November 15, 2013

FOUNDATION INVESTIGATION AND DESIGN REPORT

CULVERT AT STATION 13+080 (BC1)
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:
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REPORT

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PART A

FOUNDATION INVESTIGATION REPORT
CULVERT AT STATION 13+080 (BC1)
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FROM 10.6 KM EAST OF HIGHWAY 624 EASTERLY 3.4 KM
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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 BC1 crossing the proposed Highway 66 realignment at Station 13+080. 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 BC1 is located about 11.1 km east of Highway 624 within the High Fill area H4. The general location of the proposed Culvert BC1 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 BC1 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 BC1 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 45 m long extending across the proposed realigned Highway 66 at about STA 13+080. The land in the area of Culvert BC1 is cleared and is currently used as a corridor for power lines.

In general, the topography in the vicinity of Culvert BC1 consists of native terrain slightly sloping downward towards the south and is covered by sparse to densely populated treed areas, and wet grassy terrain adjacent to the existing Highway 66. The ground surface within the limits of the culvert alignment varies between about Elevation 305 m and 306 m. A detailed description of the subsurface conditions along the culvert alignment is presented in Section 4.0.

3.0 INVESTIGATION PROCEDURES

3.1 Foundation Investigation

The investigation for Culvert BC1 crossing the realigned Highway 66 was carried out between September 8 and 10, 2012, during which time a total of three boreholes were advanced along 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.

The field investigation was carried out using a track-mounted CME-55 drill rig supplied and operated by Landcore Drilling (Landcore) of Sudbury, Ontario. The boreholes were advanced through the overburden using 108 mm inner diameter 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, 1.5 m and 3.0 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). Samples of the cohesive soils were obtained at selected locations using 76 mm O.D. thin-walled ‘Shelby’ tubes (ASTM D1587, Standard Practice for Thin-Walled Tube Sampling) for relatively undisturbed samples. 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. Two 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 16.6 m below existing ground surface, including between 3.4 m and 3.6 m of bedrock coring.

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 BC1-3 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 sand and gravel deposit. The borehole annulus surrounding the piezometer screen was backfilled with sand and the remainder of the borehole was backfilled with a bentonite plug and cuttings. The piezometer exhibited artesian conditions and was backfilled with cement grout as required for the conditions and in accordance with the regulations.

The fieldwork was observed by a member 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 (ISRM[2]) standard

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[1] Canadian 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.

classification system. Classification of the bedrock core samples with respect to strength is based on Table 3.5 of CFEM (2006).

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		
BC1-1	5333456.8	409598.0	305.0	14.2
BC1-2	5333464.9	409579.7	306.0	10.5
BC1-3	5333449.6	409614.5	305.1	16.6

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 NOEGTS[4] 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.

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[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 a cohesive deposit comprised of a zone of firm to stiff clayey silt and soft to stiff silty clay to clay at depth. Underlying the cohesive deposit is a firm to stiff clayey silt deposit, underlain by a deposit of sandy gravel to sand and gravel, and metasediment 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

An approximately 0.1 m thick deposit of topsoil was encountered from ground surface in Boreholes BC1-1, BC1-2 and BC1-3, ranging from Elevation 306.0 m to 305.0 m.

4.2.2 Clayey Silt to Clay

A cohesive deposit consisting of an upper zone of clayey silt, a middle zone of silty clay to clay and transitioning to a lower zone of clayey silt was encountered underlying the topsoil in the boreholes. The total thickness of the cohesive deposit is between 3.1 m and 8.9 m and the surface of the deposit was encountered between Elevation 305.9 m and 304.9 m.

The upper zone is 1.3 m thick in Boreholes BC1-2 and BC1-3 and comprises brown clayey silt, some sand, some gravel to gravelly sandy clayey silt, trace organics. The middle zone, encountered in all boreholes, is comprised of brown to grey silty clay to clay, trace gravel, trace sand, and is between 1.8 m and 5.2 m thick. The lower zone is comprised of grey clayey silt in Boreholes BC1-1 and BC1-3 and is between 1.9 m and 3.0 m thick. Silt seams were encountered within the middle zone of the deposit between Elevations 304.9 m and 299.7 m in Borehole BC1-1 and within the middle and lower zones at Elevations 300.5 m, 299.1 m and 297.6 m in Borehole BC1-3. An approximately 25 mm thick sand seams was encountered at about Elevation 303.9 m in Borehole BC1-2.

4.2.2.1 Clayey Silt

The SPT ‘N’-values measured within the upper clayey silt portion of the deposit range from 6 blows to 14 blows per 0.3 m of penetration, suggesting a firm to stiff consistency.

The natural water content measured on one sample of the clayey silt portion of the deposit is about 13 per cent.

A grain size distribution test completed on one sample of the upper zone of the clayey silt deposit is shown on Figure B1 in Appendix B.

An Atterberg limits test was carried out on one sample of this portion of the deposit and the measured liquid limit is about 24 per cent, the plastic limit is about 17 per cent and the plasticity index is about 7 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.

4.2.2.2 Silty Clay to Clay

The SPT ‘N’-values measured within the lower silty clay to clay portion of the deposit range from 0 blows (weight of hammer) to 5 blows per 0.3 m of penetration. In situ field vane tests carried out within this portion of the deposit measured undrained shear strengths ranging between 23 kPa and 64 kPa, and the sensitivity is calculated to range between 3 and 5. The field vane tests results indicate that the silty clay to clay portion of the deposit has a soft to stiff consistency.

The natural water content measured on seven samples of this portion of the deposit ranges from about 37 per cent to 72 per cent.

The results of grain size distribution tests completed on four samples of the silty clay to clay portion of the deposit are shown on Figure B3 in Appendix B.

Atterberg limits tests were carried out on six samples of this portion of the deposit and measured liquid limits ranging from about 35 per cent to 64 per cent, plastic limits ranging from about 19 per cent to 27 per cent and plasticity indices ranging from about 17 per cent to 38 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B4 in Appendix B and indicate the material is classified as a silty clay of intermediate plasticity to clay of high plasticity.

4.2.2.3 Clayey Silt

The SPT ‘N’-values measured within the lower clayey silt portion of the deposit range from 0 blows (weight of hammer) to 3 blows per 0.3 m of penetration. In situ field vane tests carried out within this portion of the deposit measured undrained shear strengths ranging from 41 kPa to about 91kPa and the sensitivity is calculated to range from 3 to 5. The field vane tests results indicate that this portion of the deposit has a firm to stiff consistency.

The natural water content measured on three selected samples of this portion of the deposit is between about 28 per cent and 34 per cent.

A grain size distribution test completed on one sample of the lower clayey silt portion of this deposit is shown on Figure B1 in Appendix B.

Atterberg limits tests were carried out on two samples of this portion of the deposit and measured liquid limits of about 26 per cent and 30 per cent, plastic limits of about 18 per cent and 19 per cent and plasticity indices of about 7 per cent and 11 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.

4.2.3 Sandy Gravel to Sand and Gravel

An approximately 3.6 m to 4.0 m thick deposit of grey to brown, wet sandy gravel to sand and gravel was encountered underlying the clayey silt in Borehole BC1-1 and Borehole BC1-3 and below silty clay to clay in Borehole BC1-2. The surface of the deposit was encountered between depths of 3.2 m and 9.0 m below ground surface (corresponding to between Elevation 302.8 m and 296.1 m) and the bottom of the deposit is defined by the bedrock surface. A zone of cobbles was encountered in Boreholes BC1-1 and BC1-2 at Elevation 294.9 m and Elevation 299.7 m, with thicknesses of 0.2 m and 0.6 m, respectively.

The SPT 'N'-values measured within the sandy gravel to sand and gravel deposit range from 12 blows to 35 blows per 0.3 m of penetration, indicating a compact to dense relative density. Several samples did not penetrate the full sampler depth indicating the presence of very dense material and/or inferred cobbles.

The natural water content measured on three samples of this deposit ranges from about 2 per cent to 19 per cent.

The results of grain size distribution tests completed on three samples of this deposit are shown on Figure B5 in Appendix B.

4.2.4 Bedrock

Bedrock was encountered in all of the boreholes. The depth to the surface of the bedrock in these boreholes ranges from about 6.9 m to 13.0 m below ground surface, corresponding to between about Elevation 299.1 m to Elevation 292.1 m.

Bedrock was cored in all the boreholes for lengths between 3.4 m and 3.6 m. The retrieved bedrock core is described as very fine grained, moderately weathered to fresh, green to grey, metasediment with occasional fractured sheared zones as presented in the Record of Drillhole sheets in Appendix A. Photographs of the retrieved bedrock core samples are shown on Figure B6.

The Total Core Recovery (TCR) measured on all core samples ranges from 98 per cent to 100 per cent. The Solid Core Recovery (SCR) of the rock core samples ranges from 28 per cent to 100 per cent. The Rock Quality Designation (RQD) measured on the core samples ranges from 65 per cent to 100 per cent, indicating a rock mass of fair to excellent quality.

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

Borehole	Elevation (m)	UCS (MPa)
BC1-1	293.4	112
BC1-2	296.7	114
BC1-3	290.8	62

4.3 Groundwater Conditions

Groundwater levels were measured in the open boreholes during and upon completion of drilling and a piezometer was installed in Borehole BC1-3, sealed within the sand and gravel deposit to monitor the groundwater level. The groundwater levels measured 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)
BC1-1	Open Borehole	September 9, 2012	2.0	303.0
BC1-2	Open Borehole	September 10, 2012	2.1	303.9
BC1-3	Open Borehole	September 10, 2012	1.0	304.1
	Piezometer	November 17, 2012	-1.0 (i.e. above ground surface)	306.1
	Piezometer	May 19, 2013	-1.1 (i.e. above ground surface)	306.2

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. 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.

