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FOUNDATION INVESTIGATION AND DESIGN REPORT

Culvert C02 Replacement, Station 20+825
Highway 401, Cornwall, Ontario
GWP 4029-08-00
Ministry of Transportation, Ontario - Eastern Region

Submitted to:

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REPORT



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Table of Contents

PART A - FOUNDATION INVESTIGATION REPORT

1.0 INTRODUCTION.....	1
2.0 SITE DESCRIPTION.....	1
2.1 Site Geology	1
3.0 INVESTIGATION PROCEDURES	2
4.0 SUBSURFACE CONDITIONS.....	3
4.1 Site Stratigraphy	3
4.1.1 Fill	3
4.1.2 Topsoil	4
4.1.3 Silty Clay	4
4.1.4 Silty Sand to Sand and Gravel	4
4.1.5 Silty Sand to Sandy Silt Till	5
4.1.6 Inferred Bedrock Surface	5
4.1.7 Groundwater Conditions	5
5.0 CLOSURE.....	6

PART B - FOUNDATION DESIGN REPORT

6.0 ENGINEERING RECOMMENDATIONS.....	7
6.1 General.....	7
6.2 Options for Culvert Replacement.....	7
6.3 Culvert Foundations.....	8
6.3.1 Pipe Culvert	8
6.3.2 Precast Concrete Box Culvert.....	10
6.3.3 Cast-in-Place Open Footing Culvert	11
6.3.4 Settlement and Camber	13
6.3.5 Backfill and Cover	13
6.4 End Treatments, Filters and Scour Protection	13
6.5 Liquefaction Potential and Seismic Analysis.....	14
6.5.1 Seismic Parameters	14
6.5.2 Seismic Hazard Assessment	14



FOUNDATION INVESTIGATION AND DESIGN REPORT CULVERT C02 REPLACEMENT, STATION 20+825

6.6	Lateral Earth Pressures for Design.....	15
6.7	Construction Considerations.....	17
6.7.1	Excavation and Temporary Cut Slopes.....	17
6.7.2	Groundwater and Surface Water Control.....	17
6.7.3	Subgrade Preparation and Protection.....	18
6.7.4	Temporary Protection Systems.....	18
7.0	CLOSURE.....	19

TABLES

Table I Comparison of Culvert Replacement Alternatives

Lists of Abbreviations and Symbols

Record of Borehole Sheets

FIGURES

Figure 1 Key Plan

DRAWINGS

Drawing 1 Borehole Locations and Soil Strata

APPENDICES

APPENDIX A

Laboratory Test Data – Soils

APPENDIX B

Site Photographs

APPENDIX C

Non-Standard Special Provisions



**FOUNDATION INVESTIGATION AND DESIGN REPORT
CULVERT C02 REPLACEMENT, STATION 20+825**

PART A

FOUNDATION INVESTIGATION REPORT

CULVERT C02 REPLACEMENT, STATION 20+825
HIGHWAY 401, CORNWALL, ONTARIO
GWP 4029-08-00
MINISTRY OF TRANSPORTATION, ONTARIO - EASTERN REGION



1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by McCormick Rankin Corporation (MRC) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations as part of the detail design work for GWP 4029-08-00. The main project involves the detail design for the replacement of twin Highway 401 Overpass structures at Cornwall Centre Road within the City of Cornwall, Ontario. This report presents the results of the foundation investigation conducted for the replacement of Culvert C02 located east of the existing Cornwall Centre Road overpass structures at approximately Station 20+825.

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

2.0 SITE DESCRIPTION

The site is situated in the north-central part of the City of Cornwall, Ontario between the South Raisin River and Cornwall Township Road 31. The culvert is located at Station 20+825, approximately 175 m east of the Cornwall Centre Road overpass structures. The location of the project is shown on the Key Plan (Figure 1). Photographs of the area are presented in Appendix B.

The existing Culvert C02 is a corrugated steel pipe (CSP) 1200 mm in diameter and is generally in fair condition according to the July 2011 inspection report prepared by MRC.¹ It will be replaced as it is near the end of its service life. This culvert conveys ditch flows from north to south below Highway 401.

The land adjacent to the site is relatively flat and is at approximately Elevation 60 m. The Highway 401 embankment is approximately 5 m high in this area with the pavement grade near approximately Elevation 65 m. Land use in the area is predominantly agricultural and rural residential.

2.1 Site Geology

This project is located in a physiographic region known as the Lancaster Flats. It is a flat, poorly drained, lowland area where the till plain has been largely buried under water-lain deposits. Only the stony crests of a few drumlins and till ridges are exposed. The water-lain materials range in composition from clay to very fine sand. The nearby South Raisin River drains into the St. Lawrence River.²

The overburden consists of unconsolidated deposits mainly of Pleistocene age. The dominant consolidated deposit is glacial till composed of silt, clay and sand with pebbles and boulders included. In low-lying areas of the till surface, marine clay and silt has been deposited by the Champlain Sea during the late Pleistocene and

¹ McCormick Rankin Corporation, 2011: Culvert Inspection Report, Replacement of Centre Road Overpass, Highway 401 Detail Design, W.P. 4029-08-00.

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



FOUNDATION INVESTIGATION AND DESIGN REPORT CULVERT C02 REPLACEMENT, STATION 20+825

early Holocene age. These clayey deposits are generally less than 15 m thick but may reach 100 m in thickness in localized areas.³

The overburden thickness is approximately 11.5 m to 13.5 m in the vicinity of the site.⁴ The available bedrock topography mapping indicates that the elevation of the bedrock surface ranges from approximately 48 m to 50 m.⁵ The overburden is underlain by calcareous limestone of the Bobcaygeon Formation. Two quarries and an area of bedrock outcrops were mapped 3.5 km to 4 km west of the site.⁶

3.0 INVESTIGATION PROCEDURES

The field work for this project was conducted between November 4 and 11, 2011 during which time five boreholes were advanced at the locations shown on the Borehole Location Plan (Drawing 1). The table below summarizes the borehole locations, ground surface elevations at the borehole locations and borehole depths.

Borehole No.	Location (m)		Ground Surface Elevation	Borehole Depth
	Northing	Easting	(m)	(m)
11-14	4 991 093	203 560	59.9	7.9
11-14A	4 991 093	203 561	59.9	4.4
11-15	4 991 120	203 545	65.0	11.6
11-16	4 991 138	203 536	65.3	12.6
11-17	4 991 164	203 526	60.3	8.1

The soil stratigraphy encountered in the boreholes is shown on the attached borehole records. The investigation was carried out using a track-mounted CME 75 drill rig supplied and operated by a specialist drilling contractor. The underlying founding soils were sampled using 50 mm diameter split-spoon sampling equipment in accordance with the Standard Penetration Test (SPT) procedures of ASTM D 1586. For the investigation, samples of overburden were generally obtained at 0.75 m and 1.5 m depth intervals. In situ shear vane testing was carried out in the softer cohesive deposits to measure the undrained shear strength.

The groundwater conditions in the boreholes were observed throughout the drilling operations and standpipe piezometers were installed in Boreholes 11-14A and 11-17. The boreholes were backfilled in accordance with current MTO procedures and Ontario Regulation 903 (as amended).

The field work was supervised on a full-time basis by a member of Golder's staff who directed the drilling, sampling and in situ testing operations and logged the boreholes. The ground surface elevations and borehole

³ Brismead, R.A. 1975: Lower St. Lawrence Planning Area: Finch, Roxborough, Osnaburck, Cornwall and Charlottenburg Townships. Ontario Division of Mines Geological Branch, Open File Report 5138.

⁴ Gwyn, Q.H.J., Fraser, J.Z. and Owen, N., 1975: Drift Thickness of the Cornwall-Huntingdon Area, Southern Ontario; Ontario Division of Mines, Preliminary Map P.1073, Drift Thickness Series, Scale 1:50, 000. Geology and Compilation 1974.

⁵ Gwyn et al, 1975: Bedrock Topography of the Cornwall-Huntingdon Area, Southern Ontario; Ontario Division of Mines, Preliminary Map P.1012, Bedrock Topography Series, Scale 1:50, 000. Geology and Compilation, 1974.

⁶ Williams, D.A., Wolf, R.R. and Carson, D.M., 1985: Paleozoic Geology of the Cornwall-Huntingdon Area, Southern Ontario; Ontario Geological Survey, Map P.2720, Geological Series – Preliminary Map, Scale 1: 50 000. Geology 1981-1982.



locations were also measured by members of Golder's staff. The samples were identified in the field, placed in labelled containers and transported to Golder's Ottawa laboratory for further examination and routine classification testing. Index and classification tests, consisting of water content determinations, grain size distribution analyses and Atterberg limits determinations, were carried out on selected samples. Oedometer testing was carried out in Golder's Mississauga laboratory.

4.0 SUBSURFACE CONDITIONS

4.1 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the boreholes, together with the results of the in situ testing and the laboratory testing carried out on selected samples, are given on the borehole records following the text of this report and in Appendix A. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous samples and observations of drilling resistance and represent transitions between soil types rather than exact planes of geological change. Further, the subsurface conditions will vary between and beyond the borehole locations.

The borehole locations are shown on the attached Drawing 1 and this drawing also provides an interpreted stratigraphic profile at the site based on the subsurface conditions encountered in the boreholes. The boreholes encountered granular fill in the highway embankment area and thin layers of cohesive fill beyond the toes of the existing embankment. The fill materials are underlain by layers of silty clay, sand and silty sand to sandy silt till. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.1.1 Fill

Fill was found immediately below the ground surface in all of the boreholes. Layers of silty clay fill 0.3 m to 0.4 m thick were encountered at the surface of Boreholes 11-14, 11-14A and 11-17 which were advanced beyond the toes of the Highway 401 embankment. The cohesive fill contained organics.

Boreholes 11-15 and 11-16 were advanced through the embankment fill from the median shoulders of Highway 401. In both of the boreholes, 0.3 m thick layers of granular base fill were encountered at the surface. The granular base fill in Boreholes 11-15 and 11-16 is underlain by 0.4 m and 0.9 m, respectively, of granular subbase fill. The granular fill in both boreholes is underlain by embankment fill below Elevation 64.3 m and 64.0 m; this fill consists of sand and gravel to silty sand and gravel with cobbles. The embankment fill layers were 4.8 m and 4.2 m thick in Boreholes 11-15 and 11-16, respectively.

The granular fill is compact to very dense with SPT N values of 11 to 194 blows per 0.3 m of penetration. The granular fill had measured water contents of 6 to 9 per cent. The gradation of two samples of fill from Boreholes 11-15 and 11-16 are presented on Figure A-1 in Appendix A.



4.1.2 Topsoil

The fill in Boreholes 11-15 and 11-16 is underlain by 0.2 m and 0.4 m thick layers of buried topsoil at Elevation 59.5 m and 59.8 m, respectively. The buried topsoil has measured water contents of 33 and 24 per cent.

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

4.1.3 Silty Clay

The fill in Boreholes 11-14 and 11-17 is underlain by silty clay from Elevation 59.5 m and 60.0 m, respectively. The buried topsoil in Boreholes 11-15 and 11-16 is underlain by silty clay from Elevation 59.3 m and 59.5 m, respectively. Where fully penetrated, the silty clay deposit is 3.7 m to 5.9 m thick. Borehole 11-14A was terminated in the silty clay after exploring it for some 4.0 m.

The SPT N values in the silty clay range from 1 to 14 blows per 0.3 m of penetration. The shear strength of the “softer” zones, based on in situ shear vane tests, ranges from 27 kPa to over 96 kPa indicating a firm to stiff consistency. The sensitivity of the silty clay ranges from 4.1 to 14.0. It appears that a crust is present in the silty clay above approximately Elevation 57 m. Above this elevation, in situ shear vane strengths typically exceed 96 kPa, while below this elevation, shear strengths of less than 40 kPa were measured.

The measured water contents in the silty clay range from 44 to 89 per cent. The silty clay is generally of high plasticity based on eleven samples with plastic limits of 24 to 32 per cent, liquid limits of 47 to 89 per cent and plasticity indices of 25 to 58 per cent. The gradations of two samples of the silty clay are shown in Figure A-2. The results of the Atterberg limits determinations are presented on Figure A-5.

Consolidation testing was conducted on Sample 1 from Borehole 11-14A and Sample 5 from Borehole 11-17. The results indicated that the silty clay is vertically overconsolidated by about 80 kPa to 85 kPa beyond the existing overburden pressure. The results of the oedometer testing are presented on Figures A-6 and A-7 in Appendix A for Borehole 11-14A and Figures A-8 and A-9 in Appendix A for Borehole 11-17 and are summarized in the following table.

Borehole No.	Depth (m)	σ'_p (kPa)	σ'_{vo} (kPa)	$\sigma'_p - \sigma'_{vo}$ (kPa)	OCR	e_0	C_r	C_c	C_v (cm²/sec)
11-14A	3.2 – 3.8	118	34	84	3.5	2.02	0.077	1.548	1.8×10^{-2}
11-17	4.6 – 5.0	127	46	81	2.8	1.57	0.063	1.266	1.7×10^{-2}

4.1.4 Silty Sand to Sand and Gravel

Approximately 0.1 m to 0.6 m of very loose to loose sand was encountered below the silty clay in Boreholes 11-14, 11-15 and 11-17 below approximately Elevation 53.6 m to 55.3 m. A 0.3 m thick layer of sand and gravel was encountered below the silty clay in Borehole 11-16 below approximately Elevation 55.8 m. In Borehole 11-



14, the sand is underlain by silty sand below approximately Elevation 53.2 m; this borehole was terminated due to auger refusal after exploring the silty sand for approximately 1.2 m.

