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DRAFT PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

HIGHWAY 11 - OLD BARRIE ROAD UNDERPASS REPLACEMENT
HIGHWAY 12 MEMORIAL AVENUE TO HORSESHOE VALLEY ROAD ENVIRONMENTAL ASSESSMENT
ORILLIA, ONTARIO
GWP WO 11-20002

Submitted to:
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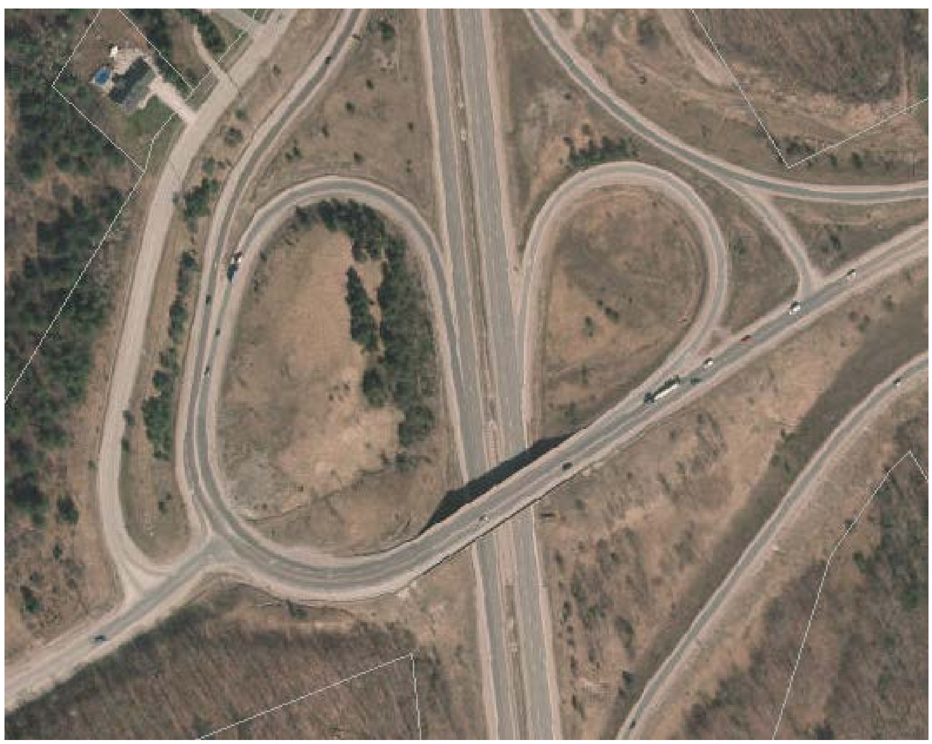
DRAFT REPORT



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PART A

PRELIMINARY FOUNDATION INVESTIGATION REPORT
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by AECOM on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the proposed replacement to the existing Highway 11 - Old Barrie Road (OBR) Underpass structure.

The terms of reference and scope of work for the foundation engineering services are outlined in Section 5.8 of MTO's Request for Proposal (RFP) for Assignment No. 2011-E-0024 dated February 2013, and in Section 5.8 of the *Technical Proposal* for this assignment.

2.0 SITE DESCRIPTION

The existing Underpass structure carries Highway 12 (Old Barrie Road - OBR) Eastbound and Westbound traffic over Highway 11, at the location shown on the Key Plan on Drawing 1.

The existing bridge was constructed in 1958 and is comprised of four approximately equal spans of 20.2 m for a total bridge length of 80.8 m. The bridge is oriented generally east / west and has a deck width of about 17.5 m, which carries four lanes of traffic. The bridge abutments are supported on 2.4 m x 0.8 m thick concrete caps of vertical and battered H-piles foundations. Each pier for the bridge comprise four 1.2 m diameter circular columns supported on a strip footing which is about 33.6 m long, 3.6 m wide, and 1.2 m thick.

3.0 INVESTIGATION PROCEDURES

3.1 Current Investigation

The field work for the subsurface investigation for the OBR Underpass was carried out between September 14 and October 1, 2015, during which time a total of three boreholes were advanced using a track-mounted drill rig, supplied and operated by Davis Drilling Ltd. of Milton, Ontario, a specialist drilling subcontractor.

The boreholes are designated as Borehole 15-S1, 15-S4, and 15-S5. Borehole 15-S5 was originally drilled adjacent to the east abutment of the existing bridge; however refusal at depth and conflicts with existing underground utilities required relocation of the borehole eastward to about 16 m from the east abutment. The locations of the three boreholes advanced at the OBR Underpass structure are shown on Drawing 1.

The boreholes were advanced to depths ranging from 13.5 m to 23.1 m below existing ground surface using hollow stem auger drilling methods, penetrating a minimum of 3 m into material which has Standard Penetration Test (SPT) 'N'-values greater than 100 blows per 0.3 m of penetration. Soil samples were obtained in the boreholes at 0.75 m and 1.5 m intervals of depth using 50 mm outer diameter split-spoon samplers driven by an automatic hammer, in accordance with the Standard Penetration Test (SPT) procedure.

The groundwater conditions were observed in the open boreholes during and immediately following the drilling operations and a standpipe piezometer was installed in Borehole 15-S1 to permit monitoring of the groundwater levels at this location. The standpipe piezometers consists of 50 mm diameter PVC pipe, with a slotted screen sealed within a sand filter pack at a selected depth interval within the borehole. The piezometer installation details and the water level recorded in the boreholes/piezometer are indicated on the borehole records contained in Appendix A. The two remaining boreholes were backfilled with bentonite upon completion, in accordance with Ontario Regulation 903 (as amended).

The field work was supervised on a full-time basis by a member of Golder's staff who observed the drilling, sampling and in situ testing operations, and logged the subsurface conditions encountered in the boreholes. The soil samples were identified in the field, placed in labelled containers and transported to Golder's laboratory in Mississauga for further examination and laboratory testing. Index and classification tests consisting of water contents, Atterberg limits and grain size distributions were carried out on selected soil samples.

The borehole locations were noted relative to identifiable site features and the coordinates and ground surface elevations were obtained from the digital terrain model provided by AECOM. The borehole locations in MTM NAD83 northing and easting coordinates, the ground surface elevations referenced to Geodetic datum and the drilled depths are summarized below and are shown on Drawing 1.

Borehole No.	NAD83 MTM Zone 10 Coordinates		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m)	Easting (m)		
15-S1	4938798.8	309031.3	268.9	20.0
15-S4	4938799.8	309083.1	260.7	13.5
15-S5	4938838.0	309128.1	266.8	23.1

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

This section of Highway 11 lies within the Simcoe Uplands, as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, Third Edition, 1984). The soil deposits are typically glacial tills comprised of sandy loam, deposited in broad, rolling plains which are separated by steeps-sided, flat-floored valleys. In some areas within the Simcoe Uplands, localized areas of sands and silts have been surficially deposited.

4.2 Subsurface Conditions

The detailed soil and groundwater conditions encountered in the boreholes, and the results of in situ and geotechnical laboratory testing, are summarized on the borehole records in Appendix A. The results of the laboratory tested samples are shown on Figures B1 to B7 in Appendix B. The stratigraphic boundaries shown on the borehole records, and on the interpreted stratigraphic profile on Drawing 1, are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole location.

In summary, the subsoils encountered in the boreholes consist of non-cohesive fill underlain by interlayered native strata comprised of sandy silt, silt and sand and gravelly silty sand, clayey silt and glacial till. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Fill

The boreholes penetrated 0.2 m of topsoil, or 0.2 m to 0.3 m of asphalt, which is underlain at each borehole location by fill materials of variable composition and deposit thickness. The elevations of the surface and base of the soil fill and the thickness of the deposit as encountered in the boreholes are summarized below.

Borehole No.	Fill Surface Elevation	Fill Thickness	Base of Fill Elevation
15-S1	268.7	8.4 m	260.3 m
15-S4	260.4	6.8 m	253.6 m
15-S5	266.6	16.0 m	250.6 m

The fill materials vary in layer thickness and composition from a 0.4 m to 1.2 m thick upper layer of gravelly sand to sand and gravel, underlain by a 6.4 m to 14.8 m thick layer of silt and sand to silty sand and sandy clayey silt. Trace organics were found in several samples of the fill materials at various depths.

The measured SPT “N”-value within the gravelly sand fill upper layer is 22 blows per 0.3 m of penetration. The measured SPT “N”-values within the silty and sand to silty sand portion of the fill materials range from 4 blows per 0.3 m of penetration to 52 blows per 0.3 m of penetration, indicating that overall the fill deposit is loose to very dense in relative density. Auger grinding was noted in the fill in Borehole 15-S5 at a depth of about 4.6 m below existing grade.

The water content of samples of the fill range from 2 per cent to 11 per cent. The results of grain size distribution tests completed on five selected samples of the fill are shown on Figure B1. Atterberg limits testing was carried out on one selected sample of the deposit and measured a plastic limit of 11 per cent, a liquid limit of 14 per cent and a plasticity index of 3 per cent. This result, which is plotted on the plasticity chart on Figure B2 in Appendix B, indicates that the tested sample of the fill consists of silt of slight plasticity.

4.2.2 Sandy Silt to Silty Gravelly Sand

An interlayered deposit of sandy silt to silty sand to silty gravelly sand was encountered below the fill in Boreholes 15-S1 and 15-S4, and below the till (described below) in Borehole 15-S5.

The elevations of the surface and base of the sandy silt to silty gravelly sand deposit and the deposit thickness encountered at the borehole locations are summarized below.

