



January 2015

FOUNDATION INVESTIGATION AND DESIGN REPORT

**Culvert Replacement, Sandusk Creek Tributary, Site
No. 9-155/C
Station 14+537, Highway 3
Contract 2 Structure Replacements and Rehabilitation
GWP 3040-11-00
Ministry of Transportation, Ontario - West 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.....	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 Pavement Structure	4
4.2.2 Fill	4
4.2.3 Topsoil	4
4.2.4 Silty Clay	5
4.2.5 Clayey Silt Till	5
4.2.6 Bedrock.....	5
4.3 Groundwater Conditions	5
5.0 MISCELLANEOUS	7

PART B - FOUNDATION DESIGN REPORT

6.0 ENGINEERING RECOMMENDATIONS.....	8
6.1 General.....	8
6.2 Replacement Culvert	8
6.2.1 Foundations	8
6.2.2 Bedding.....	10
6.2.3 Backfill and Cover	11
6.2.4 End Treatments and Camber.....	11
6.3 Retaining Walls.....	11
6.3.1 Retaining Wall Options.....	11
6.3.2 Retaining Wall Foundations	12
6.3.3 Resistance to Lateral Forces	13



6.4	Liquefaction Potential and Seismic Analysis.....	13
6.4.1	Seismic Parameters	13
6.4.2	Seismic Hazard Assessment	13
6.5	Lateral Earth Pressures for Design.....	14
6.6	Construction Considerations.....	15
6.6.1	General	15
6.6.2	Erosion and Scour Protection	15
6.7	Excavations and Groundwater Control	16
6.8	Staging and Temporary Roadway Protection	16
7.0	MISCELLANEOUS	18

TABLE I - Comparison of Structure Alternatives for Replacement Culvert

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, SANDUSK CREEK TRIBUTARY, SITE NO. 9-155/C

STATION 14+537, HIGHWAY 3

CONTRACT 2 STRUCTURE REPLACEMENTS AND REHABILITATION

GWP 3040-11-00

MINISTRY OF TRANSPORTATION, ONTARIO - 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 concrete culvert at the Sandusk Creek Tributary (Site 9-155/C) at Station 14+537 on Highway 3 east of Jarvis, 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's 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.



2.0 SITE DESCRIPTION

The subject culvert is situated at Station 14+537 on Highway 3, approximately 0.6 kilometres east of Nanticoke Road in the Township of Walpole in Haldimand County, Ontario. The town of Jarvis is about 2.1 kilometres west of the site. 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. The creek flows beneath Highway 3 from north to south through the culvert. The existing concrete structure has the following characteristics:

Dimensions (m)	Obvert Elevation (m)		Construction
	Lt	Rt	
4.90 x 1.83 x 18.75	207.12	207.09	Concrete NRFO with Concrete Arch Extensions

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

The banks of the drainage 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 rural agricultural area with low relief. Ground surface elevations in the vicinity of the culvert site range from about 207 to 208 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 deep water clay deposits that are massive to laminated and varved at the vicinity of the site.² Geological mapping also indicates that the underlying bedrock consists of medium brown microcrystalline limestone of the Dundee Formation of Middle Devonian age.³ The bedrock surface at the site is reportedly at about elevation 203.5 to 205.0 metres, with the overburden thickness being about 3 to 4 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 Barnett, P.J and Girard, C.K., 1975: Quaternary Geology of the Simcoe Area, Southern Ontario; Ontario Div. Mines., Map 2369, scale 1:50,000.

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

⁴ Barnett, P.J. and Fegruson, A.J., 1975: Bedrock Topography of the Simcoe Area, Southern Ontario; Ontario Div. Mines Preliminary Map P.1055, Bedrock Topography Series. Scale 1:50,000.



3.0 INVESTIGATION PROCEDURES

The field work for the investigation was carried out on October 16, 2014, 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 45 drilling equipment supplied and operated by a specialist drilling contractor. Samples of the overburden were typically obtained at depth intervals of 0.75 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 4.0 of this report 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 231 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 experienced members of our staff who located the boreholes in the field, obtained utility locates, 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 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		
230	4 750 008	256 928	207.41	5.03
231	4 749 991	256 925	206.46	4.11
232	4 750 001	256 955	207.43	5.03



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 and/or cohesive fill materials overlying topsoil over silty clay and clayey silt glacial till at depth.

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 Pavement Structure

Pavement granular materials were encountered at ground surface in boreholes 230 and 232. The granular materials were about 360 and 300 millimetres thick, respectively.

4.2.2 Fill

Silty clay fill material was encountered at the ground surface in borehole 231 and beneath the pavement structure in borehole 232 at elevation 207.1 metres. The firm silty clay fill material was 140 to 610 millimetres thick with an N value of 4 blows per 0.3 metres. A sample of the silty clay fill had a water content of about 13 per cent. A sheet of fibrous cloth was encountered beneath the fill material in borehole 231.

4.2.3 Topsoil

Topsoil was encountered beneath the pavement structure in borehole 230 at elevation 207.1 metres and beneath the silty clay fill material in borehole 231 at elevation 206.3 metres. The buried topsoil layers were 50 and 110 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.



4.2.4 Silty Clay

Firm to stiff silty clay was encountered beneath the buried topsoil in boreholes 230 and 231 at elevations 207.0 and 206.2 metres, respectively, and beneath the fill material in borehole 232 at elevation 206.5 metres. The silty clay was 1.1 to 1.7 metres thick. Measured N values for the silty clay were 4 to 8 blows per 0.3 metres. The silty clay had water contents of about 19 to 28 per cent.

The silty clay was of intermediate to high plasticity based on the Atterberg limits determinations carried out on three samples obtained during standard penetration testing. The plastic limit ranged from 18 to 22 per cent, the liquid limit ranged from 43 to 52 per cent and the plasticity index ranged from 23 to 30 per cent indicating intermediate to high plasticity. Grain size distribution curves for samples of the silty clay are provided on Figure A-1 and the Atterberg limits data are presented on Figure A-3.

