



September 2014

FOUNDATION INVESTIGATION AND DESIGN REPORT

**Culvert Replacement, Archer Municipal Drain
Site No. 14-439/C, Station 12+231, Highway 21
Contract 2 Structure Replacements and Rehabilitation
GWP 3040-11-00
Ministry of Transportation, West Region**

Submitted to:

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

REPORT



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LIST OF ABBREVIATIONS

LIST OF SYMBOLS

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PART A

FOUNDATION INVESTIGATION REPORT

**CULVERT REPLACEMENT, ARCHER MUNICIPAL DRAIN
SITE NO. 14-439/C, STATION 12+231, HIGHWAY 21
CONTRACT 2 STRUCTURE REPLACEMENTS AND REHABILITATION
GWP 3040-11-00
MINISTRY OF TRANSPORTATION - WEST REGION**



1.0 INTRODUCTION

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

This report addresses the proposed replacement of the culvert at Archer Municipal Drain (Site 14-439/C) at Station 12+231 on Highway 21 south of Forest in the Township of Warwick, Lambton County, Ontario.

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



2.0 SITE DESCRIPTION

The subject culvert is situated at Station 12+231 on Highway 21, approximately 0.9 kilometres north of Egremont Road in the Township of Warwick, Lambton County, Ontario. The town of Forest is 9.5 kilometres north of the site. The location of the culvert is shown on the Key Plan, Figure 1.

This section of Highway 21 is currently a two lane undivided highway with gravel shoulders. It is generally oriented north-south in the vicinity of the subject site. The creek flow direction in the culvert is from west to east beneath Highway 21. The existing culvert is a non-rigid frame open footing (NRFO) concrete structure with the following characteristics:

Dimensions (m)	Obvert Elevation (m)		Construction
	Lt ¹	Rt ¹	
3.05 x 1.52 x 19.53	224.80	224.77	NRFO

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 upstream and downstream of the culvert are grass covered and the channel flows through fields adjacent to Highway 21. Site photographs are provided in Appendix B.

The culvert is situated in a rural agricultural area with low relief. Ground surface elevations in the vicinity of the culvert site range from about 224 to 226 metres.

2.1 Site Geology

The project area is located within the Horseshoe Moraines physiographic region. This region is characterized by irregular, stony knobs and ridges composed mostly of till with some sand and gravel deposits. Deposits of pale brown, fine-textured till with a moderate degree of stoniness are mapped as covering most of the area in the vicinity of the site.¹ The quaternary geological mapping indicates that surficial soils consist of St. Joseph Till consisting of clayey silt till.² Geological mapping also indicates that the underlying bedrock consists of black bituminous shale with greenish-grey silty shale interbeds of the Kettle Point Formation of the Port Lambton Group of Upper Devonian age.³ The bedrock surface at the site is at about elevation 198 metres, with the overburden thickness being about 27 metres.⁴

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

² Fitzgerald, W.D., and assistants 1979: Quaternary Geology. Sarnia-Brights Grove Area, Southern Ontario; Ontario Geological Survey, Prelim. Map 2222, scale 1:50,000.

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

⁴ Fitzgerald, W.D., and assistants. 1979: Bedrock topography Series. Sarnia-Brights Grove Area. Southern Ontario; Ontario Geological Survey. Prelim. Map P.2206, scale 1:50,000.



3.0 INVESTIGATION PROCEDURES

The field work for the investigation was carried out on July 4, 2013, during which time 3 boreholes were drilled at the locations shown on the Borehole Location Plan, Drawing 1. The boreholes were drilled using track-mounted CME 75 drilling equipment supplied and operated by a specialist drilling contractor. Samples of the overburden were typically obtained at depth intervals of 0.75 metres using 50 millimetre outside diameter split spoon sampling equipment in accordance with the Standard Penetration Test (SPT) procedures (ASTM D1586).

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

Groundwater conditions in the boreholes were observed throughout the drilling operations and a piezometer was installed in borehole 208 as indicated on the corresponding Record of Borehole sheet. The boreholes were backfilled in accordance with current MTO procedures and Ontario Regulation 903 (as amended).

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

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

Borehole	Location (m)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting		
207	4 763 622	345 305	225.4	9.8
208	4 763 639	345 291	225.5	9.8
209	4 763 628	345 292	225.5	11.1



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.

In summary, the boreholes drilled at the site encountered topsoil and/or surficial fill overlying cohesive fill materials and clayey silt glacial till. The locations and elevations of the boreholes, together with the interpreted stratigraphic profile, are shown on the attached Drawing 1. A detailed description of the subsurface conditions encountered in the boreholes is provided on the Record of Borehole sheets and is summarized in the following sections.

4.2 Soil Conditions

4.2.1 Topsoil

Topsoil layers on average 0.2 metres thick were encountered at the ground surface of boreholes 207 and 208. A 0.8 metre thick layer of topsoil fill was found at elevation 223.4 metres in borehole 209. The fill in boreholes 207, 208 and 209 also contained trace amounts of topsoil. The topsoil fill in borehole 209 had a water content of 45 per cent.

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

4.2.2 Fill

Fill was encountered at the surface of borehole 209 and beneath the topsoil in boreholes 207 and 208. Excluding the topsoil fill found in borehole 208, the fill thickness ranged from 1.9 to 2.1 metres. The fill consisted of crushed granular road base, sand and gravel and clayey silt.