Borehole No.	Sandy Silt to Silty Gravelly Sand Surface Depth	Sandy Silt to Silty Gravelly Sand Surface Elevation	Sandy Silt to Silty Gravelly Sand Thickness	Sandy Silt to Silty Gravelly Sand Base Elevation
15-S1	8.6 m	260.3 m	4.6 m	255.7 m
	19.1 m	249.8 m	> 0.9 m	Below 248.9 m
15-S4	7.1 m	253.6 m	2.8 m	250.8 m

Borehole No.	Sandy Silt to Silty Gravelly Sand Surface Depth	Sandy Silt to Silty Gravelly Sand Surface Elevation	Sandy Silt to Silty Gravelly Sand Thickness	Sandy Silt to Silty Gravelly Sand Base Elevation
	12.6 m	248.1 m	> 0.9 m	Below 247.2 m
15-S5	21.5 m	245.3 m	> 1.6 m	Below 243.7 m

The measured SPT "N"-values within the various interlayers of the sandy silt to silty gravelly sand deposit range from 24 blows per 0.3 m of penetration to greater than 100 blows per 0.3 m of penetration, indicating this interlayered deposit is compact to very dense in relative density, and typically very dense.

The water content measured on samples of the sandy silt to silty gravelly sand deposit range from 7 per cent to 17 per cent. The results of grain size distribution tests carried out on two selected samples of the silty sand portion of the deposit from the investigation are shown on Figure B3 in Appendix B.

4.2.3 Glacial Till

A glacial till deposit was encountered underlying the silty gravelly sand in Borehole 15-S1 and below the fill in Borehole 15-S5. The till deposit is comprised of non-cohesive silty sand and silt and sand in Boreholes 15-S1 and 15-S5, and cohesive clayey silt in Borehole 15-S1; the till contains varying amounts of gravel. The till deposits in Ontario typically contain cobbles and boulders and these materials should be anticipated to be present throughout the till deposit encountered at the site.

The elevations of the surface and base of the till deposit and the deposit thickness encountered at the borehole locations are summarized below.

Borehole No.	Till Surface Depth	Till Surface Elevation	Till Thickness	Till Base Elevation
15-S1	13.2 m	255.7 m	1.5 m	254.2 m
	14.7 m	254.2 m	1.5 m	252.7 m
15-S5	16.2 m	250.6 m	5.3 m	245.3 m

The measured SPT "N" values within the non-cohesive portions of till deposit range from 38 blows to greater than 100 blows per 0.3 m of penetration, indicating this portion of the till is dense to very dense in relative density. The measured SPT "N" value within the cohesive portion of the till deposit is 51 blows per 0.3 m of penetration, suggestive a hard consistency.

The water content measured on samples of the till deposits is about 13 per cent for the clayey silt portion of the deposit and about 10 per cent for the silt and sand portion of the deposit. The results of a grain size distribution test completed a two selected sample of the clayey silt till and of the silt and sand till are shown on Figures B4 and B5, respectively.

4.2.4 Clayey Silt

A deposit of clayey silt was encountered in Borehole 15-S1 underlying the clayey silt till deposit, and in Borehole 15-S4 underlying the silty sand deposit. The elevation of the surface and base of the deposit and the thickness of the stratum as encountered in the two boreholes are summarized below.

Borehole No.	Clayey Silt Surface Depth	Clayey Silt Surface Elevation	Clayey Silt Thickness	Clayey Silt Base Elevation
15-S1	16.2 m	252.7 m	2.9 m	249.8 m
15-S4	9.9 m	250.8 m	2.7 m	248.1 m

The measured SPT "N"-values within the clayey silt deposit are all greater than 100 blows per 0.3 meters of penetration, suggesting a hard consistency. The water content measured on two samples of the clayey silt deposit is 10 per cent and 20 per cent.

The result of a grain size distribution test completed on one selected samples of the clayey silt deposit is shown on Figure B6 in Appendix B. Atterberg limits testing was carried out on one selected sample of the deposit and measured a plastic limit of 12 per cent, a liquid limit of 22 per cent and a plasticity index of 10 per cent. This result, which is plotted on the plasticity chart on Figure B7 in Appendix B, confirms that the tested sample of the deposit consists of clayey silt of low plasticity.

4.3 Groundwater Conditions

The groundwater conditions at each of the borehole locations were noted during the drilling operations. Borehole 15-S1 was dry upon completion of drilling; the standpipe piezometer installed in Borehole 15-S1 was also dry when monitored on September 30, 2015 about one week following completion of the drilling operations. The groundwater level was not able to be measured in Boreholes 15-S4 and 15-S5 on completion of the drilling operations due to the use of drilling mud within the augers.

The groundwater level at the site is expected to fluctuate seasonally in response to changes in precipitation and snow melt, and is expected to be higher during the spring and other wet periods of the year.

5.0 CLOSURE

This Foundation Investigation Report was prepared by Mr. Nick La Posta, P. Eng., and reviewed by Mr. Jorge Costa, P.Eng., a Designated MTO Foundations Contact for Golder and Principal of Golder.

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PART B

PRELIMINARY FOUNDATION DESIGN REPORT
HIGHWAY 11 - OLD BARRIE ROAD UNDERPASS REPLACEMENT
HIGHWAY 12 MEMORIAL AVENUE TO HORSESHOE VALLEY ROAD ENVIRONMENTAL ASSESSMENT
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6.0 DISCUSSION AND ENGINEERING RECOMMENDATION

6.1 General

This section of the report provides preliminary foundation design recommendations for the proposed replacement of the Old Barrie Road (OBR) Underpass Structure. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during our field investigation.

The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the preliminary design of the structure foundations. Further investigation and analysis will be required during detail design.

Where comments are made on construction, they are provided to highlight those aspects that could affect the future detail design of the project, and for which special provisions may be required in the Contract Documents. Those requiring information on the aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

The existing OBR Underpass structure carries Highway 12 Eastbound and Westbound traffic over Highway 11. The Underpass structure was constructed in 1958 and comprises four approximately equal spans of 20.2 m for a total bridge length of 80.8 m. The bridge is oriented east / west and has a deck width of about 17.5 m, which carries four lanes of traffic. The bridge abutments are supported on square 2.4 m x 0.8 m thick concrete pads of vertical and battered H-piles foundations. Each pier of the bridge is comprised of four 1.2 m diameter columns and the piers are supported on a strip footing which are about 33.6 m long, 3.6 m wide, and 1.2 m thick.

As indicated by AECOM in the draft structural design report, the preferred option is to replace the existing bridge with a new two span steel box girder structure. The single central pier will comprise five individual elliptical columns, each supporting one of the steel box girders. RSS wingwalls will be constructed at each corner of the abutments.

6.2 Overview of Foundation Options for Structure Replacement

Based on the preliminary drawings provided by AECOM, it is understood that the preferred replacement alternative for the existing Underpass structure is a two-span structure, with the eastbound lanes positioned on the same alignment as the existing structure and a new section to accommodate the westbound lanes.

Based on the proposed Underpass geometry and the subsurface conditions at this site, deep foundation are considered the preferred option for support of the abutments and centre pier for replacement of the OBR Underpass. Given the thickness of the existing fill soils at the structure location, it is anticipated that shallow spread/strip foundations would be required to be founded on a granular pad constructed within the existing or new fill materials, or on the native subgrade at great depths below the existing fill at the site. In either case, relatively low axial resistances would be available for these shallow foundations and as such, they are not considered feasible due to the depth to native materials. Foundations comprised of strip or spread footings founded on the existing fill soils at the site are not recommended due to potential for differential settlement issues.

A summary of the advantages and disadvantages associated with each option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, risks and relative costs is provided in Table 1 following the text of this report.

- **Driven steel H-piles:** Driven steel H-piles are suitable and feasible for support of the abutments and centre pier, and would permit integral abutment design. The use of driving shoes is recommended due to the hard / very dense nature of the soils and the potential presence of cobbles and boulders in the glacially derived soils.
- **Driven steel pipe (tube) piles:** Steel tube (pipe) piles could also be considered as a deep foundation option for support of the abutments and piers. However, pipe piles are considered to have a higher risk than H-piles for “hanging up” or being deflected away from their vertical or battered orientation due to the presence of cobbles and/or boulders within the glacially-derived soils at this site. Further, pipe piles do not permit the construction of integral abutments, except in specific conditions of very soft soil deposits.
- **Caissons:** Caissons are also feasible for structure support at this site as groundwater was not encountered during the preliminary geotechnical investigation. However, temporary or permanent liners would still be recommended to mitigate the potential risk for the non-cohesive granular soils caving into the drilled hole through which the caissons would be constructed.

The following sections provide recommendations for deep foundations to support the proposed works. From a foundations perspective, based on the above considerations and comparison of alternatives in Table 1, and the soil conditions at the site, the preferred option for a replacement structure from a geotechnical/foundations perspective is to support the abutments and centre pier on driven H-pile foundations.

6.2.1 Driven Steel H-Pile or Steel Pipe Pile Foundations

6.2.1.1 Founding Elevations

The abutments and centre pier for the replacement structure may be supported on steel H-piles or steel pipe piles driven to found within the “100-blow” clayey silt/silty sand deposits or within the silty sand till/sandy silt deposits, encountered in the boreholes. The following pile tip elevations may be used for preliminary design purposes, assuming about 2 m to 2.5 m of penetration into the “100-blow” deposit(s).

Foundation Element	Borehole	Estimated Design Pile Tip Elevation (Stratum)
West abutment	15-S1	250.0 m (Clayey Silt)
Centre pier	15-S4	248.0 m (Clayey Silt/Silty Sand)
East abutment	15-S5	246.0 m (Silt and Sand Till)

Based on OPSD 3090.101 (Frost Penetration Depths for Southern Ontario), the pile caps should be constructed at a minimum depth of 1.7 m below final ground surface for frost protection purposes.

For the installation of steel H-piles or steel pipe piles, consideration must be given to the potential presence of cobbles and/or boulders within the soil deposits. In this regard, steel H-piles are preferred over steel pipe piles as pipe piles are considered to pose a higher risk of “hanging up” or being deflected away from their vertical or battered orientation during installation, due to their larger end area. The piles should be reinforced at the tip with driving shoes or flange plates to reduce the potential for damage to the piles during driving, in accordance with OPSS 903 *(Deep Foundations).* In the very dense / hard soils which will be encountered at this site, driving shoes (such as OPSD 3000.100 or Titus Standard "H" Bearing Pile Points) are preferred over flange plates.