4.2.5 Clayey Silt Till

Very stiff to hard clayey silt glacial till was encountered beneath the silty clay in all of the boreholes between elevations 205.1 and 205.3 metres. Measured N values for the clayey silt till layers generally ranged from 15 to 38 blows per 0.3 metres, with N values of greater than 100 blows per 0.3 metres obtained at the inferred soil/bedrock interface. The clayey silt till had water contents of about 9 to 26 per cent. The presence of cobbles and boulders should be expected in this deposit.

The clayey silt till was of low plasticity based on the Atterberg limits determinations carried out on three samples obtained during standard penetration testing. The plastic limit was 13 per cent, the liquid limit ranged from 20 to 21 per cent and the plasticity index ranged from 7 to 9 per cent indicating low plasticity. Grain size distribution curves for samples of the clayey silt till are provided on Figure A-2 and the Atterberg limits data for the clayey silt till are presented on Figure A-3.

4.2.6 Bedrock

Each of the boreholes was terminated due to auger refusal on probable bedrock at about elevation 202.4 metres.

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 231. Installation details are provided on the corresponding Record of Borehole sheet following the text of this report. Groundwater was encountered in all boreholes at depths of 1.7 to 3.6 metres or between elevations 203.8 and 204.7 metres. A summary of the encountered and measured groundwater levels is provided in the table below:



FOUNDATION INVESTIGATION AND DESIGN REPORT
CULVERT REPLACEMENT, SANDUSK CREEK TRIBUTARY, SITE 9-155/C, HIGHWAY 3

Borehole	Ground Surface Elevation (m)	Encountered Groundwater Elevation (m)	Measured Groundwater Level Elevation (m)	
			Oct. 16, 2014	Dec. 1, 2014
230	207.41	203.8	-	-
231	206.46	204.0	203.33	205.85
232	207.43	204.7	-	-

The above-noted encountered water levels are not considered to be representative of the long-term, stabilized groundwater conditions. The water level in the Sandusk Creek Tributary was measured at elevation 205.96 metres on October 16, 2014. Following installation of the piezometer on October 16, 2014, the water level in the piezometer installed in borehole 231 was about 3.1 metres below ground surface or at about elevation 203.3 metres. On December 1, 2014, the water level in the piezometer installed in borehole 231 was about 0.6 metres below ground surface or at about elevation 205.9 metres.

Based on the observed groundwater levels, the surrounding topography, the soil colour change from brown to grey and the water level in the creek, the inferred groundwater level is at elevation 206.0 metres. The groundwater level is expected to fluctuate seasonally and 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 Ms. Nicole A. Gould, P.Eng. 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 the Project Engineer Ms. Dirka U. Prout, P.Eng. and reviewed by Mr. Azmi M. Hammoud, P.Eng., an Associate 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.

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PART B

FOUNDATION DESIGN REPORT

CULVERT REPLACEMENT, SANDUSK CREEK TRIBUTARY, SITE NO. 9-155/C

STATION 14+537, HIGHWAY 3

CONTRACT 2 STRUCTURE REPLACEMENTS AND REHABILITATION

GWP 3040-11-00

MINISTRY OF TRANSPORTATION, ONTARIO - 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 of the concrete culvert at the Sandusk Creek Tributary (Site 9-155/C). The culvert is located at Station 14+537 on Highway 3 in the Township of Walpole in Haldimand County, Ontario.

The recommendations are based on interpretation of the factual data obtained from the boreholes advanced at this site. The interpretation and recommendations are intended to provide the designers with sufficient information to design the proposed new 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 watercourse flow is conveyed beneath Highway 3 through the concrete culvert that is 18.8 metres long with a 4.9 metre span and 1.8 metres in height. The approximate invert elevations are 204.4 metres at the inlet and outlet. The existing culvert has approximately 1.1 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 concrete culvert with a new 6.0 by 3.0 metre concrete open footing culvert or a 6.0 by 2.4 metre concrete box culvert. Stantec has indicated that replacement with a single concrete box culvert is the preferred structural alternative. The replacement culvert may be precast or cast-in-place (CIP). No grade raise is proposed at this location. A comparison of the various culvert types is presented in 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 silty clay fill material overlying buried topsoil to about elevation 206.5 metres. The topsoil and fill materials were underlain by firm to stiff silty clay to approximately elevation 205 metres. The silty clay was underlain by very stiff to hard clayey silt till. Cobbles and/or boulders should be expected in the clayey silt till layer. The inferred groundwater level for design purposes is at elevation 206.0 metres. The water level in the Sandusk Creek Tributary was measured at elevation 206.0 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 these types of structures are 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 organic materials.

Based on the proposed invert elevations of 204.5 and 204.4 metres at the inlet and outlet, respectively, and allowing for a base slab thickness of about 250 millimetres, 300 millimetres of granular bedding and a 75 millimetre thick levelling course, a replacement box culvert may be founded on the very stiff clayey silt till at or below elevation 203.8 metres. An open footing culvert may be founded on the very stiff clayey silt till at or below elevation 203.2 metres. From a foundations engineering perspective, both culvert types are feasible, however, a box culvert offers the advantage of more rapid construction, reduced excavation effort and less traffic disruption. Any observed fill or organic materials should be removed to expose competent native soils. Any low areas should be brought to design grade using lean concrete fill or compacted, well graded granular materials.

