September 2013

FOUNDATION INVESTIGATION AND DESIGN REPORT

Culvert Rehabilitation, Willow Creek Under Highway 6
Site No. 2-457/C, Station 17+378
Contract 1 Structure Replacements and Rehabilitation
GWP 3101-10-00
Ministry of Transportation, West Region

Submitted to:

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REPORT

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Table of Contents

APPENDICES

PART A

FOUNDATION INVESTIGATION REPORT

CULVERT REHABILITATION, WILLOW CREEK UNDER HIGHWAY 6
SITE NO. 2-457/C, STATION 17+378
CONTRACT 1 STRUCTURE REPLACEMENTS AND REHABILITATIONS
GWP 3101-10-00
MINISTRY OF TRANSPORTATION - WEST REGION

1.0 INTRODUCTION

Golder Associates Ltd. (Golder Associates) has been retained by Stantec Consulting Ltd. (Stantec) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations as part of the preliminary design and detail design work for GWP 3101-10-00. 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 rehabilitation of the culvert at Willow Creek (Site 2-457/C) at Station 17+378 in St. Edmunds Township in Bruce County.

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

2.0 SITE DESCRIPTION

The subject culvert is situated at Station 17+378 on Highway 6, approximately 0.2 kilometres southeast of Emmett Lake Road in the Township of St. Edmunds in Bruce County, Ontario. The location of the culvert is shown on the Key Plan, Figure 1.

This section of Highway 6 is currently a two lane undivided highway with gravel shoulders. It is generally oriented northwest-southeast in the vicinity of the subject site. The flow direction in the culvert is from north to south beneath Highway 6. The construction date of the culvert is unknown. The existing culvert is a concrete rigid frame box (RFB) structure with the following characteristics:

Dimensions (m)	Obvert Elevation (m)		Construction
	Lt[1]	Rt[1]	
7.08 x 2.25 x 20.02	199.18	199.26	Concrete RFB

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 drainage channel immediately upstream and downstream of the culvert are grass covered with rock retaining walls on all four corners and the channel flows through fields adjacent to Highway 6. Site photographs are provided in Appendix B.

The culvert is situated in a rural area with low relief. Ground surface elevations in the vicinity of the culvert range from about 196 to 200 metres.

2.1 Site Geology

The project area is located within the Bruce Peninsula physiographic region. This region is characterized by typically thin overburden deposits and bedrock outcrops.[1] The overburden in the area of the site generally consists of poorly drained lacustrine silts and clays with some areas of granular bar and beach deposits.[2]

The geological mapping indicates that the underlying bedrock consists of light grey-tan to brown, tabular bedded dolostone of the Guelph Formation of Middle to Lower Silurian age.[3] A bedrock outcrop is present about 80 metres southeast of the culvert.

[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., Cowan, W.R. and Henry, A.P. 1991: Quaternary Geology of Ontario, southern sheet; Ontario Geological Survey, Map 2556, scale 1:1,000,000.

[3] Freeman, E.B., 1979: Geological Highway Map, Southern Ontario; Ontario Geological Survey Map 2441, Scale 1:800,000.

3.0 INVESTIGATION PROCEDURES

The field work for the investigation was carried out on June 6, 2013, during which time 4 boreholes were drilled at the locations shown on the Borehole Location Plan, Drawing 1.

The boreholes were drilled using track-mounted 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 procedures (ASTM D1586).

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 sand and gravel fill as discussed in the text of this report.

Groundwater conditions in the boreholes were observed throughout the drilling operations and a standpipe was installed in borehole 103 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, monitored the drilling, sampling and in situ testing operations and logged the boreholes. The samples were identified in the field, placed in labelled containers and transported to our London laboratory for further examination and testing. Index and classification tests, consisting of water content determinations, Atterberg limits, and grain size distribution analyses, were carried out on selected samples. The results of the testing are shown on the Record of Borehole sheets and in Appendix A.

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

Borehole	Location (m)		Ground Surface Elevation	Borehole Depth
	Northing	Easting	(m)	(m)
103	5 004 222	381 857	199.6	6.5
104	5 004 229	381 850	199.8	5.9
105	5 004 239	381 857	199.9	6.0
106	5 004 234	381 866	199.9	6.7

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 granular fill or topsoil overlying variable embankment fill materials then in sequence, peat, organic silt, silty sand and gravel, and sandy silt to silty sand.

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

4.2 Soil Conditions

4.2.1 Topsoil

A layer of topsoil was encountered at the ground surface in borehole 103. The topsoil was 90 millimetres thick.

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

4.2.2 Fill

Fill materials were encountered beneath the topsoil in borehole 103 at elevation 199.5 metres and at the ground surface in boreholes 104 to 106. The fill consisted of sand and gravel and sand, trace to some silt. The fill materials were 2.1 to 3.2 metres thick with standard penetration test N[4] values of 1 to 10 blows per 0.3 metres. Samples of the fill had water contents of about 17 per cent. Cobbles and boulders were encountered in the sand

[4] The SPT 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 having first penetrated 150 millimetres.

and gravel fill in borehole 103. As such, the presence of cobbles and boulders should be anticipated within the fill materials.

4.2.3 Peat

Layers of very soft peat were encountered beneath the fill in boreholes 103, 104, and 105 between elevations 197.7 and 196.3 metres. The peat layers were 0.4 to 1.2 metres thick with measured N values of 1 blow per 0.3 metres within the peat to 6 blows per 0.3 metres where partially in fill and peat and water contents of about 84 to 445 per cent, with an average water content of about 300 per cent.

4.2.4 Organic Silt

Very soft organic silt layers were encountered beneath the peat in borehole 104 at elevation 197.2 metres and beneath the fill in borehole 106 at elevation 197.4 metres. The organic silt layers were 0.4 to 0.6 metres thick with a measured N value of 1 blow per 0.3 metres and a water content of about 115 per cent.

4.2.5 Silty Sand and Gravel

A layer of loose to compact silty sand and gravel was encountered beneath the organic silt in borehole 106 at elevation 197.0 metres. The silty sand and gravel layer was 1.5 metres thick with measured N values of 4 and 16 blows per 0.3 metres and a water content of about 11 per cent.

4.2.6 Sandy Silt, Some Clay

Layers of very loose to compact sandy silt were encountered in boreholes 103 and 104 at elevations 196.0 and 196.7 metres, respectively. Boreholes 103 and 104 were terminated in the sandy silt at auger refusal after exploring it for 2.8 metres.

