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PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

Credit River Bridges Highway 401 Improvements from East of Credit River to Trafalgar Road, Regional Municipalities of Peel and Halton W.O. 07-20021

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REPORT





Table of Contents

PART A – PRELIMINARY FOUNDATION INVESTIGATION REPORT

1.0	INTRODUCTION.....	1
2.0	SITE DESCRIPTION.....	1
3.0	INVESTIGATION PROCEDURES	1
4.0	SITE GEOLOGY AND SUBSURFACE CONDITIONS	3
4.1	Regional Geology	3
4.2	Subsurface Conditions.....	3
4.2.1	Topsoil	4
4.2.2	Fill	4
4.2.3	Surficial Silty Sand	4
4.2.4	Upper Clayey Silt Till.....	5
4.2.5	Sand and Gravel.....	5
4.2.6	Lower Sandy Silt to Silty Sand Till	5
4.2.7	Lower Clayey Silt Till.....	6
4.2.8	Shale Bedrock.....	6
4.3	Groundwater Conditions	7
5.0	CLOSURE.....	8

PART B – PRELIMINARY FOUNDATION DESIGN REPORT

6.0	DISCUSSION AND ENGINEERING RECOMMENDATIONS.....	9
6.1	General.....	9
6.2	Foundation Options	9
6.3	Shallow Foundations	11
6.3.1	Founding Elevations.....	11
6.3.2	Geotechnical Resistance/Reaction	12
6.4	Driven Steel H-Pile or Steel Pipe (Tube) Foundations.....	12
6.4.1	Founding Elevations.....	12
6.4.2	Axial Geotechnical Resistance/Reaction.....	13
6.5	Caissons	13



PRELIMINARY FOUNDATION REPORT - CREDIT RIVER BRIDGES

6.5.1	Founding Elevations.....	14
6.5.2	Axial Geotechnical Resistance/Reaction.....	14
6.6	Approach Embankments	14
6.6.1	Subgrade Preparation and Embankment Construction.....	14
6.6.2	Approach Embankment Stability	15
6.6.3	Approach Embankment Settlement.....	16
6.7	Construction Considerations.....	16
6.7.1	Excavation and Temporary Protection Systems	17
6.7.2	Groundwater Control.....	17
6.7.3	Bedrock Excavation and/or Socket Formation	18
6.7.4	Subgrade Protection	18
6.7.5	Obstructions.....	18
6.7.6	Scour Protection.....	18
6.7.7	Vibration Monitoring During Construction.....	19
6.8	Recommendations for Further Work During Detail Design	19
7.0	CLOSURE.....	20

REFERENCES

TABLES

Table 1	Comparison of Foundation Options
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DRAWINGS

Drawing 1	Credit River Bridge – Borehole Locations
Drawing 2	Credit River Bridge – Soil Strata

FIGURES

Figure 1	Static Global Stability – Credit River Bridge - Approach Embankments
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APPENDIX A Borehole Records

Lists of Abbreviations and Symbols
Lithological and Geotechnical Rock Description Terminology
Records of Boreholes 11-101 to 11-108

APPENDIX B Laboratory Test Results

Figure B1	Grain Size Distribution Test Results – Clayey Silt Fill
Figure B2	Plasticity Chart – Clayey Silt Fill
Figure B3	Grain Size Distribution Test Result – Upper Clayey Silt Till
Figure B4	Plasticity Chart – Upper Clayey Silt Till



PRELIMINARY FOUNDATION REPORT - CREDIT RIVER BRIDGES

Figure B5	Grain Size Distribution Test Results – Sand and Gravel
Figure B6	Grain Size Distribution Test Results – Silty Sand to Sandy Silt Till
Figure B7	Grain Size Distribution Test Results – Lower Clayey Silt Till
Figure B8	Plasticity Chart – Lower Clayey Silt Till
Figure B9	Unconfined Compression Test
Figure B10	Unconfined Compression Test
Table B1	Unconfined Compression Test (UC)
Table B2	Unconfined Compression Test (UC)
Table B3	Point Load Test on Rock Samples



PART A

**PRELIMINARY FOUNDATION INVESTIGATION REPORT
CREDIT RIVER BRIDGES
HIGHWAY 401 WIDENING
FROM EAST OF CREDIT RIVER TO TRAFALGAR ROAD
REGIONAL MUNICIPALITIES OF PEEL AND HALTON
W.O. 07-20021**



1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by URS Canada Inc. (URS) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the future widening of Highway 401 from east of the Credit River in the Regional Municipality of Peel to Trafalgar Road (approximately 9.7 km) in the Regional Municipality of Halton, Ontario.

This report addresses the results of the subsurface investigation carried out for the proposed replacement of the existing Credit River bridges.

The terms of reference and scope of work for the preliminary foundation engineering services are outlined in MTO's Request for Proposal (RFP) for Assignment No. 2008-E-0015 dated February 2010, and in Section 5.8 of the *Technical Proposal* for this assignment.

2.0 SITE DESCRIPTION

The Credit River bridges carry the Highway 401 eastbound and westbound lanes over the Credit River in the City of Mississauga, within the Regional Municipality of Peel, Ontario. The existing bridges consist of 68 m long, three-span structures with the existing abutments and piers supported on spread footings.

The bridges are located near the eastern edge of the broad, flat floodplain of the Credit River valley, with the natural ground surface within the floodplain at approximately Elevation 161 m to 161.5 m. Immediately west of the structure site, the natural ground surface rises out of the Credit River valley to approximately Elevation 170 m. The river channel is approximately 25 m to 30 m wide at the structure site, with the channel base at approximately Elevation 160.0 m. The Credit River flows from north to south, with its typical level at approximately Elevation 160.5 m to 160.7 m (i.e., a water depth of approximately 0.5 m to 0.7 m at the existing bridges). Based on the design drawings for the existing bridges, the high water level at the existing bridge site is at approximately Elevation 163.8 m (October 1954).

On the *General Layout* drawing for the original bridges, dated May 1957, an approximately 5 m wide "Old Mill Race" channel is shown in the northeast quadrant of the structure site. This channel is not longer present at the site and it is assumed that it has been filled in. According to the 1957 drawing, it formerly intersected the Credit River channel at a point approximately 40 m to 45 m north of the Highway 401 centreline.

The existing Highway 401 grade declines from west to east across the structure site, from about Elevation 167.5 m near the west abutments to about Elevation 166.4 m near the east abutments. The existing Highway 401 embankments are therefore up to about 6 m in height at the west approaches, and up to about 5 m in height at the east approaches. The abutment foreslopes are oriented at approximately 1.5 horizontal to 1 vertical (1.5H:1V), and the Highway 401 embankment side slopes are oriented at approximately 2H:1V.

3.0 INVESTIGATION PROCEDURES

The field work for this subsurface investigation was carried out in September and December 2011, during which time eight boreholes (Boreholes 11-101 to 11-108) were advanced using a track-mounted CME-55 drill rig



PRELIMINARY FOUNDATION REPORT - CREDIT RIVER BRIDGES

supplied and operated by Geo-Environmental Drilling Inc. of Milton, Ontario. The borehole locations are shown on Drawing 1.

The boreholes were drilled using 108 mm inside diameter hollow stem augers through the overburden and then cored into the bedrock to depths ranging from 7.1 m to 15.5 m. Soil samples were obtained at 0.75 m and 1.5 m intervals of depth using a 50 mm outside diameter split-spoon sampler driven by an automatic hammer in accordance with the Standard Penetration Test (SPT) procedure. Bedrock core samples were obtained using an NQ-size rock core barrel.

The groundwater conditions were observed in the open boreholes during and immediately following the drilling operations, and standpipe piezometers were installed in three selected boreholes (Boreholes 11-102, 11-103 and 11-107) to permit monitoring of the groundwater level(s) at the structure site. The piezometers consist of 20 mm or 50 mm diameter PVC pipes, with slotted screens sealed within a sand filter pack at selected depth intervals within the boreholes. Above the sand filter pack and piezometer screen, the annulus surrounding each piezometer pipe was backfilled to the ground surface with bentonite pellets. The piezometer installation details and water level readings are indicated on the borehole records contained in Appendix A. The remaining boreholes were backfilled with bentonite pellets upon completion, in accordance with Ontario Regulation 903 (as amended).

The field work was supervised on a full-time basis by members of Golder's staff who located the boreholes in the field, completed utility clearances, directed the drilling, sampling, and in situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in labelled containers and transported to Golder's laboratories in Mississauga and Cambridge for further examination and laboratory testing. Index and classification tests consisting of water content determinations, Atterberg limits testing and grain size distribution analyses were carried out on selected soil samples. Strength testing (uniaxial compression and point load index testing) was carried out on selected rock core specimens. The geotechnical laboratory testing was completed according to applicable MTO LS and/or ASTM standards, as applicable.

The location of the boreholes and ground surface elevations were measured in the field by Callon Dietz, Ontario Land Surveyors. The borehole locations (referenced to the MTM NAD83 co-ordinate system) and ground surface elevations (referenced to geodetic datum) are summarized in the following table and are shown on Drawing 1.

Borehole No.	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)	Borehole Depth (m)
11-101	4,830,316.0	286,408.3	165.2	15.5
11-102	4,830,334.4	286,329.0	161.4	9.1
11-103	4,830,270.3	286,343.9	161.3	7.1
11-104	4,830,329.7	286,315.3	161.6	8.5
11-105	4,830,241.1	286,306.0	161.5	7.3
11-106	4,830,305.1	286,274.0	162.4	9.0
11-107	4,830,240.4	286,285.7	164.6	10.7
11-108	4,830,260.6	286,229.2	168.0	12.7



4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

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

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 meltwater 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.

The South Slope region slopes gradually downward towards Lake Ontario. The overburden immediately below ground surface within the South Slope generally consists of clayey silt till and silty clay till and at depth consists of alternating deposits of dense lacustrine sands and silts and overconsolidated lacustrine clays and clay tills overlying the bedrock.

4.2 Subsurface Conditions

As part of this subsurface investigation, eight boreholes were advanced in the vicinity of the existing Credit River bridges. The borehole locations, ground surface elevations and interpreted stratigraphic conditions at the site are shown on Drawing 1. The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of in situ and laboratory testing are given on the borehole records contained in Appendix A. The results of geotechnical laboratory testing are also presented on Figures B1 to B10 contained in Appendix B. The stratigraphic boundaries shown on the borehole records and on the interpreted stratigraphic sections on Drawing 1 are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

In summary, the subsoil conditions encountered at the site consist of fill overlying surficial silty sand, an upper clayey silt till deposit, or sand and gravel, which are underlain by a lower till deposit that grades in composition from a non-plastic sandy silt to silty sand, to a low plasticity clayey silt with sand. The lower till deposit is underlain by shale bedrock of the Georgian Bay Formation. A more detailed description of the soil and bedrock conditions encountered in these boreholes is provided in the following sections.



4.2.1 Topsoil

Approximately 100 mm to 300 mm of topsoil was encountered immediately below the ground surface in Boreholes 11-102, 11-103, and 11-105 to 11-107, which were advanced near the toe of the Highway 401 embankments on the east and west sides of the river, from at or near the floodplain grade.

4.2.2 Fill

Approximately 300 mm of asphalt was encountered immediately below the ground surface in Boreholes 11-101 and 11-108, which were advanced through the shoulder of the Highway 401 eastbound and westbound lanes in the southeast and northwest quadrants of the structure site, respectively.

Fill or reworked native soils were encountered below the asphalt or topsoil, or immediately below the ground surface in all boreholes with the exception of Boreholes 11-105 and 11-106. The fill material extends to depths of between 1.2 m and 3.0 m (Elevations 159.8 m to 165.0 m). The fill consists of layers of both cohesionless and cohesive materials, as follows:

- Silty sand to sandy silt (although predominantly silty sand) containing trace to some clay and trace to some gravel was encountered in Boreholes 11-101, 11-102 and 11-108. This fill has a loose to compact relative density, based on measured Standard Penetration Test (SPT) “N” values of 5 blows to 11 blows per 0.3 m of penetration.
- Cohesive fill, consisting of clayey silt with sand to some sand and trace to some gravel, was encountered in Boreholes 11-101, 11-103, 11-104, 11-107 and 11-108. This fill also contains cobbles, as noted on the borehole records. The results of grain size distribution tests completed on five selected samples of the clayey silt fill are shown on Figure B1 in Appendix B. Atterberg Limits tests were completed on five selected samples of the cohesive fill and measured plastic limits of 14 per cent to 19 per cent, liquid limits of 22 per cent to 34 per cent, and plasticity indices between 8 per cent and 15 per cent; these results, which are plotted on a plasticity chart on Figure B2 in Appendix B, confirm that the tested cohesive fill material consists of low plasticity clayey silt. The measured water contents in the cohesive fill materials range from approximately 8 per cent to 20 per cent, typically at or below the plastic limit for the material. The measured SPT “N” values within the cohesive fill range from 4 blows to 23 blows per 0.3 m of penetration, indicating a firm to very stiff consistency.

4.2.3 Surficial Silty Sand

An approximately 0.6 m to 1.4 m thick layer of silty sand was encountered below the topsoil or fill materials in Boreholes 11-104, 11-105 and 11-106, extending to depths of between 0.7 m and 2.2 m (Elevation 159.4 m to 161.7m). The silty sand contains trace to some gravel and trace to some clay.

The measured SPT “N” values within this deposit range from 7 blows to 17 blows per 0.3 m of penetration, suggesting a loose to compact relative density. The measured water contents of selected samples of the silty sand ranged from 8 per cent to 13 per cent.



4.2.4 Upper Clayey Silt Till

An approximately 1.5 m to 2.6 m thick upper deposit of clayey silt till was encountered below the fill in Boreholes 11-101, 11-107 and 11-108, extending to depths of between 3.7 m and 5.6 m (Elevation 160.7 m to 162.4 m).

This upper cohesive till deposit consists of clayey silt with sand to trace sand, and trace to some gravel. The result of a grain size distribution test completed on one selected sample of the upper clayey silt till is shown on Figure B3 in Appendix B. Atterberg Limits testing was carried out on one selected sample of the upper till deposit and measured a plastic limit of about 13 per cent, a liquid limit of about 20 per cent, and a plasticity index of 7 per cent; this result, which is plotted on a plasticity chart on Figure B4 in Appendix B, confirms that the deposit consists of low plasticity clayey silt till. The water content measured on selected samples of the clayey silt till ranged from approximately 8 per cent to 16 per cent.

The measured SPT “N” values within the upper clayey silt till deposit range from 12 blows to 40 blows per 0.3 m of penetration, suggesting a stiff to hard consistency.

4.2.5 Sand and Gravel

An approximately 0.7 m to 2.7 m thick deposit of sand and gravel was encountered below the fill, surficial silty sand or clayey silt till in Boreholes 11-101 to 11-103 on the east side of the river, and Borehole 11-105 on the west side of the river, extending to between Elevation 158.0 m and 159.3 m.

The sand and gravel deposit contains trace to some silt and trace clay. The results of grain size distribution tests completed on two selected samples of the sand and gravel are shown on Figure B5 in Appendix B. The water content measured on selected samples of the sand and gravel ranged from approximately 8 per cent to 12 per cent.

The measured SPT “N” values within the deposit range from 11 blows to 45 blows per 0.3 m of penetration, suggesting a compact to very dense relative density.

4.2.6 Lower Sandy Silt to Silty Sand Till

An approximately 1.5 m to 3.8 m thick deposit of sandy silt to silty sand till was encountered in all boreholes (with the exception of Borehole 11-105), below the surficial silty sand or sand and gravel on the east side of the Credit River, and below the surficial silty sand or upper clayey silt till on the west side of the river. The deposit rises from east to west, with the surface of the deposit encountered between Elevation 158.0 m and 159.4 m in the boreholes on the east side of the river, and between Elevation 160.9 m and 162.4 m in the boreholes on the west side of the river. The deposit base was encountered between Elevation 155.4 m and 157.3 m in the boreholes on the east side of the river, and between Elevation 158.7 m and 160.8 m in the boreholes on the west side of the river.