Geotechnical Resistances

A factored geotechnical resistance at Ultimate Limit States (ULS) of 300 kilopascals and a geotechnical reaction at Serviceability Limit States (SLS) of 200 kilopascals may be used for design purposes provided that the foundations have a minimum width of 0.5 metres and that the subgrade has been properly prepared in accordance with Section 6.6 below. If the footing width is increased to 0.9 metres, a factored geotechnical resistance of 350 kilopascals at ULS and a geotechnical reaction of 225 kilopascals at SLS may be used for design. 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. Foundations for open footing culverts must be provided with frost protection in the form of a minimum depth of soil cover equivalent with the frost depth or a comparable thermal alternative. 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 CIP 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:

$$H_{ri} = 0.8A'c' + 0.8V\tan\delta > H_f \text{ (for pre-cast elements)}$$

$$H_{rs} = 0.8A'c' + 0.8V\tan\phi' > H_f \text{ (for CIP concrete footings)}$$

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 CIP concrete footing
- V - unfactored vertical force, kilonewtons
- H_f - unfactored horizontal load, kilonewtons

The factored horizontal resistance may be calculated using the 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 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, or organic materials, or other deleterious materials have been removed. Should subexcavated be required below the design subgrade elevation, the material should be replaced with compacted Ontario Provincial Standard Specification (OPSS) Granular B Type II or Granular A. It is recommended that the precast 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 placed and compacted in accordance with OPSS 501 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 OPSS 501 and 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

If a box culvert is selected, cut-off walls should be provided in accordance with Clause 1.9.5.6 of the CHBDC. No grade raise is proposed as part of the culvert replacement and relatively low cover is proposed over the replacement culvert founded on very stiff clayey silt till, therefore it is not necessary to provide a camber.

6.3 Retaining Walls

The existing structure has a retaining wall consisting of gabion baskets at the northwest quadrant that is about 3.0 metres long and 1.0 metre in height. The existing retaining wall will be removed during the culvert replacement and new retaining walls will be constructed at the northeast and southwest quadrants. It is understood that consideration is being given to constructing the retaining walls at the replacement structure as gabion walls, concrete gravity or cantilever walls or reinforced soil system (RSS) walls. The retaining wall options are discussed below.

6.3.1 Retaining Wall Options

Gabion Walls

Gabion walls do not require an embedment depth equivalent to the frost depth provided they are founded on a granular pad of 300 millimetres compacted thickness, and the foundations have adequate embedment to provide a stable structure. Advantages of gabion walls compared to more rigid structures include the ability to accommodate differential settlements, dissipation of the energy of flowing water, and they are free-draining provided an adequate filter is placed behind the wall. Gabion walls can be constructed relatively quickly with minimal equipment and materials.



Reinforced Concrete Gravity and Cantilever Walls

Construction of reinforced concrete gravity or cantilever walls is geotechnically feasible. Compared to gabion walls or RSS walls, footings for gravity and cantilever walls must be constructed with a frost cover of 1.2 metres. The concrete gravity walls could consist of precast elements or CIP. Precast retaining walls are preferred for compatibility with precast culverts.

RSS Walls

The height of the retaining walls will be relatively low. Therefore, a reinforced soil system wall utilizing an interlocking block facing system and geogrid reinforcement is a geotechnically feasible alternative. RSS walls are proprietary systems which are designed by the supplier and constructed in accordance with their specifications. The internal stability of the mechanically-reinforced soil walls should be verified by the RSS supplier/designer. If an RSS block system wall is selected, the geotechnical aspects of the global stability of the detailed retaining wall design should be reviewed prior to construction. Depending on the design approach selected, an embedment depth equivalent to the frost depth may not be required for foundations of an RSS block system wall. This wall type can be constructed relatively quickly and inexpensively using small equipment.

6.3.2 Retaining Wall Foundations

Gabion walls may be founded directly on a 300 millimetre thick compacted Granular A pad. If required, a granular levelling course approximately 75 millimetres in thickness may be placed on the founding pad for gabion walls. Non-woven geotextile is to be placed between the gabions and the backfill. Gabion walls are to be constructed in accordance with OPSS 512 and 1860 and the manufacturer's specifications.

RSS walls may be designed such that the facing blocks are built on a levelling pad constructed with Granular A to a minimum thickness of 300 millimetres. Depending on the design selected by the RSS supplier, it may not be necessary to provide 1.2 metres of earth cover or thermal equivalent for frost protection. However the foundations must have adequate embedment to provide a stable structure. Typically the embedment depth, defined as the distance between the top of the levelling pad and the top of the adjoining finished grade, is a minimum of 500 millimetres.

All retaining wall foundations must be protected against scour as noted in the CHBDC Section 1.9.5. It is recommended that the replacement retaining walls be founded on the native clayey silt till encountered between elevations 205.1 and 205.3 metres which may be below the water level in the watercourse, which at the time of the investigation in October 2014 was at elevation 206.0 metres.

Retaining walls founded on the native clayey silt till may be designed using a factored geotechnical resistance at ULS of 300 kilopascals and a geotechnical reaction at SLS of 200 kilopascals. The SLS value corresponds to 25 millimetres of settlement.



6.3.3 Resistance to Lateral Forces

The resistance to lateral forces/sliding resistance between the retaining walls and the subgrade soils should be calculated in accordance with Section 6.7.5 of the CHBDC. Each retaining wall shall be checked for overturning. Assuming that the founding soils are not loosened/disturbed during excavation and footing construction, the following angles of friction and corresponding unfactored coefficient of friction, $\tan \delta$, may be used for the interaction between the base of the walls and the founding soil:

Wall Type	Interaction	Angle of Interface Friction, δ (degrees)	Coefficient of Interface Friction, $\tan \delta$
Gabion Wall	Gabion basket on Granular A leveling pad	30	0.58
Concrete Gravity and Cantilever Walls	Precast concrete walls on Granular A levelling pad	30	0.58
RSS Block System Wall	Precast concrete block facing units on Granular A levelling pad	30	0.58

6.4 Liquefaction Potential and Seismic Analysis

6.4.1 Seismic Parameters

The site is located near the towns of Jarvis and Simcoe in Southern Ontario. According to Table A.3.1.1 of the CHBDC, the zonal acceleration ratio, A , for Jarvis is not available, and for Simcoe is 0.05, which is considered applicable to this site. 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.

6.4.2 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).

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.



The potential for liquefaction occurring at this site is very low due to historically low seismicity in this area, founding soils with a normalized SPT (N_{160}) generally greater than 22 blows per 0.3 metres, and the relatively shallow depth to bedrock. Therefore, a detailed evaluation of the liquefaction potential of the foundation soils is not considered warranted.