The sandy silt had measured N values of the weight of hammer to over 100 blows per 0.3 metres and water contents of the samples ranged from 14 to 17 per cent. Grain size distribution curves for samples of the sandy silt recovered from the standard penetration testing are provided on Figure A-1. The sandy silt exhibited some plasticity as indicated by average plastic limits of 12 per cent, liquid limits of 18 per cent, and plasticity indices of 6 per cent based on Atterberg limits determinations carried out samples of the sandy silt. The Atterberg limits data are presented on Figure A-3.

4.2.7 Silty Sand

Layers of very loose to loose silty sand were encountered at elevation 196.6 metres beneath the peat in borehole 105 and at elevation 195.5 metres beneath the silty sand and gravel in borehole 106. Boreholes 105 and 106 were terminated in silty sand after exploring the layer some 2.3 to 2.7 metres. The silty sand had measured N values of 0 to 6 blows per 0.3 metres and water contents of about 10 to 19 per cent with an average water content of about 14 per cent.

Grain size distribution curves for samples of the silty sand are provided on Figure A-2. Atterberg limits determinations carried out on samples of the silty sand indicated average plastic limits of 12 per cent, liquid limits of 16 per cent and plasticity indices of 4 per cent. The Atterberg limits data are presented on Figure A-3.

4.3 Groundwater Conditions

Groundwater conditions were observed during and on completion of drilling and sampling and a groundwater observation standpipe was installed in borehole 103. Installation details are provided on the corresponding Record of Borehole sheet following the text of this report. Groundwater was encountered in boreholes 103 to 106 at depths of 1.9 to 2.1 metres or between elevations 197.7 and 197.8 metres. A summary of the encountered and measured groundwater levels is provided in the table below.

Borehole	Ground Surface Elevation (m)	Encountered Groundwater Elevation (m)	Measured Groundwater Level Elevation (m)	
			June 6, 2013	September 12, 2013
103	199.6	197.7	199.1	198.2
104	199.8	197.8	-	-
105	199.9	197.8	-	-
106	199.9	197.8	-	-

The above-noted encountered water levels are not considered to be representative of the long-term, stabilized groundwater conditions. The corresponding water level in the watercourse was measured at elevation 198.1 metres on June 6, 2013. On June 6, 2013, the water level in the standpipe installed in borehole 103 was about 0.5 metres below ground surface or at about elevation 199.1 metres. On September 12, 2013, the water level in the standpipe was about 1.4 metres below ground surface or about elevation 198.2 metres.

Based on the observed groundwater levels, the surrounding topography, and water levels in the drain, the groundwater level is inferred to typically be at about elevation 197.8 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 Aardvark Drilling Inc., an Ontario Ministry of Environment licensed well contractor. The field operations were supervised by Mr. Michael Arthur under the direction of Mr. David J. Mitchell. 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 Ms. Nicole Gould, P.Eng. under the direction of the Project Engineer Ms. Dirka U. Prout, P.Eng. and the Team Leader, Dr. Storer J. Boone, P.Eng. This report was reviewed by Mr. Azmi Hammoud, P.Eng., an Associate and a Geotechnical Engineer 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 REHABILITATION, WILLOW CREEK UNDER HIGHWAY 6
SITE NO. 2-457/C, STATION 17+378
CONTRACT 1 STRUCTURE REPLACEMENTS AND REHABILITATIONS
GWP 3101-10-00
MINISTRY OF TRANSPORTATION - WEST REGION

6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides recommendations on the foundation aspects of the design of wingwall replacements to be carried out as part of the rehabilitation of the Willow Creek culvert (Site 2-457/C) at Station 17+378 on Highway 6 in the Township of St. Edmunds in Bruce County, Ontario.

The recommendations are based on 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 in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, and scheduling.

The existing culvert is a 20.0 metre long concrete rigid frame box (RFB) structure with a 6.1 metre span and a 1.6 metres high opening with invert elevations of 197.7 and 197.6 metres at the inlet and outlet, respectively. The existing stone wingwalls are between about 1.2 and 4.4 metres long and are between about 0.8 and 1.2 metres in height. Based on information and the preliminary general arrangement drawing provided by Stantec Consulting Ltd. (Stantec), it is understood that the existing wingwalls at the site will be replaced as part of the rehabilitation of the Willow Creek culvert. The replacement wingwalls are to be 3.0 metres long and between about 1.0 and 1.4 metres in height. It has been indicated by Stantec that consideration is being given to constructing the replacement wingwalls as gabion walls. Other options that may be considered are concrete gravity or cantilever walls, armour stone walls, or reinforced soil system (RSS) walls. The various wingwall options are discussed below.

6.2 Replacement Wingwalls

6.2.1 Wingwall Options

Armour Stone Walls

Construction of armour stone block walls at the site is geotechnically feasible. Armour stone 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 founding row of stones has adequate embedment to provide a stable structure. Armour stone walls can be constructed relatively quickly as no foundation construction is required.

Gabion Walls

Similar to armour stone 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 a concrete toe wall or RSS walls, footings for gravity and cantilever walls must be constructed with a frost cover of 1.4 metres. This may result in a longer foundation construction time compared to a gabion or armour stone wall, particularly if cast-in-place (CIP) walls are constructed. The concrete gravity wall could consist of pre-cast elements or CIP.

RSS Walls

The height of the wingwalls will be relatively low. Therefore, a reinforced soil system wall utilizing an interlocking block system and geogrid reinforcement is a geotechnically feasible alternative. RSS walls are proprietary systems which are to be 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.

6.2.2 Wingwall Foundations

Armour stone walls may be founded 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 strata. A non-woven geotextile should be placed between the stone blocks and the backfill.

Gabion walls may also 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 strata for gabion walls. Non-woven geotextile is to be placed between the gabions and the backfill placed in accordance with OPSS 512, OPSS 1860, and the manufacturer’s specifications.

Retained Soil System walls may be designed such that the facing blocks are constructed 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 1.4 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 wingwalls foundations must be protected against scour as noted in the CHBDC Section 1.9.5. Peat and/or organic silt were encountered in all the boreholes to elevations 196 to 197 metres. These organic materials are very compressible and should be subexcavated from below all foundation areas.

It is recommended that the replacement wingwalls be founded on engineered fill placed on the native sandy silt, silty sand, or silty sand and gravel encountered between elevations 197.0 and 196.0 metres which is below the water level in the creek which at the time of the investigation in early June 2013 was at elevation 198.1 metres.