6.2.1.2 Axial Geotechnical Resistance

For HP 310x110 piles driven to the estimated tip elevations provided in Section 6.2.1.1, the factored axial resistance at ULS and the axial geotechnical reaction at SLS (for 25 mm of settlement) may be taken as follows for preliminary design:

Foundation Element	Borehole No.	Factored Geotechnical Resistance at ULS	Geotechnical Resistance at SLS (for 25 mm of Settlement)
West abutment	15-S1	1,600 kN	1,400 kN
Centre pier	15-S4	1,600 kN	1,400 kN
East abutment	15-S5	1,600 kN	1,400 kN

For closed-end, concrete-filled, 324 mm (12 ¾ in.) diameter steel pipe piles having a minimum wall thickness of 9.5 mm (3/8 in.), driven to the estimated tip elevations provided in Section 6.2.1.1, the factored axial resistance at ULS and the axial geotechnical reaction at SLS (for 25 mm of settlement) may be taken as follows for preliminary design:

Foundation Element	Borehole No.	Factored Geotechnical Resistance at ULS	Geotechnical Resistance at SLS (for 25 mm of Settlement)
West abutment	15-S1	1,600 kN	1,400 kN
Centre pier	15-S4	1,600 kN	1,400 kN
East abutment	15-S5	1,600 kN	1,400 kN

The preliminary geotechnical resistances provided above will have to be re-evaluated and modified as necessary during detail design in consideration of the additional subsurface investigation that will be carried out at the foundation elements.

6.2.2 Drilled Piers (Caissons)

The OBR structure could also be supported on drilled piers (caissons) founded within the hard / very dense soils at the estimated elevations provided in Section 6.2.1.1.

The performance of caissons will depend upon the final cleaning and verification of the quality of the soil at the base. Each caisson excavation must be carefully cleaned to remove all loose materials to ensure that the concrete is in intimate contact with competent bearing stratum. A temporary liner should be utilized to support the sides of the caisson excavations during drilling, cleaning and concrete placement. The till materials should be expected to contain cobbles and/or boulders which may pose difficulties during the advancement of the caissons and/or temporary liners. The concrete must be placed using tremie techniques (i.e. the concrete must be discharged at the base of the caisson excavations, and flow upward to the ground surface). The tremie discharge should be maintained a minimum of 1 m below the surface of the wet concrete during placement. All caisson caps should be provided with a minimum of 1.5 m of soil cover for frost protection.

The factored axial geotechnical resistance at Ultimate Limits States (ULS) for caissons with their tip founded in hard / very dense soils may be taken as follows:

Foundation Element	Borehole Nos.	Diameter	Factored Geotechnical Resistance at ULS	Geotechnical Resistance at SLS (for 25 mm of Settlement)
West Abutment	15-S1	0.6 m	1,900 kN	Not Required
		0.9 m	3,800 kN	Not Required
Centre Pier	15-S4	0.6 m	1,900 kN	Not Required
		0.9 m	3,800 kN	Not Required
East Abutment	15-S5	0.6 m	1,900 kN	Not Required
		0.9 m	3,800 kN	Not Required

The geotechnical reaction at SLS for 25 mm of settlement will exceed the ULS value generated from the given above, and therefore the ULS value will govern the caisson foundation design.

The bearing soil and fresh concrete should be protected from freezing during cold weather construction.

6.3 Lateral Earth Pressures

The lateral earth pressures acting on the abutment stems and any associated wing walls will depend on the type and method of placement of the backfill materials, on the nature of the soils 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 stems/wing walls. It should be noted that these design recommendations and parameters assume a level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select, free draining granular fill meeting the specifications of OPSS.PROV 1010 (Aggregates) Granular 'A' or Granular 'B' Type II should be used as backfill behind the walls. This fill should be compacted in accordance with OPSS.PROV 501 (Compacting). Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub drains and frost taper should be in accordance with OPSD 3101.150 (Walls Abutment, Backfill) and 3121.150 (Walls Retaining, Backfill).

- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with CHBDC Section 6.9.3 and Figure 6.6. Compaction equipment should be used in accordance with OPSS.PROV 510 (Compacting). Other surcharge loadings should be accounted for in the design, as required.
- The granular fill may be placed either in a zone with the width equal to at least 1.5 m behind the back of the walls for a restrained wall (Figure C6.20 (a) of the Commentary to the CHBDC), or within the wedge shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing for an unrestrained wall (Figure C6.20(b) of the Commentary to the CHBDC).
- For a restrained wall, the pressures are based on the proposed embankment fill materials and the existing native soils and the following parameters (unfactored) may be used assuming the use of granular earth fill such as OPSS.PROV 1010 (Aggregates) Select Subgrade Material (SSM) for embankment construction:

Unfactored Parameters		New Earth Fill
Soil unit weight:		21 kN/m^3
Coefficients of static lateral earth pressure:	At rest, K_o	0.47
	Active, K_a	0.31

- For an unrestrained wall where the pressures are based on OPSS.PROV 1010 (Aggregates) Granular 'A' or Granular 'B' Type II fill behind the wall, the following parameters (unfactored) may be assumed:

Unfactored Parameters		Granular A	Granular B Type II
Soil unit weight:		22 kN/m^3	21 kN/m^3
Coefficients of static lateral earth pressure:	At rest, K_o	0.43	0.43
	Active, K_a	0.27	0.27

If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design. The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.9.1 and Table C6.6 of the Commentary to the CHBDC.

A restrained structure is typically a concrete box culvert or a rigid frame bridge structure where the rotational and/or horizontal movement is not sufficient to mobilize the active pressure condition. For this condition, an at-rest pressure plus any compaction surcharge should be included in the design of the structure.

6.4 Seismic Considerations

6.4.1 Site Coefficient

For seismic design purposes, the Site Coefficient, S, for this site, based on experience and considering the guidelines in Section 4.4.6 of the CHBDC may be taken as 1.2, consistent with Soil Profile Type II.

6.4.2 Seismic Analysis Coefficient

The potential for seismic (earthquake) loading may also need to be considered for the design of abutment stems/wing walls/retaining walls and for the assessment of liquefaction potential of foundation soils in accordance with Section 4.6 of the CHBDC, as significant seismic loading will result in increased lateral earth pressures acting on the abutment stem and retaining walls.

According to Table A3.1.1 of the CHBDC (2006), this site is located in Seismic Zone 1. Based on the National Building Code (2010), the site-specific zonal acceleration ratio for Orillia is 0.05. Based on experience for the subsurface conditions at this site, a 20 percent amplification of the ground motion may occur (i.e. Site Coefficient, S=1.2 for Soil Profile II from Table 4.4 of CHBDC), resulting in an increase in the peak horizontal ground acceleration (PHA) from 0.05 g to 0.06 g at the ground surface.

Based on Section 4.4.4 of the CHBDC, this bridge structure is assigned Seismic Performance Zone 1. Given that the proposed structure is not designated as a lifeline or truss bridge, and in accordance with Section 4.4.5.3 (Table 4.2) of the CHBDC, no seismic analysis is required for structures located in Seismic Performance Zone 1.

6.5 Approach Embankments

Based on the GA drawings, the existing approach embankments will be widened along the north side as part of the bridge replacement works. For preliminary assessment purposes, side slopes inclined no steeper than 2 Horizontal to 1 Vertical (2H:1V) can be considered assuming appropriate subgrade preparation and proper placement and compaction of the embankment fill materials will be carried out. A review of the stability of the approach embankments should be carried out and the required inclination confirmed based on the subsoil conditions encountered within the proposed approach embankment widening footprint during detail design.

In addition to the global stability of the approach embankments, the magnitude of settlement under the widened approach embankments should be assessed based on the soil and groundwater conditions as determined during the detail design, with particular emphasis on the thickness and properties of the existing fill within the embankment widening footprint at the site.

6.6 Construction Considerations

The following subsections identify future construction issues that should be considered at this stage as they may impact the planning and preliminary design. Where applicable, Non-Standard Special Provisions (NSSP) should be developed during detail design for incorporation into the Contract Documents.

6.6.1 Excavation and Temporary Protection Systems

Open-cut excavations into the soils at the site should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. The existing fill and the compact silty sand to sandy silt portions of the native soils would be classified as Type 3 soil; the dense silty sand till is classified as Type 2 soil, and the hard clayey silt and cohesive till is classified as Type 1 soil, according to the

OHSA. Temporary excavations (i.e. those that are open for a relatively short time period) into Type 3 soil should be made with side slopes no steeper than 1H:1V; whereas temporary excavations into Type 1 and Type 2 soils, if required, may be made with side slopes no steeper than 1H:1V to within 1.2 m of the bottom of the excavation.

If temporary excavation protection is required as part of the Underpass replacement, the temporary excavation support system should be designed and constructed in accordance with OPSS.PROV 539 *(Temporary Protection Systems)*. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS.PROV 539.

It is considered that either a driven, interlocking sheet pile system or a soldier pile and timber lagging system would be suitable for the temporary excavation support for a replacement structure, based on the subsurface soil and groundwater conditions, although it may likely be difficult to vibrate/drive steel sheet piles adequately into the dense deposits.

6.6.2 Groundwater Control

During the drilling operations, groundwater was not noted in any of the boreholes. Further, the standpipe piezometer in Borehole 15-S1 measured dry to the bottom of the screen about one week following completion of the drilling. As such, groundwater is not anticipated to be problematic at the site and a Permit to Take Water (PTTW) would not be required for groundwater control system at this site.

6.6.3 Obstructions

The soils at this site are glacially derived and the fill deposit is comprised of variable materials, and as such should be expected to contain cobbles and boulders, as inferred by auger grinding as noted on the borehole records, which could affect the installation of deep foundations or protection systems. The frequency of the occurrence of cobbles and boulders should be identified during future investigations as part of the detail design. If conditions warrant, an NSSP should be included in the Contract Documents developed during the detail design stage to identify to the contractor the possible presence of cobbles and/or boulders within the overburden soils and fill materials.

