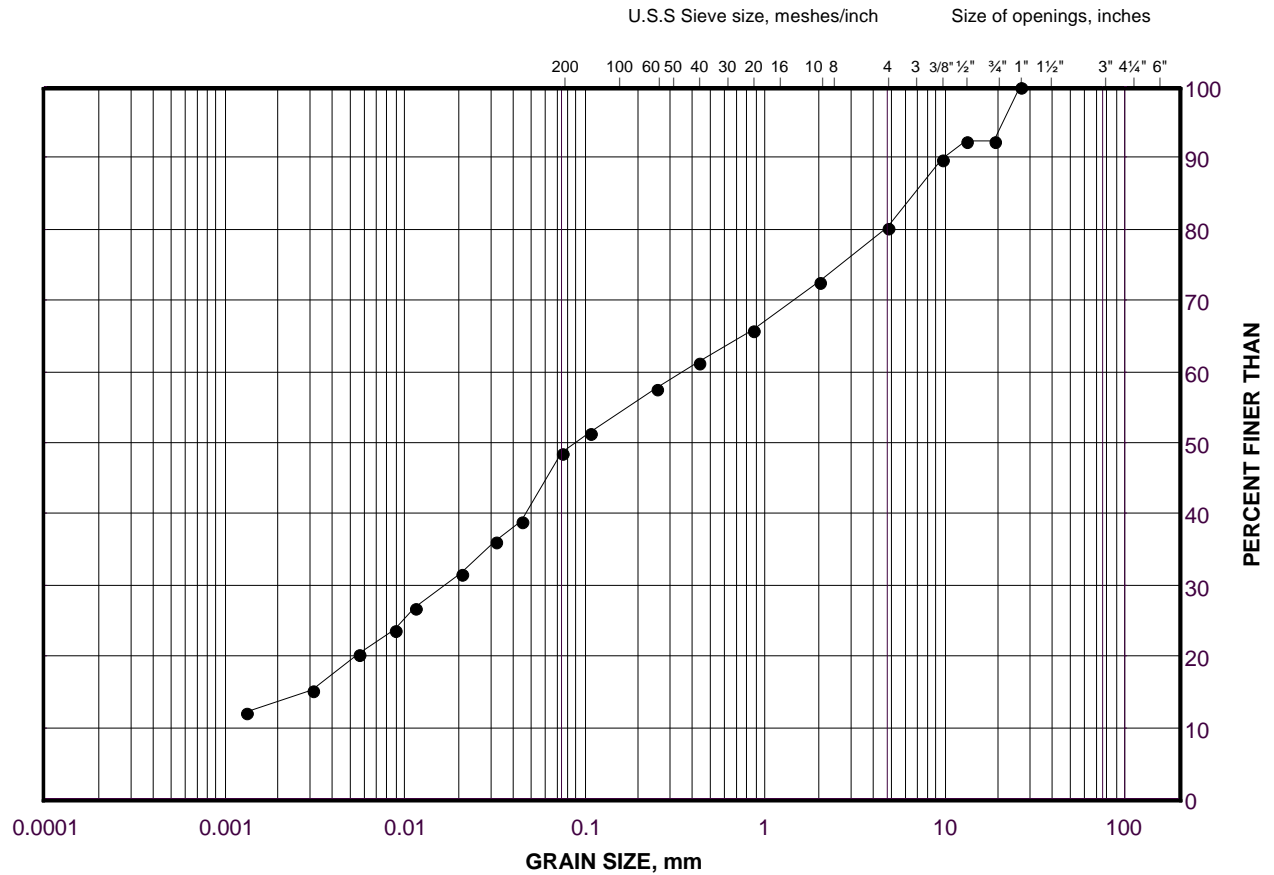
Golder Associates

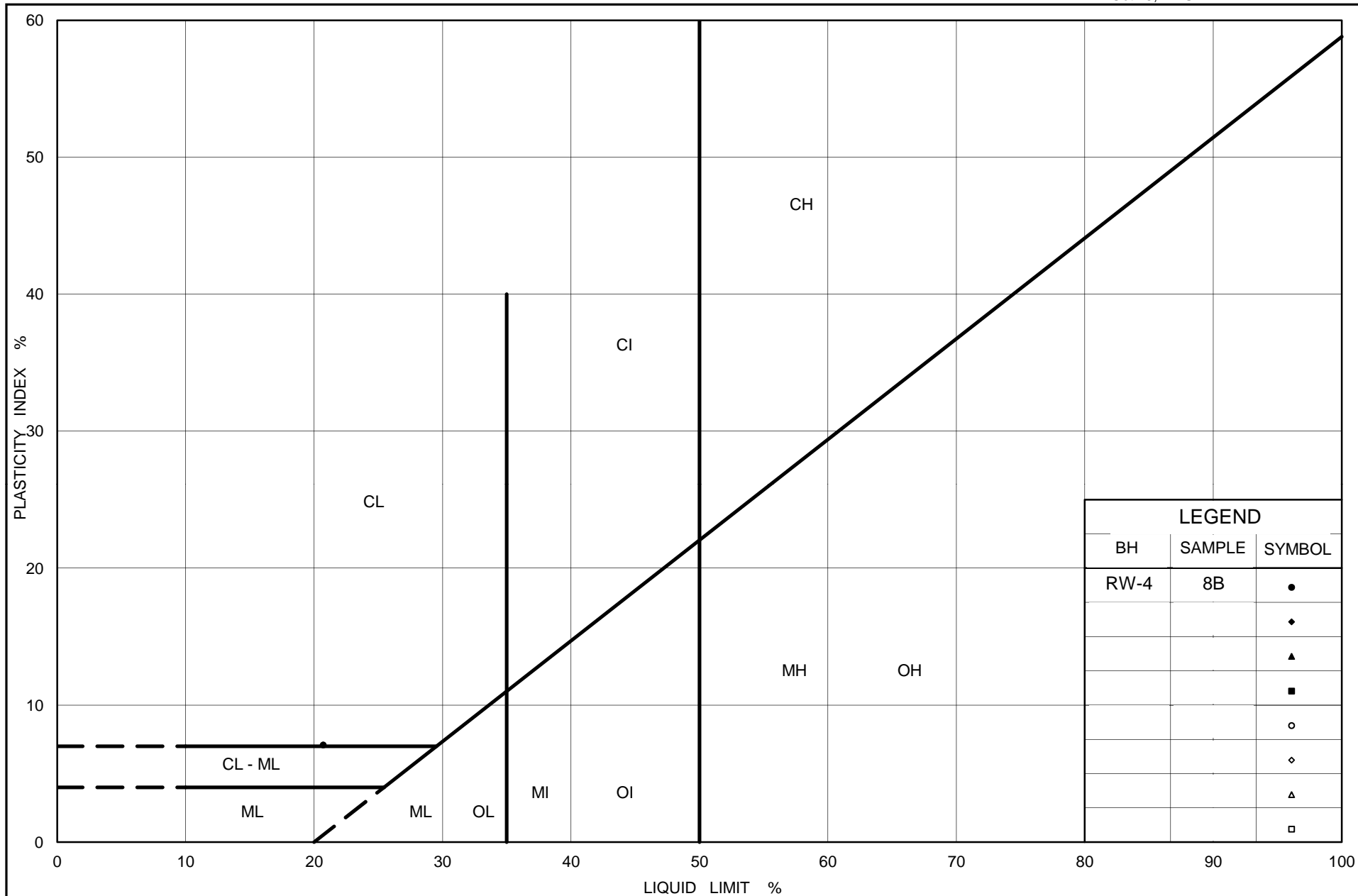
Date: 11-Mar-15

GRAIN SIZE DISTRIBUTION

Clayey Silt with Sand (Till) (Lower Deposit)
Retaining Wall 5

FIGURE E7





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PLASTICITY CHART Sandy Clayey Silt (Till) (Lower Deposit) Retaining Wall 5

Figure No. E8

Project No. 10-1111-0211

Checked By: NK



APPENDIX F

Non-Standard Special Provisions



PRELOADING

Operational Constraint

Preload Period –Embankments for Retaining Walls 2 and 3

This operational constraint applies for the northward widening of the embankments on Highway 401 to the east and west of the Fletcher's Creek bridges in the area of Retaining Walls 2 and 3. Following construction of the embankment widening to the top of the granular sub-base, the fills shall remain in place for a minimum period of sixty (60) days or until such time as determined by the results of the settlement monitoring program (whichever period of time is longer). The Contract Administrator must determine the actual end of the preload period after which time the remaining pavement structure and/or settlement sensitive structures can be constructed.

