April 2017

FOUNDATION INVESTIGATION AND DESIGN REPORT

Culvert Replacement, Hodgins Drain Culvert
Site No. 2-463/C, Highway 9
Contract 4 Structure Replacements and Rehabilitation
GWP 3042-11-00
Ministry of Transportation, West Region

Submitted to:
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REPORT

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PART A

FOUNDATION INVESTIGATION REPORT

CULVERT REPLACEMENT, HODGINS DRAIN CULVERT

SITE NO. 2-463/C, HIGHWAY 9

CONTRACT 4 STRUCTURE REPLACEMENTS AND REHABILITATION

GWP 3042-11-00

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 detailed 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 Hodgins Drain Culvert at Site 2-463/C on Highway 9, at about Station 19+405 in Bruce County, Geographic Township of Kincardine.

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 and laboratory testing on selected samples. The terms of reference for the scope of work are outlined in the MTO's Request for Proposal, in Golder Associates' proposal P2-1132-0163 dated February 25, 2013, and in the Change Order 12-1132-0163-4000-C03 revised September 13, 2016. 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 19+405 Highway 9, approximately 850 metres (m) northwest of Bervie, Ontario, in the Township of Kincardine, Bruce County, Ontario. The replacement culvert will be constructed in approximately the same location as the existing culvert. The location of the culvert is shown on the Key Plan, Figure 1.

This section of Highway 9 is a two-lane, undivided highway with paved shoulders and a post and cable fence guardrail system. The highway is generally oriented northwest-southeast in the vicinity of the subject site. The Hodgins Drain watercourse flows in the culvert from north to south beneath Highway 9. The existing culvert has an overall length of 27.3 m, including extensions. The dates of construction for both the original portion and the extensions are unknown. The original structure consists of concrete non-rigid frame open footing (NRFO) construction and the extensions consist of open footing concrete rigid frame (RFO) construction.

Existing Dimensions (m)	Obvert Elevation (m)		Construction
	Lt[1]	Rt[1]	
4.27 x 1.83 x 27.3	258.28	258.13	Concrete NRFO/RFO

NOTE: 1. Lt and Rt are defined as Left and Right of centreline when facing the direction of increasing chainage.

The banks of the watercourse and the embankments along Highway 9 near the culvert are grass-covered, with young trees and shrubs on the north embankment and isolated young trees on the south embankment. Rip-rap lines the streambed on the north side of the culvert at the inlet. The watercourse flows through fields on both sides of Highway 9. Selected site photographs are provided in Appendix B.

2.1 Site Geology

The project area is located within the physiographic region of Huron Slope, which is generally a clay plain modified by a narrow strip of land which was modified by twin beaches of glacial Lake Warren, which flank the moraine.[1] The overburden in the area of the site generally consists of modern alluvium of silt, sand and gravel.[2]

The geological mapping indicates that the underlying bedrock consists of limestone, dolostone, and shale, which is part of the Detroit River Group, Onondaga formation of the Middle Devonian epoch.[3] The bedrock surface at the site is at about elevation 238 m, [4] with the overburden thickness being about 24 m. [5]

3.0 INVESTIGATION PROCEDURES

The geotechnical field investigation was carried out on October 5 and 25, 2016, during which time four boreholes were drilled at the approximate locations shown on Drawing 1. Advancement of manual boreholes was required at the inlet and outlet since the embankment sideslopes were too steep to safely access these areas with a track-mounted drill rig and utilities were present along both sides of Highway 9. Overhead hydroelectric wires are situated on the south side and multiple underground utilities are situated at both ends of the culvert. The manual boreholes were advanced with a hand auger to approximately 1.3 metres, or the limit of the hand auger.

Boreholes labeled BH-701 and BH-702 were drilled using a track-mounted Diedrich D50T drilling rig supplied and operated by a specialist drilling subcontractor. Samples of the overburden were typically obtained at depth intervals of 0.75 or 1.5 m using 50 millimetre outside diameter split spoon sampling equipment in accordance with the Standard Penetration Test (SPT) procedures (ASTM D1586). Boreholes labeled BH-703 and BH-704 were drilled by a member of our engineering staff using manual drilling equipment; the soil profile at these locations was observed as the hole was advanced; selected grab samples of the overburden were obtained from the auger for further classification at Golder's laboratory.

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 retrieved to about 40 millimetres (mm). Therefore, particles or objects that may exist within the overburden 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 till.

Groundwater conditions in the boreholes were observed throughout the drilling operations and a piezometer was installed in BH-701 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).

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

[2] Cowan, W.R,, and Pinch, J.J. 1986. Quaternary Geology of the Walkerton-Kincardine Area. Southern Ontario; Ontario Geological Survey, Map P.2956, Geological Series-Preliminary Map, scale 1:50,000. Geology 1975-1979.

[3] Ontario Geological Survey, 1991. Bedrock geology of Ontario, southern sheet. Ontario Geological Survey, Map 2544, scale 1:1 000 000.

[4] Davies, L.L., McClymont, W.R., and Karrow, P.F., 1962: Bedrock Topography Series Kincardine – Walkerton Sheet, Ontario Department of Mines, Prelim. Map No. P.165, Scale 1:50,000.

[5] Kelly, R.I. and Carter, T.R., 1993. Drift thickness, Kincardine area, southerd Ontario; Ontario Geological Survey, Preliminary Map, P.3203, Scale 1:50,000.

The field work was monitored on a full-time basis by an experienced member of our staff who positioned 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 uniquely-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 Atterberg limits 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. Table 1, below, summarizes the coordinates, ground surface elevations, and depths of the boreholes.

Table 1: Geospatial and Borehole Exploration Summary

Borehole	Location (m)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting		
BH-701	4 888 744	383 871	262.0	11.1
BH-702	4 888 750	383 845	261.5	12.7
BH-703	4 888 765	383 353	257.0*	1.3
BH-704	4 888 736	383 847	257.1*	1.2

* Borehole elevations have been inferred based on profile ground surface elevations provided

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 and laboratory testing carried out on selected samples, are given on the attached Record of Borehole sheets 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, represent transitions between soil types rather than exact planes of geological change. Further, the subsurface conditions will vary between and beyond the borehole locations.

BH-701 and BH-702 were drilled in the roadway shoulder and traffic lane and generally encountered the pavement structure, granular fill, sandy silt, clayey silt till, silt, and clayey silt. In the hand auger borings, located adjacent to the culvert inlet and outlet, topsoil, sandy silt, silt, clayey silt, and silty sand were observed.

The locations and elevations of the boreholes and 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

Asphaltic concrete was encountered at the surface of BH-701 and BH-702. The asphaltic concrete was about 60 mm thick in BH-701 and 250 mm thick in BH-702.

Sand and crushed gravel fill, interpreted to be granular base, extends to depths of about 230 to 710 mm below pavement surface at BH-701 and BH-702, respectively. Sand and gravel fill, interpreted to be granular subbase, underlies the road base material in BH-701 and extends to 760 mm below ground surface.

Clayey silt fill was encountered below the inferred pavement structure in BH-701 and BH-702. The clayey silt fill extends to a depth of 2.9 m below ground surface. This fill is generally considered to be firm to very stiff, based on SPT N values, as determined in the standard penetration testing, ranging between 5 and 16 blows per 0.3 m. The clayey silt fill had a water content of about 13 per cent. The results of a grain size distribution analysis are provided on Figure A-1 in Appendix A.

The clayey silt fill is underlain by sandy silt fill, with topsoil and organics. Based on SPT N values that ranged between 3 and 7 blows per 0.3 m, the sandy silt fill is very loose to loose. This fill was encountered to depths of 3.7 m and 4.4 m in BH-701 and BH-702, respectively.

BH-703 and BH-704 encountered sandy topsoil to depths of about 150 mm and 250 mm, respectively. No testing to determine organic content or other nutrients was carried out. Therefore, the use of materials classified as topsoil cannot be relied upon for support and growth of landscaping vegetation.

The above-noted fill and topsoil are underlain by native soil consisting of silt, sandy silt, clayey silt till, and/or clayey silt. Silt was encountered below the topsoil at BH-704 and below the clayey silt till in BH-702 at elevations 256.8 and 256.3 m, respectively. The silt was 0.3 and 0.8 m in BH-704 and in BH-702, respectively. Based on a SPT N value of 14 blows per 0.3 m, the silt is considered to be compact. Sandy silt was encountered in BH-703 and BH-704 from elevation 256.9 and 256.5 m, respectively and was 0.9 to 1.1 m thick.

A layer of clayey silt and sand about 0.8 m thick, was encountered beneath the fill in BH-701. This stratum was firm with a SPT N value of 7 blows per 0.3 m and a water content of about 28 per cent. Results of one grain size distribution analysis are provided on Figure A-2 in Appendix A.