6.6.4 Existing Structure Foundations

Assuming a full structure replacement is adopted consideration will need to be given to potential conflicts between the existing and new foundation elements, particularly at the existing pile-supported abutments. Temporary protection systems would be required for removal of the existing pile cap and to facilitate cutting off the existing piles (where necessary) a minimum of 0.5 m below the founding level of the new abutment elements.

6.6.5 Vibration Monitoring During Pile Installation

Vibration levels up to a maximum peak particle velocity (PPV) of 100 mm/s are generally considered tolerable for bridge structures in good condition. Based on vibration monitoring experience, it is considered unlikely that vibrations induced by conventional construction activities (such as pile driving) will reach this threshold level and, therefore, vibration monitoring for the existing structure is not expected to be required during construction at this site.

6.7 Recommendations for Further Work in Detail Design

Additional boreholes are recommended within each of the foundation areas (abutments and centre pier), any new retaining wall footprints, and the approach embankment areas during the future detail design stage of the works, to further assess and/or confirm the subsurface conditions and the preliminary recommendations provided in this report, including:

- Assessment of the fill thickness at each of the abutment and centre pier locations, in particular the foundation elements for the northern portion of the bridge structure.
- Assessment of groundwater conditions(s) by the installation of an additional standpipe piezometer.
- Assessment of the global stability at the widened approach embankment locations.
- Assessment of the settlement of the subsurface/foundation soils under the widened approach embankment footprints.
- Observation of the presence of cobbles and/or boulders within the fill and native soil deposits, to assess the need for an NSSP to warn the contractor of the presence of such obstructions as they may affect excavations and the installation of driven foundations or temporary protection systems.

7.0 CLOSURE

This Foundation Design Report was prepared by Mr. Nick La Posta, P. Eng., and reviewed by Mr. Jorge Costa, P.Eng., a Designated MTO Foundations Contact for Golder and Principal of Golder.

GOLDER ASSOCIATES LTD.

Nick La Posta, P. Eng. Geotechnical Engineer	Jorge M.A. Costa, P.Eng. Designated MTO Foundations Contact, Principal

NL/JMAC/sm

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REFERENCES

Canadian Geotechnical Society, 2006. *Canadian Foundation Engineering Manual*, 4th Edition. The Canadian Geotechnical Society, BiTech Publisher Ltd., British Columbia.

Canadian Highway Bridge Design Code (CHBDC) and Commentary, Canadian Standards Association Special Publication CAN/CSA S6 06-06.

Canadian Standards Association (CSA), 2006. *Canadian Highway Bridge Design Code* and *Commentary on CAN/CSA S6 06*. CSA Special Publication, S6.1 06.

Chapman, L.J., and Putnam, D.F., 1984. *The Physiography of Southern Ontario*, 3rd Edition. Ontario Geological Survey, Special Volume 2. Ontario Ministry of Natural Resources.

Ontario Provincial Standard Specifications (OPSS)

OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material

Ontario Provincial Standard Drawings (OPSD)

OPSD 3000.100	Foundation, Piles, Steel H-Pile Driving Shoe
OPSD 3101.150	Walls, Abutments, Backfill, Minimum Granular Requirements
OPSD 3090.101	Foundation, Frost Penetration Depths for Southern Ontario
OPSD 3121.150	Walls, Retaining, Backfill, Minimum Granular Requirements

TABLE 1 – COMPARISON OF FOUNDATION ALTERNATIVES FOR STRUCTURE REPLACEMENT

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Strip or spread footings founded on a granular pad, constructed within the existing fill	• Not feasible to support either the abutments or the pier on fill due to the variable strength/thickness of the fill at the site	• Relative ease of construction • Relatively minor groundwater seepage anticipated • Lower vibration impacts on existing structures than for driven pile installation	• Much lower geotechnical resistances as compared with deep foundations • Potential for differential settlement under a foundation element and between foundation elements due to the variable strength/thickness of the fill layers. • Precludes use of integral abutments; potentially greater maintenance required at abutments • Potentially deep excavation would be required to found the granular pad on suitable fill layer.	• Conventional excavation and construction techniques	• Less expensive than deep foundations although bridge maintenance costs would be higher due to both settlement repairs and non-integral abutment configuration • Estimated cost is about $600/m^3 for a concrete unit for construction of shallow foundations, excluding deeper excavation and temporary protection system

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Steel H-piles driven into the “100-blow” soils	• Feasible for support of new abutments and centre pier	• Allows for integral abutment construction • Abutment pile caps could be maintained higher than footings founded on shallow foundations constructed on a granular pad • Minimal groundwater control required • Would minimize differential settlement between foundation elements • Most cost effective deep foundation alternative	• Potential for encountering obstructions (cobbles and/or boulders) during pile driving; this could result in piles “hanging up” and achieving lower geotechnical resistances • Existing abutment foundations must be removed to allow for pile installation	• Conventional construction methods for H-pile foundations	• Lower relative cost compared with caisson option • Estimated unit cost is approximately $250/linear metre for pile installation and $600/m^3 for pile cap construction
Steel pipe (tube) piles, driven into the “100-blow” soils	• Feasible for support of new abutments and centre pier	• Abutment pile caps could be maintained higher than footings founded on shallow foundations constructed on a granular pad • Minimal groundwater control required • Would minimize differential settlement between foundation elements	• Greater risk of piles “hanging up” or deviating from required batter/plumbness than for steel H-pile foundations if obstructions (cobbles and/or boulders) are encountered during driving; piles “hanging up” would achieve lower geotechnical resistances • Potential conflicts with existing centre pier foundations • Typically not acceptable for integral abutment construction	• Conventional construction methods	• Costs for steel pipe (tube) piles slightly higher than for steel H-piles

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Caissons founded within the hard “100-blow” soils	• Feasible but not recommended for support of abutments and centre pier	• Abutment and pier pile caps could be constructed at the underside of the bridge • Higher capacity than for steel H-piles, so reduced number of deep foundation elements compared to steel H-piles	• Potential for loss of ground if perched water is encountered within the fill deposit, and in water-bearing cohesionless lenses which are anticipated to be encountered in the cohesive units • Temporary or permanent liners would be required; likely not possible to inspect caisson base • Precludes use of integral abutments	• Conventional construction methods	• Higher cost compared with shallow foundations or steel H-piles

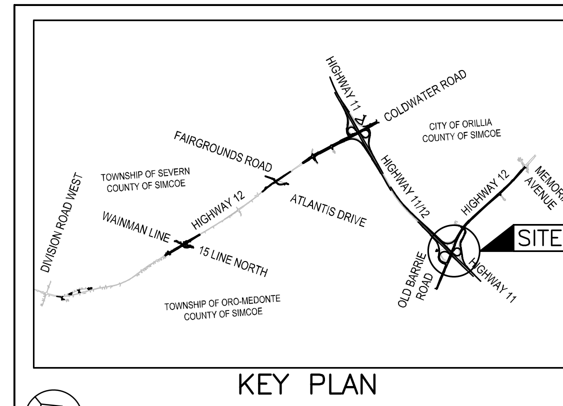
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