*This Operational Constraint will require modification based
on the construction schedule and selected method(s) of
settlement mitigation for the embankment widening areas.*



WORKING SLAB - Item No.

Non-Standard Special Provision

1.0 Scope

This Special Provision covers the requirements for the supply and placement of a concrete working slab under structure foundations.

2.0 References

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

Ontario Provincial Standard Specifications, Construction

OPSS 902 Excavating and Backfilling - Structures

3.0 Definitions - Not Used

4.0 Design And Submission Requirements - Not Used

5.0 Materials

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

6.0 EQUIPMENT - Not Used

7.0 CONSTRUCTION

7.01 Excavation

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

7.02 Protection of Founding Soil

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

7.03 Protection of Founding Bedrock

The surface of the footing founding rock shall be exposed, cleaned and any loose or fractured parts removed so that sound rock is exposed. The working slab shall be placed on the exposed cleaned sound founding rock surface as specified in the Contract Documents.

Thickness of the mass concrete pad shall depend on the slope and irregularities in the exposed founding rock surface. A nominal thickness and a footprint plan view area has been specified on the Contract Documents



7.04 Dewatering

Dewatering shall be carried out according to OPSS 902.

8.0 Quality Assurance - Not Used

9.0 Measurement For Payment - Not Used

10.0 Basis of Payment

10.01 Working Slab - Item

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

END OF SECTION



FOUNDATION REPORT - RETAINING WALLS HIGHWAY 401 WIDENING, GWP 2150-01-00

OBSTRUCTIONS - Item No.

Non-Standard Special Provision

The existing fill and the native cohesionless till contains cobbles and boulders as indicated in the Record of Borehole sheets and as inferred from difficulties in advancing and grinding augers/casing. Consideration of the presence of these obstructions must be made in the selection of appropriate equipment and procedures for driving steel H-piles/ pipe piles or advancing caissons such that the design tip levels are achieved.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION



FOUNDATION REPORT - RETAINING WALLS HIGHWAY 401 WIDENING, GWP 2150-01-00

DEWATERING FOR EXCAVATION - Item No.

Non-Standard Special Provision

The contractor shall be alerted that high artesian groundwater levels (with hydrostatic head measured up to about 5 m above ground surface to Elevation 172 m) were encountered at the sites of proposed Retaining Wall 3 and Retaining Wall 2. It is estimated that the base of temporary excavations for the foundations may be up to 7 m below the creek water level (and up to 14 m below the measured artesian hydrostatic head) estimated at the time of the geotechnical investigation in September 2012. The subsoil conditions generally consist of existing fill, loose to compact sand and silts underlain by clayey silt to clayey silt with sand, underlain by a cohesionless till (comprised of silty sand, sand and silt, gravelly sand, silty sand and gravel, sandy silt and gravel, and sand and gravel). Construction of shallow foundations, pile caps, or excavation and replacement with engineered fill must be carried out in the dry. Dewatering within (and possibly surrounding) the foundation excavations will be required and the excavation shall be kept stable during the work.

Due to the proximity of the proposed Retaining Walls 2 and 3 to the edge of the Fletcher's Creek, a groundwater cut-off system (cofferdam or similar measure) is likely required to minimize dewatering requirements and mitigate potential environmental impacts.

Basis of Payment

Payment at the contract price for the above tender item shall be full compensation for all labour, equipment and materials required to do the work.

END OF SECTION



FOUNDATION REPORT - RETAINING WALLS HIGHWAY 401 WIDENING, GWP 2150-01-00

OBSTRUCTIONS - Item No.

Non-Standard Special Provision

The native clayey silt till and silty sand to sand and gravel and sand and silt till soils at the Retaining Wall sites contain cobbles and boulders as encountered in the boreholes and also inferred from grinding of and refusal to advancement of the augers and SPT sampler during borehole drilling and in-situ testing, documented in the Record of Borehole sheets. Consideration of the presence of these obstructions must be made in the selection of appropriate equipment and procedures for sub-excavation at the high fill areas and for permanent excavation at the deep cut areas.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION

SUPPLY AND INSTALLATION OF EMBANKMENT MONITORING EQUIPMENT –
Item No.

