



November 15, 2013

FOUNDATION INVESTIGATION AND DESIGN REPORT

**CULVERT AT STATION 13+310 (BC1A)
REALIGNMENT OF HIGHWAY 66 AT VIRGINIATOWN FROM 10.6 KM EAST OF
HIGHWAY 624 EASTERLY 3.4 KM
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5091-07-00**

Submitted to:

McCormick Rankin Corporation
A Member of MMM Group Limited
2655 North Sheridan Way
Mississauga, Ontario
L5K 2P8



REPORT



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PART A

FOUNDATION INVESTIGATION REPORT
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by McCormick Rankin Corporation (MRC), a member of MMM Group Limited (MMM) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the proposed Culvert BC1A crossing the proposed Highway 66 realignment at Station 13+310. The proposed work is part of the overall Highway 66 realignment from 10.6 km east of Highway 624 easterly 3.4 km. The foundation engineering components within the overall project limits include the engineering of: high fill embankments and embankments over swamps; a deep cut section; as well as a number of culverts. The proposed Culvert BC1A is located about 11.3 km east of Highway 624 within the High Fill area H5. The general location of the proposed Culvert BC1A is shown on the Key Plan on Drawing 1.

The Terms of Reference for the foundation investigation are outlined in MTO's Request for Proposal, dated October 2010. Golder's proposal (Scope of Work) for foundation engineering services is contained in Section 6.8 of MRC's Technical Proposal for this assignment. The work has been carried out in accordance with Golder's Supplementary Specialty Plan for foundation engineering services for this project, dated February 25, 2011. The plans showing the proposed horizontal and vertical alignment as well as the General Arrangements (GAs) for the culvert were provided to Golder by MRC.

This report addresses the investigation carried out for the proposed Culvert BC1A only. Separate reports address the foundation investigations for the remaining culverts, swamp crossing/high fill areas and deep cut section.

The purpose of this investigation is to establish the subsurface conditions along the proposed culvert alignment by methods of borehole drilling, rock coring, in situ testing and laboratory testing on selected samples. The centreline of the proposed Highway 66 realignment was staked in the field by MRC and the foundation investigation was carried out at Culvert BC1A as defined in the Terms of Reference. The investigation area is shown in plan on Drawing 1.

2.0 SITE DESCRIPTION

The new Highway 66 alignment is oriented generally in an east-west direction within the Township of McGarry. The proposed culvert will be approximately 43 m long extending across the proposed realigned Highway 66 at about STA 13+310.

In general, the topography in the vicinity of Culvert BC1A consists of a small valley between two rock outcrops and is a moderately populated treed area. The ground surface within the limits of the culvert alignment varies between about Elevation 305 m and Elevation 303 m. A detailed description of the subsurface condition along the culvert alignment is presented in Section 4.0.

3.0 INVESTIGATION PROCEDURES

3.1 Foundation Investigation

The investigation for Culvert BC1A crossing the realigned Highway 66 was carried out on September 6 and 7, 2012, and May 17 and 18, 2013, during which time a total of four boreholes were advanced along or near the proposed culvert alignment. The locations of the boreholes are shown on Drawing 1 and are provided on the Record of Borehole sheets in Appendix A.



Borehole BC1A-1 was drilled using a track-mounted CME-55 drill rig supplied and operated by Landcore Drilling (Landcore) of Sudbury, Ontario and Boreholes BC1A-4 to BC1A-6 were drilled using a track-mounted CME-55 drill rig supplied and operated by George Downing Estate Drilling Ltd. of Grenville sur la Rouge, Quebec. The boreholes were advanced through the overburden using 108 mm inner diameter (I.D.) hollow-stem augers, and/or 'NW' casing with wash boring techniques. In general, soil samples were obtained at intervals of depth of about 0.75 m and 1.5 m, using a 50 mm O.D. split-spoon sampler driven by an automatic hammer, and carried out in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586, Standard Test Method for Standard Penetration Test). Field vane shear tests were carried out in cohesive soils for assessment of undrained shear strengths (ASTM D2573, Standard Test Method for Field Vane Strength Shear Test) using MTO Standard 'N' size vanes. Samples of the bedrock were obtained using an 'NQ' size rock core barrel. All open boreholes were backfilled with bentonite upon completion in accordance with Ontario Regulation 903 Wells (as amended).

The culvert boreholes were advanced to depths ranging between 10.5 m and 14.7 m below existing ground surface including between 3.2 m and 3.8 m of bedrock coring.

In Boreholes BC1A-4 to BC1A-6 the groundwater conditions and water levels in the open boreholes were observed during the drilling operations and are described on the Record of Borehole sheets provided in Appendix A. A piezometer was installed in Borehole BC1A-1 to permit monitoring of the groundwater level at this location. The piezometer consists of a 50 mm diameter PVC pipe with a 1.5 m long slotted screen sealed within the silt and gravel deposits. The borehole annulus surrounding the piezometer screen was backfilled with sand and the remainder of the borehole was backfilled with bentonite. The piezometer was decommissioned in with bentonite in accordance with the regulations.

The fieldwork was observed by members of our engineering and technical staff, who located the boreholes, arranged for the clearance of underground services, observed the drilling, sampling and in situ testing operations, logged the boreholes, and examined and cared for the soil and rock core samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Sudbury geotechnical laboratory where the samples underwent further visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected samples. Strength testing, for uniaxial compression strength (UCS), was carried out on a selected specimen of the rock core. The results of the laboratory testing on samples from the culvert boreholes are included in Appendix B.

Classification of the rock mass quality of the bedrock with respect to the Rock Quality Designation (RQD) is described based on Table 3.10 of the Canadian Foundation Engineering Manual (CFEM, 2006)¹. The degree of weathering of the bedrock samples (i.e. fresh to completely weathered) and the strength classification of the intact rock mass based on field identification (i.e. strong to very strong) are described in accordance with Table B.3 and Table B.6, respectively, of the International Society for Rock Mechanics (ISRM²) standard classification system. Classification of the bedrock core samples with respect to strength is based on Table 3.5 of CFEM (2006).

¹Canadian Geotechnical Society, 2006. Canadian Foundation Engineering Manual, 4th Edition.

² International Society for Rock Mechanics Commission on Test Methods, 1985. Int. J. Rock Mech. Min. Sci. & Geomech. Abstr. Vol 22, No. 2, pp. 51-60.



The proposed centreline of the new highway alignment was staked in the field by MRC prior to drilling. The as-drilled borehole locations, in stations and offsets, were measured in reference to the centreline alignment and were subsequently converted into MTM NAD 83 coordinates in AutoCAD. Borehole elevations were surveyed by a member of our technical staff in reference to the ground surface elevations at temporary benchmarks, which were installed by MRC prior to the commencement of fieldwork. The borehole locations given in the Record of Borehole sheets and shown on Drawing 1 are positioned relative to MTM NAD 83 northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum. The borehole locations, ground surface elevations and drilled depths are as follows:

Borehole	Location (MTM NAD 83)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting		
BC1A-1	5333690.6	409643.0	304.3	10.5
BC1A-4	5333679.6	409643.0	304.0	11.8
BC1A-5	5333670.9	409620.2	303.0	13.5
BC1A-6	5333690.4	409662.8	304.9	14.7

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

In the Quaternary Period, the Virginiatown area was encompassed by glacial Lakes Barlow and Ojibway. In areas of more turbulent waters in these lakes, coarse grained sediments of sand and gravel were deposited. In the calmer portions of the glacial lakes fine grained sediments, consisting primarily of varved clay, were deposited. After Lakes Barlow and Ojibway receded, organic materials were deposited. In the Kirkland Lake area the organic deposits are usually found as fens, bogs and swamps containing varying thicknesses of organics and are often encountered in glaciolacustrine plains (overlying the sand and gravel or clay), along creeks and streams and in bedrock basins (Baker, 1985)³.

Based on NOEGTS⁴ Mapping, the subsoils in the vicinity of the Highway 66 realignment generally consist of till deposited as a ground moraine. A primarily clay/clayey glaciolacustrine deposit is located further than 1 km north of the realignment. The soils along the Highway 66 realignment consist of variable deposits of organic materials, lacustrine sand, silt and clay and till.

Published literature indicates that the site is located in the Abitibi Subprovince of the Superior Province (OGS, 1991)⁵. The Abitibi Subprovince contains rocks of up to 2.75 Ga in age, is about 800 km by 300 km in area and lies within the southern portion of the Superior Province. Bedrock in this subprovince consists primarily of zones of mafic to intermediate metavolcanic rocks and metasedimentary rocks.

³ C.L. Baker, 1985. Quaternary Geology of the Kirkland Lake Area, Districts of Cochrane and Timiskaming; Ontario Geological Survey.

⁴ Northern Ontario Engineering Geology Terrain Study. Ontario Geological Society Map Reference Number 32DSW.

⁵ Ontario Geological Survey, 1991. Geology of Ontario, Special Volume 4, Part 1. Eds P.C. Thurston, H.R. Williams, R.H. Sutcliffe and G.M. Stott, Ministry of Northern Development and Mines, Ontario.