This till deposit varies in composition from sandy silt to sand and silt to silty sand, containing trace to some clay and trace to some gravel; portions of the till deposit contain higher proportions of gravel, and are described as “gravelly” on the borehole records. The results of grain size distribution tests completed on six selected samples of the cohesionless till are shown on Figure B6 in Appendix B. The water content measured on selected samples of the sandy silt to silty sand till ranged from approximately 7 per cent to 12 per cent.

The measured SPT “N” values within the sandy silt to silty sand till range from 31 blows to greater than 100 blows per 0.3 m of penetration, indicating a dense to very dense relative density.

4.2.7 Lower Clayey Silt Till

The silty sand to sandy silt till deposit grades with depth to clayey silt till in Boreholes 11-101, 11-106 and 11-108; this lower clayey silt till was also encountered below the sand and gravel deposit in Borehole 11-105. The clayey silt portion of the lower till deposit is approximately 1.5 m to 2.1 m thick, and where present generally rises from the east to the west, with its surface encountered between Elevation 156.5 m and 160.8 m, and its base encountered between Elevation 155.0 m and 158.8 m.

The cohesive portion of the lower till deposit consists of clayey silt with sand, containing trace to some gravel. The results of grain size distribution tests completed on two selected samples of the lower clayey silt till are shown on Figure B7 in Appendix B. Atterberg limits testing was carried out on two selected samples of the deposit and measured plastic limits of about 12 per cent and 13 per cent, liquid limits of about 17 per cent and 18 per cent, and plasticity indices of about 5 per cent; these test results, which are plotted on a plasticity chart on Figure B8 in Appendix B, confirm that this portion of the till deposit consists of low plasticity clayey silt. The water content measured in selected samples of the clayey silt till ranged from approximately 6 per cent to 10 per cent, below the plastic limit for the material.

The measured SPT “N” values within the lower clayey silt till range from 20 blows to greater than 100 blows (but are predominantly greater than 100 blows) per 0.3 m of penetration, indicating a very stiff to hard (but predominantly hard) consistency.

4.2.8 Shale Bedrock

Bedrock was encountered below the lower till deposit and cored at each of the borehole locations. The bedrock surface elevation generally rises from east to west, from about Elevation 155.0 m to 158.8 m, as summarized in the following table:

Borehole No.	Depth to Bedrock Surface (m)	Elevation of Bedrock Surface (m)
11-101	10.2 (Weathered) 12.2 (“Sound”)	155.0 (Weathered) 153.0 (“Sound”)
11-102	6.0	155.4
11-103	4.2	157.1
11-104	4.3 (Weathered) 5.1 (“Sound”)	157.3 (Weathered) 156.5 (“Sound”)



PRELIMINARY FOUNDATION REPORT - CREDIT RIVER BRIDGES

Borehole No.	Depth to Bedrock Surface (m)	Elevation of Bedrock Surface (m)
11-105	4.3	157.2
11-106	5.3 (Weathered) 5.4 ("Sound")	157.1 (Weathered) 157.0 ("Sound")
11-107	5.9	158.7
11-108	9.3	158.8

The bedrock at this site consists of grey to black shale of the Georgian Bay Formation, which is known to contain stronger limestone interbeds. The upper 0.1 m to 2.0 m of the bedrock as encountered in Boreholes 11-101, 11-104 and 11-106 is considered to be moderately to highly weathered, based on the ability to penetrate this portion of the bedrock at these locations by augering and split-spoon sampling. Based on the recovered core samples, the bedrock is generally described as slightly to moderately weathered, laminated, and weak to medium strong, containing strong fossiliferous limestone interbeds. The Rock Quality Designation (RQD) measured on the core samples is between about 7 per cent and 88 per cent, but typically greater than 35 per cent, indicating a rock mass of fair to good quality. The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of the core samples are typically between 41 per cent and 100 per cent and 16 percent and 91 per cent, respectively.

Point load strength tests were performed on selected core samples. Diametral point load strength index values are shown on the Record of Drillhole Sheets and are summarized in Table B3 in Appendix B. The point load index (Is_{50}) results from diametral laboratory tests on five samples of the shale bedrock range from approximately 0.2 MPa to 5.7 MPa, and the Is_{50} results from axial laboratory tests carried out on seven samples of the shale bedrock range from approximately 1.6 MPa to 11.4 MPa. These point load test results correspond to estimated unconfined compressive strengths of approximately 4 MPa to 262 MPa, as shown on Table B2 in Appendix B.

Unconfined compressive strength (UCS) tests were carried out on two selected specimens of the bedrock obtained from Boreholes 11-102 and 11-106, and measured values of about 21 MPa and 100 MPa, respectively, as summarized on Tables B1 and B2 in Appendix B. Photographs of two bedrock core sample before and after UCS testing are shown on Figures B9 and B10 in Appendix B.

Based on the laboratory UCS test and point load test results as summarized in Tables B1 to B3 in Appendix B, the estimated intact strength of the shale bedrock is anticipated to be very weak to medium strong with strong to very strong limestone interbeds, excluding the upper moderately to highly weathered zones of the bedrock.

4.3 Groundwater Conditions

Details of the water conditions observed in the open boreholes at the time of drilling are summarized on the borehole records in Appendix A. Standpipe piezometers were installed in Boreholes 11-102, 11-103 and 11-107 to monitor the groundwater levels at the site. The water levels measured in the piezometers are summarized in the following table:



PRELIMINARY FOUNDATION REPORT - CREDIT RIVER BRIDGES

Borehole No.	Ground Surface Elevation (m)	Depth to Water Level (m)	Groundwater Elevation (m)	Date
11-102	161.4 m	1.0 m	160.4 m	January 20, 2012
11-103	161.3 m	1.3 m*	160.0 m	December 20, 2012
		1.0 m	160.3 m	January 20, 2012
11-107	164.6 m	4.0 m*	160.6 m	September 9, 2011
		3.1 m	161.5 m	November 2, 2011
		3.0 m	161.6 m	January 20, 2012

* Measured in the piezometer shortly after completion of drilling and piezometer installation.

The groundwater level should be expected to fluctuate seasonally and should be expected to rise during wet periods of the year.

5.0 CLOSURE

This Preliminary Foundation Investigation Report was prepared by Mr. Chris Sternik and Mr. Mehdi Mostakhdemi, M.Sc., M.Eng., and reviewed by Ms. Lisa Coyne, P.Eng., a senior geotechnical engineer and Principal with Golder. Mr. Ty Garde, P.Eng., a Designated MTO Foundations Contact for Golder, conducted an independent review of this report.

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PART B

**PRELIMINARY FOUNDATION DESIGN REPORT
CREDIT RIVER BRIDGES
HIGHWAY 401 WIDENING
FROM EAST OF THE CREDIT RIVER TO TRAFALGAR ROAD
REGIONAL MUNICIPALITIES OF PEEL AND HALTON
W.O. 07-20021**



6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides preliminary foundation design recommendations for the proposed replacement of the existing Highway 401 bridges over the Credit River. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during this preliminary subsurface investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the preliminary design of the structure foundations. Further investigation and analysis will be required during detail design.

Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project, and for which special provisions may be required in the Contract Documents. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.2 Foundation Options

Based on the planning study completed to date for the widening of Highway 401 from east of Credit River to Trafalgar Road, it is understood that the future widening could consist of four and three additional lanes in the eastbound and westbound directions, respectively, on Highway 401. It is further understood that the existing 68 m long Credit River bridges will require replacement, and either single-span bridges or three-span bridges (with the new piers placed further from the edge of the creek) are under consideration.

The existing structures consist of three-span bridges, with the existing abutments and piers supported on spread footings that are founded on and dowelled to the shale bedrock. Based on the *General Layout* drawing for the original structure, dated May 1957, and the *General Arrangement, East and West Abutment Details*, and *Pier Details* drawings for the median widening of the existing structures, dated October 1976, the existing founding conditions are summarized as follows:

Foundation Element	Footing Width	Founding Elevation
West abutments	4.0 m (13.0 ft.)	157.7 m
West piers	1.8 m (6.0 ft.)	Approx. 156.5 m (founded 0.8 m into rock)
East piers	1.8 m (6.0 ft.)	Approx. 157.3 m (founded 0.8 m into rock)
East abutments	4.0 m (13.0 ft.)	156.2 m

Along with the future widening of Highway 401, the pavement grade may be raised by up to 1.5 m above the current grade (i.e., up to approximately Elevation 169 m and 168 m near the west and east abutments, respectively). This would require placement of up to about 7.5 m and 6.5 m of new fill at the west and east approaches, respectively.



Based on the subsurface conditions at this site, both shallow and deep foundation options have been considered for support of the abutments and piers for the new Credit River bridges. A summary of the advantages and disadvantages associated with each option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, risks and approximate costs is provided in Table 1 following the text of this report.

- **Spread footings founded on the dense to very dense/hard till:** Spread footings founded on the dense to very dense silty sand to sandy silt till or very stiff to hard clayey silt till are feasible for support of the new abutments and associated wingwalls/retaining walls. Spread footings founded on the native soil are not recommended at the piers, if a multi-span structure is adopted, due to the potential for scour and undermining. Temporary protection systems would be required to facilitate excavation through the existing Highway 401 embankments and the native soils, and dewatering will be required in the surficial silty sand, sand and gravel, and cohesionless till deposits.
- **Spread footings founded on the shale bedrock:** Spread footings founded on the shale bedrock, similar to the existing structure foundations, are feasible for support of the new abutments, associated wingwalls/retaining walls, and piers. Deeper temporary protection systems would be required as compared with any of the other foundation options; however, this option is advantageous over spread footings founded on the till deposit in terms of minimizing settlement at each foundation element. If a multi-span structure is adopted for the replacement, this option is considered advantageous at the piers in terms of protection against scour and erosion. This option would require the deepest excavation, with associated temporary protection systems, and dewatering and/or cofferdams.
- **Footings “perched” on a compacted granular pad in the existing approach embankments:** This option would be advantageous in minimizing the depth of excavation and groundwater control requirements as compared with footings founded on native soil or bedrock. However, due to the presence of firm to stiff and/or loose fill materials in the existing approach embankments, there is potential for 25 mm or more of settlement under the abutment loading; in a multi-span structure configuration, if the piers are founded on the bedrock, there is potential for at least 25 mm of differential settlement between the abutments and piers. Therefore, this foundation option is not recommended for the new abutments.
- **Steel H-piles driven to found on the shale bedrock:** Driven steel H-piles are suitable and feasible for support of new abutments (and would permit integral abutment design) and associated wingwalls/retaining walls at this site. Due to the relatively shallow depth to bedrock at the pier locations (within approximately 4.5 m below the floodplain level), steel H-piles are not recommended for support of the new piers. At the abutments, it is assumed that the new pile caps would be “perched” within the approach embankments above the floodplain grade, thus minimizing the depth of excavation and associated requirements for temporary protection systems and dewatering. There is a relatively minor risk associated with penetrating through or the piles “hanging up” on cobbles or boulders (although further investigation is recommended in this regard at the detail design stage).
- **Steel pipe piles driven to found on the shale bedrock:** Driven steel pipe piles could also be considered as a deep foundation option for support of new abutments (and would permit semi-integral abutment design) and associated wing walls/retaining walls at this site; pipe piles are not recommended for support of the new piers due to the relatively shallow depth to bedrock. It is assumed that the abutment pile caps would be “perched” within the Highway 401 approach embankments, minimizing the depth of excavation



and associated requirements for temporary protection and dewatering. Pipe piles are considered to have a slightly higher risk than H-piles for “hanging up” or being deflected away from their vertical or battered orientation due to the presence of cobbles and/or boulders within the glacial and glacio-fluvial soils at this site.

- **Caissons founded in the shale bedrock:** Caissons founded in the shale bedrock are feasible for support of the new abutments (although they would preclude integral abutments) and potentially for support of new piers at this site. The use of caissons for support of the new piers may reduce the overall depth of bedrock excavation and dewatering requirements as compared with a spread footing option. However, temporary or permanent liners would be required during caisson construction given the risk of running/flowing soil when excavating through the water-bearing silty sand, sand and gravel and/or cohesionless till deposits. In addition, coring and/or churn drilling techniques are expected to be required to penetrate into the bedrock to the target founding level.

The following sections provide recommendations for spread footings, driven steel H-pile or pipe pile foundations, and caisson foundations to support the proposed bridge replacement and widening. Based on the subsurface conditions at the site and the above considerations, and considering the satisfactory performance of the existing structure and its foundations, the preferred option from a geotechnical/foundations perspective is to support the abutments for the new structure on steel H-piles driven to found on the shale bedrock, in an integral abutment configuration. If three-span replacement structures are adopted, the preferred foundation option from a geotechnical/foundations perspective is to support the piers on shallow footings founded on the shale bedrock.

6.3 Shallow Foundations

6.3.1 Founding Elevations

For support of the new abutments, associated wing walls/retaining walls, strip or spread footings should be founded below any fill and firm to stiff/compact native soils, on either the dense to very dense/very stiff to hard till, or the shale bedrock. The footing founding elevation should be a minimum of 1.2 m below the lowest surrounding grade to provide adequate protection against frost penetration. For preliminary design purposes, the maximum (highest) founding elevations for spread footings may be taken as given in the table below. It is noted that the soils at the site will be susceptible to erosion and scour, and deeper founding levels may be necessary at the piers and potentially at the abutments to provide adequate protection against scour under the design hydraulic conditions.

Foundation Element	Founding Elevation (m)	
	WBL Bridge	EBL Bridge
West abutments	160.0 (Till) 157.0 (Shale)	160.0 (Till) 158.5 (Shale)
West piers	157.0 (Shale)	157.0 (Shale)
East piers	156.5 (Shale)	157.0 (Shale)
East abutments	159.0 (Till) 155.0 (Shale)	159.0 (Till) 157.0 (Shale)



The founding soils or shale bedrock will be susceptible to disturbance. If the concrete for the footings cannot be poured immediately after excavation and inspection, it is recommended that a concrete working slab be placed on the prepared subgrade within four hours of its inspection and approval, as discussed further in Section 6.7.

Scour protection will be required, and this is discussed further in Section 6.7.

6.3.2 Geotechnical Resistance/Reaction

Strip or spread footings placed on the properly prepared till or shale bedrock, at or below the design elevations given in the preceding section, should be designed based on the following factored geotechnical resistances at Ultimate Limit States (ULS) and geotechnical resistances at Serviceability Limit States (SLS). These values assume a footing width of approximately 3 m to 4 m.

Founding Stratum	Factored Geotechnical Resistance at ULS	Geotechnical Reaction at SLS
Dense to very dense/ very stiff to hard till	500 kPa	400 kPa*
Shale bedrock	800 kPa	600 kPa**

* For 25 mm of settlement where supported on till deposit

** For 10 mm of settlement where supported on shale bedrock

The preliminary geotechnical resistances should be reviewed if the selected footing width or founding elevation differs from those given above. In addition, these preliminary geotechnical resistances are provided for loads applied perpendicular to the surface of the footings; where applicable, inclination of the load should be taken into account in accordance with Section 6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC 2006)* and its *Commentary*.

The preliminary geotechnical resistance values provided above will have to be re-evaluated and modified as necessary during detail design, based on future additional subsurface investigation at the proposed new foundation elements.

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

6.4.1 Founding Elevations

For preliminary assessment purposes, it has been assumed that if pile foundations are adopted for support of the west and east abutments, the pile caps would be perched within the Highway 401 approach embankments above the natural ground surface.

