January, 2014

FOUNDATION INVESTIGATION AND DESIGN REPORT

Highway 401/Holt Road Underpass Structure
Clarington, Ontario
G.W.P. 2101-08-00

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REPORT

Golder Associates

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FOUNDATION INVESTIGATION REPORT
HIGHWAY 401/HOLT ROAD INTERCHANGE STRUCTURE
CLARINGTON, ONTARIO
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by URS Canada Inc. (URS) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the Highway 401/Holt Road Interchange reconfiguration in the Town of Clarington, Regional Municipality of Durham, Ontario.

This report addresses the results of the detail subsurface investigation carried out for the reconstruction/replacement of the Interchange underpass structure.

The Terms of Reference and Scope of Work for the foundation engineering services are outlined in MTO's Request for Proposal (RFP) for Assignment No. 2008-E-0059 dated March 2009 and associated clarifications, and in Section 6.8 of the URS *Technical Proposal* for this assignment.

2.0 SITE DESCRIPTION

The existing Highway 401/Holt Road Underpass bridge is located near the entrance to the Darlington Nuclear Power Plant approximately 10 km east of Oshawa, Ontario. According to the design drawings prepared by Department of Highways – Ontario, dated 1961, the existing four-span underpass structure is about 60 m long with inner span lengths of about 18 m and outer span lengths of about 12 m, and the bridge deck is about 10 m wide. Reportedly, the existing abutments are supported on piles driven into the very dense till deposits and the piers are supported on spread footings founded on the till deposits between about Elevation 108.2 m and 109.4 m.

Based on the General Arrangement (GA) drawing of the new Highway 401/Holt Road Interchange provided by URS on September 12, 2013, we understand that the existing bridge will be removed and a new Underpass bridge will be constructed about 30 m to the east of the existing structure.

In general, the terrain in the area of the proposed new bridge is relatively flat, with the natural ground surface in the vicinity of the structure site ranging between about Elevation 111 m and 114 m.

The Highway 401 grade in the vicinity of the existing and the new Holt Road Interchange is at about Elevation 111 m. The existing Holt Road Underpass approach embankments consist of earth fill, up to about 7.5 m high, with the Holt Road surface at about Elevation 118.5 m. The existing approach embankment side slopes are oriented at approximately 2 horizontal to 1 vertical (2H:1V), with no mid-height benches.

3.0 INVESTIGATION PROCEDURES

3.1 Current Investigation

Golder Associates completed a preliminary subsurface investigation for the new Interchange structure which was carried out on November 22, 2012, during which time two boreholes (Boreholes HR-1 and HR-2) were advanced at the proposed abutment locations as shown on Drawing 1. The results of the subsurface investigation are reported in Golder's Preliminary Foundation Investigation and Design Report (Golder, 2013). The borehole information from the preliminary investigation have been utilized to supplement the current investigation.

The field work for the current subsurface investigation was carried out between May and June 2013, during which time six boreholes (Boreholes 13-45 to 13-50) were advanced approximately at the locations shown on Drawing 1. Boreholes 13-45, 13-46, 13-49 and 13-50 were advanced using a track-mounted CME-45 drill rig, supplied and operated by KC Drilling of Innisfill, Ontario and Boreholes 13-47 and 13-48 were advanced using a truck-mounted CME-55 drill rig, supplied and operated by Strong Soil Search Inc. of Claremont, Ontario. All boreholes were drilled within the footprint of the proposed structure foundations with Boreholes 13-45 and 13-46 drilled at the north abutment and approach, respectively, Boreholes 13-47 and 13-48 drilled in the median of Highway 401 at the central pier and Boreholes 13-49 and 13-50 drilled at the south abutment and approach, respectively.

The boreholes were drilled using 120 mm diameter solid stem augers to depths ranging between 6.2 m and 9.2 m below ground surface. Soil samples were obtained at 0.75 m and 1.5 m intervals of depth in the boreholes, using a 50 mm outside diameter split-spoon sampler driven in accordance with the Standard Penetration Test (SPT) procedure (ASTM D1586)[1].

The groundwater conditions were observed in the open boreholes during and immediately following the drilling operations and are noted on the borehole records contained in Appendix A. A piezometer was installed in each of Boreholes 13-45 and 13-46 to monitor the groundwater levels at those locations. The piezometer installation details and water level readings are described on the Record of Borehole sheets in Appendix A. The boreholes were backfilled in accordance with Ontario Regulation 903 (as amended).