4.2 General Overview of Local Subsurface Conditions

The detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced during this investigation, together with the results of the laboratory tests carried out on selected soil and bedrock core samples, are presented on the attached Record of Borehole sheets and the soil laboratory test sheets provided in Appendices A and B. The results of the in situ field tests (i.e. SPT 'N'-values and undrained shear strengths from the field vanes) as presented on the Record of Borehole sheets and in Section 4 are uncorrected. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling, observations of drilling progress and the results of in situ testing. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole locations.

The inferred soil stratigraphy based on the result of the boreholes is shown in profile on Drawing 1. The orientation (i.e. north, south, east, west) stated in the text of the report is typically referenced to project north and/or up-chainage (along the proposed Highway 66 alignment). For purposes of this report, Highway 66 is oriented east-west.

In general, the subsurface conditions encountered at the site generally consist of topsoil at the ground surface underlain by deposits of loose to compact silty sand to sand and firm to very stiff clayey silt to clay. The cohesive deposit is underlain by deposits of very loose to dense silt, in turn underlain by a deposit of compact to very dense silty sand to gravelly silty sand to gravel, underlain by bedrock.

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

4.2.1 Topsoil

A 75 mm to 300 mm thick deposit of topsoil was encountered from ground surface in all the boreholes, ranging from Elevation 304.9 m to Elevation 303.0 m.

4.2.2 Silty Sand to Sand

A 0.5 m to 0.6 m thick deposit of brown to grey silty sand to sand, some gravel, trace clay, trace organics, was encountered underlying the topsoil in Boreholes BC1A-1, BC1A-4 and BC1A-5. The surface of the deposit was encountered between Elevations 304.2 m and 302.8 m.

The SPT 'N'-values measured within the silty sand to sand deposit range from 6 blows to 13 blows per 0.3 m of penetration, indicating a loose to compact relative density.

The natural water content measured on one sample of the silty sand to sand is about 11 per cent.

4.2.3 Clayey Silt to Clay

A deposit of cohesive soil comprised of brown to grey clayey silt to clay, trace to some sand, trace gravel, was encountered below the silty sand to sand deposit in Boreholes BC1A-1, BC1A-4 and BC1A-5, and below the



topsoil in Borehole BC1A-6. The top of the cohesive deposit was encountered between Elevations 304.8 m and 302.2 m and the deposit is between 2.2 m and 3.5 m thick.

The SPT 'N'-values measured within the clayey silt to clay deposit range from 4 blows to 11 blows per 0.3 m of penetration. In situ field vane tests carried out within the deposit measured undrained shear strengths ranging from about 34 kPa to greater than 100 kPa, and the sensitivity is calculated to be 3 where it was feasible to obtain remoulded undrained shear strengths. The field vane tests results indicate that the silty clay to clay deposit has a firm to very stiff consistency.

The natural water content measured on seven samples of this deposit ranges from about 26 per cent to 50 per cent.

The results of grain size distribution tests completed on three samples of the clayey silt to silty clay portions of the deposit are shown on Figure B1 in Appendix B.

Atterberg limits tests were carried out on six samples of this deposit and measured liquid limits ranging from about 33 per cent to 56 per cent, plastic limits ranging from about 19 per cent to 28 per cent and plasticity indices ranging from about 13 per cent to 29 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B2 in Appendix B and indicate the material is classified as a clayey silt of low plasticity to clay of high plasticity.

4.2.4 Silt

An approximately 1.1 m to 3.9 m thick deposit of grey silt, trace to some clay, trace to some sand, trace gravel, was encountered underlying the clayey silt to clay deposit in Boreholes BC1A-1 and BC1A-4 to BC1A-6. The surface of the silt deposit was encountered between Elevations 301.3 m and 299.2 m. Clay seams were encountered in the silt deposit, between Elevations 301.3 m and 297.4 m in Borehole BC1A-6.

The SPT 'N'-values measured within the silt deposit range from 3 blows to 47 blows per 0.3 m of penetration, indicating a very loose to dense relative density

The natural water content measured on five selected samples of this deposit is about 22 per cent and about 32 per cent.

The results of grain size distribution tests completed on three samples of the silt deposit are shown on Figure B3 in Appendix B. Atterberg limits tests on three samples of the silt deposit indicate that this material is non-plastic.

4.2.5 Silty Sand to Gravel

A deposit of brown to grey, wet silty sand to gravelly silty sand to gravel, trace to some clay was encountered underlying the silt deposit in all boreholes. The surface of the silty sand to gravel deposit was encountered between Elevations 298.7 m and 297.4 m and the deposit is between 1.7 m and 4.9 m thick. Broken rock was encountered between Elevations 297.7 m and 297.0 in Borehole BC1A-1. Boulders were encountered between Elevations 297.8 m and 297.1 m, and between Elevations 296.4 m and 295.9 m in Borehole BC1A-4 and between Elevations 294.3 m and 294.0 m in Borehole BC1A-6. Cobbles were encountered between Elevations 297.4 m and 297.2 m in Borehole BC1A-5. The bottom of the deposit in all boreholes is defined by bedrock.



The SPT 'N'-values measured within the silty sand to gravel deposit range from 13 blows to 80 blows per 0.3 m of penetration, indicating a compact to very dense relative density.

The natural water content measured on three samples of this deposit is about 8 per cent and about 12 per cent.

The results of grain size distribution tests completed on two samples of this deposit are shown on Figure B4 in Appendix B.

4.2.6 Bedrock

Bedrock was encountered in all of the boreholes at depths ranging from 7.3 m to 10.9 m below ground surface, between Elevation 297.0 m and Elevation 293.2 m.

Bedrock was cored in all the boreholes for lengths between 3.2 m and 3.8 m. The retrieved bedrock core is described as fine grained, moderately weathered to fresh, grey to green, metasediment as presented in the Record of Drillhole sheets in Appendix A. Photographs of the retrieved bedrock core samples are shown on Figure B5 in Appendix B.

The Total Core Recovery (TCR) measured on the core samples is 100 per cent. The Solid Core Recovery (SCR) of the rock core samples ranges from 0 per cent to 98 per cent. The Rock Quality Designation (RQD) measured on the core samples ranges from about 21 per cent to 100 per cent, indicating a rock mass of very poor to excellent quality. Typically, the RQD is greater than 44 per cent indicating that the rock is of poor to excellent quality.

Laboratory Uniaxial Compression Strength (UCS) tests were carried out on selected bedrock core samples from Boreholes BC1A-4 to BC1A-6. The UCS values are presented on the Record of Drillhole sheets in Appendix A and are summarized below, and indicate that the bedrock is weak to strong.

Borehole	Elevation (m)	UCS (MPa)
BC1A-4	293.0	25
BC1A-5	292.2	44
BC1A-6	293.6	77

4.3 Groundwater Conditions

Groundwater levels were measured in the open boreholes during and upon completion of drilling and a piezometer was installed in Boreholes BC1A-1, sealed within the silt and gravels deposit to monitor the groundwater level. The measured groundwater levels in the open boreholes and piezometer are presented below.



Borehole	Installation	Time and/or Date	Depth to groundwater (Below ground surface) (m)	Groundwater Elevation (m)
BC1A-1	Open Borehole	September 7, 2012	1.1	303.2
	Piezometer	November 17, 2012	0.6	303.7
	Piezometer	May 17, 2013	0.8	303.5
BC1A-4	Open Borehole	May 17, 2013	1.3	302.7
BC1A-5	Open Borehole	May 18, 2013	1.8	301.2
BC1A-6	Open Borehole	May 18, 2013	1.6	303.3

Groundwater elevations as encountered in the boreholes may not be representative of static groundwater levels since the groundwater levels in the boreholes may not have stabilized on completion of drilling. Furthermore, groundwater elevations will vary depending on seasonal fluctuations, precipitation and local soil permeability.

5.0 CLOSURE

The drilling program was supervised by Mr. Mat Riopelle and Mr. Matt Thibeault, EIT. This report was prepared by Ms. Michelle He and Mr. Matt Thibeault and reviewed by Ms. Sarah Coyne, P.Eng., a senior geotechnical engineer and Associate with Golder. Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project and Principal with Golder, conducted an independent quality control review of the report.



Report Signature Page

GOLDER ASSOCIATES LTD.

Matt Thibeault
Matt Thibeault, EIT

Geotechnical Engineering Intern



Sarah E.M. Coyne, P. Eng.
Senior Geotechnical Engineer, Associate



Jorge M.A. Costa, P. Eng.
Designated MTO Contact, Principal

MH/MT/SEMC/JMAC/kp

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PART B

FOUNDATION DESIGN REPORT

CULVERT AT STATION 13+310 (BC1A)

REALIGNMENT OF HIGHWAY 66 AT VIRGINIATOWN

FROM 10.6 KM EAST OF HIGHWAY 624 EASTERLY 3.4 KM

MINISTRY OF TRANSPORTATION, ONTARIO

GWP 5091-07-00



6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides an interpretation of the geotechnical data obtained during the investigation and recommendations on the foundation aspects of design of the proposed works. The recommendations provided are intended for the guidance of the design engineer. Where comments are made on construction, they are provided to highlight aspects of construction that could affect the design of the project. Those requiring information on aspects of construction must make their own interpretation of the subsurface information provided as it affects their proposed construction methods, costs, equipment selection, scheduling and the like.