The new abutments and associated wingwalls may be supported on steel H-piles or steel pipe (tube) piles driven to found on or in the shale bedrock. The surface elevation for the shale bedrock and the thickness of the upper, moderately to highly weathered zone varies in the boreholes, and further investigation will be required at the detail design stage to confirm these preliminary founding elevations. The following pile tip elevations may be



used for preliminary design purposes, assuming penetration through the upper, moderately to highly weathered shale bedrock, and termination on or just into the slightly weathered portion of the bedrock:

Foundation Element	Borehole Nos.	Estimated Design Pile Tip Elevation (m)	
		WBL Bridge	EBL Bridge
West abutments	11-105 11-106 11-107	157.0	157.0
East abutments	11-102 11-103 11-104	155.5	157.0

Assuming that the pile caps are constructed at a minimum depth of 1.2 m for frost protection purposes, it is anticipated that the underside of the new pile caps would be at approximately Elevation 165 m and 164 m at the west and east abutments, respectively. Based on the bedrock surface elevation and estimated design pile tip elevation as given above, a minimum pile length of 5 m (for integral abutment design) is achievable at this site, and pre-augering/pre-coring into the bedrock should not be required.

For the installation of steel H-piles or steel pipe piles, consideration must be given to the potential presence of cobbles and boulders within the soil deposits. In this regard, steel H-piles are preferred over steel pipe piles as pipe piles are considered to pose a higher risk of “hanging up” or being deflected away from their vertical or battered orientation during installation, due to their larger end area. The piles should be reinforced at the tip with driving shoes to reduce the potential for damage to the piles during driving.

As discussed further in Section 6.7 (Construction Considerations), vibration monitoring is not anticipated to be required during deep foundation construction activities, either for the existing bridges or for the nearest buildings.

6.4.2 Axial Geotechnical Resistance/Reaction

For preliminary design for HP 310x110 piles driven to the estimated tip elevations provided in Section 6.4.1, the factored axial geotechnical resistance at ULS may be taken as 1,600 kN, and the axial geotechnical reaction at SLS (for approximately 10 mm of settlement) may be taken as 1,400 kN. Similar axial resistances may be used in the design of closed-end, concrete-filled, 324 mm (12 ¾ in.) diameter steel pipe piles having a minimum wall thickness of 9.5 mm (3/8 in.).

The preliminary geotechnical resistances provided above will have to be re-evaluated and modified as necessary during detail design in consideration of the additional subsurface investigation at the new foundation elements.

6.5 Caissons

As an alternative to steel H-piles or pipe piles, caissons could be considered for support of the new abutments. Caissons may also be considered as an alternative to spread footings founded on the shale bedrock for the new



piers, if multi-span replacement structures are adopted. The use of caissons for support of new piers may reduce the overall depth of bedrock excavation and dewatering requirements as compared with a spread footing option. However, temporary or permanent liners will be required during caisson construction because of the water-bearing cohesionless soils that are present at this site. For the installation of caissons, consideration must be given to the potential presence of cobbles and boulders within the soil deposits.

6.5.1 Founding Elevations

As the surface of the bedrock varies based on the borehole results, and to accommodate some weathering in the upper portion of the bedrock, socketting into the bedrock is recommended. The table below provides caisson founding levels for preliminary design.

Foundation Element	Borehole Nos.	Bedrock Surface Elevation (m)	Design Caisson Founding Elevation (m)	
			WBL Bridge	EBL Bridge
West abutments	11-106, 11-107	157.1 to 158.7	156.0	157.5
West piers	11-105, 11-106	157.1 to 157.2	156.0	156.0
East piers	11-103, 11-104	157.1 to 157.3	155.5	156.0
East abutments	11-102, 11-103	155.4 to 157.1	153.5	156.0

The shale bedrock is weak to medium strong, with unconfined compressive strengths in the range of 5 MPa to 50 MPa, and it is expected that the sockets would have to be advanced into the rock by coring and/or churn drilling.

6.5.2 Axial Geotechnical Resistance/Reaction

For preliminary design, caissons socketted approximately 1 m or more into the shale bedrock should be designed based on end-bearing resistance, using a factored axial geotechnical resistance at ULS of 5 MPa; for a 1.2 m diameter caisson, this would equate to a factored axial geotechnical resistance at ULS of 4,400 kN. The geotechnical reaction at SLS (for less than 15 mm of settlement) may be taken as 3,300 kN.

6.6 Approach Embankments

6.6.1 Subgrade Preparation and Embankment Construction

It is recommended that all topsoil/organic material or existing surficial fill materials be stripped from the footprint of the widened Highway 401 approach embankments. The depth and extent of stripping should be assessed during detail design when additional subsurface information will be available for the widened approach embankment areas.

Additional fill for construction of the embankment widening could consist of clean earth fill or granular fill. Benching of the north and south sides of the existing Highway 401 embankment should be carried out to “key in”



the new fill materials for the realignment/widening, in accordance with OPSD 208.010 (*Benching of Earth Slopes*).

In accordance with MTO's standard practice, a minimum 2 m wide bench should be provided where the embankment side slopes are equal to or greater than 8 m in height, such that the uninterrupted slope height does not exceed 8 m. To reduce erosion of the embankment side slopes due to surface water runoff, placement of topsoil and seeding or pegged sod is recommended as soon as practicable after construction of the embankments.

6.6.2 Approach Embankment Stability

Preliminary slope stability analyses have been performed for the proposed widened/new approach embankments using the commercially available program SLIDE, produced by Rocscience Inc., to check that a minimum factor of safety of 1.3 is achieved for the proposed embankment heights and geometries under static conditions. This minimum factor of safety is considered appropriate for the proposed bridge replacement on this project, considering the design requirements and the available field and laboratory testing data.

The preliminary stability analyses were completed for a maximum 7.5 m high approach embankment, based on the subsurface conditions as encountered in Boreholes 11-101 to 11-108. The following parameters have been used in the preliminary analyses, based on field and laboratory test data as well as accepted correlations:

Soil Deposit	Bulk Unit Weight (kN/m ³)	Effective Friction Angle
Embankment fill	21	34°
Loose to compact silty sand	19	30°
Compact to very dense sand and gravel	20	32°
Dense to very dense sandy silt to silty sand till	21	34°
Very stiff to hard clayey silt till	21	34°

The preliminary stability analysis results indicate that a 7.5 m high embankment with side slopes no steeper than 2H:1V will have a factor of safety of at least 1.3 against global instability, assuming appropriate subgrade preparation and proper placement and compaction of the embankment fill materials. An example of the results from the static global stability analyses is provided on Figure 1. This preliminary assessment of the stability of the approach embankments should be reviewed and confirmed based on the additional borehole information obtained within the proposed footprint for the widened Highway 401 approach embankments during detail design.



6.6.3 Approach Embankment Settlement

The new Credit River bridges are proposed to be constructed at the location of the existing structures, with widening of three and four lanes in the north and south directions, respectively.

Preliminary settlement analyses for the anticipated soil conditions below the new/widened approach embankments were carried out using the commercially available computer program *Settle-3D* from Rocscience, using estimated elastic deformation moduli as given in the table below, based on correlations with the SPT “N” values, undrained shear strengths, Atterberg limits testing and engineering judgement from experience with similar soils in this region of Ontario (Bowles, 1984; Kulhawy and Mayne, 1990; Peck et al., 1974).

Soil Deposit	Bulk Unit Weight (kN/m ³)	Elastic Modulus (MPa)
Embankment fill	21	-
Loose to compact silty sand	19	15
Compact to very dense sand and gravel	20	20-50
Dense to very dense sandy silt to silty sand till	21	75
Very stiff to hard clayey silt till	21	75

Based on this preliminary settlement assessment, the settlement of the foundation soils under the 1.5 m grade raise over the existing embankment plus the north and south widening of 6.5 m to 7.5 m high approach embankments is estimated to be up to about 40 mm. The majority of the anticipated settlement is expected to occur relatively quickly during and immediately following construction of the widened approach embankments. This estimated magnitude and duration of settlement should be reassessed following additional investigation during detail design.

The above preliminary settlement 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 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.

6.7 Construction Considerations

The following subsections identify future construction considerations that should be considered at this stage as they may impact the planning and preliminary design. Where applicable, Non-Standard Special Provisions (NSSP) should be developed during the future detail design stage for incorporation in the Contract Documents.



6.7.1 Excavation and Temporary Protection Systems

The foundation excavations for spread footings would extend through the existing fill, loose to compact silty sand, stiff to very stiff clayey silt till, and compact sand and gravel, into dense to very dense sandy silt to silty sand till. If space permits, open-cut excavations into these materials should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. The existing fill and loose to compact/stiff soils should be classified as Type 3 soil, according to the OHSA. Temporary excavations (i.e. those that are open for a relatively short time period) through these materials should be made with side slopes no steeper than 1H:1V, assuming that appropriate groundwater control is in place.

It is anticipated that depending on traffic staging during construction, temporary protection systems will be required along Highway 401, as well as surrounding the excavations for the removal of the existing abutments and construction of the new abutments. If new pier locations can be established so that they do not conflict with the existing, and if it is feasible from a structural and environmental perspective, it is recommended that the existing pier foundations be left in place (i.e., removed only down to the top of footing) to minimize excavation and groundwater control requirements in close proximity to the river. However, it is anticipated that cofferdams will still be required for excavations for spread footings for new piers, as well as for removal of the existing pier columns, due to the proximity to the Credit River.

The selection and design of the protection system will be the responsibility of the Contractor. However, for conceptual/planning purposes, the temporary protection systems should be designed and constructed in accordance with OPSS 539 (*Construction Specification for Temporary Protection Systems*). The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS 539. 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 the abutments. At the piers, an interlocking sheetpile system would contribute to both ground and groundwater control; a soldier pile and lagging system is less practical for the pier excavations, as it would be necessary to control seepage or include measures to mitigate loss of soil particles through the lagging boards.

6.7.2 Groundwater Control

While new abutment pile caps would be maintained above the groundwater level at the site, excavations for new spread footings at the piers would extend below the groundwater level, into the water-bearing silty sand, sand and gravel, and sandy silt to silty sand till. Some water seepage is also anticipated from the base of granular fills ("perched" on top of underlying clayey silt soils).

If new pier locations can be established so that they do not conflict with the existing foundations, and if it is feasible from a structural and environmental perspective, it is recommended that the existing pier foundations be left in place (i.e., removed only down to the top of footing) to minimize excavation and groundwater control requirements in close proximity to the river.

Due to the proximity of the piers to the edge of the Credit River, a groundwater cut-off (cofferdam or similar measure) is recommended to minimize dewatering requirements and potential environmental impacts. A cut-off/cofferdam could consist of interlocking steel sheetpiles driven to the bedrock surface. The bedrock surface elevation is expected to vary slightly within the pier areas, and depending on the thickness and degree of



weathering of the upper portion of the bedrock, it is possible that “gaps” may exist at the base of some of the sheetpile sections at the bedrock interface. In addition, there is potential for groundwater seepage in the upper weathered or fractured portion of the shale bedrock. Measures would be required to control groundwater seepage and prevent loss of soil through these gaps during excavation at and below the bedrock interface.

6.7.3 Bedrock Excavation and/or Socket Formation

The upper portion of the shale bedrock, as encountered in some of the boreholes, is weathered. The “sound” shale bedrock at the site is weak to medium strong (corresponding to unconfined compressive strengths in the range of 5 MPa to 50 MPa), and contains strong limestone interbeds. Where excavation into sound bedrock is required, it is expected that hoe-ramming techniques will be needed to reach the design founding elevation. It is recommended that an NSSP be developed at the detail design stage and included in the Contract Documents to warn the contractor that the bedrock at the site is weak to medium strong, that excavation into the bedrock will require appropriate equipment and construction procedures, and that the bedrock excavation must not disturb the existing bridge footings.

Alternatively, if caissons are the selected foundation option and rock sockets are required to provide the necessary foundation capacity, it is recommended that an NSSP be included in the Contract Documents to warn the Contractor that the shale bedrock is weak to medium strong and contains strong limestone interbeds. Further, it is expected that socket formation would require coring or churn drilling to advance the hole.

6.7.4 Subgrade Protection

The till deposit and/or shale bedrock that will be exposed at the foundation subgrade level will be susceptible to disturbance from construction traffic and/or ponded water. To limit this degradation, it is recommended that a 100 mm thick concrete working slab (of 20 MPa concrete) be placed on the subgrade within four hours after preparation, inspection and approval of the footing subgrade. This requirement can be addressed with a note on the General Arrangement drawing and with an NSSP, which can be developed during the detail design stage.

6.7.5 Obstructions

The soils at this site are glacially or glacio-fluvially derived and as such should be expected to contain cobbles and boulders, which could affect the installation of deep foundations. Further observation is recommended in the next stage of investigation in support of the detail design. If conditions warrant, an NSSP should be included in the Contract Documents developed during the detail design stage to identify to the contractor the possible presence of cobbles and/or boulders within the overburden soils.

6.7.6 Scour Protection

The existing soils in the abutment and pier areas will be susceptible to erosion and scour under the design flooding conditions. In addition, the shale bedrock in which the pier footings will be founded could experience some erosion/scour throughout the design life of the structure if the overlying soils are eroded.



It is recommended that scour protection be provided on the river channel banks and around the piers. For conceptual/preliminary planning purposes, the riprap could consist of a minimum 800 mm thick layer of 200 mm to 400 mm diameter stone, placed on the surface of the shale bedrock (as the scour analyses show that the maximum scour depth extends to this level) and well compacted to the design ground level. The riprap should extend from the channel banks to 1 m above the design flood level at the structure site.

This proposed riprap treatment is similar to the riprap protection for the existing structures, which based on visual observation appears to have performed satisfactorily to the time of the current investigation.

6.7.7 Vibration Monitoring During Construction

A maximum peak particle velocity (PPV) of 100 mm/s is generally considered applicable for bridge structures in good condition. Based on vibration monitoring experience, it is considered unlikely that vibrations induced by conventional construction activities such as pile driving, coring/churn drilling, or hoe-ramming will reach this threshold level and, therefore, vibration monitoring for the existing bridges is not expected to be required during construction at this site.

Existing commercial buildings are located to the west of the structure site, approximately 250 m to 300 m from the Credit River bridges. Although a lower PPV threshold of 50 mm/s is generally considered applicable for vibration impacts on buildings, the construction zone of influence would be less than 200 m, and likely less than 100 m. Therefore, vibration monitoring is not expected to be required at the existing buildings west of the bridge site.

6.8 Recommendations for Further Work During Detail Design

Additional boreholes will be required within each of the foundation elements and within the approach embankment areas during the future detail design stage of investigation, to further assess and/or confirm the subsurface conditions and the preliminary recommendations provided herein, as follows:

- Abutments and piers:
 - Assessment of the bedrock surface elevation and thickness of highly weathered shale to confirm the founding elevation for spread footings, and the tip elevation for driven piles or caissons (if adopted)
 - Further assessment of the presence/extent and permeability of any cohesionless soil deposits that will impact groundwater control requirements for foundation excavations.
 - Observation of the presence of cobbles and/or boulders within the soil deposits, to assess the need for an NSSP to warn the contractor of the presence of such obstructions as they may affect excavations and the installation of driven steel H-pile foundations.
- Approach embankments:
 - Assessment of the depth and extent of stripping of topsoil/organics, fill materials and loosened or softened native soils within the footprint of the new approach embankments.



PRELIMINARY FOUNDATION REPORT - CREDIT RIVER BRIDGES

- Further assessment of the thickness and consolidation/elastic compression properties of the soils within the footprint of the widened approach embankments, to confirm the settlement estimates.