The field work was supervised on a full-time basis by a member of Golder's engineering staff who located the boreholes in the field, directed the drilling, sampling, and in situ testing operations, and logged 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 content determinations, Atterberg limits and grain size distribution were carried out on selected soil samples. The results of the geotechnical laboratory testing are presented in Appendix B. The geotechnical laboratory testing was completed according to MTO and/or ASTM standards as applicable.

The as-drilled borehole locations and ground surface elevations were surveyed in the field by Callon-Dietz, a licensed surveyor. The borehole locations (referenced to the MTM NAD83 coordinate system) and ground surface elevations (referenced to geodetic datum) are summarized below and are shown on the Record of Borehole Sheets in Appendix A and on Drawing 1.

Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)	Borehole Depth (m)
13-45	4,860,779.0	367,273.0	113.8	6.2
13-46	4,860,802.0	367,275.0	114.1	9.2
13-47	4,860,744.0	367,287.0	112.3	8.1
13-48	4,860,749.0	367,304.0	111.8	6.4
13-49	4,860,714.0	367,319.0	110.9	6.2
13-50	4,860,695.0	367,316.0	110.9	9.2

[1] ASTM International, ASTM D1586 – Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of Soils

3.2 Previous Investigation

The results of a previous geotechnical investigation carried out at the existing Highway 401/Holt Road bridge site were obtained from the MTO GEOCRES library, as summarized in a letter prepared by the Department of Highways – Ontario titled "Darlington Twp. Bridge No. 8, Holt Road Underpass at Highway 401 Intersection, District No. 7", dated March 7, 1961, GEOCRES No. BA851-E.

During the previous investigation, a total of seven (7) boreholes (Borehole Nos. 1 to 7, inclusive) were advanced in the general vicinity of the existing bridge as shown on Drawing 1. A copy of the original borehole records is included in Appendix C.

In general, the subsoils encountered in the above noted boreholes consist of a surficial deposit of granular fill, 0.3 m to 1.5 m thick, underlain by a 0.3 m to 1.4 m thick layer of topsoil. The topsoil is underlain by a deposit of silty sand till. The silty sand till is described in the borehole records as gravelly / pebbly. The surface of the silty sand till was encountered between the depths of about 0.6 m and 2.1 m below ground surface (between Elevations 111 m and 110 m according to the reference datum used on the borehole records). The boreholes were terminated within the silty sand till at depths ranging from about 3 m to 9 m below ground surface (Elevations 108 m to 103 m). There were no groundwater levels noted nor any indication of groundwater being encountered during drilling shown on the borehole logs.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

This section of Highway 401 is located within the Iroquois Plain physiographic region, as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984)[2] and *Urban Geology of Canadian Cities* (Karrow and White, 1998)[3]. The Iroquois Plain extends around the western shores of Lake Ontario. The Plain is comprised of the flat to undulating lakebed and beaches of the former glacial Lake Iroquois, which occupied this area during the last glacial recession.

The surficial soils in this area of the Iroquois Plain are typically comprised of glaciolacustrine clays, silts and sands to gravelly sands, which are underlain by an extensive till deposit that is mapped in this area as the Bowmanville Till. Within the area approximately bounded by Holt Road and Morgan's Road, the surficial glaciolacustrine deposits are absent or of limited thickness and the Bowmanville Till unit is frequently present immediately below the ground surface. Between these limits, an extensive surficial deposit of clayey silt to silty clay is present over the Bowmanville Till (Karrow and White, 1998). More recent alluvial deposits of gravel, sand, silt and/or clay are present in the valleys associated with Bowmanville Creek, Soper Creek, Wilmot Creek and Graham Creek.

The overburden soils are underlain by limestone bedrock of the Lindsay Formation, Simcoe Group (Geological Survey of Canada, 1997).[4]

[2] 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.

[3] Karrow, P. F., and White, O. L., 1998. *Urban Geology of Canadian Cities.* Geological Association of Canada Special Paper No. 42. St. John's, Nfld.

[4] Ontario Geological Society, 1991. *Geology of Ontario*. Special Volume 4, Part 1. Eds. P.C. Thurston, H.R. Williams, R.H. Sutcliffe and G.M. Stott. Ministry of Northern Development and Mines, Ontario.

4.2 Subsurface Conditions

The current and preliminary subsurface investigations entailed the advancement of six boreholes and two boreholes, respectively, at the proposed new Highway 401/Holt Road Underpass structure site. The borehole locations, ground surface elevations and interpreted stratigraphic conditions are shown on Drawings 1 and 2. The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of in situ and laboratory testing are given on the Record of Borehole sheets contained in Appendix A. The results of geotechnical laboratory testing are also presented on Figures B1 to B6 contained in Appendix B. The stratigraphic boundaries shown on the Record of Boreholes and on the interpreted stratigraphic sections on Drawings 1 and 2 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 locations.