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PART B

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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 BC1, to be constructed across the proposed Highway 66 realignment at Station 13+080. The proposed 45 m long culvert inlet (north side) and outlet (south side) are at Elevations 305.4 m and 304.9 m and the proposed embankment at the culvert area is up to 3.6 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 addresses 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 H4) 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 BC1 at STA 13+080 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 Options

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. The following alternatives for culvert construction can be considered (where applicable, giving due consideration to the recommended foundation mitigation option for the high fill embankment in the culvert area):

- concurrent with embankment construction;
- following the embankment preload period; and
- following full sub-excavation of compressible deposits along the culvert alignment and concurrent with embankment construction.

In areas of culvert construction where relatively small settlements are estimated to occur due to the presence of relatively thin, compressible foundation soils, culvert construction can be carried out concurrently with the proposed new embankment construction provided that any requirements for maintaining embankment stability are implemented. If required, the culvert design could include a camber. However, this option is not technically feasible at this site as the estimated settlements that the culvert will experience cannot be accommodated by a camber.

Where relatively large settlements are estimated to occur, as is the case at this site, it is recommended that the culvert be constructed subsequent to the embankment preload period or after sub-excavation of cohesive deposits, to reduce settlement and provide adequate long-term performance of the culvert and the associated overlying roadway. In the case of the embankment preloading option, a temporary culvert would be required during the preloading period and the permanent culvert would be constructed afterwards. The following sections provide a more detailed discussion on the alternatives for culvert construction and measures to mitigate settlements and improve long-term performance of the culvert.

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.

6.3.1 Culvert Construction Concurrent with Embankment Construction

A culvert that is 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. 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, for a culvert designed to include a camber there is 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. The analyses of settlement and horizontal strain at Culvert BC1 are discussed in Section 6.4.2 and Section 6.4.3, respectively, and indicate that this construction option is not technically feasible even though no additional costs would be incurred for sub-excavation and backfilling operations, provision of a temporary culvert or for a shoring system.

6.3.2 Culvert Construction Following Embankment Preload Period

At locations where the magnitudes of estimated total and differential settlements and horizontal strains cannot be tolerated by a culvert (even with a camber) and/or where removal of localized cohesive deposits and replacement with granular fill is not considered practical, the permanent culvert should be constructed after a preload period. This would require the use of a temporary culvert during the preloading period. Preloading refers to the placement of fill to the proposed height of embankment (possibly in stages), in advance of construction of the permanent culvert, in order to consolidate the underlying compressible soils. If preloading of the embankment at the culvert location is completed prior to construction of the permanent culvert, the magnitude of total and differential settlement beneath the permanent culvert and horizontal strain along the culvert will be reduced. However, this mitigation option requires excavating through the new embankment fill to the culvert founding elevation at the end of the preload period in order to construct the culvert. At this site, this additional excavation and backfilling to replace the temporary culvert with the permanent culvert would cost approximately $6,800 (400 m^3 x $7/$m^3$ for excavation of embankment fill plus 400 m^3 x $10/$m^3$ for backfilling plus the cost of the temporary culvert, assuming it cannot be reused). Provided that the final fill above the permanent culvert is properly placed and compacted, the magnitude of differential settlement between the fill embankment (that has been compressed under its self-weight for the entire preload period) and the final backfill above the culvert should be acceptable. Refer to Section 6.4 for the assessment of settlement and strain for the permanent culvert.

6.3.3 Culvert Construction Following Full Sub-Excavation of Compressible Soils

Depending on the depth and thickness of any soft, compressible foundation deposit(s), the magnitude of total and differential settlement and horizontal strain could also be reduced by means of full sub-excavation and replacement along the culvert alignment to allow for permanent culvert construction prior to embankment loading (i.e. concurrent with embankment construction). At culvert locations where the compressible deposits are thick, the resulting magnitude of settlements as well as the associated horizontal strains, even with full sub-excavation, may still be too large as a result of compression of the underlying fill itself, to accommodate standard culvert construction. However, where there is a limited thickness and depth of soft, compressible soils underlying the proposed culvert, full sub-excavation and replacement is a feasible option to reduce the settlement and allow for culvert construction in conjunction with the construction of the new embankment. The cost for sub-excavation and backfilling of the compressible deposits, although not practical at this site due to the up to 9 m depth of required sub-excavation and the potential need for shoring adjacent to the existing highway would be approximately $51,000 (3000 m^3 x $7/$m^3$ for sub-excavation and 3000 m^3 x $10/$m^3$ for backfilling plus the cost of shoring, if required).

Although full sub-excavation of the cohesive deposits will improve the settlement performance of the culvert and embankments in close proximity of the sub-excavation, adjacent areas of the embankment may not experience the same improvements in settlement performance depending on the mitigation measures adopted for the adjacent high fill embankment . As a result, the overlying embankment may experience some differential settlements along its alignment depending on the timing of embankment construction/culvert construction, type of backfill and timing of final earthworks and paving.

It should also be noted that settlement of replacement fill beneath a culvert base will occur, primarily during construction, and could constitute a significant portion of the expected settlements, depending on the depth of sub-excavation and replacement required.

We understand that at this culvert site, due to the close proximity of the existing Highway 66 embankment relative to the proposed embankment, full sub-excavation of the cohesive deposit is not considered practical.

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 3.6 m high rock fill embankment at the proposed Culvert BC1 section (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 BC1.

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 rate of settlement/consolidation of the cohesive foundation soils was assessed using Terzaghi's one-dimensional consolidation theory.

The sources of settlement at this site are:

- primary time-dependent consolidation of the cohesive deposits;
- secondary time-dependent (creep) consolidation of the cohesive deposits (long-term);
- immediate settlement of the native granular soils; and
- self-weight compression of the embankment fill materials beneath the culvert (where applicable).

The thickness of the native cohesive and granular 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 at about the level of the natural ground surface.

6.4.2.2 Parameter Selection

The immediate compression of the compact to very dense sandy gravel to sand and gravel deposit below the cohesive soils 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 deposits was assessed using the results of the laboratory index tests and in situ field vane tests in the boreholes and nearby consolidation testing 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 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 (cm^2/s), required in the settlement time-rate analysis was estimated from the U.S. Navy (1986) correlation with liquid limits assuming normally-consolidated soils.

In addition to primary consolidation within cohesive deposits, secondary compression will also occur. Secondary compression is referred to as creep settlement and occurs over a long period of time, after full dissipation of excess pore pressure under a constant stress. The following relationship has been employed for estimating the magnitude of creep settlement over the life of the embankment following the completion of primary settlement at each location.

$$S_c = HC_{\alpha\varepsilon} log(\frac{t}{t_{EoP}})$$

where:
S_c = secondary consolidation (creep) settlement (mm)
$C_{\alpha\epsilon}$ = modified secondary compression index as estimated from laboratory consolidation tests and/or from the empirical correlation by Mesri (1973)
H = initial thickness of normally consolidated portion of compressible clay deposit (mm)
t = post-construction period of interest (20 years)
t_{EoP} = time to reach end of primary consolidation (years)

The values of modified secondary compression index ($C_{\alpha\varepsilon}$) estimated from the empirical correlation were compared with the values of $C_{\alpha\varepsilon}$ calculated from the results of the laboratory consolidation tests, where necessary.

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

Stratigraphic Unit	**γ' (kN/m^3)**	**σ_p' (kPa)**	**e_o**	**C_c**	**C_r**	**E' (MPa)**	**c_v (cm^2/s)**
Topsoil*	12	-	-	-	-	-	-
Clayey Silt (Crust)	17.5	296 to 91	1.2 to1.8	0.6 to 1.0	0.03 to 0.05	-	1.4×10^{-3}
Clayey Silt to Clay (Lower, Below about Elev. 303.0 m)	16.5	91	1.8 to 0.8	1.0 to 0.6	0.05 to 0.03	-	1.4×10^{-3}
Sand and Gravel to Sandy Gravel	20	-	-	-	-	35	-

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

6.4.2.3 Results of Analysis

Given that the proposed culvert location is within High Fill H4 and therefore subjected to the same settlement issues for the embankment crossing the swamp area, it is recommended that the settlement mitigation recommendations for the proposed culvert location be consistent with the foundation mitigation recommendations for the new embankment construction (i.e. embankment preloading).

As discussed in MTO (2013) related to the High Fill H4 swamp crossing, based on an average coefficient of consolidation (c_v) of about 1.4×10^{-3} cm^2/s estimated for the cohesive deposit, the imposed loading conditions for the 3.6 m high embankment at the critical section, after removal of the organic deposits at the culvert location, and assuming two-way drainage of the 8.9 m thick cohesive deposit, it is estimated that about 90 per cent of the primary consolidation settlement will be completed in 3.8 years.

Due to the presence of the relatively thick cohesive deposit at this site, which will undergo relatively large settlement due to the new embankment loading, a temporary culvert should be installed concurrent with embankment construction and left in place during the embankment preload period. The temporary culvert would then be removed following the preload period and replaced with the permanent culvert.

For the temporary culvert constructed concurrent with embankment construction, settlements will vary along the culvert alignment and will be dependent on the length of time the temporary culvert is left in place. At this culvert site, we recommend that the preload embankment and hence the temporary culvert remain in place as long as possible to minimize the post-construction settlement of the permanent culvert. For the embankment located in High Fill H4, a minimum preload period of six months is recommended to induce sufficient settlement during the construction period and reduce the long-term settlement to meet MTO's post-construction settlement criterion of 200 mm. However, the permanent culvert would likely not tolerate post-construction settlement of 200 mm if it was installed following a six month preload, therefore, we recommend that a longer embankment preload period be extended at the culvert location. The estimated settlement of the temporary and permanent culverts for embankment preloading of either a nine or twelve month period is presented below. The permanent culvert should be designed to accommodate the post construction settlement applicable to the preload period implemented at the culvert location.

Culvert	Location	Total Estimated Settlement During*/Following Preload (mm)	
		After 9 Month Preload	After 12 Month Preload
Temporary (Installed concurrent with Embankment Preload Construction)*	Inlet (north)	Negligible	Negligible
	Near Midpoint	110	125
	Outlet (south)	15	15
Permanent (Installed following the Preload Period)	Inlet (north)	Negligible	Negligible
	Near Midpoint**	170	155
	Outlet (south)	10	10

Notes: * Includes immediate settlement of native cohesionless soils.
** Includes 50 mm of creep settlement.