6.1 General

Golder was retained by MRC to provide foundation engineering services for the design of the proposed Culvert BC1A, to be constructed across the proposed Highway 66 realignment at Station 13+310. The proposed 43 m long culvert inlet (north side) and outlet (south side) are at Elevation 305.0 m and 303.6 m and the proposed embankment at the culvert area is up to 7.1 m high.

This report presents an assessment of the stability and settlement of the embankment at the culvert location and geotechnical resistances for design of the culvert. It provides recommendations for stable embankment geometry and embankment fill materials, including implementation of mitigation alternatives that may be required as a means to reduce culvert settlements and to improve embankment stability (if necessary). The report also provides recommendations to address potential construction concerns and geotechnical problems associated with culvert and embankment construction, sub-excavating soft/organic materials and placement of new fill materials.

The culvert is located within a high fill area (designated as High Fill H5), as contained in the following report (MTO, 2013):

- *Foundation Investigation and Design Report, Swamp Crossings/High Fill Areas and Deep Cut, Realignment of Highway 66 at Virginiatown from 10.6 km east of Highway 624 easterly 3.4 km, GWP 5091-07-00, by Golder Associates Ltd.*

6.2 Culvert Types

The analyses and recommendations presented herein assume that Culvert BC1A at STA 13+310 will be a circular pipe culvert [i.e. concrete or Corrugated Steel Pipe (CSP)] having a diameter of 1.2 m. However, foundation design recommendations for a concrete box culvert are also provided in the event that an alternative culvert type is considered.

6.3 Culvert Construction Timing

In general, the foundation soils at the culvert crossing will undergo settlement as a result of loading from the new overlying embankment. Therefore, the timing of culvert construction is an essential factor in determining the preferred settlement mitigation option, if required. In areas where relatively small settlements are estimated to occur due to the presence of relatively thin, compressible foundation soils at the culvert location, as is the case at this site, culvert construction can commence concurrently with the proposed new embankment construction so



long as any requirements for maintaining embankment stability are addressed, as discussed in Section 6.4.1. If required, the culvert design could include a camber to mitigate post-construction settlement.

Culverts which are constructed concurrently with the new embankments will experience settlement (both short-term and long-term), as well as lateral spreading (or horizontal strain in the longitudinal direction) as a result of the embankment loading. The analyses of settlement and horizontal strain are discussed in Section 6.4.2 and Section 6.4.3, respectively. If the culvert structure is capable of tolerating the estimated total and differential settlements and associated strains, the culvert could be constructed with a camber (if necessary), such that once the settlement has occurred, the hydraulic flow will be maintained as originally designed. However, culverts designed to include a camber may have a relatively high risk of poor performance resulting in unfavourable drainage/surface water flow conditions at some locations. It is important to note that it is inherently difficult to predict settlements for the variable subsurface conditions along the culvert alignment with such a degree of accuracy to allow an accurate camber design. If the actual settlements are smaller than predicted, the culvert may not achieve the design grade or slope, which could impede the flow of water. If actual settlements are larger than expected, the culvert may sag below the design invert elevation and as a result some sediments may be deposited inside the culvert and could reduce the flow of water. Expansion joints may also be included along the length of the culvert to accommodate horizontal strain which will occur in conjunction with the vertical settlement.

Sub-excavation of all existing organic material is required prior to placement of any fill or culvert bedding material, as organic soils are highly compressible and can undergo significant secondary (creep) settlement.

6.4 Stability, Settlement and Horizontal Strain

The following sections summarize the methods utilized to carry out analyses of embankment stability and settlement of the culvert and methods utilized to evaluate horizontal strains along the culvert beneath the zone of influence of the proposed embankment loading.

6.4.1 Stability

The stability analysis carried out for the 7.1 m high rock fill embankment at the proposed Culvert BC1A location (using GeoStudio (Version 7.19) by Geo-Slope International) indicates that after completion of construction (including removal and replacement of the organic deposits), the embankment will have a Factor of Safety (FoS) greater than 1.3 for a deep-seated, global failure surface that would impact the operation of the highway. Therefore, stability mitigation is not required for the embankment at the location of Culvert BC1A.

6.4.2 Settlement

The following sections outline the methods used to conduct the settlement analyses at the culvert location and the results of the analyses.



6.4.2.1 Methodology

To estimate the magnitude of the expected settlements, analyses were carried out along the culvert alignment using the commercially available program Settle3D (Version 2.013) produced by Rocscience Inc.

The sources of settlement at this site are:

- primary time-dependent consolidation of the cohesive deposits;
- immediate settlement of the native granular soils.

Since the estimated preconsolidation pressure of the clayey silt to clay deposit will not be exceeded for the given embankment loading, creep settlement will not occur.

The thickness of the native compressible clayey silt to clay foundation soils and the height of the embankment vary along the proposed culvert alignment and therefore the settlements along the length of the culvert will similarly vary. As such, settlements have been assessed at the culvert inlet, mid-point (i.e. highway centreline median) and outlet.

The settlement analyses assume that all organic soils (i.e. topsoil) beneath the culvert alignment will be removed prior to construction and that granular fill will be used for replacement of sub-excavated material (as discussed in Section 6.7.1). The piezometric condition required in the analyses is based on the groundwater level observed in the piezometer installed approximately 10 m east of the proposed culvert location and is generally at about the level of the natural ground surface.

6.4.2.2 Parameter Selection

The immediate compression of the very loose to compact silty sand to sand layer overlying the cohesive deposit and the very loose to dense silt and compact to very dense silty sand to gravelly silty sand to gravel deposits underlying the cohesive deposit was modeled by estimating an elastic modulus of deformation based on the SPT 'N'-values and using correlations proposed by Bowles (1984) and Kulhawy and Mayne (1990). These estimated moduli values were compared with the typical range of expected values for similar soil types, as outlined in Canadian Highway Bridge Design Code and Commentary (CHBDC, 2006) and adjusted, if necessary.

The consolidation settlement of the cohesive deposit was assessed using the results of the laboratory index tests and in situ field vane tests in the boreholes in the area of the culvert to estimate the stress history and deformation parameters for the cohesive deposits at the culvert location. Estimates of deformation parameters (i.e. recompression and compression indices) were obtained using empirical correlations proposed in literature by Koppula (1986), Terzaghi and Peck (1967), Kulhawy and Mayne (1990) and Azzouz et al. (1976). The correlation by Koppula (1986) relating the natural water content and liquid limit to the compression index and Azzouz et al. (1976) relating the void ratio to the compression index were found to be the most consistent with the results of laboratory consolidation tests for the clayey soils in the swamp crossing areas of this highway realignment, and as such were used to represent the deformation properties at this culvert location.

The following correlation relating in situ undrained shear strength to preconsolidation stress proposed by Mesri (1975) was employed:



$$\sigma'_p = \frac{S_{u(mob)}}{0.22}$$

where:

- $S_{u(mob)}$ = $\mu S_{u(FV)}$
 σ'_p = preconsolidation stress (kPa)
 $S_{u(mob)}$ = average mobilized undrained shear strength (kPa)
 $S_{u(FV)}$ = undrained shear strength from field vane test (kPa)
 μ = Bjerrum's (1973) correction factor based on Plasticity Index

The coefficient of consolidation, c_v (cm²/s), required in the settlement time-rate analysis was estimated from the U.S. Navy (1986) correlation with liquid limits assuming over-consolidated soils.

The simplified stratigraphy together with the associated strength and unit weight values assigned to the applicable native soil types at the culvert location are summarized below.

Soil Type	γ' (kN/m ³)	σ'_p (kPa)	e_o	C_c	C_r	E' (MPa)	c_y (cm ² /s)
Topsoil *	12	-	-	-	-	-	-
Silty Sand to Sand	20	-	-	-	-	12	-
Clayey Silt to Clay	17.5	200	0.8	0.5	0.03	-	4.3×10^{-3}
Silt	18	-	-	-	-	6	-
Silty Sand to Gravelly Silty Sand to Gravel	20	-	-	-	-	35	-

Note: * The topsoil is to be removed prior to culvert/embankment construction.

6.4.2.3 Results of Analysis

Based on the results of the settlement analysis, the estimated total settlement at the centreline of the proposed realignment along the culvert alignment is 120 mm and is comprised of about 80 mm of immediate settlement due to compression of the cohesionless deposits and about 40 mm of primary consolidation of the cohesive deposit, up to 3.5 m thick at ends and perhaps 3 m thick at both shoulders. The estimated total settlement comprised of immediate and post-construction settlement, at the inlet and outlet of the proposed culvert alignment is 30 mm and 20 mm, respectively. The differential settlement between the centre of the culvert and the outlet end is therefore about 100 mm and will occur after culvert installation.

6.4.3 Horizontal Strain

The following sections outline the methods used to estimate the horizontal strain along the culvert and the results of the analysis.