In summary, the subsurface conditions encountered at the site consist of topsoil or asphalt underlain by a fill deposit comprised of sand and gravel to sandy silt to clayey silt between 0.8 m and 4.4 m thick, underlain by a dense to very dense sand and silt till deposit interlayered in places with very stiff to hard clayey silt till. A more detailed description of the soil deposits encountered in the boreholes is provided in the following sections.

4.2.1 Topsoil

A deposit of topsoil was encountered immediately below ground surface in Boreholes 13-45, 13-46, 13-49 and 13-50. The thickness of the deposit ranges between 0.4 m and 0.6 m.

The Standard Penetration Test (SPT) “N” values measured within the topsoil deposit range from 8 blows to 19 blows per 0.3 m of penetration, suggesting a firm to very stiff consistency.

The natural water content measured on one sample of the topsoil is 8 per cent.

4.2.2 Asphalt

An approximately 0.1 m thick layer of asphalt was encountered in Boreholes 13-47 and 13-48 at ground surface.

4.2.3 Sand and Gravel Fill

A fill deposit comprised of sand and gravel, trace to some silt was encountered below the asphalt in Boreholes 13-47 and 13-48. The surface of the granular fill deposit was encountered at Elevations 112.2 m and 111.7 m and the deposit is 0.7 m and 1.4 m thick in Boreholes 13-47 and 13-48, respectively.

The measured SPT “N” values within this deposit range from 20 blows to 25 blows per 0.3 m of penetration, indicating a compact relative density.

The natural water content measured on one sample of the granular fill is 5 per cent.

4.2.4 Clayey Silt Fill

A deposit of clayey silt fill was encountered below the sand and gravel fill in Borehole BH-47 and below the topsoil in Boreholes 13-49 and 13-50. The surface of the cohesive fill deposit was encountered between Elevation 111.5 m and 110.5 m and the thickness of the cohesive fill deposit is between 0.3 m and 0.7 m thick.

One measured SPT “N” value within this deposit is 16 blows per 0.3 m of penetration, suggesting a very stiff consistency.

The cohesive fill deposit consists of clayey silt with to some sand, trace to some gravel, trace organics. The results of a grain size distribution test completed on one selected sample of the clayey silt with sand fill is shown on Figure B1 in Appendix B.

Atterberg limits testing conducted on one selected sample of the clayey silt fill measured a plastic limit of about 14 per cent, a liquid limit of about 22 per cent and a plasticity index of about 8 per cent. This test result, which is plotted on a plasticity chart on Figure B2 in Appendix B, indicates that the deposit consists of clayey silt of low plasticity.

The natural water content measured on a sample of the clayey silt fill is 15 per cent.

4.2.5 Sandy Silt to Silty Sand Fill

A fill deposit comprised of sandy silt to silty sand was encountered at the ground surface in Boreholes HR-1 and HR-2, underlying the topsoil in Boreholes 13-45 and 13-46, and below the sand and gravel fill in Borehole 13-48. The surface of the sand and silt fill deposit was encountered up to 1.5 m below ground surface (Elevation 113.5 m to 110.4 m), and was measured to be between 0.7 m and 3.8 m thick.

The measured SPT “N” values within this deposit range from 7 blows to 87 blows per 0.3 m of penetration, indicating a loose to very dense relative density.

This deposit is comprised of zones of sandy silt, sand and silt and silty sand, trace to some gravel, trace to some clay and trace organics. Increased organic content/wood fibres were present in some boreholes near the interface between the fill and underlying till soils. The results of grain size distribution tests completed on three selected samples of the sand and silt portion of the fill deposit are shown on Figure B3 in Appendix B.

The natural water content measured on eight selected samples of the sandy silt to silty sand fill deposit ranges from about 6 per cent to 18 per cent. One water content of 26 per cent was measured in 13-45 and is attributed to the greater organic content of the fill in this borehole.

4.2.6 Clayey Silt (Till)

A deposit of clayey silt till was encountered below the fill in Boreholes HR-1, 13-45, 13-46, 13-49 and 13-50, and within the upper portion of the sandy silt to sand and silt till deposit in Borehole HR-2. The surface of the clayey silt till was encountered at depths between 0.8 m and 4.4 m below ground surface, corresponding to Elevations 110.9 m to 109.4 m. The thickness of this till deposit ranges from about 0.6 m to 3.9 m in Boreholes HR-1, HR-2, 13-46 and 13-49,and from about 3.3 m to 8.4 m in Boreholes 13-45 and 13-50 where it was not fully penetrated.