Clayey silt till was encountered below the sandy silt in BH-701 at about elevation 257.6 m and the sandy silt fill in BH-702 at about elevation 257.1 m. The clayey silt till was stiff to hard based on SPT N values ranging from 13 and 40 blows per 0.3 m. The clayey silt till had water contents of 17 and 21 per cent. The results of grain size distribution analyses carried out on two samples of the clayey silt till are provided on Figure A-3 in Appendix A. Although, not specifically encountered in this investigation, cobbles and boulders should be anticipated in the clayey silt till deposits.

All of the boreholes were terminated in a deposit of clayey silt. The clayey silt was encountered between elevations 255.5 and 256.2 m and was explored for 0.2 to 6.7 m. The clayey silt had water contents of about 23 and 25 per cent and SPT N values of 9 to 22 blows per 0.3 m, indicating a stiff to very stiff consistency. The results of grain size distribution analyses carried out on two samples of the clayey silt are provided on Figure A-4 in Appendix A.

A 0.1 m thick seam of grey silty sand was encountered in BH-703 below the sandy silt at elevation 256.0 m. The silty sand had a water content of about 23 per cent. The results of a grain size distribution analysis of a sample of the silty sand are presented on Figure A-5.

Results from Atterberg limits tests carried out on the clayey silt fill, clayey silt till, and clayey silt are provided on Figure A-6 in Appendix A. All tests indicate clayey soils of low plasticity.

4.3 Groundwater Conditions

Groundwater conditions were observed during and on completion of drilling, and a groundwater observation piezometer was installed in BH-701. The installation details are provided on the corresponding Record of Borehole sheet following the text of this report. Groundwater was encountered during drilling in BH-702 and BH-703 at depths of 5.4 and 1.0 m or at approximate elevation 256.1 and 256.0 m, respectively. The water level was measured in the piezometer installed in BH-701 at depths of 1.9 to 4.2 m or approximate elevaitons 257.84 to 260.17 m between October 4, 2016 and January 6, 2017. A summary of the encountered and measured groundwater levels is provided in the table below.

Table 2: Encountered and measured groundwater levels.

Borehole	Ground Surface Elevation (m)	Encountered Groundwater Elevation (m)	Measured Groundwater Level Elevation (m) October 4, 2016	Measured Groundwater Level Elevation (m) October 25, 2016	Measured Groundwater Level Elevation (m) January 6, 2017
BH-701	262.05	dry	257.84	258.87	260.17
BH-702	261.47	256.1	-	-	-
BH-703	257.04*	256.0	-	-	-
BH-704	257.08*	dry	-	-	-

* Borehole elevation inferred based on profiled ground surface elevations provided

The above 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 256.5 m. Given the presence of a clayey silt till overlain by granular soils, perched water conditions are possible at this site. In addition, 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. The elevated groundwater levels measured in BH-701 suggest a perched groundwater table in the sandy silt or fill.

5.0 MISCELLANEOUS

The investigation was carried out using equipment supplied and operated by London Soil Test Limited, an Ontario Ministry of Environment and Climate Change licensed well contractor. The field operations were supervised by Mr. William Hanson, E.I.T. and Mr. Jordan Kiss, E.I.T. under the direction of the Field Investigation Manager, Mr. Brett Thorner, P.Eng. The laboratory testing was carried out at Golder's 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 Ms. Cara Kennedy, E.I.T. under the direction of the Project Engineer Ms. Dirka U. Prout, P.Eng. The report was reviewed by Mr. Terry Nicholas, P.Eng, who is a senior consultant with Golder Associates. Mr. Fintan J. Heffernan, P.Eng, the Designated MTO Contact and Quality Control Auditor for this assignment, conducted an independent quality review of the report.

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PART B

FOUNDATION DESIGN REPORT

CULVERT REPLACEMENT, HODGINS DRAIN CULVERT

SITE NO. 2-463/C, HIGHWAY 9

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 2-463/C at about Station 19+405 on Highway 9, approximately 850 m northwest of Bervie, Ontario, in the Township of Kincardine, Bruce County, Ontario. It is understood that the replacement culvert will be constructed in approximately the same location as the existing culvert.

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

The existing culvert has an unknown date of construction and is 27.3 m long, including extensions of unknown length and unknown dates of construction. The original structure is a concrete non-rigid frame open-footing (NRFO), with open-footing concrete rigid frame extensions. The streambed elevation at the inlet of the existing open-footing culvert is approximately 256.1 m. Based on the information provided by Stantec, the replacement culvert will be a 28.06 m long precast concrete box culvert with a 4.2 m span and a 2.4 m high opening with an approximate invert elevation of 255.88 m. The replacement culvert will be installed at about the same location with respect to Highway 9, at Station 19+405. Although it is anticipated that the new culvert will be a precast structure, recommendations have also been included for an open footing culvert option.

6.1 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the Canadian Highway Bridge Design Code (CHBDC version S6-14) and its Commentary, a classification of ‘typical’ consequence has been assumed for the proposed replacement culvert and foundation system. This consequence classification should be confirmed by Stantec and the MTO.

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

6.2 Foundations

Based on the results of the investigation, the new box culvert may be founded at or below elevation 255.6 m in the clayey silt or clayey silt till. The culvert foundations may be designed using a factored geotechnical resistance at Ultimate Limit States (ULS) of 250 kilopascals (kPa) and a geotechnical resistance at Serviceability Limit States (SLS) of 150 kPa. We anticipate the SLS value corresponds to approximately 25 mm of settlement or less. Bedding for a precast culvert and a levelling pad should be provided as discussed in Section 6.2.3 below. In the event that an open footing culvert is constructed, the footings may be founded at or below elevation 254.5 m using the same geotechnical resistances provided above.

6.2.1 Frost and Scour Protection

Frost treatment in the form of a frost taper symmetrical about the culvert centreline should be provided in accordance with Ontario Provincial Standard Drawing (OPSD) 803.010. The design frost penetration depth for this area is 1.4 m below ground surface. If an open-footing culvert is installed, the footings are to be provided with soil cover equivalent to the frost depth or thermal equivalent. The culvert should also be adequately protected against scour as noted in Section 1.9.5 of the CHBDC.

6.2.2 Resistance to Lateral Forces/Sliding Resistance

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

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

$H_{ri} = \psi\phi_{gu} (A'c'_i + V_f \tan\delta'_i) > H_f$ (for pre-cast elements)

$H_{rs} = \psi\phi_{gu} (A'c' + V_f \tan\phi') > H_f$ (for open footing culverts)

Where:

Ψ = consequence factor, given in Section 6.5.2, Table 6.1 of the CHBDC

ϕ_{gu} = ultimate geotechnical resistance factor, given in Section 6.9.1, Table 6.2 of the CHBDC

A' = effective contact area, square metres

c'_i = effective cohesion along the interface between box culvert base and bedding/levelling, nil

c' = effective cohesion, nil

$\tan\delta'_i$ = coefficient of friction for interface between box culvert base and bedding/levelling pad

$\tan\phi'$ = coefficient of friction between footings and native founding soils

V_f = factored vertical force, kilonewtons

H_f = factored horizontal load, kilonewtons

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

Structure	Interaction	Angle of Friction, δ/ϕ (degrees)	Coefficient of Friction, tan δ/ϕ
Precast Box Culvert	Precast concrete on Granular A bedding/levelling pad	30	0.58
Open Footing Culvert	Cast-in-Place concrete on native clayey silt till, clayey silt or/and silt	30	0.58

6.2.3 Bedding, Backfill and Cover

Bedding, backfill and cover for the culvert shall be placed in accordance with OPSS.PROV 501, OPSD 803.010 and Special Provision (SP) 422501 Backfill. The backfill should consist of free-draining, non-frost susceptible granular materials such as Ontario Provincial Standard Specifications (OPSS) Granular B, Type III, or Granular A placed in 0.3 metre thick loose lifts and uniformly compacted. Heavy compaction equipment should not be used immediately adjacent to the walls and roof of the culvert. The height of backfill adjacent to the culvert walls should be maintained as equal as possible on both sides of the culvert during all stages of backfill placement. The height of the backfill at each side of the culvert should differ no more than 500 mm at any time.

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

6.2.4 Other Design Considerations

The fill height above the culvert roof will be less than 4.5 m and no grade raise or widening is planned. The foundation materials consist of low compressibility cohesive deposits. Therefore, differential settlement along the length of this culvert is expected to be negligible and cambering is not required. In addition, total settlement is expected to be minimal, on the order of 25 millimetres, at the culvert location.

