



November 2014

FOUNDATION INVESTIGATION AND DESIGN REPORT

**Culvert Replacement, Site No. 9-159/C
Station 31+675, Highway 3
Contract 2 Structure Replacements and Rehabilitation
GWP 3040-11-00
Ministry of Transportation, West Region**

Submitted to:

Mr. Adam Barg, P.Eng.
Stantec Consulting Ltd.
200 - 835 Paramount Drive
Stoney Creek, Ontario
L8J 0B4

REPORT



Report Number: 12-1132-0163-2000-R08

Geocres No.: 30L13-22

Distribution:

8 Copies - Stantec Consulting Ltd.

2 Copies - Golder Associates Ltd.





Table of Contents

PART A - FOUNDATION INVESTIGATION REPORT

1.0 INTRODUCTION.....	1
2.0 SITE DESCRIPTION.....	2
2.1 Site Geology	2
3.0 INVESTIGATION PROCEDURES	3
4.0 SUBSURFACE CONDITIONS.....	4
4.1 Site Stratigraphy	4
4.2 Soil Conditions.....	4
4.2.1 Topsoil	4
4.2.2 Pavement Structure	4
4.2.3 Fill	4
4.2.4 Silty Clay	5
4.2.5 Clayey Silt Till	5
4.2.6 Silty Clay Till	5
4.2.7 Sandy Silt Till	6
4.3 Groundwater Conditions	6
5.0 MISCELLANEOUS	8

PART B - FOUNDATION DESIGN REPORT

6.0 ENGINEERING RECOMMENDATIONS.....	9
6.1 General.....	9
6.2 Replacement Culvert	9
6.2.1 Foundations	9
6.2.2 Bedding.....	11
6.2.3 Backfill and Cover	12
6.2.4 End Treatments and Camber.....	12
6.2.5 Minimum Clear Spacing	12
6.3 Liquefaction Potential and Seismic Analysis.....	12
6.3.1 Seismic Parameters	12



FOUNDATION INVESTIGATION AND DESIGN REPORT CULVERT REPLACEMENT, SITE 9-159/C, HIGHWAY 3

6.3.2	Seismic Hazard Assessment	13
6.4	Lateral Earth Pressures for Design.....	13
6.5	Construction Considerations.....	14
6.5.1	General	14
6.5.2	Erosion and Scour Protection	15
6.6	Excavations and Groundwater Control	15
6.7	Staging and Temporary Roadway Protection	15
7.0	MISCELLANEOUS	17

TABLE I – Comparison of Culvert Alternatives

LIST OF ABBREVIATIONS

LIST OF SYMBOLS

RECORD OF BOREHOLE SHEETS

FIGURE 1 - Key Plan

DRAWING 1 - Borehole Locations and Soil Strata

APPENDICES

APPENDIX A

Laboratory Test Data

APPENDIX B

Site Photographs



PART A

FOUNDATION INVESTIGATION REPORT

CULVERT REPLACEMENT, HIGHWAY 3

SITE NO. 9-159/C, STATION 31+675

CONTRACT 2 STRUCTURE REPLACEMENTS AND REHABILITATION

GWP 3040-11-00

MINISTRY OF TRANSPORTATION - WEST REGION



1.0 INTRODUCTION

Golder Associates Ltd. (Golder Associates) has been retained by Stantec Consulting Ltd. (Stantec) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations as part of the detail design work for GWP 3040-11-00. The project involves the detail design of the replacement and rehabilitation of several structures along multiple highways in Southern Ontario.

This report addresses the proposed replacement of the culvert at Site 9-159/C at Station 31+675 on Highway 3 just east of Canfield in the Township of North Cayuga, Haldimand County, Ontario.

The purpose of the foundation investigation is to explore the subsurface conditions at the location of the proposed structure replacement by drilling boreholes and carrying out in situ testing and laboratory testing on selected samples. The terms of reference for the scope of work are outlined in the MTO's Request for Proposal and in Golder Associates' proposal P2-1132-0163 dated February 25, 2013. The work was carried out in accordance with our Quality Control Plan for Foundation Engineering dated March 26, 2013.



FOUNDATION INVESTIGATION AND DESIGN REPORT CULVERT REPLACEMENT, SITE 9-159/C, HIGHWAY 3

2.0 SITE DESCRIPTION

The subject culvert is situated at Station 31+675 on Highway 3, approximately 0.4 kilometres west of Haldimand-Dunn Townline in the Township of North Cayuga, Haldimand County, Ontario. The villages of Canfield and Canborough are 3.2 kilometres west and 3.0 kilometres east of the site, respectively. The location of the culvert is shown on the Key Plan, Figure 1.

This section of Highway 3 is currently a two lane undivided highway with gravel shoulders. It is generally oriented east-west in the vicinity of the subject site. A tributary of Oshwego Creek flows from north to south beneath Highway 3. The existing culvert is a concrete non-rigid frame open footing (NRFO) structure with the following characteristics, including a 2.5 metre long rigid frame box (RFB) extension at each end:

Dimensions (m)	Obvert Elevation (m)		Existing Construction
	Lt ¹	Rt ¹	
3.10 x 1.80 x 22.66	180.63	180.57	17.68 m NRFO two 2.5 m RFB extensions

NOTE: 1. When facing the direction of increasing chainage, Lt and Rt are defined as Left and Right of centreline, respectively.

The banks of the creek channel upstream and downstream of the culvert are grass covered and the channel flows through fields adjacent to Highway 3. Site photographs are provided in Appendix B.

The culvert is situated in a valley in a rural agricultural area. Ground surface elevations in the vicinity of the culvert site range from about 182 to 185 metres.

2.1 Site Geology

The project area is located within the Haldimand Clay Plain physiographic region. This region is characterized by a mixture of stratified clay and till.¹ The quaternary geological mapping indicates that surficial soils consist of glaciolacustrine clay and silt at the vicinity of the site.² Geological mapping also indicates that the underlying bedrock consists of tan dolomite with lenses of anhydrite and gypsum of the Salina Formation of Middle to Lower Silurian age.³ The bedrock surface at the site is at about elevation 167.0 to 168.0 metres, with the overburden thickness being about 12 to 13 metres.⁴

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

² Feenstra, B.H. 1974: Quaternary Geology of the Dunnville Area, Southern Ontario; Ontario Div. Mines, Prelim. Map P.981, scale 1:50,000.

³ Sanford, B.V., 1969: Geology Toronto – Windsor Area; Geological Survey of Canada Map 1263A, Scale 1:250,000.

⁴ Feenstra, B.H., 1981: Bedrock Topography of the Dunnville Area, Southern Ontario; Ontario Geological Survey Preliminary Map P.2402, Bedrock Topography Series. Scale 1:50,000.



3.0 INVESTIGATION PROCEDURES

The field work for the investigation was carried out on July 2, 2013, during which time 3 boreholes were drilled at the locations shown on the Borehole Location Plan, Drawing 1. The boreholes were drilled using track-mounted CME 75 drilling equipment supplied and operated by a specialist drilling contractor. Samples of the overburden were typically obtained at depth intervals of 0.75 to 1.5 metres using 50 millimetre outside diameter split spoon sampling equipment in accordance with the Standard Penetration Test (SPT) procedures (ASTM D1586).

The recorded SPT N values are noted on the Record of Borehole sheets. The SPT resistance, or N value, is defined as the number of blows required by a 63.5 kilogram hammer dropped from a height of 760 millimetres to drive a split-spoon sampler a distance of 300 millimetres after an initial 150 millimetres of penetration. The results of the SPT testing as presented on the Record of Borehole sheets, Drawing 1 and in Section 4 are unmodified (not standardized for hammer efficiency, borehole diameter, rod length, etc.). The samplers used in the investigations limit the maximum particle size that can be sampled and tested to about 40 millimetres. Therefore, particles or objects that may exist within the soils that are larger than this dimension will not be sampled or represented in the grain size distributions.

Groundwater conditions in the boreholes were observed throughout the drilling operations and a piezometer was installed in borehole 202 as indicated on the corresponding Record of Borehole sheet. The boreholes were backfilled in accordance with current MTO procedures and Ontario Regulation 903 (as amended).

The field work was monitored on a full-time basis by an experienced member of our staff who located the boreholes in the field, monitored the drilling, sampling, and in situ testing operations and logged the boreholes. The samples were identified in the field, placed in labelled containers, and transported to our London laboratory for further examination and testing. Index and classification tests, consisting of water content determinations, Atterberg limits and grain size distribution analyses, were carried out on selected samples. The results of the testing are shown on the Record of Borehole sheets and in Appendix A.

The as-drilled borehole locations and ground surface elevations are shown on the Record of Borehole sheets and on Drawing 1. The table below summarizes the coordinates, ground surface elevations, and depths of the boreholes.

Borehole	Location (m)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting		
201	4 760 447	287 329	179.95	8.08
202	4 760 433	287 359	180.04	9.60
203	4 760 434	287 344	181.56	11.13



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 attached Record of Borehole sheets following the text of this report and in Appendix A. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous samples and observations of drilling resistance and, therefore, may 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 boreholes drilled at the site generally encountered the existing pavement structure or topsoil overlying variable embankment fill materials, then in sequence, silty clay, silty clay till, clayey silt till and sandy silt till.

The locations and elevations of the boreholes, together with the interpreted stratigraphic profile, are shown on Drawing 1. A detailed description of the subsurface conditions encountered in the boreholes is provided on the Record of Borehole sheets and is summarized in the following sections.

4.2 Soil Conditions

4.2.1 Topsoil

Topsoil was encountered in boreholes 201 and 202 at the ground surface. The topsoil layers were about 180 and 120 millimetres thick, respectively.

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

4.2.2 Pavement Structure

Pavement granular base material was encountered in borehole 203 at the surface of the roadway shoulder. The granular base material was about 240 millimetres thick. Pavement granular subbase material was encountered beneath the granular base in borehole 203. The granular subbase material was about 400 millimetres thick.

4.2.3 Fill

Silty clay fill materials were encountered beneath the pavement structure in borehole 203 from elevation 180.9 metres and beneath the topsoil in borehole 202 from elevation 179.9 metres. The silty clay fill materials were 1.3 to 1.9 metres thick with standard penetration test N values of 4 to 6 blows per 0.3 metres. Samples of the silty clay fill had water contents of about 14 to 32 per cent.