6.4.3.1 Parameter Selection

As a result of the two-dimensional nature of the proposed embankment geometry, shear stresses will be mobilized in the foundation soils upon completion of embankment construction causing lateral spreading of the foundation soils and new embankment fill. This, in conjunction with the non-uniform vertical settlement of the foundation soils along the proposed culvert alignment, will generate horizontal straining along the newly constructed culvert. In order to maintain structural integrity of the culvert, the culvert design must incorporate a suitable allowance for extension at the joints/couplings of the culvert segments to prevent the culvert from cracking and/or failing in tension.

The research work by Rutledge and Gould (1973) on the movements on articulated conduits under earth dams on compressible foundations can be used to estimate the magnitude of the horizontal strain likely to occur as a result of the proposed embankment construction at culvert sites. The following equations have been used to obtain a relationship between vertical settlement, vertical strain, horizontal strain and maximum joint opening as a result of settlement of the foundation soils:

$$\varepsilon_v = \frac{\delta_v}{d}$$

$$\varepsilon_h = \varepsilon_v \frac{\varepsilon_h}{\varepsilon_v}$$

$$\Delta L = \varepsilon_h L$$

where:

ΔL = maximum joint opening (m)

ε_v = maximum vertical strain

ε_h = maximum horizontal strain

$\frac{\varepsilon_h}{\varepsilon_v}$ = estimated ratio of maximum horizontal strain to maximum vertical strain from Figure 2 in Rutledge and Gould, 1973)

L = length of culvert (m)

δ_v = maximum vertical settlement of culvert as a result of immediate and post-construction settlement of foundation soils and granular fill / bedding material (m)

d = thickness of compressible foundation deposits at culvert location (m)

6.4.3.2 Results of Analysis

The settlement analysis indicates that the total post-construction settlement of the foundation soils along the permanent culvert will be between about 20 mm and 120 mm, with an estimated differential settlement of about 100 mm. Therefore, the maximum post-construction horizontal strain along the 43 m long permanent culvert is estimated to be about 0.4 per cent of the culvert length, or about 175 mm. To mitigate the horizontal strain along the culvert, consideration could be given to preloading the embankment and construction of a temporary culvert, or the culvert could be constructed with a camber.

6.5 Geotechnical Axial Resistance

If a 1 m wide box culvert is considered for this site, the culvert should be designed on the basis of a factored geotechnical axial resistance at Ultimate Limit States (ULS) of 200 kPa and a geotechnical reaction at Serviceability Limit States (SLS) of 100 kPa (for 25 mm of settlement) based on the culvert being founded on a properly prepared subgrade/granular bedding (as discussed in Section 6.7). The geotechnical resistances are applicable for loads that will be applied perpendicular to the base of the culvert. Where loads are not applied



perpendicular to the base of the culvert, inclination of the loads should be taken into account in accordance with Section 6.7.4 and Section C6.7.4 of the Canadian Highway Bridge Design Code (CHBDC) and its Commentary.

The loading on the foundation soils below the culvert and the associated total settlement at the culvert location will be governed by the design height of the overlying and adjacent embankment fill. As such, it is recommended that the structural engineer exercise caution when utilizing the values of the geotechnical axial resistance at SLS in the design of the culvert. Where the culvert is constructed following completion of all foundation soil settlement due to construction of embankment fills, the SLS values as provided may be used for the culvert design for settlement of 25 mm.

6.5.1 Resistance to Lateral Loads/Sliding Resistance

Resistance to lateral forces/sliding resistance between the base of a box culvert and the granular fill/bedding placed following sub-excavation of organic deposits should be calculated in accordance with Section 6.7.5 of the CHBDC. The following summarizes the coefficient of friction for the interface materials.

Interface Materials	Coefficient of Friction ($\tan \delta$)
Precast Concrete Box Culvert on Compacted Granular 'A' or Granular 'B' Type II	$\tan \delta = 0.45$
Cast-in-Place Concrete Box Culvert on Compacted Granular 'A' or Granular 'B' Type II	$\tan \delta = 0.55$

These values represent unfactored values.

6.6 Lateral Earth Pressures

If a box culvert is selected for this site, the lateral earth pressures acting on the walls of the culvert will depend on the type and method of placement of backfill materials, the nature of the soils/embankment fill behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls.

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

- Select, free draining granular fill meeting the specifications of OPSS PROV. 1010 (Aggregates) Granular 'A' or Granular 'B' Type II but with less than 5 per cent passing the No. 200 (0.075 mm) sieve should be used as backfill behind the culvert walls, and on top of the culvert for a thickness of up to 300 mm. Backfill should be placed in a maximum of 200 mm loose lift thickness and nominally compacted. Weep holes should be installed in the walls to provide positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS 501 (Compacting) as amended by Special Provision (SP) 105S21 (Compacting).



- For a box culvert, granular fill (where utilized) should be placed in a zone with the width up to 300 mm behind the back of the culvert. The pressures are based on the proposed embankment fill materials and the following parameters (unfactored) may be used:

Fill Type	Soil Unit Weight	Coefficients of Static Lateral Earth Pressure	
		At-Rest, K_o	Active, K_a
Granular 'A'	22 kN/m ³	0.43	0.27
Granular 'B' Type II	21 kN/m ³	0.43	0.27

If the culvert structure allows for lateral yielding, active earth pressures may be used in the foundation design. If the culvert structure does not allow lateral yielding, at-rest earth pressures should be assumed for foundation design. The movement to allow active pressures to develop within the backfill, and thereby assume a restrained structure, may be taken as per Table C6.6 of the Commentary to the CHBDC.

6.7 Culverts – Construction Considerations

6.7.1 Excavation and Replacement below Culvert Bedding

Prior to the placement of any bedding material or granular fill, all organic soils should be stripped from the plan limits of the proposed works. Given the design invert elevations of the proposed culvert (Elevations 305.0 m and 303.6 m at the inlet and outlet ends) and the presence of a relatively thin up to about 300 mm thick deposit of organics/topsoil at the proposed culvert location, it is assumed that the excavation will extend to the groundwater level and granular fill, such as Granular 'B' Type II or Granular 'A', will be used to backfill the excavation.

All excavations should be carried out in accordance with OPSS 902 (Excavating and Backfilling – Structures) and must be carried out in accordance with Ontario Regulation 213 Ontario Occupational Health and Safety Act for Construction Projects (as amended).

6.7.2 Culvert Bedding and Backfill

6.7.2.1 Circular Culvert

The bedding, levelling pad and backfill for a circular concrete pipe or CSP culvert should be in accordance with OPSS 802.034 (Rigid Pipe Bedding and Cover in Embankment) and culvert construction should be in accordance with OPSS 421 (Pipe Culvert Installation in Open Cut). It is important that the backfill at the haunches be well compacted. The circular culvert should be constructed on a minimum 300 mm thick layer of OPSS PROV. 1010 (Aggregates) Granular 'A' or Granular 'B' Type II material for bedding purposes.

6.7.2.2 Precast Culvert

The bedding, levelling pad and granular backfill requirements for a precast culvert should be in accordance with OPSS 422 (Precast Reinforced Concrete Box Culverts). The bedding should be placed in lifts not exceeding 200 mm in loose thickness, and compacted to at least 98 per cent of the Standard Proctor maximum dry density of the material as specified in OPSS 501/SP 105S21 (Compacting). In addition, a minimum 75 mm thick uncompacted levelling pad consisting of OPSS PROV. 1010 Granular 'A' material or concrete fine aggregate



(meeting the grading requirements specified in SP 110S11 (Aggregates - Concrete)) should be provided as shown on OPSP 803.010 (Backfill and Cover for Concrete Culverts) for culvert construction in dry conditions.

6.7.2.3 Cast-in-Place Culvert

Should a cast-in-place culvert be preferred, the bedding and backfill requirements should be in accordance with OPSS 902 (Excavating and Backfilling – Structures). The box culvert should be provided with at least 300 mm of OPSS PROV. 1010 (Aggregates) Granular 'A' or 'B' Type II for bedding purposes and partial frost protection. The bedding should be placed in lifts not exceeding 200 mm in loose thickness, and compacted to at least 98 per cent of the Standard Proctor maximum dry density of the material as specified in OPSS 501/SP 105S21 (Compacting).

6.7.2.4 General

Backfill behind the culvert walls, should consist of granular fill meeting the specifications for OPSS PROV. 1010 (Aggregates) Granular 'A' or Granular 'B' Type II, but with less than 5 percent passing the No. 200 (0.075 mm) sieve. The granular backfill should be placed and compacted in accordance with OPSS 501/SP 105S21 (Compacting). The fill should also be placed concurrently on both sides of the culvert walls, ensuring that the backfill depth on one side does not exceed the other side by more than 400 mm.

The culverts should be designed for the full overburden stress and appropriate live loads, assuming a fill unit weight of 22 kN/m³ for Granular 'A' and 21 kN/m³ for Granular 'B' Type II placed above and surrounding the culvert.

Inspection and field density testing should be carried out by qualified geotechnical personnel during all engineered fill placement operations to ensure that appropriate materials are used, and that adequate levels of compaction have been achieved.