The engineered fill should consist of Ontario Provincial Standard Specifications (OPSS) Granular B Type II and should be compacted to at least 95 per cent standard Proctor maximum dry density.

Cast-in-place reinforced concrete gravity and cantilever walls founded on concrete strip footings must be provided with a frost cover of 1.4 metres below the adjacent ground or thermal equivalent. This deeper founding level would make for more extensive dewatering, possibly sheet piled foundation excavation. As such, these concrete walls are not the preferred option. Cast-in-place wingwalls should be kept structurally separate from a box culvert to accommodate some differential settlement.

Based on the various founding conditions noted above and the existing invert elevation of 197.6 metres, the replacement wingwalls may be founded on engineered fill at the elevations noted in the following table.

Wall Type	Armour Stone Block Walls	Gabion Walls	Concrete Gravity and Cantilever Walls	RSS Walls
Maximum Founding Elevation (m)	197.3	197.2	196.2	197.3

Wingwalls founded on at least one metre thick engineered fill placed and compacted as noted above may be designed using a factored geotechnical resistance at Ultimate Limit States (ULS) of 150 kilopascals and a geotechnical resistance at Serviceability Limit States (SLS) of 100 kilopascals. The SLS value corresponds to 25 millimetres of settlement.

6.2.3 Resistance to Lateral Forces

The lateral pressures acting on the wingwalls will depend on the backfill soils, the type and method of placement of the backfill materials behind the walls, and the subsequent lateral movement of the structures. The resistance to lateral forces/sliding resistance between the wingwalls 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 δ, may be used for the interaction between the base of the wall and the founding soil:

Wall Type	Interaction	Angle of Friction, δ (degrees)	Coefficient of Friction, Tan δ
Armour Stone Block Wall	Stone block on Granular A leveling pad	30	0.58
Gabion Wall	Gabion basket on Granular A leveling pad	30	0.58
Reinforced Concrete Gravity or Cantilever Wall	CIP concrete strip footing on engineered fill	34	0.67
RSS Block System Wall	Pre-cast concrete block facing units on Granular A levelling pad	30	0.58

6.2.4 Lateral Earth Pressures for Design

The lateral pressures acting on the proposed wingwalls 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 No. 200 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 kPa 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, 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 1.4 metres behind the walls (case (a) from commentary on CHBDC Figure C6.6).

- 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_o	0.53

- If the wall support allows lateral yielding (unrestrained structure, such as typically the case for wingwalls), 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 1.4 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

6.3 Construction Considerations

6.3.1 General

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

It is recommended that the footing excavations be carried out such that the final 0.5 metres of excavation/subexcavation is completed with a geotechnical quality verification engineer (QVE) on site. The prepared excavation bases should be inspected by the QVE and granular base materials should be placed immediately after inspection to protect the founding materials.

Sediment control such as silt fences and erosion control blankets may be required during construction. All wingwalls foundations must be protected against scour as noted in the CHBDC Section 1.9.5.

6.4 Excavations and Groundwater Control

Excavations will extend through the existing fill and organics to the underlying native granular soils. It is anticipated that excavation for the engineered fill will extend approximately 0.8 to 1.8 metres below the inferred groundwater level of elevation 197.8 metres. In order to place and compact the engineered fill in the dry, dewatering of the excavations will be required. Groundwater control may be achieved by using properly constructed and filtered sumps. Sumps should be maintained outside of the actual wall limits. Heavy pumping should be expected during high water periods. Based on the subsurface soil and groundwater conditions, it is anticipated that the dewatering rate will exceed 50 cubic metres per day (m^3/day) and therefore a Permit to Take Water (PTTW) will be required for this site. Based on empirical methods, the estimated hydraulic conductivity is 6.5×10^{-6} metres per second for the sandy silt and 1.6×10^{-5} metres per second for the silty sand.

Surficial water seepage into the excavations should be expected and will be heavier during periods of sustained precipitation. Surface water runoff should be directed away from the excavations at all times. The existing culvert flows may need to be diverted/piped during construction. The appropriate non-standard special provision (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 and organic 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 and organic materials and any native granular soils below the groundwater table would be classified as Type 3 soils. Properly dewatered native granular soils would be classified as Type 2 soils.

6.5 Staging and Temporary Roadway Protection

It is understood that two lanes are to remain open to traffic during construction; therefore, replacement of the wingwalls may require temporary support systems which could consist of soldier piles and lagging or steel sheet piles. The temporary shoring may have a maximum height of 4 metres above the excavation base. 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. Cobbles and boulders noted to be present in the fill materials encountered during the investigation may make installation of sheet piles difficult.

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:

$$p = K_a (\gamma H + q)$$

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

K_a = active coefficient of earth pressure

γ = soil unit weight

q = surcharge for traffic and other loading

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

Soil Type	Coefficient of Earth Pressure			Internal Angle of Friction (degrees)	Unit Weight (kN/m^3)
	Active, K_a	At Rest, K_o	Passive, K_p		
Fill	0.36	0.53	2.8	28	19
Peat	0.21	0.35	4.7	41	10
Organic Silt	0.49	0.66	2.0	20	18
Silty Sand & Gravel	0.27	0.43	3.7	35	22
Sandy Silt	0.36	0.53	2.8	28	20
Silty Sand	0.36	0.53	2.8	28	20

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 part of the report was prepared by and Ms. Nicole A. Gould, P.Eng. and reviewed by Mr. Azmi M. Hammoud, P.Eng., an Associate and Geotechnical Engineer with Golder. 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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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

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.)

Dynamic Cone Penetration Resistance; N_d:

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

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

Piezo-Cone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils

Consistency

	c_u, s_u kPa	c_u, s_u 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.

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
v	poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma$-u)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_3	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1+\sigma_2+\sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

(a) Index Properties (continued)

w	water content
w_l	liquid limit
w_p	plastic limit
l_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 = 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 δ
c'	effective cohesion
c_u,s_u	undrained shear strength (ϕ = 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)

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London, Ontario

RECORD OF BOREHOLE No 103

 METRIC

PROJECT 12-1132-0163
W.P. 3101-10-00 LOCATION N 5004221.9 , E 381857.0 ORIGINATED BY MA
DIST HWY 6 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY LMK
DATUM GEODETIC DATE June 6, 2013 CHECKED BY

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
199.62	GROUND SURFACE							
0.09	FILL, sandy topsoil				Cuttings			
	FILL, sand and gravel, some silt, with cobbles and boulders. Very loose to loose. Brown	1		7	Granular bentonite	199		
		2		3	Filter sand; Caved material	198		
		3		6		197	o	
196.72 / 2.90	FILL, sandy silt, some gravel, topsoil. Loose. Brown				Standpipe			
196.33 / 3.29	PEAT with organic silt pockets, trace gravel. Firm. Black and grey	4		6			84	
195.96 / 3.66	SANDY SILT, some clay. Very loose to compact. Grey					196		
		5		WH			Wp–w–WL	0 31 51 18
		6		1	Caved material	195	o	
						194		
193.13 / 6.49	END OF BOREHOLE	7		96/100mm			o	

Practical auger refusal

Groundwater encountered at about elev. 197.7m during drilling on June 6, 2013.