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 preload embankment construction and during the preload period 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*

Depending on the structural requirements of the selected culvert type and the settlement tolerance, a 9 month preload period can be considered. However, we recommend a 12 month preload period prior to construction of the permanent culvert to provide the best long term performance of the culvert.

The settlement analysis indicates over the preload period of 9 months, the 3.6 m high preload embankment and the temporary culvert would undergo settlements between 0 mm (i.e. negligible) and 125 mm and that the total post-construction settlement of the foundation soils along the permanent culvert will be between 0 mm (i.e. negligible) and 170 mm. Therefore, the maximum post-construction horizontal strain along the 45 m long permanent culvert is estimated to be about 0.60 per cent of the culvert length (or about 270 mm).

For the 3.6 m high preload embankments removed after a period of 12 months, the settlement analysis indicates that during construction, a temporary culvert would undergo settlements between 0 mm (i.e. negligible) and 110 mm and that the total post-construction settlement of the foundation soils along the permanent culvert will between 0 mm (i.e. negligible) and 155 mm. Therefore, the maximum post-construction horizontal strain along the 45 m long permanent culvert is estimated to be about 0.55 per cent of the culvert length (or about 245 mm).

Based on the estimated horizontal strain for the permanent culvert, consideration should be given to a camber for the permanent culvert.

6.5 Geotechnical Resistance

If a 1 m wide permanent 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 75 kPa and a geotechnical reaction at Serviceability Limit States (SLS) of 35 kPa (for 25 mm of settlement) based on the culvert being founded on a properly prepared subgrade/granular bedding (as discussed in Section 6.7.1). 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 fills. 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 culverts. Where culverts are 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
Precast Concrete Box Culvert on Compacted Granular ‘A’	tan δ = 0.45
Cast-in-Place Concrete Box Culvert on Compacted Granular ‘A’	tan δ = 0.55

These values represent unfactored values.

6.6 Lateral Earth Pressures

If a box culvert is selected for this site for the permanent culvert, the lateral earth pressures acting on the walls of the culverts 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 305.4 m and 304.9 m at the inlet and outlet ends, respectively) and that the organic materials (i.e. topsoil) will be removed, the excavation will extend to at least 0.1 m below existing ground surface. As the organic deposit is relatively thin at the proposed culvert location, it is recommended that granular fill be used to backfill the excavation up to the underside of the culvert.

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 culvert should be in accordance with OPSD 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 percent 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. 1010Granular ‘A’ material or fine concrete aggregate (meeting the grading requirements specified in SP 110S11 (Aggregates - Concrete)) should be provided as shown on OPSD 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 percent 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/m^3 for Granular ‘A’ and 21 kN/m^3 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

If the culvert is placed on a granular blanket, provisions should be made for scour and erosion protection (suitable non-woven geotextiles and/or rip-rap) 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 culvert should be assessed by the hydraulics design engineer. As a minimum, rip-rap treatment for the outlet of the culvert should be consistent with OPSD 810.010 (Rip-Rap Treatment for Sewer and Culvert Outlets). Erosion protection for the inlet of the culverts should follow the standard presented in OPSD 810.010 similar to the outlet. Rip-rap should be provided over the full extent of the clay blanket, including the creek side slopes and fill slope over the culvert.

6.7.4 Control of Groundwater and Surface Water

Excavation within the plan limits of the proposed culvert alignments will be required to remove organic overburden prior to placement of backfill, bedding material and the actual culvert structures. As a result of the excavation, groundwater flow into the excavation can be expected to occur due to the relatively high water levels and potential for artesian conditions. 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, as required.

Given that the design invert is approximately at or above existing ground surface and that the excavation for removal of organic materials 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.

6.8 Temporary Culvert

A temporary culvert would be required to promote drainage across the embankment during the preload period. The temporary culvert may consist of precast concrete sections (box or pipe) or corrugated steel pipe (CSP). Bedding recommendations should be in accordance with the corresponding OPSS and/or OPSD depending on the type of the temporary culvert chosen. Assuming the temporary culvert is a CSP, construction of this culvert should be carried out in accordance with OPSD 802.010 (Flexible Pipe Embedment and Backfill, Earth Excavation).

The location of the temporary culvert could be offset from the final alignment of the permanent culvert, provided that surface drainage paths are adequate. It is recommended that the temporary culvert be constructed within a temporary granular core for ease of removal after the completion of the surcharge period. Due to the potential size of the temporary culvert, it is recommended that the temporary culvert be removed following the permanent culvert construction. If it is not desirable to remove the temporary culvert, consideration could be given to backfilling the temporary culvert with OPSS 1359 (Unshrinkable Fill) material.

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.

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Ontario Geological Survey. 1991. Geology of Ontario. Special Volume 4, Part 2. Eds. P.C. Thurston, H.R. Williams, R.H. Sutcliffe and G.M. Stott. Ministry of Northern Development and Mines, Ontario.

Rutledge, P.C. and Gould, J.P. 1973. Movements of Articulated Conduits Under Earth Dams on Compressible Foundations, In: Embankment Dam Engineering – Casagramde Volume. Eds. Hirschfeld, R.C. and Poulos, S.J. John Wiley & Sons, New York.

Terzaghi, K. and Peck, R.B., 1967. Soil Mechanics in Engineering Practice, 2nd Edition, John Wiley and Sons, New York.

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

ASTM International:

ASTM D1586	Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils
ASTM D1587	Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes
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)

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 1359	Material Specification for Unshrinkable Backfill
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

Ontario Water Resources Act:

Ontario Regulation 903 Wells (as amended)

MINISTRY OF TRANSPORTATION, ONTARIO

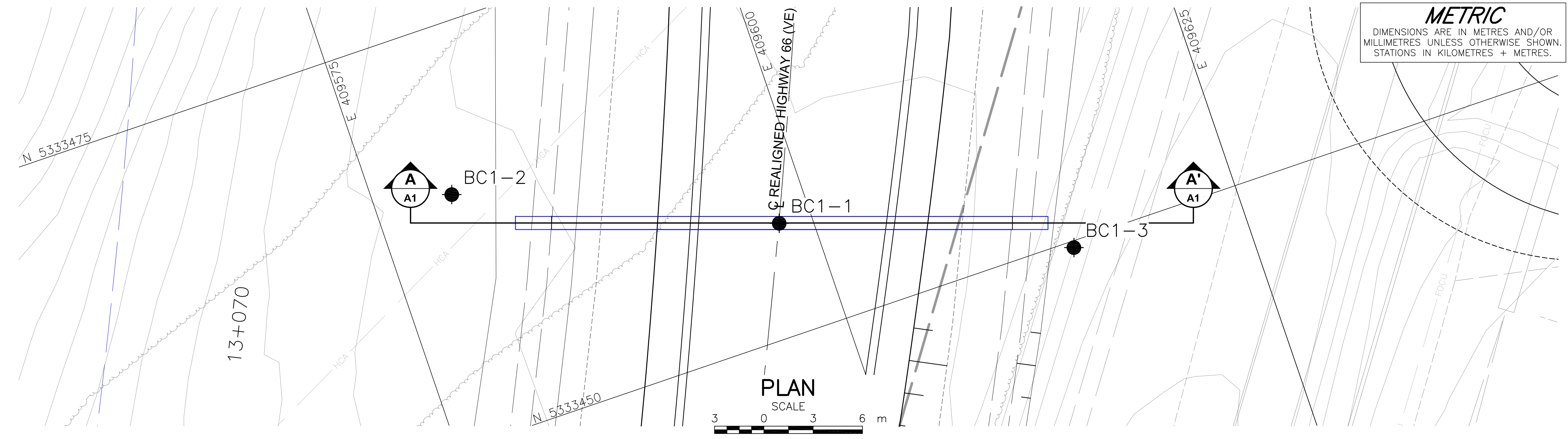
CONT No.
GWP No. 5091-07-00

SHEET

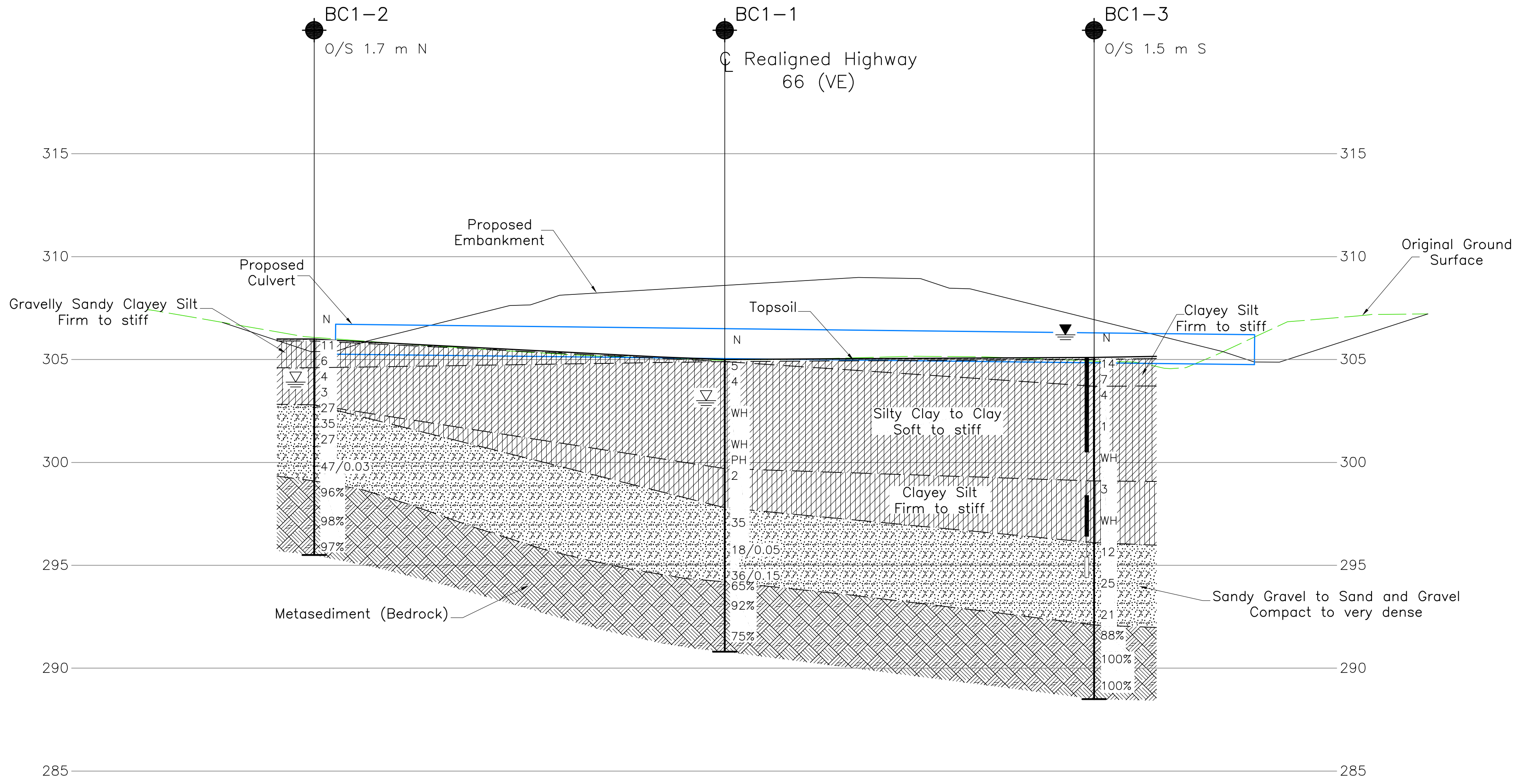
HIGHWAY 66
CULVERT AT STA 13+080
BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA

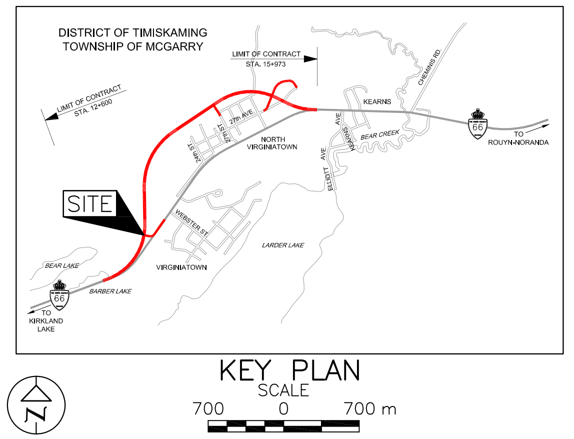
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.



PLAN
SCALE



A-A' A1 CULVERT AT STA 13+080
HIGHWAY 66
HORIZONTAL SCALE
3 0 3 6 m
3 0 3 6 m
VERTICAL SCALE



KEY PLAN
SCALE
700 0 700 m

LEGEND

●	Borehole
▮	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 19, 2013

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
BC1-1	305.0	5333456.8	409598.0
BC1-2	306.0	5333464.9	409579.7
BC1-3	305.1	5333449.6	409614.5

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.

S.E.M. COYNE
90471921
NOV 15, 2013

J.M.A.COSTA
NOV 15, 2013

NO.	DATE	BY	REVISION

Geocres No. 32D-13

HWY. 66		PROJECT NO. 10-1191-0044	DIST.
SUBM'D. MT	CHKD.	DATE: NOV 2013	SITE:
DRAWN: JJL	CHKD. SEMC	APPD. JMAC	DWG. 1

PLOT DATE: November 13, 2013
FILENAME: \\golder.gds\GAL\Sudbury\CAD-GIS\CAD\Projects\2010\10-1191-0044 Hwy 66 Virginiatown\Drawings\10119100044AA002 - 2.dwg

APPENDIX A

Highway 66 Realignment, Virginiatown— Culvert at STA 13+080 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
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_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 $= \sigma'_p / \sigma'_{vo}$

(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 multiplied by acceleration due to gravity)

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d:

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

PH:	Sampler advanced by hydraulic pressure
PM:	Sampler advanced by manual pressure
WH:	Sampler advanced by static weight of hammer
WR:	Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

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

(b) Cohesive Soils

Consistency

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

IV. SOIL TESTS

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

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

V. MINOR SOIL CONSTITUENTS

Per cent by Weight	Modifier	Example
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (non-cohesive (cohesionless)) or 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^{o} 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
FLT	Fault
SH	Shear
VN	Vein
FR	Fracture
SY	Stylolite
BD	Bedding
FO	Foliation
CO	Contact
AXJ	Axial Joint
KV	Karstic Void
MB	Mechanical Break
PL	Planar
CU	Curved
UN	Undulating
IR	Irregular
K	Slickensided
PO	Polished
SM	Smooth
SR	Slightly Rough
RO	Rough
VR	Very Rough

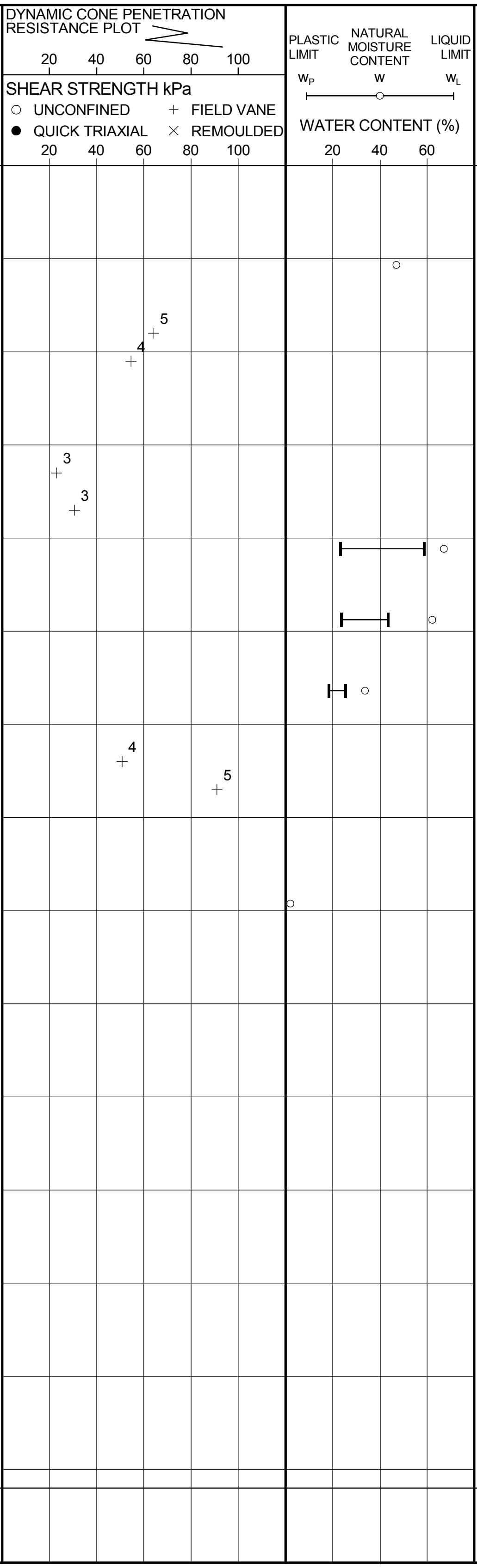
Golder Associates

RECORD OF BOREHOLE No BC1-1

1 OF 2 **METRIC**

PROJECT 10-1191-0044
G.W.P. 5091-07-00
DIST ____ HWY 66
DATUM GEODETIC
LOCATION N 5333456.8; E 409598.0
BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring
DATE September 8 and 9, 2012
ORIGINATED BY MT
COMPILED BY MT
CHECKED BY SEMC

SOIL PROFILE			SAMPLES									
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
305.0	GROUND SURFACE											
0.0	TOPSOIL											
0.1	SILTY CLAY to CLAY Firm to stiff Grey Moist to wet		1	SS	5							
			2	SS	4		304					
						▽	303					
			3	SS	WH		302					
	Silt seams encountered between 0.1 m and 5.3 m depth.		4	SS	WH		301		0	0	39	61
	Roots and silt pockets encountered at 5.2 m depth in Shelby tube Sample 5.		5	TO	PH		300		0	0	47	53
299.7 5.3	CLAYEY SILT Stiff Grey Wet		6	SS	2		299					
297.8 7.2	Sandy GRAVEL Dense to very dense Grey Wet						298					
			7	SS	35		297		78	22	0	0
			8	SS	18/0.05		296					
	Cobbles encountered between 10.1 m and 10.3 m depth.		9	SS	36/0.15		295					
294.2 10.8	METASEDIMENT (BEDROCK)		1	RC	REC 100%		294		RQD = 65%			
	Bedrock cored from 10.8 m depth to 14.2 m depth. For coring details see Record of Drillhole BC1-1.		2	RC	REC 100%		293		RQD = 92%			
			3	RC	REC 98%		292 291		RQD = 75%			
290.8 14.2												



Continued Next Page

+ 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:

RECORD OF BOREHOLE No BC1-1

2 OF 2 **METRIC**

PROJECT 10-1191-0044			
G.W.P. 5091-07-00	LOCATION N 5333456.8; E 409598.0	ORIGINATED BY MT
DIST	HWY 66	BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring	COMPILED BY MT
DATUM GEODETIC		DATE September 8 and 9, 2012	CHECKED BY SEMC

SOIL PROFILE			SAMPLES								
ELEV DEPTH	DESCRIPTION *--- CONTINUED FROM PREVIOUS PAGE ---*	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	END OF BOREHOLE Note: 1. Water level at a depth of 2.0 m below ground surface (Elev. 303.0 m) upon completion of drilling.										