The measured SPT “N” values within this deposit range from 28 blows per 0.3 m of penetration to 100 blows per 0.08 m of penetration, suggesting a very stiff to hard consistency.

The till deposit consists of clayey silt with sand to some sand, trace to some gravel and contains occasional silt seams at some locations. The presence of cobbles and boulders was inferred from grinding of the augers within this deposit as noted on the Record of Borehole sheets. The results of grain size distribution tests completed on eight selected samples of the clayey silt till are shown on Figure B4 and resemble the grain size distributions of the underlying sandy silt to sand and silt till, suggesting that the clayey silt till layer is likely a transition zone to the underlying more granular till deposit.

Atterberg limits testing was conducted on seven selected samples of the clayey silt till and measured plastic limits ranging from 10 per cent to 15 per cent, liquid limits ranging from 13 per cent to 33 per cent and plasticity indices ranging from 2 per cent to 18 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B5 and indicate that the material is a clayey silt of low plasticity with zones that may be classified as silt of slight plasticity.

The natural water content measured on samples of the clayey silt till deposit ranges from about 4 per cent to 15 per cent.

4.2.7 Sand and Silt (Till)

A deposit of sand and silt till was encountered underlying the fill deposit in Boreholes 13-47, 13-48 and HR-2 and underlying the clayey silt till deposit in Boreholes 13-46, 13-49 and HR-1. The surface of the sand and silt till deposit was encountered at depths ranging from 1.5 m to 8.3 m below ground surface, at between Elevations 110.9 m and 105.8 m. The boreholes were terminated within this till deposit at depths ranging between 6.2 m and 9.2 m below ground surface corresponding to between Elevations 105.5 m and 103.9 m.

The measured SPT “N” values within this deposit range from 42 blows per 0.3 m of penetration to greater than 50 blows per 0.03 m of penetration, indicating a dense to very dense (but typically very dense) relative density.

The glacial till deposit consists of sand and silt, trace to some clay, trace to some gravel, interlayered as noted above with clayey silt till in places. The presence of cobbles and boulders was inferred from grinding of the augers within this deposit as noted on the Record of Borehole sheets. The results of grain size distribution tests completed on seven selected samples of the sand and silt till from the current investigation are shown on Figure B6 in Appendix B.

Atterberg limits testing was conducted on five selected samples of the sand and silt till and measured plastic limits ranging from 10 per cent to 12 per cent, liquid limits of 13 per cent and plasticity indices ranging from 1 per cent to 3 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B7 and indicate that the fines portion of the material may be classified as silt of slight plasticity.

The natural water content measured on fifteen samples of the sand and silt till deposit ranges from about 4 per cent to 8 per cent.

4.3 Groundwater Conditions

Details of the water levels observed in the open boreholes at the time of drilling are summarized on the Record of Borehole sheets in Appendix A of this report. The water level in the open boreholes was measured at between 3.0 m and 7.3 m below ground surface corresponding to between Elevations 110.8 m and 105.0 m in Boreholes 13-45 to 13-47, 13-50, HR-1 and HR-2; Boreholes 13-48 and 13-49 were dry upon completion of drilling.

Standpipe piezometers were installed in Boreholes 13-45 and 13-46 to permit monitoring of the groundwater level at those locations. Details of the piezometer installations are shown on the Record of Borehole sheets in Appendix A. Groundwater levels measured in the piezometers are summarized below.

Borehole No.	Ground Surface Elevation	Depth to Groundwater Level	Groundwater Elevation	Date of Measurement
13-45	113.8 m	2.1 m	111.7 m	September 10, 2013
13-46	114.1 m	3.9 m	110.2 m	September 10, 2013

The water 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 season and periods of precipitation. Given the presence of a deposit of granular fill soils overlying very stiff to hard/very dense till, perched groundwater conditions can be expected to be present directly above the till deposits.

5.0 CLOSURE

This Foundation Investigation Report was prepared by Mr. Billy Murphy and by Mr. Matthew Kelly, P.Eng., and reviewed by Mr. Kevin Bentley, P.Eng., a geotechnical engineer and Associate with Golder. Mr. Jorge Costa, P.Eng., a Designated MTO Foundations Contact for Golder and Principal, conducted an independent review and quality control audit of this report.