6.5 Lateral Earth Pressures for Design

The lateral pressures acting on the proposed culvert and retaining walls 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 or Granular B Type II 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 C6.6.
- If the wall support does not allow lateral yielding (such is typically the case for a rigid concrete box culvert), 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).
- For Case (a), the restrained case, 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

Coefficients of lateral earth pressure:
'At rest' or restrained, K_0 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:

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

6.6 Construction Considerations

6.6.1 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 preparations 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 base should be inspected by the QVE to ensure that the clayey silt till has been reached and granular base materials or the footing should be placed immediately after inspection to protect the founding materials. The QVE should assess the foundation conditions to determine if sub-excavation of unsuitable material is required. Sub-excavation and placement and compaction fill should be carried out under the direction of the QVE.

A Non-standard Special Provision (NSSP) or Notice to the Contractor should be added to the Contract Documents to advise the Contractor of the potential for cobbles and boulders in the clayey silt till.

6.6.2 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. Temporary erosion protection and sedimentation control measures should be implemented in accordance with OPSS 805. Rip-rap treatment at the culvert outlet should be provided in accordance with OPSD 810.010. In addition, sediment control such as silt fences and erosion control blankets may be required during construction.



6.7 Excavations and Groundwater Control

Excavations will extend through the existing pavement structure, topsoil, fill and silty clay into the underlying clayey silt till. Cobbles and/or boulders should be expected in the clayey silt till deposit. It is anticipated that excavation for the culvert replacement will extend up to 0.8 metres below the inferred groundwater level of elevation 206.0 metres. Groundwater seepage from the native clayey silt till soils should be anticipated. Groundwater may be adequately controlled by pumping from properly constructed and filtered sumps located at the base of the excavation. Sumps, if used, should be maintained outside of the actual foundation limits.

Surficial water seepage into the excavation should be expected and will be heavier during periods of sustained precipitation. Surface water runoff should be directed away from the excavation at all times. The existing culvert flows will need to be diverted/piped during construction. The appropriate NSSP should be included in the contract documents to alert the contractor about the need for adequate control of surface and groundwater flows.

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 would be classified as Type 3 materials. The silty clay and clayey silt till soils would be considered Type 2 materials.

6.8 Staging and Temporary Roadway Protection

It is understood that a single lane is to remain open to traffic during construction with one half of the culvert being replaced at a time. Considering the shallow depth of bedrock and the presence of low permeability overburden material, it may be most feasible to install a soldier pile and lagging system with the soldier piles installed in pre-drilled holes. Consideration may also be given to a rapid replacement scheme where the road is closed for a 24-hour period during which the culvert is replaced within an open cut excavation. Alternatively, if a full road closure is not permissible, RSS walls may be used for shoring. 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 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.



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 (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 206.0 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			Effective Internal Angle of Friction, ϕ' (degrees)	Bulk Unit Weight, γ (kN/m^3)	Effective Unit Weight, γ' (kN/m^3)
	Active, K_a	At Rest, K_o	Passive, K_p			
Fill	0.36	0.53	2.8	28	19	9.0
Silty Clay	0.36	0.53	2.8	28	20	10.0
Clayey Silt Till	0.31	0.47	3.3	32	22	12.0

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. Azmi M. Hammoud, P.Eng., an Associate 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

ORIGINAL SIGNED

Dirka U. Prout, P.Eng.
Project Engineer

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

BT/NG/DUP/AMH/MEB/FJH/cr

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n:\active\2012\1132 - geo\1132-0100\12-1132-0163 stantec-fdns-mega culverts-3011-e-0041\ph 2000-gwp 3040-11-00\rvts\10 - site 9-155 (sandusk creek)\1211320163-2000-r10 jan 28 15 (final) part a&b fdns repl clvrt 9-155-c .docx

TABLE I

COMPARISON OF STRUCTURE ALTERNATIVES FOR REPLACEMENT CULVERT

Sandusk Creek Tributary, Site 9-155/C, Station 14+537, Highway 3
 Structure Replacements and Rehabilitation
 GWP 3040-11-00

FOUNDATION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	ESTIMATED COSTS	RISKS/ CONSEQUENCES
Precast box culvert founded on very stiff 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 CIP alternatives since there is no wait for concrete to cure • Suitable for corrosive environments • Can be more readily installed during cold weather conditions 	<ul style="list-style-type: none"> • If floor is thin and poorly reinforced, it may heave and crack • 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 box culvert founded on very stiff 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 environments • Culvert design can be customized in the field for high stress or load conditions or other site- 	<ul style="list-style-type: none"> • If floor is thin and poorly reinforced, it may heave and crack • During high flows, the concrete floor can be undermined • More expensive compared to precast box option due to increased labour associated with concrete formwork 	<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
		specific requirements <ul style="list-style-type: none"> • Can be constructed with far fewer joints than a precast box culvert 	<ul style="list-style-type: none"> • Special curing requirements for cold weather work 		
CIP open footing culvert with spread footings founded on 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 • Most suitable where maintenance of fish and/or wildlife passage and preservation of the natural stream bed is a priority 	<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 entrance 	<ul style="list-style-type: none"> • Moderate 	<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: AMH

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)

PROJECT 12-1132-0163		RECORD OF BOREHOLE No 230		1 OF 1 METRIC	
W.P. 3040-11-00		LOCATION N 4750007.5 , E 256927.5		ORIGINATED BY NG	
DIST _____ HWY 3		BOREHOLE TYPE POWER AUGER, HOLLOW STEM		COMPILED BY LMK	
DATUM GEODETIC		DATE October 16, 2014		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			
207.41	GROUND SURFACE													
0.00	FILL, roadbase, crushed sand and gravel													
207.05	Grey													
0.41	TOPSOIL, clayey													
	Black		1	SS	4						○			
	SILTY CLAY, some sand, trace gravel, trace topsoil													
	Firm		2	SS	7						○			
	Mottled brown and grey												44	2 9 49 40
205.28														
2.13	CLAYEY SILT TILL, some sand, trace gravel, silty sand to sandy silt seams		3	SS	15						○			
	Very stiff to hard													
	Brown		4	SS	15						○			
														3 24 49 24
			5	SS	38						○			
202.38			6	SS	28						○			
5.03	END OF BOREHOLE		7	SS	100/0mm						○			
	Auger refusal on probable bedrock.													
	Groundwater encountered at about elev. 203.8m during drilling on October 16, 2014.													