If the results of hydraulic analysis indicate that there will be significant difference in hydraulic head between the inlet and outlet, then the culvert inlet must be provided with a headwall, clay seal or other seepage control measure. The design of the replacement culvert should include a cut-off wall at the inlet in accordance with Section 1.9.5.6 of the CHDBC and WE-9 of the MTO's Highway Drainage Design Standards.

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.

No visual signs of slope instability were noted during the investigation. Reinstatement of the highway embankment should be carried out as per OPSS.PROV 501. The side slopes should be trimmed to a maximum final inclination of 2 horizontal to 1 vertical. A Factor of Safety against deep seated failure of greater than 1.3 is available for embankments constructed on the clayey silt till and clayey silt subgrade.

6.3 Excavations and Groundwater Control

Excavations will extend through the existing fill and topsoil into the underlying sandy silt, clayey silt till, silt, and clayey silt. Seepage volumes from the sandy silt and/or silt are anticipated to be such that groundwater control may be achieved by using properly constructed and filtered sumps in the base of the excavation. Noting that there is the potential for localized perched groundwater in the silty sand layer above the clayey silt till stratum in BH-703 and sandy silt till layer above the clayey silt till in BH-701, more proactive dewatering measures, such as use of multiple closely spaced well filtered sumps, may be required. 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.

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 and native sandy silt above the groundwater level would be classified as a Type 3 soil. The clayey silt till would be classified as a Type 2 soil. Further, the clayey silt would be classified as Type 2 soil above the groundwater level and Type 3 soil below the groundwater level. Saturated silts (native or fill), would be considered Type 4 soil.

Surface water runoff should be directed away from the excavations at all times. The existing culvert flows are anticipated to be diverted/piped during construction.

6.4 Liquefaction Potential and Seismic Analysis

6.4.1 Seismic Parameters

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

The importance category of the replacement culvert is "other" based on the CHBDC. The corresponding Seismic Category for the structure is 1 based on Table 4.10 of the CHBDC. Structures in Seismic Category 1 need not be analysed for seismic loads. However, minimum requirements as outlined in CHBDC Clause 4.4.5.1 must be followed. 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 low due to historically low seismicity in this area, founding soils with a normalized SPT $(N_1)_{60}$ generally greater than 20 blows per 0.3 m, 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 S6-14). These design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Backfill should be placed in accordance with Section 6.2.3 above.
- A compaction surcharge equal to 12 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 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 m behind the culvert walls (Case (a) from commentary on CHBDC Figure C6.20).

[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.

- For Case (a), the restrained case, the pressures are based on the existing embankment fill materials; however, since frost tapers will be provided, the pressures will be based on the materials used to construct the tapers. The following parameters (unfactored) may be used:

	GRANULAR A	GRANULAR B TYPE III
Soil unit weight:	22 kN/m³	21 kN/m³
Coefficients of lateral earth pressure:		
'At rest' or restrained, K_o	0.47	0.5

- 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 m at the base of foundation 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 III
Fill unit weight:	22 kN/m³	21 kN/m³
Coefficients of lateral earth pressure:		
'active' or unrestrained, K_a	0.31	0.33
‘passive’, K_p	3.3	3.0

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 9 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 through the clayey silt till may be difficult, with the likelihood of encountering hard material and the potential for cobbles or boulders.

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

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

The unfactored triangular earth pressure distribution (p' in kN/m²; increasing with depth) can be calculated as follows:

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

where: H = the height of the excavation at any point in m

K_a = active coefficient of earth pressure

γ = soil unit weight

γ_w = unit weight of water or 9.8 kN/m³

q = surcharge for traffic and other loading

h_w = height of groundwater level above excavation base; inferred groundwater level at time of investigation was 256.5 m

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/m³)	Effective Unit Weight γ' (kN/m³)
	Active, K_a	At Rest, K_o	Passive, K_p			
Fill	0.38	0.55	Nil	-	18.0	8.0
Sandy Silt	0.33	0.50	3.00	30	19.0	9.0
Silty Sand	0.33	0.50	3.00	30	19.0	9.0
Clayey Silt Till	0.31	0.47	3.25	32	20.5	10.5
Silt	0.32	0.48	3.12	31	20.0	10.0
Clayey Silt	0.33	0.5	3.00	30	20.0	10.0

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 frozen, soft or loose soils should be stripped from the proposed founding areas prior to placement of the bedding materials or working mat. 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 m 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 natural soils (clayey silt till, silt, and clayey silt) have 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 clayey silt till.

7.0 MISCELLANEOUS

This section of the report was prepared by Ms. Cara Kennedy, E.I.T., under the direction of the Project Engineer Ms. Dirka U. Prout, P.Eng. The report was reviewed by Mr. Terry Nicholas, 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.

LICENSED PROFESSIONAL ENGINEER
D. U. PROUT
Apr 27/17
PROVINCE OF ONTARIO

Dirka U. Prout, P.Eng
Project Engineer

Terry Nicholas, P.Eng.
Senior Consultant

REGISTERED PROFESSIONAL ENGINEER
F.J. HEFFERNAN
Apr 27/17
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Fintan J. Heffernan, P.Eng.
MTO Designated Contact

CK/DUP/TN/FJH/cr

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n:\active\2012\1132 - geo\1132-0100\12-1132-0163 stantec-fdns-mega culverts-3011-e-0041\ph 4000-gwp 3042-11-00\rpts\r07 2-463-c hodgins drain\1211320163-4000-r07 apr 27 17 (final) replaceclvrt-2-463-c.docx

LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\,\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_3	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

* 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 $= \tan\delta$
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

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

IV. SOIL TESTS

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

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

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 BH-701

METRIC

PROJECT 12-1132-0163
W.P. 3042-11-00
LOCATION N 4888744.2 , E 383871.0
ORIGINATED BY WH
DIST HWY 9
BOREHOLE TYPE POWER AUGER, HOLLOW STEM
COMPILED BY ZJB/LMK
DATUM GEODETIC
DATE October 5, 2016
CHECKED BY

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
262.05	PAVEMENT SURFACE
0.06	ASPHALT
0.23	FILL, sand and crushed gravel Brown
261.29 0.76	FILL, sand and gravel, some silt, with cobbles Brown
	FILL, clayey silt, some sand, trace gravel, with topsoil layers Firm to stiff Brown and grey
259.15 2.90	FILL, sandy silt, with topsoil and organics Very loose Grey
258.39 3.66	CLAYEY SILT AND SAND Firm Brown
257.63 4.42	CLAYEY SILT TILL, trace gravel, with cobbles, with sand seams at about elev. 256.0m Stiff to hard Brown
254.96 7.09	CLAYEY SILT, trace sand, with silt seams Stiff to very stiff Grey
250.92 11.13	END OF BOREHOLE Borehole dry during drilling on October 4, 2016 Water level measured in piezometer at elev. 257.84m on October 4, 2016. Water level measured in piezometer at elev. 258.87m on October 25, 2016. Water level measured in piezometer at elev. 260.17m on January 6, 2017.

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	SS	5				
2	SS	7				
3	SS	9				
4	SS	3				
5	SS	7	0	40	37	23
6	SS	13				
7	SS	40				
8	SS	14	0	0	66	34
9	SS	16				
10	SS	12	0	1	57	42
11	SS	12				

GROUND WATER CONDITIONS: Granular Bentonite; Cuttings; Filter Sand; Piezometer; Cuttings

ELEVATION SCALE: 262, 261, 260, 259, 258, 257, 256, 255, 254, 253, 252, 251

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/m^3

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

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RECORD OF BOREHOLE No BH-702

METRIC

PROJECT 12-1132-0163			
W.P. 3042-11-00	LOCATION N 4888750.1 , E 383844.7	ORIGINATED BY WH	
DIST	HWY 9	BOREHOLE TYPE POWER AUGER, HOLLOW STEM	COMPILED BY ZJB/LMK
DATUM GEODETIC	DATE October 5, 2016	CHECKED BY	

ELEV / DEPTH (m)	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
261.47	PAVEMENT SURFACE					
0.00	ASPHALT					
0.25	FILL, sand and crushed gravel Brown					
260.76 0.71	FILL, clayey silt, some sand, trace gravel, with topsoil layers Stiff to very stiff Brown and grey	1	SS	13		
		2	SS	16		
		3	SS	15		0 23 51 26
258.57 2.90	FILL, sandy silt, trace clay, with topsoil layers Firm Brown and grey	4	SS	7		
		5	SS	5		
257.05 4.42	CLAYEY SILT TILL, some sand, trace gravel Stiff Brown	6	SS	14		1 12 54 33
256.29 5.18	SILT, some sand, with clayey silt pockets Compact Brown	7	SS	14	∇	
255.53 5.94	CLAYEY SILT, trace sand, with silt seams Stiff to very stiff Brown to grey at about elev. 254.4m	8	SS	12		
		9	SS	9		
		10	SS	17		
		11	SS	15		0 0 68 32
248.82		12	SS	22		
12.65	END OF BOREHOLE Groundwater encountered at about elev. 256.1m during drilling on October 4, 2016.					