Non-Standard Special Provision

1.0 GENERAL

1.0.1 Scope

This non-standard special provision details the requirements for the supply and installation and baselining of the following geotechnical instruments for the monitoring of the settlement of the earth fill embankments adjacent to Retaining Walls 2 and 3:

- Temporary Survey Benchmarks (TBMs);
- Settlement Plates (SPs);
- Nail Pins (NPs).

1.0.2 Purpose

The purpose of these instruments is to monitor settlements in the embankment and foundation soils at Retaining Walls 2 and 3 after construction of the Highway 401 embankment fill up to the design pavement subgrade elevation. The locations of the retaining walls are shown on the Contract Drawings and are as follows:

- Retaining Wall 2 - 15+985 to 16+142
- Retaining Wall 3 - 16+179 to 16+250

The timing for the preload period shall begin when the pavement design subgrade elevation is achieved along the entire length of the walls and the end of the preload period shall be controlled by the instrumentation readings.

1.0.3 Personnel

The Contractor shall retain a Geotechnical Consultant with MTO classification of Geotechnical (Structures and Embankments) – Medium Complexity, to undertake the supply and installation of geotechnical instruments.

The Contractor shall be understood to refer to the Contractor and his Geotechnical Consultant.

1.0.4 Or Equal

The term, “or equal” shall be understood to indicate that the equal product is the same or better than the specified product in function, performance, reliability, quality and general configuration.

1.0.5 Notification

The Contract Administrator shall be notified a minimum of 15 working days in advance of commencing the installation of instruments.

1.0.6 Submission Requirements

The Contractor shall submit details of the proposed installation methods including locations and types of the monitoring equipment, protections systems, survey benchmarks and installation schedule, to the Contract Administrator, a minimum of 15 working days before the start of instrument installation.

1.0.7 Subsurface Conditions

The subsurface conditions at the site are described in the Foundation Investigation Report for Retaining Walls as specified elsewhere in the Contract Documents.

1.0.8 Equipment Operation and Weather Conditions

All installation and monitoring equipment and associated materials shall be capable of withstanding the range of temperatures possible for their location within the ground or on the surface.

1.1 **INSTALLATION**

The quantity and location of instruments are presented in Table 1.

Table 1 – Instrument Quantities and Locations

Monitoring Section			Quantities	
<i>Embankment Area</i>	<i>Station</i>	<i>Offset¹</i>	<i>SP</i>	<i>NP</i>
Retaining Wall 2	16+010	3	1	-
		4	-	1
	16+070	3	1	-
		4	-	1
	16+135	3	1	-
		4	-	1
Retaining Wall 3	16+185	7	1	-
		8	-	1
	16+215	7	1	-
		8	-	1
	16+240	7	1	-
		8	-	1
TOTAL			6	6

NOTES: 1. Offset distance relative to inside face of retaining wall

1.1.1 Instrument Location

Prior to the installation of instruments, the Contractor shall accurately survey and stake the location of each instrument and obtain a ground surface elevation at each instrument location.

1.1.2 Temporary Survey Benchmarks

The Contractor shall provide a minimum of two (2) non-yielding deep seated survey TBMs near the retaining walls. The Contractor shall submit proposed locations of TBMs 15 days in advance of installation to the Contract Administrator for confirmation of sight lines.

The number and locations of TBMs shall be such that direct sighting is possible from all geotechnical instruments to at least one (1) TBM. The Contractor shall establish the geodetic elevation of each such TBM.

1.1.3 Accuracy of Surveying for Elevations

Elevations shall be surveyed to an accuracy of ± 2 mm or better.

1.1.4 Materials and Equipment

The Contractor shall supply all materials and equipment required for the installation of instrumentation unless otherwise noted.

1.1.5 Underground Utilities

The Contractor shall be responsible for locating and protecting all underground utilities prior to installing the settlement monitoring instruments and drilling of boreholes for the TBMs. Any damage to underground utilities caused by the Contractor's work shall be repaired by the Contractor at no cost to the Owner or Contract Administrator.

1.1.6 Marking and Labelling

The location of any above-ground monitoring fixtures shall be made clearly visible to nearby traffic before, during and after embankment construction. Markings shall be of sufficient size to be visible from a reversing vehicle and after heavy snow falls.

Instruments shall be clearly labelled in the field, each instrument having a unique identifier. The labelling shall remain legible for the duration of the preload period.