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

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

PROJECT: 10-1191-0044

RECORD OF DRILLHOLE: BC1-1

SHEET 1 OF 1

LOCATION: N 5333456.8 ;E 409598.0
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: September 9, 2012
DRILL RIG: CME 55
DRILLING CONTRACTOR: Landcore Drilling Inc.

DATUM: GEODETIC

JN - Joint
FLT - Fault
SHR- Shear
VN - Vein
CJ - Conjugate

BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Cleavage

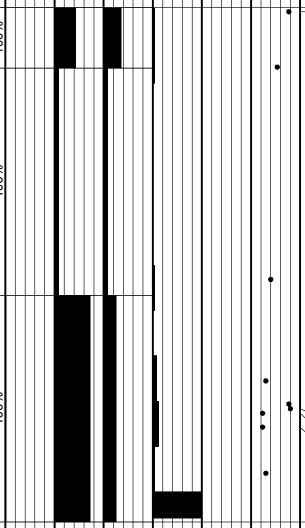
PL - Planar
CU- Curved
UN- Undulating
ST - Stepped
IR - Irregular

PO- Polished
K - Slickensided
SM- Smooth
Ro - Rough
MB- Mechanical Break

BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN (FLUSH)	NOTES WATER LEVELS INSTRUMENTATION
		REFER TO PREVIOUS PAGE				
11	September 9, 2012 NQ Coring NW	METASEDIMENT Very strong Fine grained Moderately weathered to fresh Greenish grey	294.2 10.8	1	WHITE 100%	
12				2	GREY / WHITE 100%	112 MPa
13		Sheared zone encountered between 12.7 m and 14.2 m depth.		3	GREY / WHITE 100%	
14			290.8			
		END OF DRILLHOLE	14.2			



DISCONTINUITY DATA – TYPE AND SURFACE DESCRIPTION (Jr, Ja, Jn; HYDRAULIC CONDUCTIVITY k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3}; Diametral Point Load Index (MPa) 2 4 6; RMC -Q' AVG.):

- JN,IR,Ro
- JN,IR,Ro
- JN,IR,Ro
- JN,IR,Ro
- JN,IR,Ro
- JN,IR,Ro
- JN,IR,Ro
- JN,IR,Ro
- JN,IR,Ro
- BR

15
16
17
18
19
20

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: MT
CHECKED: SEMC

RECORD OF BOREHOLE No BC1-2

1 OF 1 **METRIC**

PROJECT	10-1191-0044		
G.W.P.	5091-07-00	LOCATION	N 5333464.9; E 409579.7
DIST		HWY	66
BOREHOLE TYPE	108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring		
DATUM	GEODETIC	DATE	September 9 and 10, 2012
ORIGINATED BY	MT	COMPILED BY	MT
CHECKED BY	SEMC		

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
306.0	GROUND SURFACE						
0.0	TOPSOIL	1a	SS	11			
0.1	Gravelly Sandy CLAYEY SILT Firm to stiff Brown Moist	1b					24 26 33 17
		2	SS	6		305	
304.6 1.4	SILTY CLAY, trace gravel, trace sand Soft Brown Moist	3	SS	4	∇	304	
	Approximately 25 mm thick sand seam encountered at 2.1 m depth.	4	SS	3		303	
302.8 3.2	Sandy GRAVEL, trace silt, trace clay Compact to dense Grey to brown Wet	5a 5b	SS	27			4 3 64 29
		6	SS	35		302	
	Approximately 0.4 m thick sand seam encountered at 4.6 m depth.	7a 7b	SS	27		301	11 80 7 2
		8	SS	47/0.03		300	
299.1	Cobbles encountered between 6.3 m and 6.9 m depth.						
6.9	METASEDIMENT (BEDROCK) Bedrock cored from 6.9 m depth to 10.5 m depth. For coring details see Record of Drillhole BC1-2.	1	RC	REC 100%		299 298	RQD = 96%
		2	RC	REC 100%		297	RQD = 98%
		3	RC	REC 100%		296	RQD = 97%
295.5 10.5	END OF BOREHOLE						

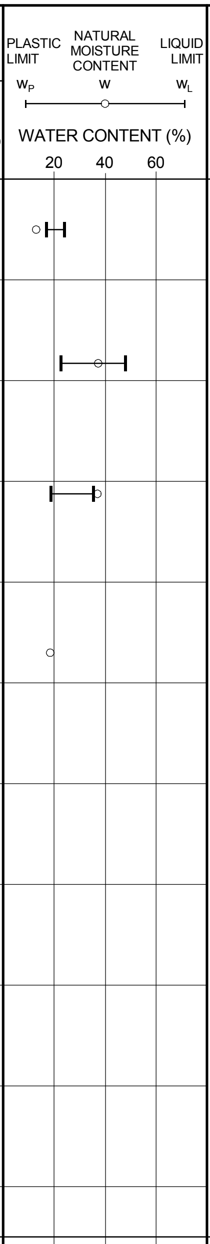
Note:

1. Water level at a depth of 2.1 m below ground surface (Elev. 303.9 m) upon completion of drilling.

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100

UNIT WEIGHT γ kN/m³



+³, ×³: Numbers refer to Sensitivity

○³% STRAIN AT FAILURE

PROJECT: 10-1191-0044

RECORD OF DRILLHOLE: BC1-2

SHEET 1 OF 1

LOCATION: N 5333464.9 ;E 409579.7

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: September 10, 2012

DRILL RIG: CME 55

DRILLING CONTRACTOR: Landcore Drilling Inc.

DATUM: GEODETIC

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

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	COLOUR % RETURN	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		REFER TO PREVIOUS PAGE	299.1					
7	NW Casing	METASEDIMENT Very strong Fine grained Fresh Grey	6.9	1	GREY 100%		VN,PL,RO	
8	NQ Coring September 10, 2012			2	GREY / WHITE 100%		JN,IR,Ro	
9							JN,IR,Ro	114 MPa
10			295.5	3	GREY 100%			
		END OF DRILLHOLE	10.5					
11								
12								
13								
14								
15								
16								

Column headings: SYMBOLIC LOG; FLUSH; RECOVERY: TOTAL CORE % (80 60 40 20), SOLID CORE % (80 60 40 20); R.Q.D. % (80 60 40 20); FRACT. INDEX METRES (5 10 15 20); DISCONTINUITY DATA: B Angle (0 90 180 270), DIP w.r.t. CORE AXIS (0 30 60 90), TYPE AND SURFACE DESCRIPTION, Jr, Ja, Jn; HYDRAULIC CONDUCTIVITY k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}); Diametral Point Load Index (MPa) (2 4 6); RMC -Q' AVG.

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: MT
CHECKED: SEMC

RECORD OF BOREHOLE No BC1-3

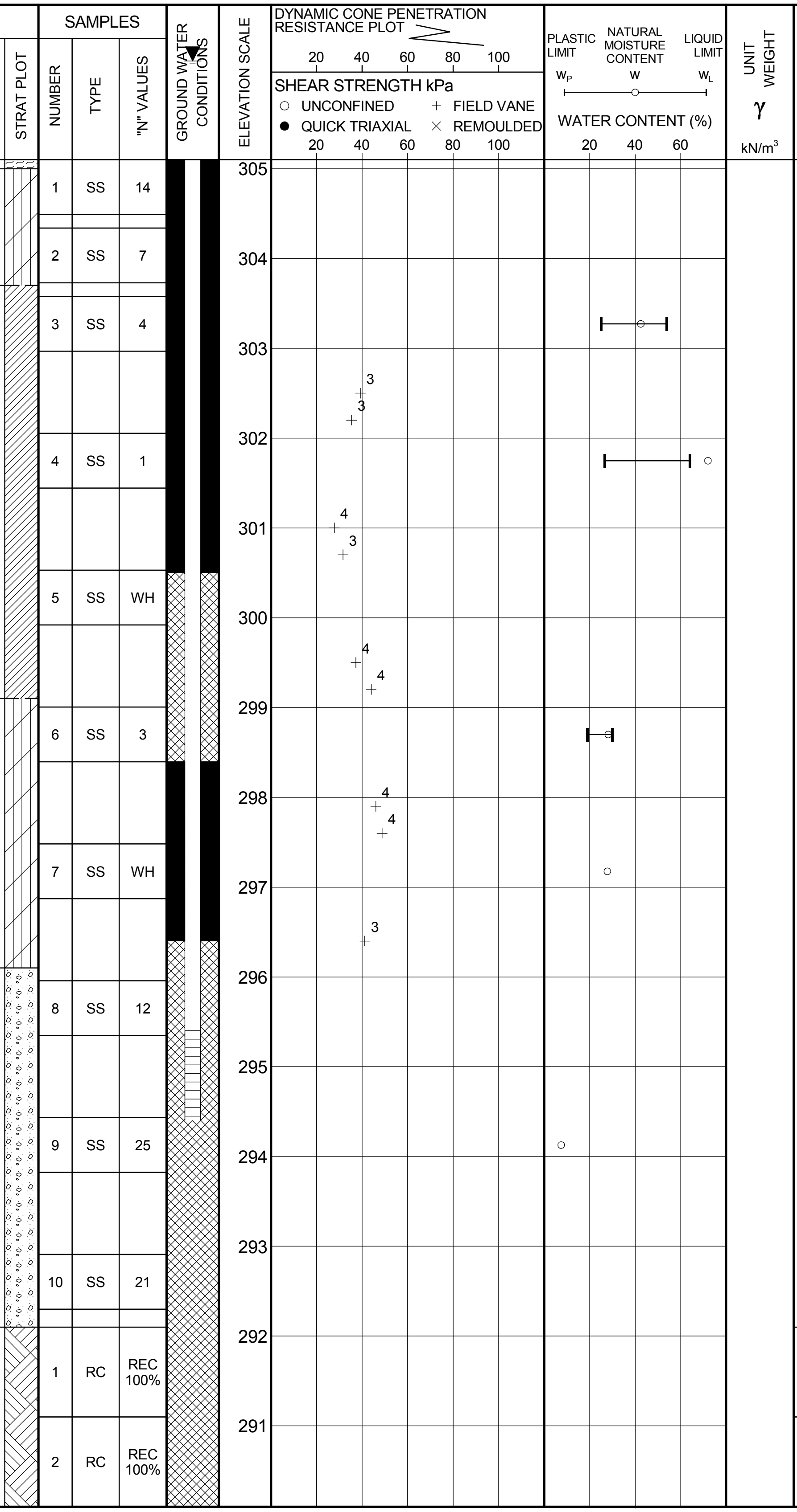
1 OF 2 **METRIC**

PROJECT 10-1191-0044
G.W.P. 5091-07-00
DIST ____ HWY 66
DATUM GEODETIC

LOCATION N 5333449.6; E 409614.5
BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring
DATE September 10, 2012