Water level measured in standpipe at elev. 199.09m after installation on June 6, 2013.

Water level measured in standpipe at elev. 198.20m on September 12, 2013.

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100
PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30
UNIT WEIGHT γ kN/m³

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

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RECORD OF BOREHOLE No 104

METRIC

PROJECT 12-1132-0163
W.P. 3101-10-00
LOCATION N 5004229.0 , E 381850.0
ORIGINATED BY MA
DIST HWY 6
BOREHOLE TYPE POWER AUGER, HOLLOW STEM
COMPILED BY LMK
DATUM GEODETIC
DATE June 6, 2013
CHECKED BY

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
199.82	GROUND SURFACE										
0.00	FILL, sand and gravel, crushed										
0.21	FILL, sand and gravel, trace silt; Loose to compact; Brown	1	SS	10		199					
		2	SS	6	∇	198					
197.69 / 2.13	PEAT, fibrous; Very soft; Black	3	SS	1			330				
197.23 / 2.59	ORGANIC SILT; Very soft; Grey					197					
196.68 / 3.14	SANDY SILT, to sand and silt, some clay, trace to some gravel; Very loose; Grey	4	SS	4		196					
		5	SS	WH				5	32	51	12
		6	SS	1		195		11	39	37	13
193.88 / 5.94	END OF BOREHOLE					194					

Practical auger refusal

Groundwater encountered at about elev. 197.8m during drilling on June 6, 2013.

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa — ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT W_L; WATER CONTENT (%) (10, 20, 30); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%); STRAT PLOT

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

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RECORD OF BOREHOLE No 105

METRIC

PROJECT 12-1132-0163

W.P. 3101-10-00 LOCATION N 5004239.0 , E 381856.9 ORIGINATED BY MA

DIST HWY 6 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY LMK

DATUM GEODETIC DATE June 6, 2013 CHECKED BY

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
199.88	GROUND SURFACE						
0.00	FILL, sand and gravel, crushed Brown						
0.30	FILL, sand, trace gravel Brown						
199.24 0.64	FILL, sand, fine to medium, trace silt, gravel Very loose to loose Brown	1	SS	5			
		2	SS	1	∇		
197.75 2.13	PEAT, fibrous Very soft Black	3	SS	1		445	
						363	
196.59 3.29	SILTY SAND, to sand and silt, some clay, trace to some gravel Very loose Grey	4	SS	4			
		5	SS	2			14 40 36 10
		6	SS	WH			
193.88 6.00	END OF BOREHOLE Practical auger refusal Groundwater encountered at about elev. 197.8m during drilling on June 6, 2013.						

ELEVATION SCALE: 199, 198, 197, 196, 195, 194

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L — WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

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

LDN_MTO_06 1211320163-1000.GPJ LDN_MTO.GDT 24/09/13

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London, Ontario

RECORD OF BOREHOLE No 106

METRIC

PROJECT 12-1132-0163
W.P. 3101-10-00 | LOCATION N 5004233.7 , E 381866.0 | ORIGINATED BY MA
DIST | HWY 6 | BOREHOLE TYPE POWER AUGER, HOLLOW STEM | COMPILED BY LMK
DATUM GEODETIC | DATE June 6, 2013 | CHECKED BY

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
199.90	GROUND SURFACE					
0.06	FILL, silty sand and gravel, Brown					
199.47 / 0.43	FILL, sand and gravel, crushed, Brown					
	FILL, sand, some gravel, trace silt, Very loose, Brown	1	SS	3		
		2	SS	3	▽ (elev. 197.8)	
197.77 / 2.13	FILL, sand, fine to medium, trace silt, Very loose, Brown	3	SS	21		
197.37 / 2.53	ORGANIC SILT AND FINE SAND, Compact, Grey					
197.00 / 2.90	SILTY SAND AND GRAVEL, trace organics, Loose to compact, Grey	4	SS	16		
		5	SS	4		
195.48 / 4.42	SILTY SAND, to sand and silt, some clay, some gravel, Very loose to loose, Grey	6	SS	1		10 43 35 12
		7	SS	6		16 39 34 11
193.19 / 6.71	END OF BOREHOLE					