1.1.7 Protection of Instruments

The Contractor shall adequately protect all instruments such that they are not damaged during construction. Any instrument damaged by the Contractor's work shall be immediately replaced at no cost to the Owner or Contract Administrator.

1.1.8 Boreholes

If applicable, the Contractor shall make a basic stratigraphic log of boreholes as they are being drilled. In situ or laboratory testing is not required.

Boreholes for the TBM shall be advanced using conventional drilling methods and shall be as vertical as practical.

1.1.9 Installation Program

Instrument installation shall commence when the elevation of the embankment fill construction is 1.5 m below the design pavement subgrade elevation.

2.0 TEMPORARY SURVEY BENCHMARKS – SUPPLY AND INSTALLATION

2.1 GENERAL

2.1.1 Scope

This Section contains the requirements for the supply and installation of TBMs.

The purpose of the benchmarks is to provide stable, non-settling reference points for the surveying of the SPs and NPs.

2.1.2 General Procedure

The TBMs shall be installed prior to embankment construction. The TBMs shall consist of a steel rod anchored to the bottom of a borehole terminated in competent (compact/stiff or better) native soils (minimum 3 m deep) at the site or approved equivalent suitable non-yielding reference point.

2.1.3 Number and Location

A minimum of two (2) Benchmarks should be provided to read each instrument located near the Retaining Walls. The number and locations of TBMs shall be adjusted in the field such that direct sighting of all SPs and NPs is possible from the TBMs.

2.2 MATERIALS

2.2.1 General

The Contractor shall supply all materials and equipment required for the installation of the TBMs.

2.2.2 Rod

The Contractor shall supply a steel pipe with an outside diameter not less than 25.4 mm, supplied in lengths as required to complete the installation as described in Section 2.3.

The top end of each length of TBM rod shall be threaded to receive a cap or to allow for connection of successive lengths of rods. A rounded cap shall be installed at the top of the rod in such a way that a single survey point can be clearly identified and returned to.

2.2.3 Sand

The Contractor shall supply clean, washed sand. The sand shall be Sakcrete washed general-purpose sand – or equal.

2.2.4 Grout

The Contractor shall supply cement-bentonite grout. A suitable grout mix design consists of 23 kg of bentonite (OPSS 1205), 143 litres of water and 40 kg of cement (Type GU – OPSS 1301).

2.2.5 Rod Anchor Grout

The Contractor shall supply cement-bentonite grout. A suitable grout mix design consists of 14 kg of bentonite (OPSS 1205), 49 litres of water and 40 kg of cement (Type GU – OPSS 1301).

2.2.6 Friction Reducing Sleeve

The Contractor shall supply a friction reducing sleeve for the full length of rod consisting of Schedule 40 – 50.8 mm (2") O.D. PVC pipe cut perpendicular to the axis of the pipe.

2.3 INSTALLATION

2.3.2 General

The Contractor shall install TBMs in accordance with the information below.

2.3.2 Borehole Installation

The borehole shall be advanced to competent compact/stiff native soils or minimum 3 m depth (whichever is greater). The diameter of the borehole shall be sufficient to fit the rod, friction reducing sleeve and rod anchor grout. The sides of the borehole shall be stable and the borehole shall be free of drilling mud and debris.

2.3.3 Rod

The coupling of the rods shall be such that all sections have the same axis and no separation or contraction will occur at the couplings.

2.3.4 Rod Anchor

The rod shall be installed vertically in the borehole with its bottom end resting at the bottom of the borehole.

The elevation of the bottom of the rod anchor shall be determined by measuring the length of the rod to the ground surface elevation. The bottom portion of the rod shall be fixed against the surrounding native soil by grouting the bottom 0.5 m of the borehole using the rod anchor grout mix to form a concrete/soil anchor.

Once grouting is completed and the rod anchor grout has set, the contractor shall pour clean sand in the lower 0.5 m length of the borehole above the concrete/soil anchor to create a base for the end of the friction reducing sleeve to rest on.

2.3.5 Friction Reducing Sleeve

The friction reducing sleeve shall be over the entire length of the rod above the rod anchor and sand.

2.3.6 Installation Details

The elevation, easting and northing of the top of the Benchmark rod shall be surveyed.

2.4 **COORDINATION WITH MONITORING**

2.4.2 Notification

The Contractor shall notify the Contract Administrator no later than three (3) working days after installing a TBM. At this time the Contractor shall also supply the following information to the Contract Administrator for each TBM:.