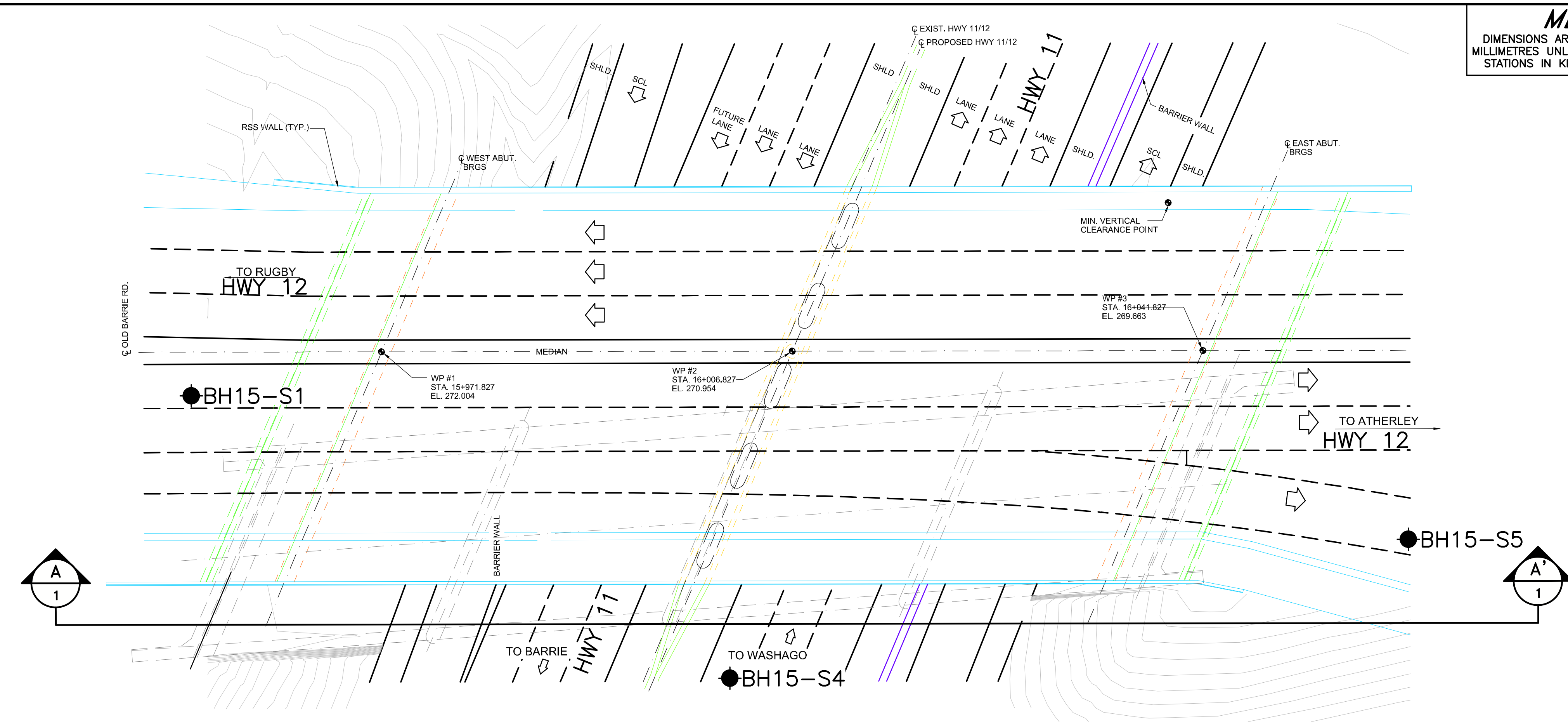
CONT No.
WP No. 011–20002

SHEET

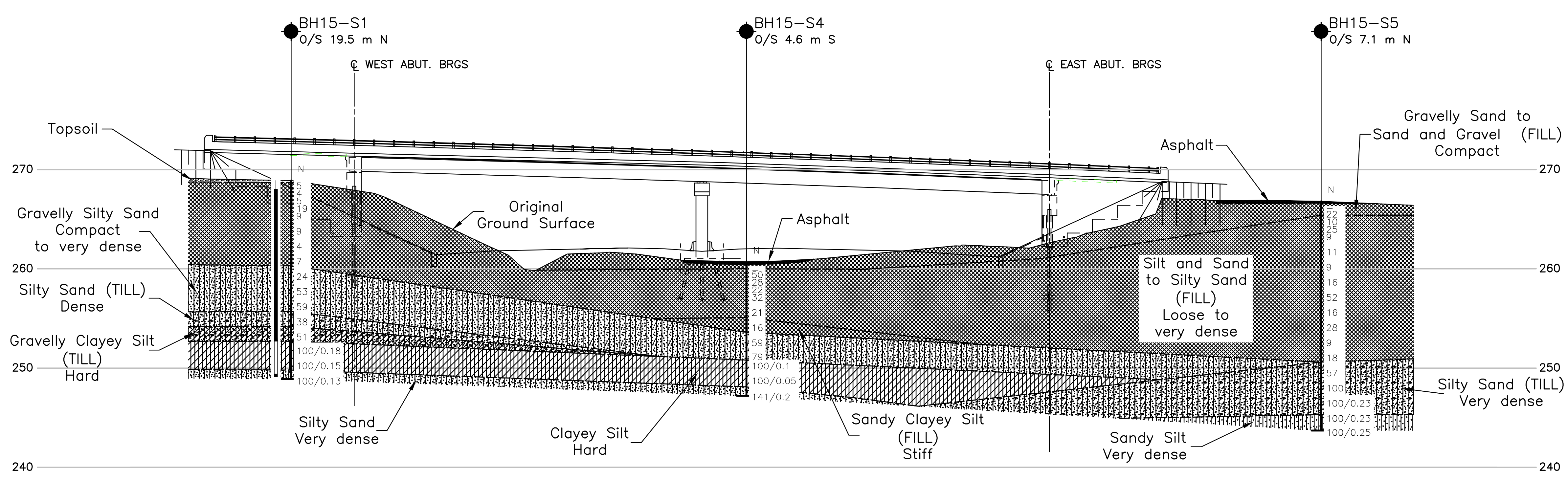
HIGHWAY 12
OLD BARRIE ROAD BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates





PLAN
SCALE
5 0 5 10 m



HORIZONTAL: 1:250 (A–A / 1) SECTION
VERTICAL 1:250

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 in piezometer, measured on Sept 30, 2015
- WL upon completion of drilling

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
BH15–S1	268.9	4938798.8	309031.3
BH15–S4	260.7	4938799.8	309083.1
BH15–S5	266.8	4938838.0	309128.1

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

Base plans provided in digital format by AECOM, drawing file no. 01–Old Barrie Road_GA.dwg, received OCT 29, 2015.

NO.	DATE	BY	REVISION
.	.	.	

Geocres No.

HWY. 12		PROJECT NO. 13–1111–0026	DIST.
SUBM'D.	CHKD.	DATE: 1/18/2016	SITE:
DRAWN: TB	CHKD. NL	APPD. JMAC	DWG. 1

DRAFT

MINISTRY OF TRANSPORTATION, ONTARIO

PLOT DATE: January 18, 2016
FILENAME: H:\Projects\2013\13-1111-0026 ...

APPENDIX A

Borehole Records

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 $= \tan\delta$
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 With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand

Golder Associates

RECORD OF BOREHOLE No BH15-S1

METRIC

PROJECT	13-1111-0026		
W.P.	011-20002	LOCATION	N 4938798.8; E 309031.3
DIST	CENTRAL HWY 12	BOREHOLE TYPE	200 mm Diameter Hollow Stem Augers
DATUM	GEODETIC	DATE	September 22, 2015
ORIGINATED BY	DM	COMPILED BY	NL
CHECKED BY	JMAC		

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
268.9	GROUND SURFACE
0.0	TOPSOIL
0.2	Silt and sand, trace clay, trace to some gravel (FILL) Loose to compact Brown Moist Trace fibrous organics from 0.2 m to 0.5 m depth. Some black mineral fragments at 4.6 m depth.
260.3 8.6	Gravelly Silty SAND, trace clay, some rock fragments Compact to very dense Brown Moist to wet Cobbles and boulders inferred from auger grinding and rock fragments in samples.

Continued Next Page

SAMPLES

NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
1A / 1B	SS	5				
2	SS	4				
3	SS	5				
4	SS	19				
5	SS	9	6	52	38	4
6	SS	9				
7	SS	4				
8	SS	7				
9	SS	24	23	54	22	1
10	SS	53				

STRAT PLOT | GROUND WATER CONDITIONS | ELEVATION SCALE: 268, 267, 266, 265, 264, 263, 262, 261, 260, 259, 258, 257

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED — 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

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

Golder Associates

RECORD OF BOREHOLE No BH15-S1

 METRIC

PROJECT 13-1111-0026
W.P. 011-20002
DIST CENTRAL HWY 12
DATUM GEODETIC
LOCATION N 4938798.8; E 309031.3
BOREHOLE TYPE 200 mm Diameter Hollow Stem Augers
DATE September 22, 2015
ORIGINATED BY DM
COMPILED BY NL
CHECKED BY JMAC

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
		11	SS	59	
255.7 / 13.2	SILTY SAND, some gravel, trace clay (TILL) Dense Brown Moist	12	SS	38	
254.2 / 14.7	Gravelly CLAYEY SILT with SAND (TILL) Hard Brown Moist	13	SS	51	24 34 26 16
252.7 / 16.2	CLAYEY SILT, trace sand Hard Brown Moist	14	SS	100/0.18	
		15	SS	100/0.15	
249.8 / 19.1	SILTY SAND, trace gravel Very dense Brown Moist	16	SS	100/0.13	
248.9 / 20.0	END OF BOREHOLE				

Note:

1. Borehole dry upon completion of drilling.

2. Piezometer dry on September 30, 2015.

Elevation scale: 256, 255, 254, 253, 252, 251, 250, 249

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 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

DRAFT

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

RECORD OF BOREHOLE No BH15-S4

 METRIC

PROJECT 13-1111-0026
W.P. 011-20002
DIST CENTRAL HWY 12
DATUM GEODETIC

LOCATION N 4938799.8; E 309083.1
BOREHOLE TYPE 200 mm Diameter Hollow Stem Augers
DATE September 14, 2015

ORIGINATED BY DM
COMPILED BY NL
CHECKED BY JMAC

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED: 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

Soil Profile

ELEV / DEPTH	DESCRIPTION
260.7	GROUND SURFACE
0.0	ASPHALT (330 mm)
260.4 / 0.3	Sand and gravel, trace silt (FILL) Brown Moist
260.0 / 0.7	Silt and sand to silty sand, some gravel, trace clay, trace organics (FILL) Compact to very dense Brown to grey Moist Presence of cobbles/boulders inferred by grinding at 1.5 m depth. Compact to dense below 1.5 m depth.
255.1 / 5.6	Sandy clayey silt, trace gravel, trace organics (FILL) Stiff Brown to grey, mottled Moist
253.6 / 7.1	Silty SAND, some gravel, trace to some clay, trace organics Very dense Brown to grey Moist
250.8 / 9.9	CLAYEY SILT, some sand, trace gravel Hard Brown Moist

Samples

NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
1	AS	-				
2	SS	50				
3	SS	28				
4	SS	22	3	55	38	4
5	SS	32				
6	SS	21				
7	SS	16				
8	SS	59	20	47	26	7
9	SS	79				
10	SS	100/0.1	3	22	62	13
11	SS	100/0.05				

ELEVATION SCALE: 260, 259, 258, 257, 256, 255, 254, 253, 252, 251, 250, 249

Continued Next Page

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

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RECORD OF BOREHOLE No BH15-S4

 METRIC

PROJECT 13-1111-0026

W.P. 011-20002 | LOCATION N 4938799.8; E 309083.1 | ORIGINATED BY DM

DIST CENTRAL HWY 12 | BOREHOLE TYPE 200 mm Diameter Hollow Stem Augers | COMPILED BY NL

DATUM GEODETIC | DATE September 14, 2015 | CHECKED BY JMAC

ELEV DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) 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
	--- CONTINUED FROM PREVIOUS PAGE ---										
248.1 12.6	Silty SAND Very dense Brown Moist		12	SS	141/0.2		248		○		
247.2 13.5	END OF BOREHOLE Note: 1. Water level could not be observed upon completion of drilling due to use of drilling mud in augers.										

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

RECORD OF BOREHOLE No BH15-S5

 METRIC

PROJECT 13-1111-0026			ORIGINATED BY DM
W.P. 011-20002	LOCATION N 4938838.0; E 309128.1		
DIST CENTRAL HWY 12	BOREHOLE TYPE 200 mm Diameter Hollow Stem Augers		COMPILED BY NL
DATUM GEODETIC	DATE October 1, 2015		CHECKED BY JMAC

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
266.8	GROUND SURFACE									
0.0	ASPHALT (230 mm)									
0.2	Gravelly sand, trace silt, trace cobbles (FILL) Compact Brown Moist	1	AS	-						
		2	SS	22	266	○				
265.4 / 1.4	Silt and sand to silty sand, trace to some clay, some gravel to gravelly (FILL) Loose to very dense Brown Moist	3	SS	10	265					
		4	SS	25	264	○	16	47	30	7
		5	SS	9	263					
	Auger grinding from 4.6 m depth.	6	SS	11	262	○				
		7	SS	9	261, 260					
		8	SS	16	259	○	23	44	26	7
		9	SS	52	258, 257					
		10	SS	16	256, 255	○				

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 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; STRAT PLOT.