7.0 CLOSURE

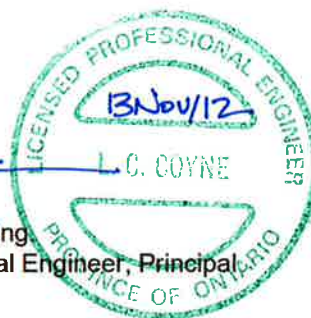
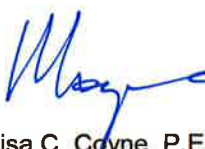
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Ontario Provincial Standard Specifications (OPSS)

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| OPSS 501 | Construction Specification for Compacting |
| OPSS 539 | Construction Specification for Temporary Protection Systems |
| OPSS 572 | Construction Specification for Seed and Cover |
| OPSS 902 | Construction Specification for Excavating and Backfilling Structures |
| OPSS 903 | Construction Specification for Deep Foundations |
| OPSS 1010 | Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material |

Ontario Provincial Standard Drawings (OPSD)

- | | |
|---------------|--|
| OPSD 3000.100 | Foundation Piles – Steel H-Pile Driving Shoe |
| OPSD 3090.101 | Foundation Frost Penetration Depths for Southern Ontario |

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

- | | |
|-----------|---|
| SP 105S21 | Amendment to OPSS 501 – Construction Specification for Compacting |
| SP 206S03 | Earth Excavation and Grading |



PRELIMINARY FOUNDATION REPORT - CREDIT RIVER BRIDGES

**TABLE 1 – COMPARISON OF FOUNDATION OPTIONS
CREDIT RIVER BRIDGE**

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Spread/strip footings on dense to very dense sand and silt till or very stiff to hard clayey silt till	<ul style="list-style-type: none"> • Feasible for support of new abutments and associated wingwalls/ retaining walls • Not recommended for new piers due to potential for scour and undermining 	<ul style="list-style-type: none"> • This option allows abutment foundations to be higher than if founded on shale, thus reducing the excavation depth, temporary protection system requirements and groundwater control requirements as compared to the bedrock option • Allows for semi-integral abutments 	<ul style="list-style-type: none"> • Significant excavations (to a depth of approximately 7.5 m below Highway 401 grade); temporary protection systems will be required • Groundwater control will be required in the silty sand and sand and gravel; some seepage is also anticipated from the sand and silt till • Lower geotechnical resistances as compared with footings on shale or deep foundations • Precludes use of integral abutments; potentially greater maintenance required at abutments 	<ul style="list-style-type: none"> • Conventional excavation and construction techniques 	<ul style="list-style-type: none"> • Anticipated to be less expensive than deep foundations, although structure maintenance costs may be higher due to non-integral abutment configuration • Estimated cost is approximately \$600/m³, excluding temporary protection system
Spread/strip footings founded on shale bedrock	<ul style="list-style-type: none"> • Feasible for support of new abutments, wingwalls/ retaining walls, and piers 	<ul style="list-style-type: none"> • Existing structure supported on shallow foundations bearing on bedrock, and has performed well • Minimizes total settlement and differential settlement between foundation elements • Allows for appropriate protection against scour/erosion at the piers 	<ul style="list-style-type: none"> • Deeper excavations and temporary protection systems would be required compared to any of the other foundation options • Groundwater control requirements expected to be more significant than for any other options • Some excavation into weak to medium strong bedrock may be required 	<ul style="list-style-type: none"> • Conventional excavation and construction techniques in soil; hoe-ramming may be required for limited excavation into weak to medium strong shale bedrock 	<ul style="list-style-type: none"> • Estimated cost for the footings itself is approximately \$600/m³; however, additional costs will apply for temporary protection systems, dewatering and bedrock excavation



PRELIMINARY FOUNDATION REPORT - CREDIT RIVER BRIDGES

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Spread/strip footings perched on compacted granular pad in approach embankment fill	<ul style="list-style-type: none"> Not recommended for support of new abutments due to presence of firm to stiff and/or loose fill materials in existing approach embankments 	<ul style="list-style-type: none"> Abutment footings could be maintained higher than footings founded on till deposit or shale bedrock, reducing excavation depth and associated requirements for temporary protection systems and groundwater control 	<ul style="list-style-type: none"> Potential for differential settlement between abutments and piers due to settlement of soils under perched abutment and widened/raised embankment loading Precludes use of integral abutments; potentially greater maintenance required at abutments 	<ul style="list-style-type: none"> Conventional excavation and construction techniques 	<ul style="list-style-type: none"> This option not recommended
Steel H-piles driven to found on shale bedrock	<ul style="list-style-type: none"> Feasible for support of abutments and wingwalls/retaining walls Not feasible for support of new piers due to shallow depth to bedrock 	<ul style="list-style-type: none"> Pier pile caps could be maintained higher than footings founded on till or shale bedrock, reducing depth of excavation and temporary protection system requirements adjacent to Highway 401 Limited groundwater control required Allows for integral abutment construction 	<ul style="list-style-type: none"> Minor risk of encountering obstructions (cobbles and/or boulders) during pile driving; this could result in piles "hanging up" and lower geotechnical resistances 	<ul style="list-style-type: none"> Conventional construction methods 	<ul style="list-style-type: none"> Lower relative cost compared with caisson option Estimated cost is approximately \$250/m length for pile installation and \$600/m³ for pile cap construction, plus cost of any temporary protection systems




PRELIMINARY FOUNDATION REPORT - CREDIT RIVER BRIDGES

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Steel pipe (tube) piles, driven to found on shale bedrock	<ul style="list-style-type: none"> • Feasible for support of new abutments and wingwalls/ retaining walls • Not feasible for support of new piers due to shallow depth to bedrock 	<ul style="list-style-type: none"> • Pier pile caps could be maintained higher than footings founded on till or shale bedrock, reducing depth of excavation and temporary excavation support requirements adjacent to Highway 401 • Limited groundwater control required • Allows for semi-integral abutment construction 	<ul style="list-style-type: none"> • Greater risk than for steel H-pile foundations of encountering obstructions (cobbles and/or boulders) during driving; this could result in piles “hanging up” and lower geotechnical resistances 	<ul style="list-style-type: none"> • Conventional construction methods 	<ul style="list-style-type: none"> • Costs for steel pipe (tube) piles similar to but slightly higher than those for H-piles
Caissons founded in shale bedrock	<ul style="list-style-type: none"> • Feasible for support of new abutments • Potentially feasible for support of new piers 	<ul style="list-style-type: none"> • Abutment pile caps could be maintained higher than footings founded on till deposit, reducing depth of excavation and temporary excavation support requirements adjacent to Highway 401 embankment • At piers, may reduce the overall depth of bedrock excavation and dewatering requirements as compared with a spread footing option 	<ul style="list-style-type: none"> • Temporary or permanent liners would be required due to risk of running/flowing soils in water-bearing silty sand, sand and gravel, and sand and silt till deposits • Coring and/or churn drilling techniques required to penetrate into the shale bedrock • Precludes use of integral abutments 	<ul style="list-style-type: none"> • Conventional construction methods with temporary liners required 	<ul style="list-style-type: none"> • Higher cost compared with shallow foundations or steel H-piles

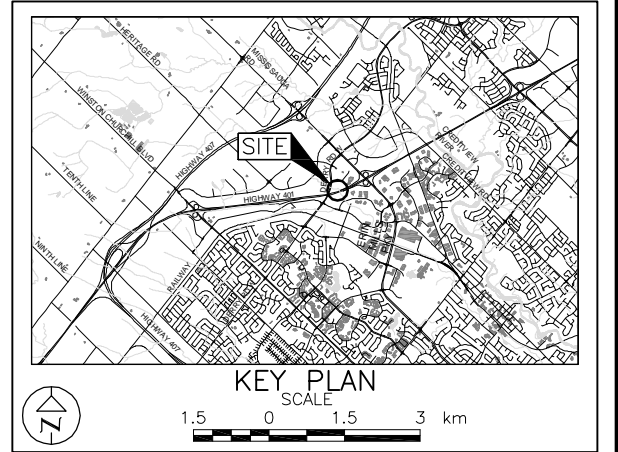
METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No.
WO No. 07-20021






CREDIT RIVER BRIDGES
HIGHWAY 401 WIDENING
BOREHOLE LOCATIONS

SHEET

**Golder Associates Ltd.**
MISSISSAUGA, ONTARIO, CANADA



LEGEND

-  Borehole - Current Investigation
-  Seal
-  Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
-  WL in piezometer, measured on January 20, 2012
-  WL upon completion of drilling

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
11-101	165.2	4830316.0	286408.3
11-102	161.4	4830334.4	286329.0
11-103	161.3	4830270.3	286343.9
11-104	161.6	4830329.7	286315.3
11-105	161.5	4830241.1	286306.0
11-106	162.4	4830305.1	286274.0
11-107	164.6	4830240.4	286285.7
11-108	168.0	4830260.6	286229.2

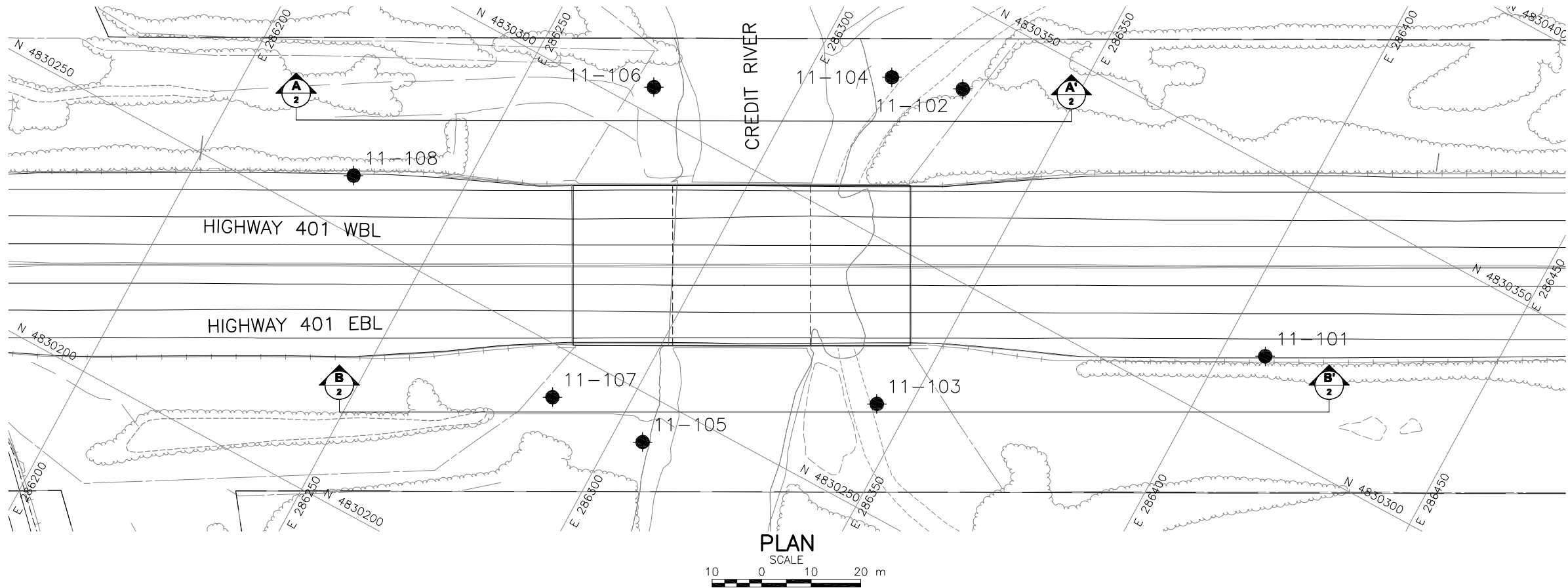
NOTES

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

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

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

NO.	DATE	BY	REVISION
Geocres No. 30M12-357			
HWY. 401		PROJECT NO. 10-1111-0040	DIST.
SUBM'D. MM	CHKD. LCC	DATE: 11/04/2012	SITE:
DRAWN: JFC	CHKD. MM	APPD. LCC	DWG. 1



REFERENCE

Base plans provided in digital format by URS, drawing file nos. ACAD-X-base1_to_Trafalgar.dwg, ACAD-Aerials_MTO_ROWL_Property Boundaries.dwg and Credit River_Structure_Alt 1-Plan.dwg, received August 17, 2011, August 29, 2011 and March 22, 2012.

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

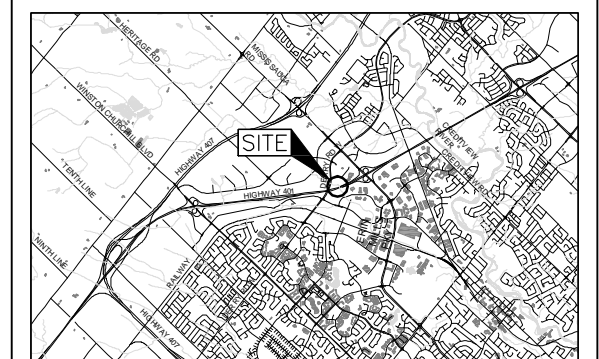
CONT No.
WO No. 07-20021

CREDIT RIVER BRIDGES
HIGHWAY 401 WIDENING
SOIL STRATA

SHEET



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN
SCALE
1.5 0 1.5 3 km

LEGEND

- Borehole - Current Investigation
- ⬮ Seal
- ⬮ Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated
(Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- ⬮ WL in piezometer, measured on January 20, 2012
- ⬮ WL upon completion of drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
11-101	165.2	4830316.0	286408.3
11-102	161.4	4830334.4	286329.0
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11-104	161.6	4830329.7	286315.3
11-105	161.5	4830241.1	286306.0
11-106	162.4	4830305.1	286274.0
11-107	164.6	4830240.4	286285.7
11-108	168.0	4830260.6	286229.2

NOTES

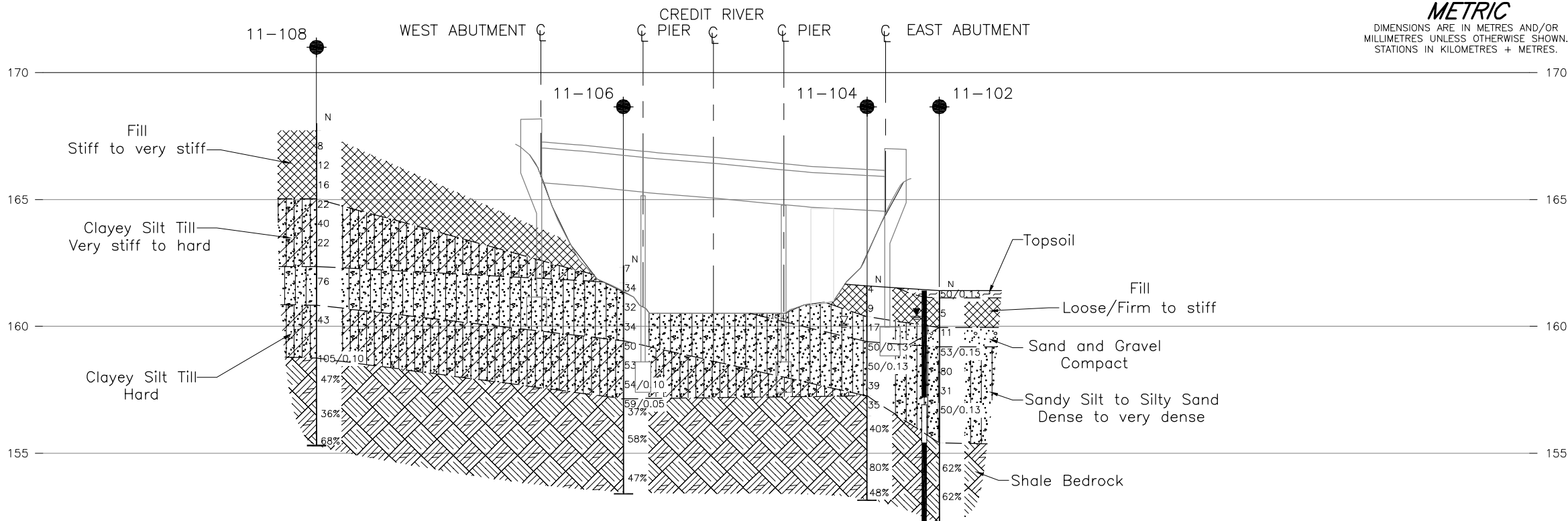
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the design configuration as shown elsewhere in the Preliminary Design Report.