RECORD OF BOREHOLE No 231

1 OF 1

METRIC

PROJECT 12-1132-0163
W.P. 3040-11-00 LOCATION N 4749990.6 , E 256924.7 ORIGINATED BY NG
DIST HWY 3 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY LMK
DATUM GEODETIC DATE October 16, 2014 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL						
206.46	GROUND SURFACE															
0.00	FILL, silty clay, some gravel															
0.14	FIBROUS MATERIAL															
0.25	TOPSOIL, clayey Brown															
	SILTY CLAY, some sand Firm to stiff Mottled brown and grey		1	SS	8								43		0 12 45 43	
205.09																
1.37	CLAYEY SILT TILL, some sand, trace gravel Very stiff to hard Brown		2	SS	21											
			3	SS	24											
			4	SS	22										6 18 48 28	
202.35			5	SS	100/ 165mm											
4.11	END OF BOREHOLE															
	Auger refusal on probable bedrock.															
	Groundwater encountered at about elev. 204.0m during drilling on October 16, 2014.															
	Water level measured in pipe after installation at elev. 203.33m on October 16, 2014.															
	Water level measured at elev. 205.85m on December 1, 2014.															

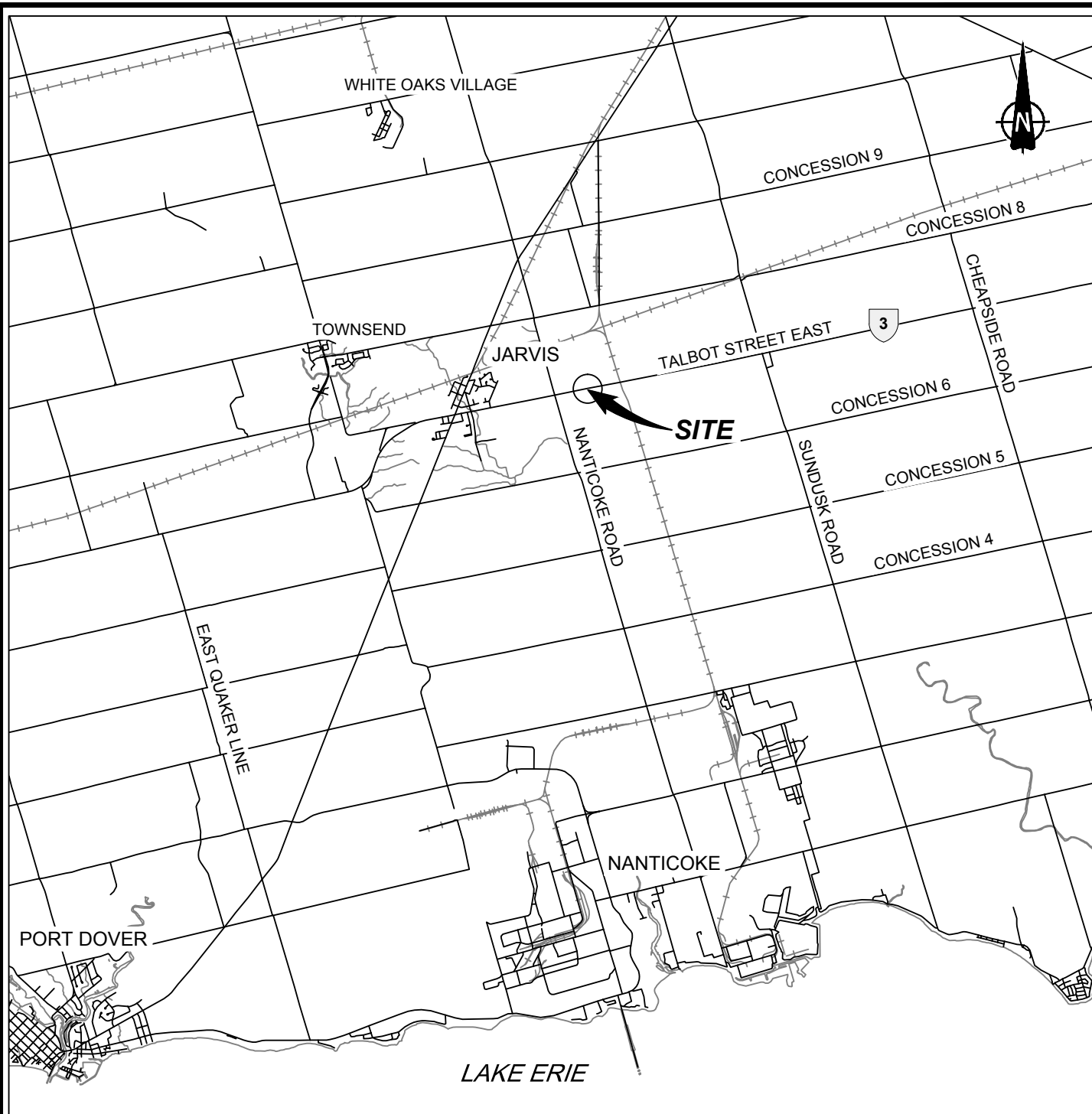
RECORD OF BOREHOLE No 232

1 OF 1

METRIC

PROJECT 12-1132-0163
W.P. 3040-11-00 LOCATION N 4750000.5 , E 256954.8 ORIGINATED BY NG
DIST HWY 3 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY LMK
DATUM GEODETIC DATE October 16, 2014 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 (%)					
207.43	GROUND SURFACE						20	40	60	80	100						
0.00	FILL, roadbase, sand and gravel Grey																
0.30	FILL, silty clay, some sand Firm																
206.52	Brown and grey		1	SS	4								○		○		
0.91	SILTY CLAY, some sand, trace gravel Firm		2	SS	5											52	
	Brown and grey mottled																0 6 46 48
205.30																	
2.13	CLAYEY SILT TILL, some sand to sandy, trace gravel, trace topsoil, fissured above about elev. 204.4m Very stiff to hard		3	SS	21								○				
	Brown and grey mottled to brown		4	SS	16								○				
			5	SS	16								○				
																	3 29 49 19
202.40			6	SS	62/ 275mm								○				
5.03	END OF BOREHOLE																
	Auger refusal on probable bedrock.																
	Groundwater encountered at about elev. 204.7m drilling on October 16, 2014.																



