



July 2012

FOUNDATION INVESTIGATION AND DESIGN REPORT

**Eastbound Overpass
Cornwall Centre Road Overpass (Site 31-209)
Highway 401, Cornwall, Ontario
GWP 4029-08-00
Ministry of Transportation, Ontario - Eastern Region**

Submitted to:

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REPORT



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PART A

FOUNDATION INVESTIGATION REPORT

EASTBOUND OVERPASS

CORNWALL CENTRE ROAD OVERPASS (SITE 31-209)

HIGHWAY 401, CORNWALL, ONTARIO

GWP 4029-08-00

MINISTRY OF TRANSPORTATION, ONTARIO - EASTERN REGION



1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by McCormick Rankin Corporation (MRC) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations as part of the detail design work for GWP 4029-08-00. The project involves the detail design for the replacement of the twin Highway 401 overpass structures (Site 31-209) at Cornwall Centre Road within the City of Cornwall, Ontario.

This report addresses the replacement of the eastbound structure, Site 31-209. The purpose of the foundation investigation is to determine the subsurface conditions at the location of the proposed replacement by drilling boreholes and carrying out in situ testing and laboratory testing on selected samples. The terms of reference for the scope of work are outlined in the MTO's Request for Proposal, in *Section 6.8* of the Technical Proposal and a letter outlining scope changes for this project dated October 5, 2011. The work was carried out in accordance with Golder's Project-Specific Supplementary Speciality Plan dated April 5, 2011.

2.0 SITE DESCRIPTION

The site is situated in the north-central part of the City of Cornwall, Ontario between the South Raisin River and Cornwall Township Road 31 (Speers Road). The closest interchange is the Highway 138/Brookdale Avenue interchange, approximately 1.2 km to the east, and the Power Dam Drive interchange is located approximately 1.75 km to the west. The location of the project is shown on the Key Plan, Figure 1. Site photographs are presented in Appendix F.

The existing Highway 401-Cornwall Centre Road overpasses consist of twin single-span structures, one for each of the two westbound and two eastbound lanes of Highway 401. Both structures are 13.26 m wide with a span of 18.37 m. The twin structures are separated by a 14.5 m wide median. The median fills between the abutments are retained by concrete cantilever retaining walls that are approximately 7 m high. The structures are skewed approximately 51 degrees to the centreline of Highway 401. The embankments are about 7 m high with side slopes oriented at approximately 2 horizontal to 1 vertical (2H:1V). The Highway 401 pavement surface at Cornwall Centre Road is at approximately Elevation 68.1 m. Cornwall Centre Road is on a vertical (sag) curve with elevations ranging from 61.5 m at a low point between the eastbound and westbound structures, Elevation 61.7m at the beginning of the vertical curve north of the overpass, to Elevation 61.9 m at the end of the vertical curve south of the overpass.

The west median retaining wall is exhibiting signs of movement and distress. It has rotated towards Cornwall Centre Road and vertical cracks are present on both the east and west median retaining walls. Remedial work was carried out on the west median retaining wall in 2009, consisting of installing tie-backs attached to deadman anchors to restrain movement of the wall. Similar distresses were not noted on the east median retaining wall, though observations indicate that there may have been some minor movement.

The land adjacent to the site is relatively flat and is near Elevation 60 m. Land use in the area is predominantly agricultural and rural residential.



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2.1 Site Geology

The Highway 401-Cornwall Centre Road overpass structures are located in a physiographic region known as the Lancaster Flats. It is a flat, poorly drained, lowland area where the till plain has been largely buried under water-lain deposits. Only the stony crests of a few drumlins and till ridges are exposed. The water-lain materials range in composition from clay to very fine sand. The nearby South Raisin River drains into the St. Lawrence River.¹

The overburden consists of unconsolidated deposits mainly of Pleistocene age. The dominant consolidated deposit is glacial till composed of silt, clay and sand with pebbles and boulders included. In low-lying areas of the till surface, marine clay and silt has been deposited by the Champlain Sea during the late Pleistocene and early Holocene age. These clayey deposits are generally less than 15 m thick but may reach 100 m in thickness in localized areas.²

The overburden thickness is approximately 11.5 m to 13.5 m in the vicinity of the site.³ The available bedrock topography mapping indicates that the bedrock surface ranges from Elevation 48 m to 50 m.⁴ The overburden is underlain by calcareous limestone of the Bobcaygeon Formation. Two quarries and an area of bedrock outcrops were mapped 3.5 km to 4 km west of the site.⁵

3.0 INVESTIGATION PROCEDURES

The field work for this project was conducted between June 7 and 25, 2011 during which time seven boreholes were advanced at the locations shown on the Borehole Location Plan (Drawing 1). A supplementary investigation was conducted in November 2011 to determine the extent and properties of any silty clay that was suspected to be present below the existing abutment footings. A total of four boreholes were drilled through the front (toe) of the existing eastbound overpass abutment footings from the Cornwall Centre Road grade. The boreholes were terminated a minimum depth of 1.1 m below the underside of footing elevation. The table below summarizes the borehole locations, ground surface elevations at the borehole locations and borehole depths.

Borehole No.	Location (m)		Ground Surface Elevation	Borehole Depth
	Northing	Easting	(m)	(m)
11-01	4 991 125	203 307	61.7	5.9
11-02	4 991 135	203 331	61.9	10.0
11-03	4 991 123	203 337	61.6	2.0
11-03A	4 991 123	203 338	61.5	6.8
11-04	4 991 131	203 346	61.9	11.5

¹ Chapman, L.J., and Putnam, D.F. 1984: Physiography of Southern Ontario; Ontario Geological Survey, Special Volume 2, 270p. Accompanied by Map P.2715 (coloured), scale 1: 600,000.

² Brismead, R.A. 1975: Lower St. Lawrence Planning Area: Finch, Roxborough, Osnaburck, Cornwall and Charlottenburg Townships. Ontario Division of Mines Geological Branch, Open File Report 5138.

³ Gwyn, Q.H.J, Fraser, J.Z. and Owen, N., 1975: Drift Thickness of the Cornwall-Huntingdon Area, Southern Ontario; Ontario Division of Mines, Preliminary Map P.1073, Drift Thickness Series, Scale 1:50, 000. Geology and Compilation 1974.

⁴ Gwyn et al, 1975: Bedrock Topography of the Cornwall-Huntingdon Area, Southern Ontario; Ontario Division of Mines, Preliminary Map P.1012, Bedrock Topography Series, Scale 1:50,000. Geology and Compilation, 1974.

⁵ Williams, D.A., Wolf, R.R. and Carson, D.M., 1985: Paleozoic Geology of the Cornwall-Huntingdon Area, Southern Ontario; Ontario Geological Survey, Map P.2720, Geological Series – Preliminary Map, Scale 1: 50 000. Geology 1981-1982.



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Borehole No.	Location (m)		Ground Surface Elevation	Borehole Depth
11-05	4 991 149	203 358	61.7	10.2
11-07	4 991 145	203 379	67.4	17.7
11-28	4 991 145	203 348	61.9	4.3
11-29	4 991 136	203 333	61.9	3.8
11-30	4 991 144	203 366	61.7	3.7
11-31	4 991 135	203 350	61.7	3.4

Discrepancies were noted for the east abutment footing between the design top of footing elevation and those determined by Golder, particularly at Borehole 11-31. A supplementary investigation was conducted on February 2, 2012 to verify the top of footing elevation at Borehole 11-31 and at a second location along the east abutment by exposing the footing using a hydro-excavator. An Ontario Licensed Surveyor (OLS) surveyed the top of footing elevation and the ground surface elevations at Boreholes 11-28 to 11-31. The location of Borehole 11-24 was re-excavated using hydrovac techniques and the top of concrete elevation surveyed by the OLS. A second hydrovac hole was completed mid-way between Boreholes 11-30 and 11-31 and the top of concrete elevation surveyed. The results of the additional investigation and have been incorporated into this report.

The soil stratigraphy encountered in the boreholes is shown on the attached Record of Borehole sheets. The investigation was carried out using a track-mounted CME 850 power auger supplied and operated by a specialist drilling contractor, Marathon Drilling Ltd. of Ottawa, Ontario. For the initial investigation, samples of overburden were generally obtained at 0.75 m and 1.5 m depth intervals using 50 mm outside diameter split-spoon sampling equipment in accordance with the Standard Penetration Test (SPT) procedures of ASTM D 1586. Rock core was obtained using an NQ-sized rock core barrel in Boreholes 11-02, 11-04, 11-05 and 11-07. The supplementary investigation was conducted using a hydro-excavator truck to advance the boreholes below the depth of a gas main that is located in front of the east abutment footings and Bell ducts that are located above the west abutment footings. The boreholes were completed below the depths of these utilities using a truck-mounted LC-60 power auger supplied and operated by the same specialist drilling contractor. The concrete footings were cored using HQ-sized rock coring equipment and the underlying founding soils were sampled using 50 mm outside diameter split-spoon sampling equipment in accordance with SPT procedures.

Groundwater conditions in the boreholes were observed throughout the drilling operations and a standpipe was installed in Borehole 11-03A. The boreholes were backfilled in accordance with current MTO procedures and Ontario Regulation 903 (as amended).

The field work was supervised on a full-time basis by members of Golder's senior technical staff who directed the drilling, sampling and in situ testing operations and logged the boreholes. The ground surface elevations and borehole locations were also determined by members of Golder's staff. The samples were identified in the field, placed in labelled containers and transported to Golder's London, Ottawa and/or Mississauga laboratories for further examination and testing. Index and classification, tests consisting of water content determinations, grain size distribution analyses and Atterberg limits determinations, were carried out on selected samples in Golder's London and Ottawa laboratories. Unconfined compressive strength testing and point load testing were carried



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out in Golder's Mississauga laboratory. The results of the testing are shown on the Record of Borehole sheets and in Appendices A and B. Photographs of the rock core are presented in Appendix C.

In addition, information from a previous investigation report prepared by Thurber Engineering Ltd. (Thurber) for this site (GEOCRE Report No. 31G-232 entitled "Foundation Investigation and Design Report, Retaining Wall Distress, Highway 401/Cornwall Centre Road, Cornwall, Ontario, Site No. 31-209" dated June 15, 2009) was reviewed and incorporated into this report. Three boreholes (Boreholes 09-05, 09-09 and 09-10) were drilled at or adjacent to the eastbound structure to depths of 7.7 m to 16.9 m. Three test pits (TP 09-5 to 09-7) were excavated at the base of the west abutment walls and southeast and southwest wingwalls/retaining walls. The records of the previous boreholes and test pits are presented in Appendix E of this report and the locations of the previous boreholes and test pits are shown on Drawing 1. The locations of the boreholes drilled for the previous investigation are as follows:

Borehole No.	Location (m)		Ground Surface Elevation*	Borehole Depth
	Northing	Easting	(m)	(m)
09-05	4 991 143	203 324	68.5	16.9
09-09	4 991 149	203 348	67.8	8.1
09-10	4 991 145	203 373	67.7	7.7

* **NOTE:** Ground surface elevations for 2009 Thurber boreholes adjusted as described below.

In an e-mail dated July 11, 2011 to MRC, the Geomatics Department of the MTO reported that the borehole elevations they provided to Thurber for GEOCRE No. 31G-232 utilized vertical control points which were in error. They reported that the elevations for Boreholes 09-05, 09-09 and 09-10 as reported in GEOCRE No. 31G-232 were 1.73 m too high while those for the test pits were 1.75 m too high. The ground surface elevations for all boreholes and test pits from GEOCRE No. 31G-232 have been corrected for this report and the ground surface elevations shown on the records of previous boreholes and test pits in Appendix E have been updated accordingly.

4.0 SUBSURFACE CONDITIONS

4.1 Site Stratigraphy

The detailed subsurface soil, rock and groundwater conditions encountered in the boreholes, together with the results of the in situ testing and the laboratory testing carried out on selected samples, are given on the borehole records following the text of this report and in Appendices A and B. The borehole and test pit records from the 2009 Thurber investigation (GEOCRE No. 31G-232) are included in Appendix E. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous samples, observations of drilling resistance and rock core and, therefore, may represent transitions between soil and rock types rather than exact planes of geological change. Further, the subsurface conditions will vary between and beyond the borehole locations.

The locations and elevations of the boreholes, together with the interpreted stratigraphic profiles, are shown on the attached Drawings 1 and 2. The four boreholes advanced through the footing area (Boreholes 11-28 to 11-31) were excluded from the stratigraphic profile shown in Drawing 1 in the interest of clarity, but are shown on



Drawing 2. The selected test pits from GEOCRETS No. 31G-232 were included solely to provide additional information on the top of footing elevation. Therefore, descriptions of the stratigraphy from these test pits have not been provided or included on the drawings. A detailed description of the subsurface conditions encountered in the boreholes is provided on the Record of Borehole sheets and is summarized in the following sections.

4.2 Soil Conditions

Boreholes 11-02, 11-04 and 11-05 were advanced adjacent to the existing eastbound structure and wingwalls during the 2011 investigation. Boreholes 11-01, 11-03 and 11-03A were advanced on the shoulders of Cornwall Centre Road on the south side of the EBL overpass. Borehole 11-07 was drilled from the Highway 401 level at the northwest corner of the eastbound structure. In the supplementary 2011 investigation, Boreholes 11-28 and 11-29 were advanced through the west abutment footing and Boreholes 11-30 and 11-31 were drilled through the east abutment footing. Boreholes 09-09 and 09-10 (31G-232) were drilled by others in 2009 behind the east median retaining wall and west median retaining wall, respectively, and Borehole 09-05 was drilled in the eastbound lanes of Highway 401.

The soils encountered in the boreholes consist of the pavement structure or surficial topsoil underlain by very loose to very dense or firm to very stiff fill to Elevation 59 m to 61 m. The embankment fill is generally underlain by firm to very stiff silty clay to Elevation 59 m to 61 m, then compact to very dense silty sand to sandy silt till to the limestone bedrock interface at approximately Elevation 53 m to 56 m.

4.2.1 Pavement Structure

Asphalt layers 40 mm to 100 mm thick were encountered at the ground surface in Boreholes 11-02, 11-05 and 11-28 to 11-31 which were drilled through Cornwall Centre Road. A 175 mm thick layer of asphalt was found at the Highway 401 pavement surface in Borehole 09-05 (31G-232) with a 325 mm thick layer of concrete below the asphalt in this borehole. The asphalt layers in Boreholes 11-02 and 11-28 to 11-31 are underlain by 120 mm to 500 mm thick layers of sand and gravel or crushed gravel fill which are considered to be granular roadbase material from Cornwall Centre Road. The granular roadbase material could not be distinguished from the fill at Boreholes 11-05 and 09-05 (31G-232).

4.2.2 Topsoil

Topsoil layers 100 mm to 500 mm thick were encountered at the ground surface in Boreholes 11-01, 11-03, 11-03A, 11-04 and 11-07. Layers of topsoil 75 mm thick were encountered at the surface of Boreholes 09-09 and 09-10 (31G-232).

Materials designated as topsoil in this report were classified solely based on visual and textural evidence. Testing of organic content or for other nutrients was not carried out. Therefore, the use of materials classified as topsoil cannot be relied upon for support and growth of landscaping vegetation.



4.2.3 Fill

The asphalt layers in Boreholes 11-02 and 11-05 are underlain by fill from Elevation 61.8 m and 61.6 m, respectively. Adjacent to Cornwall Centre Road, fill was encountered beneath the topsoil at Elevation 61.3 m and 61.6 m in Boreholes 11-01 and 11-04, respectively. Adjacent to Highway 401, fill was encountered beneath the topsoil from Elevation 67.3 m in Borehole 11-07. The granular roadbase material in Boreholes 11-28 to 11-31 is underlain by fill from Elevation 61.5 m to 61.7 m. The asphalt and concrete layers in Borehole 09-05 (31G-232) are underlain by fill from Elevation 68.0 m. The topsoil in Boreholes 09-09 and 09-10 (31G-232) is underlain by fill from approximately Elevation 67.5 m. Borehole 09-10 (31G-232) was terminated in granular fill after exploring it for approximately 7.6 m.

The fill consists of sand and gravel, silty sand, sandy silt and clayey silt. The sand and gravel fill in Borehole 11-28 contains asphalt fragments. The fill layers are about 0.9 m to 2.5 m thick in the boreholes drilled beside Cornwall Centre Road and about 6.3 m to over 7.6 m thick behind the abutment areas.

The granular fill has a very loose to very dense relative density with SPT N values of 3 to 79 blows per 0.3 m. The granular fill had water contents of 6 to 18 per cent. The gradation of two samples of granular fill retrieved from Boreholes 11-01 and 11-07 are presented on Figure A-1 in Appendix A.

The firm to very stiff clayey silt fill in Boreholes 11-01, 11-02, 11-04, 11-05 and 11-07 had SPT N values of 7 to 16 blows per 0.3 m. Water contents of 10 to 35 per cent were measured in samples of the clayey silt fill. The gradation of one sample of clayey silt fill retrieved from Borehole 11-05 is presented on Figure A-1 in Appendix A. The clayey silt fill is of low plasticity based on one sample with a plastic limit of 22 per cent, liquid limit of 35 per cent and plasticity index of 13 per cent. The results of the Atterberg limits determination are presented on Figure A-5 in Appendix A.

4.2.4 Concrete *Abutment Footings*

The top of the concrete footing at the west abutment was encountered beneath the fill in Boreholes 11-28 and 11-29 at approximately Elevation 60.2 m. The concrete was found to be 1.3 m and 1.1 m thick in these boreholes, respectively.

The top of the concrete footing at the east abutment was encountered beneath the fill in Boreholes 11-30 and 11-31 at approximately Elevation 60.2 m and 60.4 m, respectively. The concrete was found to be 0.9 m thick in these boreholes. The bottom 50 mm to 75 mm of the concrete was honeycombed in Borehole 11-31.

On February 2, 2012, Borehole 11-31 was re-excavated using hydrovac techniques to confirm the top of concrete footing elevation. The surface of the concrete was surveyed by an OLS at Elevation 60.42 m. The top of concrete footing elevation at Borehole 11-31 is considered to be a localized 'high point' since the top of concrete elevation directly surveyed in an adjacent hydrovac borehole was 60.23 m. Also, an additional hydrovac hole was advanced mid-way between Boreholes 11-30 and 11-31 to the top of the east abutment footing. The top of concrete at this location was found to be Elevation 60.21 m.



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The top and underside of concrete elevations and concrete thicknesses based on the results of the February 2012 survey and November 2011 investigation are summarized as follows:

Footing Location	Borehole	Depth to Top of Footing (m)	Elevation of Top of Footing (m)	Thickness of Concrete (m)	Elevation of Base of Footing (m)
West Abutment	11-28	1.55	60.31	1.27	59.04
	11-29	1.68	60.22	1.06	59.16
East Abutment	11-30	1.52	60.20	0.92	59.28
	11-31	1.32	60.41*	0.91	59.50

* Note: Top of footing elevation noted in table for Borehole 11-31 based on downhole measurement taken from ground surface during drilling in November 2011 and ground surface elevation surveyed in February 2012. The top of footing elevation at Borehole 11-31 and an adjacent borehole was directly surveyed by an OLS on February 2, 2012 at Elevation 60.42 m and 60.21 m, respectively.

In three of the test pits (Test Pits 09-05 to 09-07) excavated for Geocres No. 31G-232, the top of the concrete footings were exposed at depths of 1.7 m to 1.9 m or at approximately Elevation 60.0 m and 60.1 m. The top of concrete elevations encountered in these test pits are summarized as follows:

Footing Location	31G-232 Test Pit No.	Depth to Top of Footing (m)	Elevation of Top of Footing (m)
West Abutment	09-05	1.7	60.1
Southwest Wingwall	09-06	1.8	60.1
Southeast Wingwall	09-07	1.9	59.9

The design thickness of the footings was shown on the 1960 contract drawings as 0.76 m and the concrete core to that depth indicated structural concrete; a reinforcing bar was encountered within the structural concrete in Borehole 11-29 at a depth of 0.6 m below the top of the footing. The lower concrete, some 0.1 m to 0.5 m in thickness, was a slightly lower grade of concrete and honeycombing was observed at the base of the concrete core recovered from Borehole 11-31.

Unconfined compressive strength (UCS) testing was conducted on a core sample of the lower concrete retrieved from Borehole 11-28. The measured UCS was about 19 MPa. The results of the UCS testing are presented on the report in Appendix D.

Other Concrete

A 100 mm thick concrete layer was encountered beneath the fill in Borehole 11-05 at Elevation 59.2 m.



4.2.5 Silty Clay

The fill in Boreholes 11-02 to 11-04, 11-07 and 09-05 (31G-232) were underlain by silty clay from Elevation 60.4 m to 61.7 m. Borehole 09-09 (31G-232) encountered material described as silty clay at Elevation 60.3 m which was not fully penetrated after exploring it for about 0.6 m. Where fully penetrated, the silty clay layers are 0.5 m to 2.0 m thick.