The sand is very loose to loose based on SPT N values of 3 to 5 blows per 0.3 m of penetration; the sand was observed to be flowing in the augers after retrieval of Sample 5 at a depth of 6.1 m (about Elevation 53.8 m). The sand and gravel encountered in Borehole 11-16 is inferred to have a compact relative density. The silty sand in Borehole 11-14 is compact to very dense with SPT N values of 21 and over 100 blows per 0.3 m of penetration. The silty sand had a water content of 13 per cent. The gradation of a sample of the silty sand is shown on Figure A-3 in Appendix A.

4.1.5 Silty Sand to Sandy Silt Till

The sand or sand and gravel in Boreholes 11-15 to 11-17 is underlain by silty sand to sandy silt till below approximately Elevation 54.7 m to 55.5 m. Boreholes 11-15 to 11-17 were terminated due to auger refusal on inferred bedrock after exploring the silty sand to sandy silt till for approximately 2.5 m to 2.8 m.

The silty sand to sandy silt till is loose to very dense with SPT N values of 6 to over 100 blows per 0.3 m of penetration and had water contents of 9 to 13 per cent. The silty sand to sandy silt till is a borderline silt to clay of low plasticity based on a plastic limit of 18 per cent, a liquid limit of 12 per cent and a plasticity index of 6 per cent. The results of the Atterberg limits testing are shown on Figure A-5 in Appendix A. The gradations of two samples of silty sand to sandy silt till are shown on Figure A-4 in Appendix A.

4.1.6 Inferred Bedrock Surface

Boreholes 11-14 to 11-17 were terminated due to auger refusal on probable bedrock. The inferred bedrock surface based on the termination depth lies between about Elevation 52.0 m and 53.4 m.

4.1.7 Groundwater Conditions

The groundwater conditions were observed in the boreholes during and upon completion of drilling. Groundwater was encountered at Elevation 53.8 m in Borehole 11-14; the groundwater level was not established in the remaining four boreholes. Standpipe piezometers were installed in Boreholes 11-14A and 11-17 and the groundwater levels measured in the piezometers are summarized in the following table:

Borehole No.	Ground Surface Elevation (m)	Measured Groundwater Level			
		December 19, 2011		March 7, 2012	
		Depth (m)	Elevation (m)	Depth (m)	Elevation (m)
11-14A	59.9	0.2	59.7	Frozen	
11-17	60.3	0.6	59.7	Frozen	

NOTE: On March 7, 2012 frozen water was encountered at the ground surface of the standpipe piezometers of Boreholes 11-14A and 11-17.



FOUNDATION INVESTIGATION AND DESIGN REPORT CULVERT C02 REPLACEMENT, STATION 20+825

The groundwater level at this site has been inferred to be at approximately Elevation 60 m based on the colour change from brown to grey and measured groundwater levels. Groundwater levels are expected to fluctuate seasonally and are expected to rise during wet periods of the year.

5.0 CLOSURE

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

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

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

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**FOUNDATION INVESTIGATION AND DESIGN REPORT
CULVERT C02 REPLACEMENT, STATION 20+825**

PART B

FOUNDATION DESIGN REPORT

CULVERT C02 REPLACEMENT, STATION 20+825
HIGHWAY 401, CORNWALL, ONTARIO
GWP 4029-08-00
MINISTRY OF TRANSPORTATION, ONTARIO - EASTERN REGION



6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides recommendations on the foundation aspects of the design and construction of the proposed replacement of Culvert C02 located at Station 20+825 on Highway 401, approximately 175 m east of the Cornwall Centre Road overpass structures. The recommendations are based on interpretation of the factual data obtained from the five boreholes advanced during the foundation investigation at this site. The interpretation and recommendations are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to design the foundations of the proposed culvert replacement. Where comments are made on construction, they are provided in order to highlight those aspects that could affect the design of the project and for which special provisions may be required in the Contract Documents. Those requiring information on aspects of construction should make their own interpretations of the factual information provided as it may affect equipment selection, proposed construction methods and scheduling.

6.2 Options for Culvert Replacement

The existing culvert consists of a 1200 mm diameter corrugated steel pipe (CSP) which conveys ditch flows from north to south under Highway 401. The culvert is about 77.9 m long and is skewed at 30 degrees to the highway. The invert of the existing culvert is at Elevation 59.7 m at the inlet/north end and at Elevation 59.4 m at the outlet/south end. The culvert is scheduled for replacement as it is nearing the end of its service life.

The subsurface conditions encountered in the boreholes at the culvert site consist of fill overlying firm to very stiff silty clay, which extends to approximately Elevation 54 m to 56 m. The silty clay deposit is underlain by a thin layer of very loose to dense silty sand to sand and gravel which is, in turn, underlain by loose to very dense silty sand to sandy silt till. The till is underlain by limestone bedrock, the surface of which has been inferred to be between about Elevation 52 m and 53 m based on auger refusal in the boreholes. The groundwater level has been interpreted to be at approximately Elevation 60 m (near the natural ground surface at this site) for design.

It is understood that no widening or grade raise is planned as part of the current assignment for the existing Highway 401 embankment at Culvert C02. As a result, there will be no or negligible increase in net embankment loading at the culvert location and no or negligible total and differential settlement.

From a foundations perspective, a concrete box culvert, "open footing" (shallow foundation) concrete culvert or pipe culverts are all feasible for the replacement of Culvert C02; both precast concrete or cast-in-place concrete elements are feasible from a foundations perspective. Associated wing walls/retaining walls, if required, should be supported on shallow foundations. Deep foundations are not required or recommended at this culvert site as shallow foundations will provide sufficient bearing resistance and acceptable settlement performance for the proposed, unchanged highway geometry.

The advantages and disadvantages associated with a precast box culvert, a cast-in-place open footing culvert and a pipe culvert are summarized in Table 1 following the text of this report. A concrete pipe culvert is the preferred technical alternative from a foundations perspective for the reasons summarized below; it is also understood that a closed cell is preferred from an environmental perspective because the drainage channel has no fisheries value. From a hydrologic perspective, MRC has indicated that any circular rough or smooth wall



pipe 1200 mm in diameter, or a box culvert 1800 mm by 900 mm in cross-section will be a suitable replacement alternative. The technical alternative proposed by MRC is replacement of existing Culvert C02 with a 1200 mm diameter concrete pipe culvert. The final selection of the pipe material will be left to the Contractor. Materials considered by MRC to be suitable for construction of the culvert are concrete, polymer laminated steel, high density polyethylene (HDPE) and poly vinyl chloride (PVC).

- Concrete box, pipe or open footing culverts have better long-term durability than CSP culverts.
- A precast box culvert or pipe culvert replacement would minimize the depth of excavation as compared to open footings, allowing the new structure to remain higher within the silty clay “crust” at this site.
- Precast box culvert or pipe culvert segments can often be installed more expeditiously than cast-in-place open footing culverts, resulting shorter durations for excavation/protection systems and traffic staging.
- Precast box or pipe culvert segments are more tolerant of total and differential settlement than open footing culverts. It is noted that settlement is not a significant consideration for replacement of Culvert C02 as no embankment widening or grade raise is currently planned. A box culvert would better accommodate a future median widening (i.e., a grade raise in the current centre median area) than would an open footing culvert.

Recommendations for replacement with a box culvert, pipe culvert and shallow foundation (open footing) culvert are provided in the following sections. The applicability and final selection of the replacement culvert type and size would be the responsibility of the drainage, highway and structural engineers; however, reference should be made to the geotechnical/foundation engineering comments and recommendations provided below regarding settlement performance, bedding, backfill and construction considerations. It is understood that although a 1200 mm diameter concrete pipe culvert has been recommended by MRC in the drainage report, the contractor can opt to install an equivalent pipe type.

6.3 Culvert Foundations

The invert of the existing culvert is at Elevation 59.7 m at the inlet/north end and at Elevation 59.4 m at the outlet/south end. It has been assumed that the replacement culvert will be constructed to maintain the current inlet and outlet levels of Elevation 59.7 m and 59.4 m, respectively.

6.3.1 Pipe Culvert

Concrete Culvert

The use of concrete culverts as a replacement option is both geotechnically and hydrologically feasible. This option may require the most use of heavy equipment as concrete culverts are the heaviest of all pipe culvert types. The remaining pipe culvert types are quite lightweight and the use of heavy equipment during culvert construction may be eliminated or substantially reduced. Concrete pipes are the least susceptible to corrosion, abrasion and degradation by ultra-violet (UV) light and photo-oxidation. Avoidance of overly hard or soft spots in the pipe foundation is important to prevent stress concentrations which can lead to damage or future failure.



Steel Culverts

Replacement of the existing culvert with another CSP of similar diameter is a geotechnically feasible option. MRC's drainage engineers have determined that the soil/water conditions at this site are such that only a 50-year design life is projected for galvanized and aluminized CSPs. Only polymer laminated steel CSPs were found to meet the MTO's requirement of a 75 year design life for culverts.⁷ Compared to culverts manufactured from concrete, HDPE or PVC, polymer laminated steel pipes are the most susceptible to corrosion and abrasion. The long-term performance of CSPs is highly dependent on the care with which the culvert backfill is placed and compacted.

HDPE Culvert

Use of HDPE culverts as a replacement option is both geotechnically and hydrologically feasible. This culvert type has is very resistant to corrosion and abrasion but cannot be structurally reinforced. The exposed ends of HDPE culverts are subject to degradation by ultra-violet (UV) light and photo-oxidation. The long-term performance of HDPE culverts is highly dependent on the care with which the backfill is placed and compacted.

PVC Culvert

Use of PVC culverts as a replacement option is both geotechnically and hydrologically feasible. This culvert type has low susceptibility to corrosion and abrasion but cannot be structurally reinforced. Compared to HDPE culverts, the exposed ends of PVC culverts are somewhat more susceptible to UV degradation and photo-oxidation. The long-term performance of HDPE culverts is highly dependent on the care with which the backfill is placed and compacted.

Founding Level and Bedding

It is not necessary to found a pipe culvert replacement at the standard depth for frost protection purposes as pipe culverts are tolerant of small magnitude movements related to freeze-thaw cycles, should these occur. A pipe culvert replacement should, however, be founded below any existing fill and surficial organic materials.

The bedding for a pipe culvert replacement should be in accordance with the pipe manufacturer's requirements as well as OPSD 802.010 and/or 802.014. In general, it is recommended that the pipe culvert be placed on a minimum thickness of 300 mm of granular bedding material meeting OPSS 1010 Granular A.

For the replacement of Culvert C02, the following table summarizes the anticipated founding levels for a pipe assuming a bedding thickness as described above. Based on these elevations, a pipe culvert replacement will be founded on the stiff to very stiff (crust) portion of the silty clay deposit.

⁷ MRC 2012: Drainage and Hydrology Final Report Rev. 1, Highway 401, Cornwall Centre Road Structure, Detail Design Study, G.W.P. 4029-08-00.



FOUNDATION INVESTIGATION AND DESIGN REPORT CULVERT C02 REPLACEMENT, STATION 20+825

Location	Invert Elevation (m)	Subgrade Level (m)
Inlet (North End)	59.7	59.4
Outlet (South End)	59.4	58.1

Geotechnical Resistance

Pipe culverts are not designed on the basis of geotechnical resistances. Flexible pipes (CSPs, HDPE and PVC) rely on the bedding and backfill materials to support the applied loads. For rigid concrete pipe, the applied loads are primarily resisted by the pipe with some contribution from the bedding and backfill materials.

6.3.2 Precast Concrete Box Culvert

Founding Level and Bedding

It is not necessary to found a box culvert replacement at the standard depth for frost protection purposes as box structures are tolerant of small magnitude movements related to freeze-thaw cycles should these occur. A box culvert replacement should, however, be founded below any existing fill and surficial organic materials.

The bedding and/or levelling pad requirements for a box culvert replacement should be in accordance with Ontario Provincial Standard Specification (OPSS) 422 (*Construction Specification for Precast Reinforced Concrete Box Culverts and Box Sewers in Open Cut*) for precast rigid frame culverts. It is recommended that the box culvert segments be placed on a minimum thickness of 300 mm of granular bedding material meeting OPSS 1010 Granular A.

For the replacement of Culvert C02, the table below summarizes the recommended founding levels, assuming a base slab thickness of 300 mm and bedding thickness as described above. Based on these elevations, the box culvert replacement will be founded on the stiff to very stiff silty clay deposit.

Location	Invert Elevation (m)	Box Culvert Founding Elevation (m)	Subgrade Level (m)
Inlet (North End)	59.7	59.4	59.1
Outlet (South End)	59.4	59.1	58.8

Based on the founding level and subsurface conditions encountered in the boreholes at this site, no subexcavation requirements have been identified at this time. However, as discussed in Section 6.7.3, the culvert box subgrade should be inspected by a Quality Verification Engineer (QVE) and, if overly soft or wet materials are encountered within the culvert excavations, these materials should be subexcavated and replaced with compacted Granular A.



Geotechnical Resistance

The very stiff to firm silty clay at or below Elevation 58.8 m to 59.1 m is suitable for support of the proposed replacement culvert. A factored geotechnical resistance at Ultimate Limit States (ULS) of 225 kPa and a geotechnical resistance of 150 kPa at Serviceability Limit States (SLS) may be used for design purposes. The SLS value corresponds to 25 mm of settlement for an approximately 1.2 m wide box culvert.

The recommended resistances are dependent on the box culvert span and applied loads; the geotechnical resistances should, therefore, be reviewed if the culvert span or founding elevation differs from that given above.

