REPORT

February 2017

FOUNDATION INVESTIGATION AND DESIGN REPORT

Culvert Replacement, Culvert at 7th Line
Site No. 4-320/C, Highway 89
Contract 4 Structure Replacements and Rehabilitation
GWP 3042-11-00
Ministry of Transportation, West Region

Submitted to:

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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 3042-11-00. The project involves the detailed 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 7th Line (Site 4-320/C) at about Station 23+675 on Highway 89 in the Geographic Township of Mulmur in Dufferin County.

The purpose of the foundation investigation is to explore the subsurface conditions at the location of the proposed culvert replacement by drilling boreholes and carrying out in situ testing and laboratory testing on selected samples. The terms of reference for the scope of work are outlined in the MTO's Request for Proposal and in Golder Associates' proposal 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 about Station 23+675 on Highway 89, approximately 10 metres west of 7th Line in the Township of Mulmur in Dufferin County, Ontario. The Town of New Tecumseth is approximately 11.0 kilometres east of the site. The replacement culvert will be constructed about 5 metres west of the existing culvert. The location of the culvert is shown on the Key Plan, Figure 1.

This section of Highway 89 is currently a two lane, undivided highway with gravel shoulders. It is generally oriented east-west in the vicinity of the subject site. An unnamed watercourse flows in the culvert from south to north beneath Highway 89. The existing culvert has an unknown date of construction and has since been extended to the north and to the south. The dates of the extensions are also unknown. The original structure is a concrete non rigid frame, open footing (NRFO) structure and the extensions are concrete, rigid frame, open footing (RFO) structures. The south and north extensions are 8.85 and 9.30 metres long, respectively.

Dimensions (m)	Obvert Elevation (m)		Construction
	Lt[1]	Rt[1]	
3.05 x 1.53 x 26.83	352.81	352.79	Concrete RFO/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 watercourse and the embankments along Highway 89 near the culvert are grass covered. Sand bags have been placed at the inlet of the culvert. The watercourse flows through fields on the south side of Highway 89 and along 7th Line on the north side of Highway 89. Site photographs are provided in Appendix B.

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 which are composed mostly of till and with some sand and gravel deposits as well as sand and gravel terraces and swampy valley floors.[1] The overburden in the area of the site generally consists of sandy silty till with some pebbly silty sandy till.[2]

The geological mapping indicates that the underlying bedrock consists of shale, limestone, dolostone and siltstone of the Queenston Formation of Upper Ordovician age.[3] The bedrock surface at the site is at about elevation 345 metres, with the overburden thickness being about 7 to 10 metres.[4]

[1] Chapman, L.J., and Putnam, D.F., 1984: Physiography of Southern Ontario; Ontario Geological Survey, Special Volume 2, 270p.

[2] Gwyn, Q.H.J. 1972: The Quaternary geology of the Dundalk area, Southern Ontario; Ontario Department of Mines and Northern Affairs, Prelim. Map P.727, Geol. Ser., scale 1:50,000. Geology 1971.

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

[4] Gwyn, Q.H.J., and Frazer, J.Z. 1975: Bedrock Topography of the Dundalk Area, Southern Ontario; Ontario Div. Mines, Prelim. Map P.306 (Revised), Bedrock Topography Ser., Scale 1:50,000. Geological compilation, 1975.

3.0 INVESTIGATION PROCEDURES

The field work for the investigation was carried out on May 30, 2016, during which time three boreholes were drilled at the approximate locations shown on the Borehole Location Plan, Drawing 1.

The boreholes were drilled using buggy-mounted CME 55 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 results of the SPT testing, as presented on the Record of Borehole sheets, Drawing 1 and in Section 4.0 of this report, are unmodified (not standardized for hammer efficiency, borehole diameter, rod length, etc.). The samplers used in the investigation limit the maximum particle size that can be sampled and tested to about 40 millimetres. Therefore, particles or objects that may exist within the soils that are larger than this dimension will not be sampled or represented in the grain size distributions. Larger particle sizes, including cobbles and boulders, are known to be present in the glacial tills as discussed in the text of this report.

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

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

Borehole	Location (m)		Ground Surface Elevation	Borehole Depth
	Northing	Easting	(m)	(m)
101	4 886 747	264 022	354.45	9.60
102	4 886 736	264 024	354.43	9.39
103	4 886 730	264 040	352.43	7.71

4.0 SUBSURFACE CONDITIONS

4.1 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the boreholes, together with the results of the in situ testing and the laboratory testing carried out on selected samples, are given on the attached Record of Borehole sheets following the text of this report and in Appendix A. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous samples and observations of drilling resistance and, therefore, may represent transitions between soil types rather than exact planes of geological change. Further, the subsurface conditions will vary between and beyond the borehole locations.

The boreholes drilled at the site generally encountered the existing pavement structure or surficial topsoil overlying embankment fill materials, buried topsoil, sandy silt till, clayey silt till and inferred bedrock.

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

Crushed sand and gravel material, interpreted to be granular pavement materials based on visual and textural examination, was encountered at the ground surface in boreholes 101 and 102, which were drilled on the shoulders of Highway 89. The granular material was 240 and 400 millimetres thick in boreholes 101 and 102, respectively. A 30 millimetre thick layer of buried asphalt was encountered beneath the granular material in borehole 101.

Topsoil was encountered in borehole 103 at the ground surface and was 240 millimetres thick. Buried layers of topsoil were encountered beneath the fill in borehole 101 and within the fill in borehole 102 at elevations 352.3 and 353.1 metres, respectively and were about 0.8 metres 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.

Variable layers of fill were encountered beneath the buried asphalt in borehole 101 at elevation 354.2 metres, beneath the granular pavement material and buried topsoil in borehole 102 at elevation 354.0 and 352.3 metres, respectively, and beneath the topsoil in borehole 103 at elevation 352.2 metres. The thicknesses of the fill ranged from about 0.3 to 1.9 metres. The fill was generally granular in nature and ranged in gradation from sandy silt to sand and gravel. Standard penetration test (SPT) N[5] values ranged from 10 to 37 blows per 0.3 metres and water contents of samples of the fill ranged from 10 to 19 per cent. Grain sizes analyses carried out on samples of sandy silt fill are presented on Figure A-1 in Appendix A.

