



April 2017

FOUNDATION INVESTIGATION AND DESIGN REPORT

**Culvert Replacement
Culvert No. 4-174/C, Highway 10
Dufferin County, Ontario
GWP 3078-12-00
Ministry of Transportation, Ontario - West Region**

Submitted to:

Mr. Bradley Schmidt, M.E.Sc., P.Eng., Associate
Dillon Consulting Limited
1400-130 Dufferin Avenue
London, Ontario
N6A 5R2

REPORT



Report Number: 1526476-4000-R01

Geocres No.: 40P16-25

Distribution:

4 Copies - Dillon Consulting Limited
1 eCopy - Golder Associates Ltd.





Table of Contents

PART A – FOUNDATION INVESTIGATION REPORT

1.0 INTRODUCTION.....	1
2.0 SITE DESCRIPTION.....	1
2.1 Site Geology	2
3.0 INVESTIGATION PROCEDURES	2
4.0 SUBSURFACE CONDITIONS.....	3
4.1 Site Stratigraphy	3
4.2 Soil Conditions.....	3
4.3 Groundwater Conditions	4
5.0 MISCELLANEOUS	5

PART B – FOUNDATION DESIGN REPORT

6.0 ENGINEERING RECOMMENDATIONS.....	6
6.1 Consequence and Site Understanding Classification	6
6.2 Foundations.....	6
6.2.1 Frost and Scour Protection	7
6.2.2 Resistance to Lateral Forces/Sliding Resistance	7
6.2.3 Bedding, Backfill and Cover	8
6.2.4 Other Design Considerations.....	8
6.3 Excavations and Groundwater Control	9
6.4 Retaining Walls.....	9
6.4.1 Frost Protection and Embedment	10
6.4.2 Lateral Resistance	10
6.4.3 Other Design Considerations	11
6.5 Liquefaction Potential and Seismic Analysis.....	11
6.5.1 Seismic Parameters	11
6.5.2 Seismic Hazard Assessment	11
6.6 Lateral Earth Pressures for Design.....	12



FOUNDATION INVESTIGATION AND DESIGN REPORT

SITE NO. 4-174/C, HIGHWAY 10, GWP 3078-12-00

6.7	Temporary Roadway Protection	13
6.8	Construction Considerations.....	14
7.0	MISCELLANEOUS	15

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
SITE NO. 4-174/C, HIGHWAY 10
GWP 3078-12-00
MINISTRY OF TRANSPORTATION, ONTARIO - WEST REGION



1.0 INTRODUCTION

Golder Associates Ltd. (Golder Associates) has been retained by Dillon Consulting Ltd. (Dillon) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out a foundation investigation as part of the detailed design work for GWP 3078-12-00. The project involves the detailed design of the replacement and rehabilitation of several structures along Highway 10 from Orangeville to Camilla. This report addresses the proposed replacement of Culvert No. 4-174/C at about Station 14+527 on Highway 10 in the Geographic Township of Mono in Dufferin County.

The purpose of the foundation investigation is to explore the subsurface conditions at the location of the proposed culvert 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 P1526476 dated January 20, 2016. The work was carried out in accordance with our Quality Control Plan for Foundation Engineering dated March 22, 2015.

2.0 SITE DESCRIPTION

The subject site is situated at about Station 14+527 on Highway 10 approximately 240 metres south of Campbell Road in the Geographic Township of Mono in Dufferin County, Ontario. The town of Orangeville is located approximately 5 kilometres to the south of the site. The location of the culvert is shown on the Key Plan, Figure 1. Photographs of the culvert are presented in Appendix B.

This section of Highway 10 is currently a four lane, undivided highway with gravel shoulders. It is generally oriented north-south in the vicinity of the subject site. An unnamed watercourse flows in the culvert from west to east beneath Highway 10. Based on the information provided, the existing culvert has an unknown date of construction and has since been extended twice on either side, with the most recent extensions taking place in 1988. The original structure and extensions are concrete open footings culverts. The length of the first extensions on the west and east sides were 12.1 and 6.3 metres, respectively, and the second extensions were both 4.0 metres in length. The original structure has a length of about 11.9 metres.

Dimensions (m)	Obvert Elevation (m)		Construction
	Lt'	Rt'	
6.10 x 3.64 x 38.29	420.57	420.53	Concrete Open Footing

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 watercourse and the embankments along Highway 10 near the culvert are grass covered. Erosion was noted along the north sides of the culvert and is shown in Photographs 2 and 4 in Appendix B. Sand bags have been placed at the outlet of the culvert and are shown in Photograph 5 in Appendix B. The watercourse flows through a wooded area on the east side of Highway 10 and through a residential area on the west side of Highway 10.



2.1 Site Geology

The site lies within the Hillsburgh Sandhills physiographic region¹. The quaternary geology mapping indicates that the surficial materials consist primarily of alluvium, consisting of silt, sand and gravel². The underlying bedrock surface is typically found at an elevation of about 411 to 419 metres³. However, the more recent drift thickness mapping suggests that while the overburden thickness in this area typically ranges between 6 and 12 metres, the drift thickness adjacent to the creek channel is deeper at 19 to 21 metres⁴. The rock formation is mapped and described as grey or mottled brown, fine to medium crystalline dolostone of the Manitoulin Formation of the Cataract Group⁵.

3.0 INVESTIGATION PROCEDURES

The field work for the investigation was carried out on October 6 and 27, 2016, during which time three boreholes were drilled at the approximate locations shown on the Borehole Location Plan, Drawing 1.

Boreholes 101 and 102 were drilled using track-mounted D50T drilling equipment supplied and operated by a specialist drilling contractor. Borehole 103 was advanced at the culvert outlet using manual augering equipment since it was not possible to access the outlet with a track-mounted drill rig. In boreholes 101 and 102, 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). An auger sample was obtained from borehole 103.

The recorded SPT N values are noted on the relevant Record of Borehole sheets. 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 investigation 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. Larger particle sizes, including cobbles and boulders, are known to be present in the fill, glacial tills and native granular deposits as discussed in the text of this report.

Groundwater conditions in the boreholes were observed throughout the drilling operations and a piezometer was installed in borehole 101 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, 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, grain size distribution analyses and an Atterberg limits determination, were carried out on selected samples. The results of the testing are shown on the Record of Borehole sheets and in Appendix A.

¹ Chapman, L.J. and Putnam, D.F., 1984: The Physiography of Southern Ontario, Third Edition. Ontario Geological Survey, Special Volume 2.

² Ontario Division of Mines, 1975: Quaternary Geology, Orangeville, Southern Ontario, Map 2326, scale 1:50,000.

³ Ontario Department of Mines, 1962: Bedrock Topography Series, Orangeville Sheet, Preliminary Map P.170, scale 1:50,000.

⁴ Ontario Division of Mines, 1976: Drift Thickness, Orangeville, Southern Ontario, Map P.2328, scale 1:50,000.

⁵ Liberty, B.A., Bond, I.J., Telford, P.G., 1976: Paleozoic Geology, Orangeville, southern Ontario. Geological Survey of Canada, Map 2339, scale 1:50,000.



FOUNDATION INVESTIGATION AND DESIGN REPORT

SITE NO. 4-174/C, HIGHWAY 10, GWP 3078-12-00

The as-drilled borehole locations and ground surface elevations at the borehole locations 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		
101	4 868 411	256 155	422.12	13.81
102	4 868 397	256 138	422.01	14.17
103	4 868 400	256 169	419.77	1.75

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 surficial topsoil or embankment fill materials overlying buried topsoil, native granular soils, sandy silt till and clayey 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

Sand and crushed gravel, interpreted to be granular base from visual and textural examination, was encountered at the ground surface in boreholes 101 and 102 and was 610 and 380 millimetres thick, respectively.

Layers of loose to compact fill, ranging from sandy silt to silty sand and gravel, were encountered in boreholes 101 and 102 beneath the granular subbase at approximate elevation 421.6 metres, and in borehole 103 beneath the surficial topsoil at elevation 419.5 metres. The thickness of the various fill layers ranged from about 2.1 to 3.1 metres and SPT N values ranged from 5 to 18 blows per 0.3 metres. The water content of a sample of silty sand fill from borehole 101 was about 7 per cent. A grain size distribution analysis for a sample of the silty sand fill from borehole 101 is presented on Figure A-1 in Appendix A.

Buried topsoil was encountered in borehole 102 at elevation 419.1 metres beneath the fill. Surficial topsoil was encountered at the ground surface in borehole 103. The buried and surficial topsoil layers were 760 and 320 millimetres thick, respectively. Variable amounts of topsoil, organics and wood pieces were encountered in the silty sand and gravel and sand in borehole 101 at elevation 415.7 to 418.5 metres, the silty sand in borehole 102 at elevation 417.6 to 418.4 metres and the fill, silty sand and sand in borehole 103 at elevation 418.0 to 419.5



metres. 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.

Compact to very dense native granular deposits, ranging from silty sand to sand and gravel, were encountered in boreholes 101 and 103 beneath the fill at elevation 418.5 and 418.8 metres, respectively, and in borehole 102 below the topsoil at elevation 418.4. The various layers of native granular deposits ranged in thickness from about 3.4 to 6.5 metres. Standard penetration test N values ranged from 10 to over 100 blows per 0.3 metres and water contents ranged from about 10 to 28 per cent. Grain size analyses carried out on samples of the silty sand, sand, and silt and sand are presented on Figures A-2, A-3 and A-4, respectively, in Appendix A.