Resistance to Lateral Forces/Sliding Resistance

The resistance to lateral forces/sliding resistance between the culvert base and the bedding should be calculated in accordance with Section 6.7.5 of the CHBDC. The coefficient of friction, $\tan \delta$, between the precast concrete slab and compacted Granular A fill may be taken as 0.45. In accordance with the CHBDC Section 6.7.5, a factor of 0.8 is applied in the equation to calculate the factored horizontal geotechnical resistance, H_{ri} , as follows:

$$H_{ri} = 0.8A'c' + 0.8V\tan\delta > H_f$$

where:

A'	-	effective contact area, square metres
c'	=	0 kPa
$\tan \delta$	-	as given above
V	-	unfactored vertical force, kN
H_f	-	unfactored horizontal load, kN

6.3.3 Cast-in-Place Open Footing Culvert

Founding Level and Frost Protection Requirements

Strip footings for an open footing culvert replacement, and for any associated concrete wing walls/retaining walls, should be founded at a minimum depth of 1.7 m below the lowest surrounding grade to provide adequate protection against frost penetration as per Ontario Provincial Standard Drawing (OPSD) 3090.101 (*Foundation Frost Depths for Southern Ontario*). In addition, the footings should extend below any existing fill and surficial organic materials, where present. The following table summarizes the footing founding elevations along the replacement culvert:

Location	Invert Elevation (m)	Footing Founding Elevation (m)
Inlet (North End)	59.7	58.0
Outlet (South End)	59.4	57.7



The footings will be founded within the stiff to very stiff (crust) portion of the silty clay deposit. Based on the footing founding levels and subsurface conditions encountered in the boreholes, no subexcavation requirements have been identified at this time. However, as discussed in Section 6.7.3, the footing subgrade should be inspected by a Quality Verification Engineer (QVE) and, if softened or disturbed soils are encountered, these should be subexcavated. Following inspection and approval of the subgrade, it is recommended that a concrete working slab be placed to protect the subgrade from degradation during the preparation works for the cast-in-place concrete culvert; further discussion on this aspect is also provided in Section 6.7.3.

Geotechnical Resistances

The firm to stiff silty clay at or below approximately Elevation 58.0 m to 57.7 m is suitable for support of the proposed replacement culvert. A factored geotechnical resistance at ULS of 150 kPa and a geotechnical resistance at SLS of 100 kPa may be used for design purposes. The SLS value corresponds to 25 mm of settlement for an approximately 1.5 m wide footing.

The recommended resistances are dependent on the footing size, configuration and applied loads; the geotechnical resistances should, therefore, be reviewed if the footing size or founding elevation differs from that given above.

Resistance to Lateral Forces/Sliding Resistance

The resistance to lateral forces / sliding resistance between cast-in-place concrete footings and a concrete working slab, and between the concrete working slab and native silty clay, should be calculated in accordance with Section 6.7.5 of the CHBDC. The coefficient of friction, $\tan \phi'$, between cast-in-place concrete footings and the concrete working slab may be taken as 0.6, while that between the cast-in-place concrete working slab and the silty clay may be taken as 0.5. In accordance with the CHBDC Section 6.7.5, a factor of 0.8 is applied in the equation to calculate the factored horizontal geotechnical resistance, H_{ri} , as follows:

$$H_{ri} = 0.8A'c' + 0.8V\tan\delta > H_f$$

where:

A'	-	effective contact area, square metres
c'	=	0 kPa
$\tan \phi'$	-	as given above
V	-	unfactored vertical force, kN
H_f	-	unfactored horizontal load, kN

The unfactored coefficient of passive pressure of the portion of the culvert footing below the invert may be taken as 2.7 based on an unfactored effective angle of internal friction, ϕ , of 27 degrees for the silty clay.



6.3.4 Settlement and Camber

It is understood that Culvert C02 will be replaced with no modification of the existing Highway 401 embankment (i.e., no widening and no grade raise). The height of fill above the culvert crown will be maintained at less than 4.5 m. Since there will be no or negligible change in the embankment loading, differential settlement along the replacement culvert is not anticipated and cambering of the replacement culvert will not be required.

6.3.5 Backfill and Cover

Backfill, cover and construction of the frost taper (backfill transition) for concrete box culverts should be completed in accordance with OPSS 422 (*Construction Specification for Precast Reinforced Concrete Box Culverts and Box Sewers in Open Cut*) and/or OPSD 803.010 (*Backfill and Cover for Concrete Culverts*).

A pipe culvert replacement should be constructed in accordance with OPSS 421. Placement of all backfill and cover materials and construction of the frost taper (backfill transition) should be carried out in accordance with OPSD 802.010, 802.014, 802.030 and/or 802.031, as applicable. Excavations for the pipe culvert should have a clearance width that exceeds the diameter of the pipe by at least 0.5 m on each side to allow for good workmanship and effective compaction of the fill.

Backfill to culvert walls and above the culvert should consist of granular fill meeting the requirements of OPSS 1010 Granular A or Granular B Type II, but with less than 5 per cent passing the No. 200 sieve. The backfill should be placed and compacted in accordance with MTO's Special Provision SP105S21 (Amendment to OPSS 501). The fill depth during placement should be maintained equal on both sides of the culvert walls, with one side not exceeding the other by more than 500 mm. The culvert replacement should be designed for the full overburden pressure and live load assuming that the embankment fill has a unit weight of 22 kN/m³ for Granular A and 21 kN/m³ for Granular B Type II or select earth fill above and/or surrounding the culvert.

6.4 End Treatments, Filters and Scour Protection

The hydraulic assessment carried out by MRC indicated that the Culvert C02 is not flowing full at either the 25- or 50-year design flow. This culvert only handles ditch flows.

Water flowing beneath a culvert could potentially cause undermining and scouring. Seepage flowing around the culvert barrel or walls has the potential to remove fines from the embankment fill and lead to piping and erosion. Therefore the replacement culvert must be designed with the appropriate end treatment to prevent undermining, scouring and piping. The risk of piping at this location is considered low because the culvert conveys only ditch flows, the foundation soils are not susceptible to piping and the results of the hydraulic analysis indicated that water is never ponded at this culvert for an extended period of time at the 25- and 50-year design flows. However, if there will be a significant difference in hydraulic head between each end of the culvert, the use of additional anti-seepage measures such as clay seals or outlet filters may be necessary. Clay seals, if used, are to meet the requirements of OPSS 1205.

If a pipe culvert is selected and hydraulic analysis indicates that the culvert will be subject to rapid fluctuations of the water level or hydraulic uplift, additional end treatment should be provided by the design engineer. Backfill for



a pipe culvert should be graded to minimize the uncovered length of the pipe and reduce the potential for hydraulic uplift. As required by the CHBDC, precast concrete box culverts should be designed with cutoff walls, at least at the upstream end, to prevent undermining or possible collapse of the ends.

If the water flow velocities are sufficiently high, provision should be made for scour protection, in the form of non-woven geotextiles and/or rip-rap, at the inlet and outlet. Scour protection for open footing culverts should be provided in accordance with Section 1.9.5 of the CHBDC. The requirements for and specific design of scour and erosion protection measures should be assessed by the hydraulic design engineer. However, as a minimum, it is recommended that rip-rap treatment consistent with the standard OPSD 810.010 Treatment Type A should be provided at the culvert outlet. In addition, sediment control measures such as silt fences and erosion blankets may be required during construction along with diversion/piping of the watercourse to mitigate migration of fine particles. If a clay blanket is adopted as an anti-seepage measure, rip-rap should be provided over the full extent of the blanket and on the embankment fill slope adjacent to the culvert.

6.5 Liquefaction Potential and Seismic Analysis

6.5.1 Seismic Parameters

The site is located in Cornwall, Ontario. According to Table A.3.1.1 of the CHBDC, the zonal acceleration ratio, A , applicable to this site is 0.20. The corresponding acceleration related seismic zone, Z_a , is 4.

Seismic loads are generally not considered in the design of pipe culverts less than 3 m in span. Seismic analysis of precast box or cast-in-place open footing culverts is to be conducted in accordance with CHBDC Clause 7.5.5. The effects of site conditions on the culvert response are to be included in the determination of the seismic loads. The stratigraphy generally consists of surficial topsoil or pavement structure overlying a predominantly granular embankment fill. The fill is underlain in sequence by firm to very stiff silty clay then compact to dense silty sand to sandy silt till. The natural ground surface at the site is at approximately Elevation 60 m. The inferred bedrock surface for the Bobcaygeon Formation limestone was encountered at an approximate depth of 7 m to 8 m or between about Elevation 52 m to 53 m. Based on the site stratigraphy, the soil profile type is categorized as Type I with a seismic site response coefficient, S , of 1.0 based on the CHBDC criteria.

6.5.2 Seismic Hazard Assessment

A preliminary screening of the soil stratigraphy was conducted using the procedure outlined in the Federal Highway Administration recommended procedures⁸ and Canadian Foundation Engineering Manual (CFEM). The silty clay is not considered to be susceptible to liquefaction or cyclic mobility. However, the silty clay materials may undergo significant deformations during a seismic event if the cyclic shear stress is greater than the static undrained shear strength. Although saturated layers of very loose to loose sands underlie the silty clay at several locations, the layers are generally less than 0.6 m thick. The silty sand to sandy silt till is below the groundwater

⁸ Federal Highway Administration (FHWA). (1997). "Design Guidance: Geotechnical Earthquake Engineering For Highways. Volume I – Design Principles." *Geotechnical Engineering Circular No. 3: FHWA-SA-97-076*, Washington, D.C.



level but is not considered to be susceptible to liquefaction as it is generally non-plastic and has a water content to liquid limit ratio (w/w_L) less than 0.8.

6.6 Lateral Earth Pressures for Design

The lateral pressures acting on the culvert walls will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the freedom of lateral movement of the structure and on the drainage conditions behind the walls. The following recommendations are made concerning the design of the culvert walls in accordance with the CHBDC:

- Select, free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B but with less than 5 per cent passing the No. 200 sieve should be used as backfill behind the abutments and retaining walls. This fill should be compacted in accordance with MTO's SP 105S21. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the abutment granular backfill requirements with respect to subdrains and frost taper should be in accordance with Ontario Provincial Standard Drawing (OPSD) 3101.150, 3121.150 and 3190.100.
- A minimum compaction surcharge equal to 12 kPa should be included in the lateral earth pressures for the structural design of the walls in accordance with CHBDC, Figure 6.6. Compaction equipment should be used in accordance with MTO's SP 105S21.
- The granular fill may be placed either in a zone with a width equal to at least 1.7 m behind the back of the wall (Case (a) from Commentary on CHBDC, Figure C6.20) or within the wedge-shaped zone defined by a line drawn at a maximum 1 horizontal to 1 vertical extending up and back from the rear face of the footing (Case (b) from Commentary on CHBDC Figure C6.20).
- For Case (a), the restrained case, the pressures are based on the existing embankment fill materials and the following parameters (unfactored) may be used:

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

- For Case (b), the unrestrained case, the pressures are based on the granular fill as placed and the following parameters (unfactored) may be assumed:



FOUNDATION INVESTIGATION AND DESIGN REPORT CULVERT C02 REPLACEMENT, STATION 20+825

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

- If the wall support allows lateral yielding of the stem (for culvert wing walls for example), active earth pressures may be used in the geotechnical design. If the wall support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design.
- Seismic loading will result in increased lateral earth pressures acting on the walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above plus the earthquake-induced dynamic earth pressure. According to the CHBDC, the site-specific zonal acceleration ratio for Cornwall is 0.2. Based on experience, for the subsurface conditions at this site, no significant amplification of the ground motion is expected. The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio $A = 0.2$.
- In accordance with Sections 4.6.4 and C.4.6.4 of the CHBDC and its Commentary, for structures which do not allow lateral yielding, the horizontal seismic coefficient, k_h , used in the calculation of the seismic active pressure coefficient, is taken as 1.5 times the zonal acceleration ratio (i.e., $k_h = 0.30$). For structures which allow lateral yielding, k_h is taken as 0.5 times the zonal acceleration ratio (i.e., $k_h = 0.10$).
- The following seismic active pressure coefficients (K_{AE}) for the two backfill cases (Case (a) and Case (b)) may be used in design. These seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat. Where sloping backfill is present above the top of the wall, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Granular A	Granular B Type II
Yielding wall	0.30	0.30
Non-yielding wall	0.50	0.50

Note: These CHBDC seismic K_{AE} values include the effect of wall friction ($\delta = \phi'/2$)

- The above K_{AE} values for yielding walls are applicable provided that the wall can move up to 250A (mm), where A is the design zonal acceleration ratio of 0.20. This corresponds to displacements of up to 50 mm at this site.



- The earthquake induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e. an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

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

where	$\sigma_h(d)$ =	the lateral earth pressure at depth d (kPa);
	K =	either the static active earth pressure coefficient (K_a) or the static at-rest earth pressure coefficient (K_o);
	K_{AE} =	the seismic active earth pressure coefficient;
	γ' =	the effective unit weight of the soil (kN/m ³) <ul style="list-style-type: none">■ taken as soil unit weights given above for new granular fill, and■ taken as 20 kN/m³ for the existing embankment fill;
	d =	the depth below the top of the wall (m); and
	H =	the total height of the wall above the toe (m).

6.7 Construction Considerations

6.7.1 Excavation and Temporary Cut Slopes

Excavations for removal of the existing culvert and construction of its replacement will extend through the existing Highway 401 embankment fill into the silty clay deposit. All excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Ontario Occupational Health and Safety Act and Regulations for Construction Projects. The existing fill and native silty clay would be classified as Type 3 soils according to the OHSA. Temporary open-cut slopes within the fill materials and native soils should be maintained no steeper than 1H:1V, provided that appropriate groundwater control measures are in place.

6.7.2 Groundwater and Surface Water Control

Although excavations will terminate below the interpreted groundwater level of approximately Elevation 60 m, seepage volumes are expected to be low due to the relatively impermeable nature of the silty clay deposit. However, "perched" groundwater may be present at the base of the highway embankment fill (perched on top of the underlying, less permeable silty clay) and some seepage into the excavation from the granular fill materials should be expected. Such seepage will be heavier during periods of sustained precipitation. Pumping from properly filtered sumps located at the base of the excavations is expected to be sufficient to provide groundwater control. The sumps should be maintained outside of the actual foundation limits.