ORIGINATED BY MT
COMPILED BY MT
CHECKED BY SEMC

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
305.1	GROUND SURFACE				
0.0 0.1	TOPSOIL				
	CLAYEY SILT, some sand, trace organics Firm to stiff Brown Moist	1	SS	14	
		2	SS	7	
303.7 1.4	CLAY Firm Brown to grey Moist to wet	3	SS	4	0 1 40 59
		4	SS	1	
		5	SS	WH	
	Silt seams encountered between 4.6 m and 6.0 m depth.				
299.1 6.0	CLAYEY SILT Firm Grey Wet	6	SS	3	
	Silt seams encountered between 6.0 m and 7.5 m depth.				
		7	SS	WH	0 2 82 16
296.1 9.0	SAND and GRAVEL, some silt, trace clay Compact Grey Wet	8	SS	12	
		9	SS	25	36 43 17 4
		10	SS	21	
292.1 13.0	METASEDIMENT (BEDROCK) Bedrock cored from 13.0 m depth to 16.6 m depth. For coring details see Record of Drillhole BC1-3.	1	RC	REC 100%	RQD = 88%
		2	RC	REC 100%	RQD = 100%



Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

Golder Associates

PROJECT 10-1191-0044	**RECORD OF BOREHOLE No BC1-3**	2 OF 2	**METRIC**
G.W.P. 5091-07-00	LOCATION N 5333449.6; E 409614.5		ORIGINATED BY MT
DIST HWY 66	BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring		COMPILED BY MT
DATUM GEODETIC	DATE September 10, 2012		CHECKED BY SEMC

SOIL PROFILE			SAMPLES								
ELEV DEPTH	DESCRIPTION --- *CONTINUED FROM PREVIOUS PAGE* ---	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	METASEDIMENT (BEDROCK) Bedrock cored from 13.0 m depth to 16.6 m depth. For coring details see Record of Drillhole BC1-3.		2	RC			290				RQD = 100%
			3	RC	REC 100%		289				RQD = 100%
288.5 16.6	END OF BOREHOLE Note: 1. Water level at a depth of 1.0 m below ground surface (Elev. 304.1 m) upon completion of drilling. 2. Water level in piezometer measured at 1.0 m above ground surface (Elev. 306.1 m) on November 17, 2012 and at 1.1 m above ground surface (Elev. 306.2 m) on May 19, 2013. 3. Piezometer installed within heaving sand and gravel.										

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

PROJECT: 10-1191-0044

RECORD OF DRILLHOLE: BC1-3

SHEET 1 OF 1

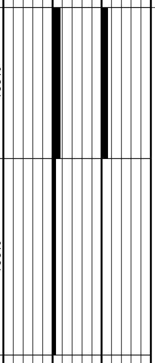
LOCATION: N 5333449.6 ;E 409614.5
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: September 10, 2012
DRILL RIG: CME 55
DRILLING CONTRACTOR: Landcore Drilling Inc.

DATUM: GEODETIC

JN - Joint	BD - Bedding	PL - Planar	PO - Polished	BR - Broken Rock
FLT - Fault	FO - Foliation	CU - Curved	K - Slickensided	**NOTE:** For additional abbreviations refer to list of abbreviations & symbols.
SHR - Shear	CO - Contact	UN - Undulating	SM - Smooth	
VN - Vein	OR - Orthogonal	ST - Stepped	Ro - Rough	
CJ - Conjugate	CL - Cleavage	IR - Irregular	MB - Mechanical Break	

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH	COLOUR % RETURN	RECOVERY TOTAL CORE % 80 60 40 20	RECOVERY SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT. INDEX METRES 5 10 15 20	DISCONTINUITY DATA B Angle 0 90 180 270	DISCONTINUITY DATA DIP w.r.t. CORE AXIS 0 30 60 90	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	HYDRAULIC CONDUCTIVITY k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3}	Diametral Point Load Index (MPa) 2 4 6	RMC -Q' AVG.	NOTES WATER LEVELS INSTRUMENTATION
		REFER TO PREVIOUS PAGE		292.1																	
13	NQ Coring September 10, 2012	METASEDIMENT Strong Fine grained Fresh Greenish Grey Sheared zone between 13.0 m and 16.6 m depth.		13.0	1		GREY 100%														
14					2		GREY 100%														62 MPa
15																					
16				288.5	3		GREY 100%														
		END OF DRILLHOLE		16.6																	
17																					
18																					
19																					
20																					
21																					
22																					
23																					

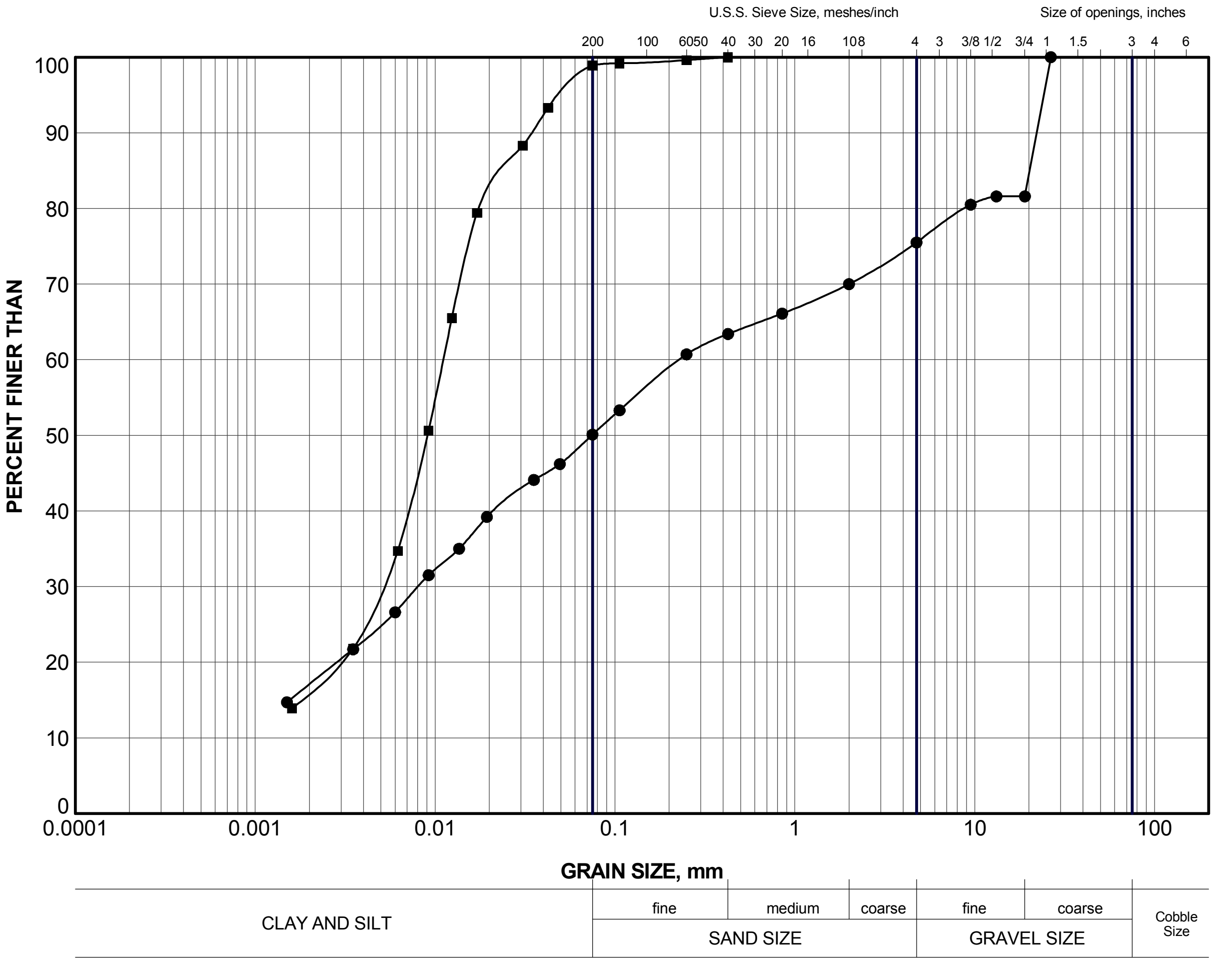


DEPTH SCALE
1 : 50

LOGGED: MT
CHECKED: SEMC

APPENDIX B

Highway 66 Realignment, Virginiatown — Culvert at STA 13+080 Laboratory Tests Results



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BC1-2	1b	305.7
■	BC1-3	7	297.2