The silty clay fill material is of intermediate plasticity based on the Atterberg limits determination carried out on a sample obtained during standard penetration testing. The plastic limit was 19 per cent, the liquid limit was 37 per cent and the plasticity index was 18 per cent. The Atterberg limits data for the silty clay fill material are



presented on Figure A-6. A grain size distribution curve for a sample of the fill material is provided on Figure A-1.

4.2.4 Silty Clay

Layers of firm to stiff silty clay were encountered beneath the fill in boreholes 202 and 203 at elevations 178.7 and 179.0 metres, respectively, and beneath the clayey silt till in borehole 201 at elevation 172.6 metres. Borehole 201 was terminated in the silty clay after exploring the layer for 0.8 metres. Where fully penetrated in boreholes 202 and 203, the silty clay was 1.1 to 2.3 metres thick. Measured N values for the silty clay layers were 4 to 11 blows per 0.3 metres. Water contents of the samples ranged from 28 to 39 per cent.

The silty clay is of intermediate to high plasticity based on the Atterberg limits determinations carried out on two samples obtained during standard penetration testing. The plastic limits were 21 and 25 per cent, the liquid limits were 41 and 50 per cent and the plasticity indices were 21 and 25 per cent. The Atterberg limits data for the silty clay are presented on Figure A-6. Grain size distribution curves for samples of the silty clay are provided on Figure A-2.

4.2.5 Clayey Silt Till

Very stiff to hard clayey silt glacial till was encountered beneath the silty clay in borehole 203 at elevation 177.9 metres and beneath the silty clay till in borehole 201 at elevation 174.5 metres. The clayey silt till was 1.5 to 1.8 metres thick. Measured N values for the clayey silt till layers were 20 to 33 blows per 0.3 metres. The clayey silt till had a water content of about 11 per cent. Cobbles and boulders were encountered in the clayey silt glacial till in boreholes 201 and 203. The presence of cobbles and boulders should be expected in this deposit.

The clayey silt till is of low plasticity based on the Atterberg limits determination carried out on a sample obtained during standard penetration testing. The plastic limit was 13 per cent, the liquid limit was 23 per cent and the plasticity index was 10 per cent. The Atterberg limits data for the clayey silt till are presented on Figure A-6. A grain size distribution curve for a sample of the clayey silt till is provided on Figure A-3.

4.2.6 Silty Clay Till

Layers of silty clay glacial till were encountered beneath the topsoil in borehole 201 at elevation 179.8 metres, beneath the silty clay in borehole 202 at elevation 176.4 metres and beneath the clayey silt till in borehole 203 at elevation 176.4 metres. The silty clay till was 3.7 to 5.3 metres thick. Measured N values for the firm to very stiff silty clay till layers were 5 to 19 blows per 0.3 metres. The shear strength of the softer silty clay till in borehole 202 was measured with an in situ field shear vane and was found to be greater than 144 kilopascals. Water contents of the samples ranged from 17 to 34 per cent. Cobbles and boulders were encountered in the silty clay glacial till in borehole 201. The presence of cobbles and boulders should be expected in this deposit.

The silty clay till is of intermediate to high plasticity based on the Atterberg limits determinations carried out on samples obtained during standard penetration testing. The plastic limit varied from 18 to 25 per cent, the liquid limit varied from 36 to 54 per cent, and the plasticity index varied from 18 to 29 per cent. The Atterberg limits



data for the silty clay till are presented on Figure A-6. Grain size distribution curves for samples of the silty clay till are provided on Figure A-4.

4.2.7 Sandy Silt Till

Layers of dense to very dense sandy silt glacial till were encountered beneath the silty clay till in boreholes 202 and 203 at elevations 172.7 and 174.6 metres, respectively. Both boreholes were terminated in the sandy silt till after exploring the layer for 2.3 to 4.1 metres. The sandy silt till had measured N values of 40 to 95 blows per 0.3 metres and water contents of about 9 to 10 per cent. Grain size distribution curves for samples of the sandy silt till are provided on Figure A-5. Cobbles were encountered in the sandy silt glacial till in borehole 203. The presence of cobbles and boulders should be expected in this deposit.

4.3 Groundwater Conditions

Groundwater conditions were observed during and on completion of drilling and sampling and a groundwater observation piezometer was installed in borehole 202. Installation details are provided on the corresponding Record of Borehole sheet following the text of this report. Groundwater was encountered in borehole 203 at a depth of 2.9 metres or elevation 178.7 metres. A summary of the encountered and measured groundwater levels is provided in the table below.

Borehole	Ground Surface Elevation (m)	Encountered Groundwater Elevation (m)	Measured Groundwater Level Elevation (m)		
			July 2, 2013	Sept. 5, 2013	April 30, 2014
201	179.95	*	-	-	-
202	180.04	*	172.39	175.47	180.52**
203	181.56	178.7	-	-	-

*Groundwater level not established.

**Area surrounding piezometer flooded and cap found missing.

The above-noted encountered and measured water levels are not considered to be representative of the long-term, stabilized groundwater conditions. The corresponding water level in the watercourse was measured at elevation 179.5 metres on July 2, 2013 and at elevation 180.2 metres on April 30, 2014. On September 5, 2013 the water level in the groundwater observation standpipe installed in borehole 202 was about 4.6 metres below the ground surface or at about elevation 175.5 metres. On April 30, 2014 the water level in the groundwater observation standpipe installed in borehole 202 was measured in the pipe stickup about 0.5 metres above the ground surface or at elevation 180.5 metres. At the time of the April 30, 2014 reading, the area surrounding the culvert was flooded and the standpipe was partially submerged with only a portion of the aboveground stickup visible. Since the cap was missing from the pipe, this reading is considered suspect. A heavy precipitation event occurred prior to the reading.



FOUNDATION INVESTIGATION AND DESIGN REPORT CULVERT REPLACEMENT, SITE 9-159/C, HIGHWAY 3

Based on the observed groundwater levels, the surrounding topography, the soil colour change from brown to grey and water levels in the drain, the inferred groundwater level has been assumed to be elevation 179.0 metres for design purposes. The groundwater levels are expected to fluctuate seasonally and are expected to be higher during periods of sustained precipitation or during spring snow melt conditions and will be influenced by flows in the watercourse.



5.0 MISCELLANEOUS

The investigation was carried out using equipment supplied and operated by Aardvark Drilling Inc., an Ontario Ministry of Environment licensed well contractor. The field operations were supervised by Mr. Brett Thorner, E.I.T. under the direction of Mr. David J. Mitchell, the Site Investigation Field Manager. The laboratory testing was carried out at Golder Associates' London laboratory under the direction of Mr. Chris M. Sewell. The laboratory is an accredited participant in the MTO Soil and Aggregate Proficiency Program and is certified by the Canadian Council of Independent Laboratories for testing Types C and D aggregates. This report was prepared by Mr. Brett Thorner, E.I.T. under direction of Ms. Dirka U. Prout, P.Eng. and reviewed by Mr. W. Michael Kellestine, P.Eng., a Principal with Golder Associates. Mr. Fintan J. Heffernan, P.Eng., the Designated MTO Contact and Quality Control Auditor for this assignment, conducted an independent quality review of the report.

GOLDER ASSOCIATES LTD.

ORIGINAL SIGNED

Dirka U. Prout, P.Eng.
Project Engineer

ORIGINAL SIGNED

W.M. Kellestine, P.Eng.
Principal

ORIGINAL SIGNED

Fintan J. Heffernan, P.Eng.
MTO Designated Contact

BT/DB/DUP/WMK/FJH/cr

Golder, Golder Associates and the GA globe design are trademarks of Golder Associates Corporation.

n:\active\2012\1132 - geo\1132-0100\12-1132-0163 stantec-fdns-mega culverts-3011-e-0041\ph 2000-gwp 3040-11-00\rpts\r08 - site 9-159\final\1211320163-2000-r08 nov 5 14 (final) part a&b fdns repl clvrt 9-159-c.docx



**FOUNDATION INVESTIGATION AND DESIGN REPORT
CULVERT REPLACEMENT, SITE 9-159/C, HIGHWAY 3**

PART B

FOUNDATION DESIGN REPORT

CULVERT REPLACEMENT, HIGHWAY 3

SITE NO. 9-159/C, STATION 31+675

CONTRACT 2 STRUCTURE REPLACEMENTS AND REHABILITATION

GWP 3040-11-00

MINISTRY OF TRANSPORTATION - WEST REGION



6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides recommendations on the foundation aspects of the design of the replacement culvert at Site 9-159/C, located at Station 31+675 on Highway 3 in the Township of North Cayuga in Haldimand County, Ontario.

The recommendations are based on interpretation of the factual data obtained from the current boreholes advanced at this site. The interpretation and recommendations are intended to provide the designers with sufficient information to design the proposed culvert foundations. As such, where comments are made on construction, they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, and scheduling.

The existing culvert is a 22.7 metre long concrete NRFO structure with a 3.1 metre span, a 1.8 metre high opening, and an approximate invert elevation of 178.8 metres. The main culvert NFRO section is 17.7 metres long. There is a 2.5 metre long RFB section at each end. The existing culvert has approximately 1.0 to 1.5 metres of fill cover.

6.2 Replacement Culvert

Based on information provided by Stantec, it is understood that consideration is being given to replacing the existing RFO culvert with new twin 4.2 by 2.1 metre concrete box culverts with a 60 millimetre gap filled with grout between the culverts or a two cell concrete open footing culvert with 4.2 by 1.8 metre cells. It has been indicated by Stantec that the preferred structural alternative involves use of concrete box culverts that may be precast or cast-in-place (CIP). Retaining wing walls were not required at the replacement culvert location from a structural engineering perspective. The design length of the new culvert has been established such that the embankments can be graded to avoid the use of retaining walls. In addition, the existing guide rail at this site will be reinstated which helps reduce the length of the culvert. The embankment side slopes were found to be stable and no erosion concerns have been identified, so the retaining wing walls were not considered necessary from a geotechnical engineering perspective. No grade raise is proposed at this location. The proposed invert elevations are 178.53 metres at the inlet and 178.47 metres at the outlet. A comparison of the various culvert types is presented on Table I following the report text.