Surface water runoff should be directed away from the excavations at all times. The existing culvert flows will need to be diverted/piped during construction. The Non-Standard Special Provision (NSSP) for groundwater



control provided in Appendix C should be included in the Contract Documents to alert the contractor about the need for adequate control of surface and groundwater flows.

6.7.3 Subgrade Preparation and Protection

All topsoil, organics and soft or loose soils should be removed from below the proposed founding elevation and wasted or reused as landscaping fill, as required. Subgrade preparation should be performed and monitored in accordance with OPSS. The cleaned excavation base should be inspected by a Quality Verification Engineer (QVE) qualified in geotechnical engineering prior to placing compacted fill or a concrete working slab.

Where the subgrade is formed in the native silty clay deposit, this soil will be susceptible to softening and disturbance due to construction activities and ponded water. If a cast-in-place concrete culvert is adopted, it is recommended that the footings be constructed on a 75 mm thick concrete working slab consisting of 10 MPa lean concrete; the Contract Documents should include the appropriate Non-Standard Special Provision for the concrete working slab item (see Appendix C).

Based on the footing founding levels and subsurface conditions as encountered in the boreholes advanced at this site, no subexcavation requirements have been identified at this time. However, in the event that subexcavation is required, the width of the required subexcavation should be defined by lines extending from 0.3 m beyond the outside edges of the proposed culvert base or footings outward and downward at 1H:1V. The subexcavated area should be backfilled with granular material meeting OPSS 1010 Granular A or Granular B Type II placed and compacted in accordance with the requirements of MTO's Special Provision SP105S10.

6.7.4 Temporary Protection Systems

It is understood that the replacement of the existing culvert is planned to be completed in stages in conjunction with the staging and traffic shifts proposed for the replacement of the Cornwall Centre Road overpasses. It is anticipated that a temporary protection system would be required in the highway median to allow for replacement of, first, one half, then the other half of the culvert; in addition, temporary protection systems may be required along the outside edges of the culvert excavation (i.e., oriented at an approximately 30 degree skew to the highway), unless the traffic staging associated with the overpass replacement permits sufficient space to allow for open-cut excavation.

The temporary protection systems are to be designed and constructed by the Contractor in accordance with OPSS 539. The lateral movement of the protection system(s) should meet Performance Level 2.

Conceptually, temporary protection systems could consist of soldier piles and lagging where the H-piles would be placed in auger holes to a suitable depth and horizontal lagging installed as the excavation proceeds. Driven steel sheet piling may also be considered at this site, although cobbles present within the existing embankment fill material pose a minor risk of obstructing sheet pile installations. Support to the wall systems could be in the form of struts and walers or rakers and anchors. The lateral support must be designed to accommodate the loads applied from pressures and surcharge pressures from area line or point loads as well as the impact of sloping ground behind the system, where applicable.



FOUNDATION INVESTIGATION AND DESIGN REPORT CULVERT C02 REPLACEMENT, STATION 20+825

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

Soil Type	Earth Pressure Coefficient			Angle of Internal Friction (degrees)	Unit Weight (Above Groundwater) (kN/m ³)
	Active, K_a	At Rest, K_o	Passive, K_p		
Embankment fill	0.33	0.50	3.0	30	20
Silty clay	0.38	0.55	2.7	27	19
Silty sand to sandy silt till	0.31	0.47	3.3	32	22

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

7.0 CLOSURE

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

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FOUNDATION INVESTIGATION AND DESIGN REPORT CULVERT C02 REPLACEMENT, STATION 20+825

TABLE I

COMPARISON OF CULVERT REPLACEMENT ALTERNATIVES

Culvert C02, Station 20+825
Highway 401
GWP 4029-08-00

REPLACEMENT OPTION	ADVANTAGES	DISADVANTAGES	RELATIVE COSTS	RELATIVE RISKS/ CONSEQUENCES
Pipe Culvert	<ul style="list-style-type: none">• Economical• Can be constructed quickly• Less excavation required compared to cast-in-place open footing option• Concrete, polymer laminated CSP, HDPE and PVC durable for required 75 year design life; concrete pipe has best overall durability• CSPs, HDPE and PVC pipe can be easily handled with little to no need for heavy equipment	<ul style="list-style-type: none">• Steel pipes subject to degradation if water/soils are corrosive – least durable option if steel pipe selected (projected design life for galvanized or aluminized CSPs is 50 years based on the water and soil conditions at this site)• HDPE and PVC pipes subject to UV degradation and photo-oxidation• Flexible pipes (CSPs, HPDE, PVC) are the options for which the long-term performance is most dependent on the care with which culvert backfill is placed• Most settlement-sensitive option; for CSPs, circumferential seams may separate due to movement of backfill• Option most susceptible to clogging by ice or debris• Buoyancy problems, particularly for non-concrete pipes, may develop if improperly designed and constructed• Alternative most susceptible to piping failures	<ul style="list-style-type: none">• Lowest Cost Option	<ul style="list-style-type: none">• Low to Moderate risk.• Risk of 75 year design life not being achieved if subject to corrosive or abrasive environment in case of CSPs.



FOUNDATION INVESTIGATION AND DESIGN REPORT CULVERT C02 REPLACEMENT, STATION 20+825

REPLACEMENT OPTION	ADVANTAGES	DISADVANTAGES	RELATIVE COSTS	RELATIVE RISKS/ CONSEQUENCES
Pre-Cast Concrete (Preferred technical alternative)	<ul style="list-style-type: none">• Can be constructed quickly• Option with greatest tolerance for settlement, particularly if embankment is widened or raised in the future• Durable for required 75 year design life• Resistant to corrosion• Less excavation required compared to CIP open footing option• Less susceptible to clogging by ice or debris than CSP	<ul style="list-style-type: none">• Culvert floor may crack and heave if inadequately reinforced• Without appropriate end treatment, floor can be undermined during high flows• Potential for leakage at joints if sections are inadequately connected	<ul style="list-style-type: none">• Cost intermediate between CSP and CIP open footing alternative.	<ul style="list-style-type: none">• Lowest risk option.
Cast-in-Place (CIP) Open Footing	<ul style="list-style-type: none">• Resistant to corrosion• Durable for required 75 year design life• Less susceptible to clogging by ice or debris than CSP	<ul style="list-style-type: none">• Most expensive and labour intensive option• Most extensive excavation required• Option requiring longest construction time• Alternative which may experience the greatest magnitude of settlement if the embankment is raised or widened since footings will be in lower crust	<ul style="list-style-type: none">• Most expensive option	<ul style="list-style-type: none">• Moderate risk.• Option with highest risk of settlement if embankment is widened or raised substantially in the future.

- NOTES:**
1. Table to be read in conjunction with accompanying report.
 2. Qualitative cost estimates are provided to illustrate the relative costs of alternatives compared to the lowest cost option. Quantitative cost estimates were not provided since final selection of the culvert type is the responsibility of the Contractor.

Prepared By: DUP
Checked By: LCC

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Consistency

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

(b) Cohesive Soils

Dynamic Cone Penetration Resistance; N_d :

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

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

IV. SOIL TESTS

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

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

LIST OF SYMBOLS

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

I. General

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation pressure
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

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

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

RECORD OF BOREHOLE No 11-14

1 OF 1

METRIC

PROJECT 10-1121-0259-2000
W.P. 4029-08-00 LOCATION N 4991093.3 ; E 203559.7 ORIGINATED BY PH
DIST HWY 401 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY AMG
DATUM GEODETIC DATE November 04, 2011 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								○ UNCONFINED	+ FIELD VANE								
						● QUICK TRIAXIAL	× LAB VANE	WATER CONTENT (%)									
						20	40	60	80	100	20	40	60				
59.9	GROUND SURFACE		1	AS		▽	59										
0.0	FILL, silty clay and organics, with roots		2	SS	7		58										
59.5			3	SS	5		57										
0.4	SILTY CLAY, Firm to stiff Grey to brown becoming grey below about elev. 56.8m						56										
							55										
			4	SS	PM		54										
							53										
53.6			5	SS	5												
6.3	SAND, Loose																
53.2	Grey		6	SS	21												
6.7	SILTY SAND, some gravel, trace clay Compact to very dense Grey		7	SS	16/ 75mm												
52.0	END OF BOREHOLE																
7.9	Auger refusal at about elev. 52.1m. Groundwater encountered at about elev. 53.8m during drilling on Nov. 4, 2011.																

RECORD OF BOREHOLE No 11-14A

1 OF 1

METRIC

PROJECT 10-1121-0259-2000

W.P. 4029-08-00

LOCATION N 4991093.3 ; E 203560.7

ORIGINATED BY PH

DIST HWY 401

BOREHOLE TYPE POWER AUGER, HOLLOW STEM

COMPILED BY AMG

DATUM GEODETIC

DATE November 07, 2011

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
59.9	GROUND SURFACE										
0.0	FILL, silty clay and organics, with roots										
59.5											
0.4	SILTY CLAY, Firm to stiff Grey to brown becoming grey below about elev. 56.8m										
			1	SS	PH						
55.5	END OF BOREHOLE										
4.4	Stratigraphy inferred from Borehole 11-14. Groundwater level not established during drilling. Water level measured at elev. 59.7m on Dec. 19, 2011. Water frozen in tubing at elev. 59.9m on March 7, 2012.										

RECORD OF BOREHOLE No 11-16

1 OF 1

METRIC

PROJECT 10-1121-0259-2000

W.P. 4029-08-00

LOCATION N 4991137.7 ; E 203536.0

ORIGINATED BY PH

DIST HWY 401

BOREHOLE TYPE POWER AUGER, HOLLOW STEM

COMPILED BY AMG

DATUM GEODETIC

DATE November 10, 2011

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
65.3	GROUND SURFACE							20 40 60 80 100		20 40 60				
0.0	FILL, crushed sand and gravel (granular roadbase) Grey		1	AS			65							
0.3	FILL, sand and gravel, trace silt, (granular subbase) Compact Brown		2	SS	22		64							
64.0	FILL, silty sand, some gravel, trace to some clay, cobbles Dense to very dense Brown to grey-brown		3	SS	43		63							
1.2			4	SS	39		62							
			5	SS	194		61							
59.8							60							
5.4	TOPSOIL, Alluvium		6	SS	9		59							
59.5			7	SS	8									
5.8	SILTY CLAY, Firm to stiff Grey to brown becoming grey below about elev. 57.0m		8	SS	2		58							
							57							
							56							
55.8			9	SS	6		55							
9.5	SAND AND GRAVEL, Compact Grey		10	SS	17		54							
55.5			11	SS	8		53							
9.8	SILTY SAND to SANDY SILT (TILL), some gravel, trace clay Loose to very dense Grey		12	SS	66/ 200mm									
52.7														
12.6	END OF BOREHOLE Auger Refusal at about elev. 52.7m. Groundwater level not established during drilling.													

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

PROJECT 10-1121-0259-2000

W.P. 4029-08-00

LOCATION N 4991163.5 ;E 203525.8

ORIGINATED BY PH

DIST _____ HWY 401

BOREHOLE TYPE POWER AUGER, HOLLOW STEM

COMPILED BY AMG

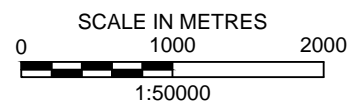
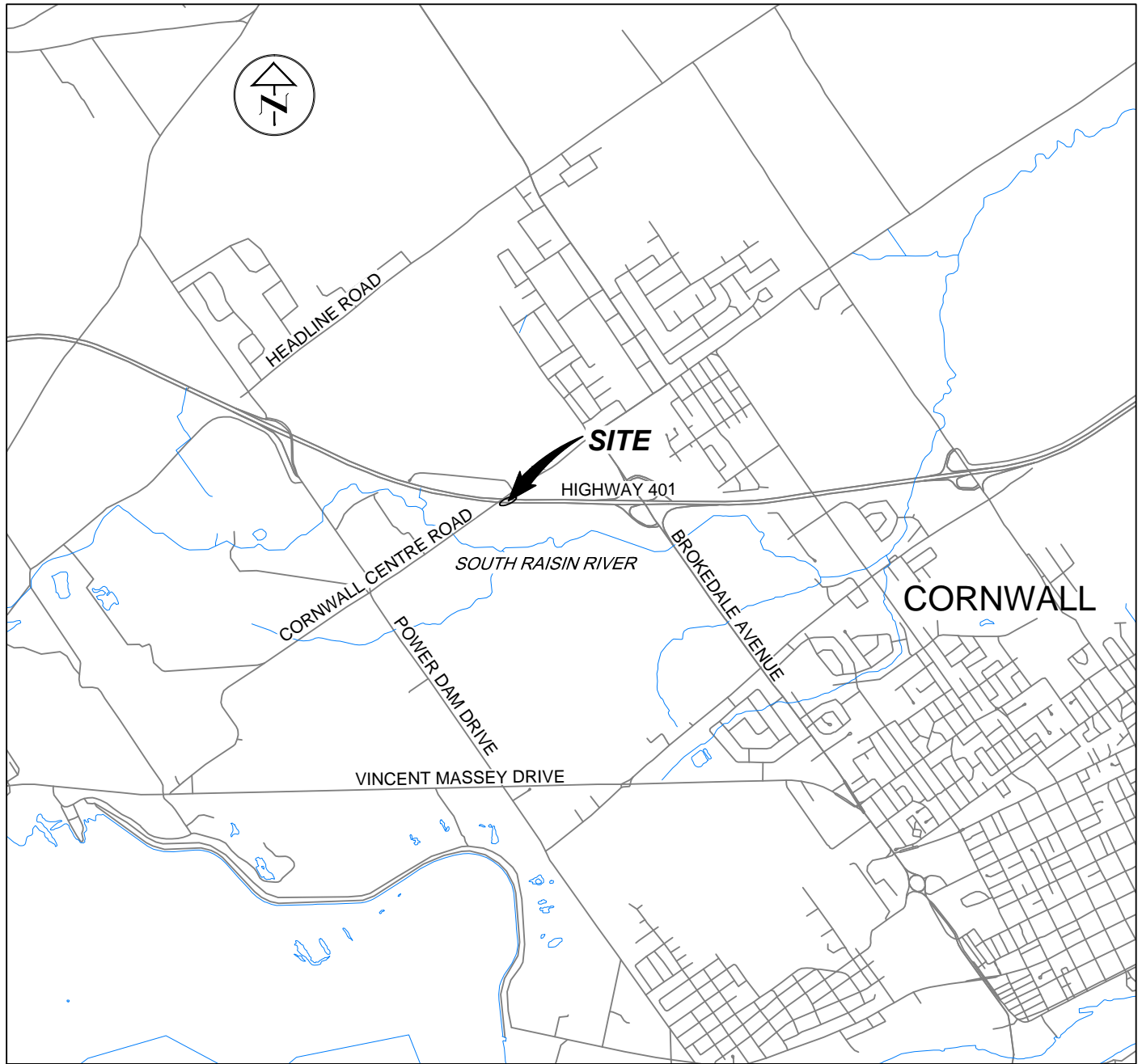
DATUM GEODETIC

DATE November 07, 2011

CHECKED BY _____

[illegible]

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



REFERENCE

DRAWING BASED ON CANMAP STREETFILES V2005.4.