Granular materials comprised the upper fill in boreholes 207 and 209. The thickness of the granular fill was 0.5 and 0.7 metres in boreholes 207 and 209, respectively.

The lower non-organic fill consisted of clayey silt. The cohesive fill was firm to stiff with recorded N values of 5 to 12 blows per 0.3 metres. Samples of clayey silt fill had measured water contents of 21 to 22 per cent.



4.2.3 Clayey Silt Till

The fill in all three boreholes was underlain by clayey silt glacial till encountered from elevation 222.6 to 223.3 metres to the termination depths of 9.8 to 11.3 metres below ground surface. The grain size distributions of nine samples of clayey silt till are presented on Figure A-1. Cobbles and boulders, although not specifically encountered in the boreholes, should be anticipated in the clayey silt till due to its depositional history.

The clayey silt till was stiff to very stiff based on recorded N values from 9 to 23 blows per 0.3 metres. The clayey silt till was generally stiff to very stiff above elevation 220.5 metres and stiff below this elevation. The water contents of the clayey silt till samples varied between 14 and 18 per cent. The clayey silt till is of low plasticity based on plastic limits of 14 to 16 per cent, liquid limits of 26 to 30 per cent and plasticity indices of 12 to 14 based on nine Atterberg limit determinations. The results of the Atterberg limit testing are shown on Figure A-2. The grain size distribution curves for selected samples of the clayey silt till are provided on Figure A-1.

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 208. Installation details are provided on the corresponding Record of Borehole sheet following the text of this report. Groundwater was encountered in borehole 208 at a depth of 9.0 metres or elevation 216.5 metres. The groundwater level could not be established in the remaining boreholes as free groundwater was not observed. 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)	
			Sept. 5, 2013	July 25, 2014
207	225.44	*	-	-
208	225.49	216.5	220.43	222.02
209	225.54	*	-	-

* Groundwater level not established.

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 222.5 metres on July 4, 2013 and at elevation 222.9 metres on April 30, 2014. On July 4, 2013 the groundwater observation piezometer installed in borehole 208 was dry to 9.8 metres below ground surface or about elevation 215.7 metres. On September 5, 2013 the water level in the groundwater observation piezometer installed in borehole 208 was about 5.1 metres below ground surface or at about elevation 220.4 metres. The groundwater observation piezometer installed in borehole 208 was damaged sometime after September 5, 2013, therefore no reading could be obtained on April 30, 2014. On July 25, 2014 the water level in the groundwater observation piezometer installed in borehole 208 was about 3.5 metres below ground surface or at about elevation 222.0 metres.



Based on the observed groundwater levels, the surrounding topography, the soil colour change from brown to grey and water levels in the drain, the inferred groundwater level has been assumed to be elevation 222 metres for design purposes. The groundwater levels are expected to fluctuate seasonally and are expected to be higher during periods of sustained precipitation or during spring snow melt conditions and will be influenced by flows in the watercourse.



5.0 MISCELLANEOUS

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

GOLDER ASSOCIATES LTD.

ORIGINAL SIGNED

ORIGINAL SIGNED

Dirka U. Prout, P.Eng.

W.M. Kellestine, P.Eng.
Principal

ORIGINAL SIGNED

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

BT/DUP/WMK/FJH/cr

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PART B

FOUNDATION DESIGN REPORT

CULVERT REPLACEMENT, ARCHER MUNICIPAL DRAIN
SITE NO. 14-439/C, STATION 12+231, HIGHWAY 21
CONTRACT 2 STRUCTURE REPLACEMENTS AND REHABILITATION
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6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides recommendations on the foundation aspects of the design of the replacement culvert at Archer Municipal Drain (Site 14-439/C), located at Station 12+231 on Highway 21 in the Township of Warwick in Lambton County, Ontario.

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

The existing culvert is a 19.5 metre long non-rigid frame open footing (NRFO) concrete structure with a 3.1 metre span, a 1.5 metre high opening, and an approximate invert elevation of 223.3 metres. The existing culvert has approximately 0.5 metres of fill cover.

6.2 Replacement Culvert

Based on information provided by Stantec, it is understood that consideration is being given to replace the existing NRFO culvert with a new 3.0 by 2.7 metre concrete open footing culvert or a 3.0 by 2.4 metre concrete box culvert. It has been indicated by Stantec that a concrete box culvert of precast or cast-in-place (CIP) construction is the preferred structural alternative. The existing stone and concrete wingwalls will be replaced with reinforced soil system (RSS) retaining walls with a typical length of 3 metres. No grade raise is proposed at this location.

6.2.1 Foundations

The subsurface conditions encountered during the investigation generally consisted of the existing pavement structure overlying variable embankment fill materials to approximate elevation 223 to 224 metres constructed on an extensive stratum of stiff to very stiff clayey silt till confirmed to elevation 214 metres. The inferred long-term groundwater level is at elevation 222 metres. The water level in the watercourse was about elevation 222.5 metres at the time of the investigation.

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

The new culvert will have proposed invert elevations of 222.02 metres at the inlet and 221.95 metres at the outlet. Based on the soil conditions encountered at the borehole locations, and the proposed invert elevations, the replacement box or open footing culvert may be founded on the stiff to very stiff clayey silt till at or below



elevation 222.6 metres. Any observed fill or organic materials should be removed to the native soils. Any low areas should be brought to design grade using lean concrete fill or well graded granular materials.