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PART B

FOUNDATION DESIGN REPORT
HIGHWAY 401/HOLT ROAD INTERCHANGE STRUCTURE
CLARINGTON, ONTARIO
G.W.P. 2101-08-00

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides foundation design recommendations for the proposed replacement of the existing Highway 401/Holt Road Interchange Underpass structure and associated approach embankments. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current and previous subsurface investigations. 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 detail design of the structure foundations and approach embankments.

Where comments are made on construction, they are provided in order to highlight those aspects that could affect the design of the project. 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.

6.1 General

As part of the future widening of Highway 401 from Courtice Road easterly to the Regional Municipality of Durham east boundary, and plans to upgrade the Highway 401/Holt Road Interchange, we understand that the design includes the removal of the existing Hwy 401/Holt Road Interchange/Underpass structure and associated ramps, and construction of a new Hwy 401/Holt Road Interchange including a new Underpass structure.

The existing four-span structure is about 60 m long with inner span lengths of about 18 m and outside span lengths of about 12 m, and the bridge deck is about 10 m wide. Available information indicates that the existing abutments are supported on piles driven into the very dense till deposit and the piers supported on spread footings founded on the till deposit between about Elevations 109.4 m and 108.2 m. The existing approach embankments are up to about 7.5 m high and the side slopes are oriented at approximately 2H:1V. Based on visual observations during the current site investigation, the existing bridge foundations appear to have performed satisfactorily to date (i.e. no signs of cracking/settlement) and the approach embankments appear to be stable.

It is understood that the new Holt Road Underpass structure will consist of a two span, pre-cast concrete girder bridge with span lengths of about 39 m, abutments located north and south of the Highway 401 westbound and eastbound alignments and a centre pier located in the median. Based on discussions with URS, for this site an integral abutment design is preferred from a structural, constructability, and maintenance perspective.

Based on the Draft General Arrangement drawing provided by URS on September 12, 2013, the replacement Underpass structure will be located approximately 30 m east of the existing structure. The finished pavement grade for Highway 401 is proposed to be maintained at approximately Elevation 112 m and the pavement grade for the new realigned Holt Road will be approximately Elevation 119 m, resulting in new approach embankments up to 8 m high relative to the adjacent ground surface.

6.2 Foundation Options

Based on the proposed Underpass geometry and the subsurface conditions at this site, both shallow and deep foundation options have been considered for support of the abutments and piers for the new Holt Road Underpass structure. 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, relative costs and risks/consequences is provided in Table 1 following the text of this report.

- **Strip or spread footings founded on the very dense sandy silt to sand and silt till:** Strip or spread footings are feasible for support of the new abutments, associated wing walls/retaining walls, and pier at this site, although this foundation type would not permit the use of integral abutments. Sub-excavation of up to 4.4 m of fill soils would be required to reach the very dense tills and tall abutment walls would be required to reach the bridge deck. In addition, to facilitate excavation for the construction of the new footings temporary protection systems would be required along the outside and median edges of the Highway 401 westbound and eastbound lanes and near the existing bridge approach embankments.

- **Footings "perched" on a compacted granular pad in the approach embankment:** "Perched" footings are feasible for support of the new abutments and associated wing walls, (but not required at the pier) which would reduce the need for temporary protection systems along the outside edges of the Highway 401 westbound and eastbound lanes and near the existing bridge approach embankments associated with the new abutment construction. Sub-excavation of up to 4.4 m of existing fill and soil replacement with engineered fill would be required at the abutments.

- **Driven steel H-piles or pipe (tube) piles:** Driven steel H-piles or steel pipe (tube) piles are feasible for support of the abutments and associated wing walls, and pier, or in combination with shallow foundations or caissons at the pier, and would permit design of integral abutments (H-piles) or semi-integral abutments. The abutment pile caps would be "perched" within the Holt Road approach embankment for frost protection and to eliminate the need for excavations for the pile caps. Due to the relatively shallow depth to the very dense/hard till (having SPT "N" values greater than 100 blows per 0.3 m of penetration), pre-augering into the "100-blow" soil would be required at the North Abutment, to achieve the minimum pile length for integral abutment design, with the piles driven from within pre-augered holes. Pile driving shoes are recommended to protect the pile tips from damage during driving into the very dense/hard till.