The firm to very stiff silty clay had SPT N values of 6 to 16 blows per 0.3 m. The shear strength, based on one in situ shear vane test, was 122 kPa indicating a very stiff consistency. The sensitivity of the silty clay at the single test location where a remoulded strength could be determined was 2.4. Water contents in the silty clay ranged from 26 to 47 per cent. The results of the grain size distribution test conducted on one sample of silty clay are shown on Figure A-2 in Appendix A.

4.2.6 Silty Sand

The silty clay in Borehole 11-03 is underlain by a 0.4 m thick layer of silty sand from Elevation 60.6 m. Hydrocarbon odour and staining was noted in the silty sand. The silty sand was loose with an SPT N value of 8 blows per 0.3 m and a water content of 22 per cent.

4.2.7 Silty Sand to Sandy Silt Till

Glacial till material, ranging in gradation from silty sand, to sand and silt, to sandy silt, was encountered beneath the concrete in Boreholes 11-05 and 11-28 to 11-31 from Elevation 59.0 m to 59.5 m, below the fill in Borehole 11-01 from Elevation 59.4 m and beneath the silty clay in Boreholes 11-02 to 11-04 and 11-07 from Elevation 58.6 m to 60.7 m. The silty sand to sandy silt till layers were 4.3 m to 5.8 m thick with Boreholes 11-01 and 11-03 being terminated due to auger refusal on possible bedrock or a boulder. Boreholes 11-28 to 11-31 were terminated in the till after penetrating it for 1.1 m to 1.5 m. Cobbles and/or boulders were noted during drilling within the silty sand to sandy silt till.

The silty sand to sandy silt till has a compact to very dense relative density with SPT N values of 10 to over 100 blows per 0.3 m. Water contents of 3 to 12 per cent were measured in samples of the silty sand to sandy silt till.

The results of grain size analyses conducted on select samples of silty sand to sandy silt till are shown on Figures A-3 and A-4 in Appendix A.

4.2.8 Bedrock

Limestone bedrock of the Bobcaygeon Formation was encountered at the base of the silty sand to sandy silt till in Boreholes 11-02 to 11-05, 11-07 and 09-05 (31G-232). The bedrock surface was inferred based on auger refusal at Elevation 55.8 m at Borehole 11-01. The bedrock surface was encountered between Elevations 54.8 m and 55.2 m along the west abutment and between Elevations 52.9 m and 55.1 m along the east abutment. The following table summarizes the bedrock surface depth and elevation as encountered at the borehole locations.



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Location	Borehole No.	Ground Surface Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
West Wingwall	11-01	61.7	5.9	55.8
West Abutment	11-02	61.9	6.9	55.0
	11-05	61.7	6.9	54.8
	09-05 (31G-232)	68.5	13.3	55.2
East Wingwall	11-03A	61.5	6.4	55.1
East Abutment	11-04	61.9	7.5	54.4
	11-07	67.4	14.5	52.9

Samples of rock core were obtained in Boreholes 11-02, 11-04, 11-05 and 11-07 using an NQ-size core barrel. GEOCRETS Report No. 31G-232 indicates that NQ2-size coring equipment was used to retrieve samples of bedrock from Borehole 09-05. The Rock Quality Designation (RQD), Total Core Recovery (TCR) and Solid Core Recovery (SCR) for each rock core run is summarized in the following table:

Borehole No.	Elevation (m)		RQD (%)	TCR (%)	SCR (%)
	From	To			
11-02	55.0	53.5	85	95	75
	53.5	51.9	95	99	95
11-04	54.4	53.2	55	90	62
	53.2	51.9	93	102	101
	51.9	50.4	96	95	91
11-05	54.8	54.5	0	30	21
	54.5	52.9	86	92	87
	52.9	51.5	80	103	96
11-07	52.9	52.0	0	66	7
	52.0	50.8	92	99	97
	50.8	49.7	85	113	85
09-05 (31G-232)	55.2	54.7	94	94	39
	54.7	54.2	100	100	0
	54.2	52.9	100	100	78
	52.9	51.6	100	100	100
Average			78	92	69

The RQD varies between 0 and 100 per cent with an average of 78 per cent indicating poor to excellent but generally good quality rock. The bedrock above Elevation 52.0 m exhibited vertical fractures and was weathered, particularly above approximately Elevation 54.5 m. The estimated unconfined compressive strength, based on point load testing of four samples of rock core, ranged from 15 MPa to 149 MPa. The results of the point load testing are summarized in Table B1 in Appendix B. The unconfined compressive strength of one sample of rock core retrieved from Borehole 11-07 was 47.5 MPa. The results of the unconfined compression



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testing are summarized in Table B2 in Appendix B and presented on Figures B1A and B1B in Appendix B. The results of the rock strength testing indicate that the limestone bedrock is very weak to very strong but generally very strong. The low strength measured in the sample from Borehole 11-02 is likely attributable to the presence of vertical fractures near Elevation 54.8 m in that borehole.

4.3 Groundwater Conditions

Groundwater conditions were observed during and upon completion of drilling. Groundwater was encountered in Boreholes 11-01 to 11-05, 11-07, 11-30 and 11-31 between Elevations 59.9 m and 61.3 m, while Borehole 09-05 was dry upon completion. These groundwater levels are not considered to be representative of stabilized groundwater levels. The following table is a summary of encountered groundwater levels.

Borehole No.	Ground Surface Elevation (m)	Encountered Groundwater Level	
		Depth (m)	Elevation (m)
11-01	61.7	1.4	60.3
11-02	61.9	1.5	60.4
11-03	61.6	1.0	60.6
11-03A	61.5	1.4	60.1
11-04	61.9	1.5	60.4
11-05	61.7	1.8	59.9
11-07	67.4	6.1	61.3
11-28	61.9	*	*
11-29	61.9	*	*
11-30	61.7	0.8	60.9
11-31	61.7	0.8	60.9
09-09 (31G-232)	67.8	6.5	61.3
09-10 (31G-232)	67.7	7.3	60.4

* Note: Groundwater level not established at Boreholes 11-28 and 11-29 due to hydro-vacuuming techniques.

A standpipe was installed in Borehole 11-03A. The most recent reading in the standpipe was obtained on March 7, 2012, at which time the groundwater level was measured at Elevation 60.5 m. The measured groundwater levels are summarized in the following table:

Borehole	Ground Surface Elevation (m)	Measured Groundwater Level							
		June 16, 2011		June 27, 2011		November 1, 2011		March 7, 2012	
		Depth (m)	Elevation (m)	Depth (m)	Elevation (m)	Depth (m)	Elevation (m)	Depth (m)	Elevation (m)
11-03A	61.5	1.5	60.0	1.4	60.1	1.4	60.1	1.0	60.5



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The average stabilized groundwater level at this site has been inferred to be at approximately Elevation 60.5 m based on the colour change from brown to grey and the observed groundwater levels in the open boreholes. Groundwater levels are expected to fluctuate seasonally and are expected to rise during wet periods of the year.

5.0 CLOSURE

This investigation was carried out using equipment supplied and operated by Marathon Drilling Ltd., an Ontario Ministry of Environment licensed well contractor. The field operations were supervised by Mr. Randy Axford and Mr. Paul Hulan under the direction of Mr. Fin Heffernan, P.Eng.

The routine laboratory testing was carried out at Golder's London and Ottawa laboratories under the direction of Mr. Chris M. Sewell and Mr. Chris Mangione, P.Eng., respectively. The point load and unconfined compressive strength testing was conducted at Golder's Mississauga laboratory under the direction of Dr. J. Paul Dittrich, P.Eng. The concrete testing was conducted at Golder's Ottawa laboratory. All three laboratories are accredited participants in the MTO Soil and Aggregate Proficiency Program and are certified by the Canadian Council of Independent Laboratories for testing Types C and D aggregates. The Mississauga laboratory is registered with the MTO in the specialty of soil and rock including testing for foundation engineering (low and high complexity).

This report was prepared by Mr. Brett Thorner and Ms. Dirka U. Prout, P.Eng. under the direction of Ms. Lisa Coyne, P.Eng., a Principal with Golder. An independent quality review of this report was conducted by Mr. Fin Heffernan, P.Eng., the Designated MTO Foundations Contact and Quality Control Auditor for this assignment.

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PART B

FOUNDATION DESIGN REPORT

EASTBOUND OVERPASS

CORNWALL CENTRE ROAD OVERPASS (SITE 31-209)

HIGHWAY 401, CORNWALL, ONTARIO

GWP 4029-08-00

MINISTRY OF TRANSPORTATION, ONTARIO - EASTERN REGION



6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides detail foundation design recommendations for the proposed replacement of the Highway 401 eastbound-Cornwall Centre Road overpass. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigations. 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 detail design of the structure foundations.

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

The twin Cornwall Centre Road overpass structures, together with the median retaining walls and wingwalls/retaining walls are to be replaced. The currently proposed preferred alternative involves the use of rapid bridge replacement (RBR) techniques for both the westbound and eastbound overpass structures. Precast footings and other structural elements will be pre-assembled in staging areas on the north and south ends of Highway 401 and transported using specialized heavy-lift equipment to their final locations. The entire structure may be assembled and transported in a single move or the footings and top rigid frame may be assembled and transported in two separate moves. Consideration is being given to re-using the existing spread footings below the precast footings for the new overpass structures, if appropriate and feasible. However, as required by the Terms of Reference for this assignment, various foundation alternatives have been considered for support of the replacement structures.

The subsoils encountered in the boreholes completed in the vicinity of the eastbound overpass and associated retaining walls generally consist of surficial fills below Cornwall Centre Road grade to approximately Elevation 59 m to 61 m. The embankment fill below the Highway 401 grade generally consists of loose to very dense granular material with very stiff cohesive fill noted in the lower 0.7 m in Borehole 11-07. The embankment fill is generally underlain by firm to very stiff silty clay to about Elevation 59 m to 61 m, then compact to very dense silty sand to sandy silt till. The surface of the limestone bedrock was encountered in the boreholes between approximately Elevation 53 m and 55 m. The stabilized groundwater level at the site is interpreted to be at approximately Elevation 60.5 m.

6.2.1 Replacement of Eastbound Overpass Structure

The General Arrangement drawing for the original structure (Drawing D-4517-1, dated March 1960) indicates that the structure was to be supported on 4.3 m wide, 0.76 m thick spread footings. The design drawing indicates that the abutment footings were to bear directly on the 'dense sandy silt till', with the design founding level at approximately Elevation 59.4 m. Based on the thickness and base elevation of the silty clay deposit as



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encountered in boreholes in the initial stage of the subsurface investigation, it was thought that the existing abutment footings might be founded on the silty clay deposit. However, the results from Boreholes 11-28 to 11-31 advanced through the footings confirm that the eastbound overpass abutment footings were constructed directly on the compact to very dense sandy silt till. It appears that any silty clay material present in the area of the footings was sub-excavated and replaced with concrete, such that the concrete is thicker than in the original footing design. The elevations of the top and base of the concrete, as encountered at the borehole locations, are summarized in the following table.

Footing Location	Borehole No.	Elevation of Top of Footing (m)	Thickness of Concrete (m)	Elevation of Base of Concrete (m)
West Abutment	11-28	60.3	1.3	59.0
	11-29	60.2	1.1	59.2
East Abutment	11-30	60.2	0.9	59.3
	11-31	60.4*	0.9	59.5

***Note:** The top of footing elevation at Borehole 11-31 is considered to be not representative. A top of footing elevation of 60.2 m should be used for the east abutment footing.

Based on the results of the boreholes and the existing overpass foundation conditions and performance, shallow foundations are considered to be the preferred alternative from a foundations perspective for the replacement of the eastbound structure and the associated wingwalls. Shallow foundations are more cost-effective than deep foundations (discussed below) for both conventional and rapid bridge replacement of the structure, whether by re-using the existing foundations, or removing the existing foundations and constructing new footings at the same founding elevation as the existing or on top of compacted Granular A fill. For the RBR option in particular, the re-use of the existing foundations is considered to be a significant advantage.

Deep foundations, including steel H-piles, steel tube piles or caissons, are not considered warranted or practical at this site in comparison to shallow foundations, since a competent stratum (the compact to very dense till deposit) is located at relatively shallow depth below the Cornwall Centre Road grade. Pre-augering would be required to install driven piles due to the very dense soil conditions and the presence of cobbles and boulders at relatively shallow depth. For example, at the south end of the west abutment, the silty sand to sandy silt till contains cobbles and boulders and has an SPT N value of greater than 100 blows per 0.3 m of penetration near Elevation 59 m (approximately 3 m below Cornwall Centre Road grade). In the northeast quadrant, the till is dense to very dense with cobbles and boulders observed or inferred around Elevation 57 m. Due to the hard driving conditions, requirement for pre-augering, and relatively short pile or caisson length, the use of deep foundations is not considered practical for support of the overpass replacement structure; therefore, design recommendations for deep foundations are not treated further in this report.

A comparison of foundation alternatives, including advantages, disadvantages and risks/consequences, is presented in Table 1. Approximate relative costs are also provided in this table; however, the costs provided are meant to provide a basis of comparison amongst the alternatives and are not indicative of actual constructions costs. Recommendations for design of shallow foundations are provided in Section 6.3, and recommendations related to backfill, drainage, lateral earth pressures and earthquake loading for design of the abutment walls are provided in Section 6.8.



6.2.2 Replacement of South Wingwalls/Retaining Walls

It is understood that there is no grade raise planned for Highway 401, and the embankment geometry will remain essentially unchanged with the exception of an approximately 1 metre widening on the median shoulders, with placement of about 1.2 m of additional fill associated with this widening. Therefore, no significant change in embankment loading will occur at the south wingwall/retaining wall locations, although it is anticipated that the wingwalls/retaining walls may extend slightly beyond the footprint of the current retaining walls, with some nominal increase in loading in these areas.

The existing west median retaining wall was exhibiting signs of movement and distress, with rotation towards Cornwall Centre Road and vertical cracking. Some vertical cracking and minor movement were also observed on the east median wall. These walls were the subject of a 2009 investigation and assessment by Thurber Engineering Ltd., following by remedial measures (tie-backs attached to dead-man anchors) to restrain the movement of the west median retaining wall. Based on Thurber's assessment, the cause of the existing median wall distress was concluded to be related to poor quality backfill behind the retaining walls; the existing retaining wall foundations were considered to be performing satisfactorily and were not considered to have contributed to the observed wall distress. It is noted that the replacement of the median retaining walls is addressed more thoroughly in the Foundation Investigation and Design Report prepared by Golder for the westbound overpass.

As for the overpass replacement, consideration may be given to constructing some types of replacement walls on top of the existing south wingwall/retaining foundations. The supplementary investigation conducted by Golder to assess the presence of silty clay below the existing footings applied specifically to the abutment footings; borings were not specifically completed through the retaining wall footings to ascertain whether subexcavation of any silty clay (if present) was completed. It is possible that silty clay, if encountered at the footing elevation during construction, was removed from within the footprints of the south wingwall/retaining wall footings. However, if these existing footings are founded on the silty clay, based on the results of other boreholes at the site, it is anticipated that the preconsolidation pressure of the silty clay on the south side of the highway will be above 300 kPa. This, together with the limited change in embankment geometry and loading, suggests that the risk of settlement associated with constructing new retaining walls on top of the existing foundations will be acceptably low, even if footings for the existing south wingwalls/retaining walls bear on silty clay.

Various wall and foundation types have been considered for the replacement of the wingwalls and retaining walls associated with the eastbound overpass structure. It is noted that the subsurface conditions at the median retaining walls are treated in greater detail in the Foundation Investigation and Design Report that addresses the westbound overpass structure; however, the discussion presented below applies equally to the retaining walls located south of the eastbound structure and to the median retaining walls. A summary of the advantages and disadvantages associated with each option is provided below, and a comparison of the alternative wall types and foundation options based on advantages, disadvantages, risks and relative costs is provided in Table 2 following the text of this report.

- **Conventional concrete cantilever or counterfort walls:** Conventional cast-in-place concrete walls are highly adaptable and well-established in their design and construction methods. However, their cost can exceed the cost of other wall options for similar project conditions, especially if the walls are relatively high. A number of precast versions of conventional cantilever or counterfort retaining walls



are also available. In general, such walls are constructed at concrete precasting plants to standard panel dimensions. Once at a construction site, the precast panels are then attached to a cast-in-place concrete footing with similar dimensions as for cast-in-place concrete walls. Precast concrete walls offer the advantages of construction speed and potentially reduced costs as the formwork is largely eliminated on site. Shallow foundations are preferred over deep foundations if concrete retaining walls are adopted at this site, for the reasons identified in Section 6.2.1 and Table 1. As discussed earlier in this section, it is also considered that the existing retaining wall footings are suitable for re-use to support new retaining walls.

- **Retained soil system (RSS) walls:** RSS walls are feasible for the replacement walls at this site, and could be constructed on top of the existing retaining wall footings without requiring their demolition and removal. This type of wall system can be constructed relatively quickly, which is a distinct advantage for this project given the limited timeline available to complete construction of the new retaining walls under a rapid bridge replacement scenario. The possibility of using cellular concrete in lieu of compacted granular backfill was explored but not adopted since it did not offer any time savings as noted in Section 6.5. RSS walls are cost effective relative to other types of retaining structures; however, it is noted that if cellular concrete is adopted the material costs would increase, but could be partially offset by reduced costs in terms of labour and overall time for construction. At this site, it is anticipated that there will be sufficient space for the reinforcing strips or geo-grids, without additional excavation into the existing Highway 401 embankment fill behind the existing abutments/walls. It is understood that there is no grade raise planned for Highway 401 and the embankment geometry will remain essentially unchanged; therefore, no significant settlement is anticipated for the walls, with the potential exception of small portions of the walls on the south side of the highway that could extend beyond the south end of the existing wall footings. Vertical slip joints could be used to accommodate any differential settlement at this transition point in the RSS walls. As an alternative, a low concrete “toe wall” could be used for those portions of the walls beyond the ends of the existing footing, if the wall is sufficiently low, to minimize the visual impact of any potential differential settlement at the transition point.
- **Soldier pile and concrete panel walls or anchored sheet-pile walls:** A soldier pile and concrete panel wall or anchored sheet-pile wall could be considered for replacement of the wingwalls/retaining walls, although these wall types are generally more advantageous in “top-down” construction applications (i.e., when excavating into existing native soils or fill materials, rather than to support embankment fill that is being constructed from the base up). Soldier pile or sheet-pile installation will be difficult in the hard ground conditions. In addition, for the anticipated wall heights at this site, lateral restraint would be required in the form of soil anchors or tie-backs; the presence of cobbles/boulders in the till deposit could impact the installation of tie-backs. It is anticipated that construction of soldier pile and concrete panel walls would be more time consuming than construction of RSS walls or precast concrete walls due to the various steps involved (i.e., auger holes; place and concrete soldier piles; affix facing panels and place and compact backfill in lifts; and install and pre-stress tie-backs, possibly at multiple levels depending on wall height).



- **Soil nail walls:** A permanent soil nail wall (in which concrete panels are affixed to the front of the steel mesh, shotcrete and soil nail assemblage) is not considered appropriate for replacement of the wingwalls/retaining walls at this site, as this type of wall is generally used only in “top-down” construction application.

From a foundations perspective, RSS walls with granular backfill are considered to be the most practicable and cost-effective option for replacement of the existing retaining structures at this site, as they can be readily constructed over the existing abutment and wall footings (i.e., excavation and removal of the existing footings would not be required), and they can be constructed quickly to meet the aggressive construction schedule that would be required for the rapid bridge replacement scheme. The use of concrete retaining walls (cast-in-place or precast) may also be appropriate, although the length of time to construct cast-in-place concrete walls may preclude their use in a rapid bridge replacement scheme. As noted in the discussion on RSS walls above, the use of low concrete toe walls may be appropriate for short, low-height sections of retaining walls that extend beyond the limits of the existing wall foundations.

Geotechnical recommendations for concrete retaining walls, RSS walls, and concrete toe walls are provided in Sections 6.4, 6.5 and 6.6, respectively, and recommendations related to lateral earth pressures and earthquake loads on concrete retaining walls are provided in Section 6.8.

6.3 Shallow Foundations for Overpass Structure

6.3.1 Founding Level and Frost Protection Requirements

The following table provides the maximum (highest) founding elevations recommended for design of new abutment footings founded directly on the compact to dense sandy silt till.

Foundation Element	Borehole Nos.	Maximum Footing Founding Elevation (m)
West Abutment	11-02, 11-05, 11-28, 11-29	59.0
East Abutment	11-04, 11-07, 11-30, 11-31	59.3

All footings should be founded at a minimum depth of 1.7 m (or provided with the thermal equivalent in insulation) for frost protection purposes. This frost protection depth is based on Ontario Provincial Standard Drawing (OPSD) 3090.101.