Geotechnical Resistances

Assuming a floor slab thickness of 250 millimetres for a box culvert and allowing for a minimum bedding thickness of 300 millimetres and a 75 millimetre thick levelling pad, the maximum founding elevation for a box culvert will be near elevation 221.4 metres. Allowing for a frost depth of 1.2 metres for open footing culvert foundations, the maximum founding elevation will be elevation 220.8 metres. The stiff to very stiff clayey silt till is suitable for support of the proposed culvert replacement. A factored geotechnical resistance at Ultimate Limit States (ULS) of 300 kilopascals (kPa) and a geotechnical reaction at Serviceability Limit States (SLS) of 200 kPa may be used for design purposes provided that any foundations have a minimum width of 0.5 metres and that the subgrade has been properly prepared (see Section 6.6). The SLS value corresponds to a maximum of 25 millimetres of total settlement for new culvert construction.

Frost Treatment and Scour Protection

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

Resistance to Lateral Forces/Sliding Resistance

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

In accordance with the CHBDC Section 6.7.5, a factor of 0.8 is applied in the equation to calculate the factored horizontal geotechnical resistance, H_{ri} , as follows:

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

where:

A'	-	effective contact area, square metres
c'	=	Nil
$\tan \delta$	-	as given below
V	-	unfactored vertical force, kilonewtons
H_f	-	unfactored horizontal load, kilonewtons



The coefficient of friction, $\tan \delta$, between the culvert base and founding material are shown in the following table.

Structure	Interaction	Angle of Friction, δ (degrees)	Coefficient of Friction, $\tan \delta$
CIP Box or Open Footing Culvert	CIP concrete on native clayey silt till	32	0.62
Precast Box Culvert	Pre-cast concrete on Granular A bedding	30	0.58

6.2.2 Bedding

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

6.2.3 Backfill and Cover

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

The backfill should consist of free-draining, non-frost susceptible granular materials such as OPSS Granular A or Granular B Type III placed and compacted in accordance with SP105S21 but with less than 5 per cent passing the 0.075 millimetre sieve. All bedding, backfill, and cover materials should be placed in accordance with SP105S21, OPSS 902 and SP422S01.

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

6.2.4 End Treatments and Camber

The culvert invert will be on the native clayey silt till. An outlet filter is not recommended due to the generally cohesive soils that will be present at the outlet, particularly if the head difference between the culvert inlet and outlet is high. No grade raising is proposed as part of the culvert replacement and relatively low cover is proposed for the replacement culvert; therefore it is not necessary to provide a camber.



6.3 Retaining Walls

The existing structure retaining walls are about 3.0 metres long and 3.0 metres in height. It is understood that consideration is being given to constructing the retaining walls of the replacement structure as reinforced soil system (RSS) walls. Concrete gravity or cantilever retaining walls are suitable alternatives from a foundation engineering perspective. The retaining wall options are discussed below.

6.3.1 Options

Reinforced Concrete Gravity and Cantilever Walls

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

RSS Walls

The height of the retaining walls will be relatively low. Therefore, reinforced soil system walls 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 a RSS block system wall is selected, the geotechnical aspects of the global stability of the 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 a RSS block system wall. This wall type can be constructed relatively quickly and inexpensively using small equipment.

6.3.2 Foundations

Cast-in-place reinforced concrete gravity or cantilever walls founded on concrete strip footings must be provided with a frost cover of 1.2 metres below the adjacent ground or thermal equivalent. Cast-in-place retaining walls should be kept structurally separate from a box culvert to accommodate some differential settlement.

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 supplier, a concrete levelling pad may also be used. Depending on the design selected by the RSS supplier, it may not be necessary to provide 1.2 metres of earth cover or thermal equivalent for frost protection. However, the foundations must have adequate embedment to provide a stable structure. Typically the embedment depth, defined as the distance between the top of the levelling pad to the top of the adjoining finished grade, is a minimum of 500 millimetres. All retaining wall foundations must be protected against scour as noted in the CHBDC Section 1.9.5.

The preliminary General Arrangement Drawings provided by Stantec indicate that the proposed grade adjacent to the new culvert will be between elevations 222.0 and 222.3 metres. The approximate founding elevation for



the RSS walls is shown at 221.8 metres. Based on this information, foundations for the various wall types may be founded on the native stiff to very stiff clayey silt till at or below the elevations noted in the following table.

Wall Type	Maximum Founding Elevation (m)
Concrete Gravity and Cantilever Walls	221.1
RSS Walls	221.5

Retaining walls founded at or below the elevations indicated above may be designed using a factored geotechnical resistance at ULS of 300 kPa and a geotechnical reaction at SLS of 200 kPa. The SLS value corresponds to 25 millimetres of settlement.

If fill materials, organics, loose/softened soils, or otherwise deleterious materials are noted in the base of footing excavations, the excavations should be extended to the native soils. The subexcavated areas should be brought to design grade using lean concrete fill or compacted OPSS Granular A or Granular B Type II or Type III.

6.3.3 Resistance to Lateral Forces

The lateral pressures acting on the retaining walls 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 retaining walls and the subgrade soils should be calculated in accordance with Section 6.7.5 of the CHBDC. Each retaining wall shall be checked for overturning. Assuming that the founding soils are not loosened/disturbed during excavation and footing construction, an angle of friction and corresponding unfactored coefficient of friction, $\tan \delta$, of 32 degrees and 0.62, respectively, may be used for the composite interaction between the base of the wall, levelling pad and the founding soil.