Elevation scale: 199, 198, 197, 196, 195, 194

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

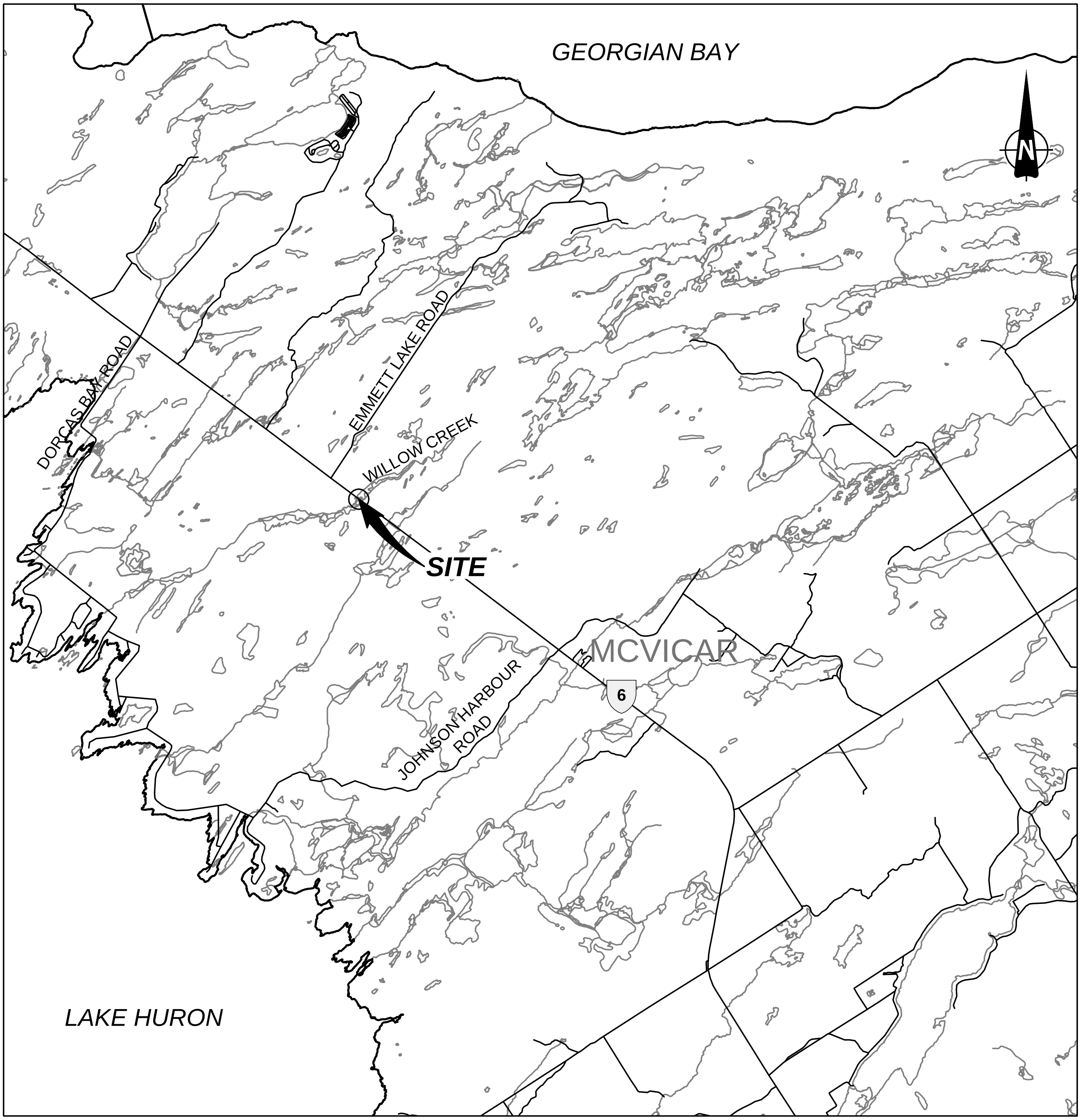
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE — 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30 (Sample 3 plotted off-scale: 115)

UNIT WEIGHT γ kN/m³

Groundwater encountered at about elev. 197.8m during drilling on June 6, 2013.

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



SCALE
0 2000 4000m
1:100,000

REFERENCE

PLAN BASED ON CANMAP STREETFILES V.2008.5.

NOTE

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

PROJECT

CULVERT REHABILITATION, WILLOW CREEK
SITE No. 2-457/C, STATION 17+378
GWP 3101-10-00

TITLE

KEY PLAN

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LONDON, ONTARIO

PROJECT No.		12-1132-0163	FILE No.	1211320163-1000-F02001
			SCALE AS SHOWN	REV. 0
CADD	LMK	July 23/13		
CHECK			**FIGURE 1**	