REFERENCE

PLAN BASED ON CANMAP STREETFILES V.2008.5.

NOTE

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

PROJECT

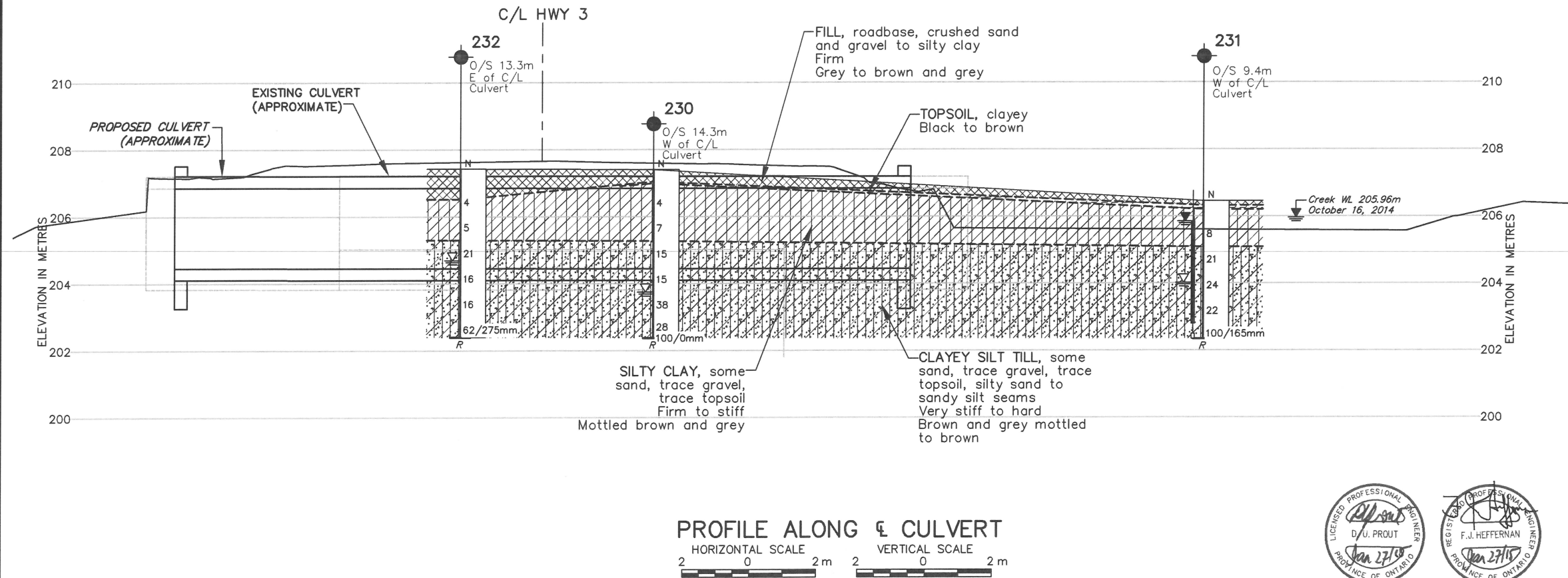
CULVERT REPLACEMENT, SITE 9-155/C
STATION 14+537, HIGHWAY 3
GWP 3040-11-00

TITLE

KEY PLAN








PROJECT No.		12-1132-0163	FILE No.		1211320163-2000-F10001
CADD	LMK	Nov. 13/14	SCALE	AS SHOWN	REV. 0
CHECK			FIGURE 1		



SHEET



- | | |
|---|--|
|  | 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 December 1, 2014. |
|  | WL encountered during drilling |
| R | Refusal on probable bedrock |

No.	ELEVATION	CO-ORDINATES (MTM NAD83 ZONE 10)	
		NORTHING	EASTING
230	207.41	4 750 007.5	256 927.5
231	206.46	4 749 990.6	256 924.7
232	207.43	4 750 000.5	256 954.8