6.4 Liquefaction Potential and Seismic Analysis

6.4.1 Seismic Parameters

The site is located near the town of Forest in southern Ontario. According to Table A.3.1.1 of the CHBDC, the zonal acceleration ratio, A , applicable to this site is 0.00. The corresponding acceleration related seismic zone, Z_a , is 0. Based on the site stratigraphy, the soil profile type is categorized as Type I with a seismic site response coefficient, S , of 1.0 based on the CHBDC criteria.

The importance category of the replacement culvert is "other" based on the current version of the CHBDC. The corresponding seismic performance zones (SPZ) to this importance category is 1. Structural culverts situated in SPZ 1 need not be analyzed for seismic loads. However, design forces for restraining elements and support lengths must meet the minimum requirements as outlined in CHBDC Clause 4.4.5.1. It should be noted that the MTO views culverts with spans of 3 metres or greater as being similar to bridges. The designer should ensure that the selected culvert design meets the seismic requirements for buried structures as outlined in Clause 7.5.5 of the CHBDC.



6.4.2 Seismic Hazard Assessment

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

6.5 Lateral Earth Pressures for Design

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

The following recommendations are made concerning the design of the walls in accordance with the current CHBDC. It should be noted that these design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select, free-draining granular fill meeting the specifications of OPSS Granular A, Granular B Type II or Granular B Type III but with less than 5 per cent passing the 0.075 millimetre sieve should be used as backfill behind the walls. The fill should be compacted in loose lifts not greater than 200 millimetres in thickness. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill.
- A compaction surcharge equal to 12 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 (such is typically the case for rigid concrete box or open footing culverts), at-rest earth pressures should be assumed for geotechnical design. The granular fill should be placed in a zone with a width equal to at least 1.2 metres behind the culvert walls (case (a) from commentary on CHBDC Figure C6.20).
- For Case (a), the restrained case, which is typical for culvert walls, the pressures are based on the existing embankment fill materials, assuming a Select Subgrade Material (SSM) is used, and the following parameters (unfactored) may be used:

Soil unit weight: 19 kN/m³

Coefficients of lateral earth pressure:
'At rest' or restrained, K_0 0.53

⁵ Federal Highway Administration (FHWA). (1997). "Design Guidance: Geotechnical Earthquake Engineering For Highways. Volume I – Design Principles." *Geotechnical Engineering Circular No. 3: FHWA-SA-97-076*, Washington, D.C.



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

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

6.6 Construction Considerations

6.6.1 General

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

It is recommended that the footing excavations be carried out such that the final 0.5 metres of excavation is completed with a Quality Verification Engineer (QVE) experienced in geotechnical engineering on site. The prepared excavation bases should be inspected by the QVE and granular base materials or a working slab should be placed immediately after inspection to protect the founding materials. The clayey silt till subgrade is sensitive to disturbance due to the foot traffic during construction and ponded water. If the granular bedding or foundation cannot be placed within 24 hours of inspecting the subgrade then a working slab shall be placed to protect the subgrade. The working slab is to consist of a minimum 100 millimetre thick mat of concrete with a minimum compressive strength of 20 megapascals. A Non-Standard Special Provision (NSSP) should be added to the contract documents to provide working slab requirements. The QVE should assess the foundation conditions to determine if sub-excavation of unsuitable material is required. Sub-excavation and placement and compacted fill should be carried out under the direction of the QVE.



6.6.2 Erosion and Scour Protection

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

6.7 Excavations and Groundwater Control

Excavations will extend through the existing pavement structure and fill materials and into the underlying clayey silt till. It is anticipated that excavation for the culvert replacement will extend below the inferred groundwater level of elevation 222 metres. Some seepage from water perched within the fill materials should be anticipated. Seepage from the clayey silt till subsoil should be minor at the excavation depths proposed. It is considered that any seepage can be controlled by pumping from properly constructed and filtered sumps located at the base of the excavations. Sumps should be maintained outside of the actual foundation limits.

Surficial water seepage into the excavations should be expected and will be heavier during periods of sustained precipitation. Surface water runoff should be directed away from the excavations at all times. The existing culvert flows will need to be diverted/piped during construction.

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

6.8 Staging and Temporary Roadway Protection

It is understood that a single lane is to remain open to traffic during construction with one half of the culvert being replaced at a time. Temporary support systems could consist of soldier piles and lagging or steel sheet piles. Excavation support systems should be designed and constructed in accordance with OPSS 539 and the design should limit the lateral movement of the temporary shoring system to meet Performance Level 2. The contractor is responsible for the complete detailed design of the protection system. While no cobbles or boulders were encountered in the boreholes, the contractor should be prepared for their presence in the clayey silt till.

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



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

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

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

K_a = active coefficient of earth pressure

γ = soil unit weight

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

q = surcharge for traffic and other loading

h_w = height of groundwater level above excavation base/water level to be taken as elevation 222 metres

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

Soil Type	Coefficient of Earth Pressure			Internal Angle of Friction (degrees)	Unit Weight (kN/m^3)
	Active, K_a	At Rest, K_o	Passive, K_p		
Fill	0.36	0.53	2.8	28	19
Clayey Silt Till	0.31	0.47	3.3	32	22

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



7.0 MISCELLANEOUS

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

GOLDER ASSOCIATES LTD.

ORIGINAL SIGNED

ORIGINAL SIGNED

Dirka U. Prout, P.Eng.