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

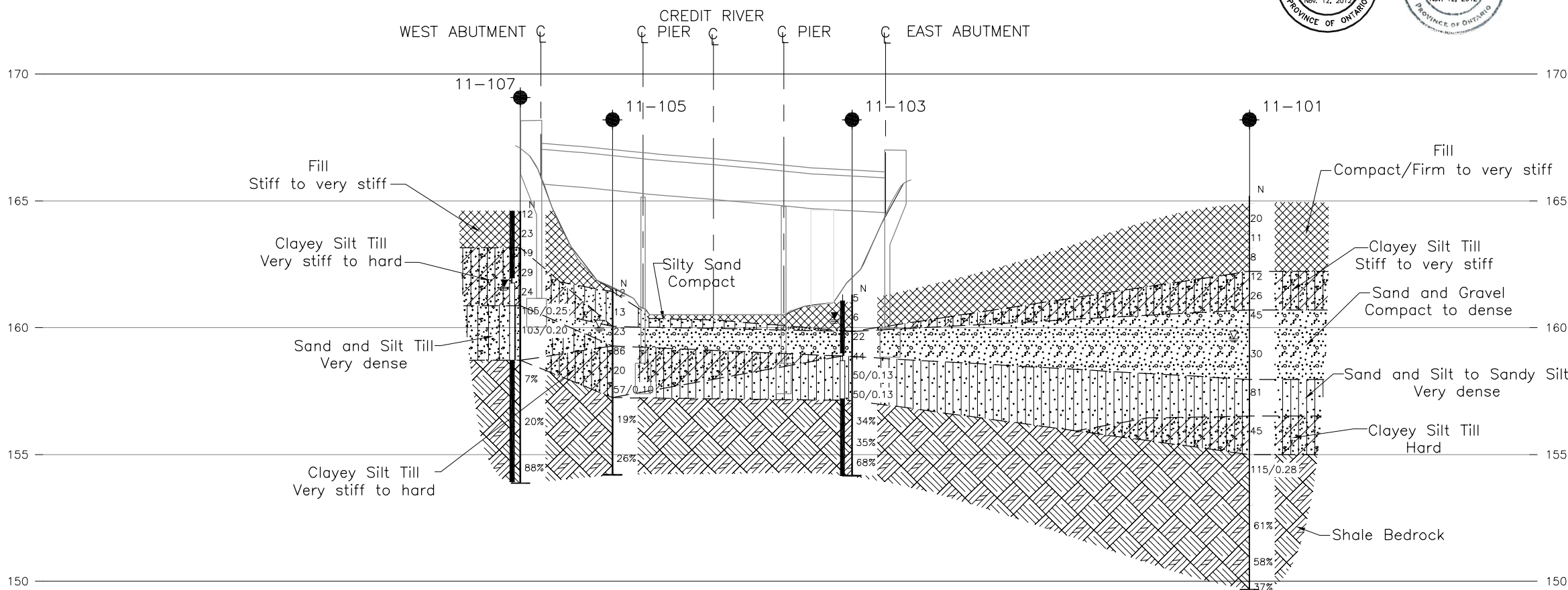
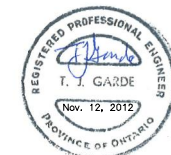
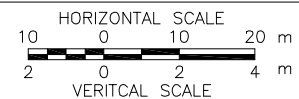
The complete Preliminary Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section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

REFERENCE

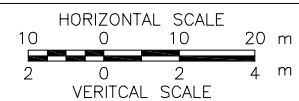
Base plan provided in digital format by URS, drawing file no.
Credit_River_Structure_Alt 1-Profile.dwg, received March 22, 2012.



PROFILE A-A' - NORTH WIDENING



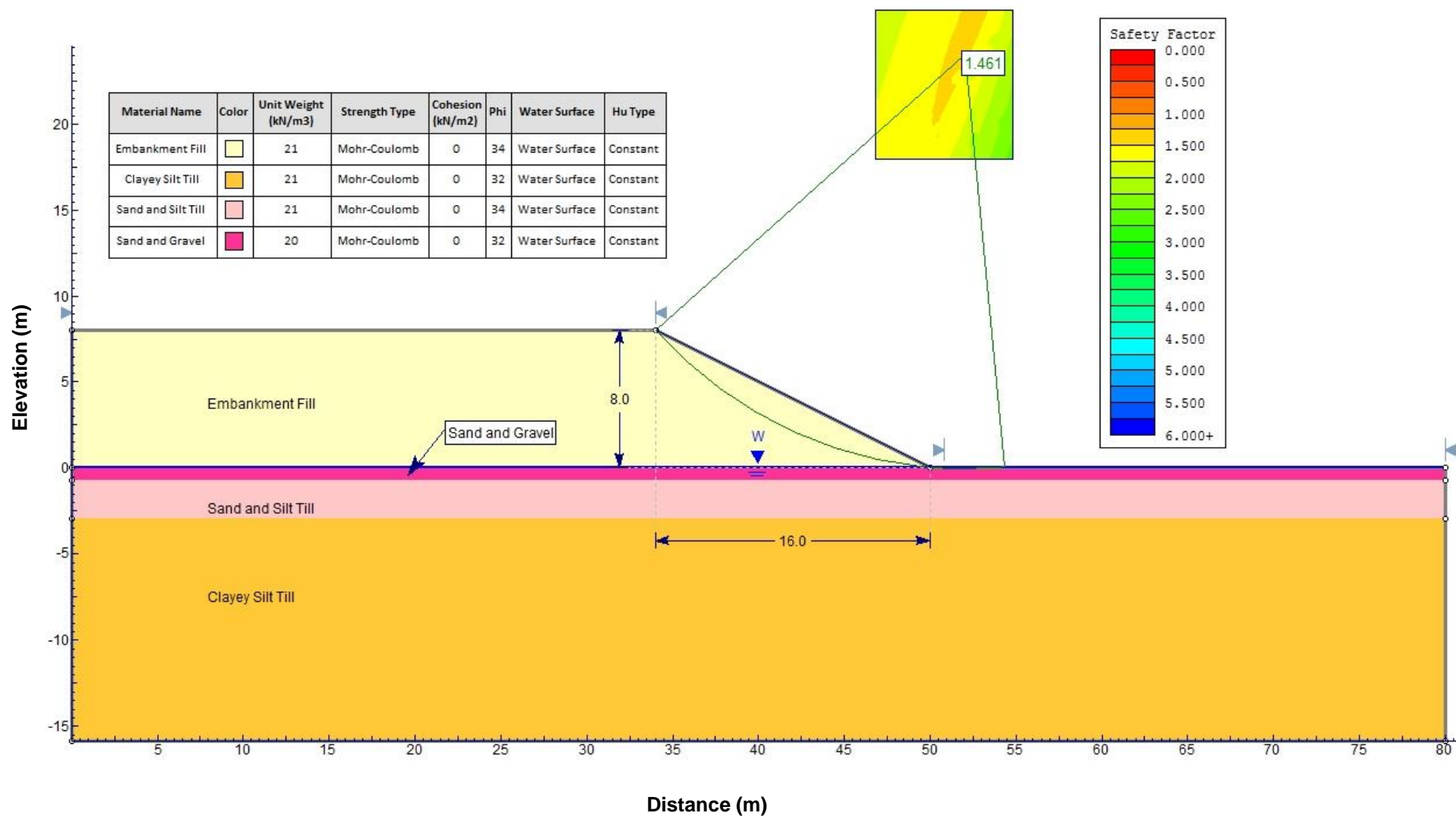
PROFILE B-B' - SOUTH WIDENING





Static Global Stability – Credit River Bridge Approach Embankments

Figure 1





APPENDIX A

Borehole Records



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
SS	Split-spoon
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
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) 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

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

IV. SOIL TESTS

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

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

V. MINOR SOIL CONSTITUENTS

Percent 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 (cohesionless) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand



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

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

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

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



LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERINGS STATE

Fresh: no visible sign of weathering

Faintly weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable.

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock and structure are preserved.

BEDDING THICKNESS

<u>Description</u>	<u>Bedding Plane Spacing</u>
Very thickly bedded	Greater than 2 m
Thickly bedded	0.6 m to 2 m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	Less than 6 mm

JOINT OR FOLIATION SPACING

<u>Description</u>	<u>Spacing</u>
Very wide	Greater than 3 m
Wide	1 m to 3 m
Moderately close	0.3 m to 1 m
Close	50 mm to 300 mm
Very close	Less than 50 mm

GRAIN SIZE

<u>Term</u>	<u>Size*</u>
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

Note: * Grains greater than 60 microns diameter are visible to the naked eye.

CORE CONDITION

Total Core Recovery (TCR)

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varied from 0% for completely broken core to 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.

Description and Notes

An abbreviation description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

JN Joint	PL Planar
FLT Fault	CU Curved
SH Shear	UN Undulating
VN Vein	IR Irregular
FR Fracture	K Slickensided
SY Stylolite	PO Polished
BD Bedding	SM Smooth
FO Foliation	SR Slightly Rough
CO Contact	RO Rough
AXJ Axial Joint	VR Very Rough
KV Karstic Void	
MB Mechanical Break	

PROJECT 10-1111-0040		RECORD OF BOREHOLE No 11-101		SHEET 1 OF 2		METRIC	
W.O. 07-20021		LOCATION N 4830316.0 ; E 286408.3		ORIGINATED BY AM			
DIST Central HWY 401		BOREHOLE TYPE Track-Mounted CME55, 108 mm I.D. Hollow Stem Augers		COMPILED BY MM			
DATUM NAD83, Geodetic		DATE September 14, 2011		CHECKED BY LCC			


SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE LIQUID LIMIT			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					
165.2	GROUND SURFACE																
164.9	Asphalt																
0.3	Silty sand, trace to some gravel (FILL)																
164.4	Compact Brown Moist		1	SS	20									18 36 32 14			
0.8	Clayey silt with sand, some gravel (FILL)																
163.8	Very stiff Brown Moist		2	SS	11												
1.5	Silty sand, trace clay, trace gravel, containing pockets of clayey silt (FILL)																
163.1	Compact Brown Moist		3	SS	8												
2.1	Clayey silt, some sand, trace gravel (FILL)																
162.2	Stiff Brown Moist		4	SS	12												
3.0	CLAYEY SILT, trace sand, trace gravel (TILL)																
	Stiff to very stiff Brown Moist		5	SS	26												
160.7	SAND and GRAVEL, trace clay, trace to some silt																
4.5	Dense Brown Moist becoming wet at 6.1 m		6	SS	45												
			7	SS	30									55 36 6 3			
158.0	Sandy SILT, trace clay, trace gravel (TILL)																
7.2	Very dense Grey Wet		8	SS	81												
156.5	CLAYEY SILT with sand, trace to some gravel (TILL)																
8.7	Hard Grey Moist		9	SS	45									8 35 42 15			
155.0	Shale (BEDROCK)																
10.2	Weathered Grey to black Wet		10	SS	115/0.28												
153.0	Shale (BEDROCK)																
12.2	Bedrock cored from depths of 10.2 m to 15.5 m		1	RC	REC 100%									RQD = 61%			
	For bedrock coring details, refer to Record of Drillhole 11-101																
			2	RC	REC 96%									RQD = 58%			

Continued Next Page

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

GTA-MTO 001 1011110040.GPJ GAL-GTA.GDT 9/13/12 DD

PROJECT <u>10-1111-0040</u>		RECORD OF BOREHOLE No 11-101		SHEET 2 OF 2		METRIC	
W.O. <u>07-20021</u>		LOCATION <u>N 4830316.0 ; E 286408.3</u>		ORIGINATED BY <u>AM</u>			
DIST <u>Central</u> HWY <u>401</u>		BOREHOLE TYPE <u>Track-Mounted CME55, 108 mm I.D. Hollow Stem Augers</u>		COMPILED BY <u>MM</u>			
DATUM <u>NAD83, Geodetic</u>		DATE <u>September 14, 2011</u>		CHECKED BY <u>LCC</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			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					W _p	W	W _L		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%)				
--- CONTINUED FROM PREVIOUS PAGE ---							20 40 60 80 100					10 20 30					
149.7			3	RC	REC 100%		150										RQD = 37%
15.5	END OF BOREHOLE NOTE: 1. Water level at a depth of 5.6 m (Elev. 159.6 m) upon completion of overburden drilling.																

PROJECT: 10-1111-0040

RECORD OF DRILLHOLE: 11-101

SHEET 1 OF 1

LOCATION: N 4830316.0 ; E 286408.3

DRILLING DATE: September 14, 2011

DATUM: NAD83, Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Track-Mounted CME 55

DRILLING CONTRACTOR: Geo-Environmental Drilling Inc.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.	RUN No.	COLOUR % RETURN	FLUSH	JN - Joint FLT - Fault SH - Shear VN - Vein CJ - Conjugate	BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage	PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular	PO - Polished K - Slickensided SM - Smooth RO - Rough VR - Very Rough	MB - Mechanical Break BR - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols.	NOTES	
				DEPTH										
				(m)										
		Continued from Record of Borehole BH11-101		153.01										
13		SHALE (BEDROCK) with fossiliferous limestone interbeds Slightly to moderately weathered Laminated Grey Medium strong		12.19	1									
14					2									
15					3									
		END OF DRILLHOLE		149.66										
16				15.54										
17														
18														
19														
20														
21														
22														

DEPTH SCALE

1 : 50



LOGGED: AM

CHECKED: LCC

GTA-RCK 018 1011110040.GPJ GAL-MISS.GDT 9/13/12 DD

GTA-MTO 001 101110040.GPJ GAL-GTA.GDT 9/13/12 DD

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

PROJECT: 10-1111-0040

RECORD OF DRILLHOLE: 11-102

SHEET 1 OF 1

LOCATION: N 4830334.4 ; E 286329.0

DRILLING DATE: December 19, 2011

DATUM: NAD83, Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Track-Mounted CME 55

DRILLING CONTRACTOR: Geo-Environmental Drilling Inc.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH	COLOUR % RETURN	JN - Joint FLT - Fault SH - Shear VN - Vein CJ - Conjugate	BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage	PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular	PO - Polished K - Slickensided SM - Smooth RO - Rough VR - Very Rough	MB - Mechanical Break BR - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols.	NOTES
6		Continued from Record of Borehole BH11-102		155.40 6.00									
7		SHAILE (BEDROCK) with fossiliferous limestone interbeds Slightly to moderately weathered Laminated Grey Medium strong to strong			1								(Axial)
8					2								
9		END OF DRILLHOLE		152.26 9.14									(Axial)
10													
11													
12													
13													
14													
15													
16													

DEPTH SCALE

1 : 50



LOGGED: CS

CHECKED:

GTA-RCK 018 1011110040.GPJ GAL-MISS.GDT 9/13/12 DD

PROJECT		10-1111-0040		RECORD OF BOREHOLE No 11-103		SHEET 1 OF 1		METRIC										
W.O.		07-20021		LOCATION		N 4830270.3 ; E 286343.9		ORIGINATED BY										
DIST		Central HWY 401		BOREHOLE TYPE		Track-Mounted CME55, 108 mm I.D. Hollow Stem Augers		COMPILED BY										
DATUM		NAD83, Geodetic		DATE		December 20, 2011		CHECKED BY										
								LCC										
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
								20 40 60 80 100	20 40 60 80 100	W _p	W	W _L	10 20 30	kN/m ³				
161.3		GROUND SURFACE																
161.0		TOPSOIL																
0.3		Clayey silt, some sand, trace gravel, containing cobbles (FILL) Firm Brown Moist		1	SS	5		161										
				2	SS	6		160										
159.9		SAND and GRAVEL, trace silt, trace clay Compact Grey Wet		3	SS	22		159										
158.9		SAND and SILT, some gravel, trace to some clay (TILL) Dense to very dense Grey Wet		4	SS	44		158										
				5	SS	50/0.13		158									14 41 33 12	
				6	SS	50/0.13		157										
157.1		Shale (BEDROCK)		1	RC	REC 100%		157									RQD = 34%	
4.2		Bedrock cored from depths of 4.2 m to 7.1 m For bedrock coring details, refer to Record of Drillhole 11-103		2	RC	REC 77%		156									RQD = 35%	
				3	RC	REC 95%		155									RQD = 68%	
154.2		END OF BOREHOLE																
7.1		NOTE: 1. Water level in monitoring well measured as follows: Date Depth (m) Elev. (m) 12/20/11 1.3 160.0 01/20/12 1.0 160.3																

[illegible]

PROJECT		10-1111-0040		RECORD OF BOREHOLE No 11-104		SHEET 1 OF 1		METRIC								
W.O.		07-20021		LOCATION		N 4830329.7 ; E 286315.3		ORIGINATED BY								
DIST		Central HWY 401		BOREHOLE TYPE		Track-Mounted CME55, 108 mm I.D. Hollow Stem Augers		COMPILED BY								
DATUM		NAD83, Geodetic		DATE		December 19, 2011		CHECKED BY								
								LCC								
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								
161.6	GROUND SURFACE															
160.4	Clayey silt with sand and gravel, containing cobbles and organic matter (FILL) Firm to stiff Brown Moist		1	SS	4											22 38 28 12
1.2	Silty SAND, trace to some gravel, trace clay Loose to compact Grey Wet		2	SS	9											
159.4			3	SS	17											
2.2	Gravelly silty SAND, trace to some clay (TILL) Dense to very dense Grey Wet		4	SS	50/0.13											23 40 25 12
			5	SS	50/0.13											
			6	SS	39											
157.3	Shale (BEDROCK) Weathered Grey Wet		7	SS	35											
156.5	Shale (BEDROCK)															
5.1	Bedrock cored from depths of 5.1 m to 8.5 m For bedrock coring details, refer to Record of Drillhole 11-104		1	RC	REC 100%											RQD = 40%
			2	RC	REC 95%											RQD = 80%
			3	RC	REC 100%											RQD = 48%
153.1	END OF BOREHOLE															
8.5	NOTE: 1. Water level at a depth of 1.5 m (Elev. 159.1 m) upon completion of overburden drilling.															

PROJECT: 10-1111-0040

RECORD OF DRILLHOLE: 11-104

SHEET 1 OF 1

LOCATION: N 4830329.7 ; E 286315.3

DRILLING DATE: December 19, 2011

DATUM: NAD83, Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Track-Mounted CME 55

DRILLING CONTRACTOR: Geo-Environmental Drilling Inc.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.	RUN No.	COLOUR % RETURN	JN - Joint FLT - Fault SH - Shear VN - Vein CJ - Conjugate	BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage	PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular	PO - Polished K - Slickensided SM - Smooth RO - Rough VR - Very Rough	MB - Mechanical Break BR - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols.	NOTES			
				DEPTH											
				(m)											
				FLUSH											
RECOVERY		R.Q.D.	FRACT	DISCONTINUITY DATA						HYDRAULIC		Diametral	RMC		
TOTAL	SOLID	%	INDEX	TYPE AND SURFACE						CONDUCTIVITY	Point Load	-Q'			
CORE %	CORE %		PER	Angle	DIP w.r.t.	CORE	AXIS	DESCRIPTION	Jr	Ja	Jn	K, cm/sec	Index	(MPa)	AVG.
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												
0.3 m	0.3 m		0.3 m												

DEPTH SCALE


1 : 50



LOGGED: CS

CHECKED:

GTA-RCK 018 1011110040.GPJ GAL-MISS.GDT 9/13/12 DD

PROJECT		10-1111-0040		RECORD OF BOREHOLE No 11-105		SHEET 1 OF 1		METRIC											
W.O.		07-20021		LOCATION		N 4830241.1 ; E 286306.0		ORIGINATED BY											
DIST		Central HWY 401		BOREHOLE TYPE		Track-Mounted CME55, 108 mm I.D. Hollow Stem Augers		COMPILED BY											
DATUM		NAD83, Geodetic		DATE		September 04, 2011		CHECKED BY											
								LCC											
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 (%)	
161.5	GROUND SURFACE							20	40	60	80	100							
0.1	Topsoil		1	SS	12	▽	161												
	Silty SAND, trace gravel, trace clay Compact Brown Moist		2	SS	13		160												
160.1																			
1.5	SAND and GRAVEL, some silt, trace clay, containing organic matter Compact Brown to black Wet		3	SS	23		159												
159.3			4	SS	86		158												
2.2	CLAYEY SILT with sand, trace to some gravel (TILL) Very stiff to hard Grey Moist becoming wet below 3.0 m		5	SS	20														
	Split-spoon bouncing at a depth of 3.9 m	6	SS	57/0 10															
157.2	Shale (BEDROCK)						157												
4.3	Bedrock cored from depths of 4.3 m to 7.3 m For bedrock coring details, refer to Record of Drillhole 11-105	1	RC	REC 98%			156											RQD = 19%	
		2	RC	REC 99%			155											RQD = 26%	
154.2	END OF BOREHOLE																		
7.3	NOTE: 1. Water level at a depth of 1.8 m (Elev. 159.7 m) upon completion of overburden drilling.																		

PROJECT: 10-1111-0040

RECORD OF DRILLHOLE: 11-105

SHEET 1 OF 1

LOCATION: N 4830241.1 ; E 286306.0

DRILLING DATE: September 04, 2011

DATUM: NAD83, Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Track-Mounted CME 55

DRILLING CONTRACTOR: Geo-Environmental Drilling Inc.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.	RUN No.	COLOUR % RETURN	JN - Joint FLT - Fault SH - Shear VN - Vein CJ - Conjugate	BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage	PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular	PO - Polished K - Slickensided SM - Smooth RO - Rough VR - Very Rough	MB - Mechanical Break BR - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols.	NOTES	
				DEPTH									
				(m)									
				FLUSH									
RECOVERY		R.Q.D. %	FRACT INDEX PER 0.3 m	DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY K, cm/sec				Diametral Point Load Index (MPa)	RMC -Q AVG.
TOTAL CORE %	SOLID CORE %			B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	T	P		
		Continued from Record of Borehole BH11-105		157.24									
		SHALE (BEDROCK) with fossiliferous limestone interbeds Slightly to moderately weathered Laminated Grey Weak to strong		4.26									
5					1								
6													
					2								
7													
		END OF DRILLHOLE		154.20									
8				7.30									
9													
10													
11													
12													
13													
14													

DEPTH SCALE

1 : 50



LOGGED: AM

CHECKED: LCC

GTA-RCK 018 1011110040.GPJ GAL-MISS.GDT 9/13/12 DD

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

PROJECT: 10-1111-0040

RECORD OF DRILLHOLE: 11-106

SHEET 1 OF 1

LOCATION: N 4830305.1 ;E 286274.0

DRILLING DATE: September 08, 2011

DATUM: NAD83, Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Track-Mounted CME 55

DRILLING CONTRACTOR: Geo-Environmental Drilling Inc.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH	COLOUR % RETURN	JN - Joint FLT - Fault SH - Shear VN - Vein CJ - Conjugate	BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage	PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular	PO - Polished K - Slickensided SM - Smooth RO - Rough VR - Very Rough	MB - Mechanical Break BR - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols.	NOTES
		Continued from Record of Borehole BH11-106		157.01									
6		SHALE (BEDROCK) with fossiliferous limestone interbeds Slightly to moderately weathered Laminated Grey Medium strong		5.39	1								
7					2								
8					3								
9		END OF DRILLHOLE		153.39									
10				9.01									
11													
12													
13													
14													
15													

DEPTH SCALE

1 : 50



LOGGED: AM

CHECKED: LCC

GTA-RCK 018 1011110040.GPJ GAL-MISS.GDT 9/17/12 DD

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

[illegible]

PROJECT <u>10-1111-0040</u>		RECORD OF BOREHOLE No 11-108		SHEET 1 OF 1		METRIC	
W.O. <u>07-20021</u>		LOCATION <u>N 4830260.6 ; E 286229.2</u>		ORIGINATED BY <u>AM</u>			
DIST <u>Central</u> HWY <u>401</u>		BOREHOLE TYPE <u>Track-Mounted CME55, 108 mm I.D. Hollow Stem Augers</u>		COMPILED BY <u>MM</u>			
DATUM <u>NAD83, Geodetic</u>		DATE <u>September 15, 2011</u>		CHECKED BY <u>LCC</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL LIMIT 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 (%)				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					w _p w w _L				
168.0	GROUND SURFACE							20	40	60	80	100					
167.7	ASPHALT																
0.3	Silty sand, trace to some gravel (FILL)																
167.2	Clayey silt with sand, trace gravel (FILL) Stiff to very stiff Brown Moist		1	SS	8												
0.8			2	SS	12												
			3	SS	16												
165.0	CLAYEY SILT with sand, trace to some gravel (TILL) Very stiff to hard Brown to grey Moist		4	SS	22												
3.0			5	SS	40												
			6	SS	22												
162.4	SAND and SILT, trace to some gravel, trace clay (TILL) Very dense Grey Moist		7	SS	76												
5.6																	
160.8	CLAYEY SILT with sand, trace to some gravel (TILL) Hard Grey Moist		8	SS	43												
7.2																	
158.8	Shale (BEDROCK) Bedrock cored from depths of 9.3 m to 12.7 m For bedrock coring details, refer to Record of Drillhole 11-108		9	SS	105/0.10												
9.3			1	RC	REC 97%												
			2	RC	REC 100%												
			3	RC	REC 82%												
155.3	END OF BOREHOLE																
12.7	NOTE: 1. Borehole dry on completion of overburden drilling.																

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

GTA-MTO 001 1011110040.GPJ GAL-GTA.GDT 9/13/12 DD

SHEET 1 OF 1

DATUM: NAD83, Geodetic

DRILL RIG: Track-Mounted CME 55
DRILLING CONTRACTOR: Geo-Environmental Drilling Inc.

LOGGED: AM
CHECKED: LCC



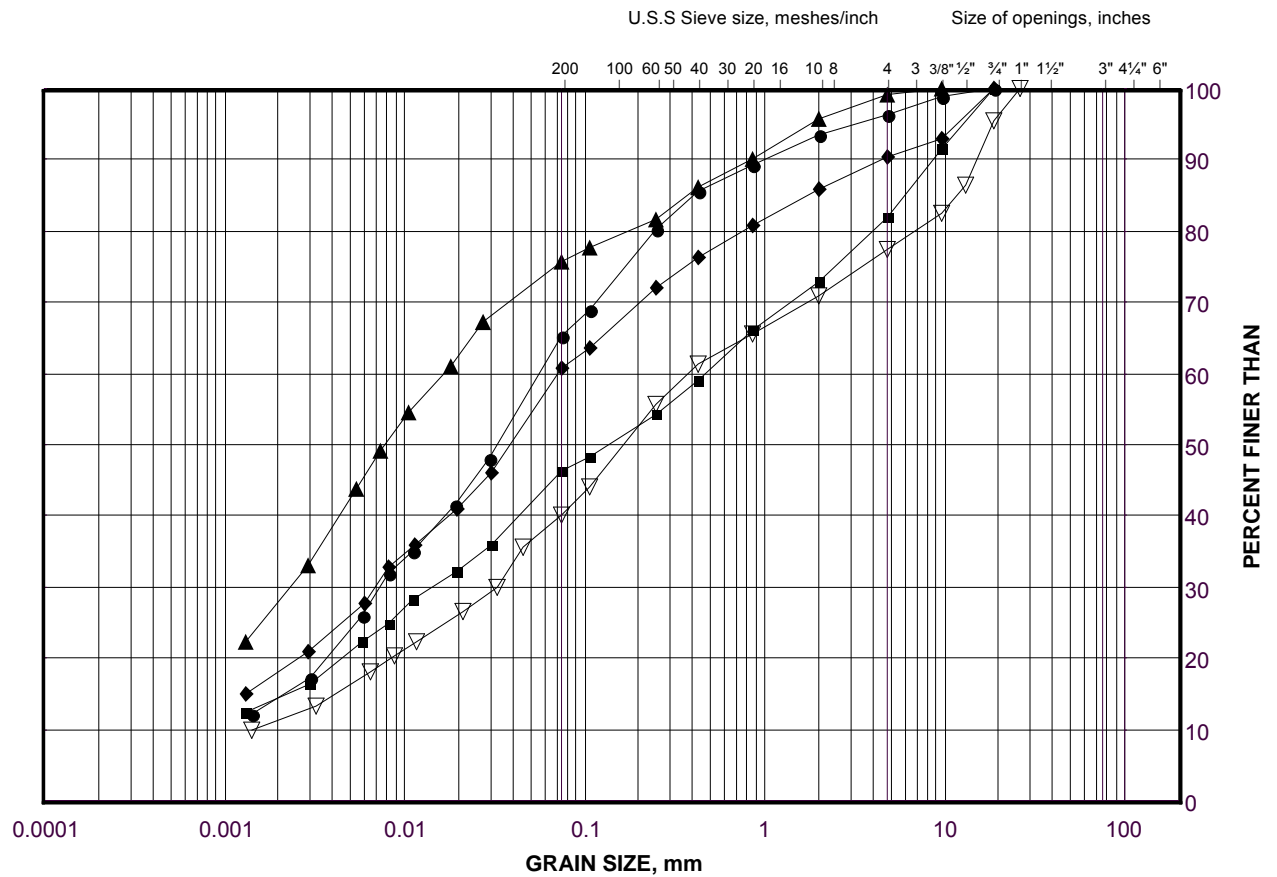
APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION

Clayey Silt Fill

FIGURE B1



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

LEGEND

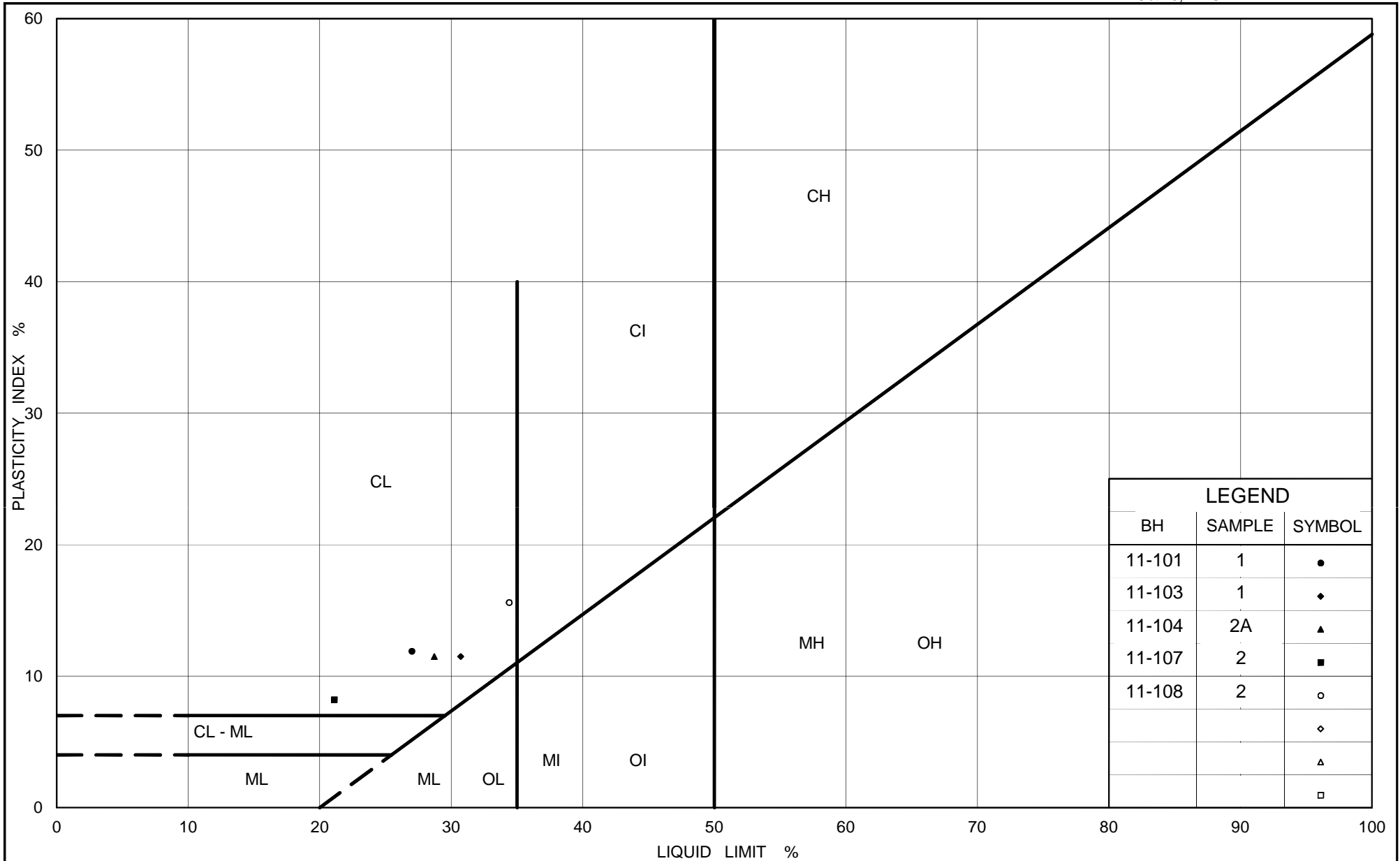
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	11-107	1	164.3
■	11-101	1	164.1
◆	11-107	2	163.5
▲	11-108	2	166.2
▽	11-104	2A	160.6

Project Number: 10-1111-0040-1

Checked By: _____

Golder Associates

Date: 29-Mar-12



Ministry of Transportation

Ontario

PLASTICITY CHART

Clayey Silt Fill

Figure No. B2

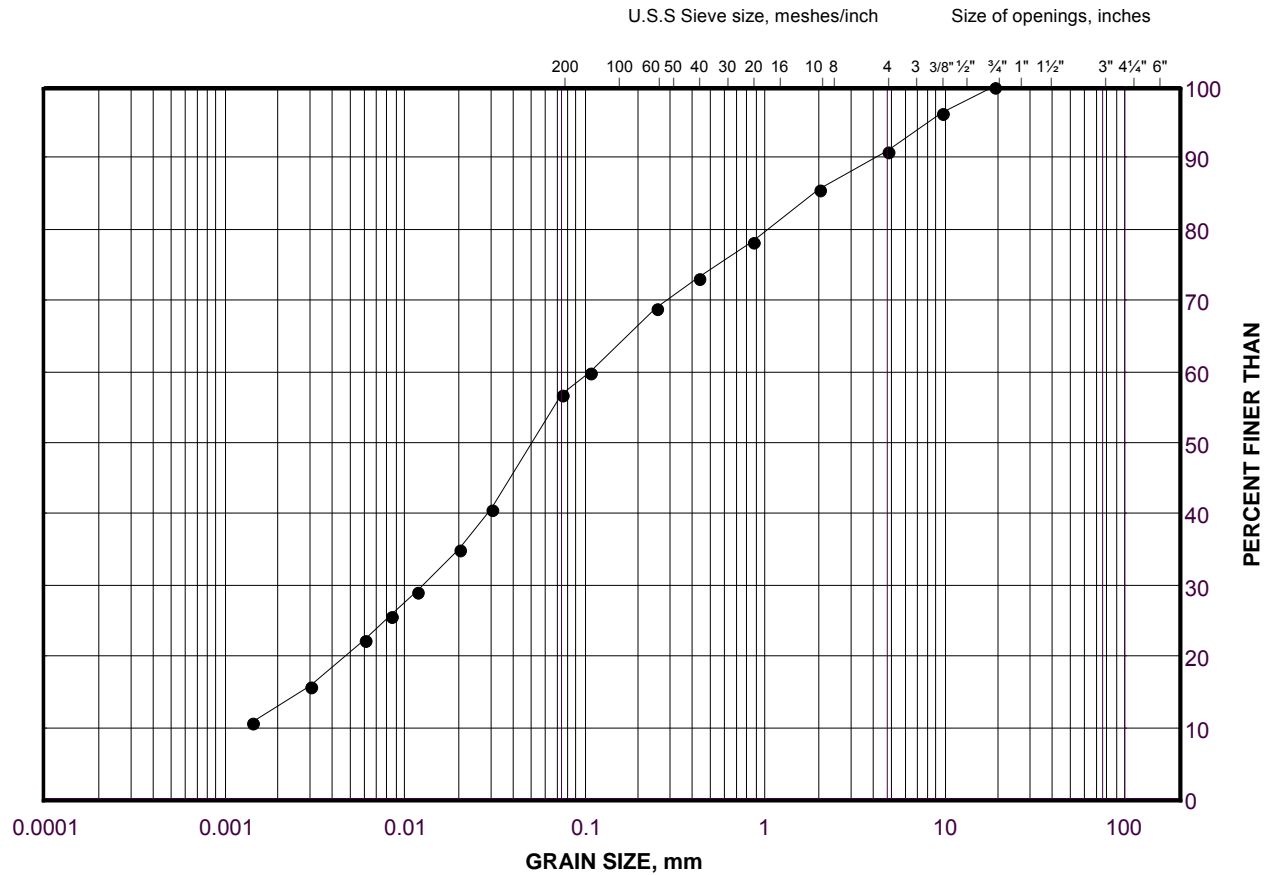
Project No. 10-1111-0040-1

Checked By:

GRAIN SIZE DISTRIBUTION

Upper Clayey Silt Till

FIGURE B3



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

LEGEND

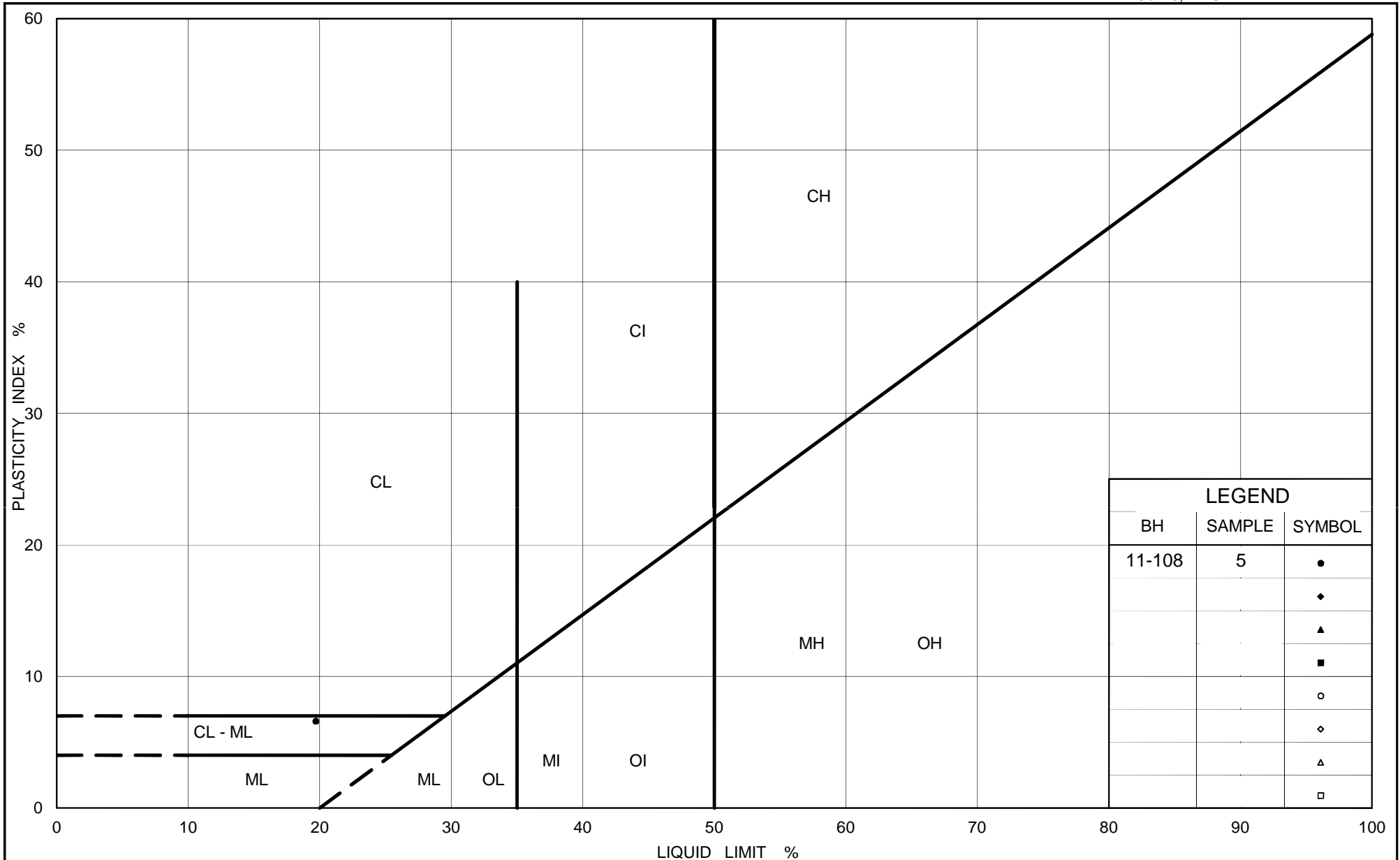
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	11-108	5	163.9

Project Number: 10-1111-0040-1

Checked By: _____

Golder Associates

Date: 29-Mar-12



Ministry of Transportation

Ontario

PLASTICITY CHART

Upper Clayey Silt Till

Figure No. B4

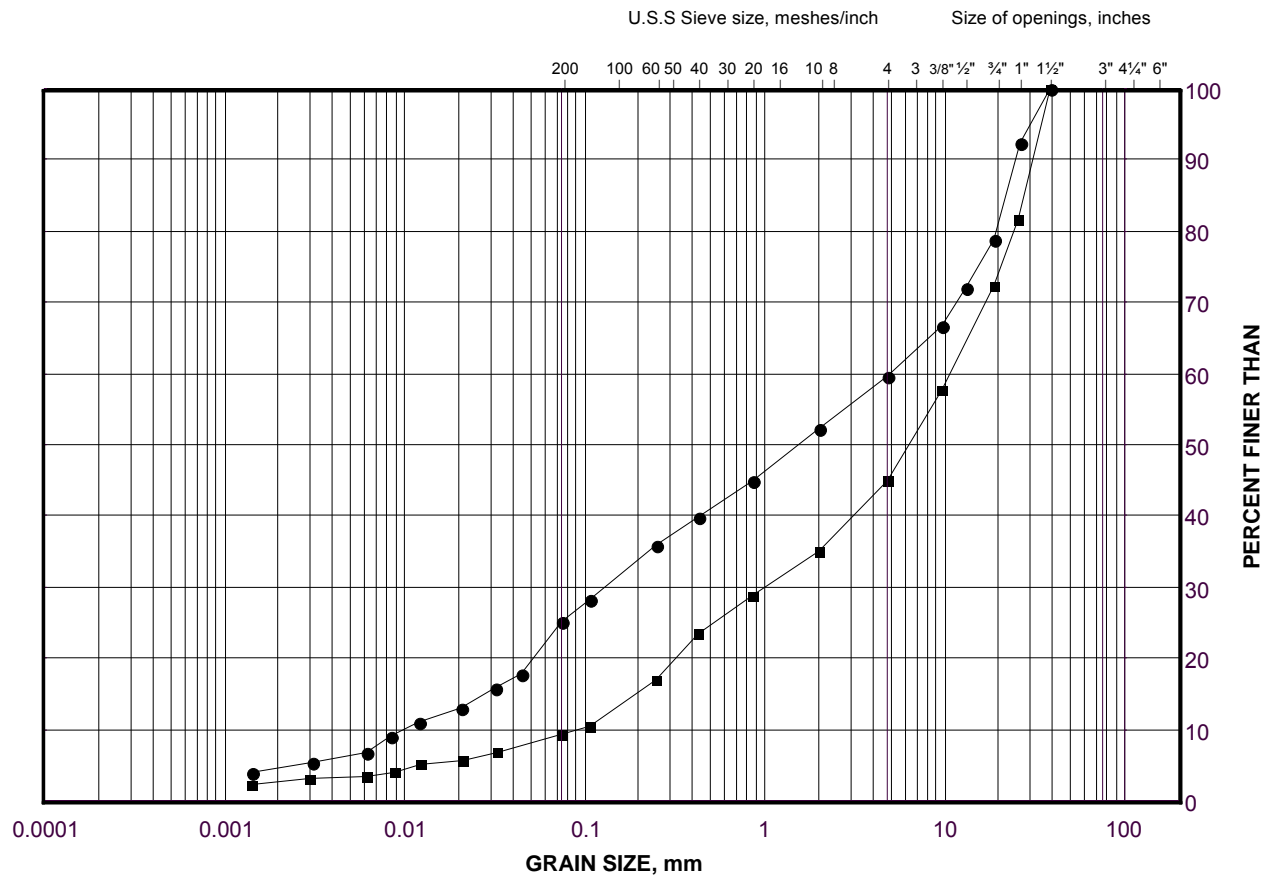
Project No. 10-1111-0040-1

Checked By:

GRAIN SIZE DISTRIBUTION

Sand and Gravel

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)
●	11-105	3	159.7
■	11-101	7	158.8