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; ELEVATION SCALE (261–249); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³

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

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London, Ontario

RECORD OF BOREHOLE No BH-703

METRIC

PROJECT 12-1132-0163
W.P. 3042-11-00
LOCATION N 4888765.0 , E 383852.8
ORIGINATED BY JK
DIST HWY 9
BOREHOLE TYPE
COMPILED BY ZJB/LMK
DATUM GEODETIC
DATE October 25, 2016
CHECKED BY

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
257.04	GROUND SURFACE						257							
0.00	TOPSOIL, sandy, with organics													
0.15	Brown SANDY SILT, trace clay, trace gravel. Brown to grey at about elev. 257.0m													
255.97 1.17	SILTY SAND, trace clay. Grey		1	AS		∇	256		o (~22)		0	21	71	8
1.32	CLAYEY SILT, trace sand, trace gravel. Grey		2	AS										
	END OF BOREHOLE													

Groundwater encountered at about elev. 256.0m during drilling on October 25, 2016.

Borehole elevation has been inferred based on profile ground surface elevations provided.

Legend: ○ UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × LAB VANE. PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L.

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

LDN_MTO_06 1211320163-4000.GPJ LDN_MTO.GDT 15/02/17

Golder Associates
London, Ontario

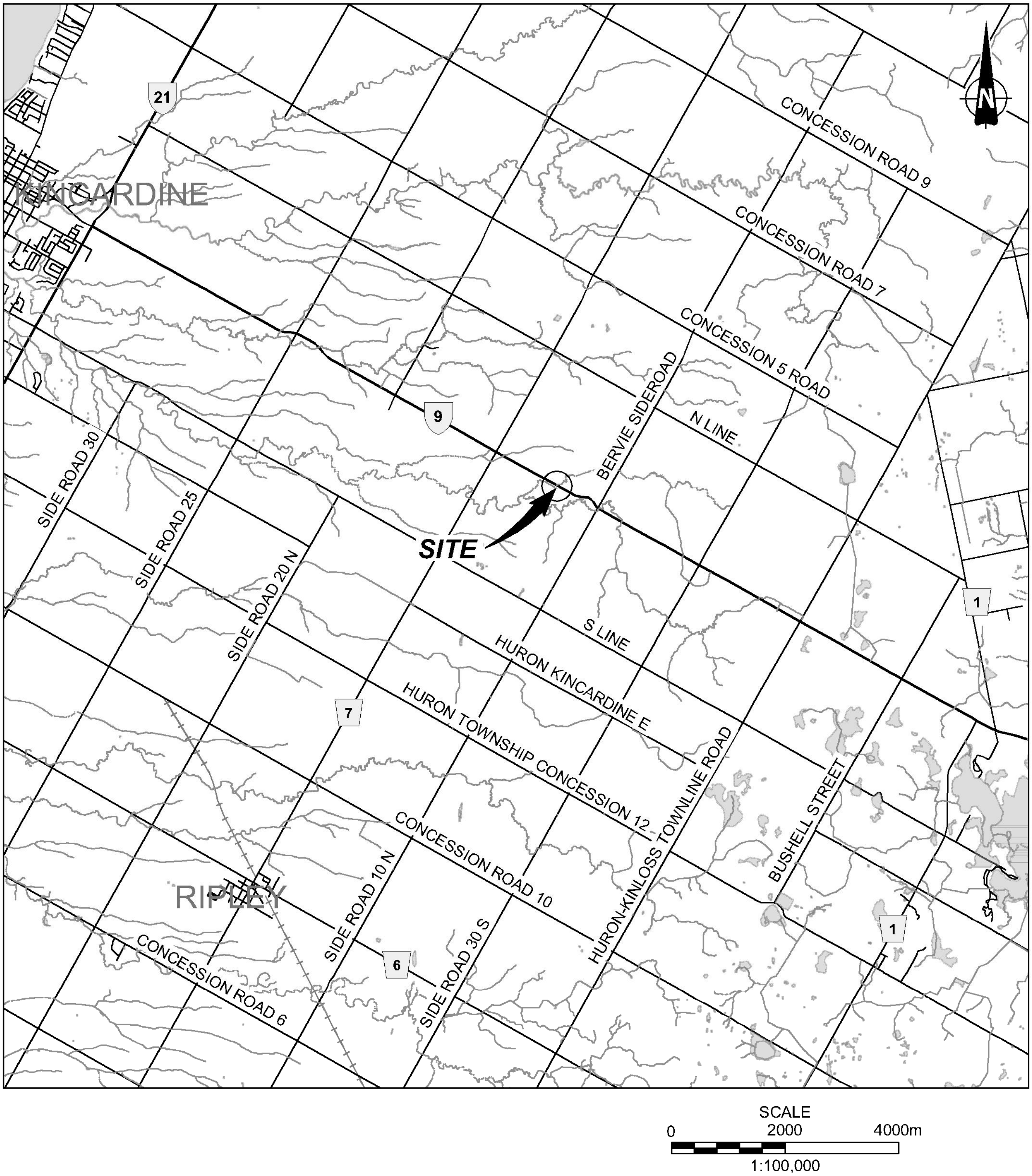
RECORD OF BOREHOLE No BH-704

METRIC

PROJECT 12-1132-0163			
W.P. 3042-11-00	LOCATION N 4888735.9 , E 383847.2	ORIGINATED BY JK	
DIST	HWY 9	BOREHOLE TYPE	COMPILED BY ZJB/LMK
DATUM GEODETIC	DATE October 25, 2016	CHECKED BY	

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	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	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
257.08	GROUND SURFACE										
0.00	TOPSOIL, sandy Brown						257				
0.25	SILT, some sand, some clay Brown										
0.51											
256.17	SANDY SILT, some gravel, trace clay, with sand layers Brown										
0.91			1	AS			256				
255.84 1.24	CLAYEY SILT, trace sand, some gravel Grey										
	END OF BOREHOLE Borehole dry during drilling on October 25, 2016 Borehole elevation has been inferred based on profile ground surface elevations provided.										

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

HODGINS DRAIN CULVER REPLACEMENT, SITE NO. 2-463/C
HIGHWAY 9
GWP 3042-11-00

TITLE

KEY PLAN

Golder Associates

PROJECT No.		12-1132-0163	FILE No.	1211320163-4000-F07001
			SCALE AS SHOWN	REV. 0
CADD	ZJB/LMK	Jan. 12/17		
CHECK			**FIGURE 1**	