W.M. Kellestine, P.Eng.
Principal

ORIGINAL SIGNED

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

BT/DUP/WMK/FJH/cr

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n:\active\2012\1132 - geo\1132-0100\12-1132-0163 stantec-fdns-mega culverts-3011-e-0041\ph 2000-gwp 3040-11-00\rpts\r05 - site 14-439 (archer)\1211320163-2000-r05 sept 12 14 (final) part a&b fdns repl clvrt 14-439-c (archer).docx

TABLE I

COMPARISON OF STRUCTURE ALTERNATIVES FOR REPLACEMENT CULVERT

Archer Municipal Drain, Site 14-439/C, Station 12+231, Highway 21
 Structure Replacements and Rehabilitation
 GWP 3040-11-00

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

COMPARISON OF STRUCTURE ALTERNATIVES FOR REPLACEMENT CULVERT

FOUNDATION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	ESTIMATED COSTS	RISKS/ CONSEQUENCES
		specific requirements. • Can be constructed with far fewer joints than a precast box culvert.	requirements for cold weather work.		
CIP open footing culvert with spread footings founded on stiff to very stiff clayey silt till	• Feasible	• Higher costs compared to box culvert options. • Suitable for corrosive environments. • Culvert design can be customized in the field for high stress or load conditions or other site-specific requirements. • Can be constructed with far fewer joints than a precast box culvert. • Most suitable where maintenance of fish and/or wildlife passage and preservation of the natural stream bed is a priority.	• More expensive compared to both box culvert options due to increased labour associated with concrete formwork and deeper excavation required to provide frost protection for footings. • Footings may be susceptible to scour and undermining especially at entrance.	• Moderate	• Relatively low to moderate risk • Improper alignment or transition between stream and culvert may lead to problems with scour • Option with foundations most susceptible to damage by scour

NOTES:

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

Prepared By: DUP
 Checked By: WMK

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Consistency

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

(b) Cohesive Soils

Dynamic Cone Penetration Resistance; N_d :

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

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

IV. SOIL TESTS

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

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

LIST OF SYMBOLS

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

I. General

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

METRIC

PROJECT 12-1132-0163

W.P. 3040-11-00

LOCATION N 4763622.2 , E 345304.9

ORIGINATED BY BT

DIST _____ HWY 21BOREHOLE TYPE POWER AUGER, HOLLOW STEM

COMPILED BY AMG/LMK

DATUM GEODETIC

DATE July 4, 2013

CHECKED BY _____

[illegible]

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

RECORD OF BOREHOLE No 208

1 OF 1

METRIC

PROJECT 12-1132-0163
W.P. 3040-11-00 LOCATION N 4763639.4 , E 345291.3 ORIGINATED BY BT
DIST HWY 21 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY AMG/LMK
DATUM GEODETIC DATE July 4, 2013 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
225.49	GROUND SURFACE															
0.00	TOPSOIL, silty Brown															
0.24	FILL, clayey silt, trace to some sand, trace gravel, trace topsoil Firm to stiff		1	SS	12											
			2	SS	6											
223.36																
2.13	CLAYEY SILT TILL, some sand, trace gravel Stiff to very stiff Brown becoming grey at about elev. 220.3m		3	SS	16											0 14 54 32
			4	SS	19											
			5	SS	23											1 13 51 35
			6	SS	23											
			7	SS	11											1 13 50 36
			8	SS	11											
			9	SS	9											
215.74	END OF BOREHOLE															
9.75	Groundwater encountered at about elev. 216.5m during dirlling on July 4, 2013. Water level measured at elev. 220.43m on September 5, 2013. Water level measured at elev. 222.02m on July 25, 2014.															

PROJECT 12-1132-0163

W.P. 3040-11-00

LOCATION N 4763627.6 , E 345292.2

ORIGINATED BY BT

DIST HWY 21

BOREHOLE TYPE POWER AUGER, HOLLOW STEM

COMPILED BY AMG/LMK

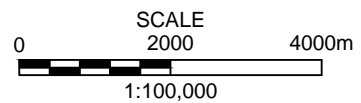
DATUM GEODETIC

DATE July 4, 2013

CHECKED BY _____

[illegible]

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



REFERENCE

PLAN BASED ON CANMAP STREETFILES V.2008.5.