6.2.1 Foundations

The subsurface conditions encountered during the investigation generally consisted of the existing pavement structure or surficial topsoil overlying variable embankment fill materials to elevations of between 178.7 and 179.8 metres. The fill at boreholes 202 and 203 was underlain by firm to stiff silty clay to elevation 176.4 to 177.9 metres, then glacial till. In borehole 201, the topsoil was underlain by glacial till to elevation 172.6 metres then firm silty clay. The glacial till was variable in composition but primarily consisted of stiff to very stiff silty clay till or very stiff to hard clayey silt till. Dense to very dense sandy silt till was present at depth in boreholes 202



and 203. The inferred groundwater level is at elevation 179.0 metres. The water level in the watercourse was about elevation 179.5 metres at the time of the investigation.

The culvert replacement should be designed to withstand the appropriate vertical weight of fill and traffic loading. It is not necessary to found a box culvert at the standard depth for frost protection purposes as this type of structure is tolerant of small magnitude movements related to freeze-thaw cycles, should these occur. A box or open footing culvert should, however, be founded below any existing fill and surficial organic materials.

Based on the soil conditions encountered at the borehole locations, and assuming that the design culvert invert elevations will be similar to those of the existing culvert, the replacement box or open footing culvert may be founded on the stiff silty clay, stiff to very stiff silty clay till or very stiff to hard clayey silt till at or below elevation 177.9 metres. Any observed fill or organic materials should be removed to the native soils. Any low areas should be brought to design grade using lean concrete fill or well graded granular materials.

Geotechnical Resistances

Assuming a floor slab thickness of 350 millimetres for a box culvert and allowing for a minimum bedding thickness of 300 millimetres and a 75 millimetre thick levelling pad, the maximum founding elevation for a box culvert will be near elevation 177.8 metres. The majority of the box culvert units/segments will likely be founded in stiff silty clay till and hard clayey silt till with those at the south end founded in stiff silty clay. A factored geotechnical resistance at Ultimate Limit States (ULS) of 300 kilopascals and a geotechnical reaction at Serviceability Limit States (SLS) of 200 kilopascals are recommended for box culvert foundations founded near elevation 177.8 metres.

Allowing for a frost depth of 1.2 metres for open footing culvert foundations, the maximum founding elevation will be elevation 177.3 metres. Stiff to very stiff silty clay till or very stiff clayey silt till will be the predominant materials exposed in footing excavations for an open footing structure. Stiff silty clay is expected to be the founding material at the south end. A factored geotechnical resistance at ULS of 350 kilopascals and a geotechnical reaction at SLS of 225 kilopascals are recommended for open footing foundations constructed at or below elevation 177.3 metres. These values assume that foundations have a minimum width of 0.5 metres and that the subgrade has been properly prepared (see Section 6.6). The SLS values correspond to a maximum of 25 millimetres of total settlement for new culvert construction.

Frost Treatment and Scour Protection

Frost treatment in the form of a frost taper symmetrical about the culvert centreline must be provided in accordance with Ontario Provincial Standard Drawing (OPSD) 803.010 for a box or open footing culvert. The design frost penetration depth for this area is 1.2 metres below ground surface. The culvert base should be adequately protected against scour as noted in Section 1.9.5.2 of the Canadian Highway Bridge Design Code (CHBDC). Scour protection for the culvert backfill, bedding, and stream bank should be provided to protect the roadway, approach embankments, and culvert approaches.



Resistance to Lateral Forces/Sliding Resistance

The resistance to lateral forces/sliding resistance between the base of the replacement box culvert and the bedding or concrete open footing and native soils should be calculated in accordance with Section 6.7.5 of the CHBDC.

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} or H_{rs} , as follows:

$$H_{ri} = 0.8A'c' + 0.8V\tan\delta > H_f \text{ (for box culverts)}$$

$$H_{rs} = 0.8A'c' + 0.8V\tan\phi' > H_f \text{ (for open footing culverts)}$$

where:

A'	-	effective contact area, square metres
c'	=	Nil
$\tan \delta$	-	coefficient of friction for interface between box culvert base and bedding/levelling pad
$\tan \phi'$	-	coefficient of internal friction for soil close to the underside of the spread/strip footing
V	-	unfactored vertical force, kilonewtons
H_f	-	unfactored horizontal load, kilonewtons

The factored horizontal resistance may be calculated using the following parameters in the following table:

Structure	Interaction	Angle of Friction, δ (degrees)	Coefficient of Friction, $\tan \delta$	Effective Angle of Internal Friction, ϕ' (degrees)	Coefficient of Internal Friction, $\tan \phi'$
CIP Box or Open Footing Culvert	CIP concrete on native silty clay	-	-	26	0.49
	CIP concrete on native silty clay till	-	-	28	0.53
	CIP concrete on native clayey silt till	-	-	32	0.62
Precast Box Culvert	Precast concrete on Granular A bedding/levelling pad	30	0.58	-	-

6.2.2 Bedding

For precast box culverts, bedding should be placed above a properly prepared subgrade from which all frozen, soft, uncompacted fill, organic materials, or other deleterious materials have been removed. Subexcavated material below the design subgrade elevation should be replaced with compacted Ontario Provincial Standard Specification (OPSS) Granular B Type II or Type III. It is recommended that the box culvert units be placed on a minimum thickness of 300 millimetres of Granular A bedding material and a minimum 75 millimetre thick levelling course consisting of uncompacted Granular A or fine aggregates as specified in MTO Special Provision (SP) 422S01.



6.2.3 Backfill and Cover

Backfill, cover and construction of the frost taper (backfill transition) should be completed in accordance with OPSD 803.010 for a concrete box or open footing culvert. The excavation for the culvert replacement should exceed the culvert dimensions by at least one metre on each side to promote good workmanship and effective compaction of the fill.

The backfill should consist of free-draining, non-frost susceptible granular materials such as OPSS Granular A or Granular B Type II or III placed and compacted in accordance with SP 105S21 but with less than 5 per cent passing the 0.075 millimetre sieve. All bedding, backfill and cover materials should be placed in accordance with SP 105S21, OPSS 902 and SP 422S01.

Heavy compaction equipment should not be used immediately adjacent to the walls and roof of the culvert. The height of backfill adjacent to the culvert walls should be maintained equal on both sides of the structure during all stages of backfill placement with one side not exceeding the other by more than 500 millimetres.

6.2.4 End Treatments and Camber

The culvert invert will be on the native silty clay, clayey silt till or silty clay till. An outlet filter is not necessary due to the generally cohesive soils that will be present at the outlet, unless the head difference between the culvert inlet and outlet is high. Box culverts, if used, shall be provided with cut-off walls in accordance with CHBDC Section 1.9.5.6. No grade raise is proposed as part of the culvert replacement and relatively low cover is proposed for the replacement culvert; therefore it is not necessary to provide a camber.

6.2.5 Minimum Clear Spacing

The spacing between twin culverts is to be sufficient to permit proper grouting between the culverts. The preliminary general arrangement drawing indicates that if the twin culvert option is adopted, the clearance between culverts will be 60 millimetres and this gap will be filled with grout. This clearance is considered adequate as it meets the requirements of OPSS 422 which requires a minimum spacing of 60 millimetres plus/minus 10 millimetres.

6.3 Liquefaction Potential and Seismic Analysis

6.3.1 Seismic Parameters

The site is located near the Town of Dunnville in southern Ontario. According to Table A.3.1.1 of the CHBDC, the zonal acceleration ratio, A , applicable to this site is 0.05. The corresponding acceleration related seismic zone, Z_a , is 1. 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.

The importance category of the replacement culvert is “other” based on the current version of the CHBDC. The corresponding seismic performance zones (SPZ) to this importance category is 1. Structural culverts situated in SPZ 1 need not be analyzed for seismic loads. However, design forces for restraining elements and support lengths must meet the minimum requirements as outlined in CHBDC Clause 4.4.5.1. It should be noted that the



MTO views culverts with spans of 3 metres or greater as being similar to bridges. The designer should ensure that the selected culvert design meets the seismic requirements for buried structures as outlined in Clause 7.5.5 of the CHBDC.

6.3.2 Seismic Hazard Assessment

A preliminary screening of the soil stratigraphy was conducted using the procedure outlined in the Federal Highway Administration recommended procedures⁵ and Canadian Foundation Engineering Manual (CFEM). The soils at this site are not considered to be susceptible to liquefaction. Therefore, a detailed evaluation of the liquefaction potential of the foundation soils is not considered warranted.

6.4 Lateral Earth Pressures for Design

The lateral pressures acting on the proposed culverts will depend on the type and method of placement of the backfill materials, on the nature of the soil behind the backfill, on the magnitude of surcharge including construction loadings, 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 walls in accordance with the current CHBDC. It should be noted that these design recommendations and parameters assume 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 Granular A, Granular B Type II or Granular B Type III but with less than 5 per cent passing the 0.075 millimetre sieve should be used as backfill behind the walls. The fill should be compacted in loose lifts not greater than 200 millimetres in thickness. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill.
- A compaction surcharge equal to 12 kilopascals should be included in the lateral earth pressures for the structural design in accordance with CHBDC Figure 6.6.
- If the wall support does not allow lateral yielding (such is typically the case for a rigid concrete box or open footing culverts), at-rest earth pressures should be assumed for geotechnical design. The granular fill should be placed in a zone with a width equal to at least 1.2 metres behind the culvert walls (case (a) from commentary on CHBDC Figure C6.20).

⁵ 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.



FOUNDATION INVESTIGATION AND DESIGN REPORT CULVERT REPLACEMENT, SITE 9-159/C, HIGHWAY 3

- For Case (a), the restrained case, which is typical for culvert walls, the pressures are based on the existing embankment fill materials, assuming a Select Subgrade Material (SSM) is used, and the following parameters (unfactored) may be used:

Soil unit weight: 19 kN/m³

Coefficient of lateral earth pressure:
'At rest' or restrained, K_o 0.53

- If the wall support allows lateral yielding (unrestrained structure, such as typically the case for retaining walls), active earth pressures may be used in the geotechnical design of the structure. The granular fill should be placed in a wedged shaped zone with a width equal to at least 1.2 metres at the footing level against a cut slope which begins at the footing level and extends upwards at a maximum inclination of 1 horizontal to 1 vertical (case (b) from commentary on CHBDC Figure C6.20).
- For walls backfilled using granular materials in accordance with Case (b), the following parameters (unfactored) may be assumed:

	<u>GRANULAR A</u>	GRANULAR B <u>TYPE II</u>	GRANULAR B <u>TYPE III</u>
Fill unit weight:	22 kN/m ³	21 kN/m ³	21 kN/m ³
Coefficients of lateral earth pressure:			
'active' or unrestrained, K_a	0.27	0.27	0.31
'passive', K_p	3.7	3.7	3.3

6.5 Construction Considerations

6.5.1 General

Care should be taken during construction to avoid disturbance of the subgrades prior to constructing foundations for the replacement culvert. All existing fill and any topsoil, organics, and soft or loose soils should be stripped from the proposed founding areas prior to placement of base materials. Subgrade preparation should be performed and monitored in accordance with OPSS 902 and as modified by these recommendations.