Continued Next Page

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

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RECORD OF BOREHOLE No BH15-S5

 METRIC

PROJECT 13-1111-0026
W.P. 011-20002
DIST CENTRAL HWY 12
DATUM GEODETIC
LOCATION N 4938838.0; E 309128.1
BOREHOLE TYPE 200 mm Diameter Hollow Stem Augers
DATE October 1, 2015
ORIGINATED BY DM
COMPILED BY NL
CHECKED BY JMAC

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	GR	SA	SI	CL
	Silt and sand to silty sand, trace to some clay, some gravel to gravelly (FILL) Loose to very dense Brown Moist Minor organic staining, trace fibrous organics observed at 12.2 m depth.	11	SS	28	254				
		12	SS	9	253	16	51	26	7
	Organic staining, some fibrous organics observed at 15.2 m depth.	13	SS	18	252, 251				
250.6 / 16.2	SILT and SAND, trace to some gravel, trace to some clay (TILL) Very dense Grey Moist	14	SS	57	250	4	60	31	5
		15	SS	100	249, 248				
	A 150 mm thick silty sand pocket observed at 20.0 m depth.	16	SS	100/0.23	247, 246				
245.3 / 21.5	Sandy SILT Very dense Brown Moist	17 A / B	SS	100/0.23	245				
		18	SS	100/0.25	244				
243.7 / 23.1									

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 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

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No BH15-S5

METRIC

PROJECT 13-1111-0026			
W.P. 011-20002	LOCATION N 4938838.0; E 309128.1		ORIGINATED BY DM
DIST CENTRAL HWY 12	BOREHOLE TYPE 200 mm Diameter Hollow Stem Augers		COMPILED BY NL
DATUM GEODETIC	DATE October 1, 2015		CHECKED BY JMAC

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 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
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
	--- CONTINUED FROM PREVIOUS PAGE ---										
	END OF BOREHOLE Note: 1. Water level could not be observed upon completion of drilling due to the use of drilling mud.										

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

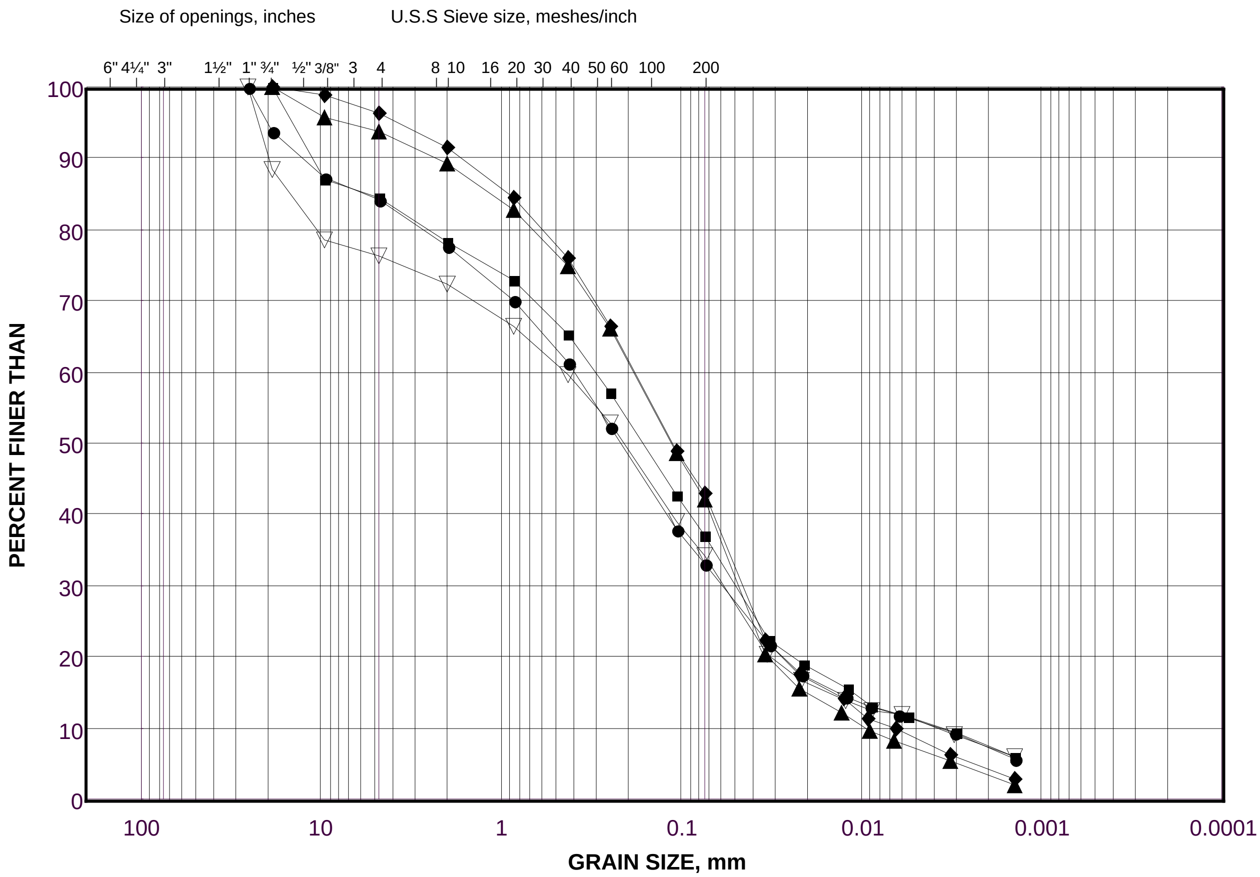
APPENDIX B

Geotechnical Laboratory Test Results

GRAIN SIZE DISTRIBUTION

SILT and SAND (FILL)

FIGURE B1



COBBLE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
SIZE	GRAVEL SIZE		SAND SIZE			FINE GRAINED

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	15-S5	12	252.9
■	15-S5	4	264.3
◆	15-S4	4	258.2
▲	15-S1	5	265.7
▽	15-S5	8	258.9

Project Number: 13-1111-0026

Checked By: NL

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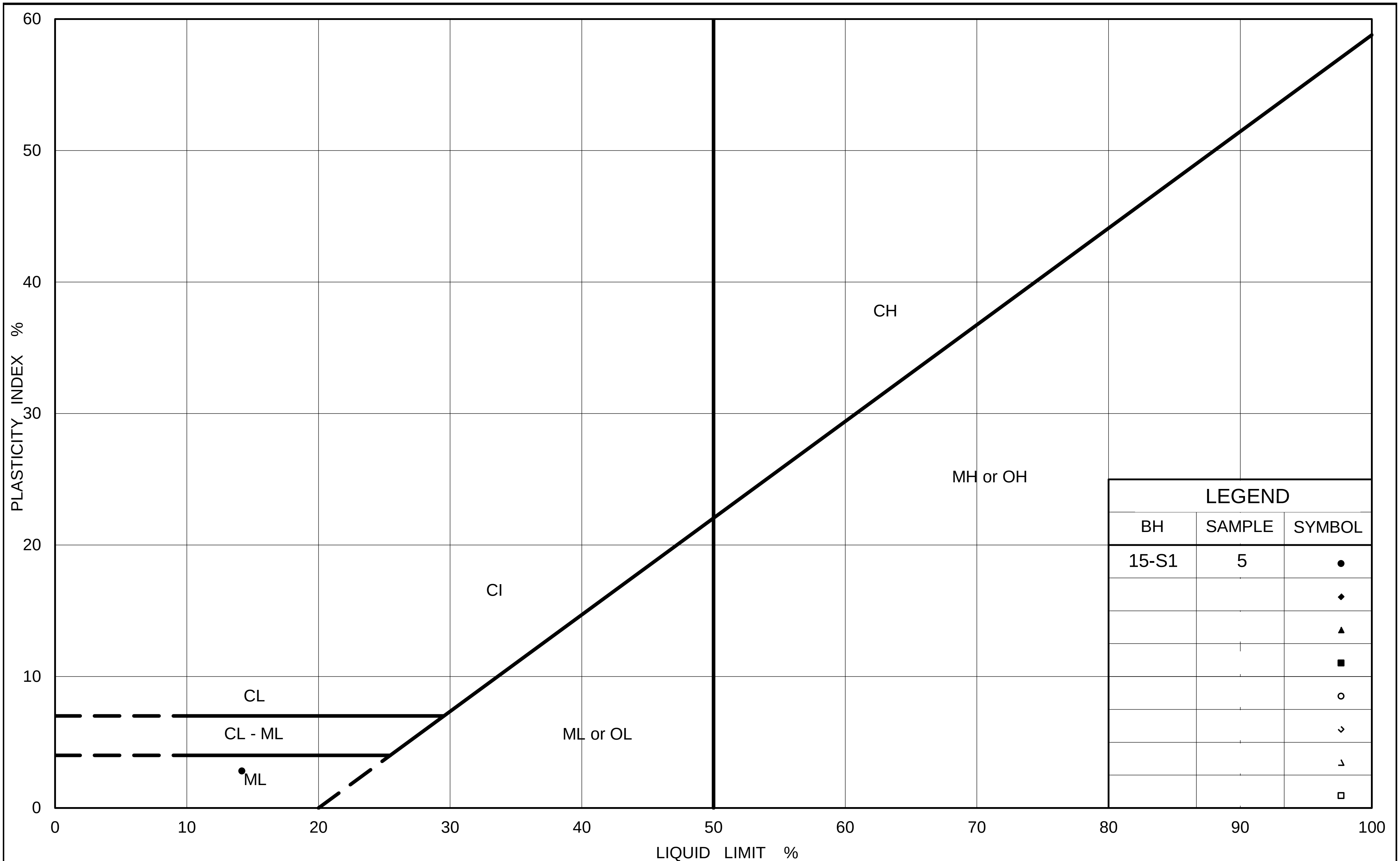
Date: 20-Jan-16