6.7.3 Erosion Protection

Provisions should be made for scour and erosion protection at the culvert location. In order to prevent surface water from flowing either beneath the culvert (potentially causing undermining and scouring) or around the culvert (creating seepage through the embankment fill, and potentially causing erosion and loss of fine soil particles), a concrete cut-off wall or clay seal should be provided at the upstream end of the culvert. If a clay seal is adopted, the clay material should meet the requirements of OPSS 1205 (Clay Seal), and the seal should be a minimum 1 m thick if constructed of natural clay or soil-bentonite mix and extend from a depth of 1 m below the scour level to a minimum horizontal distance of 2 m on either side of the culvert inlet opening, and a minimum vertical height equivalent to the high water level including along the embankment slope. Alternatively, a 0.6 m thick clay blanket (if constructed of natural clay or a soil-bentonite mix) may be constructed, extending upstream three times the culvert height and along the adjacent slopes to a height of two times the culvert height or the high water level, whichever is greater.

The requirements for and design of erosion protection measures for the inlet and outlet of the culverts should be assessed by the hydraulics design engineer. As a minimum, rip-rap treatment for the outlet of the culverts should be consistent with OPSP 810.010 (Rip-Rap Treatment for Sewer and Culvert Outlets). Erosion protection for the



inlet of the culverts should generally follow the standard presented in OPSD 810.010, with the rip-rap placed up to the toe of slope level, in combination with the cut-off measures noted above. Similarly, rip-rap should be provided over the full extent of the clay blanket, including the creek side slopes and fill slope over the culverts.

6.7.4 Control of Groundwater and Surface Water

Excavation within the plan limits of the proposed culvert alignment will be required to remove organics/topsoil prior to placement of backfill, bedding material and the actual culvert structure. As the excavation likely will extend to the groundwater level, some groundwater flow into the excavation can be expected to occur due to the relatively permeable near-surface subsoils. Therefore, control of surface water and groundwater will be necessary at the culvert location to allow for construction to be carried out in dry conditions, if required. Given that the design invert is approximately at or above existing ground surface and the excavation for removal of organics/topsoil is relatively shallow, it is not anticipated that any specialized measures will be required to control groundwater and allow construction in the dry. Surface water should be directed away from the excavations areas to prevent ponding of water.

7.0 CLOSURE

This report was prepared by Mr. Matt Thibeault, EIT and Mr. André Bom, P.Eng. Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project and Principal with Golder, carried out a technical review and quality control of the report.



Report Signature Page

GOLDER ASSOCIATES LTD.


Matt Thibeault, EIT
Geotechnical Engineering Intern



André Bom, P.Eng.
Geotechnical Engineer



Jorge M.A. Costa, P. Eng.
Designated MTO Contact, Principal

MT/AB/JMAC/kp

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ASTM International:

ASTM D1586	Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils
ASTM D2573	Standard Test Method for Field Vane Shear Test in Cohesive Soil

Commercial Software:

Settle3D (Version 2.013) by Rocscience Inc.
GeoStudio (Version 7.19) by Geo-Slope International Ltd.

Contract Design Estimating and Documentation (CDED):

Special Provision 105S21	Amendment to OPSS 501 – Compacting
Special Provision 110S11	Amendment to OPSS 1002 – Material Specification for Aggregates - Concrete

Ontario Occupational Health and Safety Act:

Ontario Regulation 213	Construction Projects (as amended)
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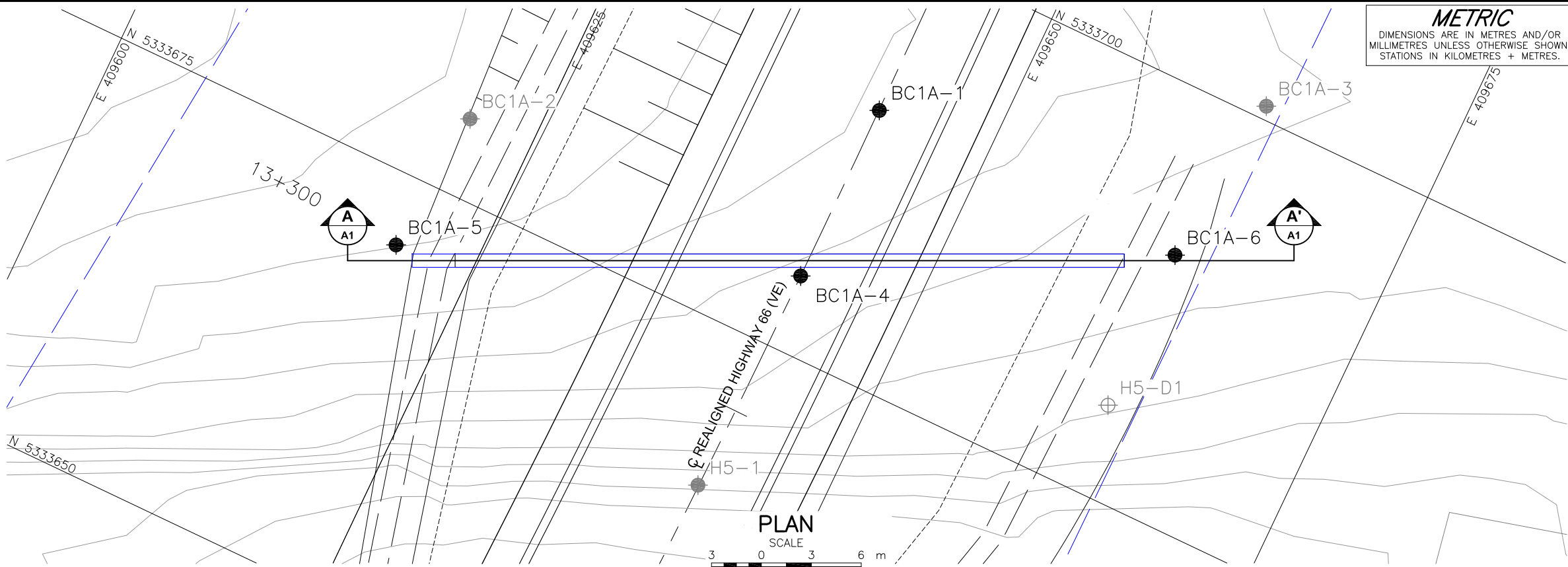
Ontario Provincial Standard Drawing:

OPSD 802.034	Rigid Pipe Bedding and Cover in Embankment, Original Ground: Earth or Rock
OPSD 803.010	Backfill and Cover for Concrete Culverts with Spans Less Than or Equal to 3.0m
OPSD 810.010	Rip-Rap Treatment for Sewer and Culvert Outlets

Ontario Provincial Standard Specification:

OPSS 421	Construction Specification for Pipe Culvert, Installation in Open Cut
OPSS 422	Construction Specification for Precast Reinforced Concrete Box Culverts and Box Sewers in Open Cut
OPSS 501	Construction Specification for Compacting
OPSS 902	Construction Specification for Excavating and Backfilling – Structures
OPSS 1205	Material Specification for Clay Seal
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

Ontario Water Resources Act Ontario Regulation 903 Wells (as amended)



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

CONT No.
GWP No. 5091-07-00

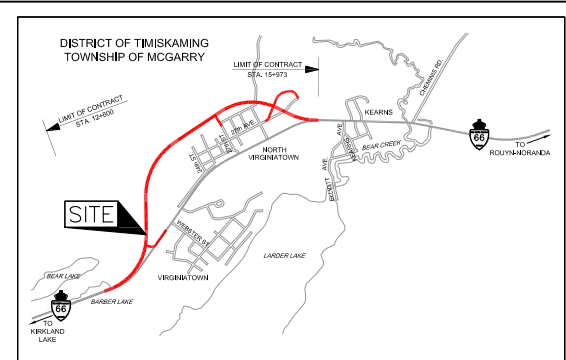


HIGHWAY 66
CULVERT AT STA 13+310
BOREHOLE LOCATIONS AND
SOIL STRATA

SHEET



Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA



KEY PLAN
SCALE
700 0 700 m

LEGEND

- Borehole
- ⊕ Dynamic Cone Penetration Test - Previous Investigation
- Borehole - Previous Investigation
- ⊥ Seal
- ⊥ Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated
(Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- ≡ WL upon completion of drilling
- ≡ WL in piezometer, measured on MAY 17, 2013

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
BC1A-1	304.3	5333690.6	409643.0
BC1A-4	304.0	5333679.6	409643.0
BC1A-5	303.0	5333670.9	409620.2
BC1A-6	304.9	5333690.4	409662.8