Alternatively, the existing foundations may be left in place and new cast-in-place footings constructed or precast footings placed on top of the existing concrete. For this case, the surface of the existing west and east abutment footings for the eastbound overpass structure are as follows:



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Footing Location	Borehole No.	Elevation of Top of Footing (m)
West Abutment	11-28	60.3
	11-29	60.2
	TP 09-05	60.1
East Abutment	11-30	60.2
	11-31	60.2*

* **Note:** As noted in Section 4.2.4, the top of footing elevation at Borehole 11-31 was directly surveyed at Elevation 60.22 m on February 2, 2012 by an OLS.

The surface of the existing concrete footings would not constitute a frost-susceptible subgrade, therefore it would not be necessary for the new footings to be founded below the frost depth as identified above.

6.3.2 Geotechnical Resistance

New Footings Supported on Glacial Till Deposit

The eastbound overpass replacement structure, together with associated wingwalls/retaining walls, can be supported on cast-in-place strip or spread footings founded on the compact to dense silty sand to sandy silt till at or below the elevations provided in Section 6.3.1. The design should be based on a factored geotechnical resistance of 600 kPa at Ultimate Limit States (ULS) and a geotechnical resistance of 400 kPa at Serviceability Limit States (SLS), assuming 25 mm of settlement. These geotechnical resistances are provided for loads applied perpendicular to the surface of the footings. For inclined loads, the geotechnical resistances should be modified as per Section 6.7.4 of CHBDC and its Commentary.

New Footings Supported on Existing Footings

If the existing footings are left in place and new footings (cast-in-place or precast) constructed on top, the geotechnical resistances provided above are also applicable. The existing footings themselves could be treated simply as mass concrete below the new footings.

It is understood that for an RBR approach, it is likely that 2.4 m wide precast footings would be used on top of the existing 4.3 m wide abutment footings. Additional assessment of footing settlement performance during the course of the rapid bridge replacement operations may be conducted based on the bulk modulus of subgrade reaction for the glacial till deposit. This value will be variable due to the range of SPT N values encountered in the glacial till deposit in the boreholes drilled for this investigation. However, for the purposes of this design assessment, a minimum modulus of elasticity of 30 MPa may be used. The structural design may also be checked using a subgrade reaction modulus of approximately 16 MPa/m.



6.3.3 Resistance to Lateral Forces/Sliding Resistance

Cast-in-Place Footings on Glacial Till Deposit

Resistance to lateral forces/sliding between new cast-in-place concrete footings and the silty sand to sandy silt till foundation soils should be calculated in accordance with Section 6.7.5 of the CHBDC.

- For cast-in-place concrete footings constructed on the compact to very dense silty sand to sandy silt till, the coefficient of friction, $\tan \phi'$, can be taken as 0.60.

In accordance with CHBDC, a factor of 0.8 is to be applied in the calculation of the factored horizontal resistance (this factor is included in the equation given in Section 6.7.5 of CHBDC).

New Footings Supported on Existing Footings

Cast-in-place or precast footings could be constructed on top of the existing footings for either a conventional or rapid bridge replacement. It is recommended that consideration be given to roughening the surface of the existing footings prior to constructing cast-in-place footings. In the case of new precast footings, it is anticipated that it would be necessary to place a concrete levelling pad on top of the existing foundations, to ensure a level and even surface on which to place the new footings. Based on discussions with MRC regarding the use of precast footings for an RBR approach, it is understood that a post-grouting scheme would be proposed to fill any "gap" created by placement of 10 mm shims between the concrete levelling pad and the new precast footings.

For the assessment of sliding resistance between precast (formed) concrete footings on screeded concrete, and assuming the use of post-grouting, it is recommended that a coefficient of friction of 0.6 be used.

To supplement the sliding resistance and provide additional resistance to lateral forces, mechanical attachments such as dowels may be used to secure the new footings to the existing footings; the dowels should be designed by the structural engineer. Lightweight fill could also be used behind the abutment walls or retaining walls to reduce the active thrust on the walls.

6.4 Shallow Foundations for Conventional Concrete Retaining Walls

6.4.1 Founding Level and Frost Protection Requirements

The following table provides the maximum (highest) founding elevations recommended for design of new retaining wall footings founded directly on the compact to dense silty sand to sandy silt till.

Foundation Element	Borehole Nos.	Maximum Footing Founding Elevation (m)
Southwest Wingwall/ Retaining Wall	11-01, 11-02	59.4
Southeast Wingwall/ Retaining Wall	11-03A, 11-04	60.0*

* Note: Frost protection depth of 1.7 m, or equivalent insulation, required.



All footings should be founded at a minimum depth of 1.7 m (or provided with the thermal equivalent in insulation) for frost protection purposes. This frost protection depth is based on Ontario Provincial Standard Drawing (OPSD) 3090.101.

Alternatively, the existing foundations may be left in place, and new cast-in-place footings constructed or precast footings placed on top of the existing concrete. The surface of the existing concrete footings would not constitute a frost-susceptible subgrade, therefore it would not be necessary for the new footings to be founded below the frost depth as identified above. If this wall type is adopted, the required footing width would be dependent on the wall height; if at any location the new footings were wider than the existing footings, it is recommended that subexcavation of the soils within the footprint of the new footings (outside of the existing footing) be completed and replaced with a minimum thickness of 0.5 m of compacted Granular A.

6.4.2 Geotechnical Resistance

New Footings Supported on Glacial Till Deposit or Compacted Granular Fill

The geotechnical recommendations provided in Section 6.3.2 for the eastbound overpass replacement are applicable for new concrete wingwall/retaining wall footings supported on the glacial till deposit. In the case of footings constructed on a granular pad, the pad is to comprise Granular A placed and compacted in accordance with MTO's SP 105S21. The pad is to have a minimum thickness of 0.5 m and is to extend horizontally a minimum of 1 m beyond the all edges of the footing then slope at 1 horizontal to 1 vertical, for a distance equivalent to the thickness of the fill.

New Footings Supported on Existing Footings

If the existing footings are left in place and new footings (cast-in-place or precast) constructed on top, the geotechnical resistances provided in Section 6.3.2 are also applicable.

6.4.3 Resistance to Lateral Forces/Sliding Resistance

Cast-in-Place Footings on Glacial Till Deposit or Compacted Granular Fill

Recommendations for lateral resistance for retaining wall footings supported on the glacial till are provided in Section 6.3.3. For cast-in-place footings constructed on compacted Granular A, the coefficient of friction, $\tan \phi'$, can be taken as 0.70.

New Footings Supported on Existing Footings

Cast-in-place or precast footings could be constructed on top of the existing footings for new concrete retaining walls. In the case of new precast footings, it is anticipated that it would be necessary to place a concrete levelling pad on top of the existing foundations to ensure a level and even surface on which to place the new footings. It is recommended that consideration be given to roughening the surface of the existing footings prior to constructing cast-in-place footings, roughening the surface of the new concrete levelling pad prior to attaching



precast footings, and/or using a fluid grout to “post grout” any gap between the existing and new foundations. For the assessment of sliding resistance between precast (formed) concrete footings on screeded concrete, and assuming the use of post-grouting, it is recommended that a coefficient of friction of 0.6 be used.

To supplement the sliding resistance and provide additional resistance to lateral forces, mechanical attachments such as dowels may be used to secure the new retaining wall footings to the existing footings; the dowels should be designed by the structural engineer. Lightweight fill could also be used behind the retaining walls to reduce the active thrust on the walls.

6.4.4 Lateral Earth Pressures for Design of Concrete Retaining Walls

Recommendations are provided in Section 6.8 regarding backfill, drainage, lateral earth pressure design and earthquake loading for concrete retaining walls.

6.5 Reinforced Soil System (RSS) Walls

If RSS walls are adopted for the replacement of the southwest and southeast wingwalls/retaining walls associated with the replacement of the eastbound overpass structure, they could be constructed over the existing footings; it is understood that depending on the geometry, the southern portions of the walls may extend beyond the limit of the existing wall footings.

The RSS retaining walls are to be designed for high performance and appearance in accordance with MTO Special Provision (SP) 599S22 and the Non-Standard Special Provision for the design and construction of RSS walls dated September 2005.

In conventional RSS wall construction, the retained soil mass is constructed using Granular A material that is placed and compacted in lifts, with reinforcing strips placed at regular intervals within the soil mass. In a rapid bridge replacement scheme, there will be a limited timeline and limited working area for reconstruction of the median retaining walls; the trucks and construction equipment to deliver, spread and compact the granular fill may further add to the congestion in the median area during the wall reconstruction.

As an alternative to granular fill, the use of cellular concrete backfill was considered in the reconstruction of the median retaining walls (the geotechnical recommendations for which are addressed in greater detail in the Foundation Investigation and Design Report for the westbound overpass structure). Cellular concrete is a lightweight material that is typically used in applications where its reduced weight improves settlement performance and can accelerate the construction schedule by reducing preloading time, although this factor is not a significant consideration for the Cornwall Centre Road overpass site. Cellular concrete does present advantages for the schedule associated with a rapid bridge replacement scheme in that it can be placed remotely (pumped from a distance) and is self-compacting and self-supporting once cured, thus reducing construction traffic to/from the site, congestion on site, and required labour forces relative to conventional placement and compaction of granular fill. Based on conversations with cellular concrete companies, it is understood that the foam concrete pumps easily with relatively low pressures via an approximately 75 mm diameter hose. Depending on the foam concrete plant used, placed volumes of approximately 75 m³/hour to



120 m³/hour can be achieved, pumping from distances of up to 250 m or more away from the placement area. Although an initial set is usually attained within 90 minutes, a subsequent lift cannot be placed until a further curing period of six to ten hours has elapsed. Unlike conventional granular fill, “set” cellular concrete does not apply active lateral pressure against the back of the wall when used as backfill for bridge abutments and retaining walls.⁶

For highway applications, a higher density (stronger) foaming agent is typically recommended. This higher density foam is recommended to be placed in approximately 0.6 m thick lifts, which can be “stepped” using formwork, with the anchor points for the permanent wall facing panels embedded into the cellular concrete during placement of the lifts. Compared to using granular backfill, the use of cellular concrete will increase the ease of backfill placement by removing the need for compactive effort. However, it has been estimated that it may take longer to place cellular concrete backfill due to the length of the curing period prior to placement of each additional lift. Therefore use of cellular concrete backfill is not considered to be an overall time saving option compatible with the RBR process.

A comparison of the advantages and disadvantages of cellular concrete relative to granular fill in an RSS wall application is included as part of Table 2 following the text of this report. This table also provides approximate costs for comparison purposes, which have been developed based on discussions with a cellular concrete company in Canada based on the approximate working area and volume required for reconstruction of the median retaining walls at this site.

6.5.1 Founding Elevations

A typical RSS wall has a front facing supported on a strip footing placed at shallow depth below the ground surface in front of the wall; this footing, and the reinforced soil mass, should be founded below any topsoil, loose fill or unsuitable native soils.

For this site, where the RSS wall (the facing footing plus the reinforced soil mass) overlies the existing footing, it is recommended that it be supported directly on the footing, although it could also be constructed on a granular pad placed on top of the existing footing. The top of the existing footing elevation may be taken to be approximately Elevation 60.3 m (assuming that the top of existing footing elevation for the wingwalls is similar to that for the top of the existing abutment footings).

Where the RSS wall extends beyond the limit of the existing footings, it is recommended that the native soils be subexcavated a further 0.5 m and replaced with compacted Granular A or Granular B Type II. The proposed underside of the 0.5 m thick granular pad would therefore be at Elevation 59.8 m, relative to the top of the existing wall footings. Based on the boreholes advanced on the south side of the structure, the facing footings would be constructed on dense to very dense glacial till, which will result in good differential settlement performance at the transition point between the portion of the RSS wall supported on the existing footing and the portion that extends beyond the footing.

⁶ American Concrete Institute, 2006: Guide for Cast-in-Place Low-Density Cellular Concrete, ACI 523.1R-06, American Concrete Institute, Farmington Hills.



However, this would require the subexcavation and placement of granular fill to extend below the water level at the site. To minimize the requirements for groundwater control for those portions of the RSS walls that extend beyond the existing footings, consideration could be given to subexcavating to approximately Elevation 60.5 m (above the existing top of footing, and roughly at the groundwater level at the site) and backfilling with compacted Granular A or Granular B Type II as before. There is a slightly greater risk for the presence of silty clay below the RSS wall footing and soil mass with this higher founding elevation, and therefore a slightly greater risk of differential settlement at the transition point. However, based on the subsurface conditions encountered in the boreholes, it is anticipated that the silty clay may be absent in some areas on the south side of the highway or where present would most likely have a very stiff, weathered consistency ("crust").

6.5.2 Geotechnical Resistance

It has been assumed that each RSS wall will act as a unit and utilize the full width of the reinforced soil mass, which has been taken to be 0.8 times the height of the wall, with a facing footing width of at least 0.5 m.

- For those RSS walls that are constructed over the existing footings, the geotechnical resistances provided in Section 6.3.2 are applicable.
- Where the RSS walls extend beyond the limits of the existing footings with subexcavation to Elevation 59.8 m (with appropriate groundwater control) and construction of a compacted granular pad, the design can be based on a factored geotechnical resistance at ULS of 300 kPa and a geotechnical resistance at SLS (for 25 mm of settlement) of 200 kPa.
- Where the RSS walls extend beyond the limits of the existing footings but are founded above the groundwater level with excavation to Elevation 60.5 m and placement of a compacted granular pad, the design can be based on a factored geotechnical resistance at ULS of 225 kPa and a geotechnical resistance at SLS (for 25 mm of settlement) of 150 kPa to account for the potential presence of some silty clay below the founding level.

6.5.3 Settlement

No grade raises or other changes in embankment geometry have been proposed. Therefore, it is anticipated that the settlement performance for RSS walls and facing panels that are founded on the existing footings or following subexcavation to Elevation 59.8 m will be acceptable, as these footings have already been subjected to similar loading. Some minor differential settlements along the retaining wall segments should be expected, although the differential settlements along the wall are expected to be less than 1 per cent of the wall length; therefore, for this founding level, precast concrete panels or block facings could be used. If a higher founding level is used for the portion of the RSS walls that extend beyond the existing footing, the potential for differential settlement is slightly higher. It is recommended that a vertical slip joint be provided where the RSS wall extends beyond the existing footing.

Differential settlement(s) will also occur where RSS retaining walls meet existing cast-in-place structures such as where the RSS wingwalls/retaining walls join the overpass structure. The RSS wall design must accommodate the anticipated differential settlements and prevent loss of fines from the backfill.



6.5.4 Resistance to Lateral Forces

The resistance to lateral forces/sliding resistance between the compacted fill of the reinforced soil mass (assumed to be Granular A) and the subgrade (taken as the existing concrete footing, Granular A or Granular B Type II) should be calculated in accordance with Section 6.7.5 of the CHBDC. The coefficient of friction, $\tan \phi'$, between the compacted granular fills of the RSS wall and the properly prepared subgrade may be taken as 0.6. This represents an unfactored value; in accordance with the CHBDC, a factor of 0.8 is to be applied in calculating the horizontal resistance.

6.5.5 Stability

The internal stability of the mechanically-reinforced soil walls should be verified by the RSS supplier/designer. The external (global) stability of a typical RSS retaining wall height of maximum 8 m has been assessed assuming a reinforcement length of 0.8 times the wall height. The walls were found to be stable based on achieving minimum Factors of Safety of 1.5 for sliding and 2.0 for eccentricity/overturning and bearing. Therefore, the walls are considered to be globally stable having met or exceeded the aforementioned minimum factors of safety normally used for design under static conditions. For the dynamic case, the minimum Factors of Safety for sliding, eccentricity/overturning and bearing are reduced to approximately 75 per cent of the static values. The walls were found to be stable under dynamic conditions, based on achieving a target factor of safety of 1.1 against global failure under dynamic conditions. These minimum factors of safety are considered appropriate for the RSS walls at this site, considering the design requirements and the field data available.

6.5.6 Utilities and Highway Infrastructure

It is preferred that utilities with alignments parallel to the wall face not be placed within the reinforcement zone. The design of the RSS retaining walls must consider the proposed highway infrastructure. This may require construction of a structural frame around the obstruction or splaying or full or partial omission of reinforcement in the area of the obstruction. The adjacent reinforcement must be designed to accommodate the additional loading resulting from removal of reinforcing elements in the area of the obstruction.

6.6 Concrete Toe Walls

Concrete toe walls may be considered for low-height sections of retaining walls south of the eastbound structure, specifically for those portions of the walls that extend beyond the limits of the existing wall foundations. Concrete toe walls should be designed in accordance with the requirements shown on OPSD 3120.100 (*Retaining Walls – Concrete Toe Wall*).

Where walls less than approximately 0.8 m in height are required, Type I toe walls may be considered. This wall type requires a minimum embedment depth of 0.45 m but also requires a minimum factored geotechnical resistance at Ultimate Limit States of 200 kPa. In order to achieve this geotechnical resistance for walls on the south side of the eastbound overpass structure, the wall footprint should be subexcavated to Elevation 60.5 m; the excavation can then be backfilled with compacted Granular A or Granular B Type II up to such level as provides for the minimum embedment depth of 0.45 m for this type of concrete toe wall.



Where walls of greater than 0.8 m in height but less than 1.8 m in height are required, Type II or Type III toe walls may be considered. This wall type requires a minimum embedment depth of 0.45 m, but also requires a minimum factored geotechnical resistance at Ultimate Limit States of 300 kPa. In order to achieve this geotechnical resistance for walls on the south side of the eastbound overpass structure, the wall footprint should be subexcavated to Elevation 59.8 m; the excavation can be backfilled with a 0.5 m thick layer of compacted Granular A or Granular B Type II, with the toe walls then constructed at approximately Elevation 60.3 m.

6.7 Liquefaction Potential and Seismic Analysis

6.7.1 Seismic Parameters

The site is located in Cornwall, Ontario. According to Table A.3.1.1 of the CHBDC, the zonal acceleration ratio, A , applicable to this site is 0.20. The corresponding acceleration related seismic zone, Z_a is 4. The following seismic performance zones (SPZ) are applicable to the proposed structure based on the assigned importance category:

Importance Category	Seismic Performance Zone
Emergency route and other bridges	3
Lifeline bridge	3

The replacement eastbound structure and wingwalls must meet the minimum requirements for earthquake analysis as outlined in CHBDC Clause 4.4.5.1. The effects of site conditions on the bridge response are to be included in the determination of the seismic loads. The stratigraphy generally consists of surficial topsoil or pavement structure overlying a predominantly granular embankment fill. The fill is underlain in sequence by firm to very stiff silty clay then compact to dense silty sand to sandy silt till. The bedrock of the Bobcaygeon Formation was encountered at an approximate depth of 6 m to 9 m below the Cornwall Centre Road grade or between Elevation 53 m to 56 m. Based on the site stratigraphy, the soil profile type is categorized as Type I with a seismic site response coefficient, S , of 1.0 based on the CHBDC criteria.

6.7.2 Seismic Hazard Assessment

A preliminary screening of the soil stratigraphy was conducted using the procedure outlined in the Federal Highway Administration recommended procedures⁷ and Canadian Foundation Engineering Manual (CFEM). A saturated layer of loose sand was only encountered in Borehole 11-03. This layer is only 0.9 m thick but will be removed if encountered within the footprint of the foundations. Compact to very dense silty sand to sandy silt till is the predominant overburden material. Even though the till is saturated, the average normalized SPT value is greater than 22 blows per 0.3 m and often greater than 30 blows per 0.3 m. The overlying silty clay is not considered to be susceptible to liquefaction or cyclic mobility. However, the silty clay materials may undergo

⁷ Federal Highway Administration (FHWA). (1997). "Design Guidance: Geotechnical Earthquake Engineering For Highways. Volume I – Design Principles." *Geotechnical Engineering Circular No. 3: FHWA-SA-97-076*, Washington, D.C.



significant deformations during a seismic event if the cyclic shear stress is greater than the static undrained shear strength.

6.8 Lateral Earth Pressures

The lateral pressures acting on the eastbound overpass abutment walls and associated wingwalls and retaining walls will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls. Seismic (earthquake) loading must also be taken into account in the design.