NOTE

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

PROJECT

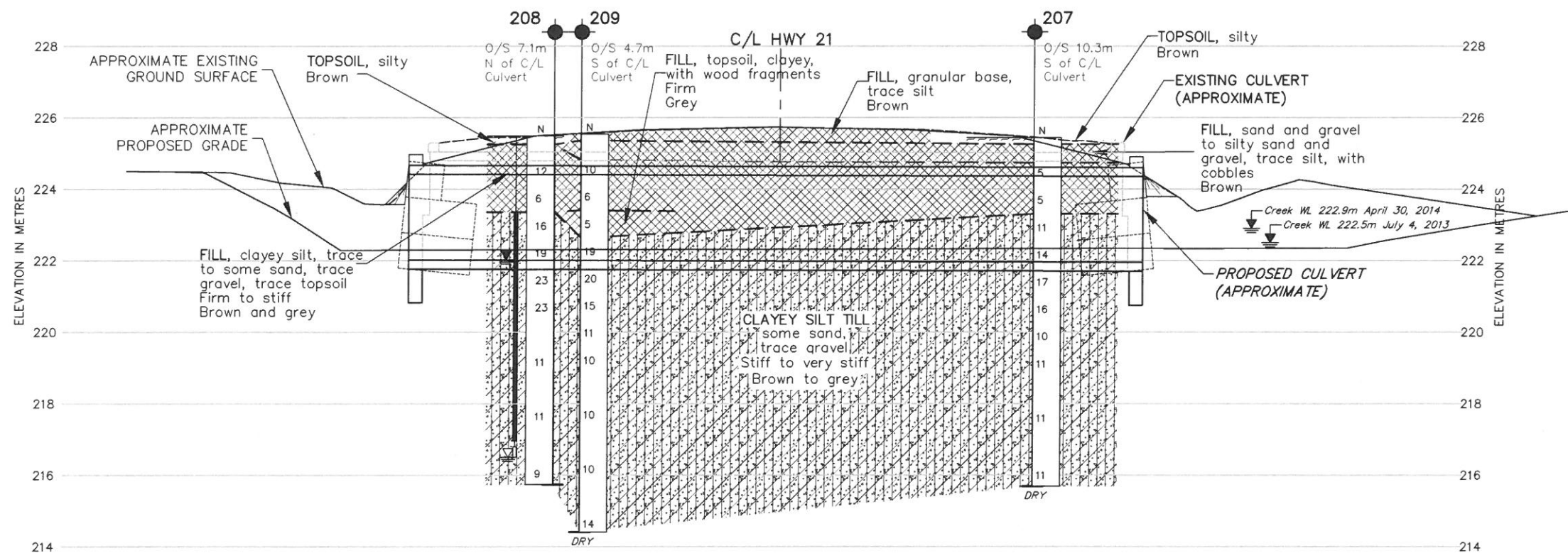
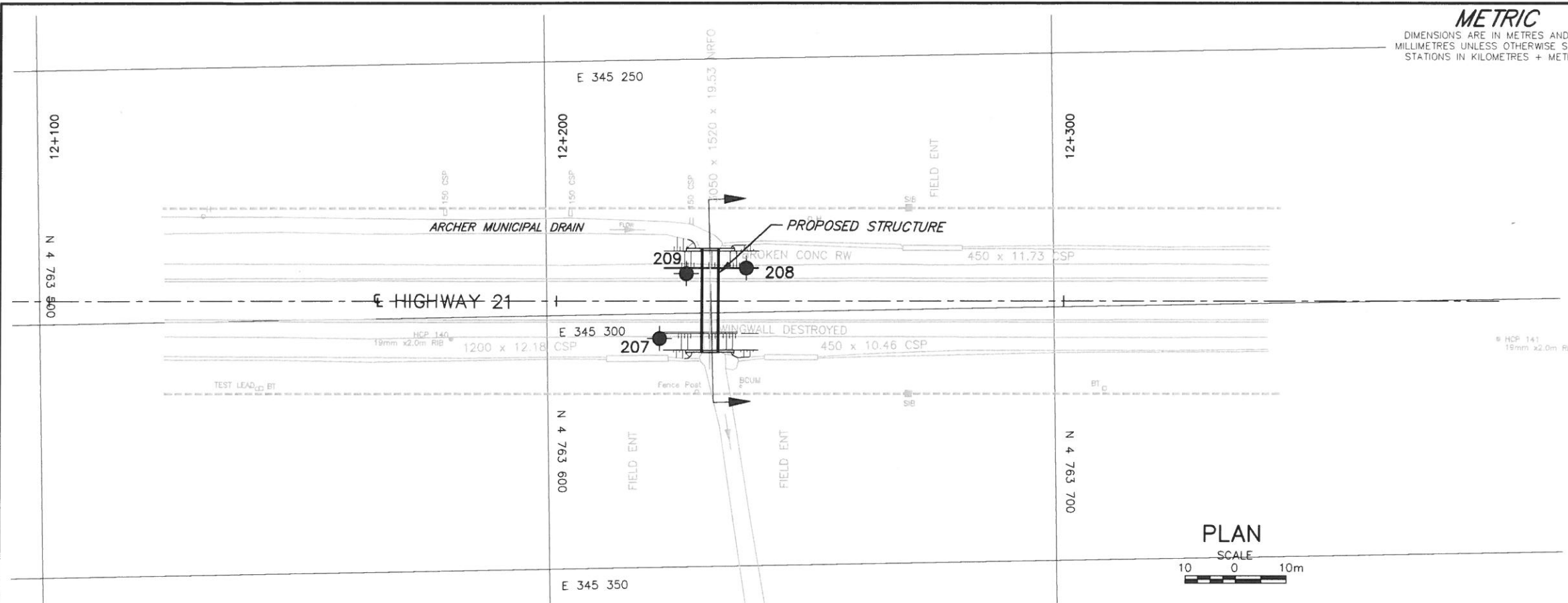
ARCHER MUNICIPAL DRAIN, SITE No. 14-439/C
STATION 12+231, HIGHWAY 21
GWP 3040-11-00

TITLE

KEY PLAN



PROJECT No. 12-1132-0163			FILE No. 1211320163-2000-F05001	
CADD	AMG/LMK	Nov. 18/13	SCALE AS SHOWN	REV. 0
CHECK			FIGURE 1	

CONT No.
WP No. 3040-11-00ARCHER MUNICIPAL DRAIN
STATION 12+231, HIGHWAY 21
STRUCTURE REPLACEMENTS AND REHABILITATION
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

**Golder Associates Ltd.**
LONDON, ONTARIO, CANADA

KEY PLAN

LEGEND

- Borehole - Current Investigation
- Seal
- Standpipe
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL measured on July 25, 2014
- WL encountered during drilling
- DRY Water level not established

No.	ELEVATION	CO-ORDINATES (MTM NAD83 ZONE 11)	
		NORTHING	EASTING
207	225.44	4 763 622.2	345 304.9
208	225.49	4 763 639.4	345 291.3
209	225.54	4 763 627.6	345 292.2

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

Drawings provided by Stantec.