Drawing file: 1211320163-1000-F02001.dwg Sep 24, 2013 - 4:58pm

MINISTRY OF TRANSPORTATION, ONTARIO

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

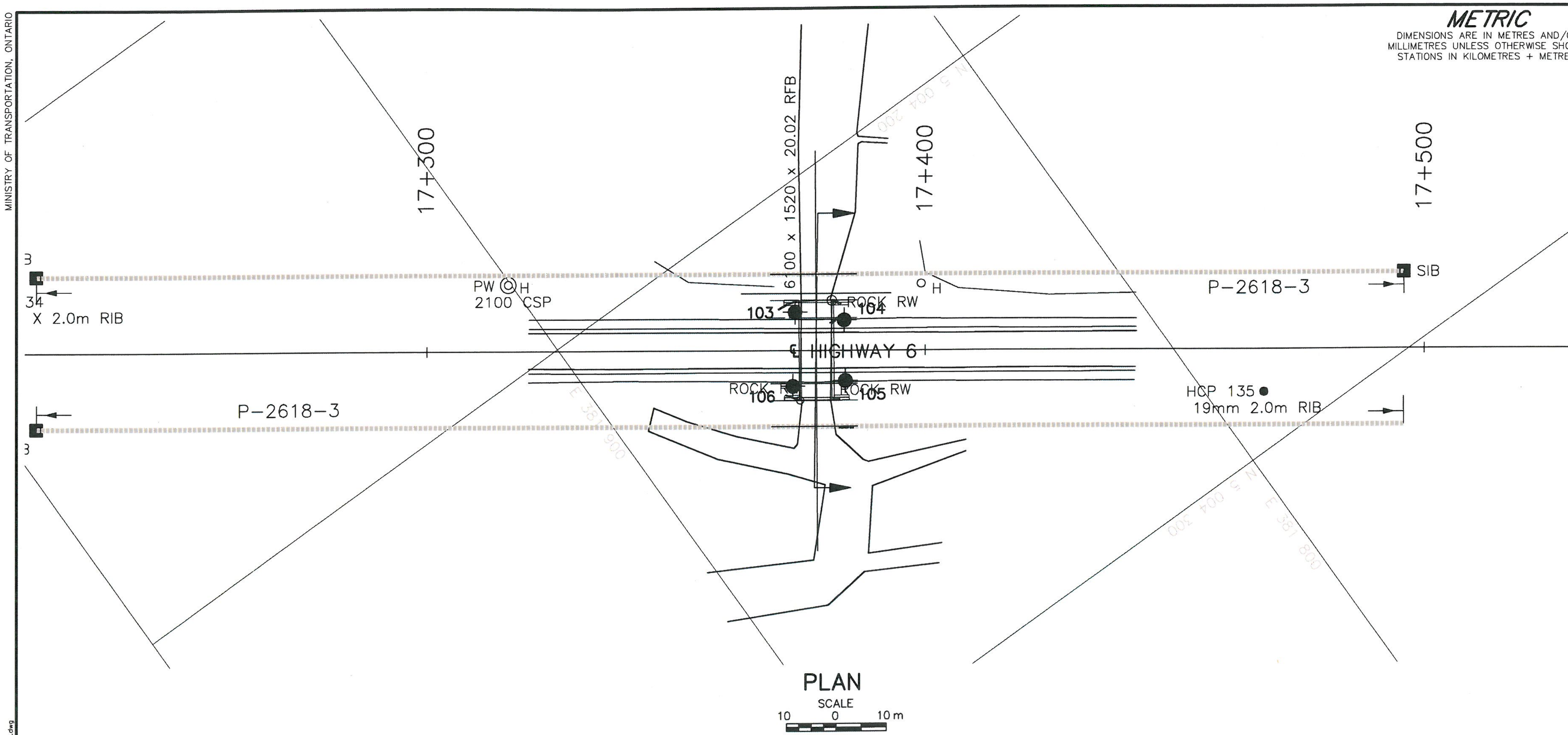
CONT No.
WP No. 3101–10–00

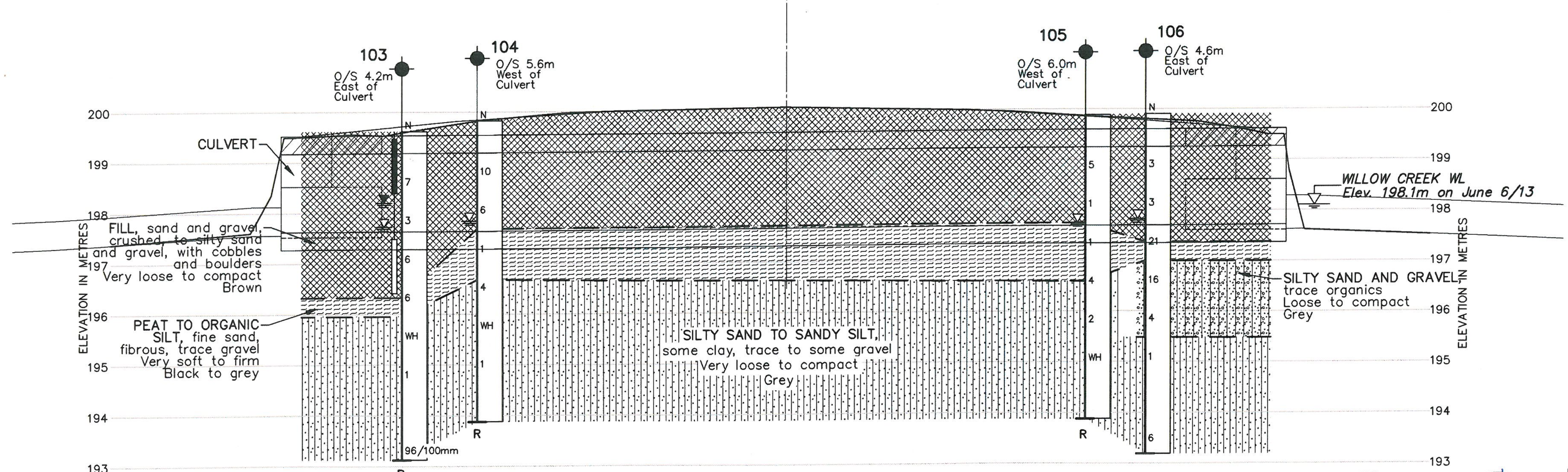
SHEET

CULVERT REHABILITATION WILLOW CREEK

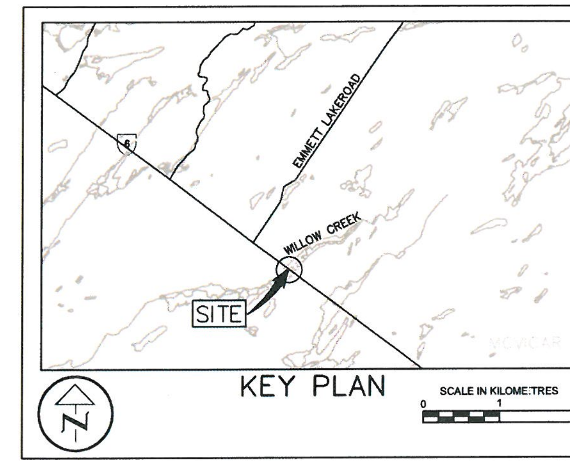
STRUCTURE REPLACEMENTS AND REHABILITATION
BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates Ltd.
LONDON, ONTARIO, CANADA





PROFILE ALONG ℄ CULVERT
HORIZONTAL SCALE 1 0 1 m
VERTICAL SCALE 1 0 1 m



KEY PLAN

LEGEND

- Borehole – Current Investigation
- Seal
- Standpipe
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL measured on June, 2013
- WL encountered during drilling
- DRY Water level not established
- R Refusal

No.	ELEVATION	CO-ORDINATES (MTM ZONE 10) NORTHING	EASTING
103	199.62	5 004 221.9	381 857.1
104	199.82	5 004 229.0	381 850.0
105	199.88	5 004 239.0	381 856.9
106	199.90	5 004 233.7	381 866.0

NOTES

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

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

REFERENCE

Base plans provided by Stantec.

LICENSED PROFESSIONAL ENGINEER N. A. GOULD 100122043 PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER F. J. HEFFERNAN PROVINCE OF ONTARIO