NOTE

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

PROJECT

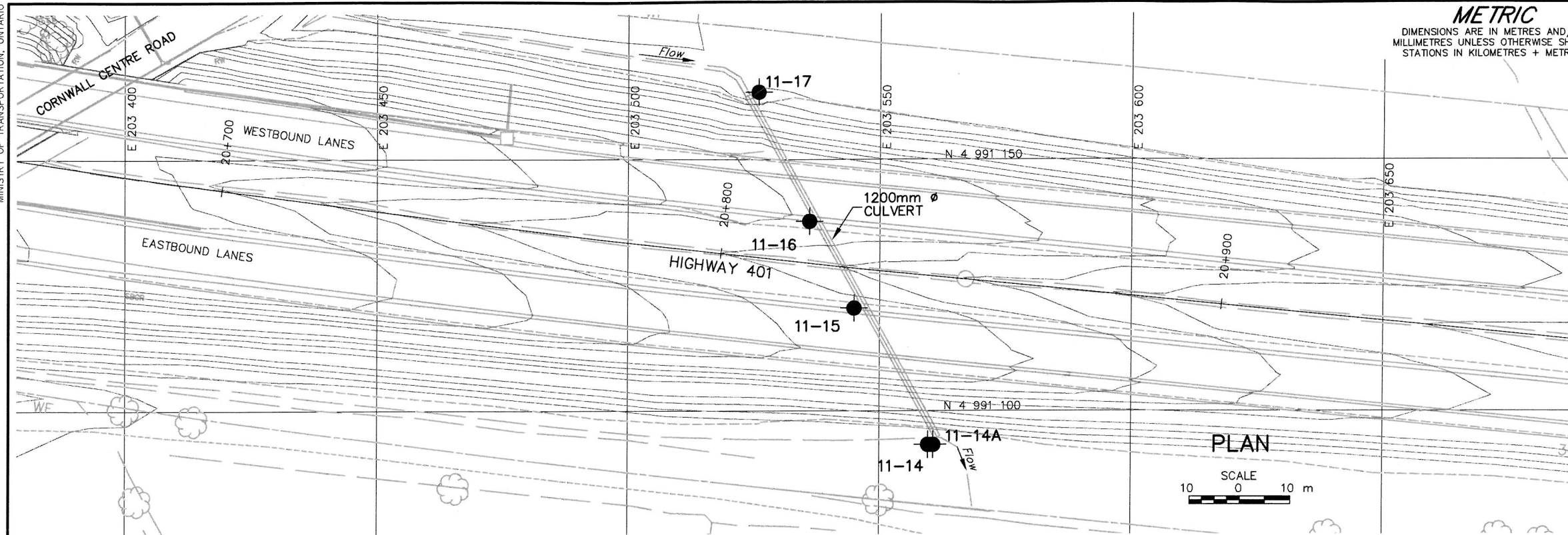
CULVERT C02 REPLACEMENT
HIGHWAY 401
GWP 4029-08-00

TITLE

KEY PLAN



PROJECT No.		10-1121-0259	FILE No.		1011210259-2000-F03001
CADD	DCH	Jan. 12/12	SCALE	AS SHOWN	REV.
CHECK			FIGURE 1		



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

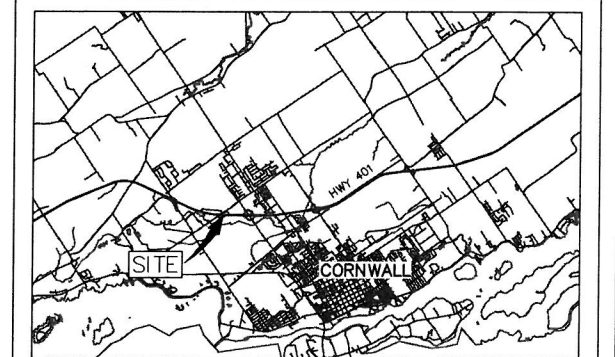
CONT No.
WP No. 4029-08-00



CULVERT C02 REPLACEMENT
HIGHWAY 401
BOREHOLE LOCATIONS AND SOIL STRATA



Golder Associates Ltd.
LONDON, ONTARIO, CANADA



SCALE IN KILOMETRES

LEGEND

- Borehole - Current Investigation
- ⊥ Seal
- ⊥ Standpipe
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- ≡ WL upon completion of drilling
- ≡ WL in standpipe (December 19, 2011)
- R Refusal

No.	ELEVATION(m)	CO-ORDINATES (MTM ZONE 8)	
		NORTHING	EASTING
11-14	59.9	4991093.3	203559.7
11-14A	59.9	4991093.3	203560.7
11-15	65.0	4991120.4	203544.9
11-16	65.3	4991137.7	203536.0
11-17	60.3	4991163.5	203525.8

NOTES

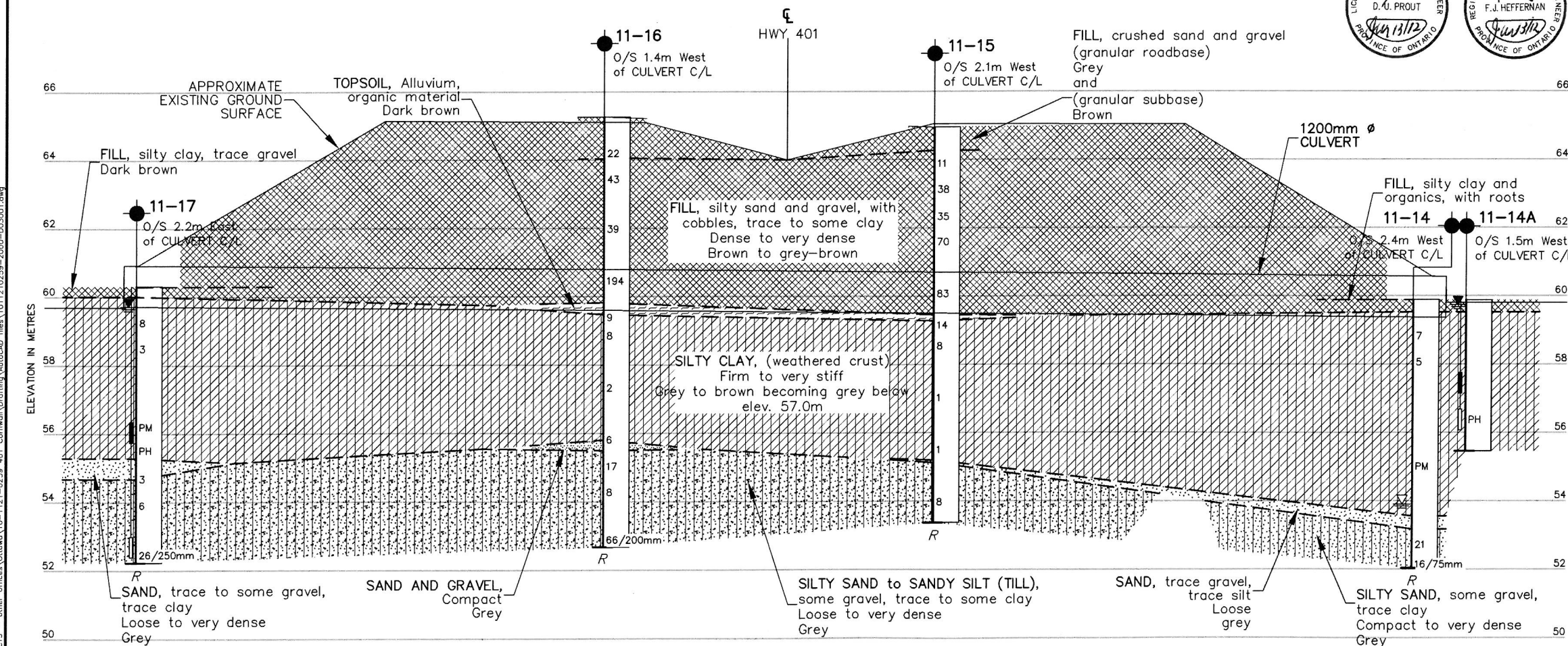
This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

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

REFERENCE

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

NO.	DATE	BY	REVISION
Geocres No.	31G-244		
HWY.	401	PROJECT NO.	10-1121-0259
SUBM'D.	DUP	CHKD.	DUP
DRAWN:	DCH	CHKD.	FJH
DATE:	Jan. 12/12	APPD.	
DIST.		SITE:	
DWG.	1		