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

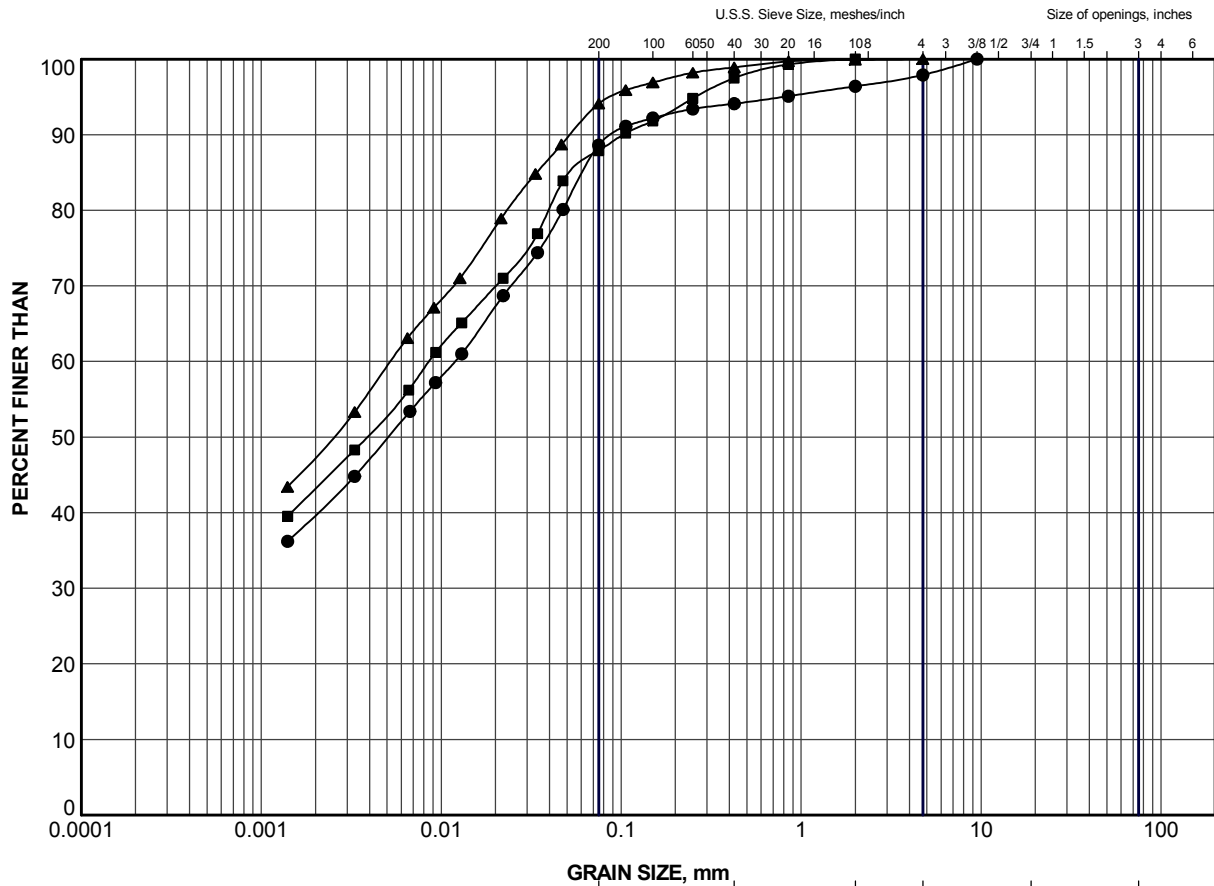
Base plans provided by Stantec.

NO.		DATE		BY		REVISION			
Geocres No. 40116-525									
HWY. 3				PROJECT NO. 12-1132-0163				DIST.	
SUBM'D. BT			CHKD. BT			DATE: Dec. 02/14		SITE: 9-155/C	
DRAWN: LMK			CHKD. DUP			APPD. FJH		DWG. 1	




APPENDIX A

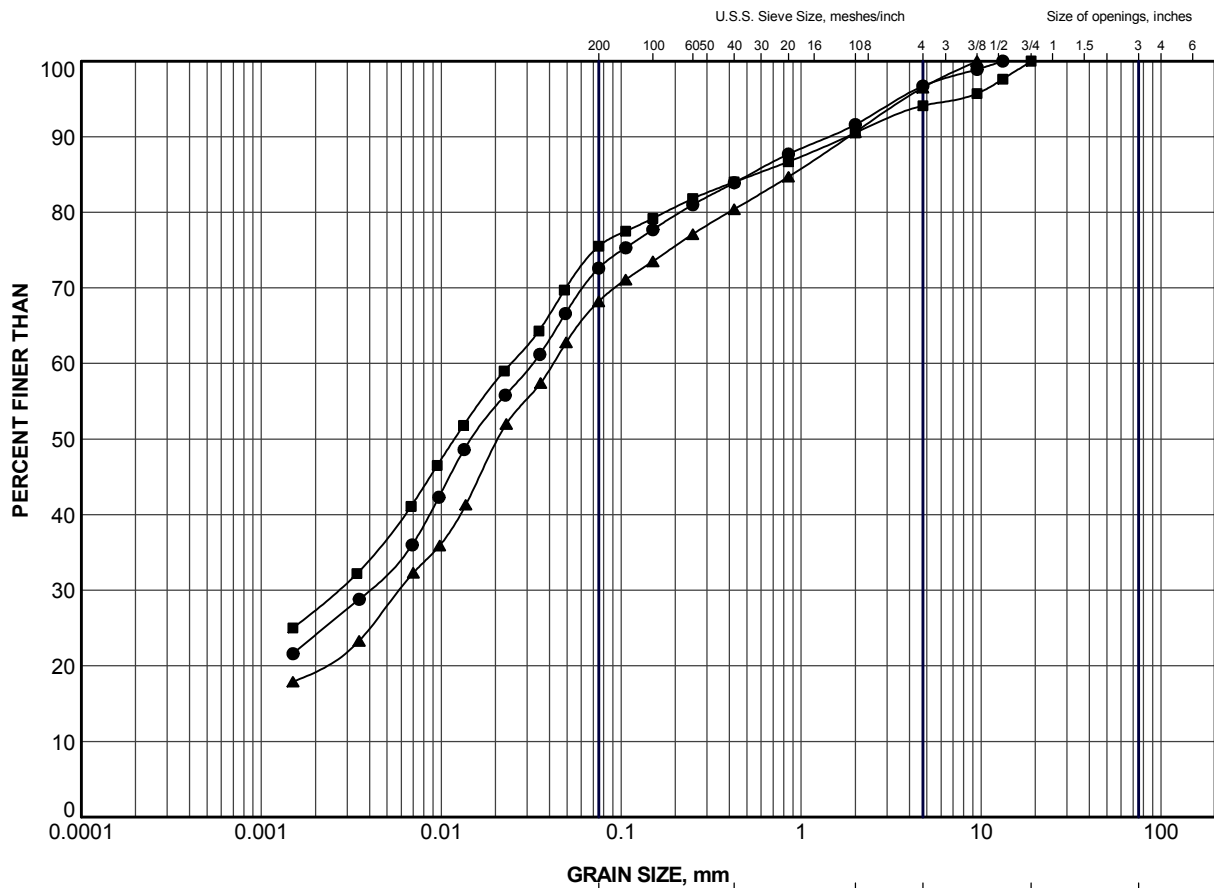
Laboratory Test Data



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	230	2	207.7
■	231	1	205.5
▲	232	2	205.7