Glacial till, ranging from compact to very dense sandy silt till to very stiff to hard clayey silt till, was encountered in borehole 101 at elevation 412.0 metres beneath the silty sand and in borehole 102 below elevation 414.9 metres. Both boreholes were terminated in the glacial till after exploring it for 3.7 metres in borehole 101 and for 1.4 metres in borehole 102 below a 0.4 metre silt layer. Above the silt layer in borehole 102, the glacial till was 5.3 metres thick. The glacial till had SPT N values ranging from 23 to over 100 blows per 0.3 metres. Samples of sandy silt till had a water content of about 10 per cent. An Atterberg limits determination carried out on this sample yielded a liquid limit of about 17 per cent, a plastic limit of about 12 per cent and a plasticity index of about 5 per cent. A grain size analysis and the results of the Atterberg limits determination for a sample of the sandy silt till from borehole 101 are presented on Figures A-5 and A-6, respectively, in Appendix A.

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 101. The installation details are provided on the corresponding Record of Borehole sheet following the text of this report. Groundwater was encountered in the boreholes during drilling between depths of 1.3 and 4.2 metres, or between elevations of 417.9 and 418.5 metres. On October 27, 2016, the water level in the piezometer installed in borehole 101 was about 3.28 metres below ground surface or at about elevation 418.84 metres. The water level in the watercourse was measured at the outlet at elevation 418.7 metres on October 6 and 27, 2016. The General Arrangement Drawing indicated an approximate creek water level of 418.68 metres near the outlet and 419.05 metres near the inlet in June 2016. 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)
			October 27, 2016
101	422.12	417.9	418.84
102	422.01	418.4	-
103	419.77	418.5	-

The above-noted encountered water levels are not considered to be representative of the long-term, stabilized groundwater conditions.



FOUNDATION INVESTIGATION AND DESIGN REPORT SITE NO. 4-174/C, HIGHWAY 10, GWP 3078-12-00

Based on the observed groundwater levels and the surrounding topography, the groundwater level is inferred to typically be at about elevation 419.0 metres. 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.

5.0 MISCELLANEOUS

The investigation was carried out using equipment supplied and operated by London Soil Test Ltd., an Ontario Ministry of Environment and Climate Change licensed well contractor. The field operations were supervised by Mr. Daniel Hyland, E.I.T. under the direction of the Field Investigation Manager, Mr. Brett Thorner, P.Eng. The laboratory testing was carried out at Golder Associates' London laboratory under the direction of Mr. Michael Arthur. 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. Daniel Hyland, E.I.T. under the direction of the Project Engineer Ms. Dirka U. Prout, P.Eng. The report was reviewed by Mr. W. Kellestine, P.Eng., a Senior Consultant 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.


Dirka U. Prout, P.Eng.
Senior Geotechnical Engineer

W.M. Kellestine, P.Eng.
Senior Consultant


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

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

DH/DUP/WMK/FJH/CR

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

n:\active\2015\3 proj\1526476 dillon_3014-e-0028 orangeville_hwy 10\ph 4000-fdns part b\2-correspondence\5-rpts\final\1526476-4000-r01 apr 17 17 (final) hwy 10 culverts 4-174-c.docx



PART B

FOUNDATION DESIGN REPORT

CULVERT REPLACEMENT

SITE NO. 4-174/C, HIGHWAY 10

GWP 3078-12-00

MINISTRY OF TRANSPORTATION, ONTARIO - WEST REGION



6.0 ENGINEERING RECOMMENDATIONS

This section of the report provides our recommendations on the foundation aspects of the design of the proposed culvert replacement at Site 4-174/C at Station 14+527 on Highway 10 in the Geographic Township of Mono in Dufferin County, Ontario. This section of the final report will be amended, as required, based on any changes to the design of the culvert.

The recommendations are based on our interpretation of the factual data obtained from the boreholes advanced during the investigation at this site. The interpretation and recommendations are intended to provide the designers with sufficient information to design the proposed foundations. As such, where comments are made on construction, they are provided only 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.

Based on the information provided, the existing culvert has an unknown date of construction and has since been extended twice on either side, with the most recent extensions taking place in 1988. The original structure and extensions are concrete open footings culverts. The first extensions on the west and east sides were 12.1 and 6.3 metres, respectively, and the second extensions were both 4.0 metres. The original structure has a length of about 11.9 metres. The culvert has a span of about 6.10 metres and a 2.44 metres high opening with an approximate invert elevation of 418.1 metres.

The proposed replacement culvert will have a span of about 6.1 metres and a rise of 2.8 metres. The 2.8 metre rise will consist of 0.3 metres of compacted native material and a 2.5 metre high opening. The length of the replacement culvert will remain unchanged at about 38.3 metres. The proposed invert elevation of the replacement structure is at about elevation 418.1 metres.

6.1 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the Canadian Highway Bridge Design Code (CHBDC 2014) and its Commentary, a classification of 'typical' consequence has been assumed for the proposed replacement culvert section and foundation system. This consequence classification should be confirmed by Dillon and the MTO.

The degree of understanding based on the scope of the foundation investigation and proximity of the boreholes to the culvert is considered 'typical' as described in Clause 6.5.3.2 of the 2014 CHBDC. The appropriate Ultimate Limit States (ULS) and Serviceability Limits (SLS) consequence factor Ψ , geotechnical resistance factors at ULS (ϕ_{gu}) and SLS (ϕ_{gs}), respectively from Tables 6.1 and 6.2 of the CHBDC have been used for design.

6.2 Foundations

The proposed invert of the replacement structure is about elevation 418.1 metres. However, it is expected that in order to remove the existing footings, excavations will extend deeper. Considering the proposed invert elevation and the proposed box culvert base slab thickness of 300 millimetres, the culvert may be founded on sand and gravel or sand at or below elevation 417.4 metres. The footings should be founded on competent native soils below all fill or organic layers. The culvert foundations may be designed using a factored geotechnical resistance at ULS of 350 kilopascals (kPa) and a geotechnical reaction at SLS of 200 kPa. The SLS values corresponds to



25 millimetres of settlement. If unsuitable material is present at the founding level, it must be subexcavated and replaced with suitable granular fill. The fill should consist of a uniformly compacted Granular A base or Granular B Type II placed in maximum 300 millimetre thick loose lifts in accordance with Ontario Provincial Standard Specification (OPSS).PROV 501. The engineered fill should extend a minimum distance of 1 metres beyond the outside walls of the box culvert and slope from the edge of the fill surface at 1 horizontal to 1 vertical (1H:1V) for a distance equivalent to the height of fill placed.

6.2.1 Frost and Scour Protection

Frost treatment in the form of a frost taper symmetrical about the culvert centreline should be provided in accordance with Ontario Provincial Standard Drawing (OPSD) 3101.150. The design frost penetration depth for this area is 1.4 metres below ground surface. The culvert should also be adequately protected against scour as noted in Section 1.9.5 of the CHBDC.

6.2.2 Resistance to Lateral Forces/Sliding Resistance

The resistance to lateral forces/sliding resistance between the base of the culvert and the bedding should be calculated in accordance with Section 6.10.5 of the CHBDC.

The factored horizontal geotechnical resistance, H_{ri} , is calculated as follows:

$$H_{ri} = \psi \phi_{gu} (A' c'_i + V_f \tan \delta_i) > H_f$$

Where:

ψ	=	consequence factor, given in Section 6.5.2, Table 6.1 of the CHBDC
ϕ_{gu}	=	ultimate geotechnical resistance factor frictional, given in Section 6.9.1, Table 6.2 of the CHBDC
A'	=	effective contact area, square metres
c'_i	=	effective cohesion intercept, nil
$\tan \delta_i$	=	friction factor, given in the table below
V_f	=	factored vertical force, kilonewtons
H_f	=	factored 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, $\tan \delta$
Precast Box Culvert	Precast concrete on Granular A levelling pad/Granular B, Type II bedding	30	0.58

6.2.3 Bedding, Backfill and Cover

Backfill for the culvert shall be placed in accordance with OPSS.PROV 501 and Special Provision (SP) 422S01. Backfill for the culvert should consist of free-draining, non-frost susceptible granular materials such as OPSS Granular B, Type II or III or Granular A placed in 0.3 metre thick loose lifts and uniformly compacted. 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 as equal as possible on both sides of the culvert during all stages of backfill placement. The height of the backfill at each side of the culvert should differ no more than 500 millimetres at any time.

The excavations for this culvert should have a clearance width that exceeds the width of the culvert by at least 1.0 metre on each side to allow for good workmanship and effective compaction of the fill. Bedding for the precast box culvert is to be placed on properly prepared native competent materials or approved compacted granular materials. At no time should the culvert be constructed on frozen materials. Granular A would be considered suitable for use as bedding material where a precast box culvert is to be installed, however due to the high water level Granular B, Type II is recommended. The bedding materials should be 300 millimetres thick. The levelling course can consist of a 75 millimetre thick layer of Granular A or materials meeting the gradation requirements for fine concrete aggregates.