PROFILE ALONG CENTRELINE OF CULVERT C02

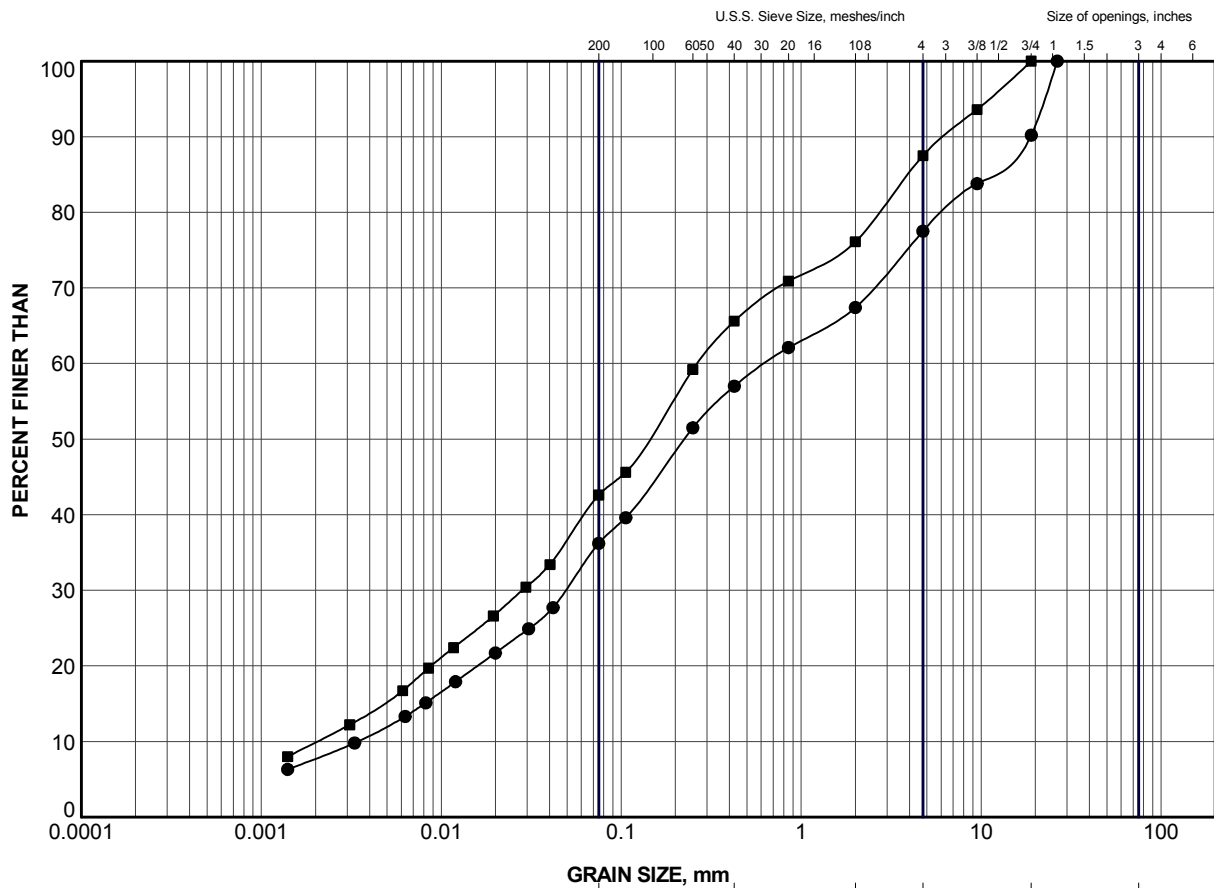
HORIZONTAL SCALE
3 0 3 m

VERTICAL SCALE
1.5 0 1.5 m



APPENDIX A


Laboratory Test Data – Soils

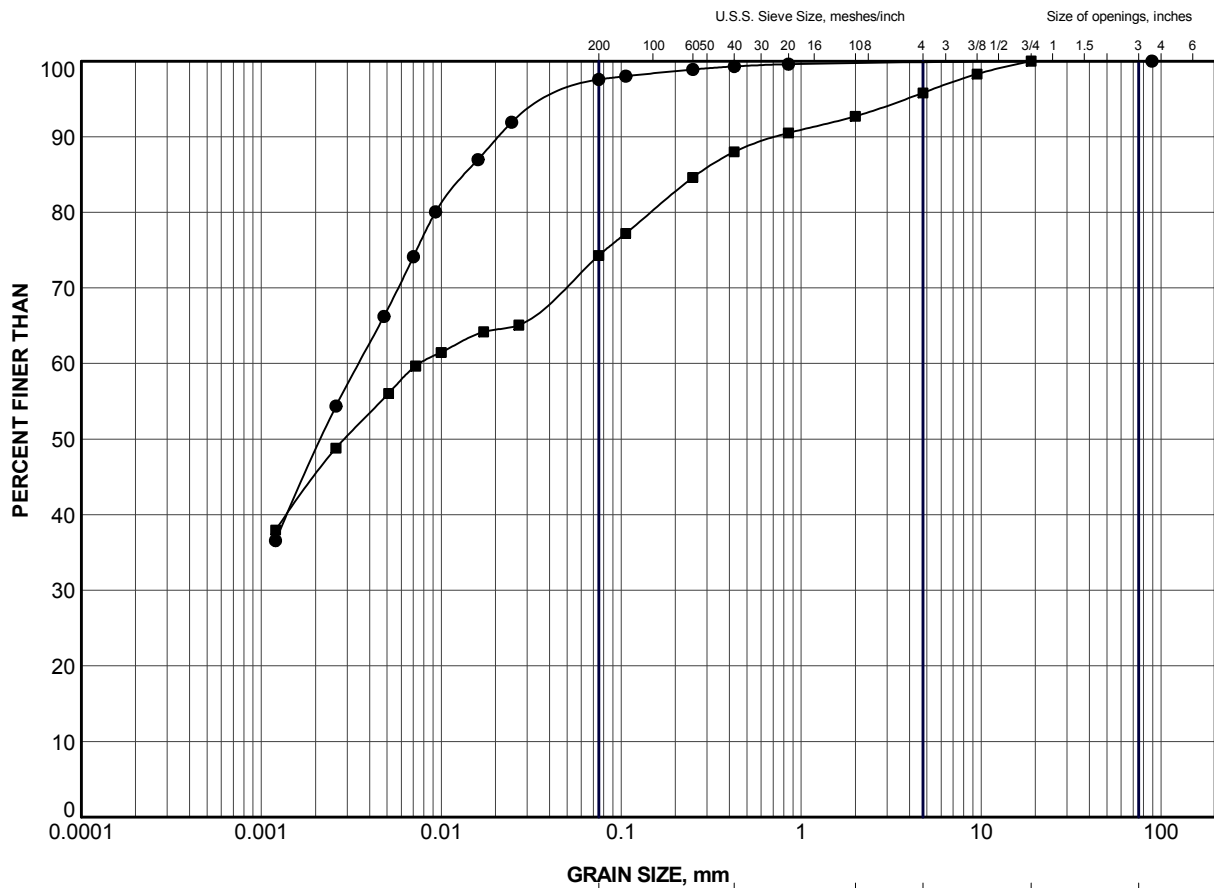


GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	11-15	4	62.4
■	11-16	3	63.4


PROJECT				CULVERT C02 REPLACEMENT HIGHWAY 401 GWP 4029-08-00			
TITLE				GRAIN SIZE DISTRIBUTION FILL			
PROJECT No.10-1121-0259-2000				FILE No.1011210259-2000-F030A1			
DRAWN		AMG		DEC. 8/11		SCALE N/A REV.	
CHECK						FIGURE A-1	
 Golder Associates LONDON, ONTARIO							

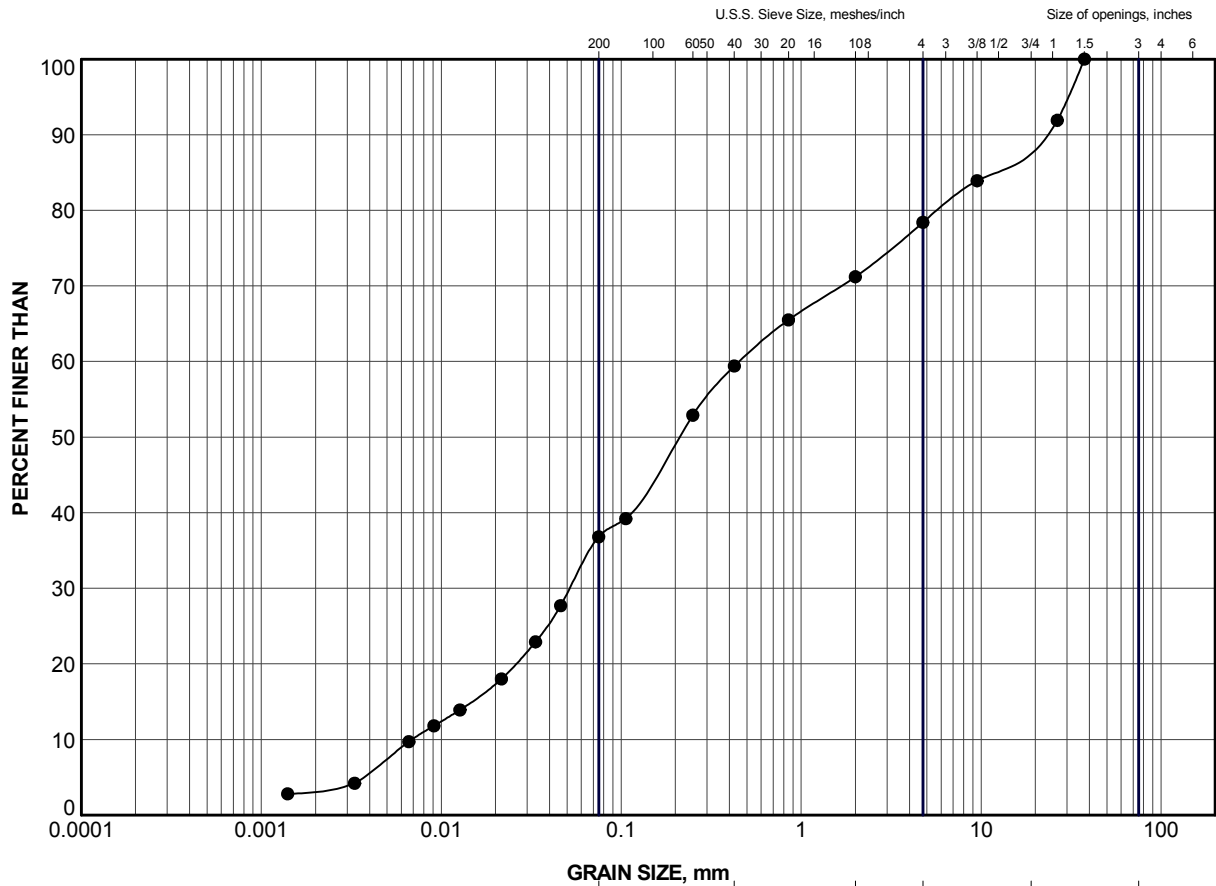


GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND


SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	11-14A	1	56.4
■	11-17	5	55.5

PROJECT				CULVERT C02 REPLACEMENT HIGHWAY 401 GWP 4029-08-00			
TITLE				GRAIN SIZE DISTRIBUTION SILTY CLAY			
PROJECT No.10-1121-0259-2000				FILE No.1011210259-2000-F030A2			
DRAWN		DCH		Jan 12/12		SCALE N/A REV.	
CHECK						FIGURE A-2	
 Golder Associates LONDON, ONTARIO							

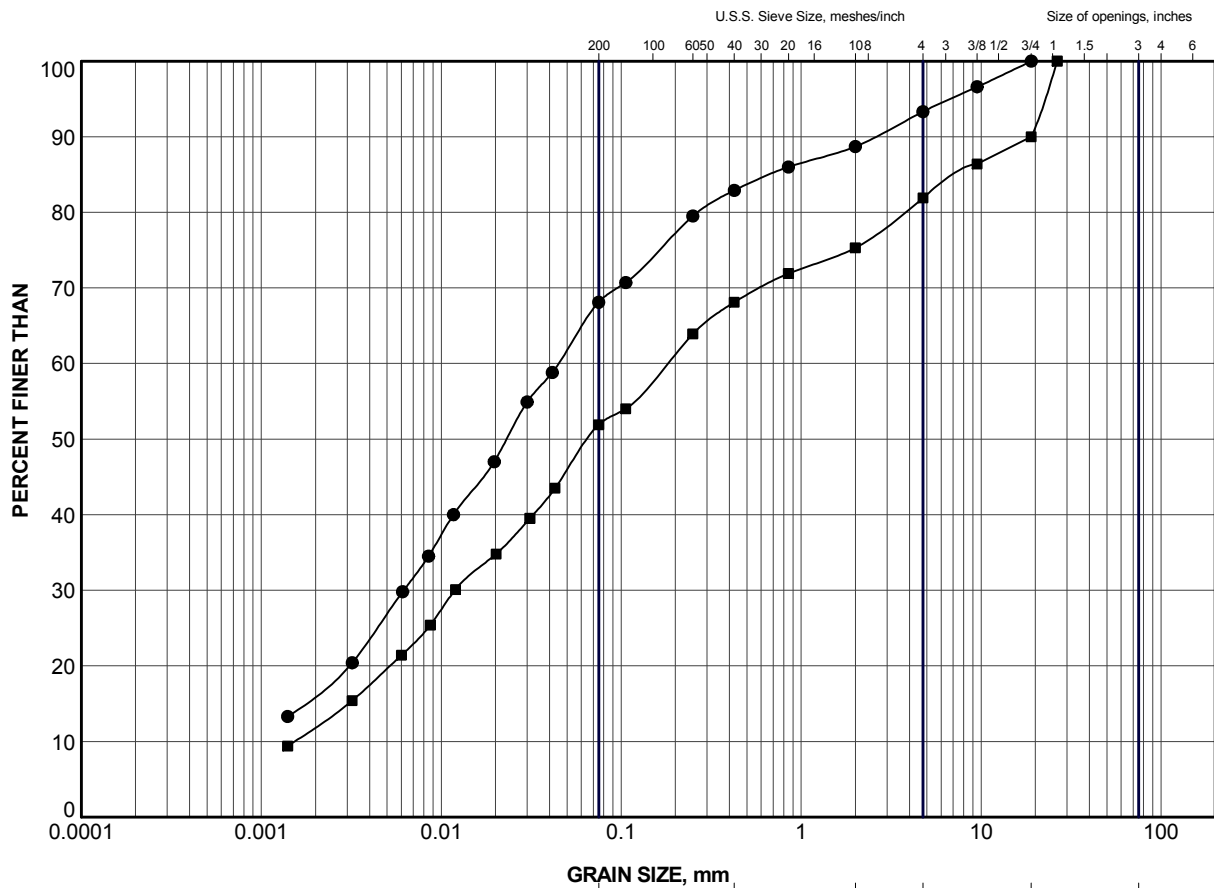


CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND			
SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	11-14	6	52.7

PROJECT				CULVERT C02 REPLACEMENT HIGHWAY 401 GWP 4029-08-00					
TITLE				GRAIN SIZE DISTRIBUTION SILTY SAND, some gravel					
 Golder Associates LONDON, ONTARIO				PROJECT No.10-1121-0259-2000		FILE No.1011210259-2000-F030A3			
				DRAWN	DCH	Jan 12/12	SCALE	N/A	REV.
				CHECK			FIGURE A-3		