Compact to very dense sandy silt glacial till was encountered in borehole 101 beneath the buried topsoil at elevation 351.6 metres and in boreholes 102 and 103 beneath the fill at elevations 351.5 and 351.9 metres, respectively. Borehole 102 was terminated in the sandy silt till after exploring it for about 6.5 metres. The sandy silt till was 5.6 metres thick in borehole 101 and 6.8 metres thick in borehole 103. Standard penetration test N

[5] 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 into the soil after having first penetrated 150 millimetres.

values in the sandy silt till ranged from 16 to 78 blows per 0.3 metres with water contents of about 7 to 11 per cent. Grain sizes analyses carried out on samples of sandy silt glacial till are presented on Figure A-2 in Appendix A.

Hard clayey silt glacial till was encountered beneath the sandy silt till in borehole 101 at elevation 345.9 metres. Borehole 101 was terminated in the clayey silt till after exploring it for about 1.1 metres. The clayey silt till had an N value of 39 blows per 0.3 metres.

Cobbles and boulders should be expected in the glacial till strata.

Inferred bedrock was encountered in borehole 103 beneath the sandy silt till at elevation 345.1 metres which was explored for about 0.4 metres.

4.3 Groundwater Conditions

Groundwater conditions were observed during and on completion of drilling and sampling and a groundwater observation piezometer was installed in borehole 103. The installation details are provided on the corresponding Record of Borehole sheet following the text of this report. Groundwater was not encountered in boreholes 101 and 103 during drilling. Borehole 102 encountered groundwater at a depth of 2.7 metres, or at elevation 351.7 metres during drilling on May 30, 2016. On July 6, 2016, the water level in the piezometer installed in borehole 103 was about 1.6 metres below ground surface or at about elevation 350.83 metres. A summary of the encountered and measured groundwater levels is provided in the table below.

Borehole	Ground Surface Elevation (m)	Encountered Groundwater Elevation (m)	Measured Groundwater Level Elevation (m)
			July 6, 2016
101	354.45	-	-
102	354.43	351.7	-
103	352.43	-	350.83

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

Based on the observed groundwater levels, the change in soil colour from brown to grey and the surrounding topography, the groundwater level is inferred to typically be at about elevation 351.0 metres. The groundwater levels are expected to fluctuate seasonally and are expected to be higher during periods of sustained precipitation or during spring snow melt conditions.

5.0 MISCELLANEOUS

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

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PART B

FOUNDATION DESIGN REPORT

CULVERT REPLACEMENT, CULVERT AT 7TH LINE
SITE NO. 4-320/C, HIGHWAY 89
CONTRACT 4 STRUCTURE REPLACEMENTS AND REHABILITATION
GWP 3042-11-00
MINISTRY OF TRANSPORTATION - WEST REGION

6.0 ENGINEERING RECOMMENDATIONS

This section of the report provides recommendations on the foundation aspects of the design of the proposed culvert replacement at Site 4-320/C at Station 23+675 on Highway 89, adjacent to 7th Line, in the Geographic Township of Mulmur in Dufferin County, Ontario.

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

The existing culvert is a 26.8 metres long concrete structure with a 3.05 metres span and a 1.53 metres high opening with an approximate invert elevation of 351.1 metres. The original structure is a non-rigid frame, open footing (NRFO) with an unknown date of construction. The structure was subsequently extended 8.85 metres to the south and 9.30 metres to the north. The extensions are rigid frame open footing (RFO) structures. Based on the information provided by Stantec, the replacement culvert will be a 29.3 metres long precast concrete box culvert with a 3.00 metres span and a 1.80 metres high opening with an approximate invert elevation of 350.9 metres. The replacement culvert will be installed at about Station 23+670.

6.1 Foundations

The founding soil is expected to consist of sandy silt till at the approximate founding elevation at or below 350.5 metres. The culvert foundations may be designed using a factored geotechnical resistance at Ultimate Limit States (ULS) of 450 kilopascals (kPa) and a geotechnical reaction at Serviceability Limit States (SLS) of 300 kPa. The SLS value corresponds to 25 millimetres of settlement. The new precast concrete box culvert should be constructed on a 300 millimetre thick base of compacted Granular A. A 75 millimetre thick levelling pad of Granular A or fine aggregates should also be provided.

If an open footing culvert is considered, the appropriate founding elevation is about 349.5 metres. Footings constructed at this elevation can be designed using the above-noted geotechnical resistances. If the footings cannot be poured promptly following excavation and inspection, a 100 millimetre thick working slab of lean concrete should be provided to protect the integrity of the founding soils.

6.1.1 Backfill

Backfill for the culvert should consist of free-draining, non-frost susceptible granular materials such as OPSS Granular B, Type II or III or Granular A placed in 0.3 metre thick loose lifts and uniformly compacted. Heavy compaction equipment should not be used immediately adjacent to the walls and roof of the culvert. The height of backfill adjacent to the culvert walls should be maintained as equal as possible on both sides of the culvert during all stages of backfill placement.

6.1.2 Frost and Scour Protection

Frost treatment in the form of a frost taper symmetrical about the culvert centreline should be provided in accordance with OPSD 803.010. The design frost penetration depth for this area is 1.4 metres below ground surface. The culvert should also be adequately protected against scour as noted in Section 1.9.5 of the Canadian Highway Bridge Design Code (CHBDC).