It is recommended that the footing excavations be carried out such that the final 0.5 metres of excavation is completed with a Quality Verification Engineer (QVE) experienced in geotechnical engineering on site. The prepared excavation bases should be inspected by the QVE and granular base materials or a working slab should be placed immediately after inspection to protect the founding materials. The clayey subgrade is sensitive to disturbance by foot traffic and excessive moisture uptake. If bedding materials or the foundation cannot be placed within 24 hours of excavating the footing then a working slab must be placed to protect the founding area. The working slab is to be 100 millimetres thick and formed with concrete with a minimum 28-day compressive strength of 20 megapascals. The QVE should assess the foundation conditions to determine if sub-excavation of unsuitable material is required. Sub-excavation and placement and compacted fill should be carried out under the direction of the QVE.



6.5.2 Erosion and Scour Protection

Erosion and scour protection for the culvert inlet and outlet should be provided, as appropriate. Consideration could be given to using suitable non-woven geotextile and rip-rap, as required, to provide erosion protection based on hydraulic requirements. Rip-rap treatment at the culvert outlet should be provided in accordance with OPSD 810.010. In addition, temporary sediment control such as silt fences and erosion control blankets may be required during construction in accordance with OPSS 805.

6.6 Excavations and Groundwater Control

Excavations will extend through the existing pavement structure, fill, and organics and into the underlying silty clay, clayey silt till and silty clay till. It is anticipated that excavation for the culvert replacement will extend below the inferred groundwater level of elevation 179.0 metres. Some perched groundwater seepage from the fill materials should be anticipated. Given the presence of low permeability cohesive materials, it is expected that groundwater can be controlled by pumping from properly constructed and filtered sumps located at the base of the excavations. Sumps should be maintained outside of the actual foundation limits.

Surficial water seepage into the excavations should be expected and will be heavier during periods of sustained precipitation. 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.

Temporary open cut slopes within the fill materials should be maintained no steeper than 1 horizontal to 1 vertical and localized sloughing and ground movements should be expected. All excavations should be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety Act and Regulations for Construction Projects. The fill materials and native firm to stiff silty clay would be classified as Type 3 soils. The very stiff to hard glacial till soils would be considered Type 2 soils.

6.7 Staging and Temporary Roadway Protection

It is understood that a single lane is to remain open to traffic during construction. Temporary support systems could consist of soldier piles and lagging or steel sheet piles. The subsoils contain cobbles and boulders which may make the installation of steel sheet piling difficult. A Non-standard Special Provision (NSSP) should be added to the Contract Documents to warn the contractor about the presence of cobbles and boulders and the need to take special precautions when installing soldier piles or sheet piles. Excavation support systems should be designed and constructed in accordance with OPSS 539 and the design should limit the lateral movement of the temporary shoring system to meet Performance Level 2. The contractor is responsible for the complete detailed design of the protection system.

Where cantilevered sheet piles are used or where the support to the wall is provided by anchors or rakers, the wall design should be based on a triangular earth pressure distribution using the design parameters given below. The raker/anchor support must be designed to accommodate the loads applied from pressures and surcharge pressures from area, line, or point loads as well as the impact of sloping ground behind the system. Passive toe restraint to the soldier piles may be determined using a triangular pressure distribution acting over an equivalent width equal to three times the pile socket diameter.



FOUNDATION INVESTIGATION AND DESIGN REPORT CULVERT REPLACEMENT, SITE 9-159/C, HIGHWAY 3

The unfactored triangular earth pressure distribution (p' in kN/m^2 ; increasing with depth) can be calculated as follows:

$$p' = K_a (H - h_w) \gamma + K_a (\gamma - \gamma_w) h_w + \gamma_w h_w + K_a q$$

where: H = the height of the excavation at any point in metres

K_a = active coefficient of earth pressure

γ = soil unit weight

γ_w = unit weight of water or 9.8 kN/m^3

q = surcharge for traffic and other loading

h_w = height of groundwater level above excavation base/water level to be taken as elevation 179.0 metres

The support systems may be designed using the parameters provided in the table below. These parameters are provided to assist with design for the unfactored ultimate resistance and loading conditions and may not result in a temporary support design that adequately controls ground and structure displacements. Achieving adequate displacement control in accordance with the MTO performance criteria may require designs that result in a system that is stiffer than might otherwise be required based on the soil parameters provided in the table below.

Soil Type	Coefficient of Earth Pressure			Internal Angle of Friction (degrees)	Unit Weight (kN/m^3)
	Active, K_a	At Rest, K_o	Passive, K_p		
Fill	0.36	0.53	2.8	28	19
Silty Clay	0.39	0.56	2.6	26	19
Clayey Silt Till	0.31	0.47	3.3	32	21
Silty Clay Till	0.36	0.53	2.8	28	20
Sandy Silt Till	0.28	0.44	3.5	34	22

The earth pressure coefficients identified above may be applied assuming a horizontal ground surface behind the retaining structure. Where the ground surface behind the retaining structure is sloped, the earth pressure coefficients provided in the table above must be increased.



7.0 MISCELLANEOUS

This section of the report was prepared by Mr. Brett Thorner, E.I.T. under the direction of the Project Engineer Ms. Dirka U. Prout, P.Eng. and reviewed by Mr. W. Michael Kellestine, P.Eng., a Principal with Golder Associates. Mr. Fintan J. Heffernan, P.Eng., the Designated MTO Contact and Quality Control Auditor for this assignment, conducted an independent quality review of the report.

GOLDER ASSOCIATES LTD.

ORIGINAL SIGNED

Dirka U. Prout, P.Eng.
Project Engineer

ORIGINAL SIGNED

W.M. Kellestine, P.Eng.
Principal

ORIGINAL SIGNED

Fintan J. Heffernan, P.Eng.
MTO Designated Contact

BT/DB/DUP/WMK/FJH/cr

Golder, Golder Associates and the GA globe design are trademarks of Golder Associates Corporation.

n:\active\2012\1132 - geo\1132-0100\12-1132-0163 stantec-fdns-mega culverts-3011-e-0041\ph 2000-gwp 3040-11-00\rpts\r08 - site 9-159\final\1211320163-2000-r08 nov 5 14 (final) part a&b fdns repl clvrt 9-159-c.docx

TABLE I

COMPARISON OF STRUCTURE ALTERNATIVES FOR REPLACEMENT CULVERT

Site 9-159/C, Station 31+675, Highway 3
 Structure Replacements and Rehabilitation
GWP 3040-11-00

FOUNDATION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	ESTIMATED COSTS	RISKS/ CONSEQUENCES
Precast twin box culvert founded on stiff silty clay, stiff silty clay till or hard clayey silt till	<ul style="list-style-type: none"> • Feasible • Preferred technical alternative 	<ul style="list-style-type: none"> • Least expensive option due to shallower excavation compared to the open footing option and use of precast elements • Allows for most rapid construction compared to the two other alternatives since there is no wait for concrete to cure • Suitable for corrosive environments • Can be more readily installed during cold weather conditions • More tolerant of differential settlement, given the variability in founding materials along the length of the culvert 	<ul style="list-style-type: none"> • If floor is thin and poorly reinforced, it may crack and heave • During high flows, the concrete floor can be undermined • Susceptible to defects/leakage at joints 	<ul style="list-style-type: none"> • Low 	<ul style="list-style-type: none"> • Relatively low risk • Improper alignment or transition between stream and culvert may lead to problems with scour
CIP twin box culvert founded on stiff silty clay, stiff silty clay till or hard clayey silt till	<ul style="list-style-type: none"> • Feasible 	<ul style="list-style-type: none"> • Intermediate in cost between a pre-cast box and CIP open footing culvert • Less excavation required compared to an open footing culvert • Suitable for corrosive 	<ul style="list-style-type: none"> • If floor is thin and poorly reinforced, it may crack and heave • During high flows, the concrete floor can be undermined • More expensive 	<ul style="list-style-type: none"> • Low to Moderate 	<ul style="list-style-type: none"> • Relatively low risk • Improper alignment or transition between stream and culvert may lead to problems with scour

COMPARISON OF STRUCTURE ALTERNATIVES FOR REPLACEMENT CULVERT

FOUNDATION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	ESTIMATED COSTS	RISKS/ CONSEQUENCES
		environments <ul style="list-style-type: none"> • Culvert design can be customized in the field for high stress or load conditions or other site-specific requirements • Can be constructed with far fewer joints than a precast box culvert • More tolerant of differential settlement, given the variability in founding materials along the length of the culvert 	compared to precast box option due to increased labour associated with concrete formwork <ul style="list-style-type: none"> • Special curing requirements for cold weather work 		
CIP two cell open footing culvert with spread footings founded on stiff silty clay, stiff silty clay till or very stiff clayey silt till	<ul style="list-style-type: none"> • Feasible 	<ul style="list-style-type: none"> • Higher costs compared to box culvert options • Suitable for corrosive environments • Culvert design can be customized in the field for high stress or load conditions or other site-specific requirements • Can be constructed with far fewer joints than a precast box culvert 	<ul style="list-style-type: none"> • More expensive compared to both box culvert options due to increased labour associated with concrete formwork and deeper excavation required to provide frost protection for footings • Footings may be susceptible to scour and undermining especially at inlet 	<ul style="list-style-type: none"> • Moderate to High 	<ul style="list-style-type: none"> • Relatively low to moderate risk • Improper alignment or transition between stream and culvert may lead to problems with scour • Option with foundations most susceptible to damage by scour

- NOTES:
1. Qualitative estimates are based on 2014 construction costs and are intended to provide a comparison between alternatives rather than actual construction costs.
 2. Table to be read in conjunction with accompanying report.