The following recommendations are made concerning the design of the abutment stems and/or retaining walls:

- Select, free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B but with less than 5 per cent passing the No. 200 sieve should be used as backfill behind the abutments and retaining walls. This fill should be compacted in accordance with MTO's SP 105S21. Longitudinal subdrains should be installed to provide positive drainage of the granular backfill. Drainage can be enhanced through the use of wall drains with weep holes, although use of weep holes may not be permitted as there are sidewalks in front of abutment walls and retaining walls along Cornwall Centre Road. In accordance with Section 5.2.1 of MTO's Structural Manual, the weep holes and wall drain may be replaced with an alternative drainage measure such as a geocomposite sheet drains (e.g. Miradrain). Alternatively, if weep holes are not used, the size of the wall drains should be enlarged to accommodate the flow that would otherwise be relieved through the weep holes. Other aspects of the abutment granular backfill requirements with respect to subdrains, wall drains, weep holes and frost taper should be in accordance with Ontario Provincial Standard Drawing (OPSD) 3101.150, 3121.150 and/or 3190.100.
- A minimum compaction surcharge equal to 12 kPa should be included in the lateral earth pressures for the structural design of the abutment walls or retaining walls, in accordance with CHBDC, Figure 6.6. Compaction equipment should be used in accordance with MTO's SP 105S21.
- The granular fill may be placed either in a zone with a width equal to at least 1.7 m behind the back of the stem (Case (a) from Commentary on CHBDC, Figure C6.20) or within the wedge-shaped zone defined by a line drawn at a maximum 1 horizontal to 1 vertical extending up and back from the rear face of the footing (Case (b) from Commentary on CHBDC Figure C6.20).
- For Case (a), the restrained case, the pressures are based on the existing or proposed embankment fill materials and the following parameters (unfactored) may be used assuming the use of Select Subgrade Material (SSM) or similar earth fill:

Soil unit weight:	20 kN/m ³
Coefficients of lateral earth pressure:	
Active, K_a	0.33
At rest, K_o	0.50
Passive, K_p	3.0



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- For Case (b), the unrestrained case, the pressures are based on the granular fill as placed and the following parameters (unfactored) may be assumed for Granular A or Granular B Type II. If cellular concrete is used as backfill behind the walls (unit weight of 5 kN/m^3), once set, it would not exert lateral earth pressures against the structure unless load is being transferred from behind the cellular concrete fill.

	Granular A	Granular B Type II	Cellular Concrete
Soil unit weight:	22 kN/m^3	21 kN/m^3	5 kN/m^3
Coefficients of lateral earth pressure:			
Active, K_a	0.27	0.27	0.17
At rest, K_o	0.43	0.43	0.29
Passive, K_p	3.7	3.7	5.8

- If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the wall support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design.
- Seismic loading will result in increased lateral earth pressures acting on the walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure. According to the CHBDC, the site-specific zonal acceleration ratio for Cornwall is 0.2. Based on experience, for the subsurface conditions at this time, no significant amplification of the ground motion is expected. The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio $A = 0.2$.



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- In accordance with Sections 4.6.4 and C.4.6.4 of the CHBDC and its Commentary, for structures which do not allow lateral yielding, the horizontal seismic coefficient, k_h , used in the calculation of the seismic active pressure coefficient, is taken as 1.5 times the zonal acceleration ratio (i.e., $k_h = 0.30$). For structures which allow lateral yielding, k_h is taken as 0.5 times the zonal acceleration ratio (i.e., $k_h = 0.10$).
- The following seismic active pressure coefficients (K_{AE}) for the two backfill cases (Case (a) and Case (b)) may be used in design. These seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat, and include the effect of wall friction. Where sloping backfill is present above the top of the wall, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Granular A	Granular B Type II	Cellular Concrete
Yielding wall	0.30	0.30	0.21
Non-yielding wall	0.50	0.50	0.34

- The above K_{AE} values for yielding walls are applicable provided that the wall can move up to 250A (mm) where A is the design zonal acceleration ratio of 0.20. This corresponds to displacements of up to 50 mm at this site.
- For the case of a yielding wall, the earthquake induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e. an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

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

where $\sigma_h(d)$ = the lateral earth pressure at depth d (kPa);

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

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

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

- taken as soil unit weights given above for fill materials;

- taken as 21 kN/m^3 for residual soil and 20 kN/m^3 for the existing fill material, where encountered;

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

H = the total height of the wall above the toe (m).



- For non-yielding walls, there will be insufficient movement to mobilize the shear strength of the soils and allow development of minimum active or maximum passive earth pressures.

6.9 Other Design Considerations

It has been confirmed by boreholes advanced through the abutment footings that these footings are founded on the native silty sand to sandy silt till. It is likely that the existing footings for the south wingwalls/retaining walls are similarly founded on the native till, although confirmation of the founding conditions in these areas was outside the scope of work. The potential for rebound of the silty clay was assessed to account for the possibility that footings for the south wingwalls/retaining walls are founded on the silty clay as there will be an approximately one month delay between the end of excavation and demolition of the structure and completion of the backfill placement. Negligible rebound is anticipated since the silty clay crust in the south wingwall/retaining wall area is overconsolidated.

6.10 Construction Considerations

6.10.1 Excavation and Temporary Cut Slopes

Excavations for construction of new foundations and/or for removal of existing foundations will extend primarily through the existing embankment fill materials and terminate in the native silty clay (where present) or in the silty sand to sandy silt till. All excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Ontario Occupational Health and Safety Act and Regulations for Construction Projects. The fill materials at this site would be classified as Type 3 soils as would any cohesionless materials below the groundwater level. The native clayey materials, properly dewatered cohesionless materials and glacial till would be classified as Type 2 soils. Temporary open-cut slopes within the fill materials and native soils should be maintained no steeper than 1H:1V, provided that appropriate groundwater control is in place in the glacial till deposit if excavation will extend into the till (see Section 6.10.2).

In addition to the existing concrete footings, a concrete layer was encountered in Borehole 11-05. This layer may be associated either with temporary works for the original construction or buried utilities in the vicinity of those boreholes. Consideration may be given to including an NSSP in the Contract Documents to warn the Contractor of the presence of a potential obstruction in this area.

6.10.2 Groundwater Control

If adopted, excavations for new spread footings supported directly on the glacial till deposit would extend below the groundwater level which has been interpreted to be at approximately Elevation 60.5 m. Alternatively, excavations to expose the top of the existing abutment footings would extend to approximately Elevation 60.2 m to 60.4 m which is also slightly below the groundwater level at the site. In addition, groundwater may be 'perched' within the granular fill soils that overlie the less permeable silty clay or glacial till deposits. The volume of perched water and the groundwater level will fluctuate based on prevailing weather conditions.



For new footings on the till deposit, groundwater control will be required to minimize disturbance of the subgrade soils. The silty sand to sandy silt till deposit is water-bearing but has a relatively low permeability; higher flows should be expected in the coarser granular fill layers, although this flow may be limited in duration. Due to the lower permeability nature of the till, it is expected that sumps may not be sufficient to lower the groundwater level to minimize subgrade disturbance; rather, it would likely be necessary to use well points or an eductor system to draw down the water level in this material. If this foundation type is adopted, an appropriate NSSP should be included in the Contract Documents for control of groundwater. An NSSP for groundwater control has been provided in Appendix C.

For new footings constructed on top of the existing footings, the excavation would only need to extend to about Elevation 60.2 m to expose the surface of the footing. It is anticipated that properly constructed and filtered sumps placed around the outside of the existing footings would be sufficient to lower the groundwater level within the excavation to below the surface of the existing concrete footings. Depending on the operations required on top of the existing footings, it may be desirable to include an appropriate NSSP, such as the one supplied in Appendix G, in the Contract Documents to address control of groundwater for this foundation type.

Excavations for the granular pads that support portions of the wingwalls that extend beyond the existing footings will extend below the groundwater level and dewatering will be required as described above for new footings. Alternatively, it may be possible to extend the roadway protection system (which could consist of sheetpiles installed in excavated trenches) to encompass the 2 m to 3 m long granular pad area; in this case, the sheetpiles should provide a sufficient groundwater cut-off to make active dewatering unnecessary, although unwatering of the excavation using properly filtered sumps would still be required to allow placement of the granular fill in dry conditions. As another alternative, consideration could be given to the use of levelling concrete or 10 MPa lean concrete in lieu of the granular material for the pads beyond the existing footings; this option would minimize or eliminate the need for dewatering, provided the excavations are opened up in short sections and the concrete is “tremie-pumped” into the excavations to displace any seepage water. The use of clear stone in lieu of compacted Granular A or lean concrete is not recommended due to the potential for loss/migration of fine soil particles from the water-bearing silty subgrade over time (including the difficulty of properly placing a geotextile filter/separator to prevent migration of such fines) and the resulting potential for settlement of the portion of the wingwalls extending beyond the existing footings.

6.10.3 Subgrade Protection

The native silty sand to sandy silt till soils will be sensitive to disturbance and loosening due to water seepage and/or ponding. Where cast-in-place footings are to be constructed on the till materials, a concrete working slab is recommended to protect the footing subgrade. In accordance with OPSS 902, the cleaned excavation base should be inspected by a Quality Verification Engineer (QVE) experienced in geotechnical engineering prior to placing the concrete working slab. It is recommended that the footing excavation be carried out such that the final 0.5 m of excavation is completed with the geotechnical personnel on site and that the working slab be placed immediately after footing inspection. An appropriate Special Provision or contract item should be included in the Contract Documents to address this requirement if the selected foundation type involves construction of new footings directly on the glacial till subgrade. However, as the selected replacement option



involves placement of new footings on top of the existing footings under a rapid bridge replacement scheme, such an NSSP is not required for this structure site.

Where a granular pad is placed for construction beyond the south limits of the existing retaining wall footings, no special subgrade protection is required provided that proper groundwater control is in place prior to and during excavation and construction of the granular pad and these portions of the retaining wall. Alternatively, as discussed in Section 6.10.2, levelling concrete or lean concrete could be used in place of the granular pad to minimize the requirements for groundwater control associated with excavations beyond the limits of the existing footings.

6.10.4 Temporary Protection Systems

Temporary protection systems will be required to execute the work which will be carried out using staged construction. At a minimum, it is anticipated that temporary protection systems will be required along the Highway 401 median parallel to the highway to allow for excavation to replace first one then the other overpass structure; temporary protection will also be required along the front edge of the existing abutment footings adjacent to Cornwall Centre Road for the heavy-lift construction. It is not expected that temporary protection systems will be required along the back of the abutment footings as there should be sufficient space to complete open-cut excavations as part of the staged construction.

The temporary protection systems are to be designed and constructed by the Contractor in accordance with OPSS 539. The lateral movement of the protection system(s) along the Highway 401 median should meet Performance Level 2. For protection systems along Cornwall Centre Road, the deformation criteria may need to be more stringent than Performance Level 2 if rapid bridge replacement is adopted with heavy-lift equipment to prevent movement of the roadway underneath the heavy wheel loading.

Conceptually, temporary protection systems could consist of soldier piles and lagging where the H-piles would be driven to a suitable depth and horizontal lagging installed as the excavation proceeds. The H-piles should be installed within pre-augered holes in order to avoid damage or other installation problems due to the presence of very dense till with cobbles and boulders near the ground surface. Driven steel sheet piling would be difficult to install due to the presence of cobbles and boulders within the silty sand to sandy silt till. Support to the systems could be in the form of struts and walers in the case of footing excavations or rakers and anchors in the case of roadway protection. The raker/anchor support must be designed to accommodate the loads applied from pressures and surcharge pressures from area line or point loads as well as the impact of sloping ground behind the system.

For preliminary assessment of costs for protection systems that are to be designed by the Contractor, a conceptual design may be completed using the following parameters:

Soil Type	Earth Pressure Coefficient			Angle of Internal Friction (degrees)	Unit Weight (Above Groundwater) (kN/m ³)
	Active, K_a	At Rest, K_o	Passive, K_p		
Embankment fill (granular)	0.33	0.50	3.0	30	20



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Soil Type	Earth Pressure Coefficient			Angle of Internal Friction (degrees)	Unit Weight (Above Groundwater) (kN/m ³)
	Active, K _a	At Rest, K _o	Passive, K _p		
Silty clay	0.38	0.55	2.7	27	19
Silty sand to sandy silt till	0.31	0.47	3.3	32	22

The earth pressure coefficients noted above are based on a horizontal surface adjacent to the excavation. If sloped surfaces are present, the coefficients should be adjusted accordingly.

7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Dirka U. Prout, P.Eng. and reviewed by Ms. Lisa Coyne, P.Eng., a senior geotechnical engineer and Principal with Golder. Mr. Fintan Heffernan, P.Eng., the Designated MTO Foundations Contact for Golder, conducted an independent review of this report.

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TABLE 1 – COMPARISON OF FOUNDATION ALTERNATIVES FOR REPLACEMENT OF EASTBOUND OVERPASS STRUCTURE

Foundation Option	Advantages	Disadvantages	Estimated Cost	Risks/Consequences
New cast-in-place or precast spread footings supported on existing spread footings or re-use of existing spread footings	<ul style="list-style-type: none"> • Least expensive foundation option • Avoids demolition and removal of existing foundations allowing reduced excavation support and groundwater control requirements • No potential for subgrade disturbance during placement of new footings • Compatible with rapid bridge replacement (RBR) techniques • Facilitates shorter construction time compared to casting new footings 	<ul style="list-style-type: none"> • Structural design must counteract sliding of new footings on existing foundations • Future increases in design loading limited to capacity of existing footings (although this is not considered to be a significant limitation) • Incompatible with integral abutment design 	<ul style="list-style-type: none"> • Estimated cost of approximately \$25,000 per abutment for anticipated footing size • Less expensive than deep foundation options 	<ul style="list-style-type: none"> • Low risk of settlement; existing overpass structures have performed satisfactorily • Low to moderate risk of variation in elevation of top surface of existing footing, though can be addressed through use of concrete levelling layer on top of footing
Spread footings supported on compact to very dense native silty sand to sandy silt till	<ul style="list-style-type: none"> • Conventional excavation and construction • Can design footings to accommodate future increases in structure loading and to accommodate specific configuration of new abutments and associated wingwalls and retaining walls • Also compatible with RBR techniques but longer construction schedule would be required to accommodate demolition of existing footings and construction of new footings 	<ul style="list-style-type: none"> • Would require demolition and removal of existing footings unless new footings are located behind existing with a longer bridge span length (which would increase structure costs) • Deeper temporary protection and longer/more significant groundwater control requirements than for re-use of existing footings; some potential for subgrade disturbance in fine-grained till soils • Results in increased time for construction compared to re-using existing foundations due to excavation, demolition/ removal, forming, reinforcing and casting stages • Incompatible with full integral abutment design 	<ul style="list-style-type: none"> • Estimated cost \$37,500 per abutment • Less expensive than deep foundation options 	<ul style="list-style-type: none"> • Low risk of settlement; existing overpass structures have performed satisfactorily



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Foundation Option	Advantages	Disadvantages	Estimated Cost	Risks/ Consequences
End bearing steel H-piles pre-augered and/or driven into very dense silty sand to sandy silt till or to bedrock	<ul style="list-style-type: none"> • High bearing resistance • Negligible settlement • Allows for conventional, integral or semi-integral abutments 	<ul style="list-style-type: none"> • More costly than shallow foundations • Would require at least partial demolition and removal of existing footings or coring through existing footings to allow H-piles to extend to required founding level or installation of deep foundations behind the existing footings (necessitating a longer bridge deck) • Groundwater control and excavation protection systems required for partial or full demolition of existing footings • Hard driving conditions expected due to the presence of cobbles and boulders; may not be possible to drive piles to required depth • Pre-augering would likely be required, contributing to increased costs; temporary liner recommended for pre-augering to minimize disturbance in water-bearing silty sand to sandy silt till; potential for cobbles or boulders to impact auger holes 	<ul style="list-style-type: none"> • Estimated cost per pile of \$270/metre plus cost of concrete pile cap 	<ul style="list-style-type: none"> • Very low risk of settlement • Moderate to high risk of encountering obstructions during pile installation (whether by driving or pre-augering) • Moderate risk of pile damage during driving; driving shoes required



FOUNDATION REPORT - CORNWALL CENTRE ROAD EASTBOUND OVERPASS

Foundation Option	Advantages	Disadvantages	Estimated Cost	Risks/ Consequences
End-bearing concrete-filled steel tube piles pre-augered and/or driven into very dense silty sand to sandy silt till	<ul style="list-style-type: none"> • High bearing resistance • Negligible settlement 	<ul style="list-style-type: none"> • More costly than shallow foundations • Would require at least partial demolition and removal of existing footings or coring through existing footings to allow steel tube piles to extend to required founding level or installation of deep foundations behind the existing footings (necessitating a longer bridge deck) • Groundwater control and excavation protection systems required for partial or full demolition of existing footings • Hard driving conditions expected due to the presence of cobbles and boulders; potentially more difficult to install than H-piles due to larger effective end area • Pre-augering would likely be required contributing to increased costs; temporary liner recommended for pre-augering to minimize disturbance in water-bearing silty sand to sandy silt till; cobbles/boulders could also impact auger holes • Incompatible with full integral abutment design 	<ul style="list-style-type: none"> • Estimated cost per pile of \$275/metre, plus cost of concrete pile cap 	<ul style="list-style-type: none"> • Very low risk of settlement • Moderate to high risk of encountering obstructions during pile installation (whether by driving or pre-augering) – risk slightly higher than for H-piles in the case of driven piles • Moderate risk of pile damage during driving; driving shoes required



FOUNDATION REPORT - CORNWALL CENTRE ROAD EASTBOUND OVERPASS

Foundation Option	Advantages	Disadvantages	Estimated Cost	Risks/Consequences
Concrete augered piles (caissons) founded in silty sand to sandy silt till	<ul style="list-style-type: none"> • High bearing resistance • Negligible settlement 	<ul style="list-style-type: none"> • More costly than shallow foundations • Would require at least partial demolition and removal of existing footings or coring through existing footings to allow H-piles to extend to required founding level or installation of deep foundations behind the existing footings (necessitating a longer bridge deck) • Groundwater control and excavation protection systems required for partial or full demolition of existing footings • Temporary liners required during caisson construction to minimize disturbance in water-bearing silty sand to sandy silt till; also potential for disturbance of soils at base of casing due to groundwater pressures, although this may be controlled by used of drilling mud • Augers may be obstructed by cobbles and boulders • Incompatible with full integral abutment design 	<ul style="list-style-type: none"> • Estimated cost per caisson of \$375/metre 	<ul style="list-style-type: none"> • Very low risk of settlement • Moderate risk of ground disturbance along sides or at base during caisson construction, although this can be accommodated through use of temporary liner and drilling mud • Moderate to high risk of encountering obstructions, although these may be removed more easily in larger diameter caisson holes than in driven pile installations (including smaller diameter pre-augering for driven pile installations)

- NOTES:
1. Costs are very preliminary estimates and are intended to provide a comparison between alternatives rather than actual construction costs.
 2. Table to be read in conjunction with accompanying report.