LDN_MTO_GSD_GLDR_LDNGDT

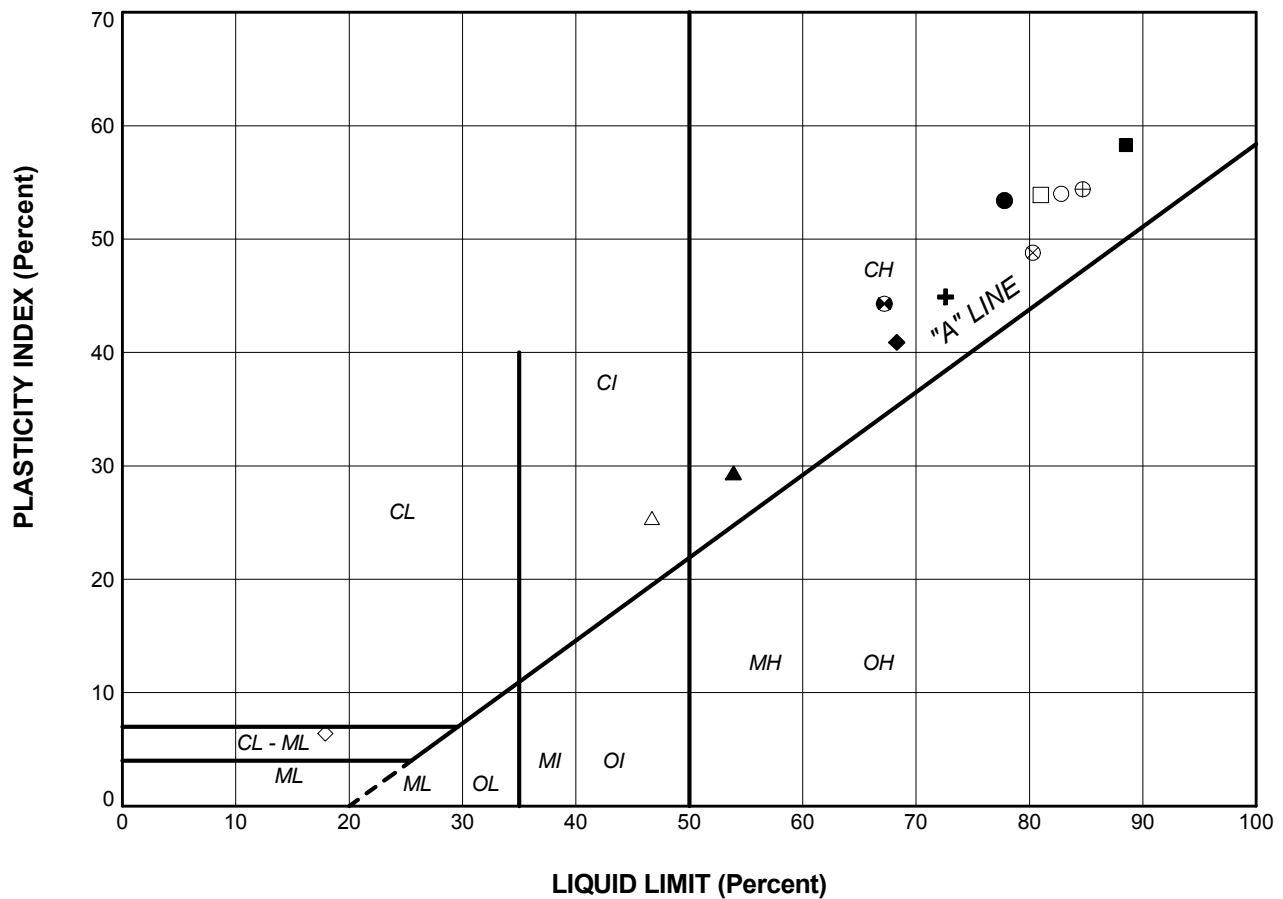


CLAY AND SILT	GRAIN SIZE, mm			Cobble Size
	fine	medium	coarse	
	SAND SIZE		GRAVEL SIZE	

LEGEND


SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	11-15	11	54.0
■	11-16	11	54.3

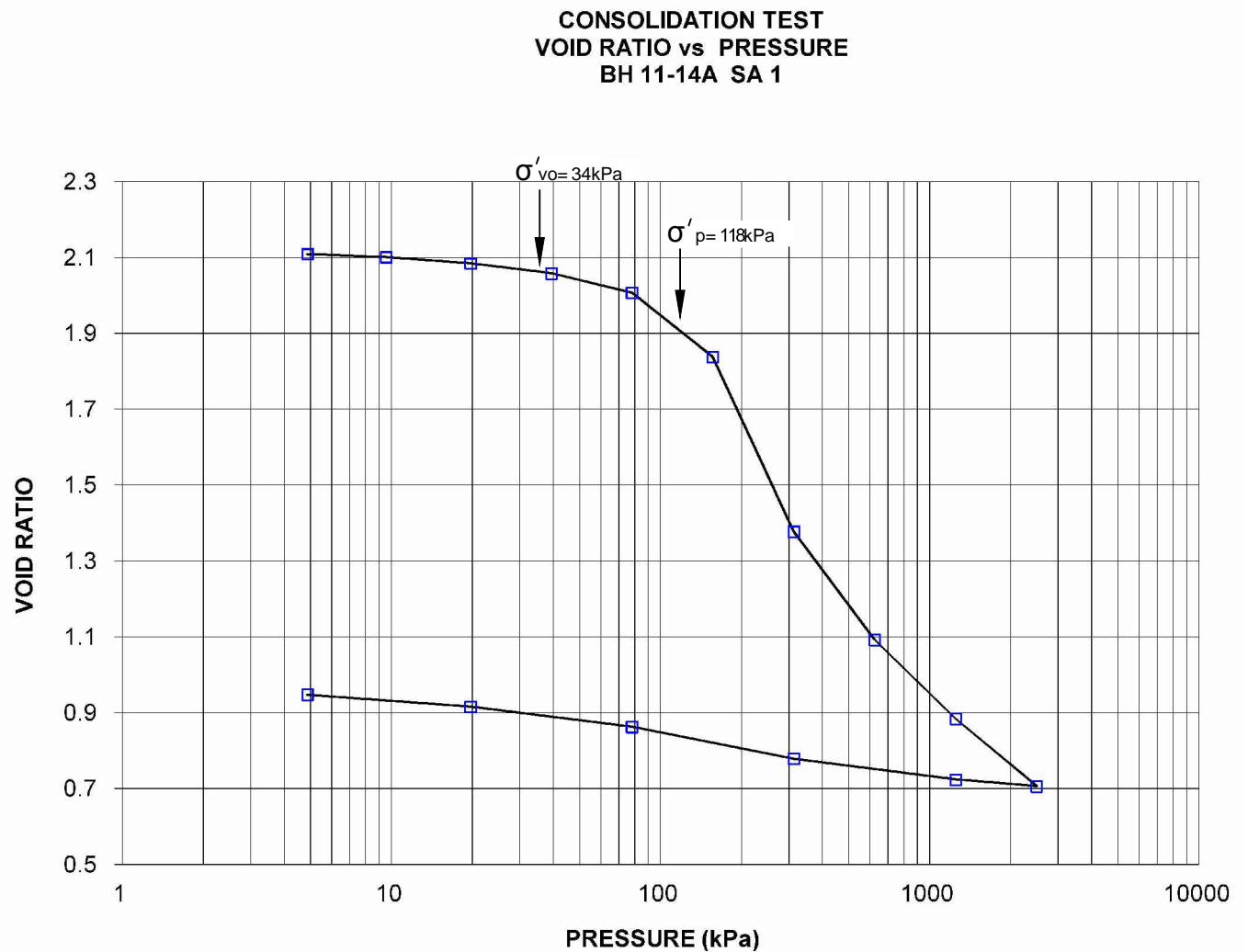
PROJECT				CULVERT C02 REPLACEMENT HIGHWAY 401 GWP 4029-08-00			
TITLE				GRAIN SIZE DISTRIBUTION SILTY SAND to SANDY SILT (TILL)			
PROJECT No.10-1121-0259-2000				FILE No. 1011210259-2000-F030A4			
DRAWN		DCH		Jan 12/12		SCALE N/A	
CHECK						REV.	
 Golder Associates LONDON, ONTARIO				FIGURE A-4			



LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
SILTY CLAY					
●	11-14	3	77.8	24.4	53.4
■	11-14	4	88.5	30.2	58.3
▲	11-14A	1	53.9	24.5	29.4
+	11-15	9	72.6	27.7	44.9
◆	11-15	10	68.3	27.4	40.9
○	11-16	7	82.8	28.8	54.0
△	11-16	9	46.7	21.3	25.4
⊗	11-17	2	80.3	31.5	48.8
⊕	11-17	3	84.7	30.3	54.4
□	11-17	4	81.0	27.1	53.9
●	11-17	5	67.2	22.9	44.3
SILTY SAND to SANDY SILT (TILL)					
◇	11-15	11	17.9	11.5	6.4

PROJECT		CULVERT C02 REPLACEMENT HIGHWAY 401 GWP 4029-08-00	
TITLE		PLASTICITY CHART	
PROJECT No.10-1121-0259-2000		FILE No.1011210259-2000-F030A5	
DRAWN	DCH	Jan 12/12	SCALE N/A REV.
CHECK			
 Golder Associates LONDON, ONTARIO		FIGURE A-5	

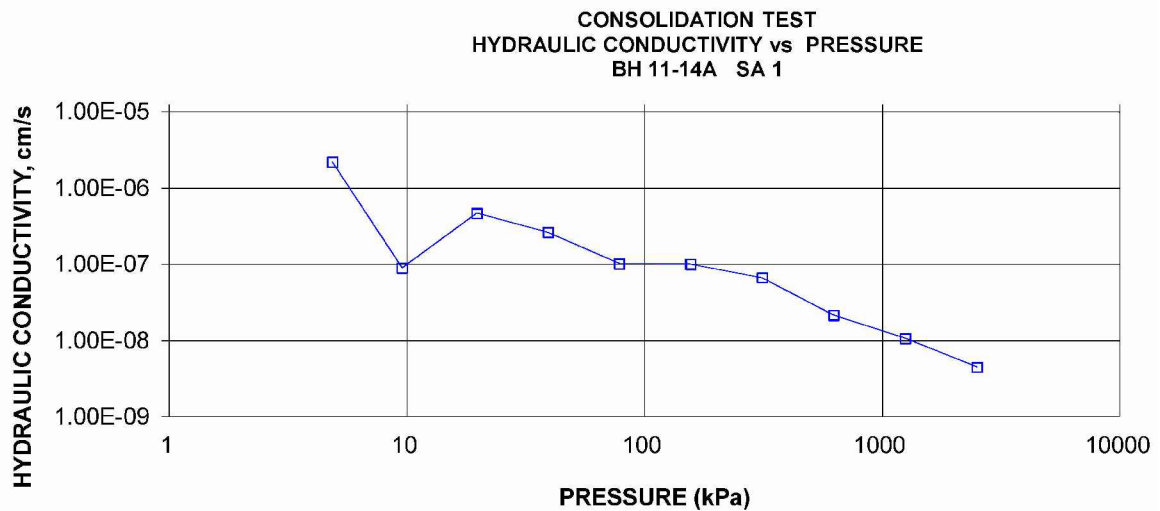
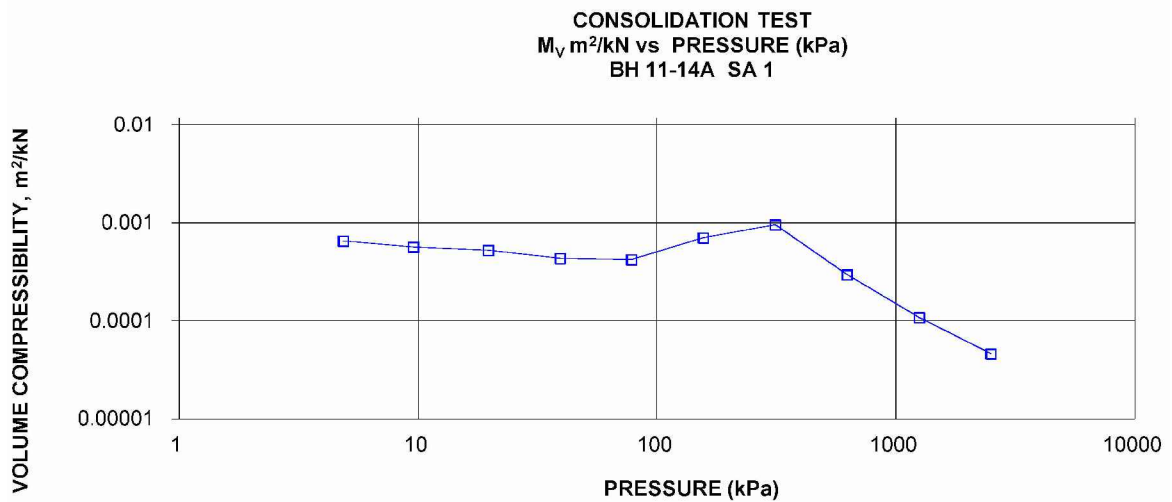
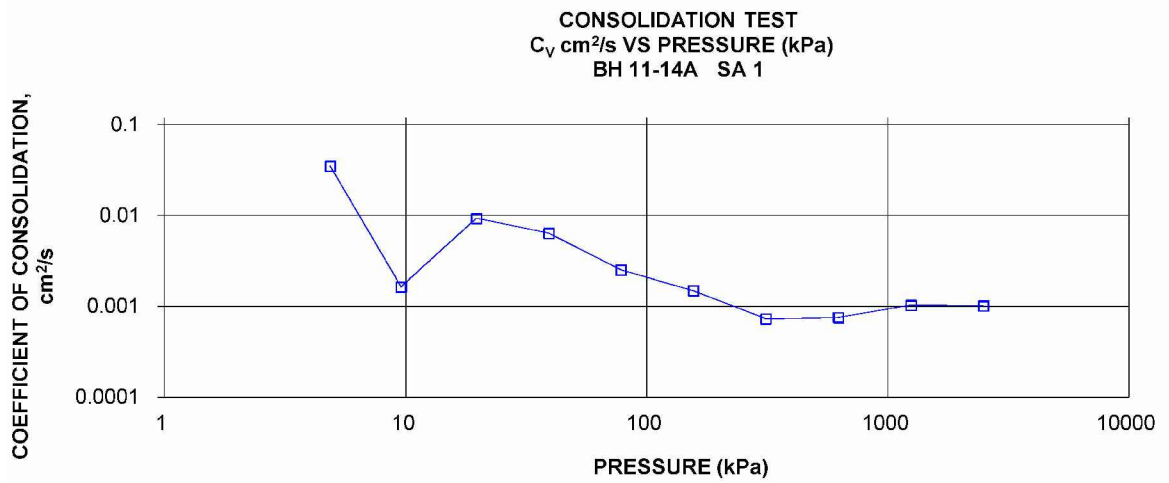


NOTE

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

PROJECT		CULVERT C02 REPLACEMENT HIGHWAY 401 GWP 4029-08-00	
TITLE		CONSOLIDATION TEST (BH 11-14A SA 1)	
PROJECT No.		10-1121-0259	FILE No. 1011210259-2000-F03006
CADD		DCH	Jan. 12/12
CHECK			
			SCALE AS SHOWN REV.
		FIGURE A-6	