6.1.3 Resistance to Lateral Forces/Sliding Resistance

The resistance to lateral forces/sliding resistance between the base of the culvert and the bedding or the footings and the founding soils should be calculated in accordance with Section 6.7.5 of the CHBDC. For box culverts, cut-off walls should be provided in accordance with Clause 1.9.5.6 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:

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

where:

A'	-	effective contact area, square metres
c'	=	Nil
$\tan\delta$	-	coefficient of friction for interface between box culvert base and bedding/levelling pad or footings and the founding soils
V	-	unfactored vertical force, kilonewtons
H_f	-	unfactored horizontal load, kilonewtons

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

Structure	Interaction	Angle of Friction, δ (degrees)	Coefficient of Friction, tan δ
Precast Box Culvert	Precast concrete on Granular A bedding/levelling pad	30	0.58
Open Footing Culvert	Poured concrete on sandy silt till	32	0.62

6.1.4 Other Construction Considerations

The excavation base should be free of debris, loose or frozen material and ponded water. The cleaned excavation base should be inspected by the geotechnical engineer prior to placement of the granular bedding materials or pouring concrete for footings.

Erosion protection for the culvert backfill should be provided to protect the roadway, approach embankments and culvert, as appropriate. Consideration could be given to using suitable non-woven geotextile and rip rap, as required, to provide erosion protection based on hydraulic requirements. Temporary erosion protection and sedimentation control measures should be implemented in accordance with OPSS 805. Rip-rap treatment at the culvert outlet should be provided in accordance with OPSD 810.010. In addition, sediment control such as silt fences and erosion control blankets may be required during construction together with diversion of any flows to mitigate migration of fine soil particles.

6.2 Excavations and Groundwater Control

Excavations will extend through the existing fill and topsoil into the underlying native sandy silt till. Seepage volumes from the sandy silt till are anticipated to be such that groundwater control may be achieved by using properly constructed and filtered sumps in the base of the excavation. Sumps should be maintained outside of the actual wall footing limits. Some seepage from the granular fill layers should be expected particularly during and following 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.

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 sandy silt till would be classified a Type 1 soil above the groundwater level and may be classified as Type 3 if water seepage occurs or below the groundwater level.

6.3 Gabion Retaining Walls

Based on the information provided to Golder, the proposed replacement culvert may feature gabion walls. Stantec has indicated that the gabion walls may be up to 2 metres in height and could be located in each quadrant, although the most probable location for them would be in the northeast quadrant. From a geotechnical perspective, armour stone walls, concrete cantilever walls, concrete gravity walls, including precast toe walls, and reinforced soil system wall (RSS) are suitable alternatives. Considering wall heights of less than 2 metres, gabion walls, armour stone walls and precast concrete toe walls are more economical than RSS walls and concrete cantilever or other concrete gravity walls. Gabion walls, armour stone walls and RSS walls need not be founded at the frost depth and are the wall types most tolerant of movement. Although somewhat labour intensive to construct, installation of gabion walls will be more economical, rapid, and require less excavation and disruption to traffic than most other wall types. Gabion walls can be supported on the compact to very dense sandy silt till at or below elevation 351.5 metres. A factored geotechnical resistance of 275 kPa at ULS and a geotechnical reaction of 175 kPa at SLS may be used for design of the gabion wall footings. The SLS value corresponds to 25 millimetres of total settlement. If required, a granular levelling course approximately 75 millimetres in thickness can be placed on the founding strata for the gabion walls.

6.3.1 Frost Protection and Embedment

The frost depth applicable to this site is 1.4 metres. Gabion walls do not require an embedment depth equivalent to the frost depth provided they are founded on granular pads with a compacted thickness of 300 millimetres. In addition, the gabion walls should have sufficient embedment to provide stability and adequate protection against scour and erosion.

6.3.2 Lateral Resistance

The resistance to lateral forces/sliding resistance between the underside of the gabion baskets and levelling pads or subgrade soil, as applicable, should be calculated in accordance with Section 6.7.5 of the CHBDC. Also, the retaining walls shall be checked for overturning. The following angles of friction and corresponding unfactored coefficient of friction, tan ϕ', may be used for the interaction between the gabion baskets and the founding soil:

Subgrade Material	Effective Angle of Friction, ϕ' (degrees)	tan (ϕ')	K_p
Sandy silt till	28	0.53	2.8
Granular A levelling pad	30	0.58	3.0

In accordance with the CHBDC, a factor of 0.8 is to be applied in calculating the horizontal resistance. The factored horizontal geotechnical resistance, H_{rs}, should be based on CHBDC 6.7.5 as follows:

$$H_{rs} = 0.8A'c' + 0.8V\tan\phi' > H_f$$

Where:

A'- effective contact area, square metres
c' = Nil
δ = angle of interface friction
V - unfactored vertical force, kilonewtons
H_f - factored horizontal load, kilonewtons

The unfactored coefficients of passive pressure, K_p, for the portion of the retaining walls below the ground surface are given in the above table using unfactored effective angles of internal friction, ϕ'.

6.3.3 Other Design Considerations

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

6.4 Liquefaction Potential and Seismic Analysis

6.4.1 Seismic Parameters

The site is located near the towns of Rosemont, Shelburne and New Tecumseth/Alliston in Ontario. According to Table A.3.1.1 of the CHBDC (version S6-06), the zonal acceleration ratio, A, for New Tecumseth/Alliston is 0.05, which is considered applicable to this site. The zonal acceleration ratio for Rosemont and Shelburne were not available. The corresponding acceleration related seismic zone, Z_a, is 1. Based on the site stratigraphy, the soil profile type is categorized as Type I with a seismic site response coefficient, S, of 1.0 based on the CHBDC criteria.

The importance category of the replacement culvert is "other" based on the 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 procedures[6] and Canadian Foundation Engineering Manual (CFEM). The potential for liquefaction occurring at this site is very low due to historically low seismicity in this area, founding soils with a normalized SPT $(N_1)_{60}$ generally greater than 26 blows per 0.3 metres, and the relatively shallow depth to bedrock. Therefore, a detailed evaluation of the liquefaction potential of the foundation soils is not considered warranted.

6.5 Lateral Earth Pressures for Design

The lateral pressures acting on the proposed culvert 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 CHBDC (version S6-06). It should be noted that these design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Backfill should be placed in accordance with Section 6.1.1 above.