6.2.4 Other Design Considerations

The fill height above the culvert roof will be less than 4.5 metres. The foundation materials consist of granular deposits underlain by low compressibility glacial tills. Therefore, differential settlement along the length of this culvert is expected to be negligible and cambering is not required.

If the results of hydraulic analysis indicate that there will be significant difference in hydraulic head between the inlet and outlet, then the culvert inlet must be provided with a headwall, clay seal or other seepage control measure. The design of the replacement box culvert should include a cut-off wall at the inlet in accordance with Section 1.9.5.6 of the CHDBC.

Erosion protection for the culvert backfill should be provided to protect the roadway, approach embankments and culvert, 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. In addition, sediment control such as silt fences and erosion control blankets may be required during construction together with diversion of any flows to mitigate migration of fine soil particles.



6.3 Excavations and Groundwater Control

Excavations for the proposed work will extend to approximate elevation 416.7 metres and will encounter fill materials, silty sand, sand and sand and gravel. Cobbles and boulders should be expected in the fill, granular materials and glacial tills. The Contractor should be alerted to the need to excavate cobbles and boulders within these materials with the appropriate Non-Standard Special Provision (NSSP) or Notice To Contractor.

Seepage volumes from the native granular deposits are expected to be such that groundwater control may not be achieved solely by the use of sumps due to the high permeability of the soils at the site. It may be necessary to use more proactive measures such as wellpoint systems, deep wells, or the like. The groundwater level is to be lowered to a minimum of 0.3 metres below the elevation of the founding subgrade. Use of steel sheet piles as a groundwater cut-off may reduce the dewatering effort provided the sheets are driven into the relatively low permeability glacial till or the silty sand at or below elevation 414 metres. An NSSP requiring the Contractor to provide groundwater and surface water control should be added to the Contract Documents. The existing culvert flows will need to be diverted/piped during construction and a Category 3 Permit to Take Water (PTTW) will likely be required. 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.

Temporary open cut slopes within the granular fill and native materials should be maintained no steeper than 1 horizontal to 1 vertical above the groundwater level. Use of flatter slopes or blanketing of the sideslopes with coarse granular material may be required to improve stability of the cut slopes. Localized sloughing of the sideslopes may occur since the granular fills and native deposits are expected to be raveling to cohesive running above the groundwater level and running to flowing below the groundwater level.

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 a Type 3 soil. The native granular deposits would be classified as Type 2 soils above the groundwater level or if properly dewatered and Type 4 soils below the groundwater level.

6.4 Retaining Walls

Based on the information provided to Golder, it is understood that retaining walls will be constructed in the southeast and northeast quadrants in conjunction with the culvert replacement. It is understood that the retaining walls would be about 2.5 to 3.5 metres in height. From a geotechnical perspective, gabion walls, precast concrete block walls, armour stone walls, concrete cantilever walls, concrete gravity walls, and precast toe walls are suitable alternatives. Gabion walls, precast concrete block walls, armour stone walls and precast concrete toe walls are probably more economical than RSS walls and concrete cantilever or other concrete gravity walls. Gabion walls, precast concrete block walls, armour stone walls and RSS walls need not be founded at the frost depth and these wall types are most tolerant of movement. Based on the information to Golder provided it is expected that gabion walls will be constructed.

The retaining walls can be supported on the compact to very dense granular soils at or below elevation 417.4 metres. A factored geotechnical resistance of 350 kPa at ULS and a geotechnical resistance of 200 kPa at SLS may be used for design of the retaining walls. The SLS value corresponds to 25 millimetres of total settlement.



6.4.1 Frost Protection and Embedment

The frost depth applicable to this site is 1.4 metres. Gabion walls, precast concrete block walls, armour stone walls and RSS walls do not require an embedment depth equivalent to the frost depth provided they are founded on granular pads with a compacted thickness of at least 300 millimetres. In addition, the retaining walls should have sufficient embedment to provide stability and adequate protection against scour and erosion. Proprietary systems may have additional foundation preparation requirements and should be designed and constructed in accordance with the supplier's requirements.

6.4.2 Lateral Resistance

The resistance to lateral forces/sliding resistance between the underside of the retaining walls and levelling pads should be calculated in accordance with Section 6.10.5 of the CHBDC. In addition, the retaining walls should be checked for overturning. The equations and soil parameters provided below should be used for the interaction between the retaining walls and the founding soil. The passive resistance of the portion of the retaining walls below grade should be neglected. Based on our preliminary calculations in the case of gabion walls, in order to provide adequate protection against overturning and sliding, the base should be at least 3 metres wide, decreasing gradually in width towards the top of the wall.

The factored horizontal geotechnical resistance, H_{ri} , is calculated as follows:

$$H_{ri} = \psi \phi_{gu} (A' c'_i + V_f \tan \delta_i) > H_f$$

Where:

ψ	=	consequence factor, given in Section 6.5.2, Table 6.1 of the CHBDC
ϕ_{gu}	=	ultimate geotechnical resistance factor frictional, given in Section 6.9.1, Table 6.2 of the CHBDC
A'	=	effective contact area, square metres
c'_i	=	effective cohesion intercept, nil
$\tan \delta_i$	=	friction factor, given in the table below
V_f	=	factored vertical force, kilonewtons
H_f	=	factored 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, $\tan \delta$
Concrete Footing	Cast-in-Place concrete on Granular A levelling pad/Granular B Type II bedding	30	0.58
Gabion Basket	Gabion Basket on Granular A levelling pad/Granular B Type II bedding	32	0.62

6.4.3 Other Design Considerations

The retaining walls must incorporate surface drainage measures to minimize infiltration of surface water into the backfill behind the wall. It is recommended that a drainage swale be incorporated at the top of each wall with the flow directed to a positive outlet. Free draining backfill must be used behind the walls. For gabion and armour stone walls, an approved, non-woven geotextile should be placed at the rear of the walls in order to minimize clogging and or loss of fines through the stone. Gabion walls would need to be designed and constructed in accordance with OPSS.PROV512. Armour stones should be properly chinked and fitted.

6.5 Liquefaction Potential and Seismic Analysis

6.5.1 Seismic Parameters

For the purposes of this project, Site Class D is appropriate based on the results of the investigation. Seismic performance should be calculated in accordance with Section 4.4.3 of the CHBDC (version S6.1-14).

The importance category of the structure is “other” based on the CHBDC. The corresponding Seismic Category for the structure is 1 based on Table 4.10 of the CHBDC. Structures in Seismic Category 1 need not be analysed for seismic loads. However, minimum requirements as outlined in CHBDC Clause 4.4.5.1 must be followed.

6.5.2 Seismic Hazard Assessment

A preliminary screening of the soil stratigraphy was conducted using the procedure outlined in the Federal Highway Administration recommended procedures⁶ and Canadian Foundation Engineering Manual (CFEM). Although compact saturated soils are present, the potential for liquefaction occurring at this site is very low due to historically low seismicity in this area and an overall soil profile with a normalized SPT (N_1)₆₀ generally greater than 30 blows

⁶ 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.



per 0.3 metres, the threshold value above which liquefaction is generally not expected to occur. Therefore, a detailed evaluation of the liquefaction potential of the foundation soils is not considered warranted.

6.6 Lateral Earth Pressures for Design

The lateral pressures acting on the proposed and existing sections of the culvert will depend on the type and method of placement of the backfill materials, the nature of the soil behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls.

The following recommendations are made concerning the design of the walls in accordance with the CHBDC (version S6.1-14). 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.

- Backfill should be placed in accordance with Section 6.2.3 above.
- 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 culvert), at-rest earth pressures should be assumed for geotechnical design. The granular fill should be placed in accordance with OPSD 803.010.
- 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 accordance with OPSD 803.010.
- For walls backfilled using granular materials in accordance with Case (b), the following parameters (unfactored) may be assumed:



	<u>GRANULAR A</u>	<u>GRANULAR B TYPE II</u>	<u>GRANULAR B 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

6.7 Temporary Roadway Protection

It is understood that temporary roadway protection is required in order to maintain traffic on Highway 10 at the culvert location during construction. Temporary support systems could consist of cantilevered soldier piles and lagging or steel sheet piles. Cobbles and wood pieces have been noted in the fill and sand and gravel deposits which may hinder installation of soldier piles and sheet piles. The presence of cobbles and boulders should be anticipated within the glacial tills. It may be preferable to install the soldier piles in pre-augered holes. An NSSP should be included in the Contract Documents to warn the Contractor about the presence of cobbles and wood debris within the fills and native sand and gravel deposits as well as cobbles and boulders within the glacial till.

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/m²; 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/m³

q = surcharge for traffic and other loading

h_w = height of groundwater level above excavation base; water level to be taken as elevation 419 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)	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	0.38	0.55	2.7	27	19.0	9.0
Silty Sand	0.32	0.49	3.1	31	20.0	9.0
Sand and Gravel	0.27	0.43	3.7	35	21.0	11.0
Clayey Silt Till	0.31	0.47	3.3	32	21.0	11.0
Sandy Silt Till	0.27	0.43	3.7	35	22.0	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.