Project Number: 10-1111-0040-1

Checked By: _____

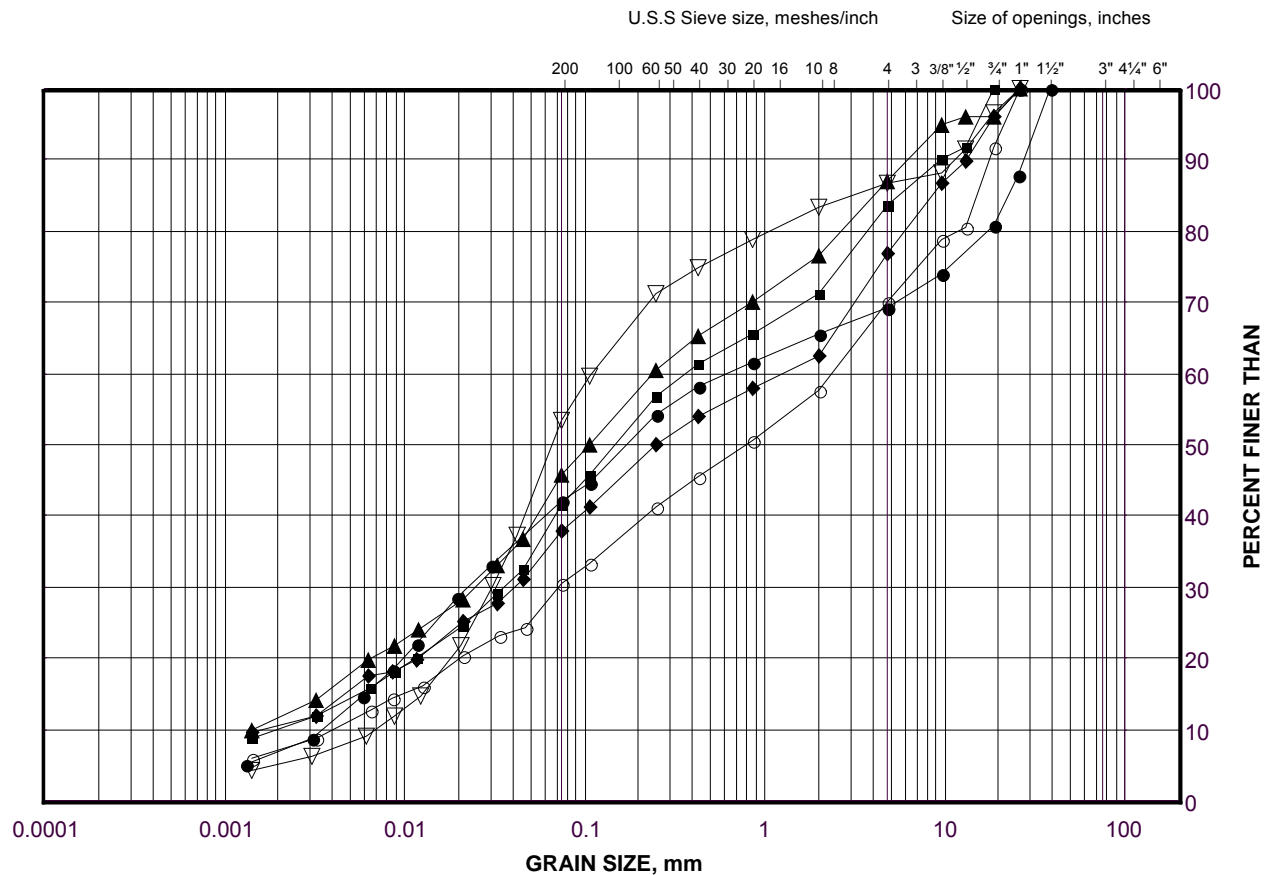
Golder Associates

Date: 09-Apr-12

GRAIN SIZE DISTRIBUTION

Silty Sand to Sandy Silt Till

FIGURE B6



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	11-106	3	160.6
■	11-102	5	158.1
◆	11-104	5	158.3
▲	11-103	5	158.1
▽	11-107	6	160.5
○	11-102	7	156.6

Project Number: 10-1111-0040-1

Checked By: _____

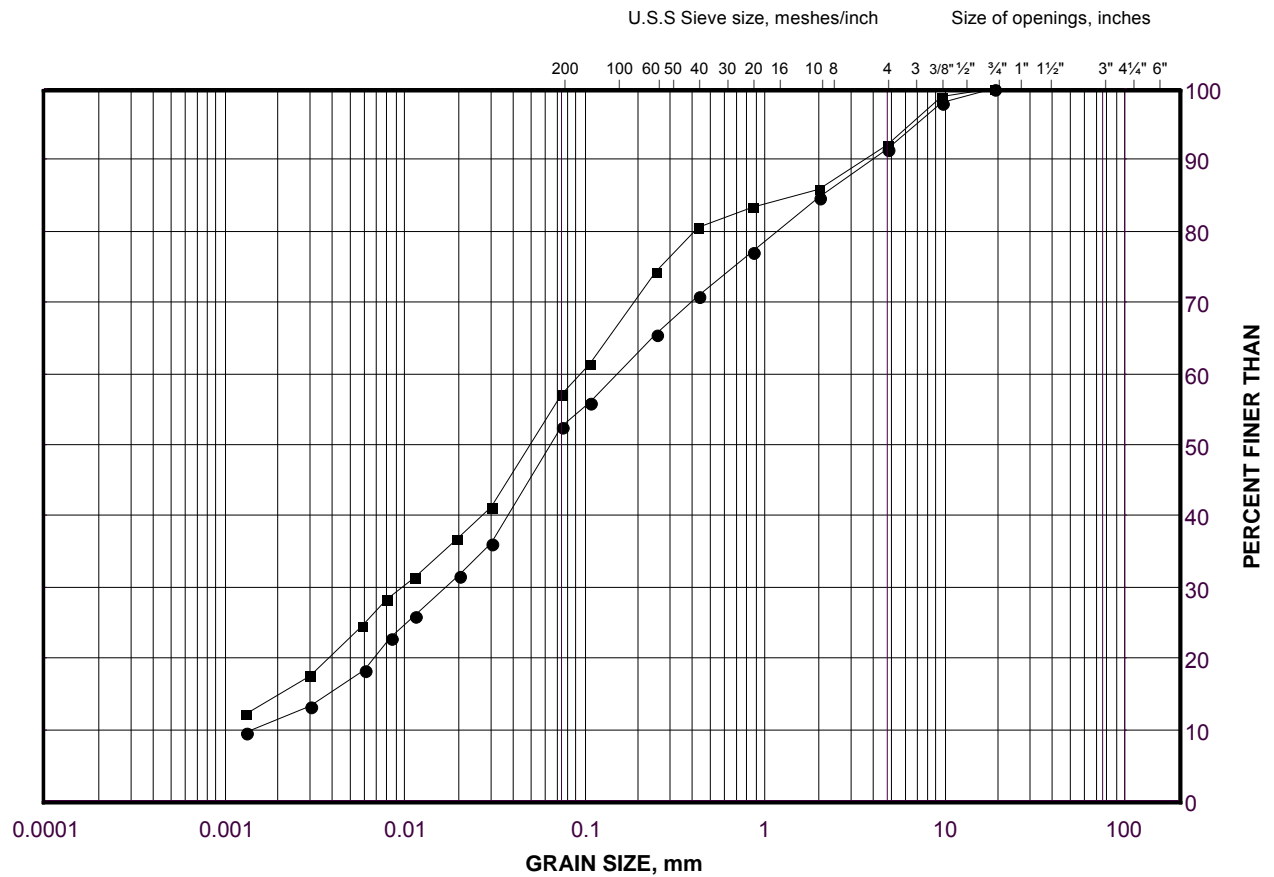
Golder Associates

Date: 09-Apr-12

GRAIN SIZE DISTRIBUTION

Lower Clayey Silt Till

FIGURE B7



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

LEGEND

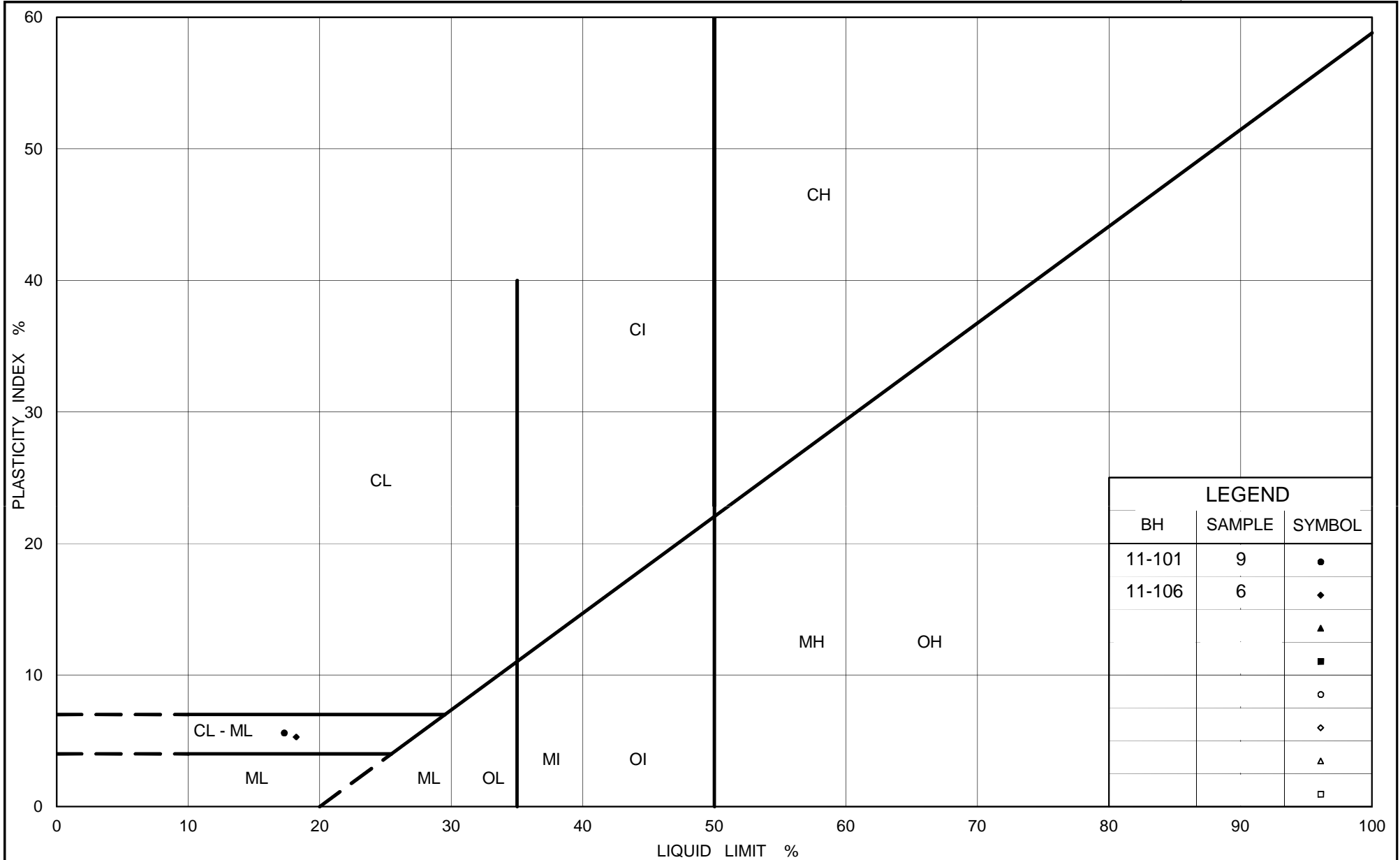
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	11-106	6	158.3
■	11-101	9	155.8

Project Number: 10-1111-0040-1

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Date: 29-Mar-12



Ministry of Transportation

Ontario

PLASTICITY CHART

Lower Clayey Silt Till

Figure No. B8

Project No. 10-1111-0040-1

Checked By:

UNCONFINED COMPRESSION TEST

ASTM D7012-07

FIGURE B9



BEFORE COMPRESSION



AFTER COMPRESSION

Date 10/19/2011
Project 10-1111-0040-1

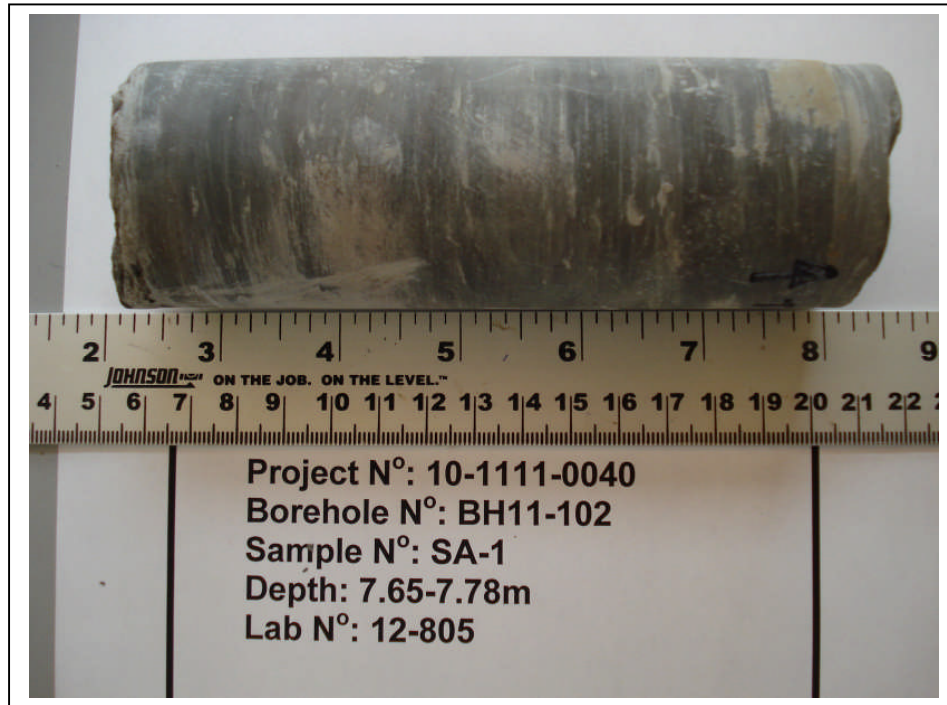
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Chkd.

UNCONFINED COMPRESSION TEST

ASTM D7012-07

FIGURE B10



BEFORE COMPRESSION




AFTER COMPRESSION

Date 1/31/2012
Project 10-1111-0040-1

Golder Associates

Drawn Frank
Chkd.

TABLE B1: UNCONFINED COMPRESSION TEST (UC)**ASTM D 7012-07**


SAMPLE IDENTIFICATION			
PROJECT NUMBER	10-1111-0040-1	SAMPLE NUMBER	-
BOREHOLE NUMBER	11-106	SAMPLE DEPTH, m	6.17-6.37
TEST CONDITIONS			
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.40
SPECIMEN INFORMATION			
SAMPLE HEIGHT, cm	11.35	WATER CONTENT, (specimen) %	0.14
SAMPLE DIAMETER, cm	4.73	UNIT WEIGHT, kN/m ³	26.40
SAMPLE AREA, cm ²	17.56	DRY UNIT WT., kN/m ³	26.36
SAMPLE VOLUME, cm ³	199.35	SPECIFIC GRAVITY, assumed	2.70
WET WEIGHT, g	536.82	VOID RATIO	0.00
DRY WEIGHT, g	536.07		
VISUAL INSPECTION		FAILURE SKETCH	
			
TEST RESULTS			
STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	100.0
REMARKS:		DATE:	10/19/2011

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TABLE B2: UNCONFINED COMPRESSION TEST (UC)

ASTM D 7012-07

SAMPLE IDENTIFICATION			
PROJECT NUMBER	10-1111-0040-1	SAMPLE NUMBER	1
BOREHOLE NUMBER	11-102	SAMPLE DEPTH, m	7.65-7.78
TEST CONDITIONS			
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST, min	>2 <15	L/D	2.35
SPECIMEN INFORMATION			
SAMPLE HEIGHT, cm	11.05	WATER CONTENT, (specimen) %	2.40
SAMPLE DIAMETER, cm	4.71	UNIT WEIGHT, kN/m ³	25.46
SAMPLE AREA, cm ²	17.42	DRY UNIT WT., kN/m ³	24.87
SAMPLE VOLUME, cm ³	192.56	SPECIFIC GRAVITY, assumed	2.70
WET WEIGHT, g	500.20	VOID RATIO	0.06
DRY WEIGHT, g	488.48		
VISUAL INSPECTION		FAILURE SKETCH	
			
TEST RESULTS			
STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	20.9
REMARKS:		DATE:	1/30/2012

Checked By:

Golder Associates

At Golder Associates we strive to be the most respected global group of companies specializing in ground engineering and environmental services. Employee owned since our formation in 1960, we have created a unique culture with pride in ownership, resulting in long-term organizational stability. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees now operating from offices located throughout Africa, Asia, Australasia, Europe, North America and South America.

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