PROJECT

HIGHWAY 66 - CULVERT BC1
STA 13+080

TITLE

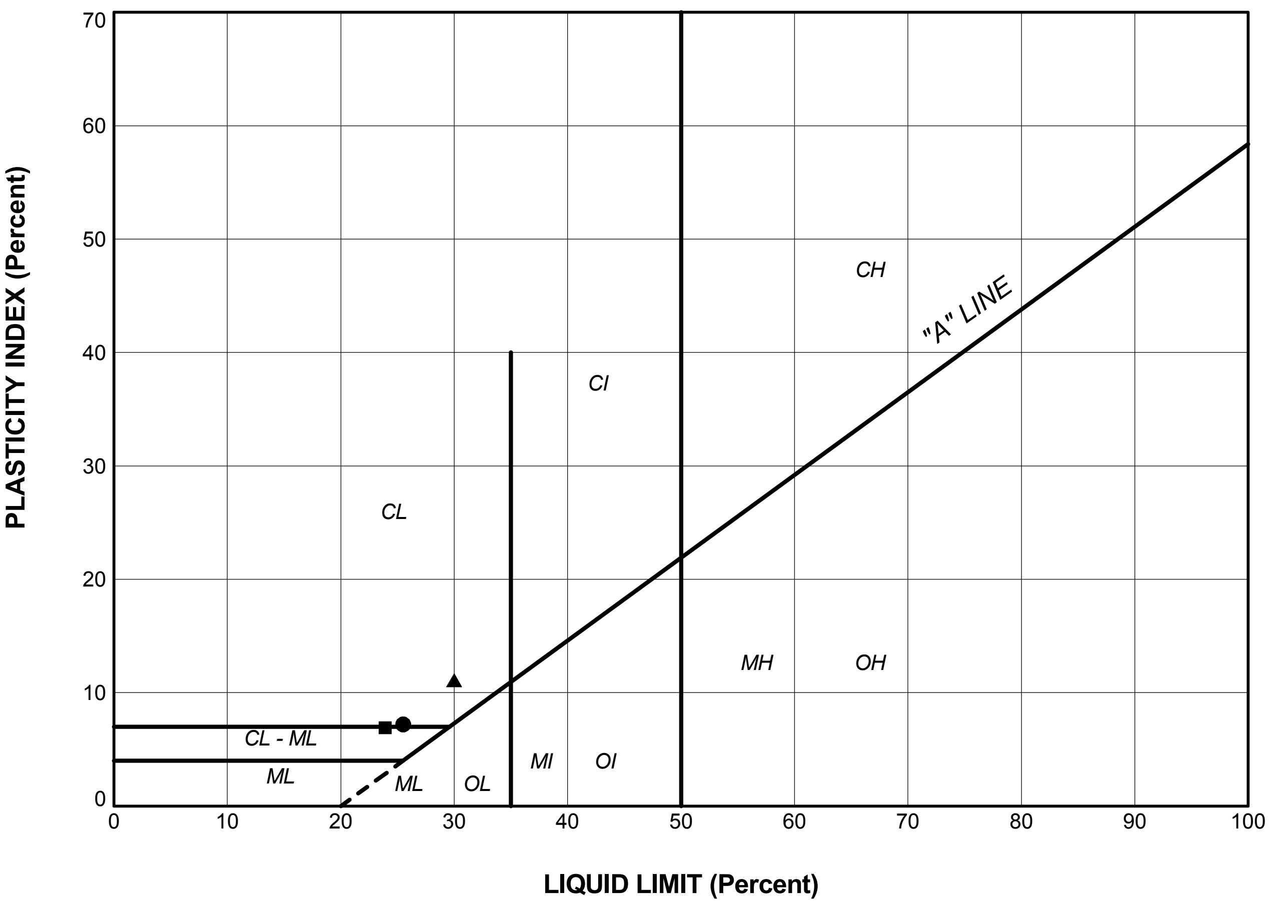
GRAIN SIZE DISTRIBUTION

CLAYEY SILT

Golder Associates
SUDBURY, ONTARIO

PROJECT No.	10-1191-0044		FILE No.	10-1191-0044C.GPJ
DRAWN	JJL	May 2013	SCALE N/A	REV.
CHECK	SEMC	May 2013	**FIGURE B1**	
APPR		May 2013		

SUD-MTO GSD GLDR_LDN.GDT



SOIL TYPE	**PLASTICITY**
C = Clay	L = Low
M = Silt	I = Intermediate
O = Organic	H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	BC1-1	6	25.5	18.3	7.2
■	BC1-2	1b	23.9	17.0	6.9
▲	BC1-3	6	30.0	18.9	11.1

PROJECT

HIGHWAY 66
STA 13+080

TITLE

PLASTICITY CHART

CLAYEY SILT

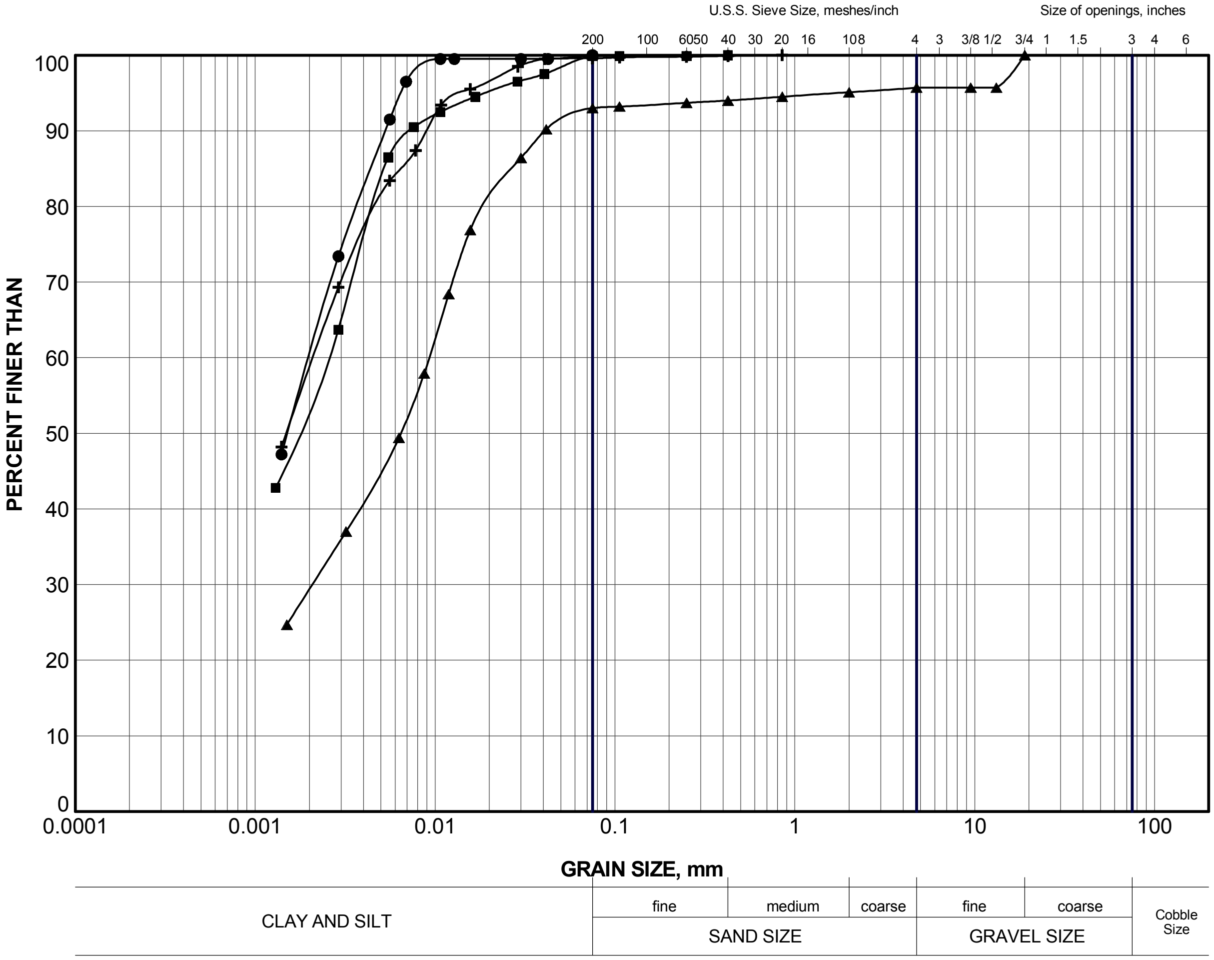
Golder Associates
SUDBURY, ONTARIO

PROJECT No.	10-1191-0044	
DRAWN	JJL	May 2013
CHECK	SEMC	May 2013
APPR	JMAC	May 2013

FILE No.	10-1191-0044C.GPJ	
SCALE	N/A	REV.

FIGURE B2

SUD-MTO PL (NEW) GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BC1-1	4	300.9
■	BC1-1	5	300.1
▲	BC1-2	5a	302.9
+	BC1-3	3	303.3