Prepared By: DP
Checked By: LC



FOUNDATION REPORT - CORNWALL CENTRE ROAD EASTBOUND OVERPASS

TABLE 2 – COMPARISON OF WALL TYPES AND FOUNDATION OPTIONS FOR REPLACEMENT OF WINGWALLS/RETAINING WALLS

Wall Type	Advantages	Disadvantages	Estimated Cost	Risks/ Consequences
Retained soil system (RSS) walls	<ul style="list-style-type: none">• Can be constructed on existing wall footings or new shallow footings• Rapid construction; compatible with Rapid Bridge Replacement (RBR) scenario• Backfilling may be accelerated by using cellular concrete or stabilized fill in lieu of compacted granular backfill. For this site, given lift thickness requirements and curing time for cellular concrete, it is estimated that compacted granular fill would still be faster; however, cellular concrete could reduce construction traffic and equipment congestion in backfill area as it can be placed from a more distant location• Lowest cost alternative• Flexible structure type with good performance under seismic loading	<ul style="list-style-type: none">• Higher material costs and longer placement time if cellular concrete used as backfill• Highway infrastructure restricted in reinforced zone	<ul style="list-style-type: none">• \$650 per m²	<ul style="list-style-type: none">• Low overall risk• Low risk of settlement based on subsurface conditions and negligible change in embankment loading; if RSS wall constructed on new footings outside of existing footprint, risk of minor differential settlement at transition point beyond existing footings



FOUNDATION REPORT - CORNWALL CENTRE ROAD EASTBOUND OVERPASS

Wall Type	Advantages	Disadvantages	Estimated Cost	Risks/ Consequences
Conventional concrete cantilever or counterfort walls	<ul style="list-style-type: none"> Well established design and construction methodologies Could use precast walls constructed on the existing footings for the RBR option; with this alternative labour costs and construction time are reduced Also, cast-in-place walls could be constructed on the existing footings Reduced backfill requirements compared to RSS wall; could also use cellular concrete or stabilized fill in lieu of compacted granular backfill to attempt to reduce congestion and placement time 	<ul style="list-style-type: none"> Highest cost option if cast-in-place walls on new footings constructed due to labour and excavation costs Relatively long construction time compared to most wall alternatives Dewatering required For a precast option, size required would be more than most plants could handle in terms of weight and very narrow sections would be needed due to handling restrictions; also, would have limited room for a crane in the working area Increased susceptibility to damage due to seismic loading compared to RSS, anchored or soil nail structures 	<ul style="list-style-type: none"> \$725 per m² 	<ul style="list-style-type: none"> Low to moderate overall risk
Soldier pile and concrete panel walls with soil anchors or tie-backs	<ul style="list-style-type: none"> Minimal excavation required if used in top-down construction, though not applicable for this site where existing median walls must be removed Negligible backfill required Minimal to no dewatering required Can allow for more rapid construction compared to cast-in-place gravity wall Permanent wall without facings can serve as temporary wall Good performance under seismic loading 	<ul style="list-style-type: none"> Most expensive option Pre-augering required to install soldier piles due to presence of dense to very dense glacial till containing cobbles and boulders near surface Vertical tolerance of soldier piles may be difficult to maintain due to hard ground conditions Lateral restraint in the form of soil anchors or tie-backs required due to height of wall Longer construction time compared with RSS or precast concrete walls Specialized equipment and skilled labour required Highway infrastructure restricted in tie-back/anchor locations 	<ul style="list-style-type: none"> \$1,500 per m² 	<ul style="list-style-type: none"> Moderate Construction costs and time may escalate if cobbles and boulders are found to be extensive at the soldier pile locations



FOUNDATION REPORT - CORNWALL CENTRE ROAD EASTBOUND OVERPASS

Wall Type	Advantages	Disadvantages	Estimated Cost	Risks/ Consequences
Anchored sheet pile walls with soil anchors or tie-backs	<ul style="list-style-type: none"> Minimal excavation required if used in top-down construction, though not applicable at this site where existing walls must be removed Negligible backfill required Minimal to no dewatering required Can allow for more rapid construction compared to cast-in-place gravity wall Permanent wall without facings can serve as temporary wall Good performance under seismic loading 	<ul style="list-style-type: none"> Sheet pile walls will be very difficult to install due to hard ground conditions in the glacial till Lateral restraint in the form of soil anchors required due to height of wall Longer construction time compared with RSS or precast concrete walls Specialized equipment and skilled labour required Highway infrastructure restricted in tie-back/anchor locations 	<ul style="list-style-type: none"> \$855 per m² 	<ul style="list-style-type: none"> High overall risk Construction costs and time may escalate if cobbles and boulders are found to be extensive at the soldier pile locations
Soil nail walls with concrete panels	<ul style="list-style-type: none"> Minimal excavation required if used in top-down construction, though not applicable for this site where existing walls must be removed Negligible backfill required Can allow for more rapid construction compared to cast-in-place gravity wall Permanent wall without facings can serve as temporary wall Procedure adaptable to any wall alignment Good performance under seismic loading No excavation required below footing level 	<ul style="list-style-type: none"> Special expensive measures such as casing may be required to maintain open hole in granular embankment fill Non-standard application – commonly used for construction of cut walls only; experienced contractor required Soil nails cannot be installed below the groundwater level without permanent dewatering Specialized equipment and skilled labour required Highway infrastructure restricted in soil nailed zone More robust design may be required due to silt content of existing backfill which may be more susceptible to frost action when saturated and will exert increased pressures on the wall 	<ul style="list-style-type: none"> Not evaluated as not considered suitable for this site 	<ul style="list-style-type: none"> Not evaluated as not considered suitable for this site

NOTES: 1. Costs are very preliminary estimates and are intended to provide a comparison between alternatives rather than actual construction costs.

2. Table to be read in conjunction with accompanying report.

Prepared By: DP

Checked By: LC

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index (Relative Density)	N Blows/300 mm or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

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.) split spoon sampler for a distance of 300 mm (12 in.)

Consistency

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

(b) Cohesive Soils

Dynamic Cone Penetration Resistance; N_d :

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

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

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

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

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
F	factor of safety
V	volume
W	weight

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	liquid limit
w_p	plastic limit
I_p	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_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation pressure
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

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

- Notes:**
- 1 $\tau = c' + \sigma' \tan \phi'$
 - 2 shear strength = (compressive strength)/2
 - * density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING 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 texture and structure are preserved.

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

<u>Description</u>	<u>Spacing</u>
Very wide	> 3 m
Wide	1 – 3 m
Moderately close	0.3 – 1 m
Close	50 – 300 mm
Very close	< 50 mm

GRAIN SIZE

<u>Term</u>	<u>Size*</u>
Very Coarse Grained	> 60 mm
Coarse Grained	2 – 60 mm
Medium Grained	60 microns – 2 mm
Fine Grained	2 – 60 microns
Very Fine Grained	< 2 microns

Note: *Grains >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 varies 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 (W.R.T.) 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 abbreviated 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

Abbreviations

B – Bedding	P - Polished
FO - Foliation Schistosity	S - Slickensided
CL - Cleavage	SM - Smooth
SH - Shear Plane Zone	R - Ridged / Rough
VN - Vein	ST - Stepped
F - Fault	PL - Planar
CO - Contact	FL - Flexured
J - Joint	UE - Uneven
FR - Fracture	W - Wavy
M F - Mechanical Fracture	C - Curved
- Parallel To	
⊥ - Perpendicular To	

RECORD OF BOREHOLE No 11-01

1 OF 1

METRIC

PROJECT 10-1121-0259-2000

W.P. 4029-08-00

LOCATION N 4991125.0 ; E 203306.8

ORIGINATED BY RA

DIST HWY 401

BOREHOLE TYPE POWER AUGER, HOLLOW STEM

COMPILED BY WDF

DATUM GEODETIC

DATE June 7, 2011

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)						
								20	40	60	80	100							
61.7	GROUND SURFACE																		
0.0	TOPSOIL, clayey silt																		
61.3	Black																		
0.4	FILL, sandy silt with clayey silt, trace topsoil																		
60.9	Brown																		
0.8	FILL, clayey silt, some sand, trace to some gravel		1	SS	7										○				
60.3	Firm Brown																		
1.4	FILL, sand and gravel, trace clay, some silt		2	SS	10										○				50 26 20 4
59.4	Compact Brown																		
2.3	SILTY SAND TO SANDY SILT (TILL), some clay, some gravel, with cobbles		3	SS	45														
	Dense to very dense																		
	Grey		4	SS	103														
															○				10 36 42 12
			5	SS	101														
			6	SS	102														
56.6	COBBLES AND BOULDERS																		
5.1																			
56.2	SILTY SAND TO SANDY SILT (TILL), some clay, some gravel, with cobbles																		
5.5	Very dense																		
55.8	Grey		7	SS	50/0mm														
5.9	END OF BOREHOLE																		
	Split-spoon and auger refusal at elev. 55.8m.																		
	Groundwater encountered at about elev. 60.3m during drilling on June 7, 2011.																		

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

RECORD OF BOREHOLE No 11-02

1 OF 1

METRIC

PROJECT 10-1121-0259-2000

W.P. 4029-08-00

LOCATION N 4991134.5 ; E 203330.7

ORIGINATED BY RA

DIST HWY 401

BOREHOLE TYPE POWER AUGER, HOLLOW STEM / ROTARY DRILLING, NQ/NW CASING

COMPILED BY WDF

DATUM GEODETIC

DATE June 8, 2011

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)									
								20	40	60	80	100		20	40	60		GR	SA	SI	CL	
61.9	GROUND SURFACE																					
0.1	ASPHALT																					
61.3	FILL, silty sand and gravel Brown																					
0.6	FILL, clayey silt, some sand, trace gravel, trace asphalt Stiff Brown and Grey		1	SS	11		61							○								
60.4																						
1.5	SILTY CLAY, trace sand, trace topsoil Stiff Brown		2	SS	9		60							○								
59.6																						
2.3	SILTY SAND TO SANDY SILT (TILL), some clay, some gravel Dense to very dense Brown to Grey at about elev. 58.9m.		3	SS	33		59							○								
58.6																						
3.4	COBBLES AND BOULDERS		4	SS	115									○								
58.2																						
3.7	SILTY SAND TO SANDY SILT (TILL), trace to some clay, trace to some gravel, sand seams below elev. 56.6m. Compact to very dense Grey		5	SS	27		58															
			6	SS	61		57							○					9	35	44	12
			7	SS	139																	
			8	SS	43		56							○								
55.0																						
6.9	LIMESTONE BEDROCK, fresh, thinly laminated Grey		9	NQ RC	-		55															
							54	95	75	85												
			10	NQ RC	-		53	T.C.R. (%)	S.C.R. (%)	R.Q.D. (%)												
								99	95	95												
51.9							52															
10.0	END OF BOREHOLE																					
	Groundwater encountered at about elev. 60.4m during drilling on June 8, 2011.																					

RECORD OF BOREHOLE No 11-03

1 OF 1

METRIC

PROJECT 10-1121-0259-2000
W.P. 4029-08-00 LOCATION N 4991122.7 ; E 203336.6 ORIGINATED BY RA
DIST HWY 401 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY WDF
DATUM GEODETIC DATE June 7, 2011 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)			GR	SA	SI	CL
								○ UNCONFINED	+	FIELD VANE	● QUICK TRIAXIAL	×					LAB VANE						
61.6	GROUND SURFACE						20	40	60	80	100												
0.0	TOPSOIL, clayey silt Black																						
61.1																							
0.5	SILTY CLAY, with fine sand lenses Firm Brown																						
60.6																							
1.0	SILTY SAND, trace gravel Loose Brown		1	SS	8																		
60.2																							
1.4	<i>Hydro carbon odour and staining</i>																						
59.6	SILTY SAND TO SANDY SILT (TILL), trace clay, some gravel Very dense Brown		2	SS	93																		
2.0	END OF BOREHOLE																						
	Groundwater encountered at about elev. 60.6m during drilling on June 7, 2011.																						

RECORD OF BOREHOLE No 11-03A

1 OF 1

METRIC

PROJECT 10-1121-0259-2000

W.P. 4029-08-00

LOCATION N 4991123.1 ; E 203338.1

ORIGINATED BY RA

DIST HWY 401

BOREHOLE TYPE POWER AUGER, HOLLOW STEM

COMPILED BY WDF

DATUM GEODETIC

DATE June 7, 2011

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE							
61.5	GROUND SURFACE														
0.0	TOPSOIL, clayey silt Black														
61.0															
0.5	SILTY CLAY, with fine sand lenses Stiff Brown		1	SS	9										
60.1															
1.4	SILTY SAND TO SANDY SILT (TILL), trace clay, some gravel, with cobbles Dense to very dense Brown to Grey at about elev. 58.5m		2	SS	73										
			3	SS	72										
			4	SS	34										
			5	SS	31										
			6	SS	49										
			7	SS	47										
			8	SS	100/ 125mm										
55.1															
6.4	LIMESTONE BEDROCK, fragments Grey														
54.7															
6.8	END OF BOREHOLE		9	SS	100/ 25mm										
	Split-spoon and auger refusal at elev. 54.7m.														
	Borehole dry during drilling on June 7, 2011.														
	Water level measured at elev. 60.0m on June 16, 2011.														
	Water level measured at elev. 60.1m on June 27, 2011.														
	Water level measured at elev. 60.1m on Nov. 1, 2011.														
	Water level measured at elev. 60.5m on Mar. 7, 2012.														

RECORD OF BOREHOLE No 11-04

1 OF 1

METRIC

PROJECT 10-1121-0259-2000

W.P. 4029-08-00

LOCATION N 4991130.8 ; E 203345.8

ORIGINATED BY RA

DIST HWY 401

BOREHOLE TYPE POWER AUGER, HOLLOW STEM / ROTARY DRILLING, NQ/NW CASING

COMPILED BY WDF

DATUM GEODETIC

DATE June 25, 2011

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									
61.9	GROUND SURFACE						20	40	60	80	100						
0.0	TOPSOIL, silty clay																
61.6	Black																
0.3	FILL, sand and gravel, some silt																
61.1	Brown																
0.8	FILL, clayey silt, trace sand and gravel, trace topsoil		1	SS	7												
60.7	Firm																
1.2	Brown																
60.2	SILTY CLAY, trace topsoil		2	SS	29												
1.7	Firm																
	Brown																
	SILTY SAND TO SANDY SILT (TILL), some clay, trace to some gravel, with cobbles and boulders		3	SS	95												
	Compact to very dense																
	Brown to Grey at about elev. 59.3m																
	Cobbles and boulders from about elev. 59.2 to 58.9m		4	SS	29											9 42 37 12	
			5	SS	109												
			6	SS	62											12 35 41 12	

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

RECORD OF BOREHOLE No 11-05

1 OF 1

METRIC

PROJECT 10-1121-0259-2000

W.P. 4029-08-00

LOCATION N 4991149.4 ; E 203357.7

ORIGINATED BY RA

DIST HWY 401

BOREHOLE TYPE POWER AUGER, HOLLOW STEM / ROTARY DRILLING, NQ/NW CASING

COMPILED BY WDF

DATUM GEODETIC

DATE June 8, 2011 - June 13, 2011

CHECKED BY


SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)	
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE									
61.7	GROUND SURFACE						20 40 60 80 100											
0.1	ASPHALT																	
	FILL, sand and gravel, some silt, asphalt fragments Compact Brown		1	SS	19							○						
60.3																		
1.4	FILL, clayey silt, some sand and gravel, trace concrete fragments Very stiff Brown		2	SS	16							○	┌──┐			21 15 39 25		
59.2													○					
2.6	CONCRETE		3	SS	23							○						
	SILTY SAND TO SANDY SILT (TILL), some clay, trace to some gravel Compact to very dense Brown becoming grey below about elev. 58.7m		4	SS	10							○				10 40 38 12		
			5	SS	13													
			6	SS	19													
			7	SS	40							○				11 39 39 11		
			8	SS	109													
54.8			9	SS	69/													
6.9	LIMESTONE BEDROCK, weathered, thinly laminated		10	NQ RC	50mm		30		21		0							
54.5	Dark grey, non-intact				-													
7.2	LIMESTONE BEDROCK, fresh, thinly laminated, vertical fractures from about elev. 52.7m to 52.6m, 25mm silt infill at about elev. 52.3m Grey		11	NQ RC	-		TCR (%) 92		SCR (%) 87		RQD (%) 86							
			12	NQ RC	-													
							103		96		80							
51.5																		
10.2	END OF BOREHOLE																	
	Auger and split-spoon refusal at about elev. 54.8m on bedrock.																	
	Groundwater encountered at about elev. 59.9m during drilling on June 8, 2011.																	

RECORD OF BOREHOLE No 11-07

2 OF 2

METRIC

PROJECT 10-1121-0259-2000
W.P. 4029-08-00 LOCATION N 4991144.6 ; E 203378.5 ORIGINATED BY RA
DIST HWY 401 BOREHOLE TYPE POWER AUGER, HOLLOW STEM / ROTARY DRILLING, NQ/NW CASING COMPILED BY WDF
DATUM GEODETIC DATE June 15, 2011 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE											
							20	40	60	80	100	20	40	60	GR	SA	SI	CL	
52.0	LIMESTONE BEDROCK, weathered, thinly laminated, non-intact Grey LIMESTONE BEDROCK, fresh, thinly laminated Grey						52	66	7	0									
15.4			12	NQ RC	-		51	99	97	92									
			13	NQ RC	-		50	113	85	85									
49.7	END OF BOREHOLE																		
17.7	Auger refusal at about elev. 52.9m on bedrock. Groundwater encountered at about elev. 61.3m during drilling on June 15, 2011.																		

RECORD OF BOREHOLE No 11-28

1 OF 1

METRIC

PROJECT 10-1121-0259-2000
W.P. 4029-08-00 LOCATION N 4991144.5 ; E 203348.3 ORIGINATED BY PH
DIST HWY 401 BOREHOLE TYPE HYDROVAC / ROTARY DRILLING, HQ CORE, POWER AUGER, HOLLOW STEM COMPILED BY WDF
DATUM GEODETIC DATE November 2, 2011 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× LAB VANE	20						40	60	80
61.9	GROUND SURFACE																			
0.0	ASPHALT																			
0.2	FILL, crushed gravel Grey																			
	FILL, silty sand and gravel, trace asphalt fragments Brown																			
60.3							61													
1.6	CONCRETE, footing			HQ RC			60													
59.0																				
2.8	SILTY SAND TO SANDY SILT (TILL), some gravel, trace clay Compact Grey		1	HQ RC			59									UC				
			2	SS	30								○			18 40 33 9				
			3	SS	28		58													
57.6																				
4.3	END OF BOREHOLE																			
	Groundwater not established November 2, 2011.																			

RECORD OF BOREHOLE No 11-29

1 OF 1

METRIC

PROJECT 10-1121-0259-2000 W.P. 4029-08-00 LOCATION N 4991136.2 ; E 203333.3 ORIGINATED BY PH
DIST HWY 401 BOREHOLE TYPE HYDROVAC / ROTARY DRILLING, HQ CORE, POWER AUGER, HOLLOW STEM COMPILED BY WDF
DATUM GEODETIC DATE November 2, 2011 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× LAB VANE	WATER CONTENT (%)					
61.9	GROUND SURFACE						20	40	60	80	100						
0.0	ASPHALT																
0.2	FILL, crushed gravel Grey																
	FILL, silty sand and gravel Brown																
60.2																	
1.7	CONCRETE, footing																
	50mm diam rebar at elev. 59.6m				HQ RC												
59.2																	
2.7	SILTY SAND TO SANDY SILT (TILL), some gravel, trace clay Very dense Grey				HQ RC												
			1	SS	164								○				
58.1			2	SS	168/ 150mm												
3.8	END OF BOREHOLE																
	Groundwater not established November 2, 2011.																

RECORD OF BOREHOLE No 11-30

1 OF 1

METRIC

PROJECT 10-1121-0259-2000
W.P. 4029-08-00 LOCATION N 4991144.1 ; E 203365.7 ORIGINATED BY PH
DIST HWY 401 BOREHOLE TYPE HYDROVAC / ROTARY DRILLING, HQ CORE, POWER AUGER, HOLLOW STEM COMPILED BY WDF
DATUM GEODETIC DATE November 3, 2011 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
								20 40 60 80 100						
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
								20 40 60 80 100						
61.7	GROUND SURFACE													
0.4	ASPHALT													
0.2	FILL, crushed gravel Grey													
	FILL, sand and gravel to silty sand and gravel Brown													
60.2														
1.5	CONCRETE, footing			HQ RC										
59.3														
2.4	SILTY SAND TO SANDY SILT (TILL), some gravel, trace clay Very dense Brown to grey at elev. 58.5m		1	SS	52									
			2	SS	75									
58.1														
3.7	END OF BOREHOLE													
	Groundwater encountered at about elev. 60.9m during drilling on November 3, 2011.													