- TBM Northing and Easting in MTM NAD 83 coordinates;
- Elevation of the rod anchor bottom rod anchor length and top of rod in Geodetic datum;
- Date of installation;
- Stratigraphic log of subsurface conditions at the TBMs, including notes on drilling method obstructions it encountered;
- Installation notes/sketches; and,
- Description of TBM (rod), sleeves and rod anchors.

2.4.3 Monitoring

Monitoring of settlements with reference to the TBMs shall be done by others. Monitoring shall be conducted during the embankment construction immediately after the instruments are installed (i.e. when grades are 1.5 m below pavement subgrade elevation). The Contractor shall provide installation information as specified above and provide access to the TBMs for monitoring including, but not limited to, snow clearing in the winter. The Contractor shall provide electric power and general area lighting as needed.

2.5 **REPORTING**

The Contractor shall record and report relevant installation details to the Contract Administrator. These include, but are not limited to:

- TBM Northing and Easting in MTM NAD 83 coordinates;
- Elevation of the rod anchor bottom rod anchor length and top of rod in Geodetic datum;
- Dates of installation;
- Stratigraphic log of subsurface conditions at the TBMs, including notes on drilling obstructions it encountered;
- Installation notes / sketches; and,
- Description of TBM (rod), sleeves and rod anchors.

3.0 SETTLEMENT PLATES (SPs) – SUPPLY AND INSTALLATION

3.1 GENERAL

3.1.1 Scope

This section contains the requirements for the supply and installation of SPs.

The purpose of the SPs is to monitor settlements of the embankment fills adjacent to the retaining walls. The settlement readings shall help to establish the end of the preload period and subsequent timing for placement of the pavement structure and any other settlement sensitive structures. Settlement is measured by survey of the top of the rod with reference to stable, non-settling TBMs.

3.1.2 General Procedure

The SPs shall be placed on undisturbed native soil or embankment fills (or on top of engineered fill if sub-excavation and replacement is needed) at a depth of 1.5 m below the design pavement subgrade level which varies across the length of the retaining walls. As embankment construction proceeds the settlement measuring rods shall be extended above the new top of embankment.

A protective surround shall be extended with the rods as embankment construction proceeds.

3.1.3 Location

The locations of the SPs are shown in Table 1.

3.2 MATERIALS

3.2.1 General

The Contractor shall supply all materials and equipment required for the installation of the SPs.

3.2.2 Plate

The Contractor shall supply a steel plate with a thickness of at least 6.35 mm. The plate shall be at least 0.5 m wide by 0.5 m long.

3.2.3 Rod

The Contractor shall supply a steel pipe with an outside diameter not less than 25.4 mm, supplied in lengths as required to complete the installation as described in Section 3.3.

The top end of the full length of rod shall be threaded to receive a cap. A rounded cap shall be installed at the top of the rod in such a way that a single survey point can be clearly identified and returned to.

3.2.4 Protective Surround

The Contractor shall supply a protective surround for the portion of the rod within the embankment. Unless approved by the CA, the protective surround should consist of a corrugated steel pipe (300 mm diameter) with the space between the CSP and SP filled with sand.

3.3 INSTALLATION

3.3.1 General

The Contractor shall install SPs as described below.

3.3.2 Settlement Plate

The SPs shall be installed (horizontally) at 1.5 m below the design pavement subgrade elevation.

The elevation of the base of the plate shall be surveyed before backfilling.

3.3.3 Rod

The SP rod shall be fixed to the centre of the plate and perpendicular to the plate.

The coupling of the rods shall be such that all sections have the same axis and that no separation or contraction will occur at the couplings.

3.3.4 Extension of Rod

The SP rods shall be extended upwards as the embankment is constructed so that the top of the rod is always at least 0.3 m above the surrounding fill.

3.3.5 Protective Surround

The protective surround shall be extended concurrent with the rods.

The SP rod shall be in the centre of the CSP.

The annulus between the CSP and the SP shall be filled with sand to a level not higher than the top of the rod.

3.3.6 Installation Details

The elevation, easting and northing of the centre of the base of the plate shall be surveyed.

The elevation, easting and northing of the top of the rod shall be surveyed.

The total length from the base of the plate to the top of the rod shall be measured to an accuracy of ± 2 mm or better.