NOTES

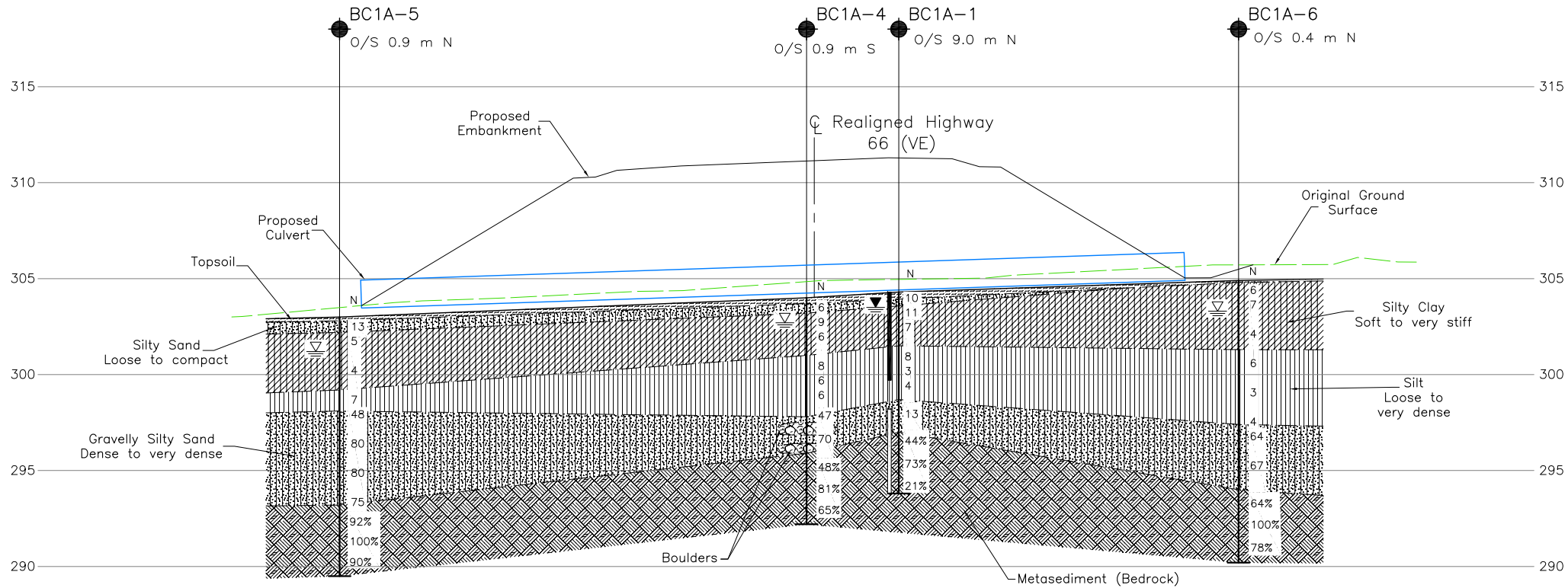
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

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

REFERENCE

Base plans provided in digital format by MMM, drawing file nos. H3211009D16 ROLL PLAN-ULTIMATE and PDR.dwg, received DEC 3, 2012. Keyplan drawing file nos. H3211009G02 received JAN 24, 2013.



A-A'
A1
CULVERT AT STA 13+310
HIGHWAY 66
HORIZONTAL SCALE
3 0 3 6 m
VERTICAL SCALE
3 0 3 6 m



NO.	DATE	BY	REVISION
Geocres No. 32D-14			
HWY. 66	PROJECT NO. 10-1191-0044		DIST.
SUBM'D. MT	CHKD.	DATE: NOV 2013	SITE:
DRAWN: TB	CHKD. SEMC	APPD. JMAC	DWG. 1



APPENDIX A

Highway 66 Realignment, Virginiatown—Culvert at STA 13+310 Record of Boreholes



LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

Notes: 1
2

$$\tau = c' + \sigma' \tan \phi'$$
$$\text{shear strength} = (\text{compressive strength})/2$$



LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

Dynamic Cone Penetration Resistance; N_d :

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

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

(b) Cohesive Soils Consistency

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

IV. SOIL TESTS

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

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

V. MINOR SOIL CONSTITUENTS

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



LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERINGS STATE

Fresh: no visible sign of weathering

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

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

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

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

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

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

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

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

CORE CONDITION

Total Core Recovery (TCR)

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

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

DISCONTINUITY DATA

Fracture Index

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

Dip with Respect to Core Axis

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

Description and Notes

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

Abbreviations

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

PROJECT		10-1191-0044		RECORD OF BOREHOLE No BC1A-1				1 OF 1 METRIC					
G.W.P.		5091-07-00		LOCATION		N 5333690.6; E 409643.0		ORIGINATED BY					
DIST		HWY 66		BOREHOLE TYPE		108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring		COMPILED BY					
DATUM		GEODETIC		DATE		September 6 and 7, 2012		CHECKED BY					
								SEMC					
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)			
304.3	GROUND SURFACE												
0.0	TOPSOIL		1	SS	10								
303.6	SAND, some silt, some gravel, trace clay, trace organics Compact Brown Moist		2	SS	11								
0.7	CLAY, trace sand Firm to stiff Grey to brown Moist		3	SS	7								
301.5	CLAYEY SILT, trace sand Stiff Grey to brown Moist to wet		4	SS	8								
300.6	SILT, some clay, trace sand Very loose Grey Wet		5	SS	3								
			6	SS	4								
298.7	GRAVEL, some sand, some silt Compact Grey Wet		7	SS	13								
	Spoon bouncing at 6.6 m depth. Broken rock encountered between 6.6 m and 7.3 m depth.												
297.0	METASEDIMENT (BEDROCK)		1	RC	REC 100%								RQD = 44%
	Bedrock cored from 7.3 m depth to 10.5 m depth. For coring details see Record of Drillhole BC1A-1.		2	RC	REC 100%								RQD = 73%
			3	RC	REC 85%								RQD = 21%
293.8	END OF BOREHOLE												
10.5	Note: 1. Water level at a depth of 1.1 m below ground surface (Elev. 303.2 m) upon completion of drilling. 2. Water level in piezometer at a depth of 1.1 m below ground surface (Elev. 303.2 m) after installation of piezometer, at 0.6 m below ground surface (Elev. 303.7 m) on November 17, 2012 and at 0.8 m below ground surface (Elev. 303.5 m) on May 17, 2013.												

SUD_MTO 003 10-1191-0044SUD.GPJ GAL-MISS.GDT 16/08/13 DATA INPUT:

PROJECT: 10-1191-0044

RECORD OF DRILLHOLE: BC1A-1

SHEET 1 OF 1

LOCATION: N 5333690.6 ; E 409643.0

DRILLING DATE: September 7, 2012

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: Landcore Drilling Inc.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN	JN - Joint FLT - Fault SHR- Shear VN - Vein CJ - Conjugate BD- Bedding FO- Foliation CO- Contact OR- Orthogonal CL - Cleavage PL - Planar CU- Curved UN- Undulating ST - Stepped IR - Irregular PO- Polished K - Slickensided SM- Smooth Ro - Rough MB- Mechanical Break BR - Broken Rock										DIP w.r.t. CORE AXIS °	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	HYDRAULIC CONDUCTIVITY k, cm/s	Diametral Point Load Index (MPa)	RMC -Q' AVG.	NOTES WATER LEVELS INSTRUMENTATION				
							RECOVERY		R.Q.D. %	FRACT. INDEX METRES	DISCONTINUITY DATA															10 ⁰	10 ¹	10 ²	10 ³
							TOTAL CORE %	SOLID CORE %			B Angle °	DIP w.r.t. CORE AXIS °	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn													

DEPTH SCALE

1 : 50



LOGGED: MT

CHECKED: SEMC

SUD-RCK 10-1191-0044SUD.GPJ GAL-MISS.GDT 16/08/13 DATA INPUT:

[illegible]

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

SHEET 1 OF 1

DATUM: GEODETIC

DRILLING CONTRACTOR: George Downing Estate Drilling Ltd.

LOGGED: MR
CHECKED: SEMC

SUD-RCK 10-1191-0044SUD.GPJ GAL-MISS.GDT 16/08/13 DATA INPUT:

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

SHEET 1 OF 1

DATUM: GEODETIC

DRILLING CONTRACTOR: George Downing Estate Drilling Ltd.

CHECKED: SEMC

SUD-RCK 10-1191-0044SUD.GPJ GAL-MISS.GDT 16/08/13 DATA INPUT:

[illegible]



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

SUD MTO 003 10-1191-0044SUD.GPJ GAL-MISS.GDT 16/08/13 DATA INPUT:

PROJECT: 10-1191-0044

RECORD OF DRILLHOLE: BC1A-6

SHEET 1 OF 1

LOCATION: N 5333690.4 ;E 409662.8

DRILLING DATE: May 18, 2013

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: George Downing Estate Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN	JN - Joint FLT - Fault SHR- Shear VN - Vein CJ - Conjugate BD- Bedding FO- Foliation CO- Contact OR- Orthogonal CL - Cleavage PL - Planar CU- Curved UN- Undulating ST - Stepped IR - Irregular PO- Polished K - Slickensided SM- Smooth Ro - Rough MB- Mechanical Break BR - Broken Rock														NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
							RECOVERY		R.Q.D. %	FRACT INDEX METRES	DISCONTINUITY DATA					HYDRAULIC CONDUCTIVITY			Diametral Point Load Index (MPa)	RMC -Q' AVG.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
							TOTAL CORE %	SOLID CORE %			B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	k, cm/s	10 ⁻⁶				10 ⁻⁶	10 ⁻⁶																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																														
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DEPTH SCALE