PROJECT

HIGHWAY 66 - CULVERT BC1
STA 13+080

TITLE

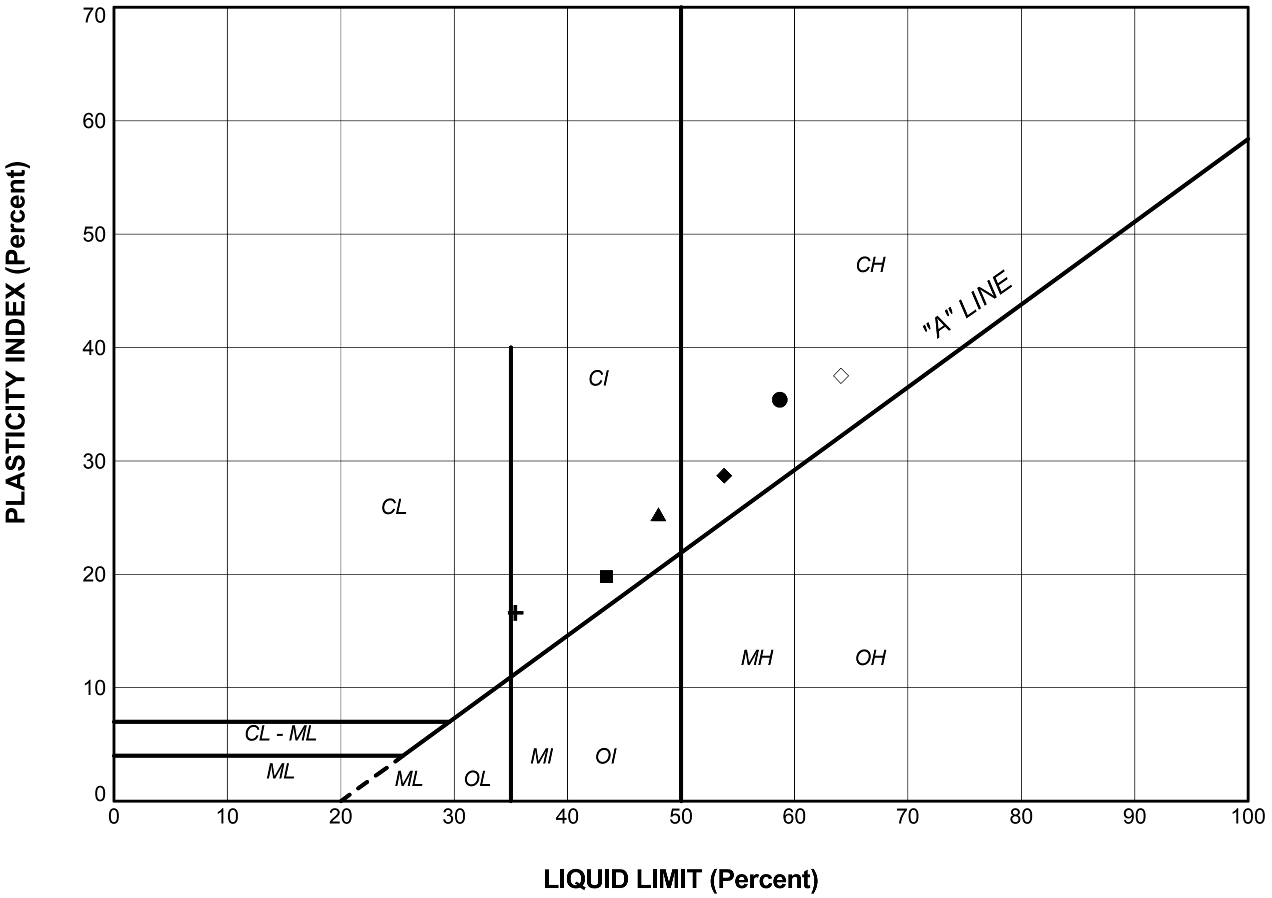
GRAIN SIZE DISTRIBUTION

SILTY CLAY to CLAY

PROJECT No.	10-1191-0044	FILE No.	10-1191-0044C.GPJ
DRAWN JJL	May 2013	SCALE N/A	REV.
CHECK SEMC	May 2013		
APPR	May 2013	**FIGURE B3**	

Golder Associates
SUDBURY, ONTARIO

SUD-MTO GSD GLDR_LDN.GDT



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	BC1-1	4	58.7	23.3	35.4
■	BC1-1	5	43.4	23.6	19.8
▲	BC1-2	3	48.0	22.7	25.3
+	BC1-2	5a	35.4	18.8	16.6
◆	BC1-3	3	53.8	25.1	28.7
◇	BC1-3	4	64.1	26.6	37.5

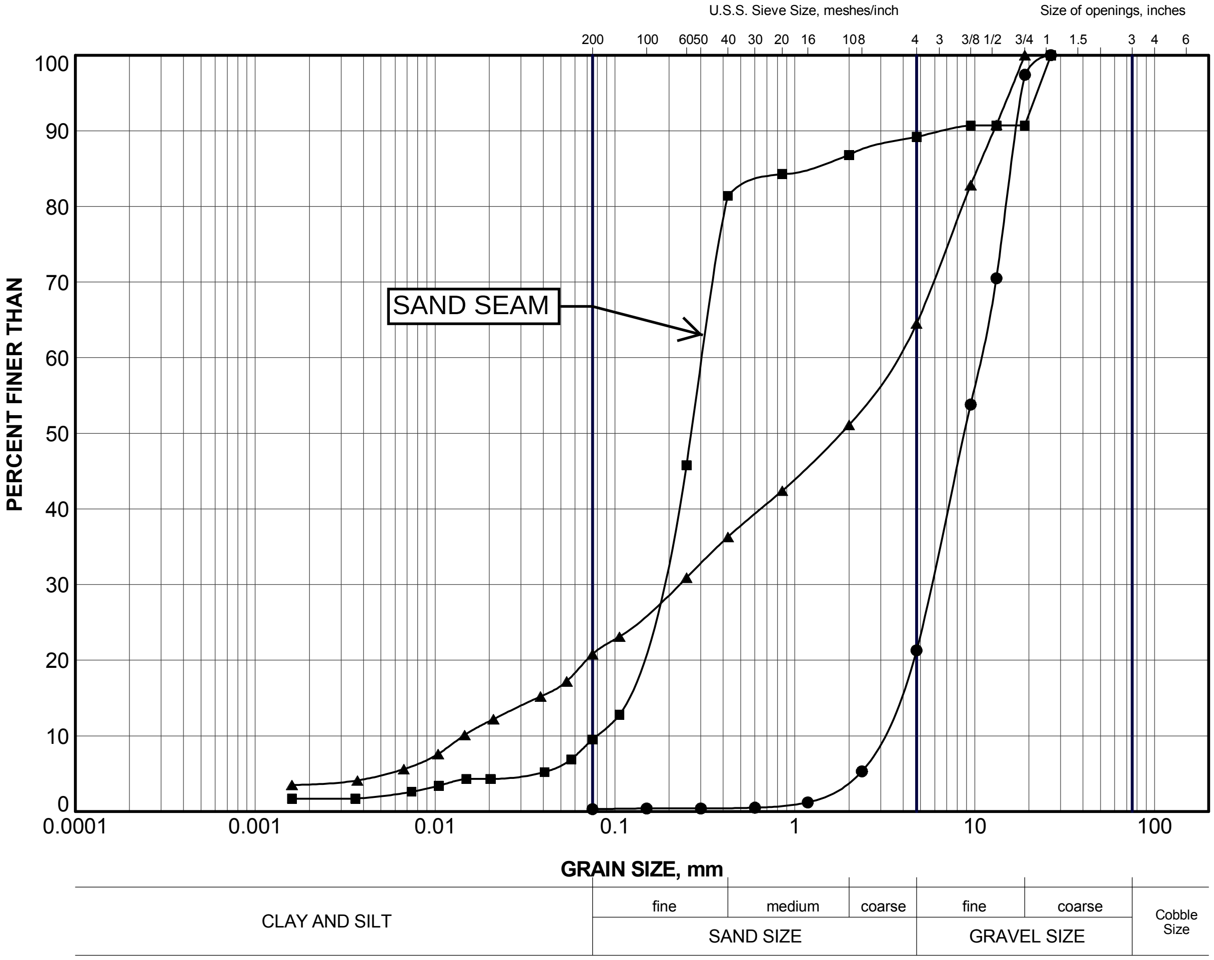
PROJECT
HIGHWAY 66 - CULVERT BC1
STA 13+080

TITLE
PLASTICITY CHART
SILTY CLAY to CLAY

Golder Associates
SUDBURY, ONTARIO

PROJECT No.	10-1191-0044		FILE No.	10-1191-0044C.GPJ	
DRAWN	JJL	May 2013	SCALE	N/A	REV.
CHECK	SEMC	May 2013	**FIGURE B4**		
APPR		May 2013			

SUD-MTO PL_GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BC1-1	7	297.1
■	BC1-2	7	301.2
▲	BC1-3	9	294.1

PROJECT

HIGHWAY 66
STA 13+080

TITLE

GRAIN SIZE DISTRIBUTION

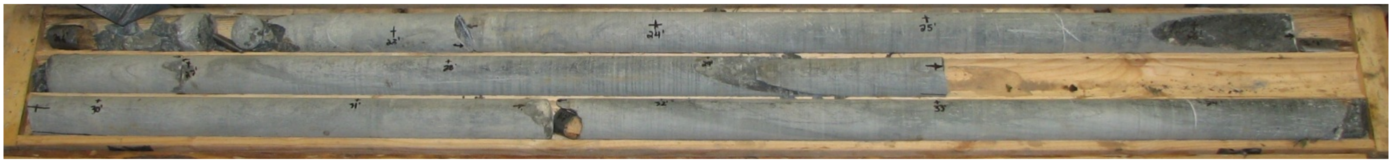
SANDY GRAVEL to SAND AND GRAVEL

Golder Associates
SUDBURY, ONTARIO

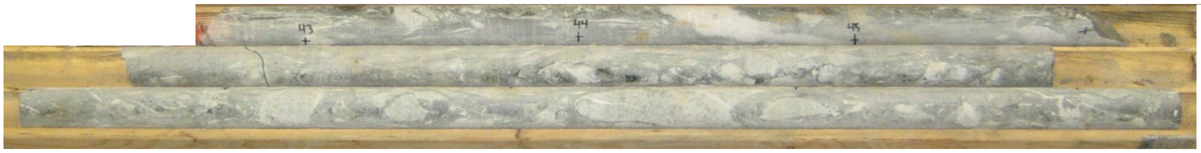
PROJECT No.		10-1191-0044	FILE No.	10-1191-0044C.GPJ
DRAWN	JJL	May 2013	SCALE N/A	REV.
CHECK	SEMC	May 2013		
APPR	JMAC	May 2013	FIGURE B5	

SUD-MTO GSD (NEW) GLDR_LDN.GDT

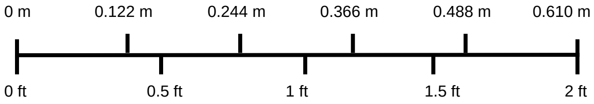
Borehole BC1-1
Elevation 294.2 m to 290.8 m



Borehole BC1-2
Elevation 299.1 m to 295.5 m



Borehole BC1-3
Elevation 292.1 m to 288.5 m



PROJECT
Highway 66 – Culvert BC1
STA 13+080

TITLE
BEDROCK CORE PHOTOGRAPHS

Golder Associates

PROJECT No. 10-1191-0044			FILE No. ----
DESIGN	MT	Apr 2013	SCALE AS SHOWN REV.
CADD	--		
CHECK	SEMC	Apr 2013	**FIGURE B6**
REVIEW	JMAC	Apr 2013	

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

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