3.4 COORDINATION WITH MONITORING

3.4.1 Notification

The Contractor shall notify the Contract Administrator no later than three (3) working days after installing a SP. At this time the Contractor shall also supply the following information to the Contract Administrator for each SP:

- SP Northing and Easting in MTM NAD 83 coordinates;
- Elevation of base of plate and top of rod in Geodetic datum;
- Date of installation;
- Installation notes/sketches; and,
- Description of SP rods, sleeves and plates.

Adjustments in the length of any SP rod shall be coordinated with the Contract Administrator to allow for confirmatory surveying by others if deemed required. This surveying is necessary to accurately track the settlement data.

3.4.2 Monitoring

Monitoring of the SPs shall be done by others. Monitoring shall be conducted during the embankment construction and preload period. The Contractor shall provide installation information as specified above and provide access to the SPs for monitoring including, but not limited to a scaffolding platform and ladder if required and snow clearing in the winter. The Contractor shall provide electric power and general area lighting as needed for reading the instruments.

3.5 REPORTING

The Contractor shall record and report relevant installation details to the Contract Administrator. These include, but are not limited to:

- SP Northing and Easting in MTM NAD 83 coordinates;
- Elevation of base of plate and top of rod in Geodetic datum;
- Date of installation;
- Installation notes/sketches; and,
- Description of SP rods, sleeves and plates.

5.0 NAIL PINS (NPs) – SUPPLY AND INSTALLATION

5.1 GENERAL

5.1.1 Scope

This Section details the requirements for the supply and installation of NPs.

The purpose of the NPs is to monitor settlement of the ground surface at the elevation of the design pavement subgrade level. Settlement is measured by survey of the top of the NP with reference to stable non-settling TBMs.

5.1.2 General Procedure

NPs shall be installed on the top of the embankment fill when the pavement design subgrade elevation is reached.

5.1.3 Location

The locations of the NPs are given in Table 1.

5.2 MATERIALS

5.2.1 General

The Contractor shall supply all materials and equipment required for the installation of the Nail Pins.

5.2.2 Pin

The Contractor shall supply a 25.4 mm minimum diameter reinforcing steel bar (OPSS 905) cut 0.15 m long or equivalent.

The top of the reinforcing steel bar shall be angled or rounded in such a way that a single survey point can be clearly identified and repeated.

5.2.3 Concrete

The Contractor shall supply concrete (OPSS.Prov 1350) of minimum 25 MPa compressive strength and set time sufficient to secure the Nail Pin within two (2) days of pouring.

5.3 INSTALLATION

5.3.1 General

The Contractor shall install NPs within the wet concrete (prior to set) poured in a minimum 0.15 m diameter by 0.3 m deep hole. The top of the reinforcing bars shall protrude above the concrete such that an accurate single survey point can be clearly identified and repeated.

5.4 COORDINATION WITH MONITORING

5.4.1 Notification

The Contractor shall notify the Contract Administrator no later than three (3) working days after installing a Nail Pin. At this time, the Contractor shall also supply the following information to the Contract Administrator for each NP:

- NP Northing and Easting in MTM NAD 83 coordinates;
- Elevation of pin in Geodetic datum;
- Dates of installation; and,
- Installation notes / sketches.

5.4.2 Monitoring

Monitoring of the NPs shall be done by others. Monitoring shall be conducted during the preload period. The Contractor shall provide installation information as specified above and provide access to the NPs for monitoring.

5.5 REPORTING

The Contractor shall record and report relevant installation details to the Contract Administrator. These include, but are not limited to:

- NP Northings and Eastings in MTM NAD 83 coordinates;
- Elevation of pin in Geodetic datum;
- Date(s) of installation; and,
- Installation notes / sketches.

6.0 DECOMMISSIONING OF INSTRUMENTS

6.1 GENERAL

The Contractor shall decommission all the TBMs SPs, and NPs at the end of the monitoring program unless advised otherwise by the Contract Administrator. Decommissioning of instrumentation shall be carried out according to the Ontario Water Resources Act, R.R.O. 1990, Regulation 903, if applicable.

7.0 MEASUREMENT FOR PAYMENT

Measurement for Payment will be made on the basis of the number of units of surveyed TBMs, SPs, and NPs installed.

8.0 BASIS OF PAYMENTS

Payment at the Lump Sum price for this tender item shall be full compensation for all labour, monitoring equipment and material to do the work.

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