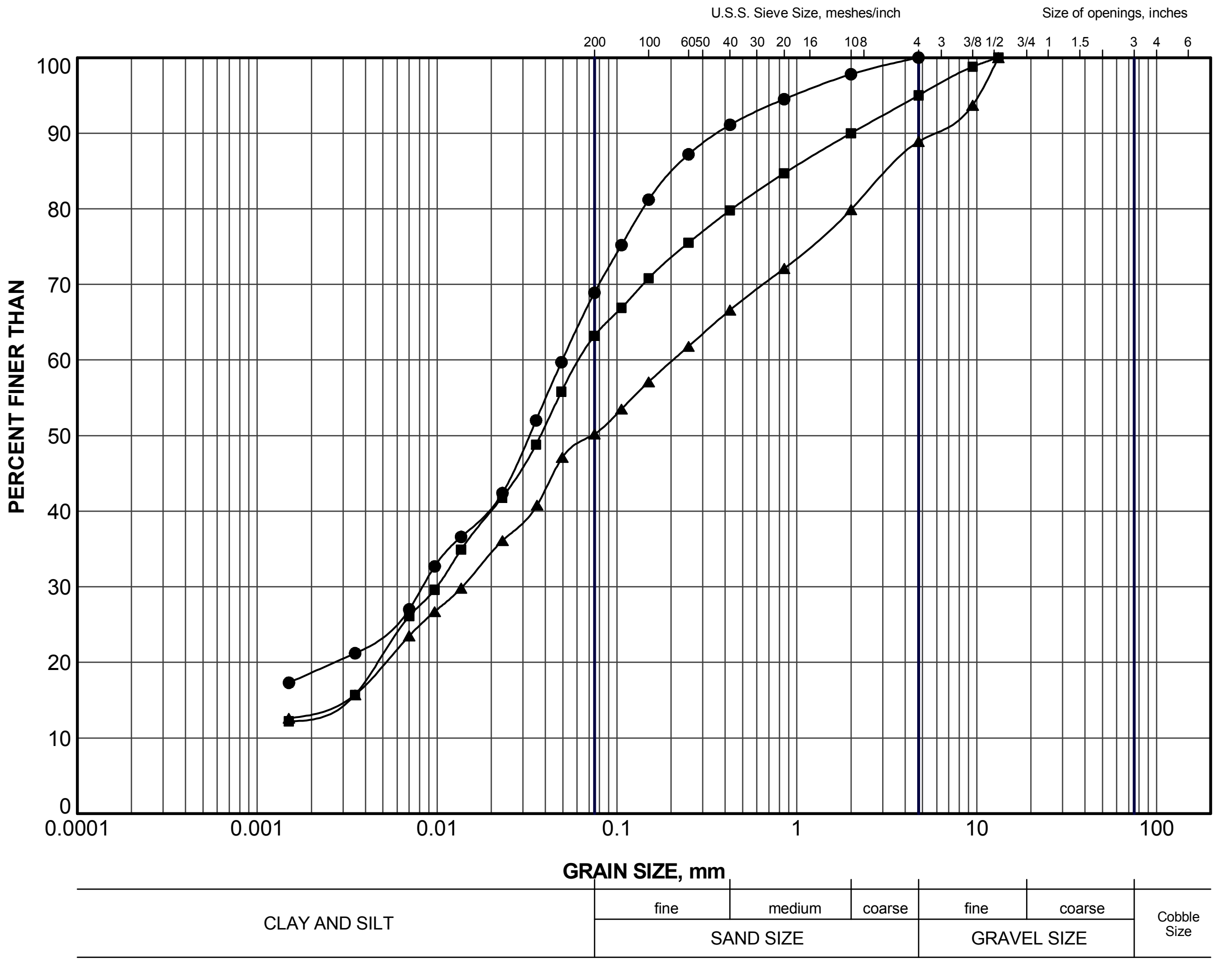
NO.	DATE	BY	REVISION

Geocres No. 41H–136
HWY. 6 | PROJECT NO. 12-1132-0163 | DIST.
SUBM'D. BT | CHKD. | DATE: Sept. 20/13 | SITE: 2-457/C
DRAWN: LMK | CHKD. | APPD. | DWG. 1

PLOT DATE: September 27, 2013

APPENDIX A

Laboratory Test Data



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	103	5	195.6
■	104	5	195.8
▲	104	6	195.0

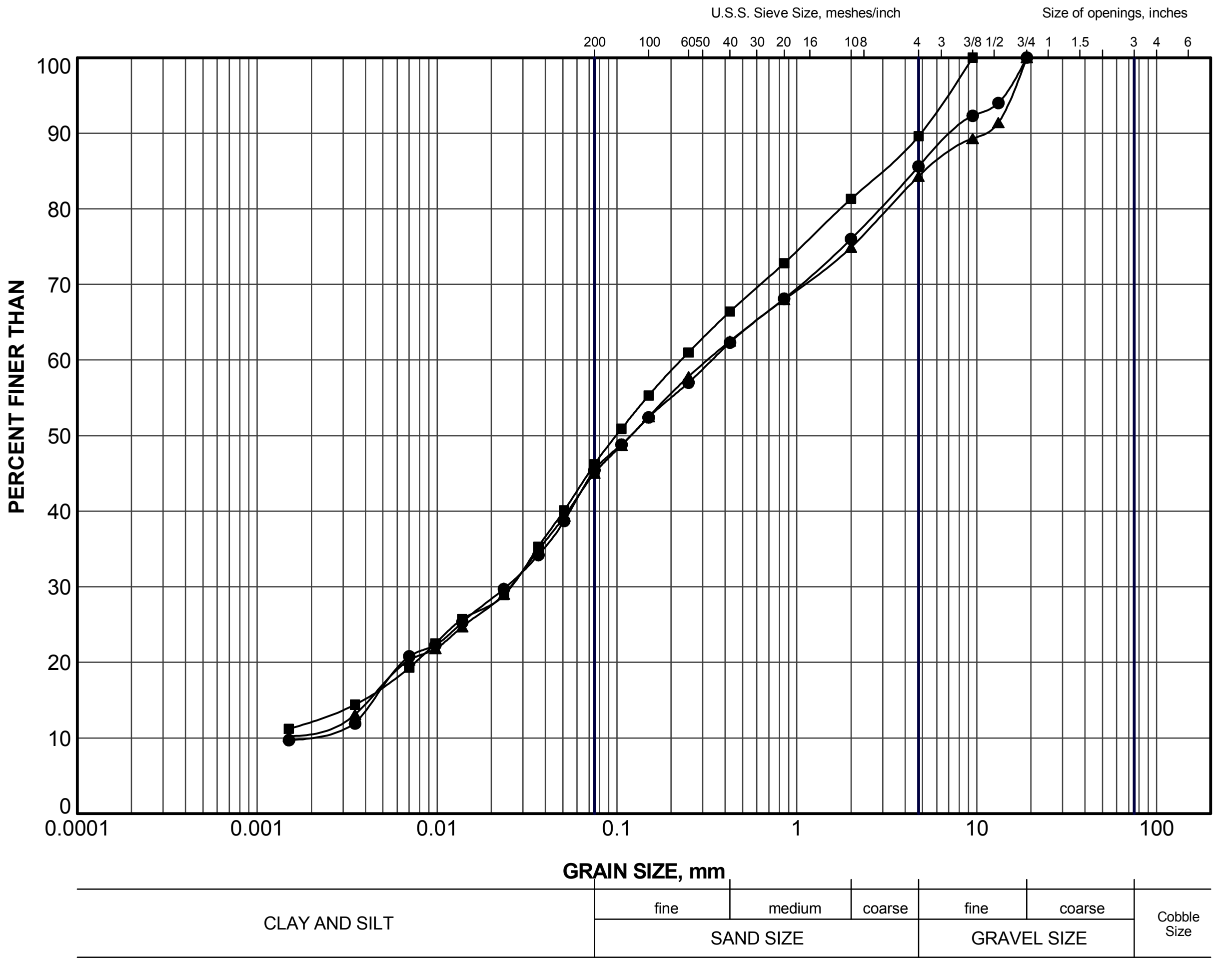
PROJECT
CULVERT REHABILITATION, WILLOW CREEK
SITE No. 2-457/C, STATION 17+378
GWP 3101-10-00