Prepared By: BT
 Checked By: DUP

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 201

1 OF 1

METRIC

PROJECT 12-1132-0163
W.P. 3040-11-00 LOCATION N 4760447.4 , E 287329.0 ORIGINATED BY BT
DIST HWY 3 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY WDF/LMK
DATUM GEODETIC DATE July 2, 2013 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									
179.95	GROUND SURFACE						20	40	60	80	100						
0.00	TOPSOIL, silty Brown																
0.18	SILTY CLAY TILL, trace to some sand, trace gravel, with cobbles and boulders Stiff to very stiff Brown		1	SS	8												
			2	SS	11												
			3	SS	9												
			4	SS	11												
			5	SS	19												
			6	SS	18												
174.46	CLAYEY SILT TILL, some sand, trace gravel, with cobbles and boulders Hard Grey		7	SS	33												
172.63	SILTY CLAY, trace sand Firm Grey		8	SS	6												
171.87	END OF BOREHOLE																
8.08	Borehole dry during drilling on July 2, 2013.																

RECORD OF BOREHOLE No 202

1 OF 1

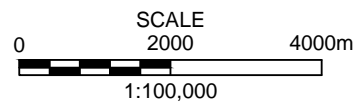
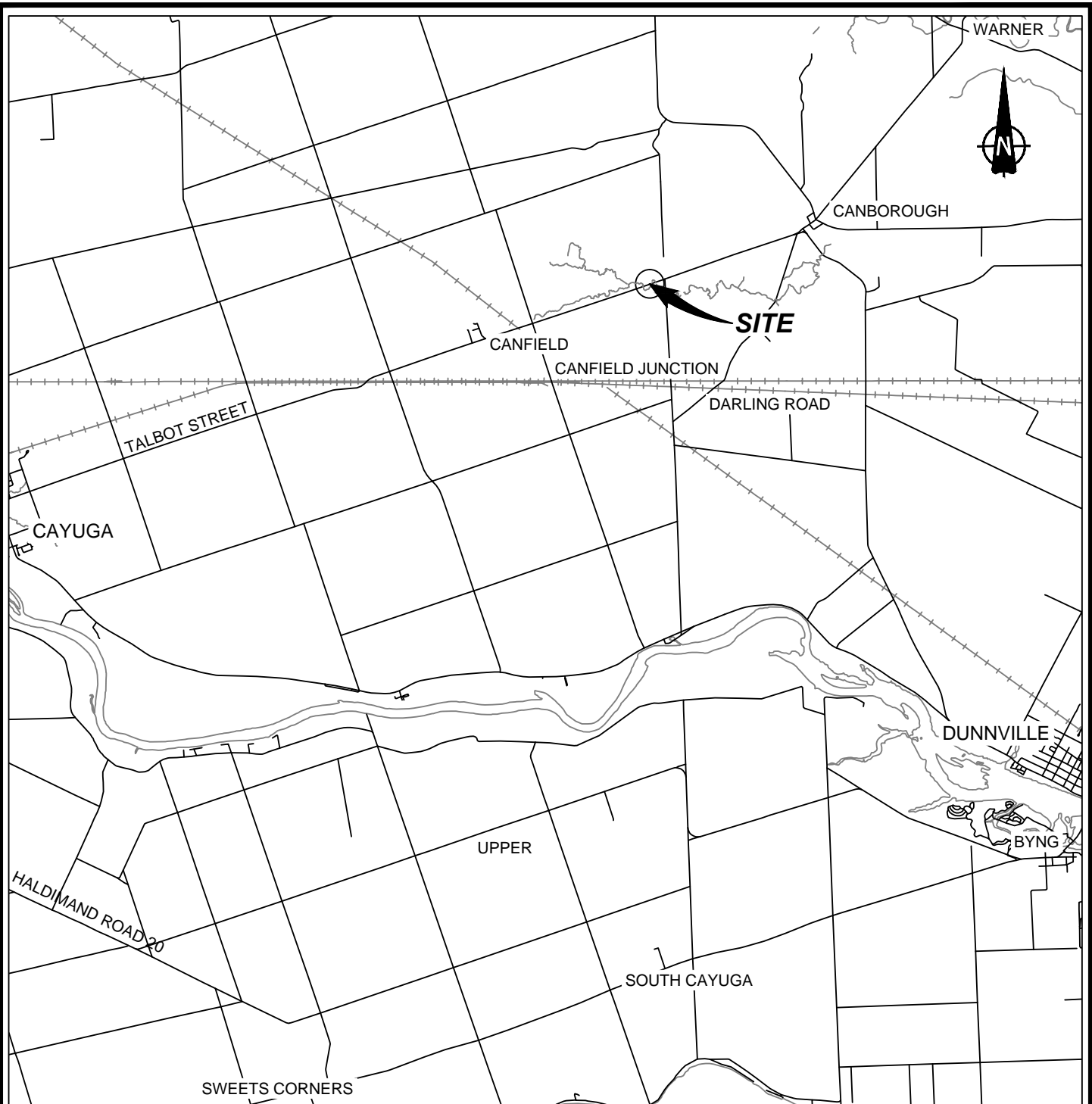
METRIC

PROJECT 12-1132-0163
W.P. 3040-11-00 LOCATION N 4760432.7, E 287358.8 ORIGINATED BY BT
DIST HWY 3 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY WDF/LMK
DATUM GEODETIC DATE July 2, 2013 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		W _P	W		
180.04	GROUND SURFACE						20 40 60 80 100						
0.00	TOPSOIL, silty Brown						○ UNCONFINED + FIELD VANE						
0.12	FILL, silty clay, trace sand, trace gravel, roots Firm Brown		1	SS	6		● QUICK TRIAXIAL × LAB VANE						
178.67	SILTY CLAY, trace sand Stiff Brown		2	SS	11								
1.37			3	SS	9								
			4	SS	8								
176.38	SILTY CLAY TILL, some sand, trace gravel Firm to very stiff Grey		5	SS	17								
3.66			6	SS	14								
			7	SS	5								
172.73	SANDY SILT TILL, some clay, trace gravel Very dense Grey		8	SS	54								
7.31			9	SS	94								
170.44	END OF BOREHOLE												
9.60	Borehole dry during drilling on July 2, 2013. Water level measured at elev. 172.39m following installation on July 2, 2013. Water level measured at elev. 175.47m on September 5, 2013. Water level measured at elev. 180.52m on April 30, 2014. "Open pipe and area flooded at time of measurement"												

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

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




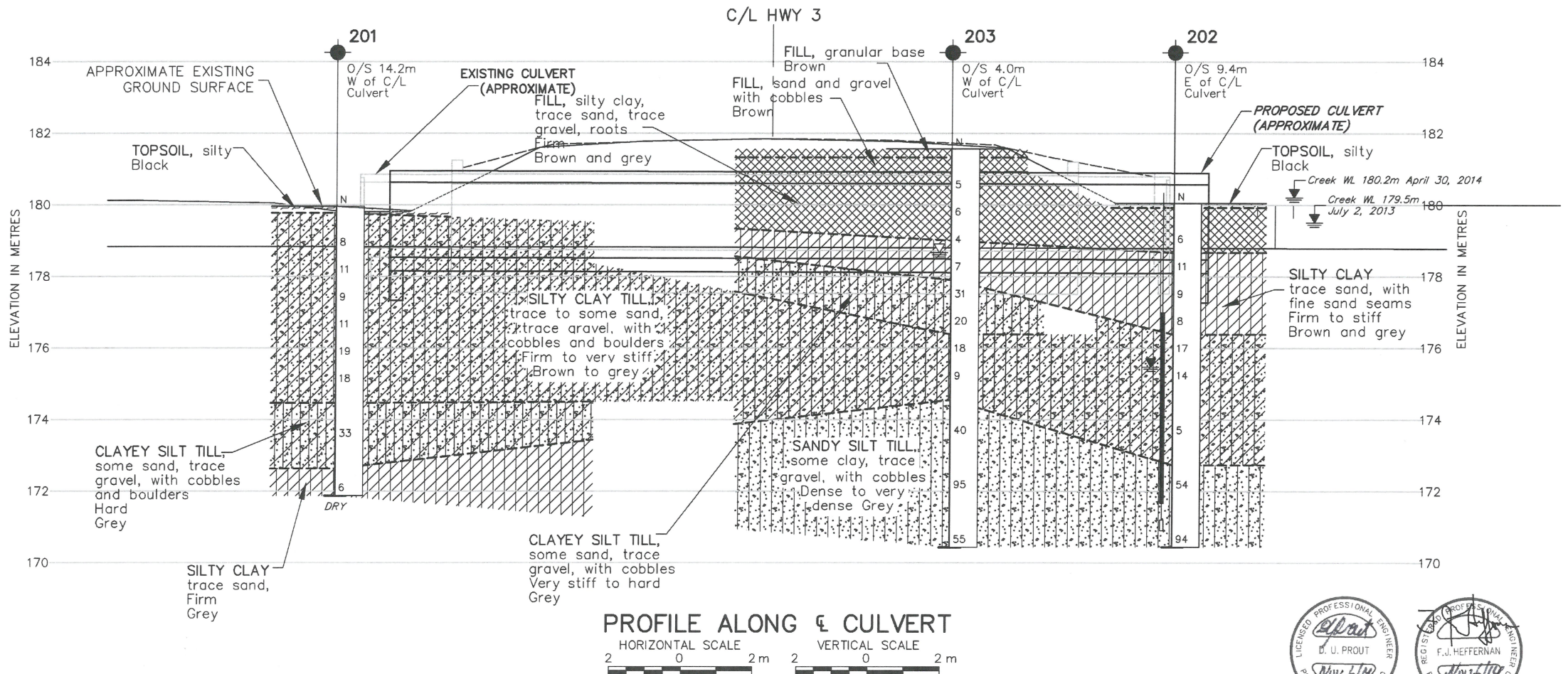
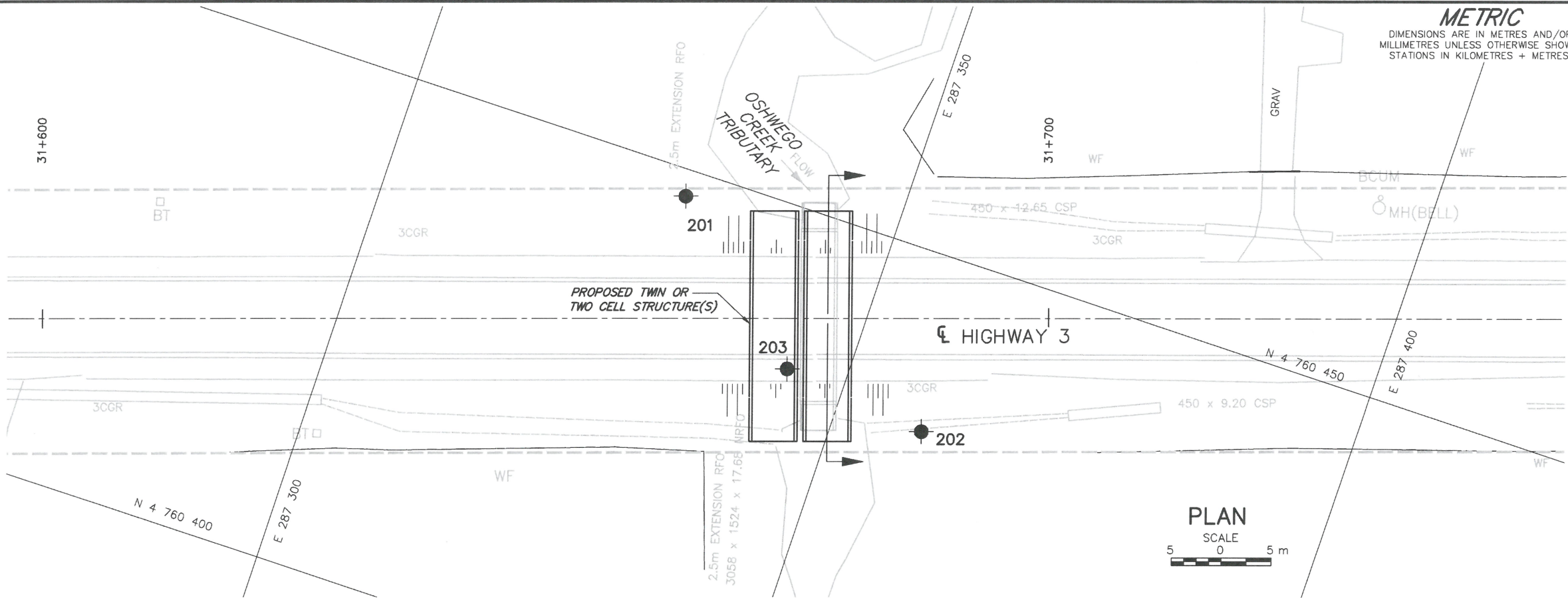
REFERENCE