Drawing file: 1211320163-4000-F07001.dwg Jan 12, 2017 - 9:14am

MINISTRY OF TRANSPORTATION, ONTARIO

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

CONT No.
WP No. 3042-11-00

SHEET

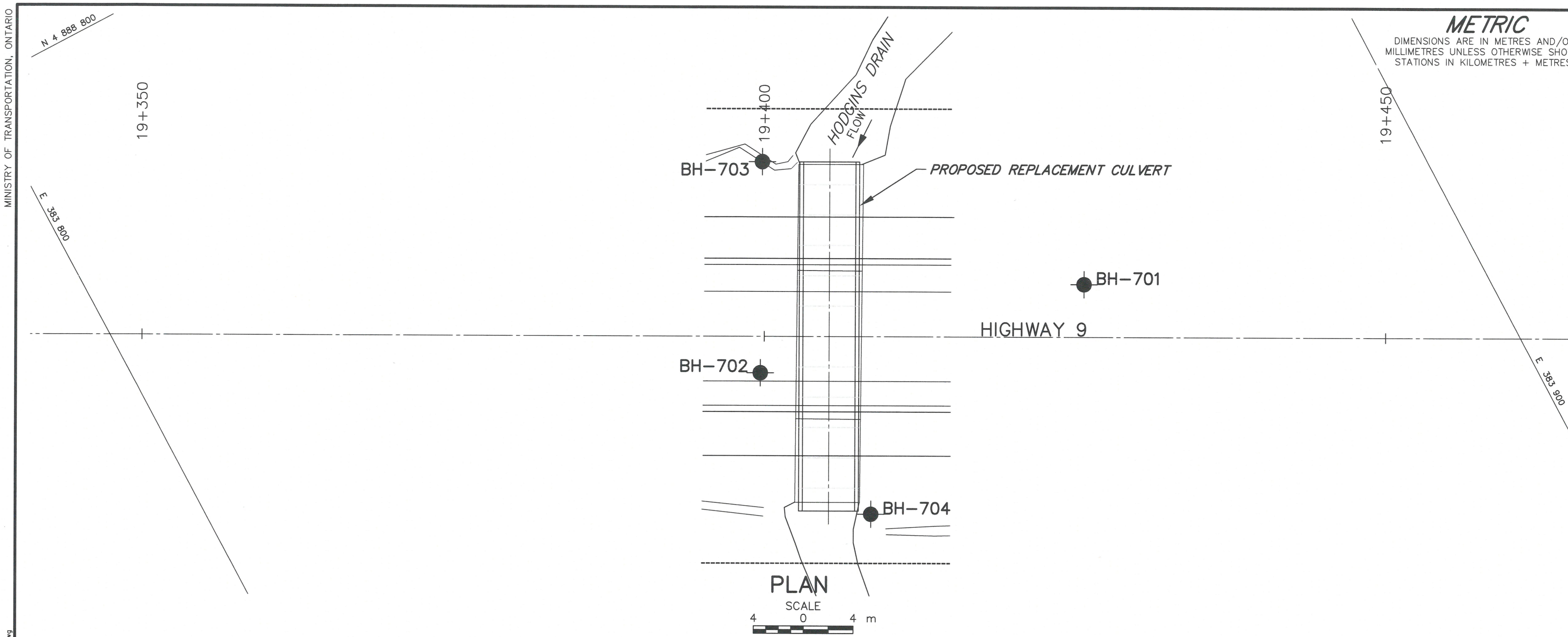
HODGINS DRAIN CULVERT REPLACEMENT

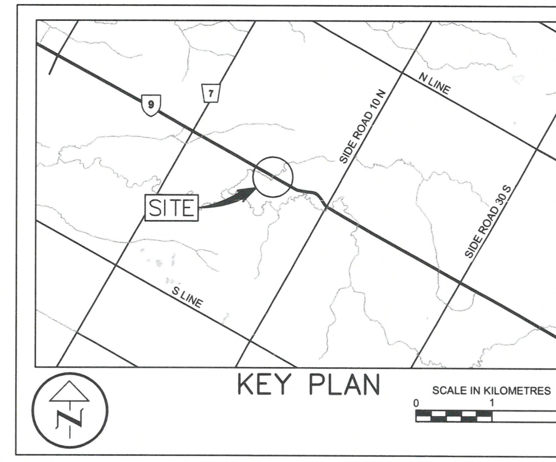
HIGHWAY 9 SITE No. 2-463/C

BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates

Golder Associates Ltd.
LONDON, ONTARIO, CANADA





LEGEND

Borehole – Current Investigation

Seal

Piezometer

N Standard Penetration Test Value

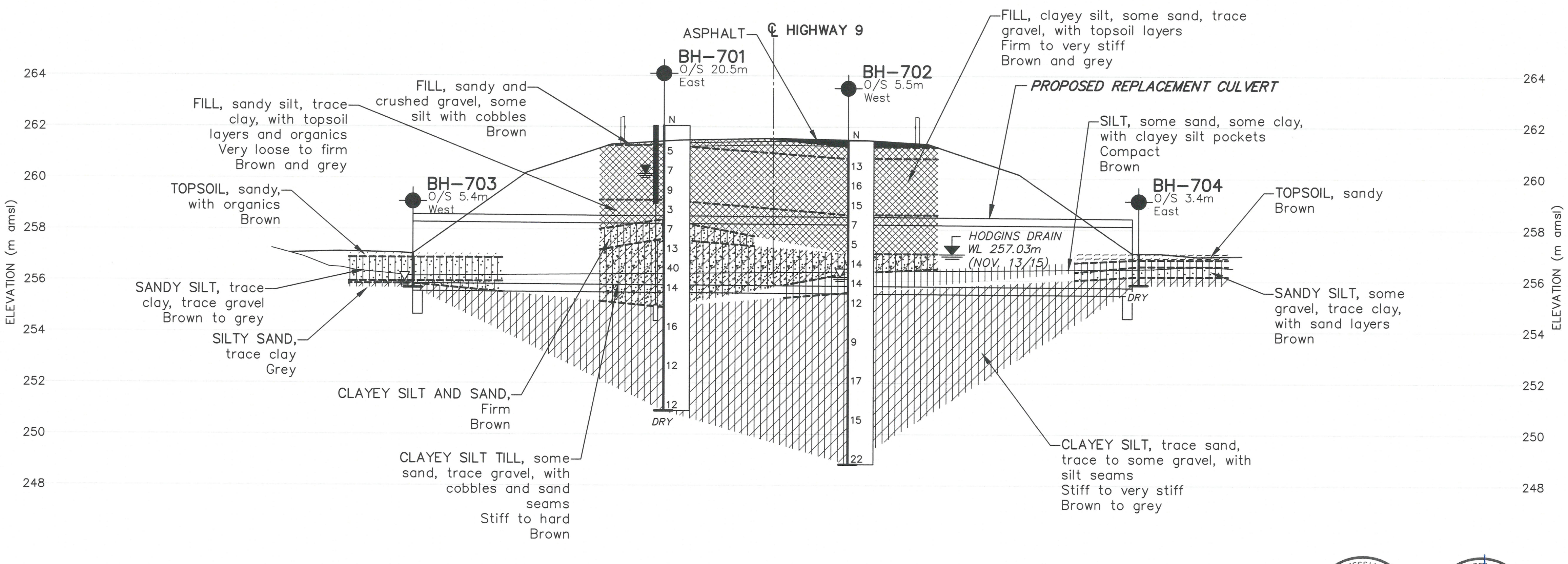
16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)

WL measured on January 6, 2017

WL encountered during drilling

DRY Borehole dry during drilling

No.	ELEVATION	CO-ORDINATES (MTM ZONE 11)	
		NORTHING	EASTING
BH-701	262.05	4 888 744.2	383 871.0
BH-702	261.47	4 888 750.1	383 844.7
BH-703	257.04*	4 888 765.0	383 852.8
BH-704	257.08*	4 888 735.9	383 847.2



PROFILE ALONG ℄ OF CULVERT

HORIZONTAL SCALE 2 0 2 m   VERTICAL SCALE 2 0 2 m

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.

* Borehole elevations have been inferred based on profile ground surface elevations provided.

REFERENCE

Base plans provided in digital format by Stantec.