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

- A compaction surcharge equal to 12 kilopascals should be included in the lateral earth pressures for the structural design in accordance with CHBDC Figure C6.6.
- If the wall support does not allow lateral yielding (such is typically the case for a rigid concrete box culvert), at-rest earth pressures should be assumed for geotechnical design. The granular fill should be placed in a zone with a width equal to at least 1.4 metres behind the culvert walls (Case (a) from commentary on CHBDC Figure C6.20).
- For Case (a), the restrained case, the pressures are based on the existing embankment fill materials, assuming a Select Subgrade Material (SSM) is used, and the following parameters (unfactored) may be used:

Soil unit weight:	19 kN/m³
Coefficients of lateral earth pressure:	
'At rest' or restrained, K_o	0.53

- If the wall support allows lateral yielding (unrestrained structure, such as typically the case for retaining walls), active earth pressures may be used in the geotechnical design of the structure. The granular fill should be placed in a wedged shaped zone with a width equal to at least 1.4 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	GRANULAR B TYPE III
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	0.31	0.27
'passive', K_p	3.7	3.3	3.7

6.6 Temporary Roadway Protection

It is understood that temporary roadway protection is required should a single lane of traffic need to be maintained on Highway 89 at the culvert location during construction. Temporary support systems could consist of cantilevered soldier piles and lagging or steel sheet piles. Installation of steel sheets into the sandy silt till may not be feasible.

Excavation support systems should be designed and constructed in accordance with OPSS 539 and the design should limit the lateral movement of the temporary shoring system to meet Performance Level 2. The contractor is responsible for the complete detailed design of the protection system.

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

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

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

where
- H = the height of the excavation at any point in metres
- K_a = active coefficient of earth pressure
- γ = soil unit weight
- γ_w = unit weight of water 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 351.0 metres

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

Soil Type	Coefficient of Earth Pressure			Internal Angle of Friction (degrees)	Bulk Unit Weight γ (kN/m^3)	Effective Unit Weight γ' (kN/m^3)
	Active, K_a	At Rest, K_o	Passive, K_p			
Fill	0.38	0.55	Nil	-	19	9.0
Topsoil	0.38	0.55	Nil	-	16	6.0
Sandy Silt Till	0.31	0.47	3.3	32	21	11.0

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

6.7 Construction Considerations

Care should be taken during construction to avoid disturbance of the subgrades prior to constructing foundations or placing bedding. All existing fill and any topsoil, organics, and soft or loose soils should be stripped from the proposed founding areas prior to placement of the bedding 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 base should be inspected by the QVE to ensure that the sandy silt till has been reached and granular base materials or working mat should be placed immediately after inspection to protect the founding materials. The QVE should assess the foundation conditions to determine if sub-excavation of unsuitable material is required. Sub-excavation, placement and compaction of fill should be carried out under the direction of the QVE.

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

7.0 MISCELLANEOUS

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

GOLDER ASSOCIATES LTD.

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Feb 23/17
PROVINCE OF ONTARIO

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

DH/DUP/MEB/FJH/cr

n:\active\2012\1132 - geo\1132-0100\12-1132-0163 stantec-fdns-mega culverts-3011-e-0041\ph 4000-gwp 3042-11-00\rpts\r01 4-320c 7th line\1211320163-4000-r01 feb 22 17 (final) part a&b fdns replace clvrt 4-320.docx

LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\,\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	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

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(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 δ
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

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d:

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 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) Non-Cohesive (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	**psf**
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

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

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

V. MINOR SOIL CONSTITUENTS

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

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 101

METRIC

PROJECT 12-1132-0163

W.P. 3042-11-00 | LOCATION N 4886746.6 , E 264022.4 | ORIGINATED BY DH

DIST ___ HWY 89 | BOREHOLE TYPE POWER AUGER, HOLLOW STEM | COMPILED BY LMK

DATUM GEODETIC | DATE May 30, 2016 | CHECKED BY ___

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
354.45	GROUND SURFACE						
0.00	FILL, sand and gravel, crushed Brown						
0.27	ASPHALT				354		
353.69 0.76	FILL, sand and gravel, some silt Brown						
	FILL, sandy silt, some clay, some gravel Compact to dense Brown	1	SS	13		o (~10)	10 37 37 16
		2	SS	37	353	o	
352.32 2.13	TOPSOIL, silty Very loose Brown	3	SS	3	352		
351.55 2.90	SANDY SILT TILL, some clay, trace to some gravel Compact to very dense Brown to grey at about elev. 349.3m	4	SS	20	351		
		5	SS	27		o (~10)	9 28 39 24
		6	SS	78	350		
		7	SS	44	349		
		8	SS	46	348	o (~10)	7 24 42 27
		9	SS	54	347		
345.92 8.53	CLAYEY SILT TILL, trace gravel Hard Grey				346		
		10	SS	39	345		
344.85 9.60	END OF BOREHOLE Borehole dry during driling on May 30, 2016.						

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; 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   O $^{3\%}$ STRAIN AT FAILURE

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RECORD OF BOREHOLE No 102

METRIC

PROJECT 12-1132-0163

W.P. 3042-11-00 | LOCATION N 4886736.4 , E 264024.2 | ORIGINATED BY DH

DIST ______ HWY 89 | BOREHOLE TYPE POWER AUGER, HOLLOW STEM | COMPILED BY LMK

DATUM GEODETIC | DATE May 30, 2016 | CHECKED BY ______

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
354.43	GROUND SURFACE										
0.00	FILL, sand and gravel, crushed Brown										
354.03 0.40	FILL, sandy silt, some gravel, with cobbles Brown										
353.67 0.76	FILL, sandy silt, some clay, trace gravel, with topsoil Compact Brown	1	SS	12		o		5	34	42	19
353.06 1.37	TOPSOIL, silty Loose Brown	2	SS	8							
352.30 2.13	FILL, sandy silt, some clay, some gravel, with topsoil and silty sand layers Compact Brown	3	SS	10	∇	o					
351.53 2.90	SANDY SILT TILL, trace to some clay, trace to some gravel Compact to very dense Brown to grey at about elev. 350.8m	4	SS	16							
		5	SS	45							
		6	SS	46		o ⊢⊣		10	28	39	23
		7	SS	49		o					
		8	SS	40		o					
		9	SS	39							
345.04		10	SS	80/ 100mm							
9.39	END OF BOREHOLE Groundwater encountered at about elev. 351.7m during drilling on May 30, 2016.										