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PLASTICITY CHART
SILT and (FILL)

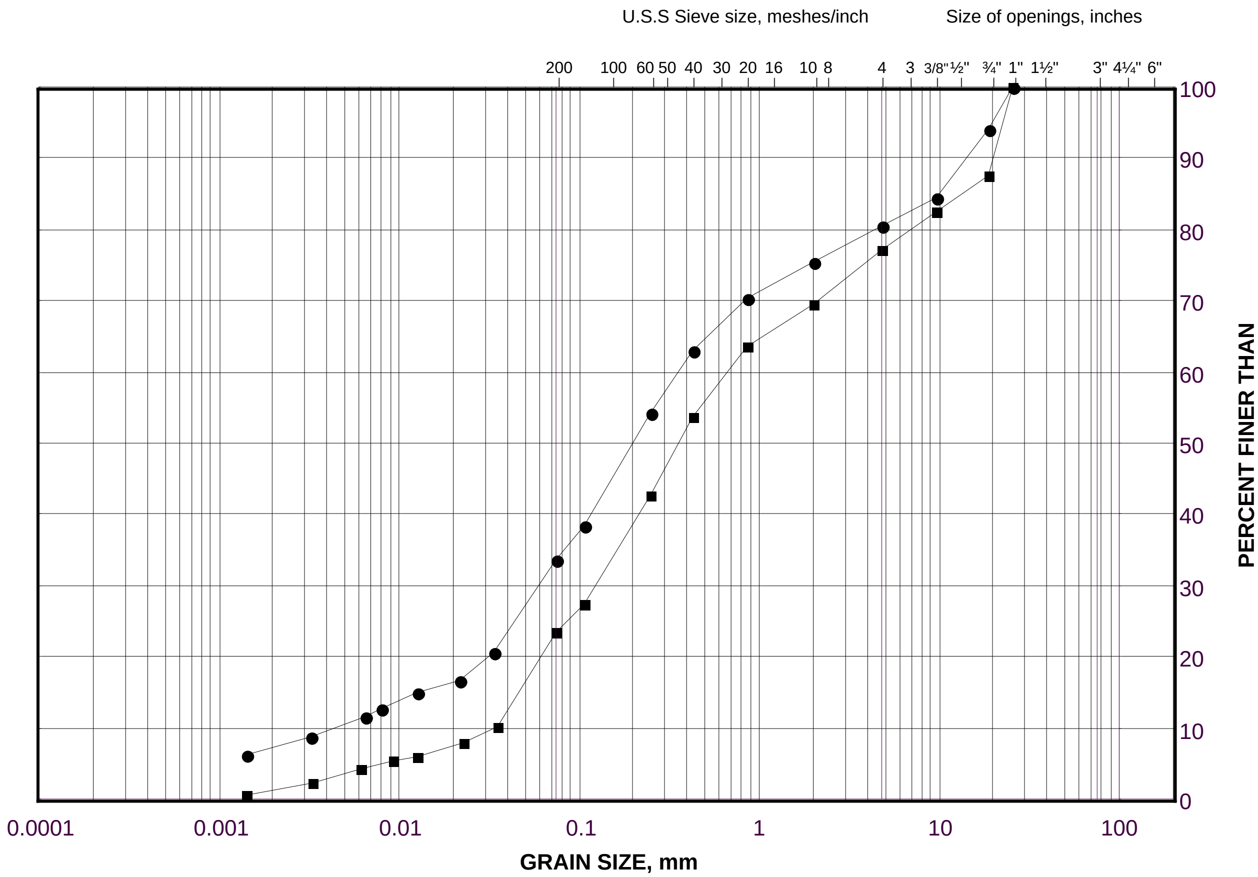
Figure No. B2

Project No. 131111-0026

Checked By: NL

GRAIN SIZE DISTRIBUTION
SILTY SAND

FIGURE B3



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	**COBBLE**
FINE GRAINED	**SAND SIZE**			**GRAVEL SIZE**		**SIZE**

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	15-S4	8A	252.9
■	15-S1	9	259.6

Project Number: 13-1111-0026

Checked By: NL

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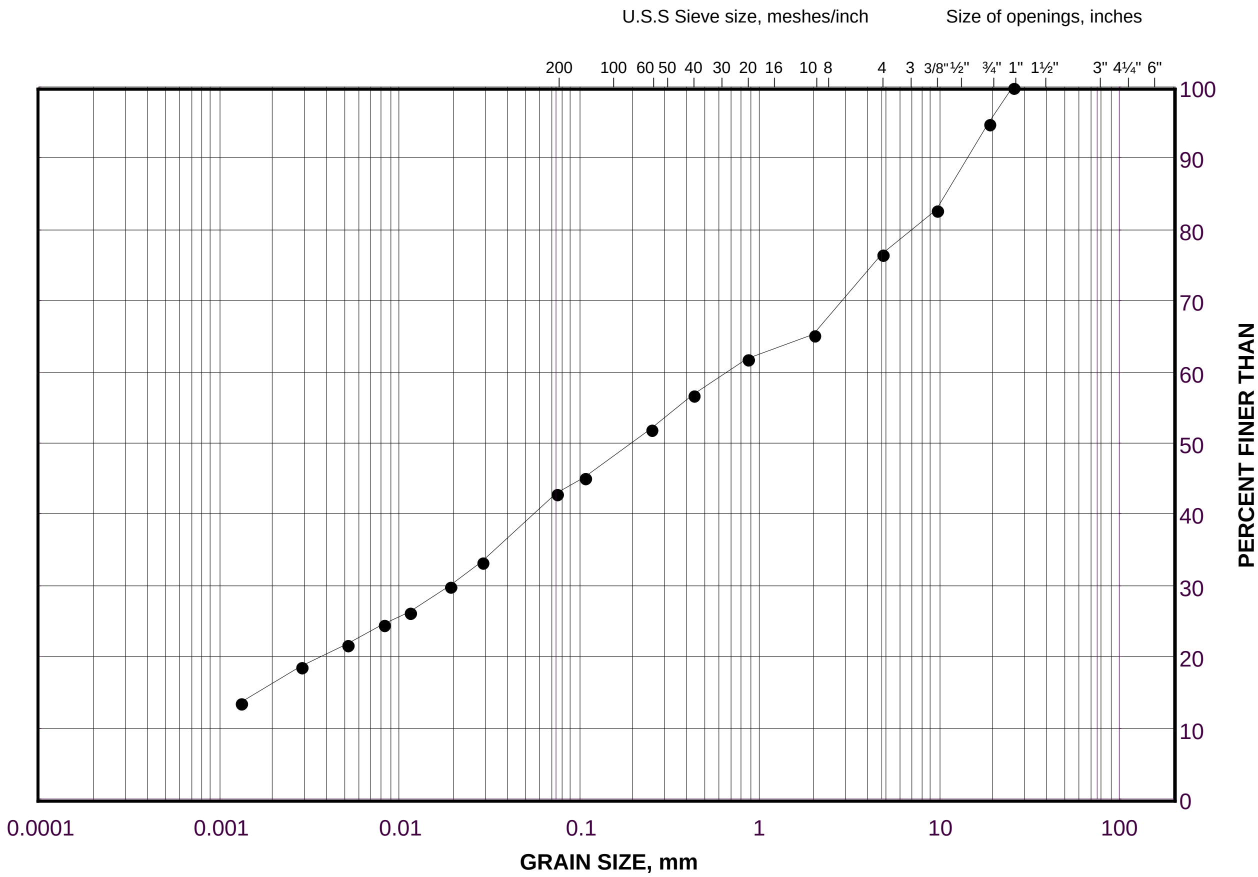
Date: 19-Dec-15

GRAIN SIZE DISTRIBUTION

CLAYEY SILT (TILL)

FIGURE B4



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	**SAND SIZE**			**GRAVEL SIZE**		**SIZE**