LICENSED PROFESSIONAL ENGINEER D. U. PROUT PROVINCE OF ONTARIO Apr 3/17

REGISTERED PROFESSIONAL ENGINEER F.J. HEFFERNAN PROVINCE OF ONTARIO Apr 3/17

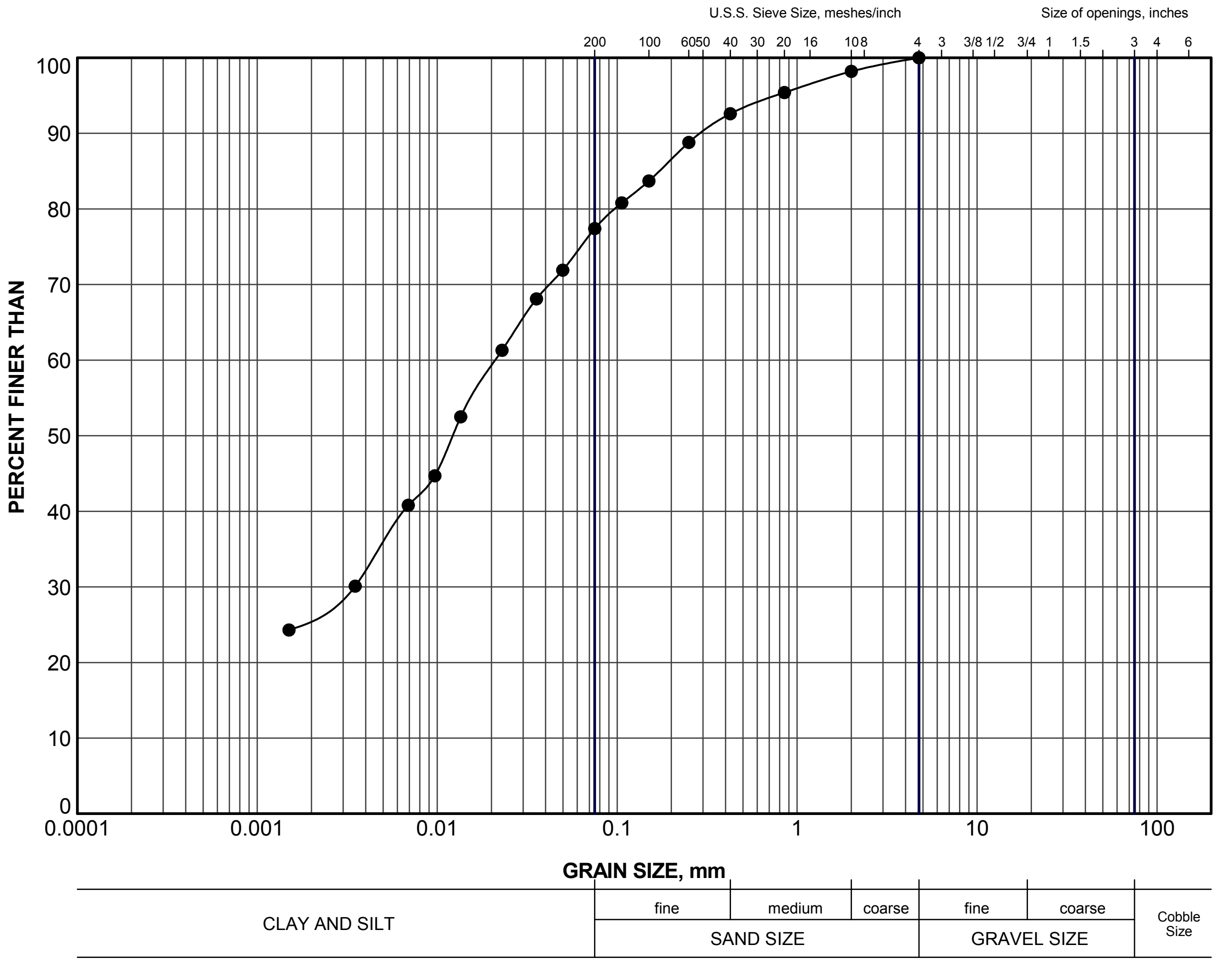
NO.	DATE	BY	REVISION

Geocres No. 41A-241			
HWY. 9		PROJECT NO. 12-1132-0163	DIST.
SUBM'D. DH	CHKD. DH	DATE: Feb. 16/17	SITE: 2-463/C
DRAWN: LMK	CHKD. DUP	APPD. FJH	DWG. 1

PLOT DATE: March 31, 2017
FILENAME: \\golder.gds\gal\London\active\2012\1132 – Geo\1132-0100\12-1132-0163 STANTEC-FONS-MEGA CULVERTS-3011-E-0041\Drafting\AutoCAD files\1211320163-4000-D07001.dwg

APPENDIX A

Laboratory Test Data



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BH-702	3	259.0

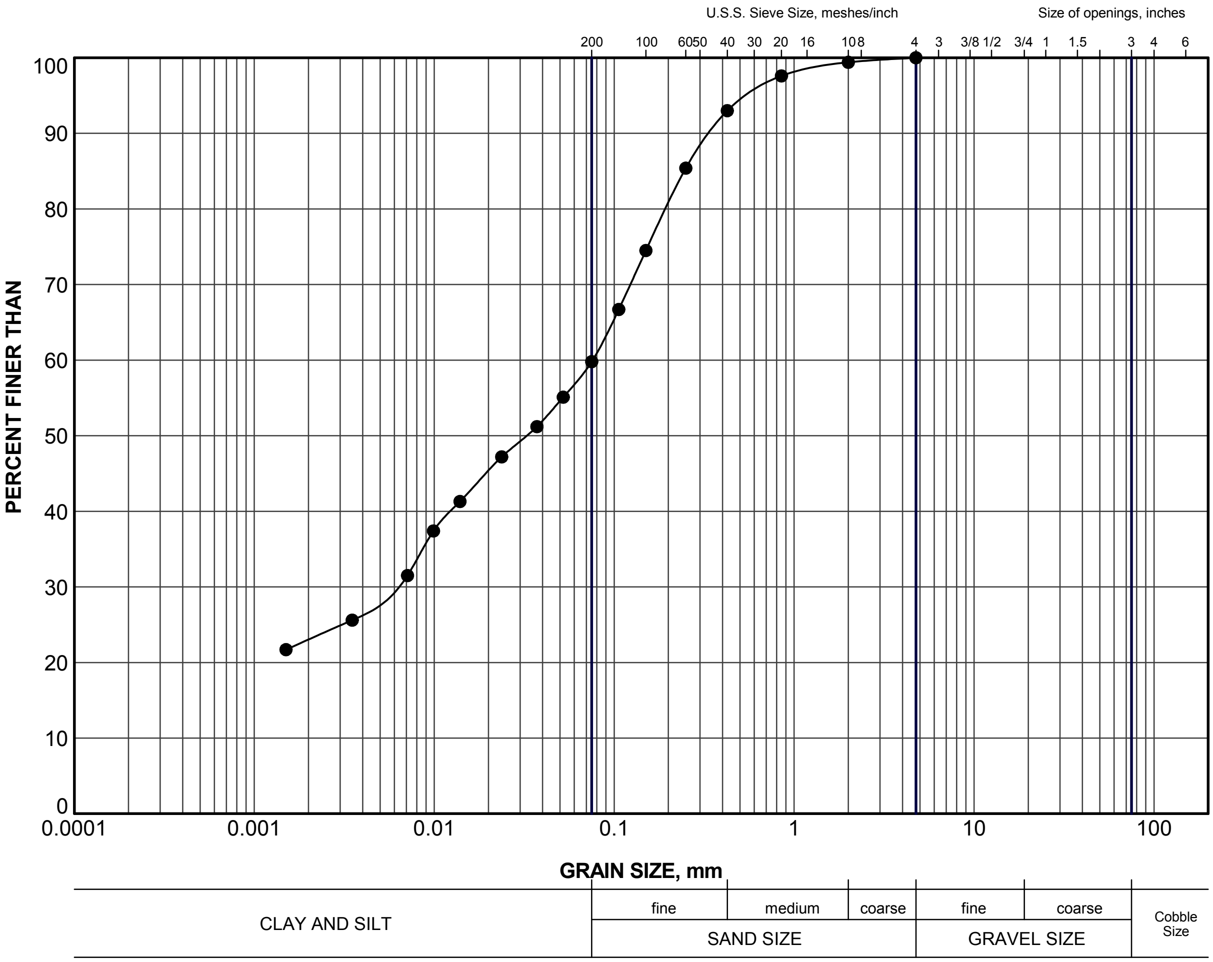
PROJECT
HODGINS DRAIN CULVERT REPLACEMENT, SITE NO. 2-463/C
HIGHWAY 9
GWP 3042-11-00