1 : 50



LOGGED: MR

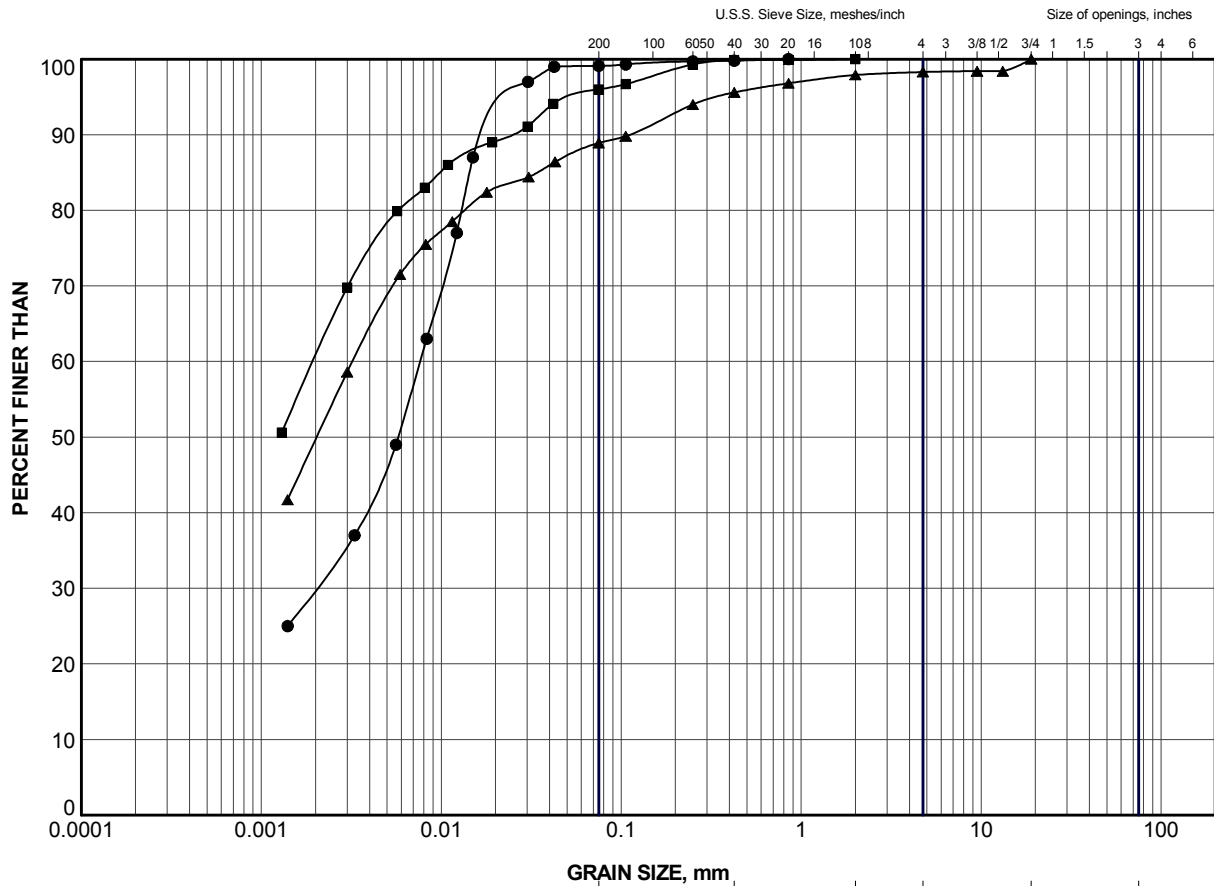
CHECKED: SEMC

SUD-RCK 10-1191-0044SUD.GPJ GAL-MISS.GDT 16/08/13 DATA INPUT:



APPENDIX B

Highway 66 Realignment, Virginiatown—Culvert at STA 13+310 Laboratory Tests Results



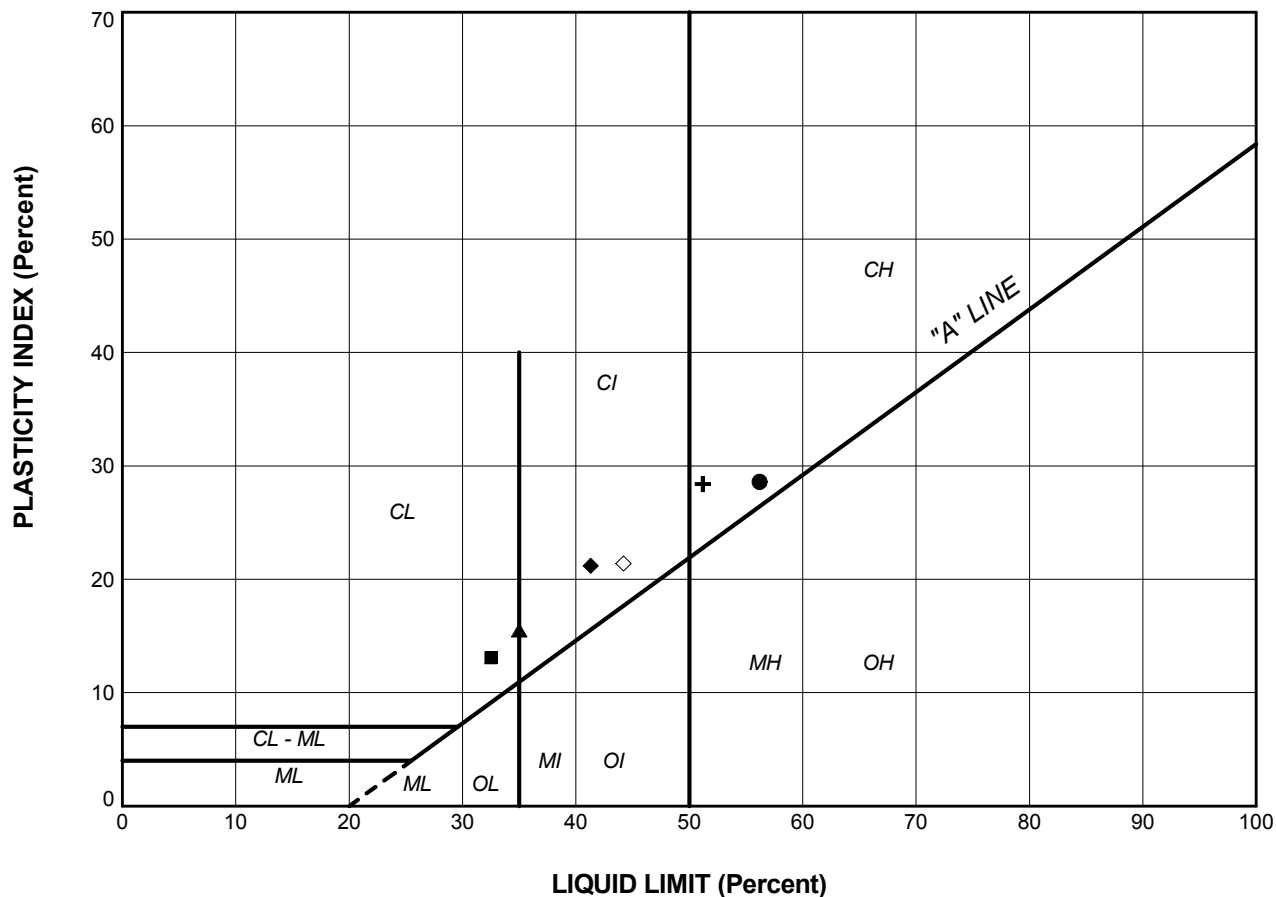
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BC1A-1	4	300.8
■	BC1A-4	2	302.9
▲	BC1A-6	1b	304.5

PROJECT					
HIGHWAY 66 - CULVERT BC1A STA 13+310					
TITLE					
GRAIN SIZE DISTRIBUTION CLAYEY SILT to SILTY CLAY					
PROJECT No.		10-1191-0044		FILE No. 10-1191-0044C.GPJ	
DRAWN	TB	Jul 2013	SCALE	N/A	REV.
CHECK	SEMC	Jul 2013	FIGURE B1		
APPR	JMAC	Jul 2013			