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

ELEVATION SCALE: 354, 353, 352, 351, 350, 349, 348, 347, 346

REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

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RECORD OF BOREHOLE No 103

METRIC

PROJECT 12-1132-0163

W.P. 3042-11-00 LOCATION N 4886729.9 , E 264040.3 ORIGINATED BY DH

DIST HWY 89 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY LMK

DATUM GEODETIC DATE May 30, 2016 CHECKED BY

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
352.43	GROUND SURFACE										
0.00	TOPSOIL, silty Brown										
0.24 / 351.88	FILL, sand, some gravel, trace organics Brown					352					
0.55	SANDY SILT TILL, some clay, trace to some gravel Compact to very dense Brown to grey at about elev. 351.1m	1	SS	17	Backfill						
		2	SS	41		351	○	12	29	37	22
		3	SS	32		350					
		4	SS	43		349					
		5	SS	47	Piezometer	348					
		6	SS	40			○	7	43	34	16
		7	SS	83		347					
		8	SS	40		346					
345.11 / 7.32	BEDROCK, shale (inferred) Red					345					
344.72 / 7.71	END OF BOREHOLE	9	SS	100/ 76mm							

Auger refusal on inferred bedrock

Borehole dry during driling on May 30, 2016.

Piezometer dry after drilling on May 30, 2016.

Water level measured in piezometer at elev. 350.83m on July 6, 2016.

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

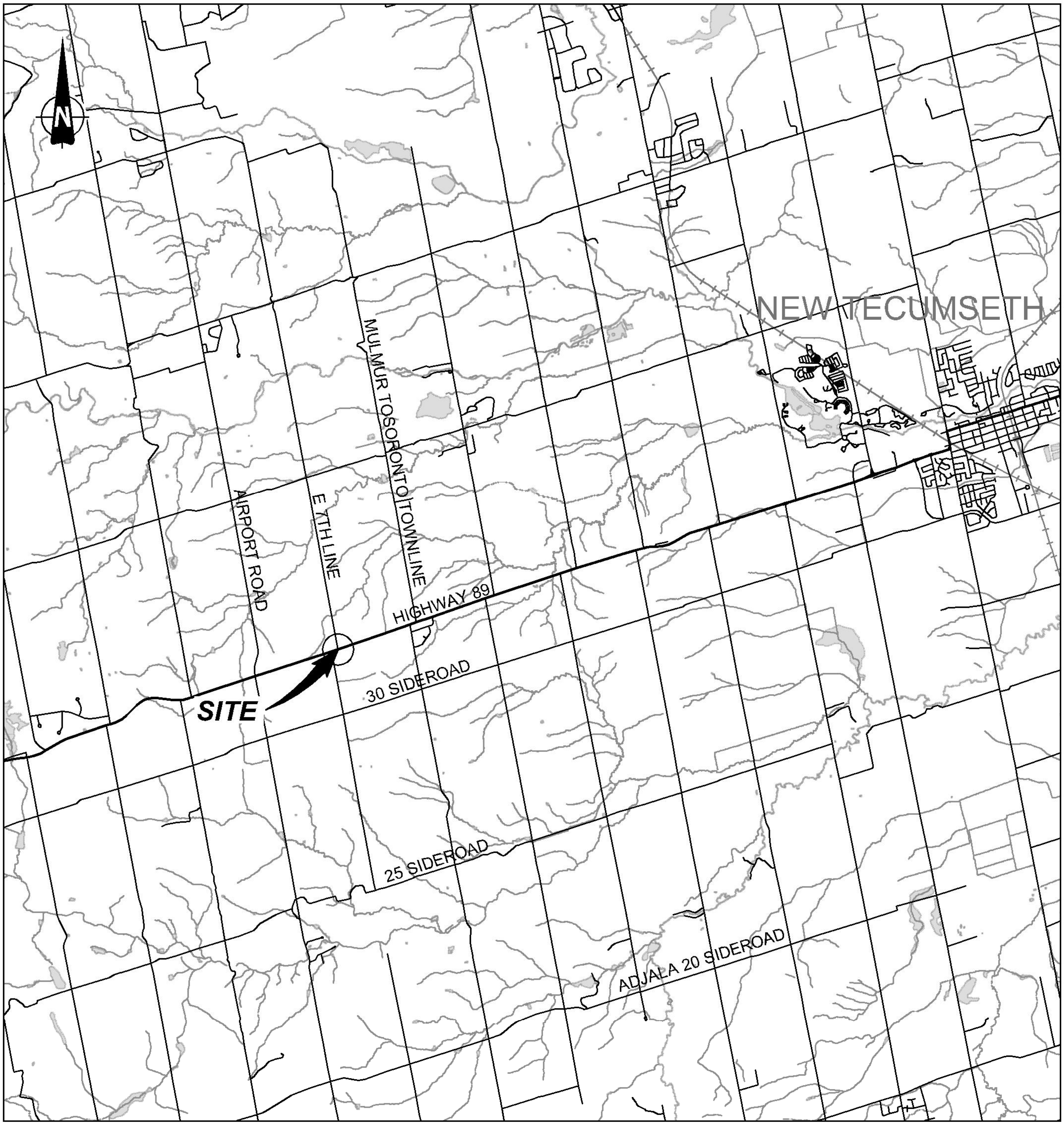
SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100

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 (%)

$+^3, \times^3$: 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 AT 7TH LINE REPLACEMENT, SITE NO. 4-320/C
HIGHWAY 89
GWP 3042-11-00

TITLE
KEY PLAN

Golder Associates

PROJECT No.		12-1132-0163	FILE No.	1211320163-4000-F01001
			SCALE AS SHOWN	REV. 0
CADD	LMK	July 13/16		
CHECK			**FIGURE 1**	