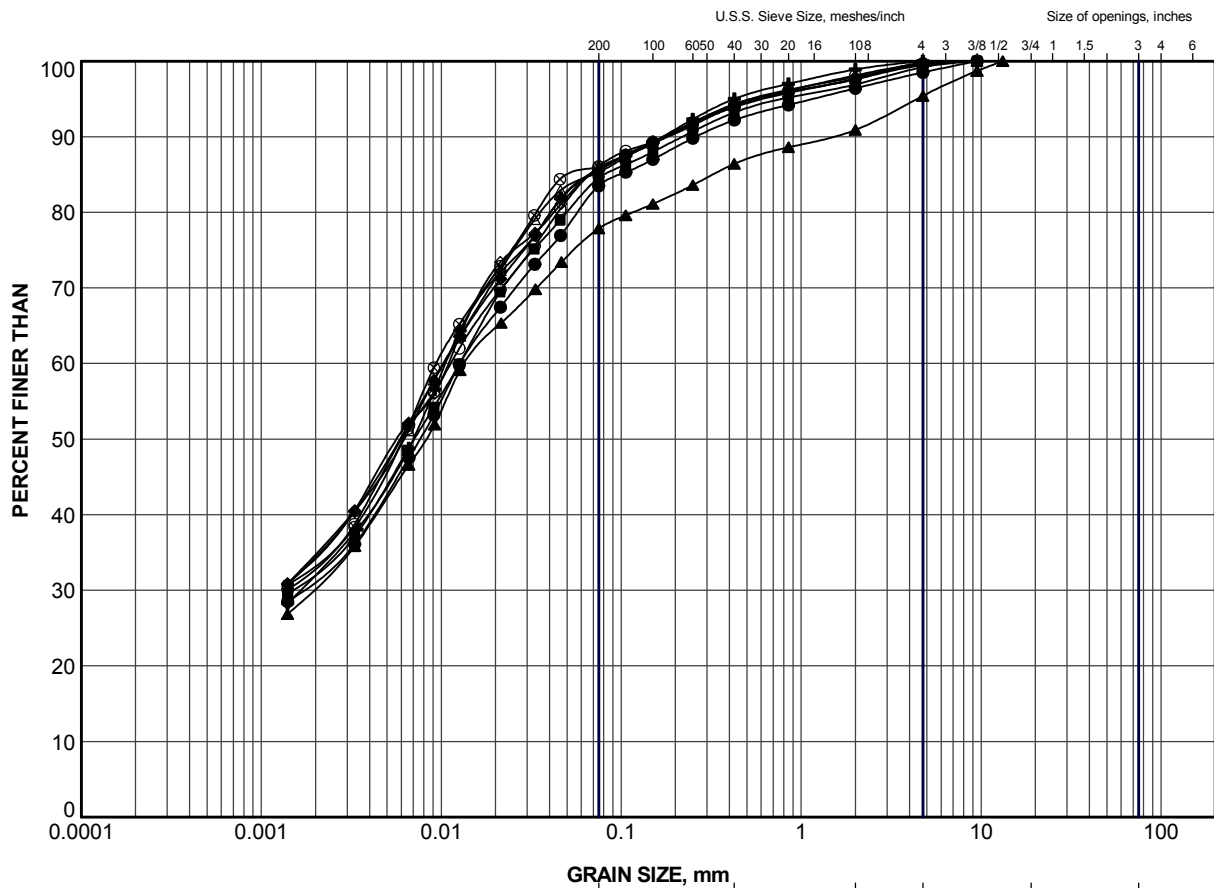


NO.	DATE	BY	REVISION
Geocres No. 4001-13			
HWY.	21	PROJECT NO.	12-1132-0163
SUBM'D.	BT	CHKD.	DUP
DRAWN:	WDF	CHKD.	WMK
DATE:	July 28/14	APPD.	FJH
SITE:	14-439/C	DWG.	1