PLAN BASED ON CANMAP STREETFILES V.2008.5.

NOTE

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

PROJECT		CULVERT REPLACEMENT, SITE 9-159/C STATION 31+675, HIGHWAY 3 GWP 3040-11-00	
TITLE		KEY PLAN	
PROJECT No.		12-1132-0163	FILE No. 1211320163-2000-F08001
CADD	LMK	Nov. 19/13	SCALE AS SHOWN REV. 0
CHECK			
 Golder Associates LONDON, ONTARIO		FIGURE 1	



CONT No. WP No. 3040-11-00

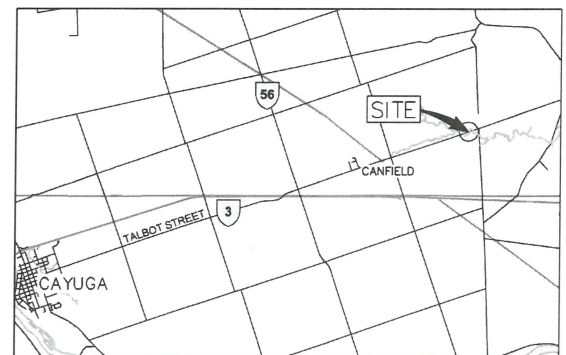
CULVERT REPLACEMENT STATION 31+675, HIGHWAY 3

STRUCTURE REPLACEMENTS AND REHABILITATION BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



Golder Associates Ltd.
LONDON, ONTARIO, CANADA



KEY PLAN



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 measured on September 5, 2013.
- WL encountered during drilling
- DRY Water level not established

No.	ELEVATION	CO-ORDINATES (MTM NAD83 ZONE 11)	
		NORTHING	EASTING
201	179.95	4760447.4	287 329.0
202	180.04	4760432.7	287 358.8
203	181.56	4760434.3	287 344.1

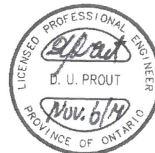
NOTES

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 by Stantec.

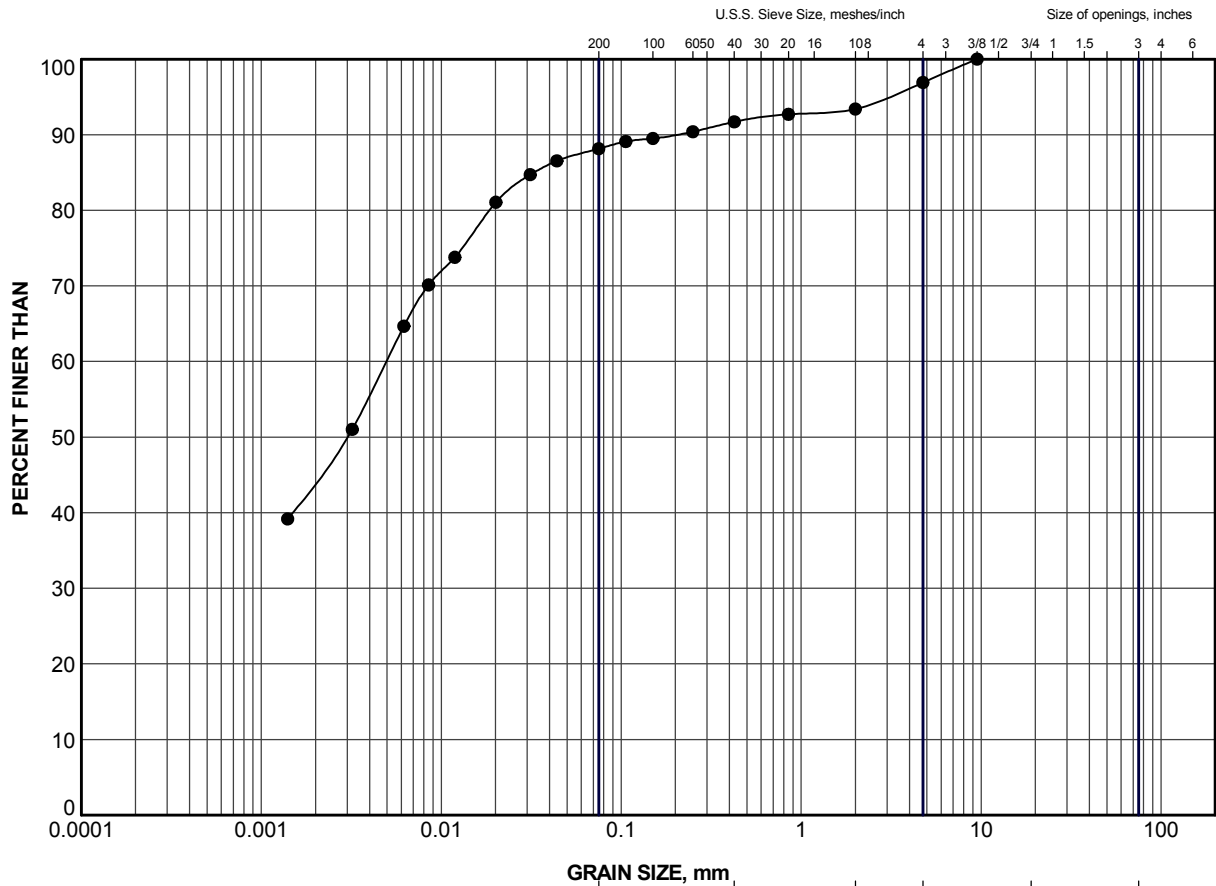


NO.	DATE	BY	REVISION
Geocres No.	30L13-22		
HWY.	3	PROJECT NO.	12-1132-0163
SUBM'D.	BT	CHKD.	BT
DRAWN:	WDF	CHKD.	DUP
DATE:	June 24/14	APPD.	FJH
SITE:	9-159/C	DWG.	1