6.8 Construction Considerations

When excavating near the portion of the existing structure to remain in place, care should be taken to ensure that the footings and its founding soils are not disturbed or undermined. Care should also be taken during construction to avoid disturbance of the subgrades prior to placing the bedding material. All existing fill and any topsoil, organics, and soft or loose soils should be stripped from the proposed founding areas prior to placement of the base. 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 base should be inspected by the QVE to ensure that competent founding soil has been reached and Granular A or Granular B, Type II 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, placement and compaction of fill should be carried out under the direction of the QVE.

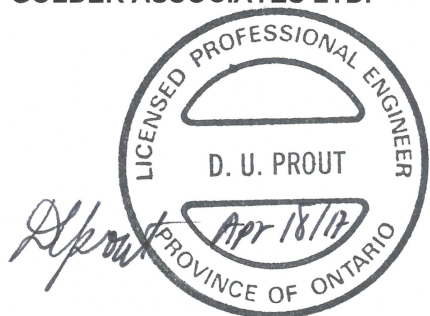
Temporary erosion protection and sedimentation control measures should be implemented in accordance with OPSS 805. In addition, sediment control such as silt fences and erosion control blankets may be required during construction together with diversion of any flows to mitigate migration of fine soil particles.



7.0 MISCELLANEOUS

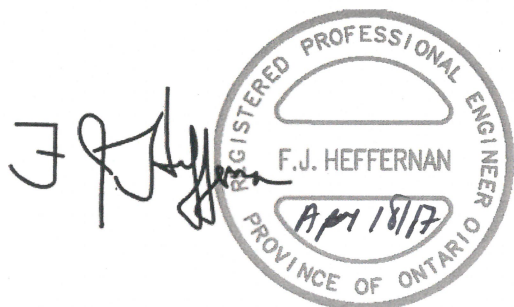
This section of the report was prepared by Mr. Daniel W. Hyland, E.I.T. under the direction of the Project Engineer Ms. Dirka U. Prout, P.Eng. The report was reviewed by Mr. W.M. Kellestine, P.Eng., and Mr. Fintan J. Heffernan, P.Eng., the Designated MTO Contact and Quality Control Auditor for this assignment, who conducted an independent quality review of the report.

GOLDER ASSOCIATES LTD.



Dirka U. Prout, P.Eng.
Senior Geotechnical Engineer

W.M. Kellestine, P.Eng.
Senior Consultant



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

DH/DUP/WMK/FJH/CR

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

n:\active\2015\3 proj\1526476 dillon_3014-e-0028 orangeville_hwy 10\ph 4000-fdns part b\2-correspondence\5-rpts\final\1526476-4000-r01 apr 17 17 (final) hwy 10 culverts 4-174-c.docx



LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

Notes: 1
2

$$\tau = c' + \sigma' \tan \phi'$$

$$\text{shear strength} = (\text{compressive strength})/2$$



LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d :

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

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

(b) Cohesive Soils Consistency

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

IV. SOIL TESTS

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

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

V. MINOR SOIL CONSTITUENTS

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

RECORD OF BOREHOLE No BH-101

1 OF 2

METRIC

PROJECT 1526476
W.P. 3078-12-00 LOCATION N 4868410.9 , E 256155.3 ORIGINATED BY DH
DIST HWY 10 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY LMK
DATUM GEODETIC DATE October 6, 2016 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								○ UNCONFINED + FIELD VANE									
								● QUICK TRIAXIAL × LAB VANE									
							WATER CONTENT (%)										
							20 40 60 80 100					10 20 30					
422.12	GROUND SURFACE						422										
0.00	FILL, sand and crushed gravel Brown																
421.51																	
0.61	FILL, silty sand, trace to some gravel, trace clay Loose to compact Brown		1	SS	6		421										
			2	SS	8												
			3	SS	15		420										
419.07																	
3.05	FILL, silty sand and gravel, with cobbles Compact Brown		4	SS	17		419										
418.46																	
3.66	SILTY SAND AND GRAVEL, with cobbles and wood pieces Very dense Grey		5	SS	50/ 75mm		418								9 59 27 5		
			6	SS	50/ 50mm												
416.69							417										
5.43	SILTY SAND Compact Grey		7	SS	13												
416.18							416										
5.94	SAND, fine to medium, trace to some silt, with wood pieces Very dense Grey		8	SS	62										0 91 8 1		
415.66							415										
6.46	SAND AND GRAVEL, some silt Very dense Grey																
415.02							414										
7.10	SILT AND SAND, trace clay Compact Grey		9	SS	27										0 36 59 5		
413.49																	
8.63	SILTY SAND, some gravel, trace clay, with cobbles Dense Grey and red		10	SS	44		413								18 53 25 4		
411.97																	
10.15	SANDY SILT TILL, some clay, trace to some gravel, with cobbles Compact to very dense Grey		11	SS	28		412								7 34 47 12		
							411										
			12	SS	55		410										
							409										
408.31			13	SS	50/ 75mm												
13.81	END OF BOREHOLE																
	Groundwater encountered at about elev. 417.9m during drilling on October 6, 2016.																

Continued Next Page

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

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

METRIC

PROJECT 1526476

W.P. 3078-12-00

LOCATION N 4868397.1 , E 256138.2

ORIGINATED BY DH

DIST _____ HWY 10BOREHOLE TYPE POWER AUGER, HOLLOW STEM

COMPILED BY LMK

DATUM GEODETIC

DATE October 27, 2016

CHECKED BY

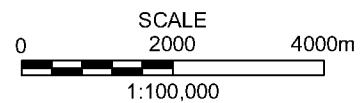
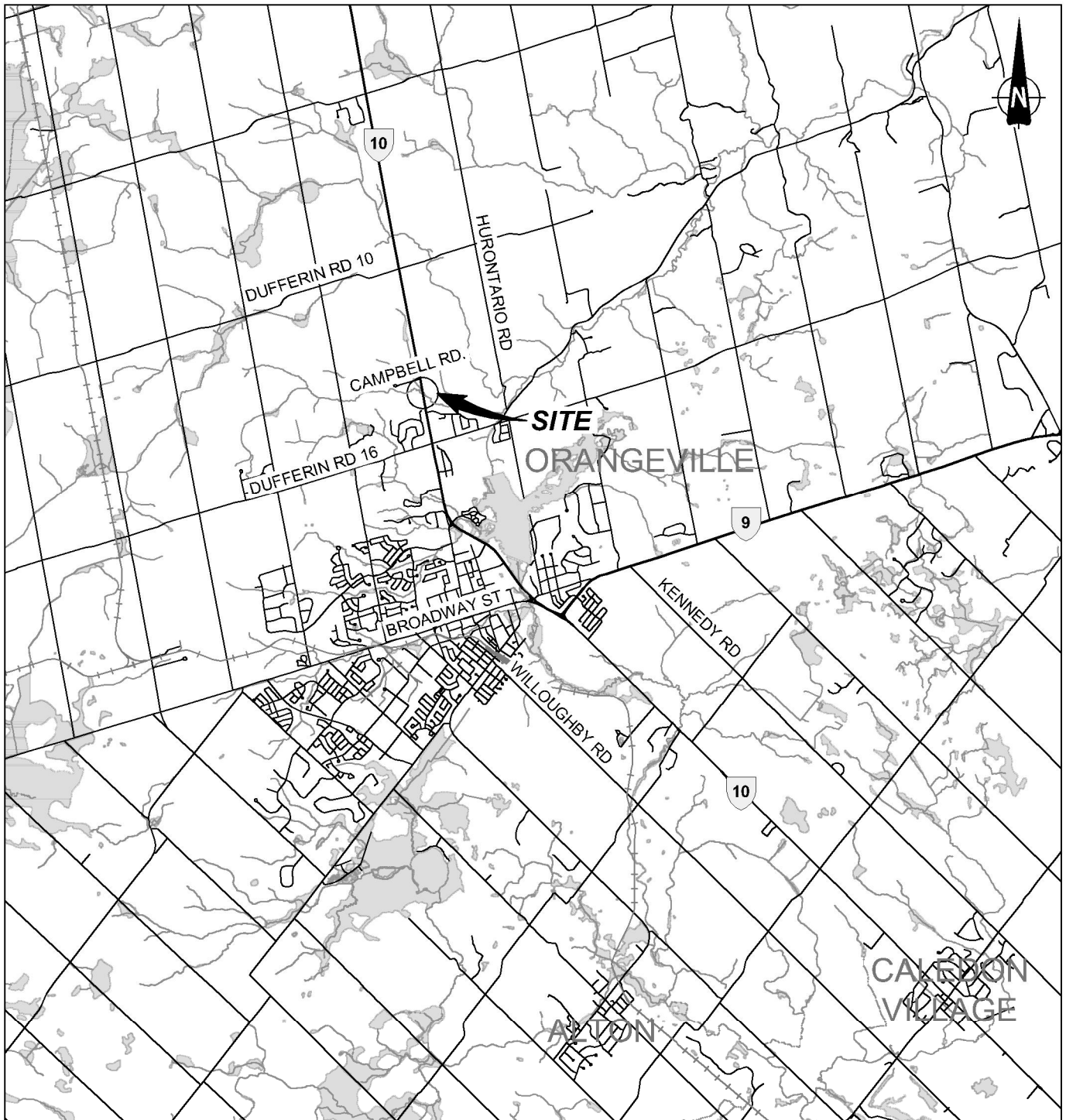
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	W _P W W _L	WATER CONTENT (%)				GR	SA	SI	CL	
								SHEAR STRENGTH kPa										
							○ UNCONFINED + FIELD VANE											
							● QUICK TRIAXIAL × LAB VANE											
422.01	GROUND SURFACE						422											
0.00	FILL, sand and crushed gravel																	
421.63	Brown																	
0.38	FILL, sand and gravel, trace silt,																	
421.25	with cobbles																	
0.76	Brown		1	SS	18		421											
	FILL, silty sand, some gravel, trace																	
	to some clay		2	SS	8													
	Loose to compact						420											
	Brown																	
			3	SS	5													
419.11							419											
2.90	TOPSOIL, silty, with wood pieces		4	SS	5													
	Loose																	
	Black																	
418.35																		
3.66	SILTY SAND, trace to some gravel,		5	SS	19		418								8	64	25	3
	trace clay, with organics																	
	Compact																	
	Grey																	
417.59			6	SS	22		417											
4.42	SAND AND GRAVEL, some silt																	
	Compact																	
	Brown																	
416.83			7	SS	13		416											
5.18	SAND, fine to medium, trace to		8	SS	10													
	some silt																	
	Compact																	
	Brown																	
							415											
414.91																		
7.10	CLAYEY SILT TILL, some sand,		9	SS	23		414											
	some gravel, with cobbles																	
	Very stiff to hard																	
	Brown to grey at about																	
	elev. 410.4m																	
							413											
			10	SS	100/ 280mm													
							412											
			11	SS	112		411											
							410											
409.60			12	SS	47		409											
12.41	SILT, some sand, some clay																	
409.21	Dense																	
12.80	Grey																	
	CLAYEY SILT TILL, some sand,																	
	some gravel																	
	Hard																	
	Grey																	
			13	SS	53		408											
407.84																		
14.17	END OF BOREHOLE																	
	Groundwater encountered at about																	
	elev. 418.4m during drilling on																	
	October 27, 2016.																	