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

RECORD OF BOREHOLE No 11-31

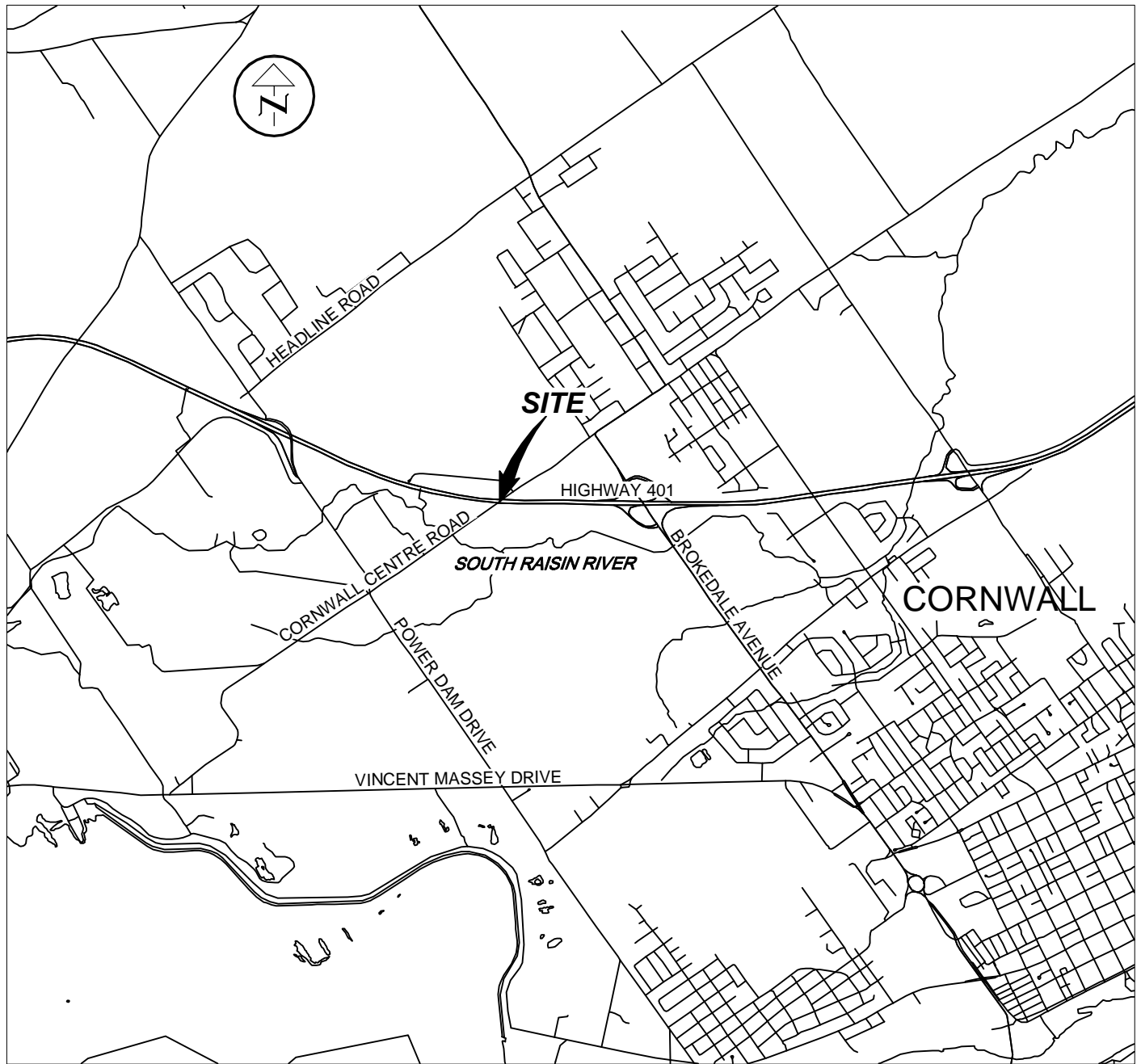
1 OF 1

METRIC

PROJECT 10-1121-0259-2000
W.P. 4029-08-00 LOCATION N 4991134.7 ; E 203349.5 ORIGINATED BY PH
DIST HWY 401 BOREHOLE TYPE HYDROVAC / ROTARY DRILLING, HQ CORE, POWER AUGER, HOLLOW STEM COMPILED BY WDF
DATUM GEODETIC DATE November 3, 2011 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)		
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× LAB VANE								
61.7	GROUND SURFACE																		
0.0	ASPHALT																		
0.2	FILL, crushed gravel Grey																		
	FILL, sand and gravel to silty sand and gravel Brown																		
60.4																			
1.3	CONCRETE, footing																		
59.5	Honeycombed in bottom 50-75mm																		
2.2	SILTY SAND TO SANDY SILT (TILL), some gravel, trace clay Very dense Brown to grey at elev. 58.7m																		

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



SCALE IN METRES
0 1000 2000
1:50000

REFERENCE

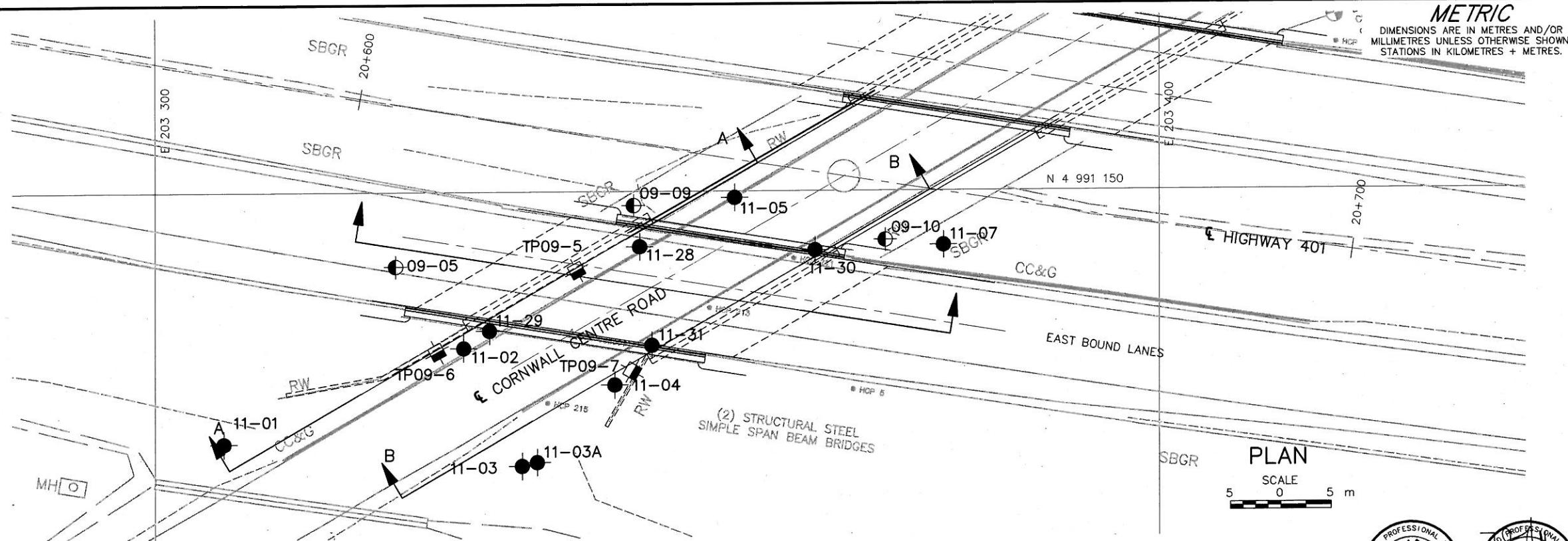
DRAWING BASED ON CANMAP STREETFILES V2005.4.

NOTE

THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING TEXT.

PROJECT		EASTBOUND OVERPASS CORNWALL CENTRE ROAD OVERPASS (SITE 31-209) HIGHWAY 401 GWP 4029-08-00	
TITLE		KEY PLAN	
PROJECT No.		10-1121-0259	FILE No. 1011210259-2000-F02001
CADD		LMK	Aug. 22/11
CHECK			
SCALE		AS SHOWN	REV.
		FIGURE 1	





CONT No. 4029-08-00
 WP No. 4029-08-00

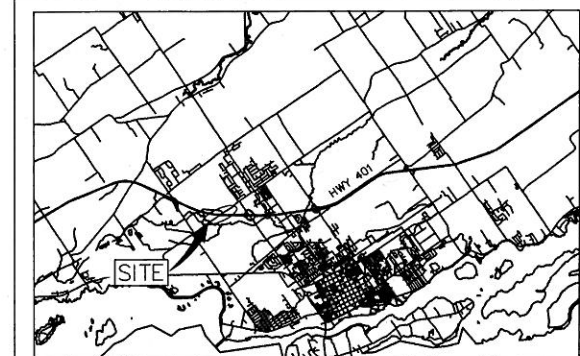


CORNWALL CENTRE OVERPASS EBL
 HIGHWAY 401
 BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



Golder Associates Ltd.
 LONDON, ONTARIO, CANADA



KEY PLAN

LEGEND

- Borehole - Current Investigation
- Borehole - (By others Geocres No. 31G-232)
- Test pit - (By others Geocres No. 31G-232)
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES (MTM ZONE 8)	
		NORTHING	EASTING
11-01	61.7	4991125.0	203306.8
11-02	61.9	4991134.5	203330.7
11-03	61.6	4991122.7	203336.6
11-03A	61.5	4991123.1	203338.1
11-04	61.9	4991130.8	203345.8
11-05	61.7	4991149.4	203357.7
11-07	67.4	4991144.6	203378.5
11-28	61.9	4991144.5	203348.3
11-29	61.9	4991136.2	203333.3
11-30	61.7	4991144.1	203365.7
11-31	61.7	4991134.7	203349.5
(Geocres No. 31G-232)			
09-05	68.5	4991142.6	203323.9
09-09	67.8	4991148.6	203347.7
09-10	67.7	4991145.1	203372.7

NOTES

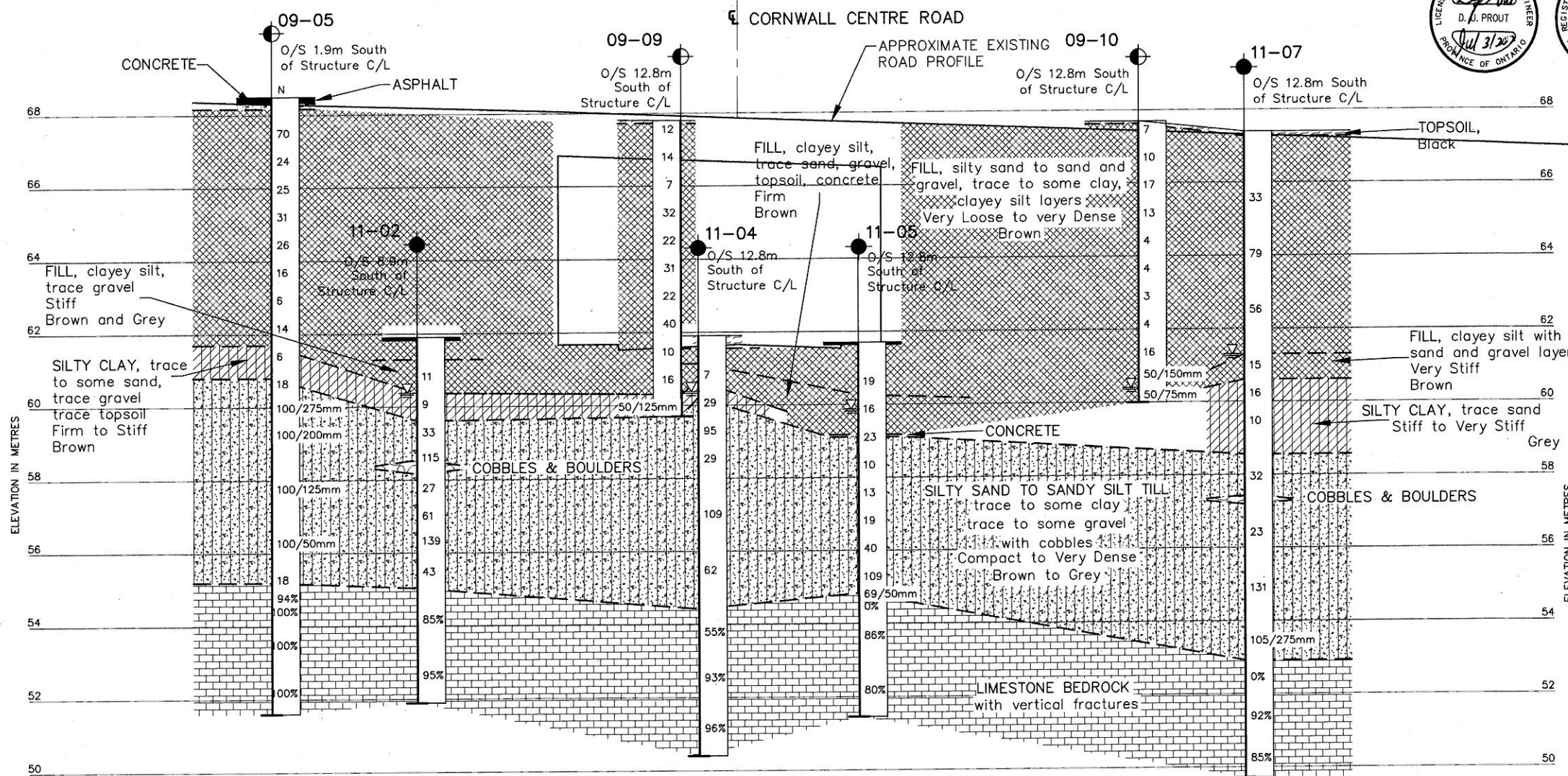
This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

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

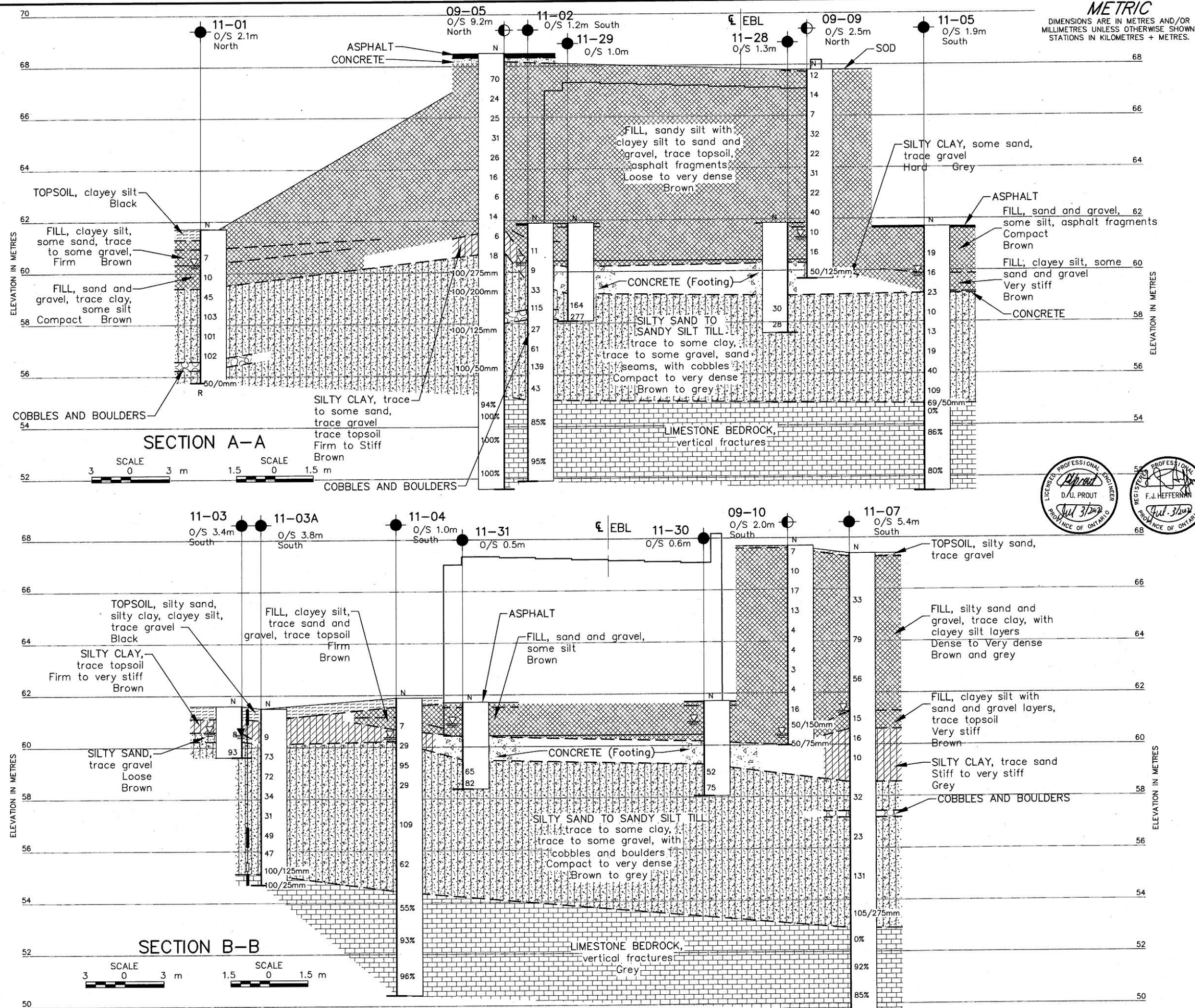
REFERENCE

Base plans provided in digital format by McCormick Rankin Corporation. Drawing file H3211037XB01 rec'd July 10, 2011.

NO.	DATE	BY	REVISION
1			
Geocres No. 31G-242			
HWY.	401	PROJECT NO.	10-1121-0259
SUBM'D.	DUP	CHKD.	DUP
DRAWN:	WDF	CHKD.	FJH
		DATE:	Mar. 20/12
		APPD.	
		DWG.	1
		SITE:	31-209



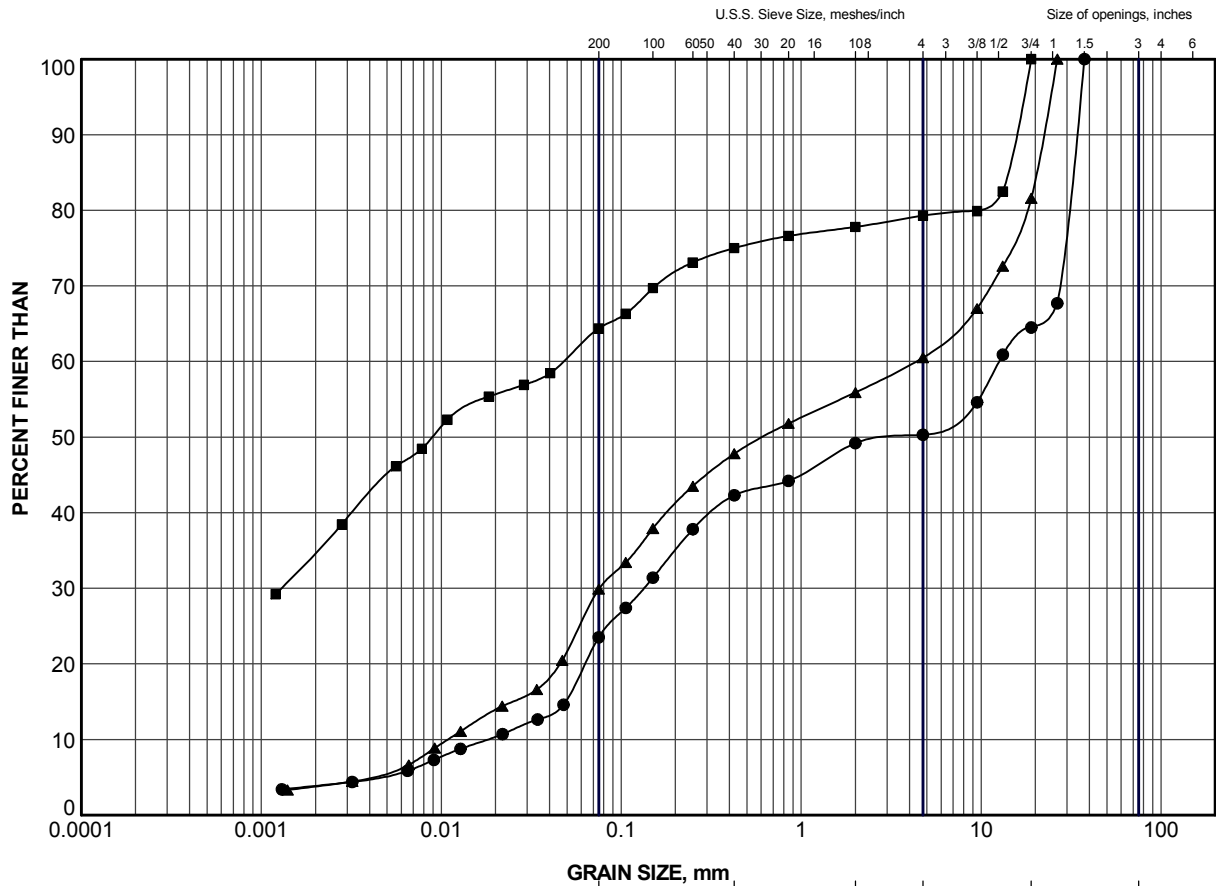
PROFILE ALONG CENTRELINE OF STRUCTURE EBL