- **Caissons:** Caissons are feasible for the support of the abutments and pier but preclude the use of integral abutments, unless used in combination with steel H-piles at the abutments. This option will be more expensive than either shallow foundations or driven pile foundations, although fewer caisson elements would be required in comparison to the number of driven steel piles. The pile caps could be "perched" within the Holt Road approach embankment at the abutments, and the caissons could extend to the underside of the bridge at the pier which would significantly reduce subexcavation costs and the requirement for a temporary excavation support system.

- **Micropiles:** Micropiles are feasible for support of the pier and abutments at this site but would require additional borehole investigation, design and load testing. Soil bonded micropiles have been assumed for this site as the depth to bedrock is unknown, and while soil bonded micropiles have been used on some projects throughout the Greater Toronto Area, there are a limited number of contractors that have sufficient experience and the expertise to construct this type of foundation and the risk of improper installation is much greater than for micropiles founded in bedrock. Due to these risks and potential construction difficulties, and the higher relative costs associated with this foundation alternative, micropiles are not considered to be a preferred foundation system for this structure site and therefore are not discussed in detail in subsequent sections of this report. However, the relative advantages and disadvantages of micropile foundations are summarized in Table 1.

Based on the above considerations, both shallow and deep foundation options are considered technically feasible for the support of the new foundation elements. From a foundations perspective, shallow foundations are preferable for the support of the new abutments and pier; however, the requirement for deep subexcavation and temporary excavation support systems make this option less desirable from a constructability point of view. Given that the new bridge is to be constructed within the footprint of the existing bridge approach embankments and the present Highway 401 right-of-way, and the fact that integral abutments are preferred from a structural perspective, deep foundations comprised of steel H-piles at the abutments and caissons designed as continuous columns at the pier are considered to be a preferred option to reduce impact on traffic and ease of constructability.

The ranking of the foundation alternatives and the primary advantage or disadvantage for the foundation alternatives at the abutments and pier locations from an overall foundations, constructibilty and performance perspectives are summarized as follows:

Abutment Foundation

Rank 1: H-Piles: driven to found within the very dense sand and silt till to hard clayey silt till – allows for integral abutment design but requires pre-augering at the North Abutment to achieve the required pile length;

Rank 2: Shallow Foundations: either founded on the very dense sand and silt till to hard clayey silt till or on a Granular A pad – precludes integral abutment design; requires subexcavation of existing fill materials to depths of about 4.4 m and high abutment walls, and temporary excavation support systems near Highway 401 and the existing bridge approach embankments;

Rank 3: Caissons: augered to found within the very dense sand and silt till to hard clayey silt till – allows for semi-integral abutment design; requires liners.

Pier Foundation

Rank 1: Caissons: augered to found within the very dense sand and silt till to hard clayey silt till – can be used in conjunction with steel H-Piles to allow for integral abutment design when used at the pier only and eliminates subexcavation and temporary excavation support system when extended to the bridge level (i.e. no buried caisson cap); requires liners.

Rank 2: H-Piles: drive to found within the very dense sand and silt till to hard clayey silt till – relatively short pile lengths (less than 5 m) and lower capacity than caissons requiring more units and increased traffic disruption / staging in Highway 401 median.

Rank 3: Shallow Foundations: either founded on the very dense sand and silt till to hard clayey silt till or on a Granular A pad – can be used in conjunction with steel H-Piles to allow for integral (or semi-integral) abutment design; requires subexcavation of existing fill materials and temporary excavation support systems in Highway 401 median to reduce traffic disruption.

6.3 Shallow Foundations

6.3.1 Founding Elevations

For support of the new abutments, pier, and any associated concrete wing walls/retaining walls, strip or spread footings should be founded below any existing fill or softened/loosened surficial soils, on the very dense sand and silt/hard clayey silt till deposit. The founding elevation should be a minimum of 1.2 m below the lowest surrounding grade to provide adequate protection against frost penetration, per Ontario Provincial Standard Drawing (OPSD) 3090.101 (*Frost Penetration Depths for Southern Ontario)*.

The following maximum (highest) founding elevations are recommended for design of footings founded on very dense silty sand to sand and silt till, or hard clayey silt till.

Foundation Element	Reference Boreholes	Maximum (Highest) Founding Elevation (m)	Approximate Excavation Depth Below Existing Grade (m)
North abutment	13-45; HR-1	110.5 to 107.9	3.3 to 3.8
Pier	13-48; 13-47	109 to 108.5	2.8 to 3.8
South abutment	13-49; HR-2	109.3 to 108.8	1.6 to 2.9

At the North abutment, the design founding elevation ranges from 110.5 m at the west limit to 107.9 m at the east limit of the footing due to the presence of a layer of clayey silt till of variable consistency at Borehole HR-1. Consideration could be given to founding the spread/strip footing at higher elevation at the east end of the abutment on the very stiff to hard clayey silt till (i.e. where SPT ‘N’-values are greater than 30 blows per 0.3 m of penetration), however the geotechnical resistance value will decrease accordingly and differential settlement (up to 25mm) across the width of the north abutment footing and between the east and west wing wall foundations of the north abutment is expected. Therefore, the north abutment footings should be founded on the ”100 blow” till soils at the elevations as given above.