APPENDIX A

Laboratory Test Data

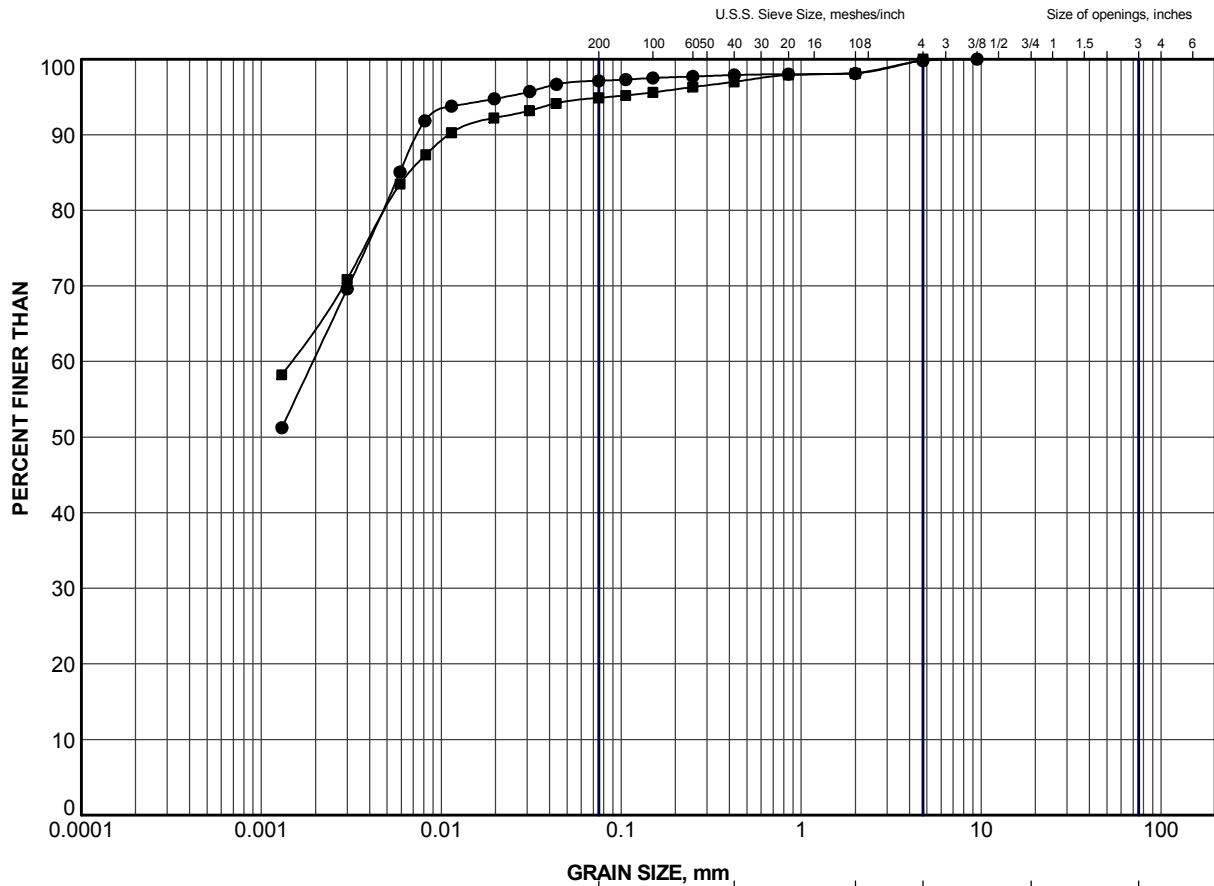


CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	203	2	179.8

PROJECT		CULVERT REPLACEMENT, SITE 9-159/C STATION 31+675, HIGHWAY 3 GWP 3040-11-00			
TITLE		GRAIN SIZE DISTRIBUTION FILL			
PROJECT No.		12-1132-0163		FILE No. 1211320163-2000-F080A1	
DRAWN		LMK		Nov 18/13	
CHECK					
Golder Associates LONDON, ONTARIO		SCALE		N/A REV.	
				FIGURE A-1	



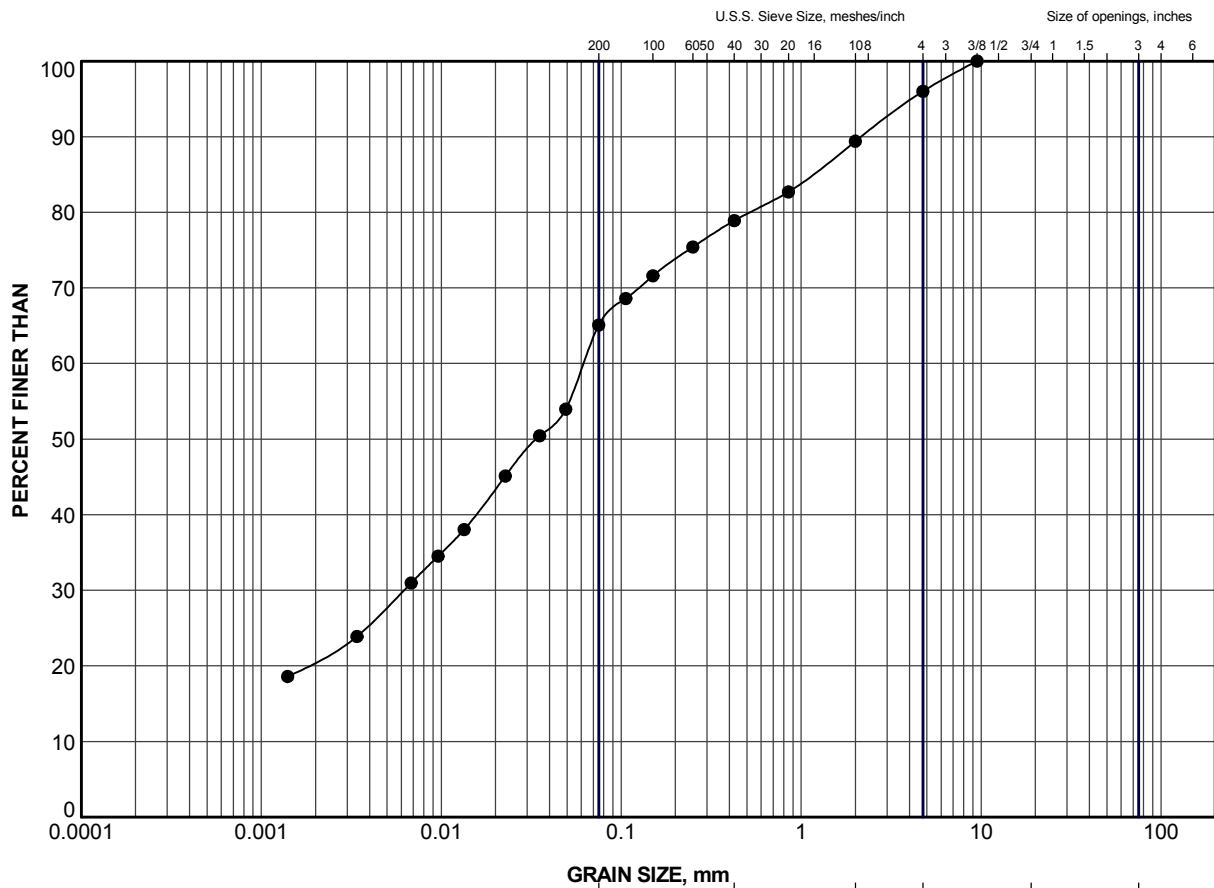
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	201	8	172.1
■	202	3	177.5

PROJECT				CULVERT REPLACEMENT, SITE 9-159/C STATION 31+675, HIGHWAY 3 GWP 3040-11-00			
TITLE				GRAIN SIZE DISTRIBUTION SILTY CLAY			
PROJECT No.		12-1132-0163		FILE No. 1211320163-2000-F080A2			
DRAWN		LMK		Nov 18/13		SCALE N/A REV.	
CHECK						FIGURE A-2	




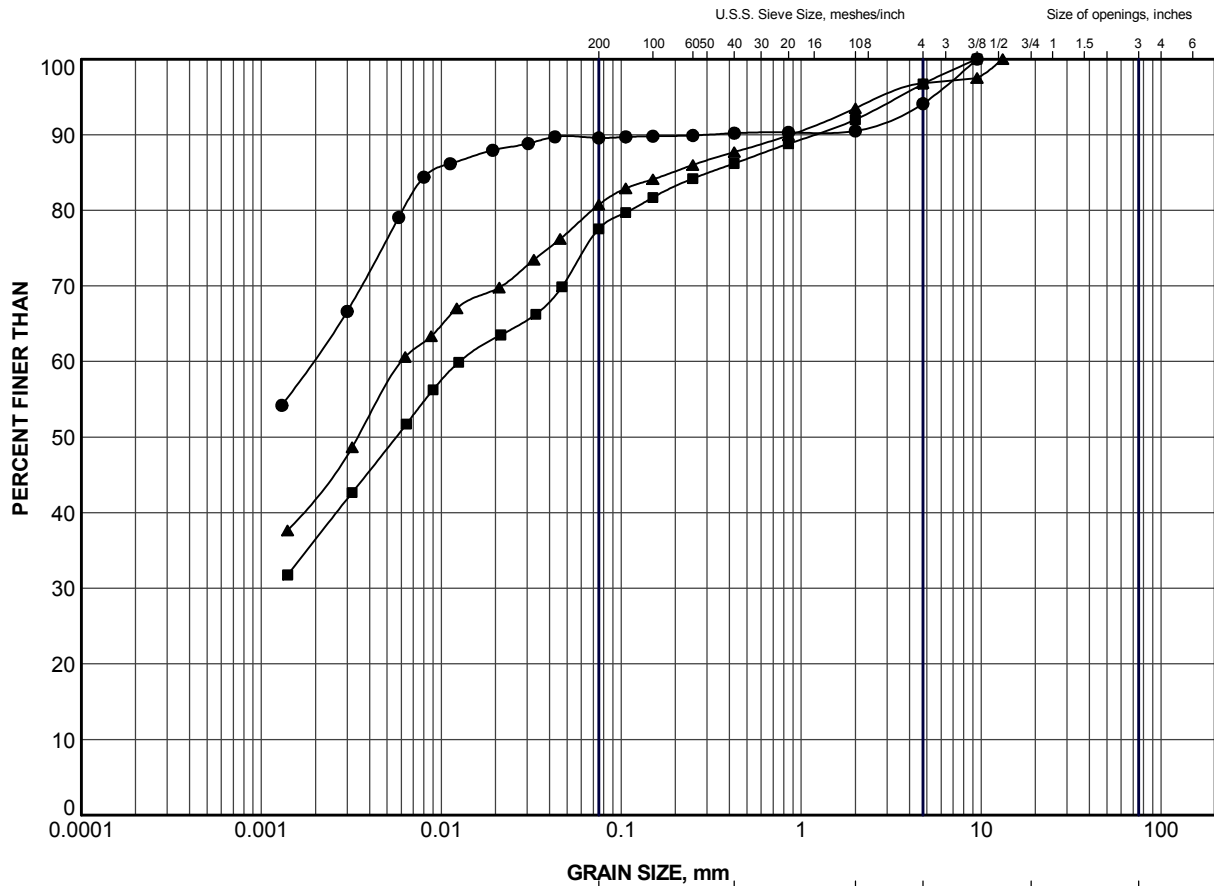


CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	203	5	177.5

PROJECT	CULVERT REPLACEMENT, SITE 9-159/C STATION 31+675, HIGHWAY 3 GWP 3040-11-00		
TITLE	GRAIN SIZE DISTRIBUTION CLAYEY SILT TILL		
	PROJECT No.	12-1132-0163	FILE No. 1211320163-2000-F080A3
	DRAWN	LMK	Nov 18/13
	CHECK		
			SCALE N/A REV.
			FIGURE A-3