Drawing file: 1211320163-4000-F01001.dwg Dec 08, 2016 - 8:38am

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

CONT No.
WP No. 3042–11–00

SHEET

CULVERT AT 7TH LINE

STRUCTURE REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

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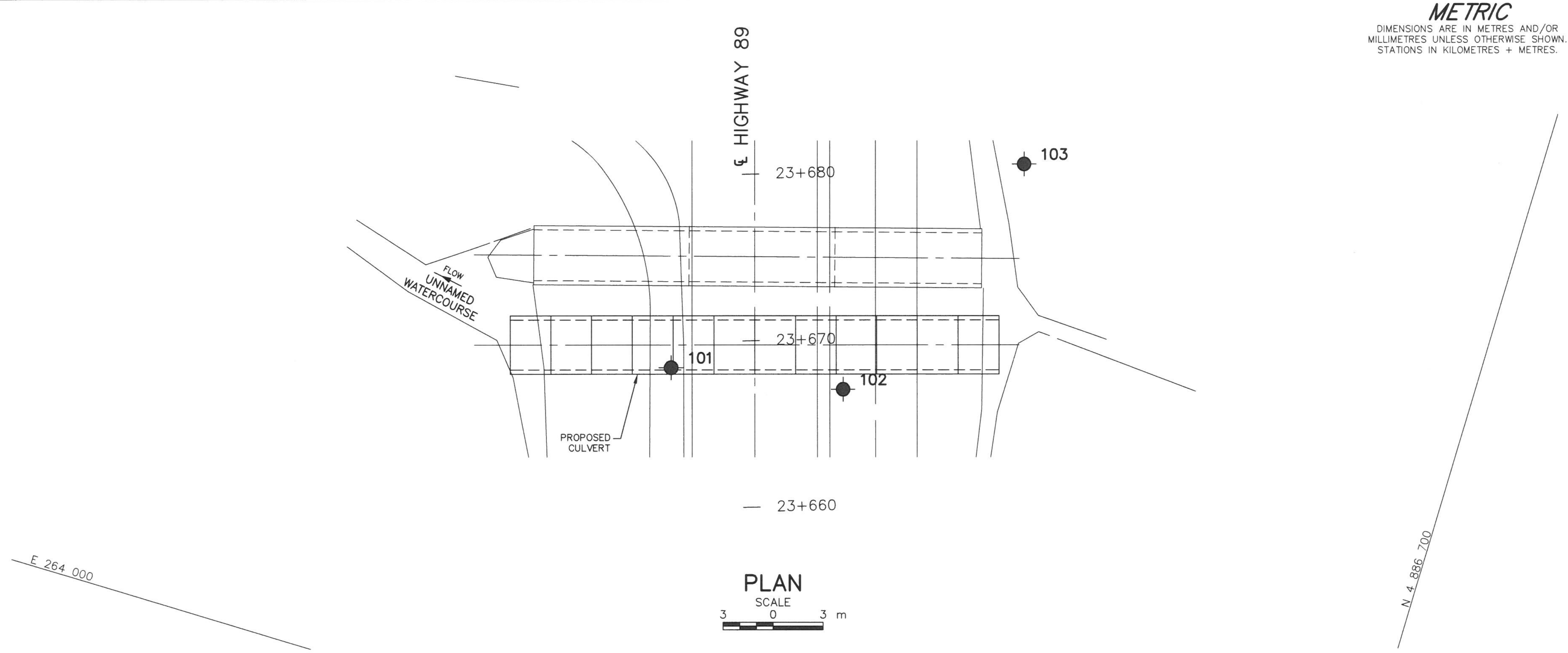
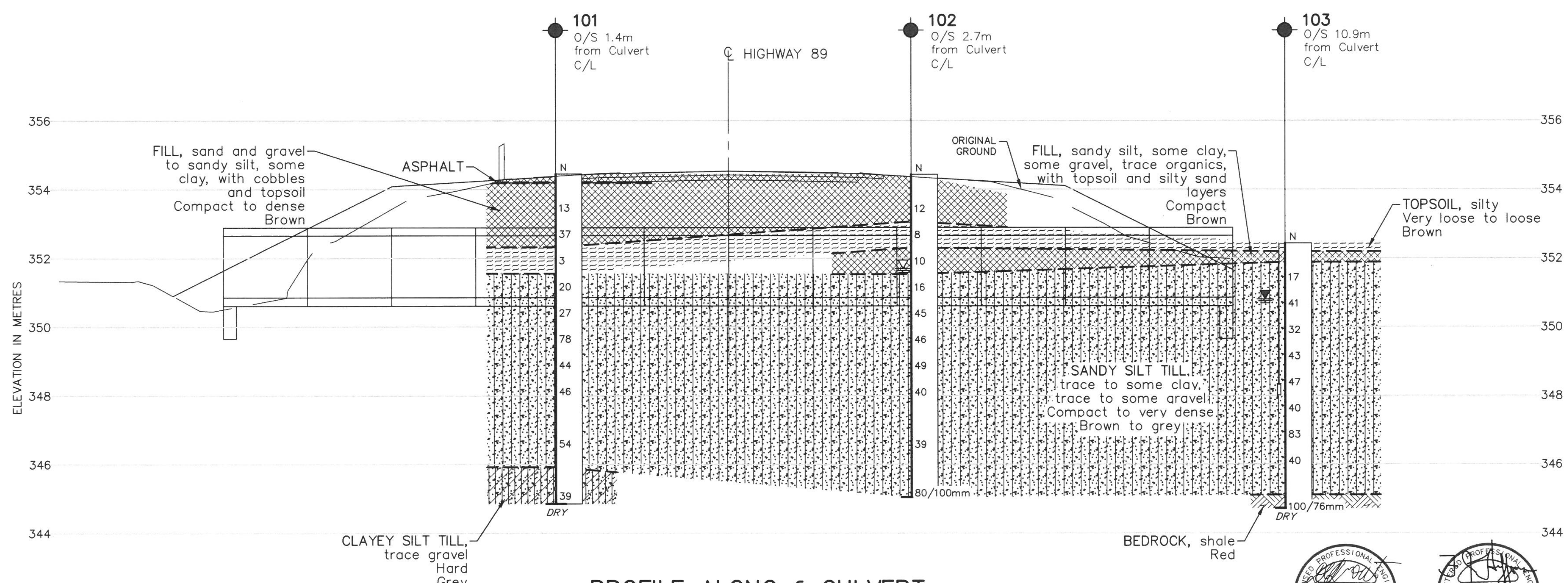
LONDON, ONTARIO, CANADA