The footing subgrade should be inspected by the Quality Verification Engineer following excavation, in accordance with OPSS 902 *(Excavating and Backfilling Structures)* to check that all existing fill, softened/loosened soils or other unsuitable material have been removed. The founding soils will be susceptible to disturbance, therefore a concrete working slab should be placed on the prepared subgrade as described in Section 6.8.4.

Alternatively, the abutment foundations could be “perched” on a compacted granular pad in the approach embankments above the Highway 401 grade. In this case, the compacted granular pad should have a minimum thickness of 2 m, such that the pad extends below any existing fill and/or loose soil to found on the compact to very dense/very stiff to hard till deposit, encountered between Elevations 110.9 m and 109.4 m at the north abutment, and between Elevation 110.1 m and 109.4 m at the south abutment. Sub-excavation of existing fill up to 4.4 m and 1.8 m below existing ground surface at the North and South Abutments would be required prior to placement of the granular pad. The pad should consist of OPSS. Prov. 1010 (Aggregates) Granular ‘A’ material extending at least 1 m beyond the edges of the footing(s), then outward and downward at 1H:1V. The granular fill should be placed in accordance with OPSS 501 (Compacting) and Special Provision (SP) 105S21.

6.3.2 Geotechnical Resistance/Reaction

Strip or spread footings placed on the native very dense sand and silt till, hard clayey silt till, or perched on a compacted Granular ‘A’ pad within the approach embankments founded at or below the design elevations given in the preceding section, should be designed based on the factored geotechnical resistances at Ultimate Limit States (ULS) and geotechnical reactions at Serviceability Limit States (SLS for 25 mm of settlement) given below.

Founding Stratum	Assumed Footing Width	Factored Geotechnical Resistance at ULS	Geotechnical Reaction at SLS*
Abutments, pier, and/or retaining wall footings on very dense sand and silt till or hard clayey silt till	3 m	700 kPa	450 kPa
Abutments or retaining wall perched in approach embankments on compacted Granular ‘A’ pad	3 m	900 kPa	350 kPa

* For 25 mm of settlement

The geotechnical resistances should be reviewed if the selected footing width or founding elevations differ from those given above. In addition, these geotechnical resistances are provided for loads applied perpendicular to the surface of the footings; where applicable, inclination of the load should be taken into account in accordance with Section 6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC 2006)* and its *Commentary.*

6.3.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footings and the subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC*. For cast-in-place concrete footings constructed on a concrete working slab that is placed on top of the very dense sandy silt to sand and silt till or on OPSS PROV 1010 Granular ‘A’ (Aggregates), the coefficient of friction, tan δ, can be taken as follows for design:

- Cast-in-place concrete footing on concrete working slab: tan δ = 0.60
- Cast-in-place footing or concrete working slab on Granular ‘A’ or sand and silt till: tan δ = 0.60

6.4 Steel H-Pile or Steel Pipe (Tube) Foundations

6.4.1 Founding Elevations

The abutments, pier and any associated wing walls can be supported on steel H-piles or steel pipe (tube) piles founded within the very dense sand and silt till or hard clayey silt till (having SPT “N” values greater than 100 blows per 0.3 m of penetration).

The surface of the “100-blow” soils ranges from about Elevation 110.9 m to 107.9 m across the footprint of the proposed new structure. The following pile tip elevations may be used for design, assuming piles are driven at least 1.5 m into the “100-blow” material:

Foundation Element	Reference Boreholes	Elevation of Underside of Pile Cap*	Estimated Design Pile Tip Elevation	Estimated Pile Length*
North Abutment	13-45; HR-1	115.0 m	109.4 m ** (west limit) to 106.4 m (east limit)	5.6 m ** to 8.6 m
Centre Pier ***	13-48; BH-47	110.8 m	107.3 m to 107.0 m	3.5 m to 3.8 m
South Abutment	13-49; HR-2	115.5 m	107.8 m to 107.2 m	7.7 m to 8.3 m

* Based on G.A. Drawings dated December 20, 2013.