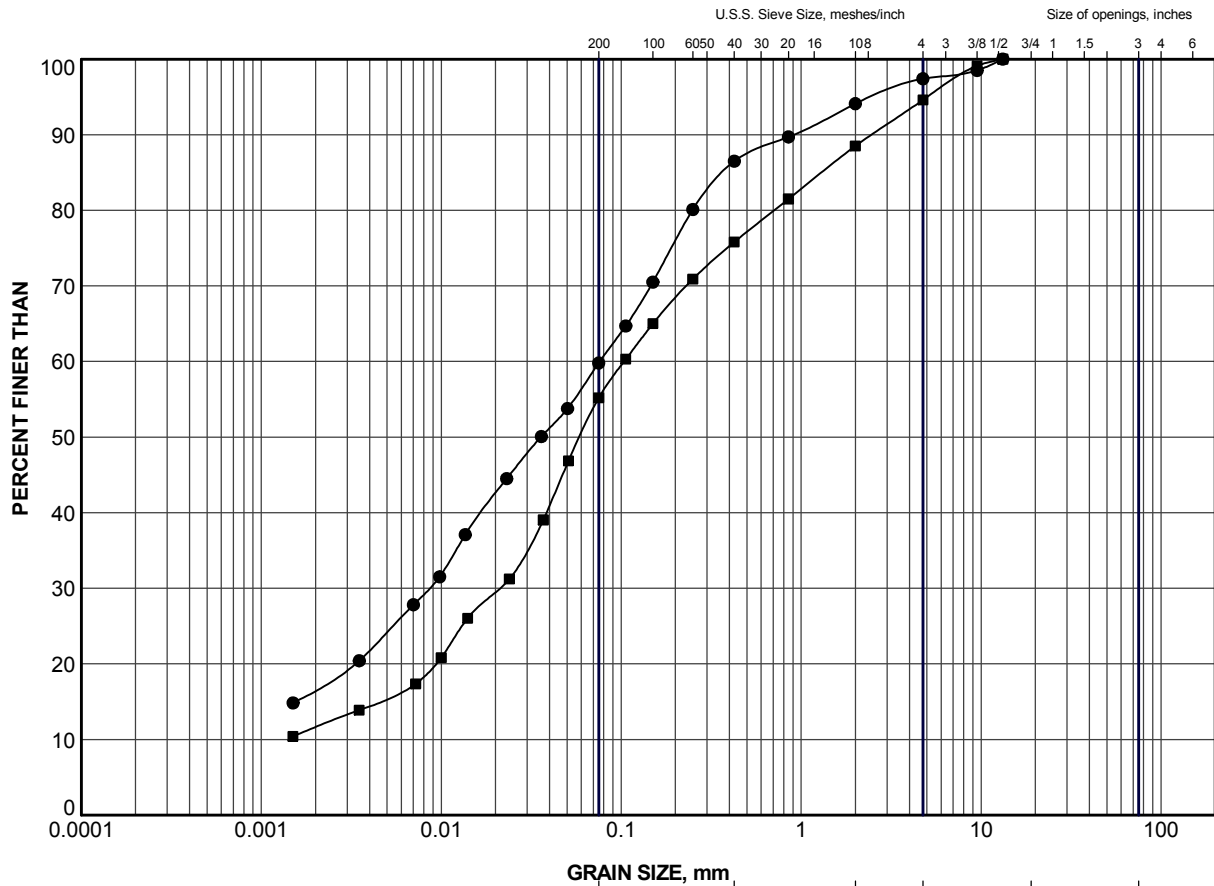


CLAY AND SILT	GRAVEL SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	201	3	177.4
■	201	6	175.2
▲	202	5	176.0


PROJECT	CULVERT REPLACEMENT, SITE 9-159/C STATION 31+675, HIGHWAY 3 GWP 3040-11-00		
TITLE	GRAIN SIZE DISTRIBUTION SILTY CLAY TILL		
 Golder Associates LONDON, ONTARIO	PROJECT No.	12-1132-0163	FILE No. 1211320163-2000-F080A4
	DRAWN	LMK	Nov 18/13
	CHECK		
			SCALE N/A REV.
			FIGURE A-4

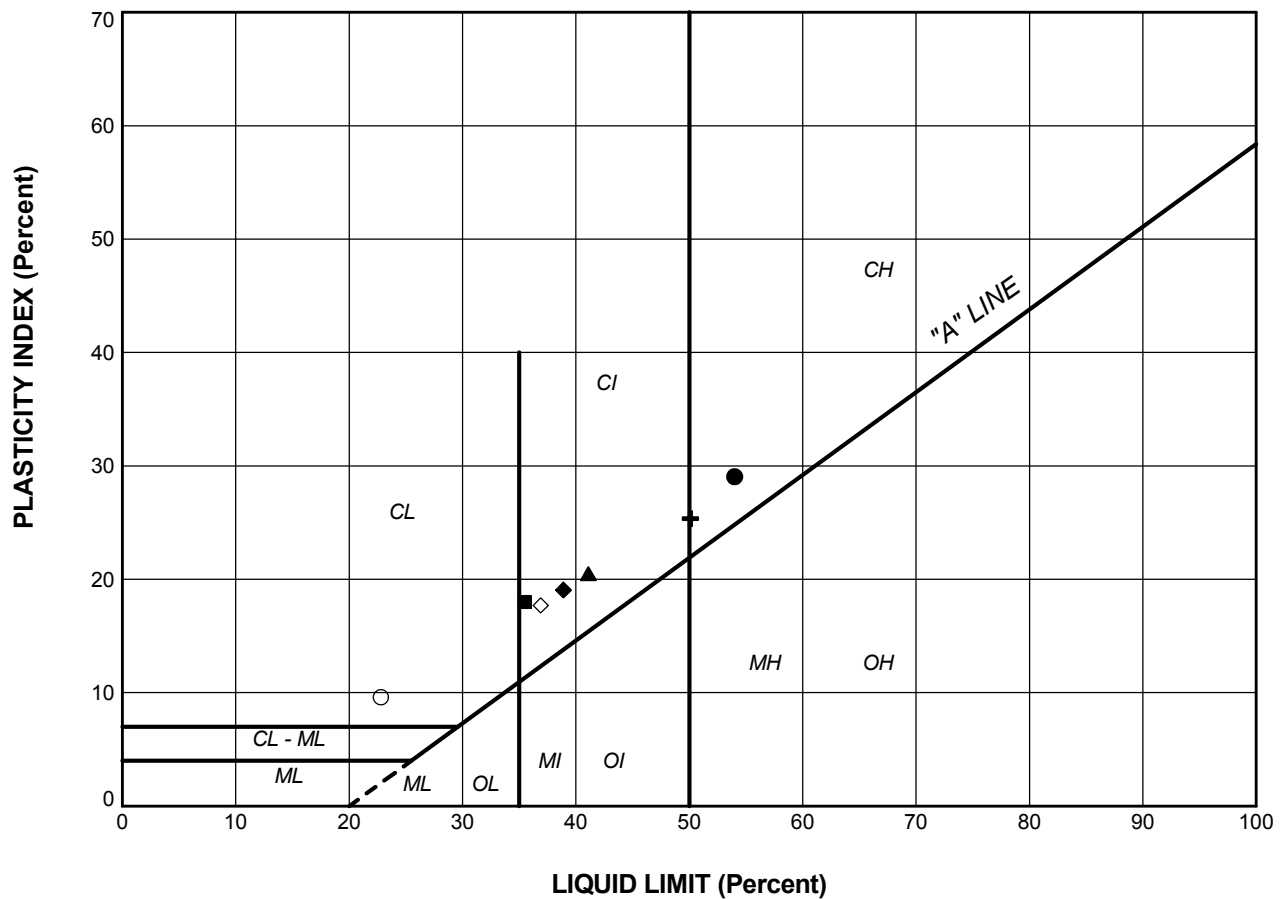


CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	202	8	172.2
■	203	9	173.7

PROJECT				CULVERT REPLACEMENT, SITE 9-159/C STATION 31+675, HIGHWAY 3 GWP 3040-11-00			
TITLE				GRAIN SIZE DISTRIBUTION SANDY SILT TILL			
PROJECT No.		12-1132-0163		FILE No. 1211320163-2000-F080A5			
DRAWN		LMK		Nov 18/13		SCALE N/A REV.	
CHECK						FIGURE A-5	
 Golder Associates LONDON, ONTARIO							



LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
FILL					
◇	203	2	36.9	19.2	17.7
SILTY CLAY					
▲	201	8	41.1	20.6	20.5
+	202	3	50.1	24.8	25.4
SILTY CLAY TILL					
●	201	3	54.0	25.0	29.1
■	201	6	35.5	17.5	18.0
◆	202	5	38.9	19.9	19.1
CLAYEY SILT TILL					
○	203	5	22.8	13.2	9.6

PROJECT				CULVERT REPLACEMENT, SITE 9-159/C STATION 31+675, HIGHWAY 3 GWP 3040-11-00			
TITLE							
PLASTICITY CHART							
PROJECT No.		12-1132-0163		FILE No. 1211320163-2000-F080A6			
DRAWN	LMK	Nov 18/13		SCALE	N/A	REV.	
CHECK				FIGURE A-6			





APPENDIX B

Site Photographs



APPENDIX B PHOTOGRAPHS



Photograph 1: North elevation (inlet) of Culvert Site 9-159/C.



Photograph 2: South elevation (outlet) of Culvert Site 9-159/C.



APPENDIX B PHOTOGRAPHS



Photograph 3: Highway 3 looking east from north shoulder at Culvert Site 9-159/C.

n:\active\2012\1132 - geo\1132-0100\12-1132-0163 stantec-fdns-mega culverts-3011-e-0041\ph 2000-gwp 3040-11-00\rpts\r08 - site 9-159\final\1211320163-2000-r08 nov 4 14 (final) app
b - photos.docx

At Golder Associates we strive to be the most respected global company providing consulting, design, and construction services in earth, environment, and related areas of energy. Employee owned since our formation in 1960, our focus, unique culture and operating environment offer opportunities and the freedom to excel, which attracts the leading specialists in our fields. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees who operate from offices located throughout Africa, Asia, Australasia, Europe, North America, and South America.

Africa	+ 27 11 254 4800
Asia	+ 86 21 6258 5522
Australasia	+ 61 3 8862 3500
Europe	+ 356 21 42 30 20
North America	+ 1 800 275 3281
South America	+ 55 21 3095 9500

solutions@golder.com
www.golder.com

Golder Associates Ltd.
309 Exeter Road, Unit #1
London, Ontario, N6L 1C1
Canada
T: +1 (519) 652 0099