TITLE
GRAIN SIZE DISTRIBUTION
SANDY SILT

Golder Associates
LONDON, ONTARIO

PROJECT No.	12-1132-0163		FILE No.	1211320163-1000-F020A1
			SCALE N/A	REV.
DRAWN	LMK	Jun 26/13	**FIGURE A-1**	
CHECK				

LDN_MTO_GSD GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	105	5	195.8
■	106	6	195.1
▲	106	7	193.5

PROJECT

CULVERT REHABILITATION, WILLOW CREEK
SITE No. 2-457/C, STATION 17+378
GWP 3101-10-00

TITLE

GRAIN SIZE DISTRIBUTION

SILTY SAND

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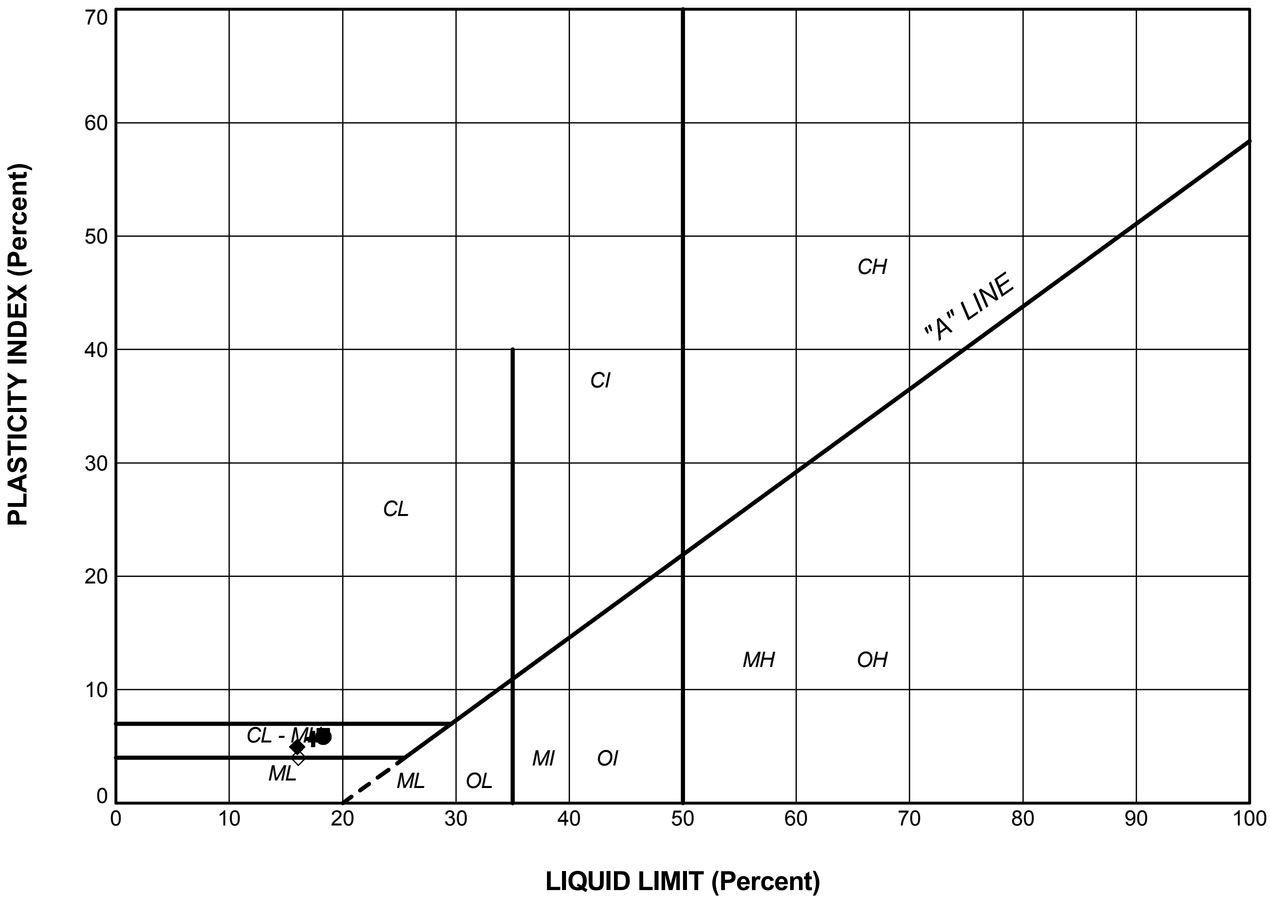
LONDON, ONTARIO

PROJECT No.		12-1132-0163	FILE No.	1211320163-1000-F020A2
			SCALE N/A	REV.
DRAWN	LMK	Jun 24/13	**FIGURE A-2**	
CHECK				

LDN_MTO_GSD GLDR_LDN.GDT



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	103	5	18.3	12.5	5.9
■	104	5	18.3	12.3	6.1
▲	104	6	18.1	12.0	6.1
✚	105	5	17.4	11.8	5.7
◆	106	6	16.0	11.1	5.0
◇	106	7	16.1	12.1	4.0

PROJECT
CULVERT REHABILITATION, WILLOW CREEK
SITE No. 2-457/C, STATION 17+378
GWP 3101-10-00

TITLE
PLASTICITY CHART

PROJECT No.	12-1132-0163		FILE No.	1211320163-1000-F020A3
			SCALE N/A	REV.
DRAWN	LMK	Jun 24/13		
CHECK			**FIGURE A-3**	

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LDN_MTO_PI GLDR_LDN.GDT

APPENDIX B

Site Photographs

Photograph 1: North elevation (inlet) of Culvert Site 2-457/C.

Photograph 2: South elevation (outlet).

Photograph 3: Highway 6 looking southeast from Culvert Site 2-457/C.

n:\active\2012\1132 - geo\1132-0100\12-1132-0163 stantec-fdns-mega culverts-3011-e-0041\ph 1000-gwp 3101-10-00\rpts\r02\1211320163-1000-r02 sept 26 13 (final) app b - photos.docx

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North America	+ 1 800 275 3281
South America	+ 55 21 3095 9500

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