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	15-S1	13	253.5

Project Number: 13-1111-0026

Checked By: NL

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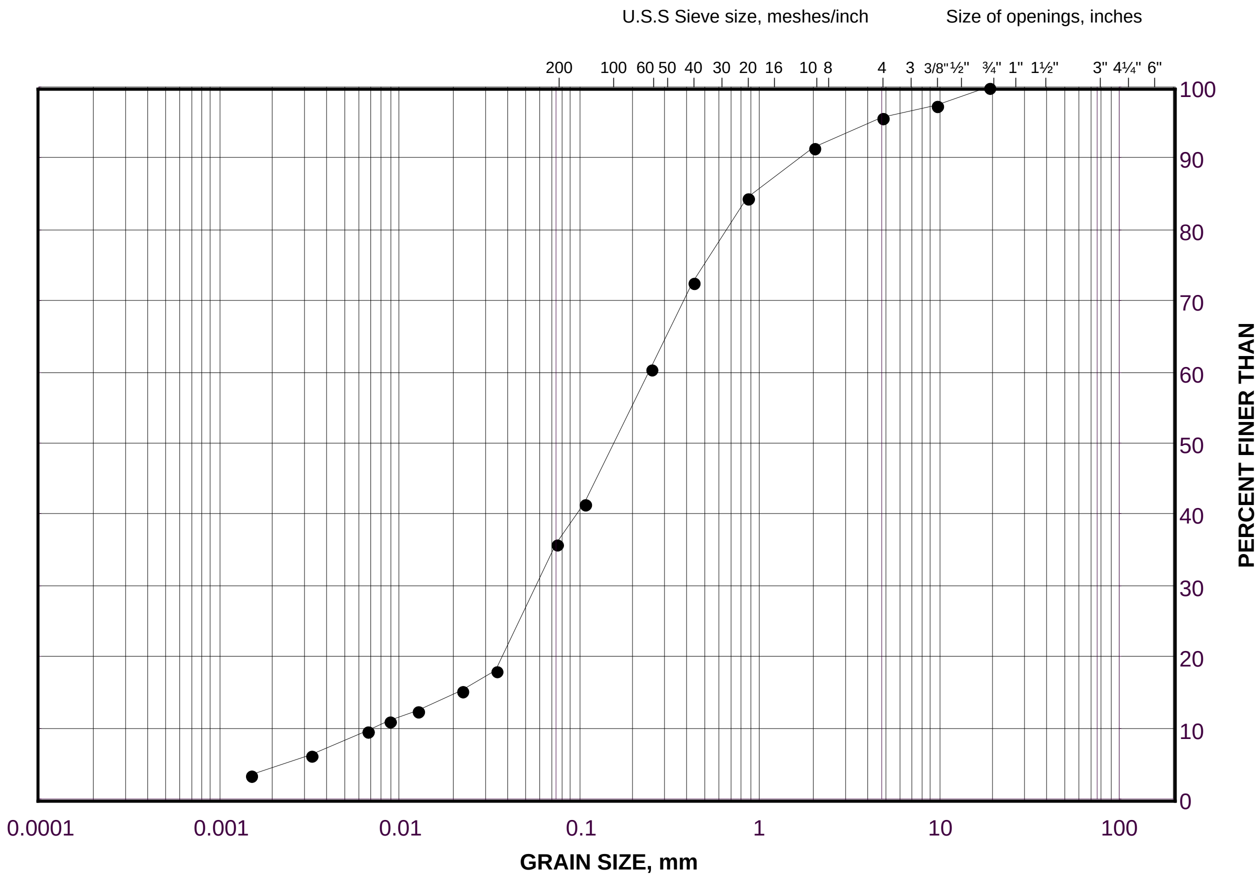
Date: 19-Dec-15

GRAIN SIZE DISTRIBUTION

SILT and SAND (TILL)

FIGURE B5



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	**COBBLE**
FINE GRAINED	**SAND SIZE**			**GRAVEL SIZE**		**SIZE**

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	15-S5	14	249.9

Project Number: 13-1111-0026

Checked By: NL

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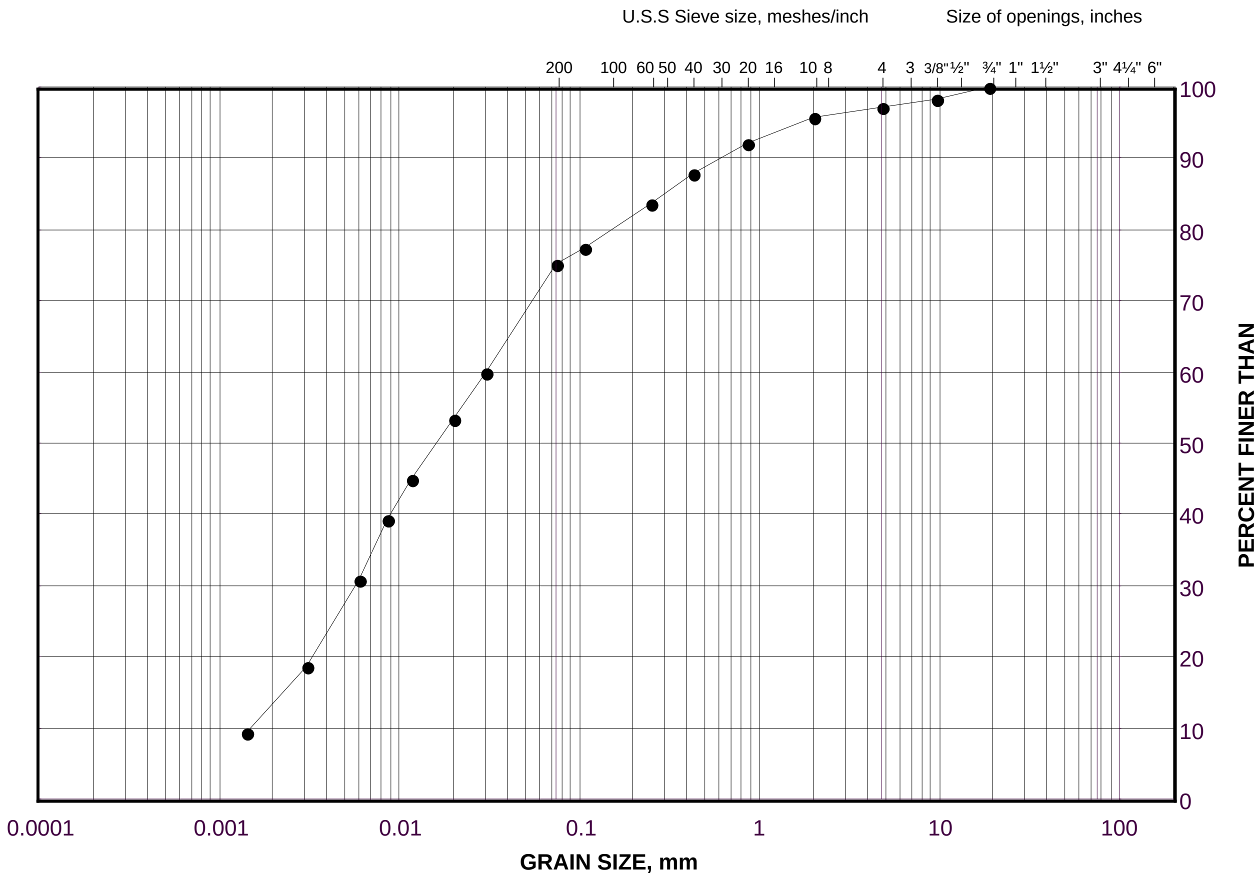
Date: 19-Dec-15

GRAIN SIZE DISTRIBUTION

CLAYEY SILT

FIGURE B6



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	**SAND SIZE**			**GRAVEL SIZE**		**SIZE**

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	15-S4	10	250.5

Project Number: 13-1111-0026

Checked By: NL

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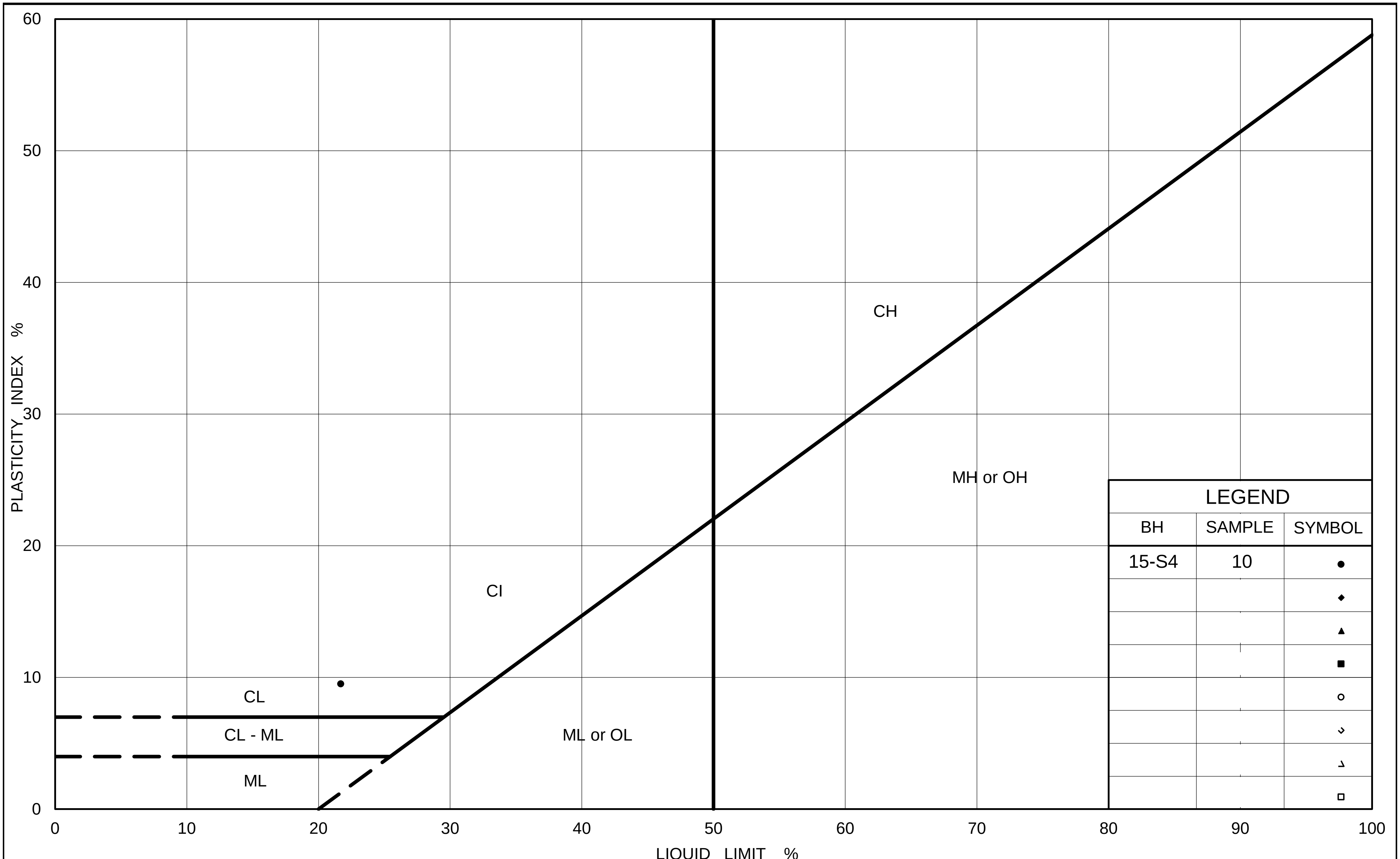
Date: 19-Dec-15



PLASTICITY CHART
CLAYEY SILT

Figure No. B7

Project No. 13-1111-0026

Checked By: NL

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