PLAN

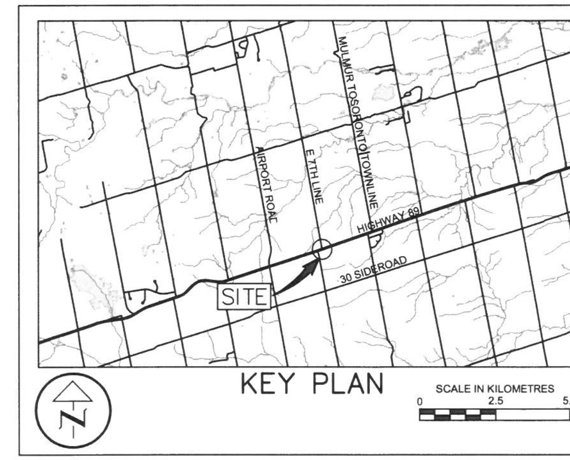
SCALE
3 0 3 m



PROFILE ALONG ℄ CULVERT

HORIZONTAL SCALE
1.5 0 1.5 m

VERTICAL SCALE
1.5 0 1.5 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 July 6, 2016
- WL encountered during drilling
- *DRY* — Water level not established

No.	ELEVATION	CO-ORDINATES (MTM NAD83 ZONE 10)	
		NORTHING	EASTING
101	354.5	4 886 746.6	264 022.4
102	354.4	4 886 736.4	264 024.2
103	352.4	4 886 729.9	264 040.3

NOTES

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

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

REFERENCE

Base plans provided by Stantec.

D. U. PROUT
F.J. HEFFERNAN

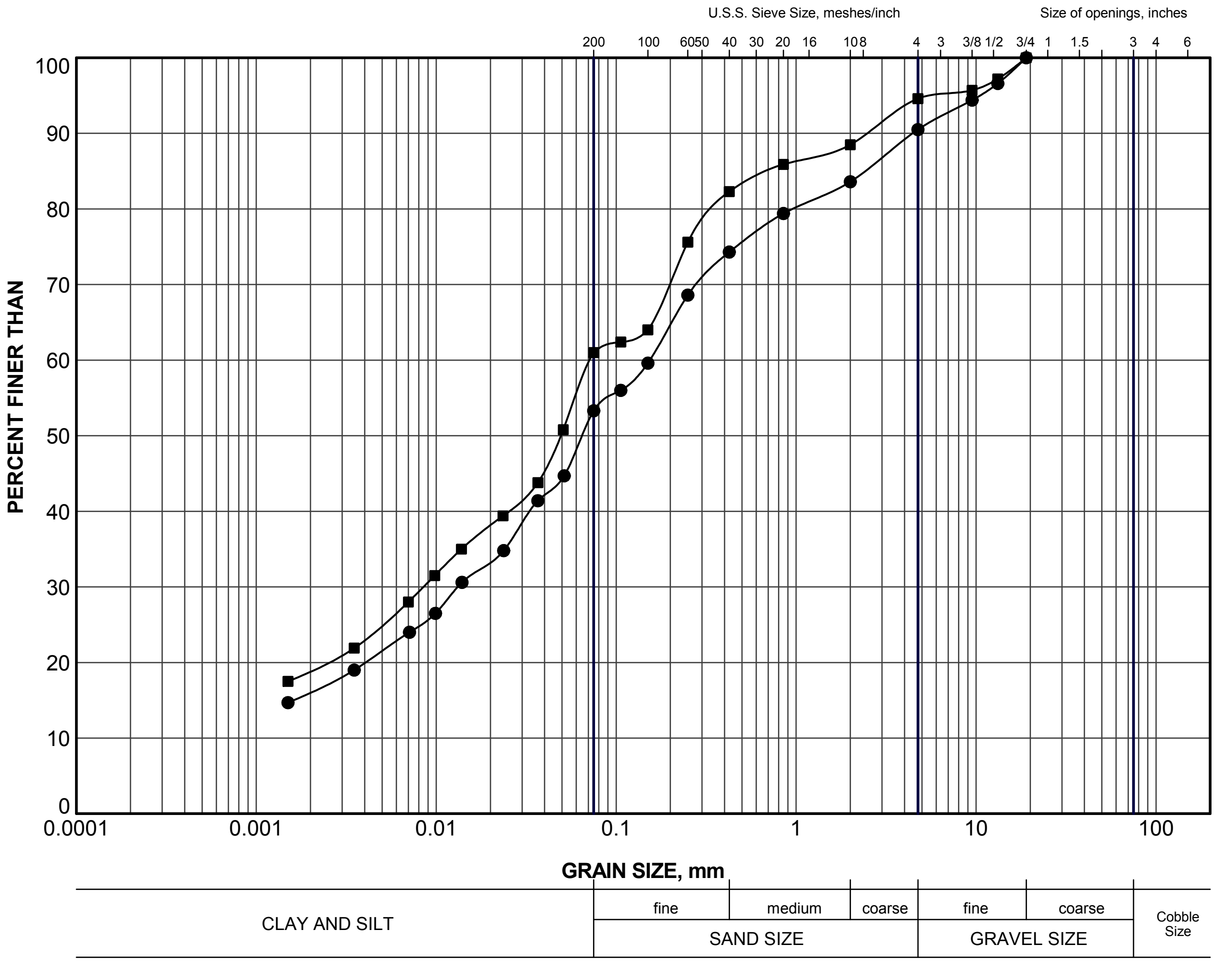
NO.	DATE	BY	REVISION

Geocres No. 41A–238

HWY. 89		PROJECT NO. 12–1132–0163	DIST.
SUBM'D. DH	CHKD. DH	DATE: Sept. 29/16	SITE: 4–320/C
DRAWN: LMK	CHKD. MEB	APPD. FJH	DWG. 1