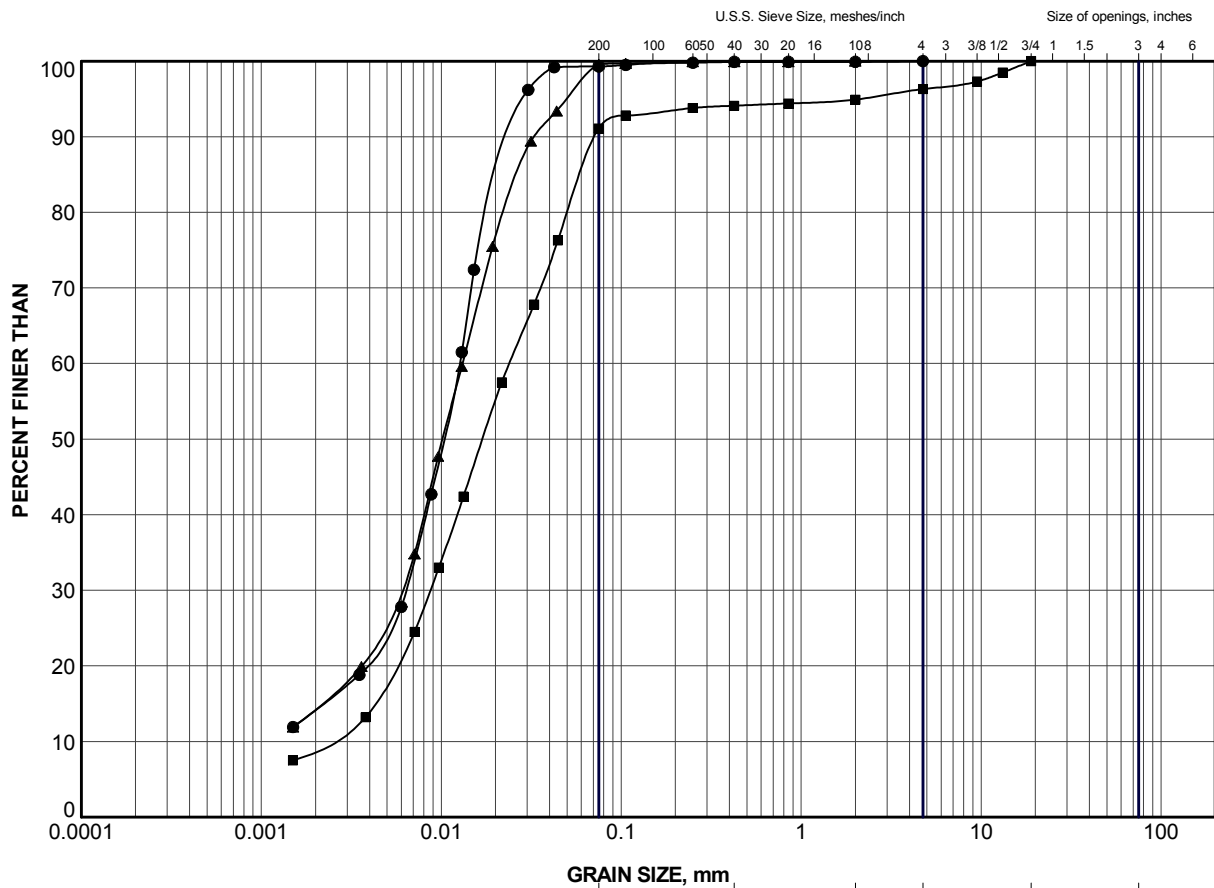


LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	BC1A-1	2	56.2	27.6	28.6
■	BC1A-1	4	32.5	19.4	13.1
▲	BC1A-4	2	35.0	19.5	15.5
+	BC1A-5	3	51.2	22.8	28.4
◆	BC1A-6	1b	41.3	20.1	21.2
◇	BC1A-6	3	44.2	22.8	21.4


PROJECT						HIGHWAY 66 - CULVERT BC1A STA 13+310					
TITLE						PLASTICITY CHART CLAYEY SILT to CLAY					
PROJECT No.			10-1191-0044			FILE No.			10-1191-0044C.GPJ		
DRAWN	TB	Jun 2013	CHECK	SEMC	Jun 2013	SCALE	N/A	REV.			
APPR	JMAC	Jun 2013				FIGURE B2					

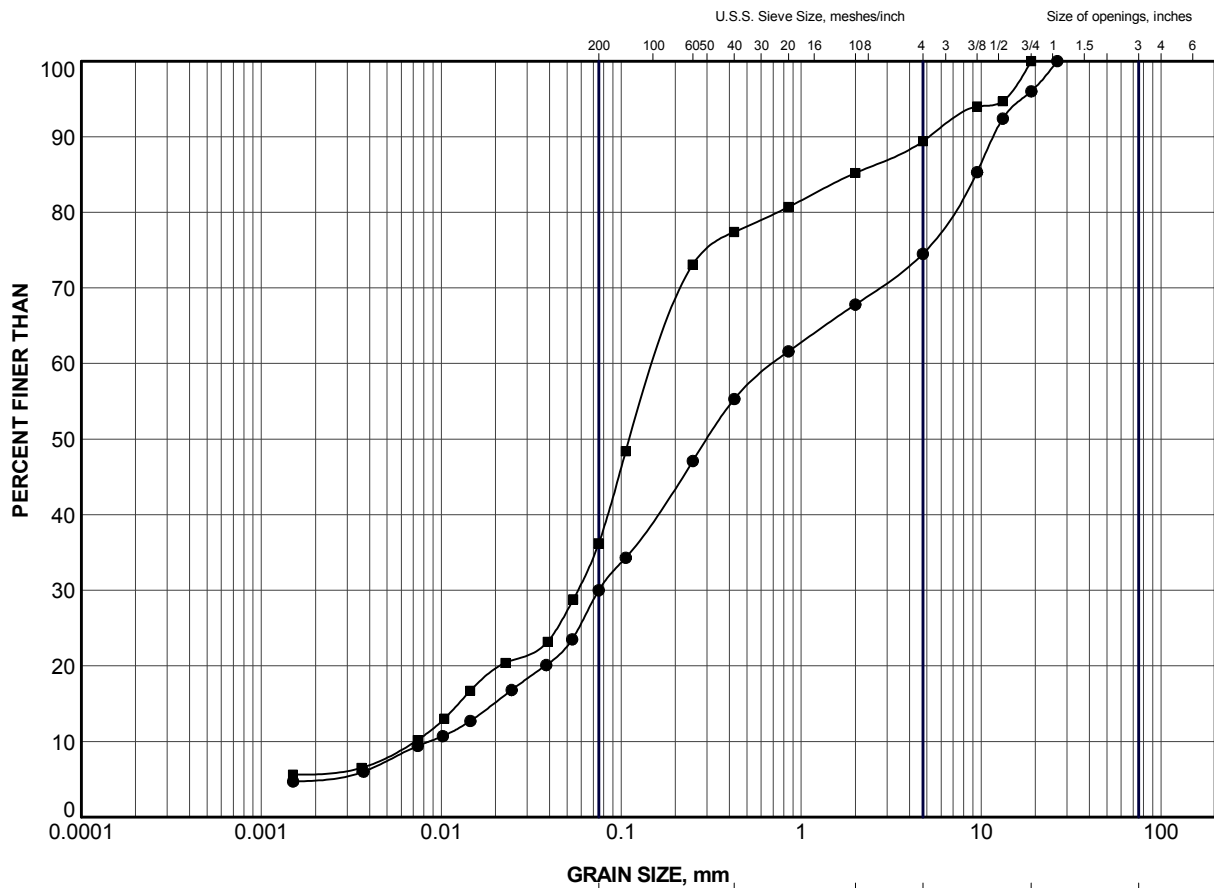




LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BC1A-1	6	299.4
■	BC1A-4	7	298.1
▲	BC1A-6	5	299.3

PROJECT				
HIGHWAY 66 - CULVERT BC1A STA 13+310				
TITLE				
GRAIN SIZE DISTRIBUTION SILT				
PROJECT No.		10-1191-0044		FILE No.
DRAWN		TB	Jun 2013	SCALE
CHECK		SEMC	Jun 2013	N/A
APPR		JMAC	Jun 2013	REV.
 Golder Associates SUDBURY, ONTARIO		FIGURE B3		



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BC1A-5	7	295.1
■	BC1A-6	7	297.0

PROJECT					
HIGHWAY 66 - CULVERT BC1A STA 13+310					
TITLE					
GRAIN SIZE DISTRIBUTION GRAVELLY SILTY SAND to SILT and SAND					
PROJECT No.		10-1191-0044		FILE No. 10-1191-0044C.GPJ	
DRAWN	TB	Jul 2013	SCALE	N/A	REV.
CHECK	SEMC	Jul 2013	FIGURE B4		
APPR	JMAC	Jul 2013			





Borehole BC1A-1
Elevation 297.0 m to 293.8 m



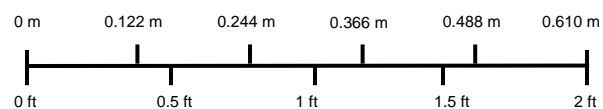
Borehole BC1A-4
Elevation 295.9 m to 292.2 m




Borehole BC1A-5
Elevation 293.2 m to 289.5 m



Borehole BC1A-6
Elevation 294.0 m to 290.2 m



PROJECT		Highway 66 – Culvert BC1A STA 13+310			
TITLE		BEDROCK CORE PHOTOGRAPHS			
	PROJECT No.	10-1191-0044		FILE No. ----	
	DESIGN	MH	June 2013	SCALE	AS SHOWN
	CADD	--		REV.	
	CHECK	SEMC	June 2013	FIGURE B5	
	REVIEW	JMAC	June 2013		

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For more information, visit golder.com

Africa	+ 27 11 254 4800
Asia	+ 86 21 6258 5522
Australasia	+ 61 3 8862 3500
Europe	+ 356 21 42 30 20
North America	+ 1 800 275 3281
South America	+ 56 2 2616 2000

solutions@golder.com
www.golder.com

Golder Associates Ltd.
1010 Lorne Street
Sudbury, Ontario, P3C 4R9
Canada
T: +1 (705) 524 6861