TITLE
GRAIN SIZE DISTRIBUTION
FILL

Golder Associates

PROJECT No.	12-1132-0163	FILE No. 1211320163-4000-F070A1	
		SCALE N/A	REV.
DRAWN	ZJB	Nov 4/16	**FIGURE A-1**
CHECK			

LDN_MTO_GSD GLDR_LDN.GDT 24/10/16



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BH-701	5	258.0

PROJECT
HODGINS DRAIN CULVERT REPLACEMENT, SITE NO. 2-463/C
HIGHWAY 9
GWP 3042-11-00

TITLE

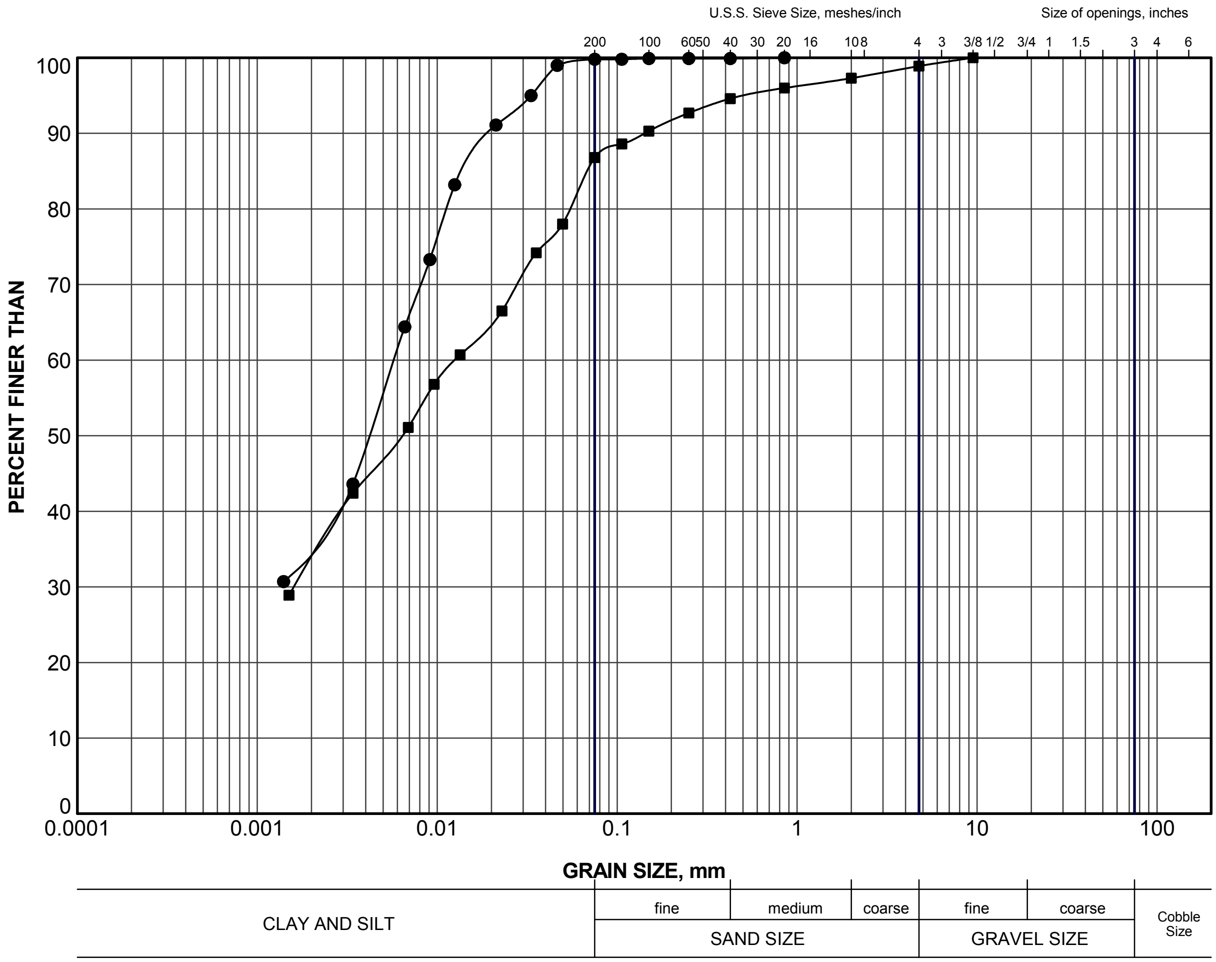
GRAIN SIZE DISTRIBUTION

CLAYEY SILT AND SAND

Golder Associates

PROJECT No.	12-1132-0163	FILE No. 1211320163-4000-F070A2	
		SCALE N/A	REV.
DRAWN	ZJB/LMK	Feb 15/17	
CHECK			FIGURE A-2

LDN_MTO_GSD GLDR_LDN.GDT 24/10/16



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BH-701	8	255.8
■	BH-702	6	256.7

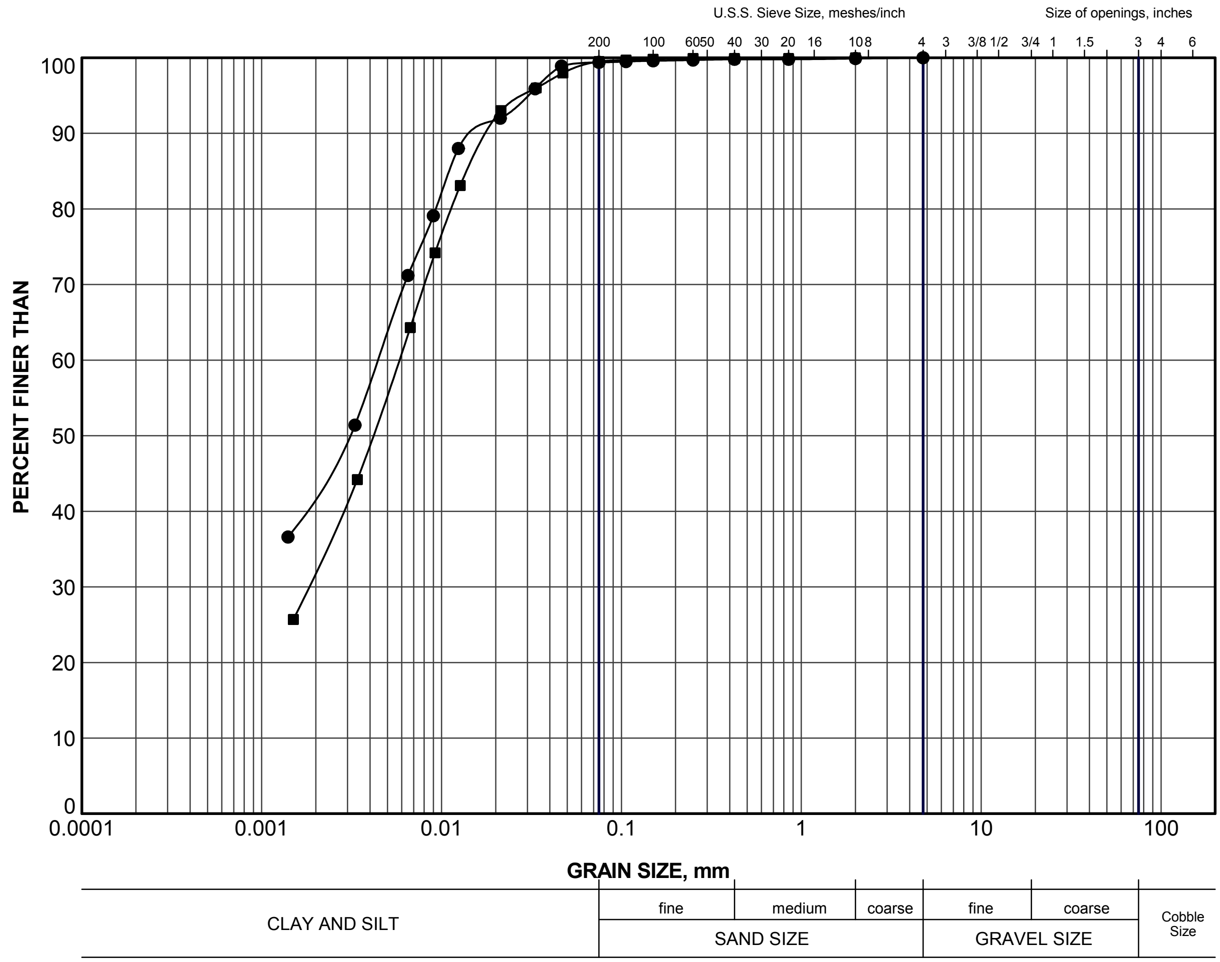
PROJECT
HODGINS DRAIN CULVERT REPLACEMENT, SITE NO. 2-463/C
HIGHWAY 9
GWP 3042-11-00

TITLE
GRAIN SIZE DISTRIBUTION
CLAYEY SILT TILL

PROJECT No.	12-1132-0163	FILE No. 1211320163-4000-F070A3	
		SCALE N/A	REV.
DRAWN	ZJB	Dec 21/16	FIGURE A-3
CHECK			

Golder Associates

LDN_MTO_GSD GLDR_LDN.GDT 24/10/16



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BH-701	10	252.7
■	BH-702	11	250.6