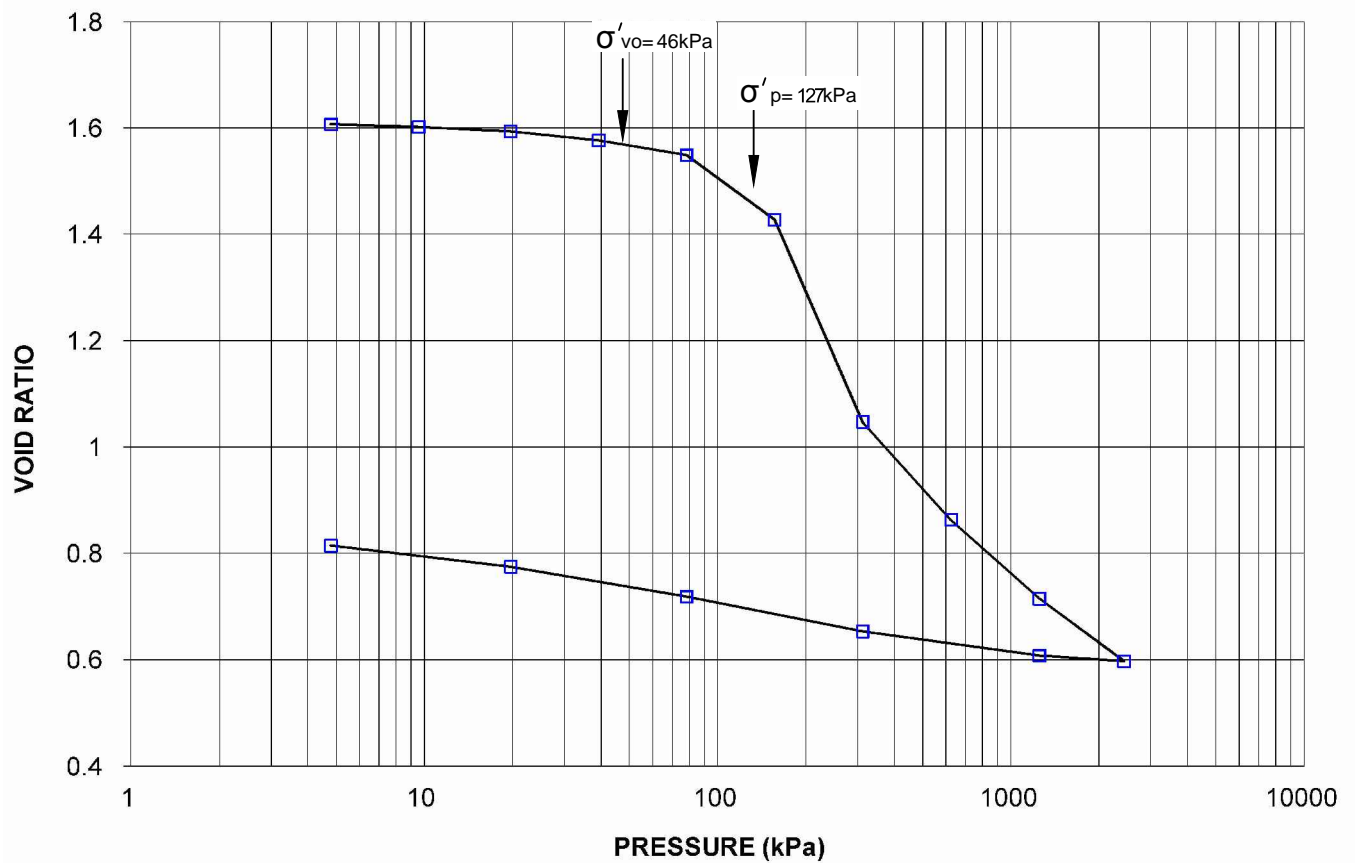
NOTE

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

PROJECT				CULVERT C02 REPLACEMENT HIGHWAY 401 GWP 4029-08-00			
TITLE				CONSOLIDATION TEST SUMMARY (BH 11-14A SA 1)			
PROJECT No.		10-1121-0259		FILE No.		1011210259-2000-F03006	
CADD		DCH		Jan. 12/12		SCALE AS SHOWN REV.	
CHECK						FIGURE A-7	




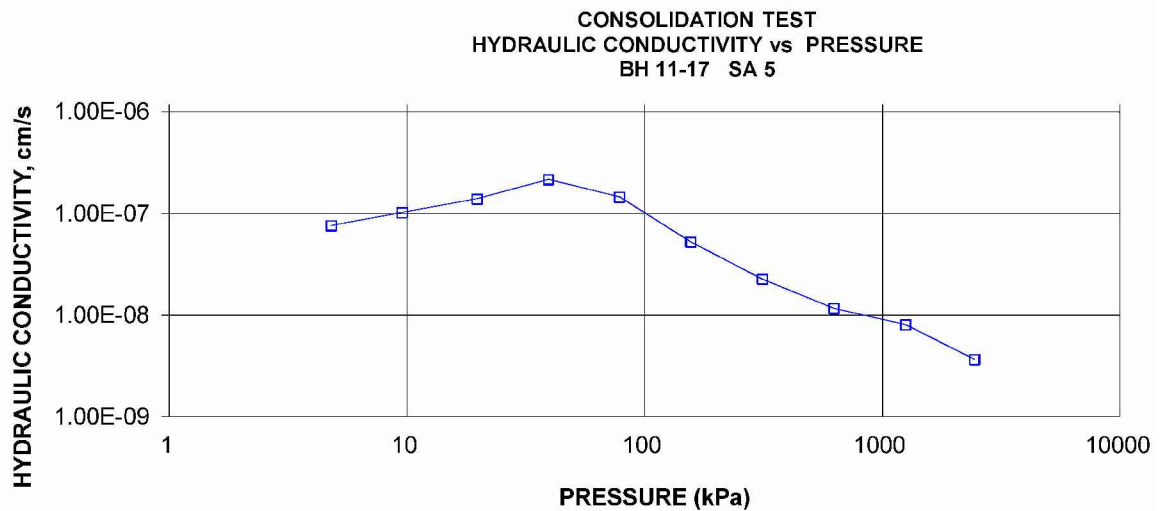
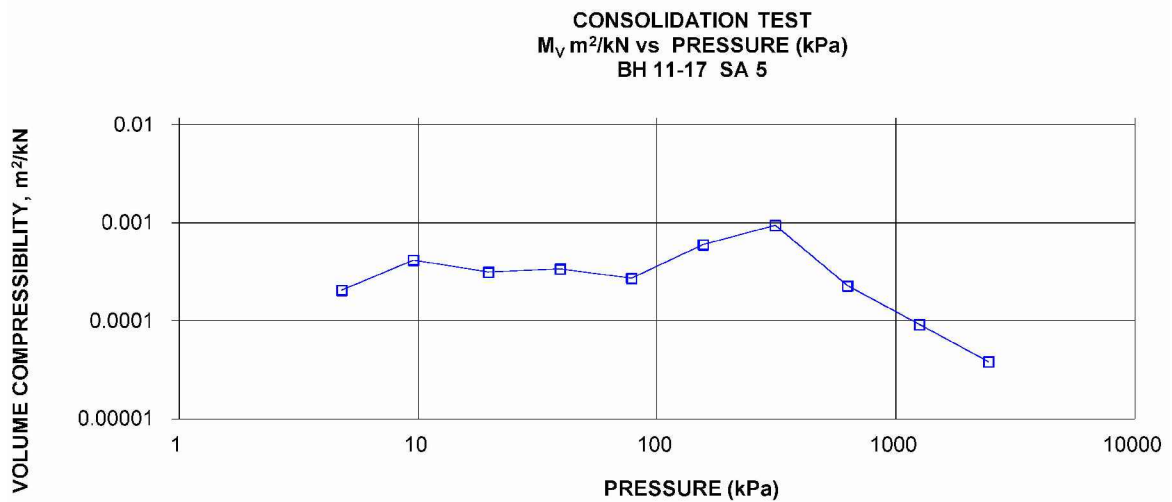
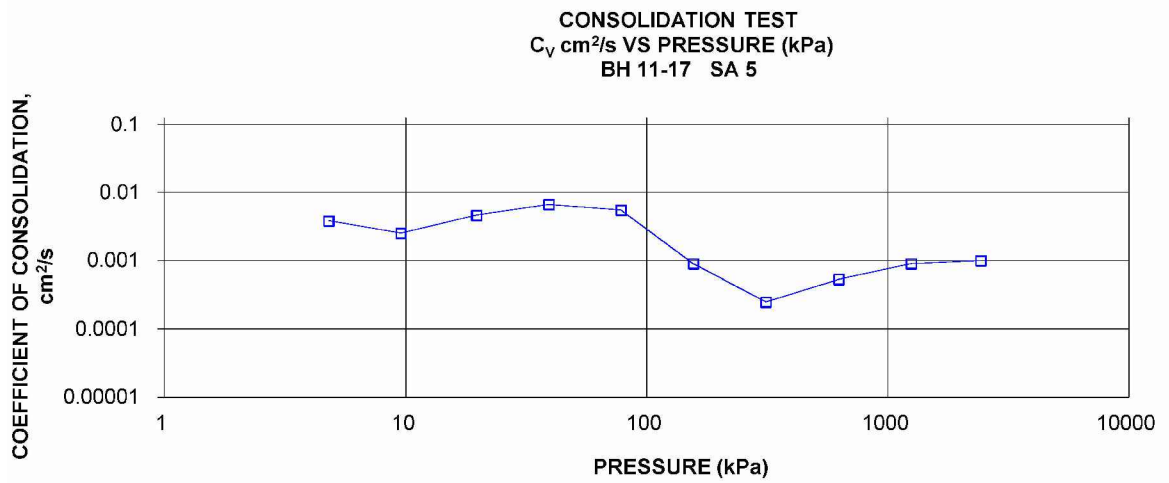
**CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BH 11-17 SA 5**



NOTE


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

PROJECT		CULVERT C02 REPLACEMENT HIGHWAY 401 GWP 4029-08-00			
TITLE		CONSOLIDATION TEST (BH 11-17 SA 5)			
 Golder Associates LONDON, ONTARIO		PROJECT No.	10-1121-0259	FILE No.	1011210259-2000-F03006
		CADD	DCH	Jan. 12/12	SCALE AS SHOWN REV.
		CHECK			
					FIGURE A-8



NOTE

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

PROJECT		CULVERT C02 REPLACEMENT HIGHWAY 401 GWP 4029-08-00	
TITLE		CONSOLIDATION TEST SUMMARY (BH 11-17 SA 5)	
PROJECT No.		10-1121-0259	FILE No. 1011210259-2000-F03006
CADD	DCH	Jan. 12/12	SCALE AS SHOWN REV.
CHECK			
 Golder Associates LONDON, ONTARIO		FIGURE A-9	



APPENDIX B

Site Photographs



APPENDIX B PHOTOGRAPHS



Photograph 1: Drilling borehole 14 at south (outlet) end of Culvert C02.



Photograph 2: Looking upstream from outlet.



APPENDIX B PHOTOGRAPHS



Photograph 3: Drilling borehole 11-17 at north (inlet) end of Culvert C02.

\\ott1-s-filesrv1\data\active\2010\1121 - geotechnical\10-1121-0259 mrc cornwall centre rd\foundations\6 - reports\r03 culvert c02\1011210259-2000-r03 jun 12 12 (final) app b-photos.docx



APPENDIX C

Non-Standard Special Provisions



APPENDIX C

NSSP - GROUNDWATER CONTROL

GROUNDWATER CONTROL, Item No.

Non-Standard Special Provision

SCOPE

Foundations for the replacement of Culvert C02 will require excavations to extend below the groundwater level. Excavations for the replacement culvert are expected to terminate within the silty clay. Although seepage volumes from this material are expected to be low, 'perched' groundwater may be present in the base of the more permeable highway embankment fill. Therefore some seepage into the excavation from the granular fill should be expected. Depending on where the perched groundwater level is at the time of construction, unwatering may be required during the construction of cast-in-place elements or placement of bedding and backfill for Culvert C02 to minimize groundwater inflow into the excavation and construction area as well as to ensure construction is carried out in dry conditions. The Contractor is to design and install an appropriate unwatering system for the culvert site to prevent disturbance of the founding soils and enable construction in dry conditions.

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

BASIS OF PAYMENT

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

\\ott1-s-filesrv1\data\active\2010\1121 - geotechnical\10-1121-0259 mrc cornwall centre rd\foundations\6 - reports\r03 culvert c02\1011210259-2000-r03 jun 12 12 - (final) app c - dewatering.docx



APPENDIX C

NSSP - WORKING SLAB

WORKING SLAB, Item No.

Non-Standard Special Provision

SCOPE

This Special Provision covers the requirements for the supply and placement of the concrete working slab under the structure foundations. The purpose of the working slab is to protect the subgrade from disturbance and loosening due to construction traffic and ponded water and also to provide a level working surface.

CONSTRUCTION

Following inspection and approval of the prepared subgrade, a working slab with a minimum thickness of 75 mm shall be placed on the foundation subgrade as per the contract drawings and documents. The concrete shall have a minimum 28-day compressive strength of 10 MPa.

Unwatering of the excavation for the footing construction, including the construction of the working slab, might be required and is covered under separate Tender Item. The dewatering scheme shall be done in such a manner as to prevent any disturbance to the surrounding original soil.

BASIS OF PAYMENT

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

\\ott1-s-filesrv1\data\active\2010\1121 - geotechnical\10-1121-0259 mrc cornwall centre rd\foundations\6 - reports\r03 culvert c02\1011210259-2000-r03 jun 12 12 (final) app c - working slab.docx

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Australasia	+ 61 3 8862 3500
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North America	+ 1 800 275 3281
South America	+ 55 21 3095 9500

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