DN_MTO_06 1526476.GPJ LDN_MTO.GDT 29/11/16

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

PROJECT <u>1526476</u>		RECORD OF BOREHOLE No BH-103		1 OF 1		METRIC	
W.P. <u>3078-12-00</u>		LOCATION <u>N 4868399.9 , E 256169.3</u>		ORIGINATED BY <u>DH</u>			
DIST <u> </u> HWY <u>10</u>		BOREHOLE TYPE <u>MANUAL AUGER</u>		COMPILED BY <u>LMK</u>			
DATUM <u>GEODETIC</u>		DATE <u>October 27, 2016</u>		CHECKED BY <u></u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	W _p W W _L	20 40 60 80 100	10 20 30			
419.77	GROUND SURFACE													
0.00	TOPSOIL, sandy													
419.45	Brown													
0.32	FILL, silty sand, trace gravel, trace													
419.12	topsoil													
0.65	Brown													
0.95	FILL, sandy silt, with topsoil and													
418.47	organics													
1.30	Brown													
418.02	SILTY SAND, trace gravel, with		1	AS										
1.75	topsoil and organics													
	Dark grey													
	SAND, some gravel, some silt,													
	trace clay, with organics													
	Dark grey													
	END OF BOREHOLE													
	Groundwater encountered at about elev. 418.5m during drilling on October 27, 2016.													



REFERENCE

PLAN BASED ON CANMAP STREETFILES V.2008.5.

NOTE

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

PROJECT

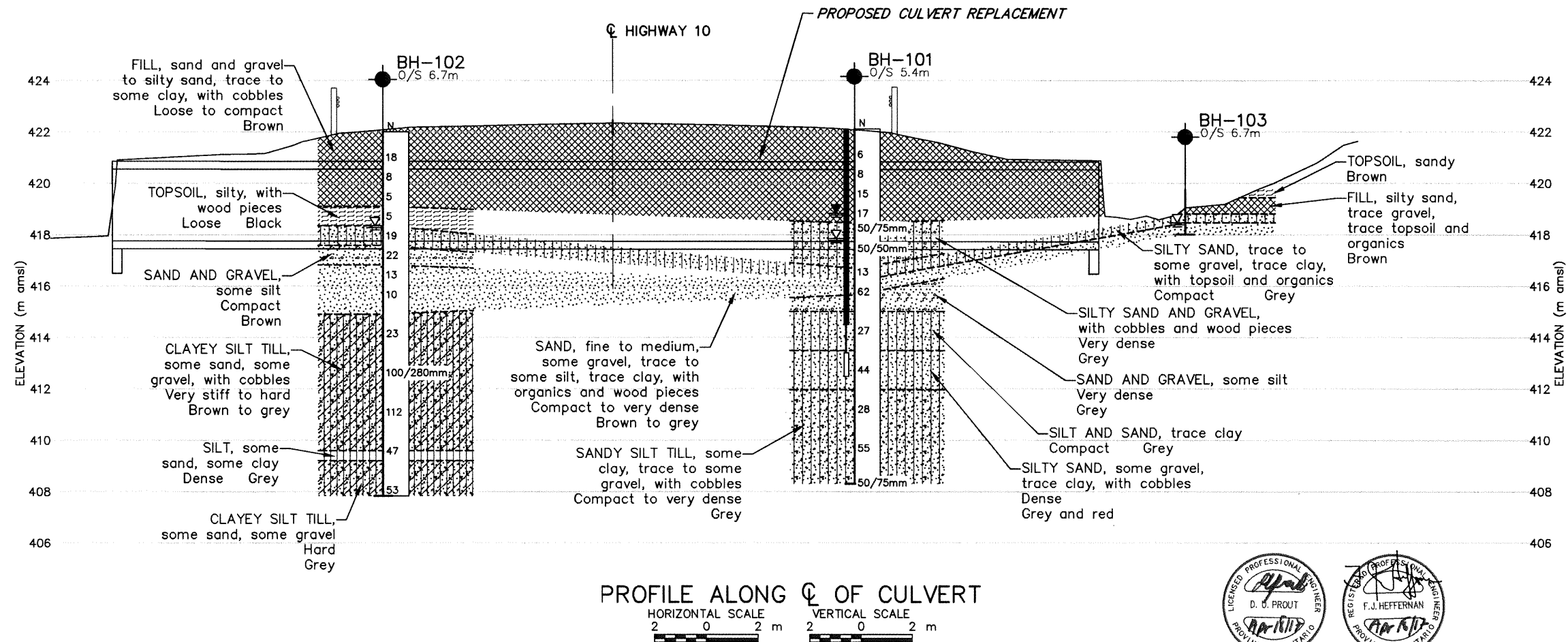
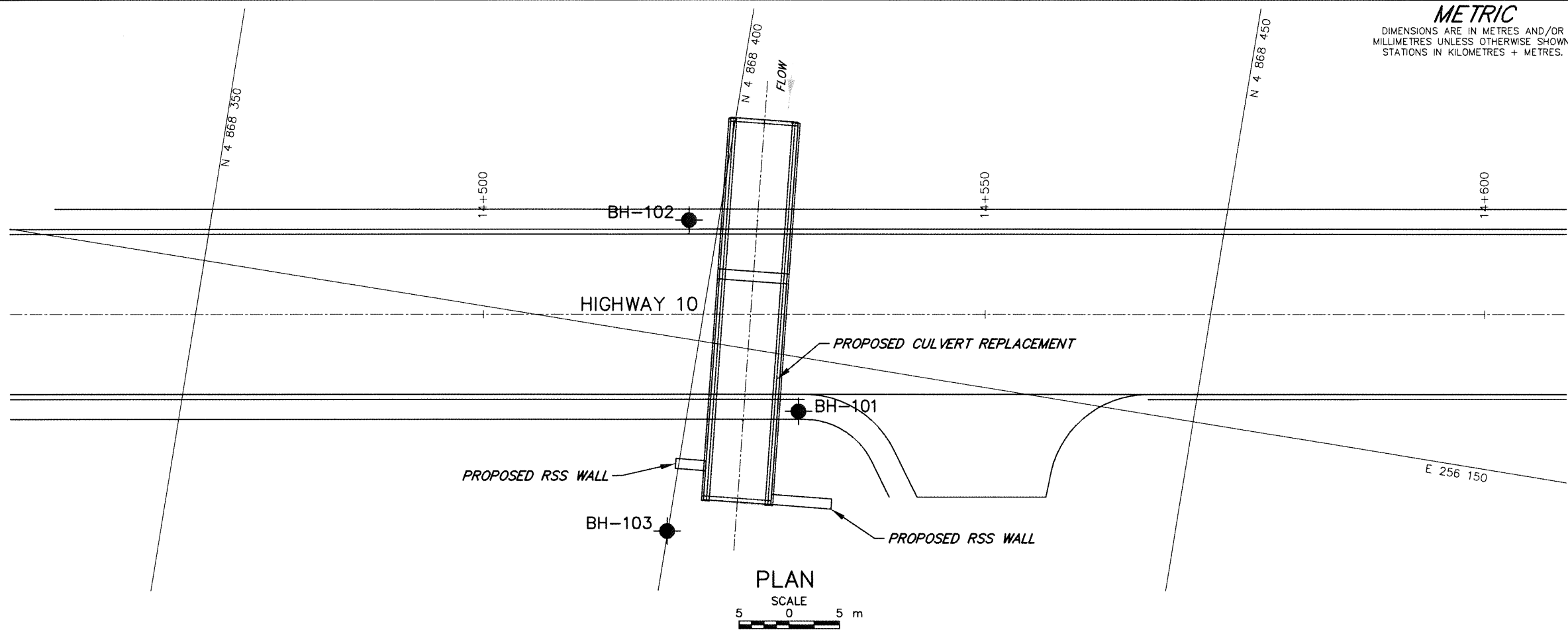
CULVERT REPLACEMENT, SITE 4-174/C
HIGHWAY 10
GWP 3078-12-00