APPENDIX A

Laboratory Test Data - Soils



CLAY AND SILT	GRAVEL SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND

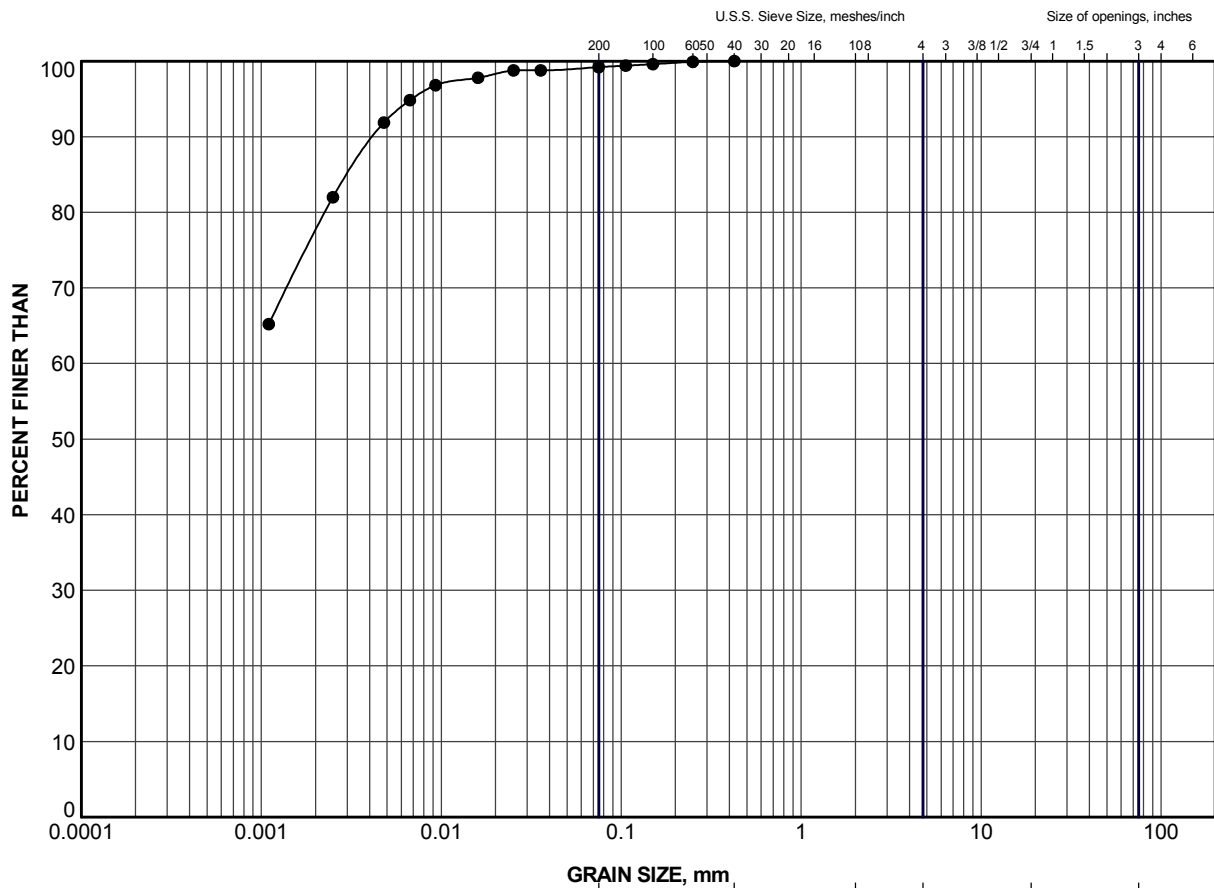
SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	11-01	2	59.9
■	11-05	2	59.9
▲	11-07	3	62.5

PROJECT EASTBOUND OVERPASS
CORNWALL CENTRE ROAD OVERPASS (SITE 31-209)
HIGHWAY 401
GWP 4029-08-00

GRAIN SIZE DISTRIBUTION FILL




PROJECT No.10-1121-0259-2000			FILE No. 1011210259-2000-F020A1		
DRAWN	LMK	Aug 09/11	SCALE	N/A	REV.
CHECK			FIGURE A-1		

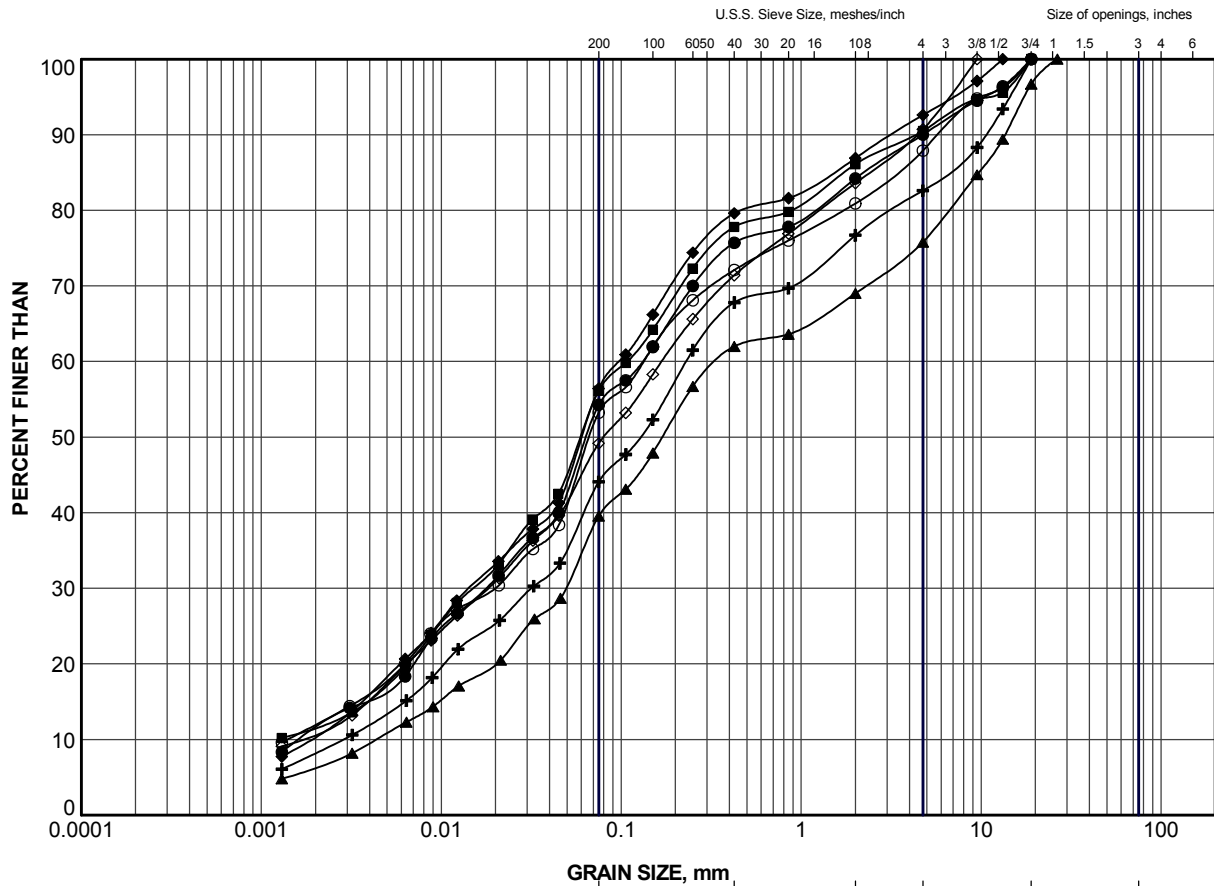


GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	11-07	6	59.5

PROJECT		EASTBOUND OVERPASS CORNWALL CENTRE ROAD OVERPASS (SITE 31-209) HIGHWAY 401 GWP 4029-08-00		
TITLE		GRAIN SIZE DISTRIBUTION SILTY CLAY		
PROJECT No:10-1121-0259-2000		FILE No. 1011210259-2000-F020A2		
DRAWN	LMK	Aug 09/11	SCALE	N/A
CHECK			REV.	
 Golder Associates LONDON, ONTARIO		FIGURE A-2		



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	11-01	5	57.6
■	11-02	6	57.0
▲	11-02	8	55.5
+	11-03A	4	58.2
◆	11-03A	7	55.9
◇	11-04	4	58.6
○	11-04	6	55.5

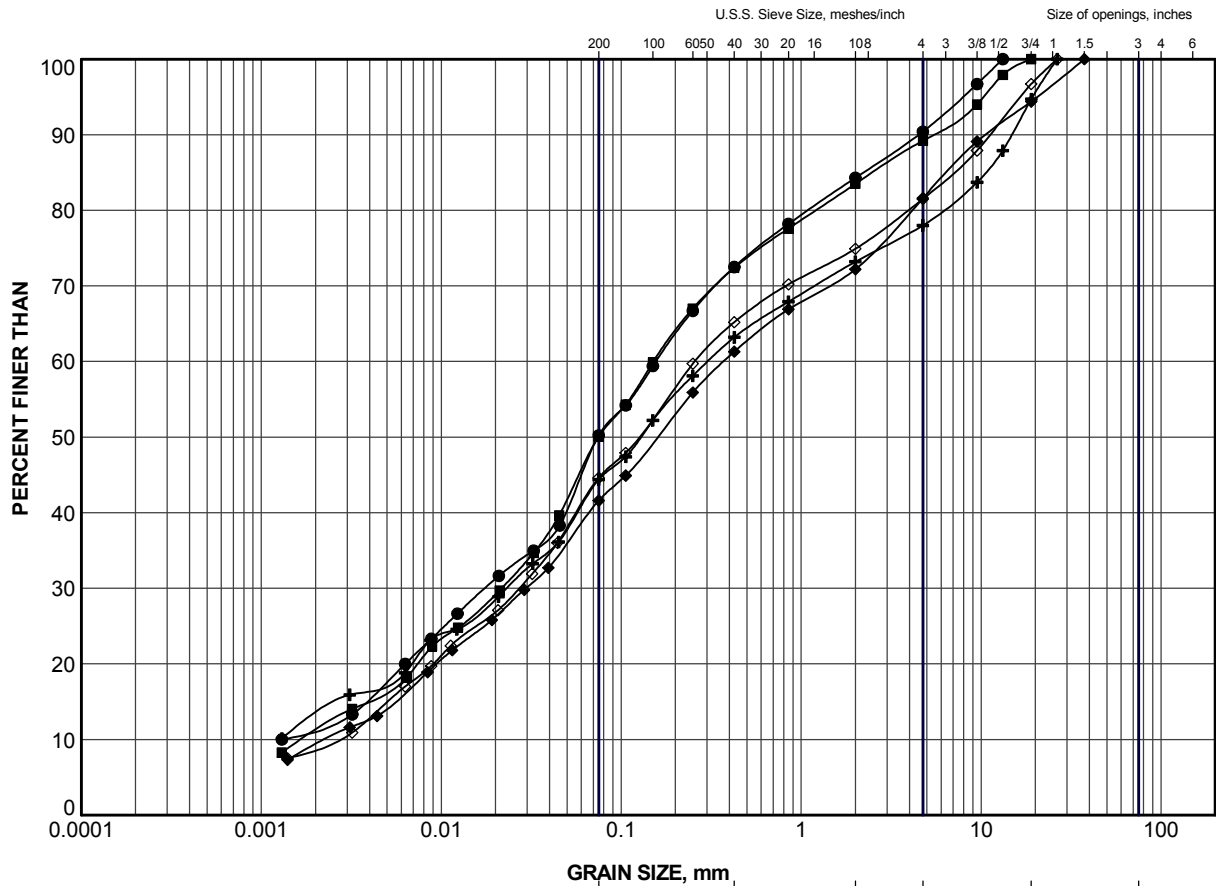
PROJECT EASTBOUND OVERPASS
CORNWALL CENTRE ROAD OVERPASS (SITE 31-209)
HIGHWAY 401
GWP 4029-08-00

TITLE
GRAIN SIZE DISTRIBUTION
SILTY SAND TO SANDY SILT (TILL)



PROJECT No.10-1121-0259-2000	FILE No. 1011210259-2000-F020A3
DRAWN LMK	Aug 09/11
CHECK	
SCALE N/A	REV.


FIGURE A-3



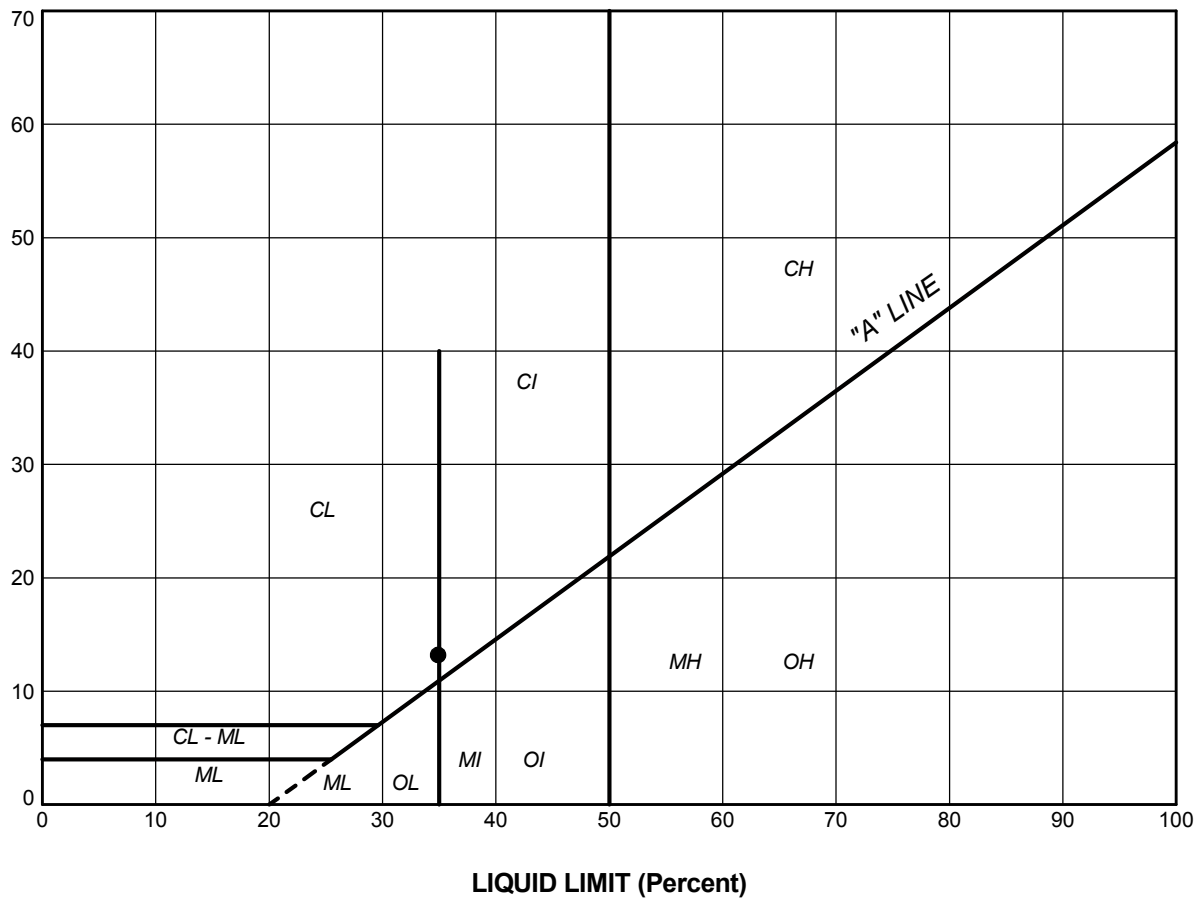
CLAY AND SILT	GRAVEL SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	11-05	4	58.4
■	11-05	7	56.1
+	11-07	8	56.4
◆	11-28	2	58.5
◇	11-31	1	59.1

PROJECT		EASTBOUND OVERPASS CORNWALL CENTRE ROAD OVERPASS (SITE 31-209) HIGHWAY 401 GWP 4029-08-00		
TITLE		GRAIN SIZE DISTRIBUTION SILTY SAND TO SANDY SILT (TILL)		
PROJECT No.10-1121-0259-2000		FILE No. 1011210259-2000-F020A4		
DRAWN	LMK	Mar 20/12	SCALE	N/A
CHECK			REV.	
 Golder Associates LONDON, ONTARIO		FIGURE A-4		

PLASTICITY INDEX (Percent)




SOIL TYPE
 C = Clay
 M = Silt
 O = Organic

PLASTICITY
 L = Low
 I = Intermediate
 H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	FILL 11-05	2	34.9	21.7	13.2

PROJECT		EASTBOUND OVERPASS CORNWALL CENTRE ROAD OVERPASS (SITE 31-209) HIGHWAY 401 GWP 4029-08-00		
TITLE		PLASTICITY CHART		
PROJECT No:10-1121-0259-2000		FILE No. 1011210259-2000-F020A5		
DRAWN	LMK	Aug 09/11	SCALE	N/A
CHECK			REV.	
 Golder Associates LONDON, ONTARIO		FIGURE A-5		



APPENDIX B

Laboratory Test Data - Rock

TABLE B1

SUMMARY OF POINT LOAD TESTING

Eastbound Overpass
 Cornwall Centre Road Overpass (Site 31-209E)
 Highway 401
GWP 4029-08-00

POINT LOAD TEST ON ROCK SAMPLES

Borehole Number	Sample Number	Sample Depth (m)	Test Type	Core Length (mm)	Core ⁽²⁾ Diameter (mm)	Equivalent Diameter (mm)	Ram Pressure (kPa)	Load (P) (kN)	Is Axial (MPa)	Is Diametral (MPa)	Is (50mm) (MPa)	Approx. ⁽¹⁾ UCS (MPa)
11-2	9	7.16-7.32	D	41.96	45.23	-	1,620.00	1.54	-	0.751	0.718	15
11-4	7	7.53-7.62	D	42.10	40.39	-	8,480.00	8.04	-	4.928	4.477	94
11-5	11	7.35-7.50	D	42.57	41.10	-	13,220.00	12.53	-	7.419	6.793	143
11-7	13	16.46-16.98	D	39.21	39.19	-	12,840.00	12.17	-	7.925	7.103	149

⁽¹⁾ $Is_{50} \times C$ (actual value will have to be confirmed by UCS testing), from ISRM ("Suggested Methods for Determining Point Load Strength", International Society for Rock Mechanics Commission on Testing Methods, Int. J. Rock. Mech. Min. Sci. and Geomechanical Abstr., Vol 22, No. 2 1985, pp. 51-60.

⁽²⁾ Actual distance between point load cones at time of failure.

Prepared By: DUP
 Checked By: LC

NOTE: 1. Table to be read in conjunction with accompanying report.

TABLE B2

SUMMARY OF UNCONFINED COMPRESSION TESTING

Eastbound Overpass
Cornwall Centre Overpass (Site 31-209E)
Highway 401
GWP 4029-08-00

BOREHOLE	SAMPLE NUMBER	DEPTH INTERVAL (m)	SAMPLE HEIGHT (mm)	SAMPLE DIAMETER (mm)	WATER CONTENT (%)	UNIT WEIGHT (kN/m³)	VOID RATIO	UNCONFINED COMPRESSIVE STRENGTH (MPa)
11-7	12	15.32 – 15.51	108.0	47.6	0.08	26.12	0.01	47.5

- NOTES:
1. Detailed test reports shown on Figures B1A and B1B.
 2. Table to be read in conjunction with accompanying report.