APPENDIX A


Laboratory Test Data

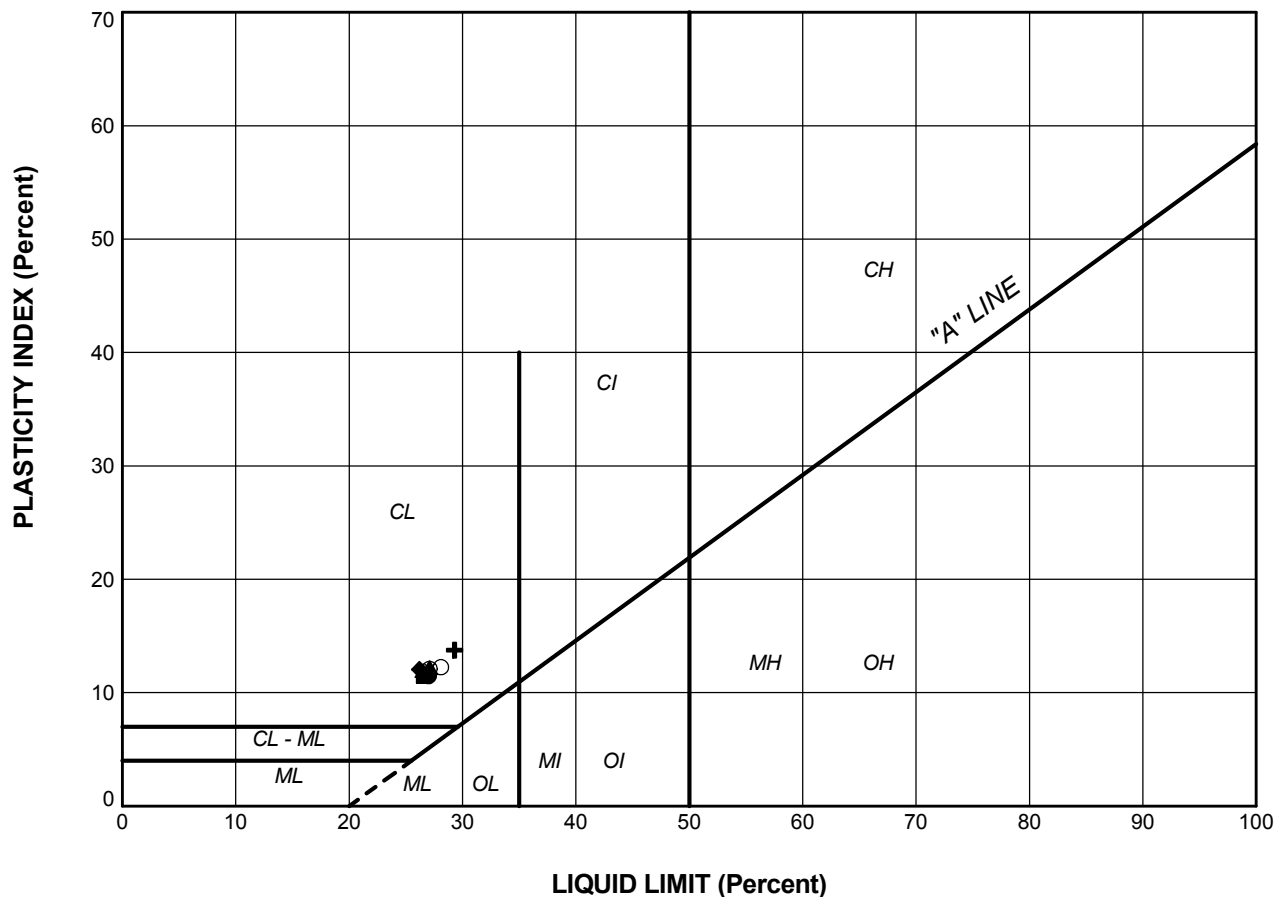


CLAY AND SILT	GRAVEL SIZE, mm						Cobble Size
	fine	medium	coarse	fine	coarse		
	SAND SIZE			GRAVEL SIZE			

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	207	4	222.2
■	207	6	220.6
▲	207	7	219.9
+	208	3	223.0
◆	208	5	221.5
◇	208	7	219.2
○	209	4	222.3
△	209	6	220.7
⊗	209	7	220.0

PROJECT				ARCHER MUNICIPAL DRAIN, SITE No. 14-439/C STATION 12+231, HIGHWAY 21 GWP 3040-11-00			
TITLE				GRAIN SIZE DISTRIBUTION CLAYEY SILT TILL			
PROJECT No.		12-1132-0163		FILE No. 1211320163-2000-F050A1			
DRAWN		LMK		Apr. 14/14		SCALE N/A REV.	
CHECK						FIGURE A-1	
 Golder Associates LONDON, ONTARIO							



LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
CLAYEY SILT TILL					
●	207	4	27.0	15.5	11.5
■	207	6	26.5	15.1	11.4
▲	207	7	27.1	14.9	12.2
+	208	3	29.3	15.6	13.8
◆	208	5	26.2	14.2	12.1
◇	208	7	26.9	15.2	11.7
○	209	4	28.1	15.9	12.3
△	209	6	26.5	14.6	12.0
⊗	209	7	27.1	15.0	12.1

PROJECT ARCHER MUNICIPAL DRAIN, SITE No. 14-439/C
 STATION 12+231, HIGHWAY 21
 GWP 3040-11-00

TITLE

PLASTICITY CHART



PROJECT No.	12-1132-0163	FILE No. 1211320163-2000-F050A2
DRAWN	LMK	Nov 18/13
CHECK		
SCALE	N/A	REV.

FIGURE A-2



APPENDIX B

Site Photographs



APPENDIX B PHOTOGRAPHS



Photograph 1: West elevation (inlet) of Culvert Site 14-439/C.



Photograph 2: East elevation (outlet).



APPENDIX B PHOTOGRAPHS



Photograph 3: Highway 21 looking north from east shoulder at Culvert Site 14-439/C.

n:\active\2012\1132 - geo\1132-0100\12-1132-0163 stantec-fdns-mega culverts-3011-e-0041\ph 2000-gwp 3040-11-00\rpts\r05 - site 14-439 (archer)\1211320163-2000-r05 sept 12 14
(final) app b - photos.docx

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Africa	+ 27 11 254 4800
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Europe	+ 356 21 42 30 20
North America	+ 1 800 275 3281
South America	+ 55 21 3095 9500

solutions@golder.com
www.golder.com

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