TITLE

KEY PLAN



PROJECT No. 1526476			FILE No. 1526476-4000-F01001		
CADD	LMK	Nov. 24/16	SCALE	AS SHOWN	REV. 0
CHECK			FIGURE 1		



CONT No.
WP No. 3078-12-00

CULVERT REPLACEMENT

HIGHWAY 10 SITE No. 4-174/C
BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates Ltd.
LONDON, ONTARIO, CANADA

KEY PLAN
SCALE IN KILOMETRES

LEGEND

- Borehole
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL measured on October 27, 2016
- WL encountered during drilling

No.	ELEVATION	CO-ORDINATES (MTM ZONE 10)	
		NORTHING	EASTING
BH-101	422.12	4 868 410.9	256 155.3
BH-102	422.01	4 868 397.1	256 138.2
BH-103	419.77	4 868 399.9	256 169.3

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 Dillon.

NO.	DATE	BY	REVISION

Geocres No. 40P16-25

HWY. 10	PROJECT NO. 1526476	DIST.
SUBM'D. DUP	CHKD. DH	DATE: Mar. 17/17
DRAWN: LMK	CHKD. DUP	APPD. FJH
		DWG. 1

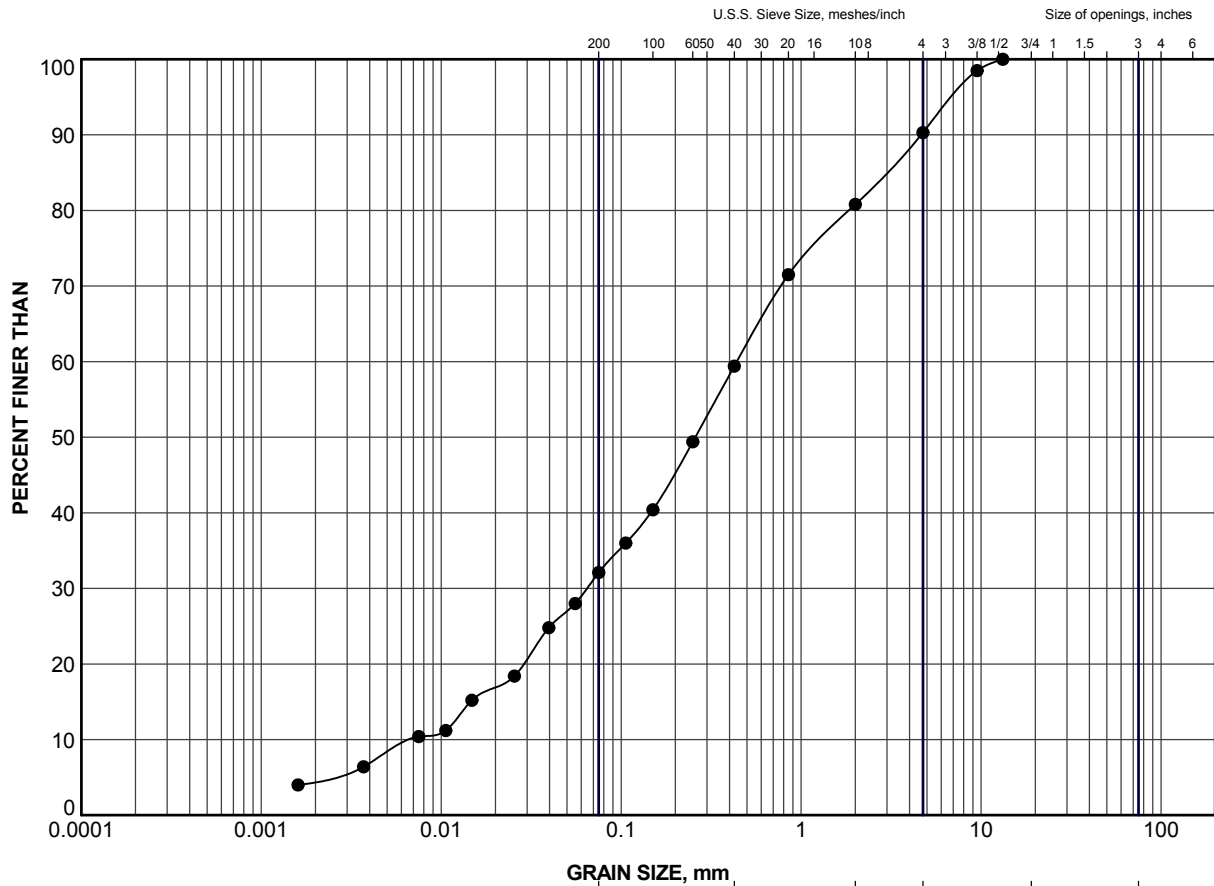
PROFESSIONAL ENGINEER
D. O. PROUT
PROVINCE OF ONTARIO

PROFESSIONAL ENGINEER
F. J. HEFFERNAN
PROVINCE OF ONTARIO



APPENDIX A


Laboratory Test Data

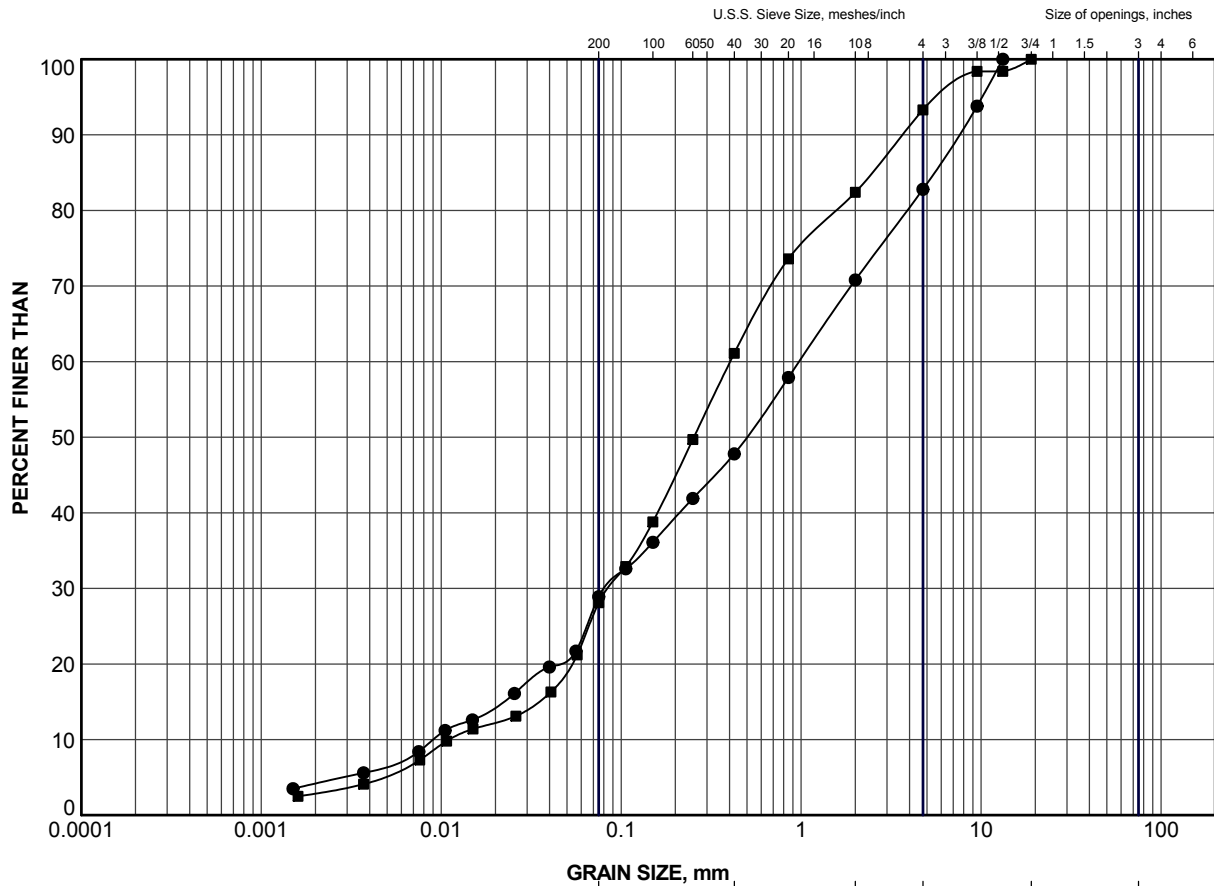


GRAVEL SIZE, mm							Cobble Size
CLAY AND SILT	fine	medium	coarse	fine	coarse		
	SAND SIZE			GRAVEL SIZE			