APPENDIX A

Laboratory Test Data



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	101	1	353.5
■	102	1	353.4

PROJECT

CULVERT AT 7TH LINE REPLACEMENT, SITE NO. 4-320/C
HIGHWAY 89
GWP 3042-11-00

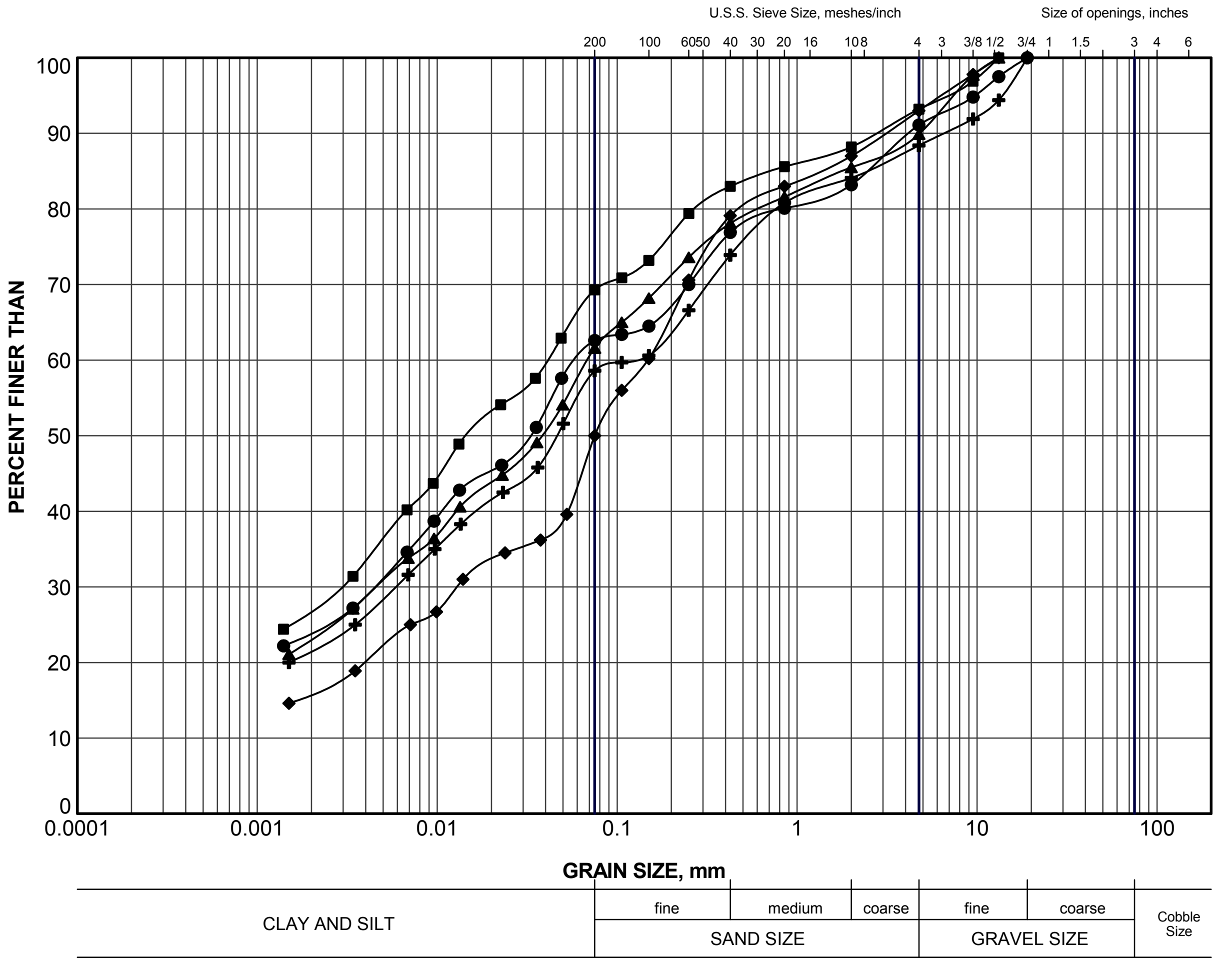
TITLE

GRAIN SIZE DISTRIBUTION
FILL

Golder Associates

PROJECT No.	12-1132-0163	FILE No. 1211320163-4000-F010A1
		SCALE N/A REV.
DRAWN LMK	Jul 12/16	**FIGURE A-1**
CHECK		

LDN_MTO_GSD GLDR_LDN.GDT 12/07/16



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	101	5	350.4
■	101	8	348.1
▲	102	6	349.6
✚	103	2	350.7
◆	103	6	347.6

PROJECT

CULVERT AT 7TH LINE REPLACEMENT, SITE NO. 4-320/C
HIGHWAY 89
GWP 3042-11-00

TITLE

GRAIN SIZE DISTRIBUTION

SANDY SILT TILL

Golder Associates

PROJECT No.	12-1132-0163	FILE No.	1211320163-4000-F010A2
		SCALE	N/A
DRAWN	LMK	Jul 12/16	REV.
CHECK			

FIGURE A-2

LDN_MTO_GSD GLDR_LDN.GDT 12/07/16

APPENDIX B

Site Photographs

Photograph 1: South elevation (inlet) of Culvert Site 4-320/C.

Photograph 2: North elevation (outlet) of Culvert Site 4-320/C.

Photograph 3: Looking southwest from 7th Line towards Culvert Site 4-320/C.

n:\active\2012\1132 - geo\1132-0100\12-1132-0163 stantec-fdns-mega culverts-3011-e-0041\ph 4000-gwp 3042-11-00\rpts\r01 4-320c 7th line\1211320163-4000-r01 feb 22 17 (final) app b - photos.docx

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