PROJECT	CULVERT REPLACEMENT, SITE 9-155/C STATION 14+537, HIGHWAY 3 GWP 3040-11-00			
TITLE	GRAIN SIZE DISTRIBUTION SILTY CLAY			
 Golder Associates LONDON, ONTARIO	PROJECT No.	12-1132-0163	FILE No.	1211320163-2000-F100A1
	DRAWN	LMK	Nov 04/14	SCALE N/A REV.
	CHECK			
			FIGURE A-1	



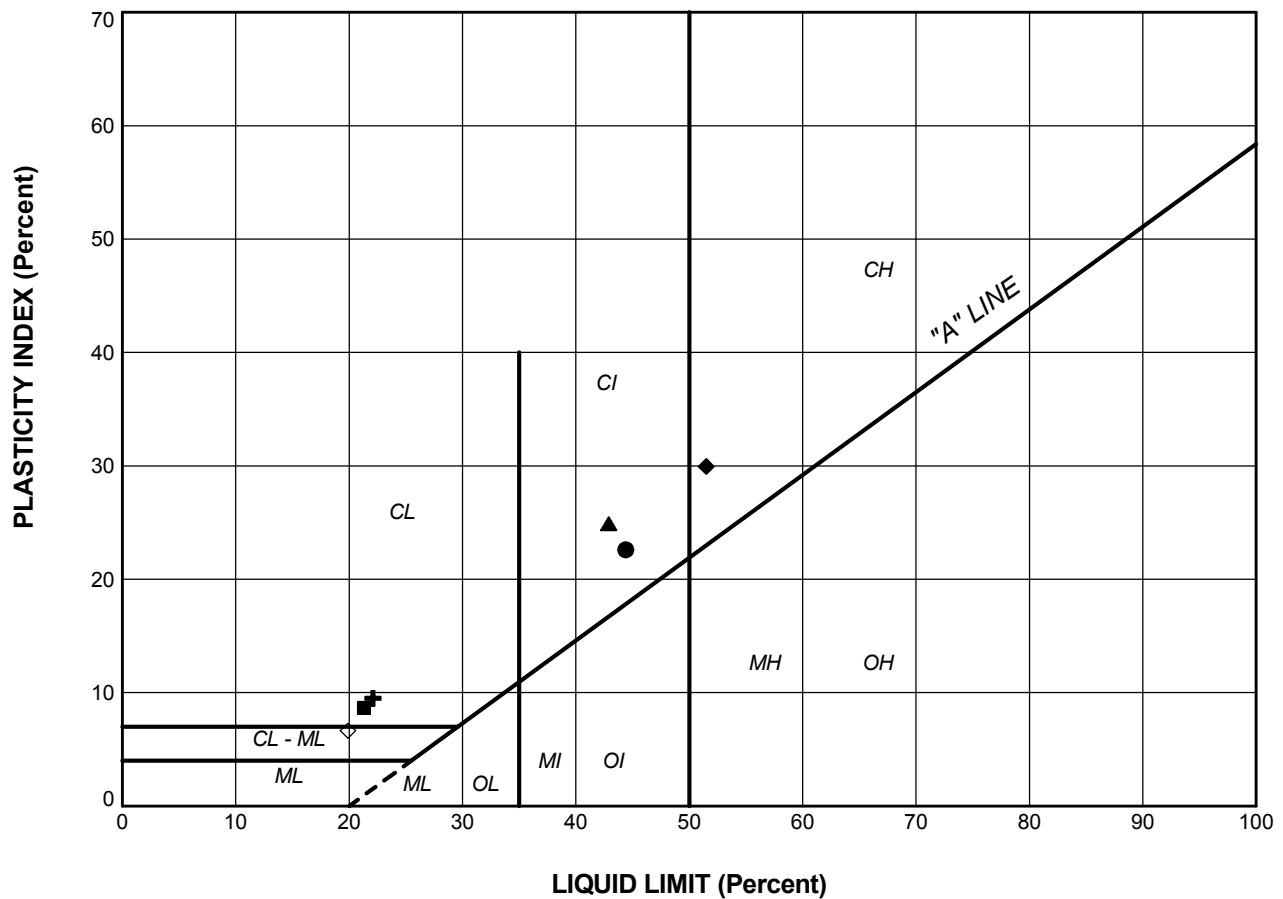
GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	230	4	204.1
■	231	4	203.2
▲	232	5	203.4

PROJECT				CULVERT REPLACEMENT, SITE 9-155/C STATION 14+537, HIGHWAY 3 GWP 3040-11-00			
TITLE				GRAIN SIZE DISTRIBUTION CLAYEY SILT TILL			
PROJECT No.		12-1132-0163		FILE No. 1211320163-2000-F010A2			
DRAWN		LMK		Nov 14/14		SCALE N/A REV.	
CHECK						FIGURE A-2	





SOIL TYPE
 C = Clay
 M = Silt
 O = Organic

PLASTICITY
 L = Low
 I = Intermediate
 H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	230	2	44.4	21.8	22.6
■	230	4	21.3	12.7	8.7
▲	231	1	42.9	18.0	24.9
⊕	231	4	22.1	12.6	9.5
◆	232	2	51.5	21.6	30.0
◇	232	5	19.9	13.3	6.7

PROJECT				CULVERT REPLACEMENT, SITE 9-155/C STATION 14+537, HIGHWAY 3 GWP 3040-11-00			
TITLE							
PLASTICITY CHART							
PROJECT No.		12-1132-0163		FILE No. 1211320163-2000-F100A3			
DRAWN	LMK	Nov 04/14		SCALE	N/A	REV.	
CHECK				FIGURE A-3			





APPENDIX B

Site Photographs



APPENDIX B PHOTOGRAPHS



Photograph 1: North elevation (inlet) of the Sandusk Creek Tributary Culvert Site 9-155/C.



Photograph 2: South elevation (outlet) of Culvert Site 9-155/C.



APPENDIX B PHOTOGRAPHS



Photograph 3: Highway 3, looking east from north shoulder at Culvert Site 9-155/C.

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15 (final) app b - photos.docx

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