Prepared By: DUP
Checked By: LC

SAMPLE IDENTIFICATION

PROJECT NUMBER	10-1121-0259	SAMPLE NUMBER	12
BOREHOLE NUMBER	11-7	SAMPLE DEPTH, m	15.32-15.51

TEST CONDITIONS

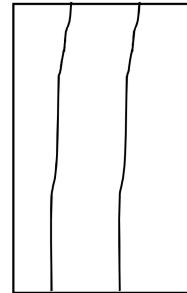
MACHINE SPEED, mm/min	0.00	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.27

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.80	WATER CONTENT, (specimen) %	0.08
SAMPLE DIAMETER, cm	4.76	UNIT WEIGHT, kN/m ³	26.12
SAMPLE AREA, cm ²	17.80	DRY UNIT WT., kN/m ³	26.10
SAMPLE VOLUME, cm ³	192.19	SPECIFIC GRAVITY, assumed	2.70
WET WEIGHT, g	512.15	VOID RATIO	0.01
DRY WEIGHT, g	511.74		

VISUAL INSPECTION

FAILURE SKETCH

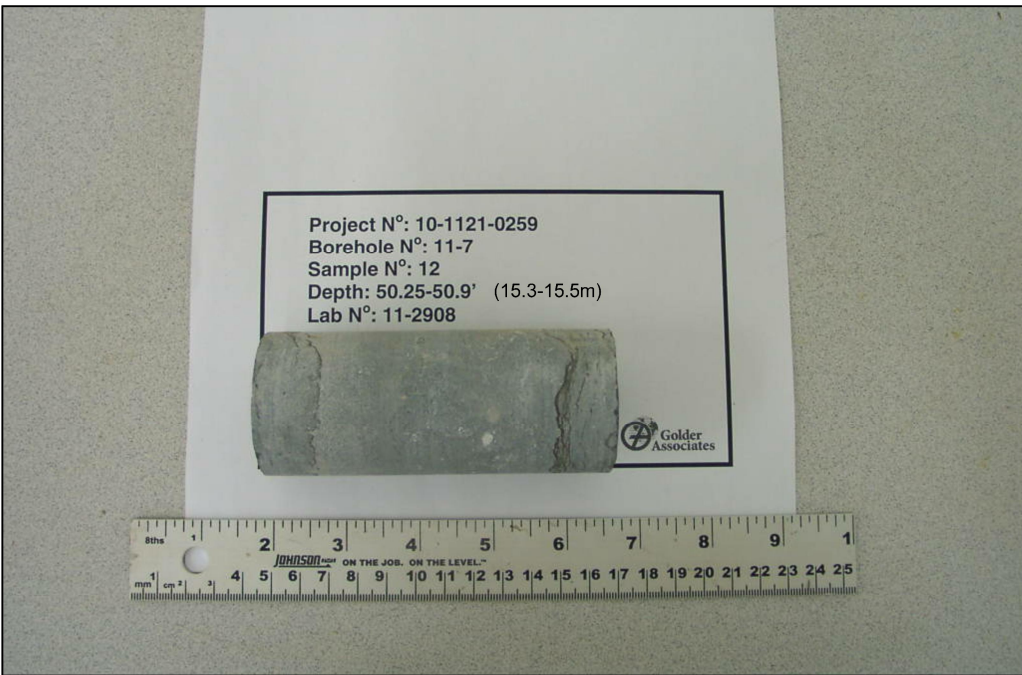


TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	47.5
----------------------	---	-------------------------	------

REMARKS: L/D Ratio not in accordance with ASTM Standard


PROJECT	EASTBOUND OVERPASS CORNWALL CENTRE ROAD OVERPASS (SITE 31-209) HIGHWAY 401 GWP 4029-08-00				
TITLE	UNCONFINED COMPRESSION TEST ASTM D7012-07				
 Golder Associates LONDON, ONTARIO	PROJECT No.	10-1121-0259	FILE No.	1011210259-2000-F02B1A	
	CADD	WDF	Oct. 03/11	SCALE	AS SHOWN
	CHECK			REV.	0
				FIGURE B1A	



BEFORE COMPRESSION



AFTER COMPRESSION

PROJECT	EASTBOUND OVERPASS CORNWALL CENTRE ROAD OVERPASS (SITE 31-209) HIGHWAY 401 GWP 4029-08-00				
	TITLE UNCONFINED COMPRESSION TEST ASTM D7012-07				
 Golder Associates LONDON, ONTARIO	PROJECT No.		10-1121-0259	FILE No.	1011210259-2000-F02B1B
	CADD	WDF	Oct. 03/11	SCALE	AS SHOWN
	CHECK			REV.	0
FIGURE B1B					

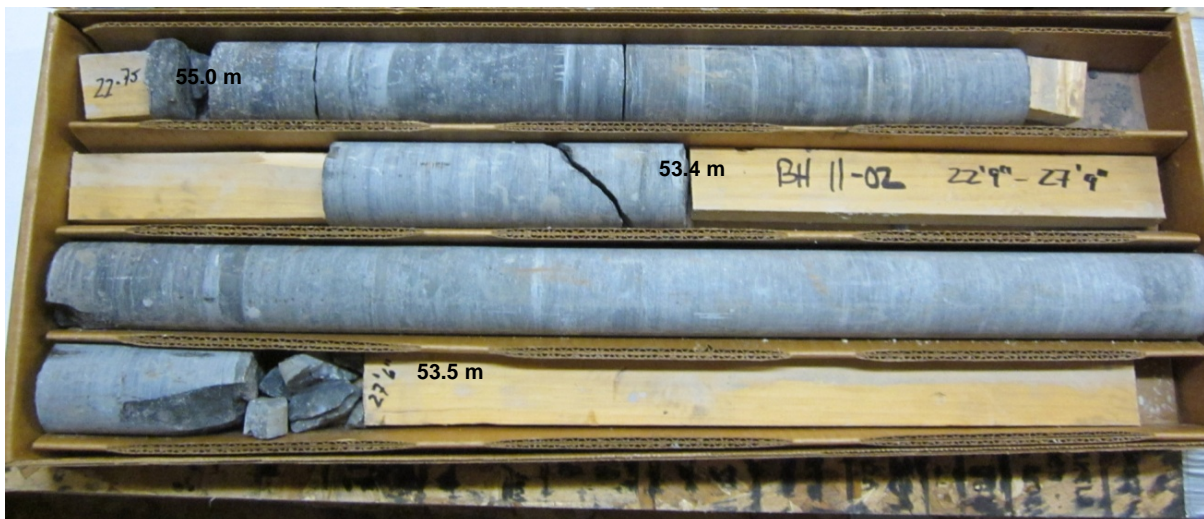


APPENDIX C

Photographs of Rock Core



APPENDIX C ROCK CORE PHOTOGRAPHS



Photograph 1: BH 11-02 - Rock Core Box 1: Elevation 55.0 to 53.5 metres.



Photograph 2: BH 11-02 - Rock Core Box 2: Elevation 53.5 to 51.9 metres.



APPENDIX C ROCK CORE PHOTOGRAPHS



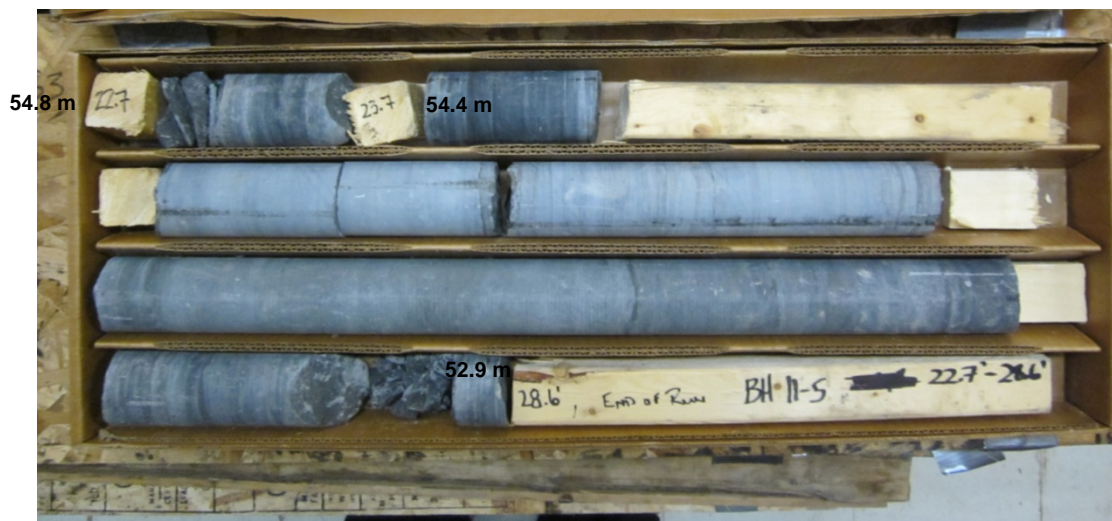
Photograph 3: BH 11-04 - Rock Core Box 1: Elevation 54.4 to 52.1 metres.



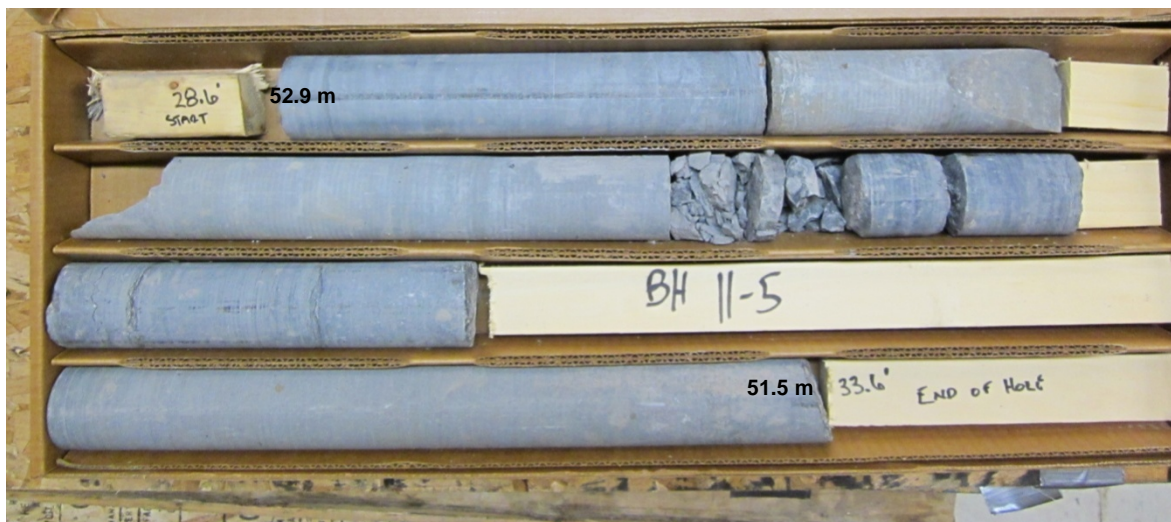
Photograph 4: BH 11-04 - Rock Core Box 2: Elevation 52.1 to 50.5 metres.



APPENDIX C ROCK CORE PHOTOGRAPHS



Photograph 5: BH 11-05 - Rock Core Box 1: Elevation 54.8 to 52.9 metres.



Photograph 6: BH 11-05 Rock Core Box 2: Elevation 52.9 to 51.5 metres.



APPENDIX C ROCK CORE PHOTOGRAPHS



Photograph 7: BH 11-07 - Rock Core Box 1: Elevation 52.9 to 50.8 metres.



Photograph 8: BH 11-07 - Rock Core Box 2: Elevation 50.5 to 49.8 metres.

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APPENDIX D

Laboratory Test Data - Concrete

Golder Associates Ltd.
32 Steacie Drive
Kanata, Ontario
K2K 2A9



CONCRETE CORE COMPRESSIVE STRENGTH REPORT

Project: Hwy 401 - Cornwall - Centre Road Structures

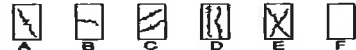
Project No.: 10-1121-0259

Date: November 15, 2011

Client: McCormick Rankin Corporation

Borehole No.	Depth (m)	Date Cast	Date Tested	Age (days)	Diameter (mm)	Density (kg/m ³)	Type of Fracture	Compressive Strength (MPa)
BH 11-25	0.86-1.00 Below T.O.F.	N/A	15-Nov-11	N/A	62.9	2325	E	26.6
BH 11-28	2.50-2.64	N/A	15-Nov-11	N/A	60.5	2299	E	19.0

Type of Fracture Codes :



REMARKS : - Compressive Strength Corrected for L/D Ratio.
- Cores tested in moist condition.

CERTIFIED CONCRETE TESTING LABORATORY - CSA Standard A283
TESTING WAS CARRIED OUT IN GENERAL ACCORDANCE WITH CSA A23.2-14C

SIGNED:

A handwritten signature in blue ink, appearing to read "C.N. Mangione".
C.N. Mangione P.Eng.



APPENDIX E

**Records of Previous Boreholes and Test Pits
(GEOCRES No. 31G-232)**

RECORD OF BOREHOLE No 09-05

1 OF 2

METRIC

G.W.P. 15-64-19 LOCATION 4 991 142.6 N 203 323.9 E ORIGINATED BY SLL
 HWY 401 BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Rods COMPILED BY SLL
 DATUM Geodetic DATE 2009-05-13 - 2009-05-14 CHECKED BY DEE

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
70.2	Corrected Ground Surface elev. 68.5m MTO Geomatics Section												
0.0	ASPHALT: (175mm)												
0.2	CONCRETE: (325 mm)												
69.7													
0.5	SAND & GRAVEL, trace to some silt Very Dense to Compact Brown Moist (FILL)		1	SS	70								37 45 18 (SI+CL)
			2	SS	24								
			3	SS	25								
	Dense		4	SS	31								44 42 13 (SI+CL)
			5	SS	26								38 42 19 (SI+CL)
			6	SS	16								
	Loose		7	SS	6								37 48 15 (SI+CL)
			8	SS	14								
63.4													
6.8	Silty CLAY, some sand, trace gravel, Loose Brown Moist		9	SS	6								2 36 42 20
62.5													
7.7	SAND & SILT, some clay, trace to some gravel Compact to Very Dense Grey Moist (TILL)		10	SS	18								7 45 39 9
			11	SS	100/ 275								
			12	SS	100/ 200								15 42 42 (SI+CL)

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

METRIC

[illegible]



+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 09-09

1 OF 1

METRIC

G.W.P. 15-64-19 LOCATION 4 991 148.6 N 203 347.7 E ORIGINATED BY GA
 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY MFA
 DATUM Geodetic DATE 2009-05-19 - 2009-05-19 CHECKED BY DEE

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
69.5	Corrected Ground Surface elev. 67.8m MTO Geomatics Section														
0.0	SOD: 75 mm														
0.2	SAND & GRAVEL, trace to some silt Compact Brown Moist (FILL)		1	SS	12		69								23 63 14 (SI+CL)
			2	SS	14										
			3	SS	7										
			4	SS	32										
			5	SS	22										
			6	SS	31										
			7	SS	22										
			8	SS	40										
			9	SS	10										
			10	SS	16										
62.0															
7.5	Silty CLAY, some sand, trace gravel Hard Grey Moist					62									
61.4		11	SS	50/ 125										5 37 37 21	
8.1	END OF BOREHOLE AT 8.1m Open borehole groundwater level at 8.1m Open borehole groundwater level at 6.5m depth upon completion. Borehole backfilled with holeplug to ground surface.														

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+³, ×³: Numbers refer to
Sensitivity

20
15
10

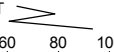
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 09-10


1 OF 1

METRIC

G.W.P. 15-64-19 LOCATION 4 991 145.1 N 203 372.7 E ORIGINATED BY GA
 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY MFA
 DATUM Geodetic DATE 2009-05-20 - 2009-05-20 CHECKED BY DEE

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES										PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															
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STRATIGRAPHY		Corrected Ground Surface elev. 61.78m MTO Geomatics Section				Photo
DEPTH	ELEV.	Sample Depth (m)	Soils Class.	C _{pen} (kPa)	Water Content (%)	
- 0	ASPHALT: (50mm)	63.53				
-	SAND, some gravel, grey, moist, (19mm CRL): (FILL)					
-	CLAY, silty, topsoil stained, dark brown, moist (FILL)					
- 0.4						
-						
- 0.6	SAND, some gravel with occasional cobbles, brown, wet (FILL)					
-						
- 0.8						
-						
- 1.0		62.53				
-						
- 1.2	CLAY, silty, with occasional cobbles, brown to grey, moist to wet (FILL)					
-						
- 1.4						
-						
- 1.6	TOP OF CONCRETE FOOTING					
-	END OF TEST PIT AT 1.68m (Elev. 61.85 m).					
-						
- 1.8	There was no inclination noted on the 1.2m level along the retaining wall.					
-	Asphalt debris encountered at footing level.					
- 2.0	No obvious cracks observed at footing/wall interface.	61.53				

THURBER ENGINEERING LTD.

Location: Cornwall Centre Road, Cornwall, Ontario

Date: May 5, 2009

Excavation Co: Bob Buitting Co. Ltd

Client: MTO


Weather: Cloudy

Inspector: SLL

Project: 15-64-19

Method: Backhoe

**LOG OF TEST PIT: NO. TP 09-5
JOB NO.: 15-64-19**

STRATIGRAPHY		Corrected Ground Surface elev. 61.83m MTO Geomatics Section				
DEPTH	ELEV.	Sample Depth (m)	Soils Class.	C _{pen} (kPa)	Water Content (%)	Photo
- 0	ASPHALT: (50mm)	63.58				
-	SAND, some gravel, grey, moist, (19mm CRL): (FILL)					
-	CLAY, silty, topsoil stained, dark brown, moist (FILL)					
- 0.4				150 kPa		
-						
- 0.6	SAND, some gravel with occasional cobbles, brown, wet (FILL)					
-						
- 0.8						
-						
- 1.0		62.58				
-						
- 1.2	CLAY, silty, with occasional cobbles, brown to grey, moist to wet (FILL)			50 - 75 kPa		
-						
- 1.4						
-						
- 1.6						
-	TOP OF CONCRETE FOOTING					
- 1.8	END OF TEST PIT AT 1.75m (Elev. 61.83 m).					
-	No obvious cracks observed at footing/wall interface.					
- 2.0		61.58				

THURBER ENGINEERING LTD.

Location: Cornwall Centre Road, Cornwall, Ontario

Date: May 5, 2009

Excavation Co: Bob Buitting Co. Ltd

Client: MTO


Weather: Cloudy

Inspector: SLL

Project: 15-64-19

Method: Backhoe

LOG OF TEST PIT: NO. TP 09-6
JOB NO.: 15-64-19

STRATIGRAPHY		Corrected Ground Surface elev. 61.84m MTO Geomatics Section					
DEPTH	ELEV.	Sample Depth (m)	Soils Class.	Cpen (kPa)	Water Content (%)	Photo	
- 0	ORGANICS, trace roots and rootlets: (150mm)	63.59					
-	CLAY, silty, with occasional cobbles and construction debris (re-bar), dark brown to brown, moist (FILL)			50 kPa			
- 0.4							
-							
- 0.6							
-							
- 0.8							
-							
- 1.0		62.59					
-							
- 1.2				75 kPa			
-							
- 1.4							
- 1.6							
- 1.8	TOP OF CONCRETE FOOTING – 1.0 m width						
- 2.0	END OF TEST PIT AT 1.93m (Elev. 61.66 m).	61.59					
	No obvious cracks observed at footing/wall interface.						

THURBER ENGINEERING LTD.

Location: Cornwall Centre Road, Cornwall, Ontario

Date: May 5, 2009

Excavation Co: Bob Buiting Co. Ltd

Client: MTO

Weather: Cloudy

Inspector: SLL

Project: 15-64-19

Method: Backhoe

LOG OF TEST PIT: NO. TP 09-7
JOB NO.: 15-64-19



APPENDIX F

Site Photographs



APPENDIX F PHOTOGRAPHS



Photograph 1: Looking south along Cornwall Centre Road at EBL Highway 401 overpass.



Photograph 2: North elevation, Cornwall Centre Road EBL overpass.



APPENDIX F PHOTOGRAPHS



Photograph 3: South elevation, looking northwest from southeast quadrant (Photo courtesy of McCormick Rankin Corporation).

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APPENDIX G

Non-Standard Special Provisions



APPENDIX G-1 NSSP - OBSTRUCTIONS

OBSTRUCTIONS - Item No.

Non-Standard Special Provision

A layer of concrete 100 mm thick was encountered at approximate elevation 59.2 m in borehole 11-05. Adjacent to the existing structure, there may also be other buried remnants relating to temporary works for the original Cornwall Centre Road Overpass or associated with utilities buried along Cornwall Centre Road. Consideration of the presence of these obstructions must be made in the selection of appropriate equipment and procedures for sub-excavation for spread footings and pre-drilling for deep foundations.

Basis of Payment

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

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DEWATERING, Item No.

Non-Standard Special Provision

SCOPE

The work under this item includes the design, installation, operation, maintenance and removal of temporary dewatering systems to facilitate construction of new foundations on the silty sand to sandy silt till deposit.

New foundations for the overpass abutments and associated wingwalls/retaining walls will require removal of fill and excavation into the native silty sand to sandy silt till below the groundwater level. The cohesionless soils will be subjected to conditions of unbalanced hydrostatic head and can slough, boil and cave in during temporary excavation work in cases where the excavation extends below the groundwater level.

REFERENCES

OPSS 517 Construction Specification for Dewatering of Pipeline, Utility, and Associated Structure Excavation

OPSS 518 Construction Specification for Control of Water from Dewatering Operations

SUBMISSION AND DESIGN REQUIREMENTS

Written details for the proposed dewatering system shall be submitted to the Contract Administrator for information purposes a minimum of ten business days prior to commencing dewatering operations. The Contractor shall reference borehole logs included in the contract documents as a guide in determining requirements.

CONSTRUCTION

Dewatering System

The Contractor is responsible for the design, installation, operation and maintenance of an adequate dewatering system to lower the groundwater level to at least 0.3 m below the footing founding level for abutments and wingwalls/retaining walls or top of subgrade level for granular pads founded on native overburden materials to allow excavation, foundation subgrade preparation and foundation construction in dry conditions.

Water pumped from trenches shall be redirected into the watercourse downstream of the work area in a manner that is not injurious to public health or safety, to property, to the environment or to any part of the work already completed or under construction.

Measures shall be implemented to prevent inundation of the footing excavations by surface water.



APPENDIX G-2

NSSP - DEWATERING

Operation

A continuous dewatering operation shall be provided to facilitate the installation of footings for the abutments, wingwalls/retaining walls or placement of the granular pad at all times during the work. All components of the dewatering system shall be maintained in an effective, functioning and stable condition at all times during the work. Notwithstanding the above, the work shall be completed in accordance with the environmental and operational constraints specified elsewhere in the contract.

Restoration

All equipment and materials placed shall be removed from the right-of-way upon the completion of the work and all areas disturbed as part of this work shall be restored to their pre-construction conditions, unless specified otherwise.

BASIS OF PAYMENT

Payment at the contract price for this Tender Item shall include full compensation for all labour, equipment and material required to do the work.

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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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