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BH-101	2	420.4


PROJECT		CULVERT REPLACEMENT, SITE 4-174/C HIGHWAY 10 GWP 3078-12-00			
TITLE		GRAIN SIZE DISTRIBUTION FILL			
	PROJECT No.	1526476		FILE No.	1526476-4000-F010A1
	DRAWN	LMK	Nov 11/16	SCALE	N/A
	CHECK			REV.	
	FIGURE A-1				

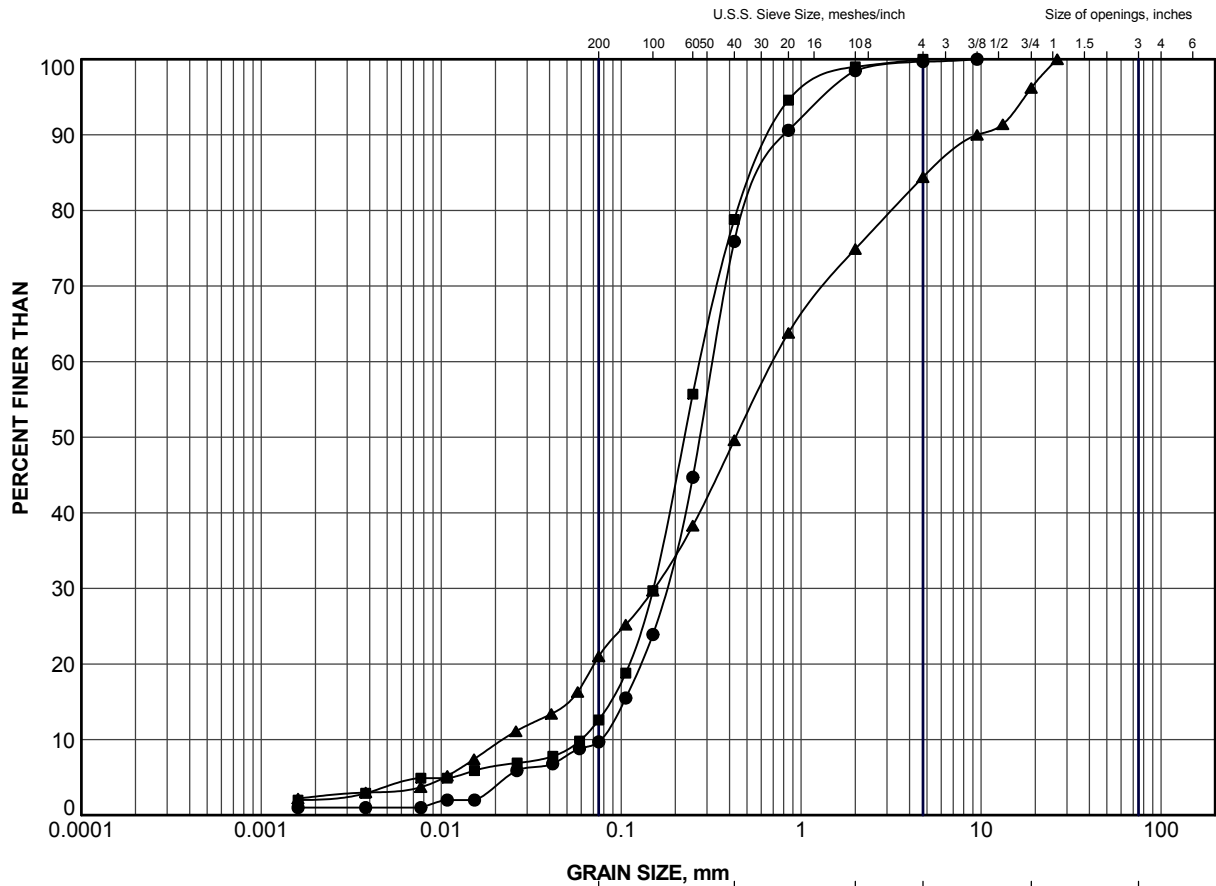


CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BH-101	10	412.8
■	BH-102	5	418.0


PROJECT		CULVERT REPLACEMENT, SITE 4-174/C HIGHWAY 10 GWP 3078-12-00			
TITLE		GRAIN SIZE DISTRIBUTION SILTY SAND			
		PROJECT No.		1526476	
		FILE No.		1526476-4000-F010A2	
		SCALE		N/A	
		REV.			
DRAWN		LMK		Nov 11/16	
CHECK					
		FIGURE A-2			

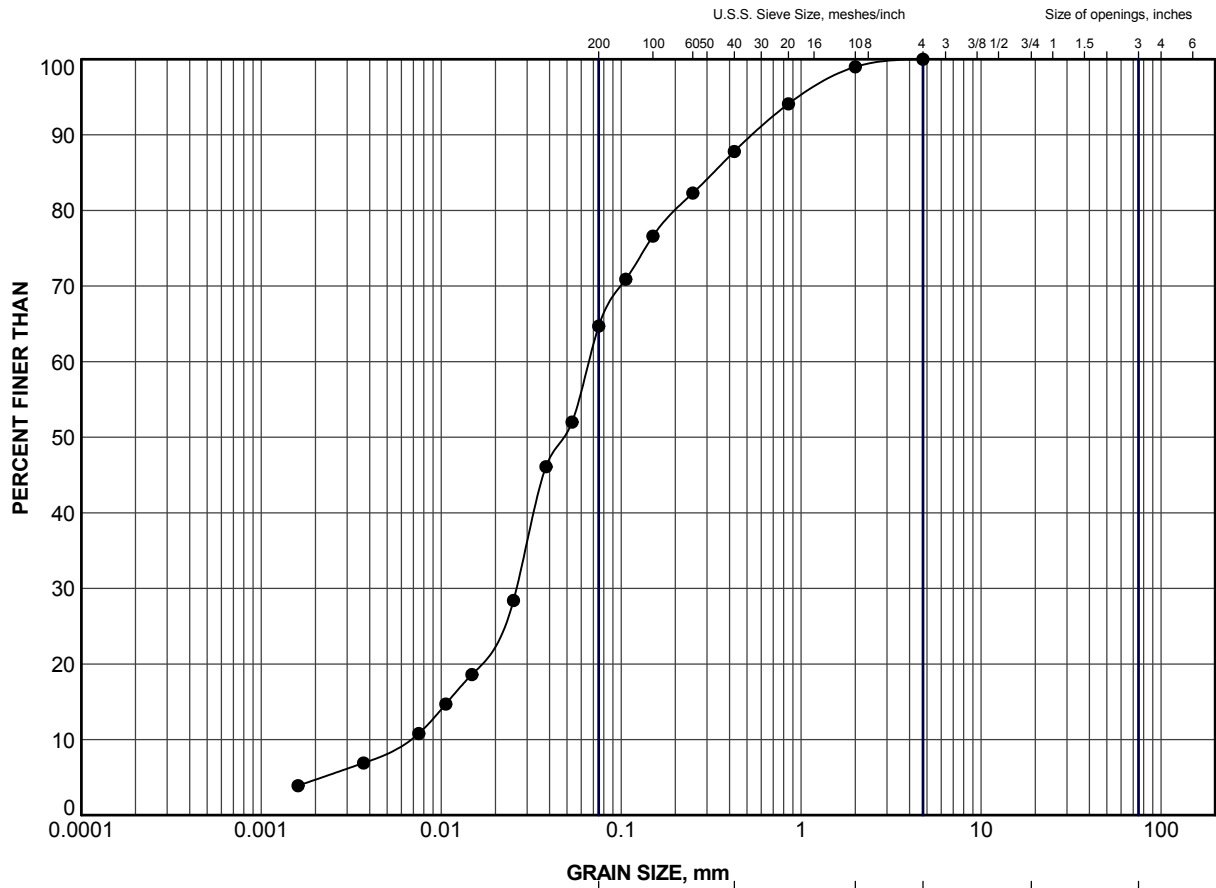


CLAY AND SILT		SAND SIZE, mm			GRAVEL SIZE, mm		Cobble Size
		fine	medium	coarse	fine	coarse	
		SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BH-101	8	415.8
■	BH-102	7	416.5
▲	BH-103	1	418.2


PROJECT		CULVERT REPLACEMENT, SITE 4-174/C HIGHWAY 10 GWP 3078-12-00			
TITLE		GRAIN SIZE DISTRIBUTION SAND			
		PROJECT No.		1526476	
		FILE No.		1526476-4000-F010A3	
		SCALE		N/A	
		REV.			
DRAWN		LMK	Nov 24/16		FIGURE A-3
CHECK					