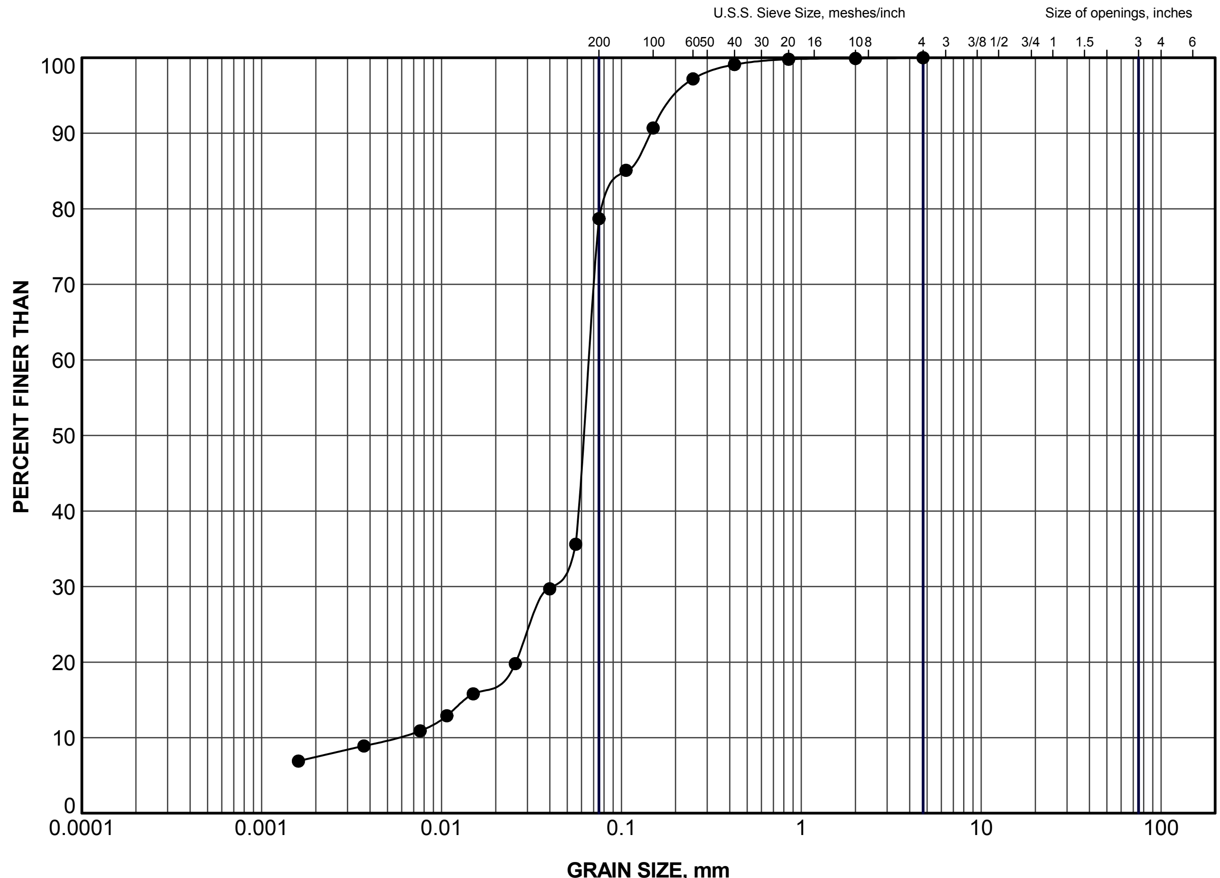
PROJECT
HODGINS DRAIN CULVERT REPLACEMENT, SITE NO. 2-463/C
HIGHWAY 9
GWP 3042-11-00

TITLE
GRAIN SIZE DISTRIBUTION
CLAYEY SILT

Golder Associates

PROJECT No.	12-1132-0163	FILE No. 1211320163-4000-F070A4	
		SCALE N/A	REV.
DRAWN	ZJB	Nov 4/16	
CHECK			**FIGURE A-4**

LDN_MTO_GSD GLDR_LDN.GDT 24/10/16



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	BH-703	1	1.1

PROJECT
HODGINS DRAIN CULVERT REPLACEMENT, SITE NO. 2-463/C
HIGHWAY 9
GWP 3042-11-00

TITLE

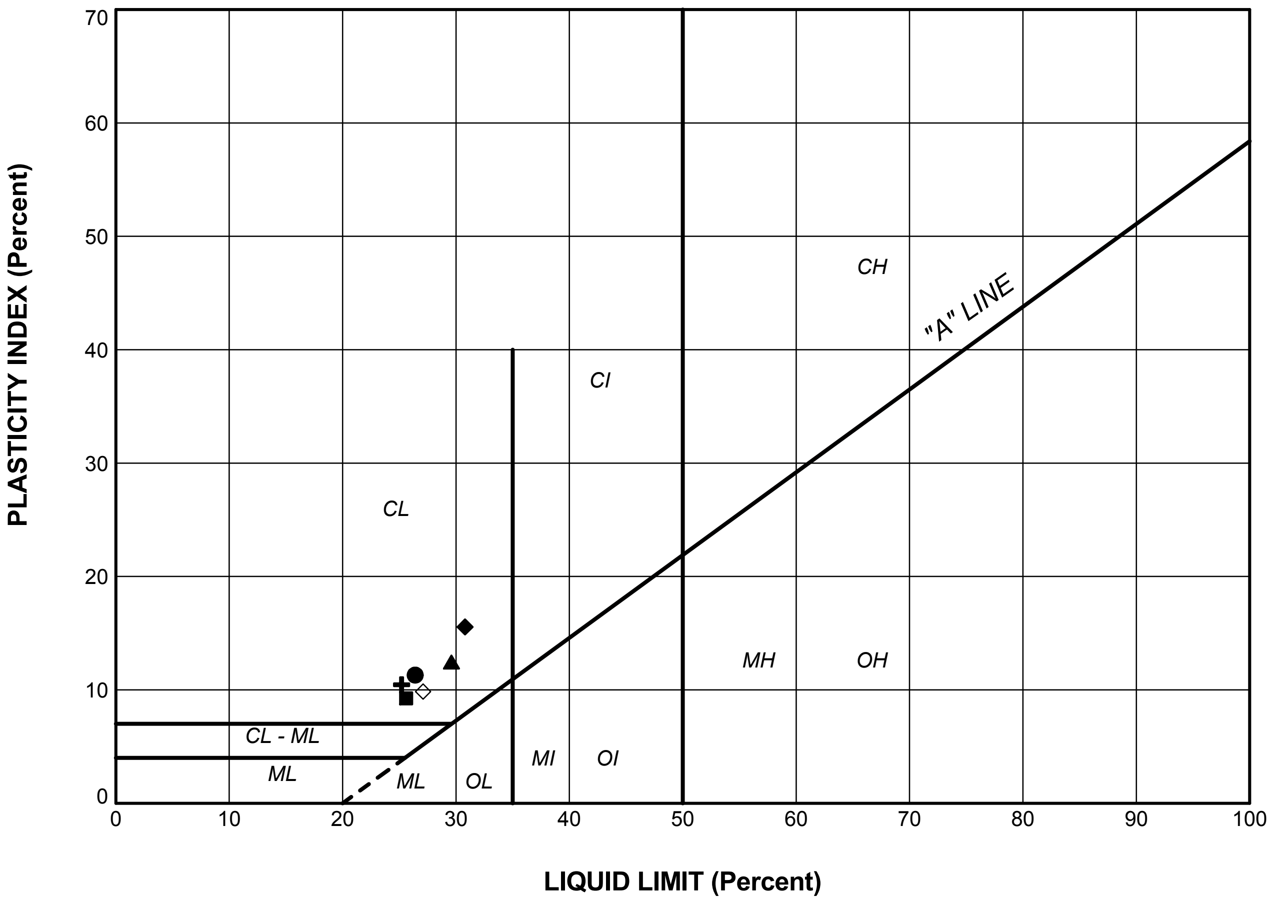
GRAIN SIZE DISTRIBUTION

SILTY SAND

Golder Associates

PROJECT No.	12-1132-0163	FILE No. 1211320163-4000-F070A5
		SCALE N/A REV.
DRAWN ZJB	Nov 4/16	FIGURE A-5
CHECK		

LDN_MTO_GSD GLDR_LDN.GDT 04/11/16



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
FILL					
✚	BH-702	3	25.2	14.8	10.5
CLAYEY SILT AND SAND					
●	BH-701	5	26.4	15.1	11.3
CLAYEY SILT TILL					
■	BH-701	8	25.6	16.4	9.3
◆	BH-702	6	30.8	15.3	15.6
CLAYEY SILT					
▲	BH-701	10	29.6	17.2	12.5
◇	BH-702	11	27.1	17.3	9.9

PROJECT
HODGINS DRAIN CULVERT REPLACEMENT, SITE NO. 2-463/C
HIGHWAY 9
GWP 3042-11-00

TITLE

PLASTICITY CHART

Golder Associates

PROJECT No.	12-1132-0163	FILE No.1211320163-4000-F070A6	
		SCALE N/A	REV.
DRAWN	ZJB/LMK	Feb 15/17	**FIGURE A-6**
CHECK			

LDN_MTO_PI GLDR_LDN.GDT 24/10/16

APPENDIX B

Site Photographs

Photograph 1: North elevation (inlet) of Culvert Site 2-463/C.

Photograph 2: South elevation (outlet) of Culvert Site 2-463/C.

Photograph 3: Looking northwest along Highway 9, north ditch toward inlet of Culvert Site 2-463/C.

Photograph 4: Looking southeast along Highway 9, south ditch toward outlet of Culvert Site 2-463/C.

APPENDIX C

Suggested Text for NSSP

NOTICE TO CONTRACTOR – Soil Conditions - Item No.

Non-Standard Special Provision

The Contractor is alerted to the presence of cobbles and boulders within the native cohesive till soils. All associated work relating to this shall be included in the applicable tender items.

BASIS OF PAYMENT

Payment at the contract price for this Tender Item shall include full compensation for all labour, equipment and material required to do the work.

n:\active\2012\1132 - geo\1132-0100\12-1132-0163 stantec-fdns-mega culverts-3011-e-0041\ph 4000-gwp 3042-11-00\rpts\r07 2-463-c hodgins drain\1211320163-4000-r07 apr 19 17 (final) app c nssp cobbles and boulders.docx

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

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