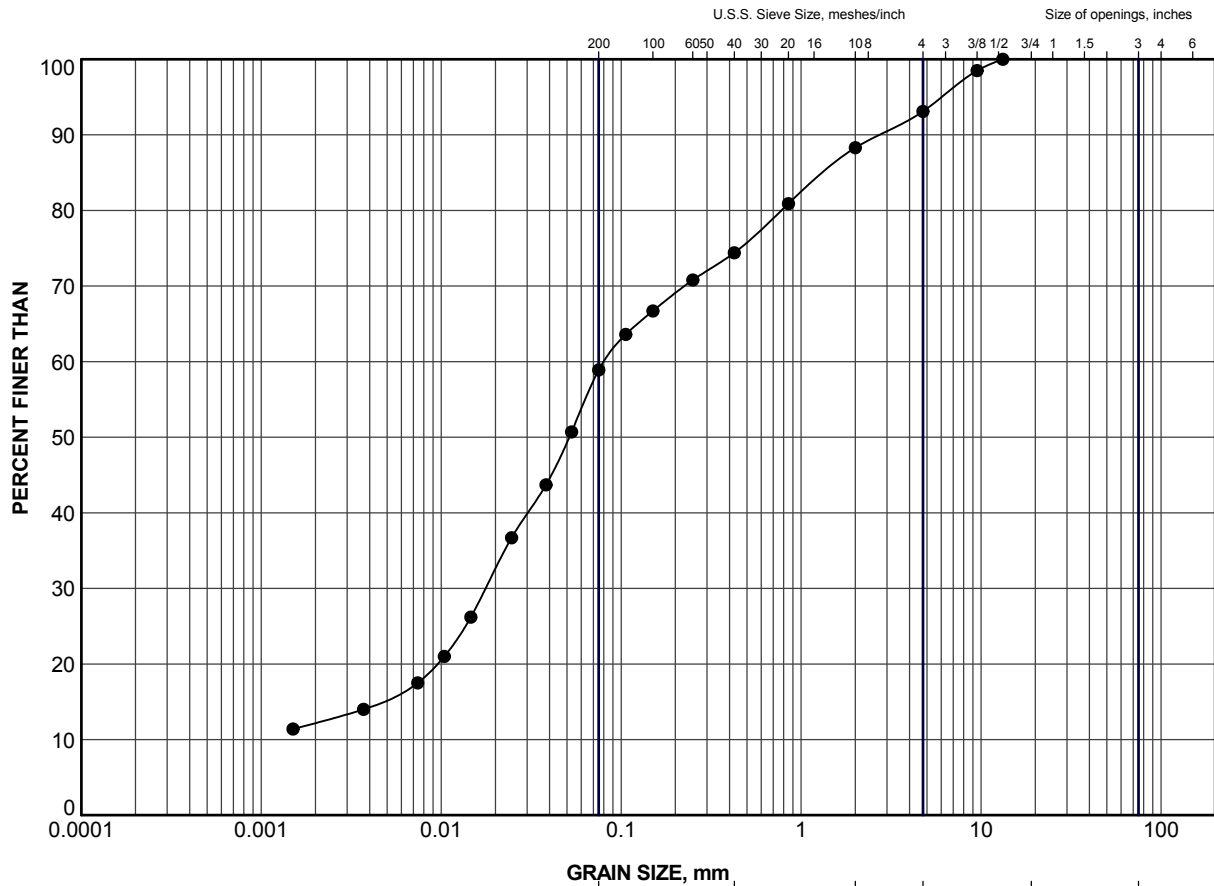


CLAY AND SILT	GRAIN SIZE, mm		medium	coarse	GRAIN SIZE, mm		Cobble Size
	fine	coarse			fine	coarse	
	SAND SIZE				GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BH-101	9	414.3


PROJECT		CULVERT REPLACEMENT, SITE 4-174/C HIGHWAY 10 GWP 3078-12-00			
TITLE		GRAIN SIZE DISTRIBUTION SILT AND SAND			
		PROJECT No.		1526476	
		FILE No.		1526476-4000-F010A4	
		SCALE		N/A	
		REV.			
DRAWN		LMK	Nov 24/16		FIGURE A-4
CHECK					

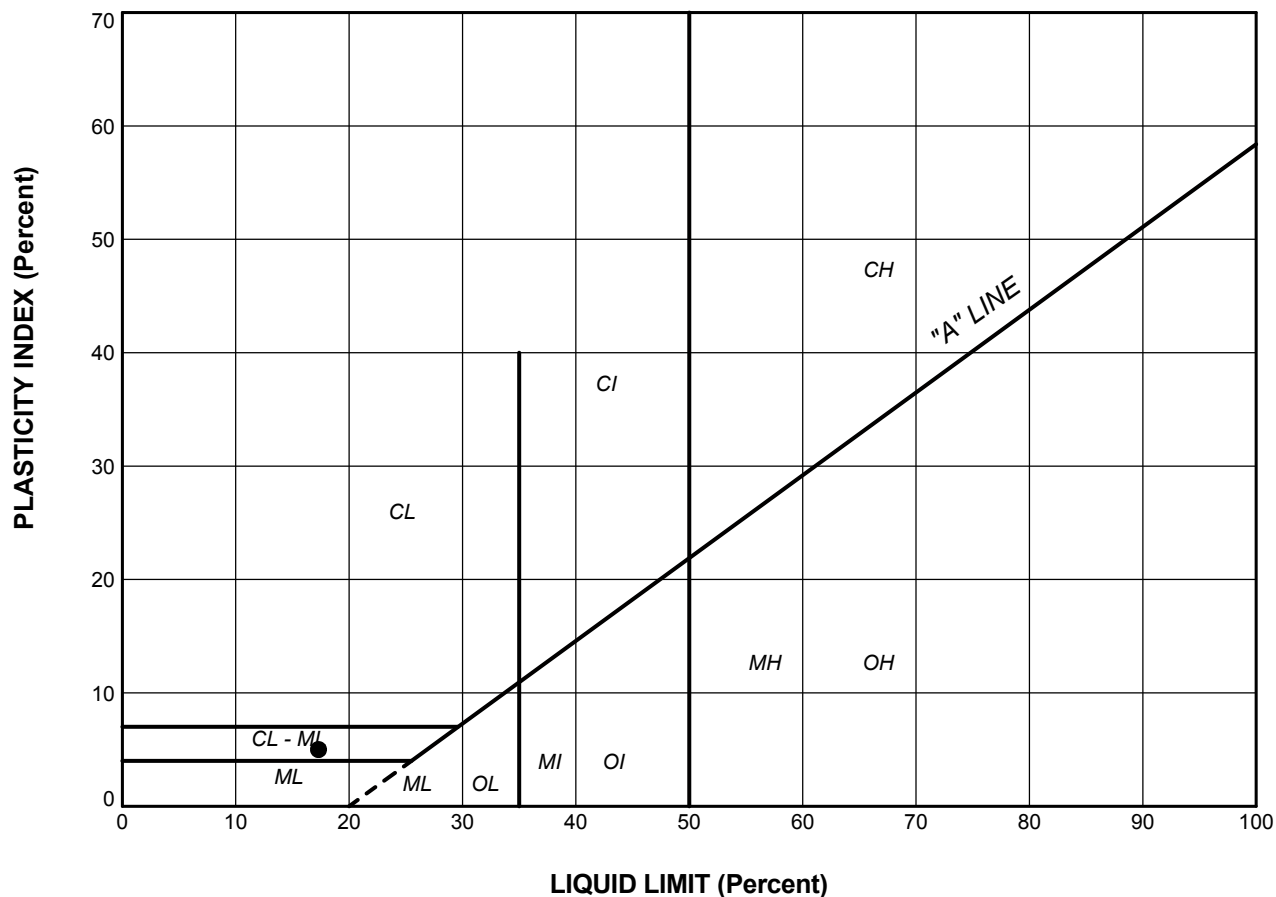


CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BH-101	11	411.2

PROJECT		CULVERT REPLACEMENT, SITE 4-174/C HIGHWAY 10 GWP 3078-12-00			
TITLE		GRAIN SIZE DISTRIBUTION SANDY SILT TILL			
		PROJECT No.		1526476	
		FILE No.		1526476-4000-F010A5	
		SCALE		N/A	
		REV.			
DRAWN		LMK	Nov 10/16		FIGURE A-5
CHECK					



LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	BH-101	11	17.3	12.3	5.0

PROJECT					CULVERT REPLACEMENT, SITE 4-174/C HIGHWAY 10 GWP 3078-12-00				
TITLE									
PLASTICITY CHART									
PROJECT No.		1526476		FILE No.		1526476-4000-F010A6			
DRAWN	LMK	Nov 10/16		SCALE	N/A		REV.		
CHECK				FIGURE A-6					





APPENDIX B

Site Photographs



APPENDIX B PHOTOGRAPHS



Photograph 1: East elevation (outlet) of Culvert Site 4-174/C.



Photograph 2: West elevation (inlet) of Culvert Site 4-174/C. Note erosion at north elevation.



APPENDIX B PHOTOGRAPHS



Photograph 3: Asphalt riding surface of Highway 10 above Culvert Site 4-174/C, looking west.



Photograph 4: East side (outlet) of Culvert Site 4-174/C. Photo taken in April 2016. Note erosion along north elevation.



APPENDIX B PHOTOGRAPHS



Photograph 5: East side (outlet) of Culvert Site 4-174/C. Photo taken in October 2016. Note sand bags and fill placed along north elevation.

n:\active\2015\3 proj\1526476 dillon_3014-e-0028 orangeville_hwy 10\ph 4000-fdns part b\2-correspondence\5-rpts\final\1526476-4000-r01 apr 13 17 (final) app b - photos.docx

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

For more information, visit golder.com

Africa	+ 27 11 254 4800
Asia	+ 86 21 6258 5522
Australasia	+ 61 3 8862 3500
Europe	+ 44 1628 851851
North America	+ 1 800 275 3281
South America	+ 56 2 2616 2000

solutions@golder.com
www.golder.com

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