** Min. 6.0 m pile length to founding Elevation 109.0 m for integral abutment design (as indicated by URS).

*** Assuming underside of pile cap is minimum1.2 m below final ground/pavement surface which is at about Elevation 112 m.

For integral abutment design, pre-auguring into the very dense/hard till soils will be required at the North Abutment to achieve a minimum 6 m pile length (and approximate 3 m embedment below adjacent ground surface required as per the structural design and reduce the potential for driving the piles out of alignment, or damaging the pile tips in the very dense/hard till deposit. The purpose of the pre-augering is to create an open hole to ensure a minimum 6 m pile length (and 3 m embedment depth) is achieved and to allow for the driving of the steel H-piles from the bottom of the hole to achieve the required geotechnical resistance and for fixity of the pile tip. It is recommended that an NSSP such as the example presented in Appendix D be included in the Contract Documents to alert the Contractor of the presence of cobbles and/or boulders and specify the requirement for pre-augering at the west half of the north abutment.

For the installation of steel H-piles or steel pipe piles, consideration must be given to the potential presence of cobbles and boulders within the till deposits (as inferred to be present in Boreholes 13-45, 13-46, 13-49 and 13-50). 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 such as OPSD 3000-201 (HP310 Oslo Point), Titus Injector Bearing Pile Point design or equivalent in accordance with OPSS 903 *(Deep Foundations).* If steel pipe piles are used, driving shoes should be in accordance with OPSD 3001.100 Type II *(Steel Tube Pile Driving Shoe).* The requirement for driving shoes should be included in the Contract Documents.

If corrugated steel pipes (CSPs) are installed as part of the integral abutment design, the CSPs should be backfilled with loose, fine to medium sand. An NSSP detailing the installation method and gradation of this sand should be included in the contract documents; an example is included in Appendix D. The annulus between the pre-augered hole and the CSP should be backfilled with OPSS.Prov.1010 Granular B Type II material.

The pile caps should be provided with a minimum of 1.2 m soil cover to provide adequate protection against frost penetration (as per OPSD 3090.101).

6.4.2 Geotechnical Axial Resistance/Reaction

For HP 310x110 piles driven to or below the estimated tip elevations provided in Section 6.4.1, the factored geotechnical axial resistance at ULS may be taken as 1,600 kN, and the geotechnical axial reaction at SLS (for 25 mm of settlement) may be taken as 1,400 kN. Similar axial resistances may be used in the design for closed-end, concrete filled 324 mm (12 ¾ in.) diameter steel pipe piles having a minimum wall thickness of 6.4 mm (1/4 in.).

Pile installation should be in accordance with OPSS 903 *(Deep Foundations).* The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile; the criteria must therefore be established at the time of construction after the piling equipment is known. The pile capacity should then be verified in the field by the use of the Hiley formula (MTO Standard Structural Drawing SS103-11) during the final stages of driving to achieve the appropriate ultimate capacity. The pile driving note to be included in the foundation drawing, as per MTO's Structural Manual (2008) Section 3.3.3 is Note 2:

- Piles to be driven in accordance with Standard SS103-11 using an ultimate geotechnical resistance of 3,200 kN per pile but must be driven below Elevations 109.0 m to 106.4 m at the North Abutment, Elevations 107.3 m to 107.0 m at the Pier and Elevations 107.8 m to 107.2 m at the South Abutment.

6.4.3 Resistance to Lateral Loads

The design of piles subjected to lateral loads should take into account such factors as the batter of the piles (if any), the relative rigidity of the pile to the surrounding soil, the fixity condition at the head of the pile (pile cap level), the structural capacity of the pile to withstand bending moments, the soil resistance that can be mobilised, the tolerable lateral deflections at the head of the pile and pile group effects. For a longer, more flexible pile, the maximum yield moment of the pile may be reached prior to mobilisation of the lateral geotechnical resistance. For design purposes, both the structural and geotechnical resistances should be evaluated to establish the governing case.

Lateral loading could be resisted fully or partially by the use of battered piles. For vertical piles, the resistance to lateral loading will have to be derived from the soil in front of the piles. For integral abutment design, there will also be a requirement for the piles to move sufficiently to accommodate the bridge deck deflections.

The resistance to lateral loading in front of the piles may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h (kPa/m), is based on the following equation (CFEM, 2002 as referenced in CHBDC, 2006):

$$k_h = \frac{n_h z}{B}$$

Where k_h is the coefficient of horizontal subgrade reaction (kPa/m);

n_h is the constant of subgrade reaction (kPa/m);

z is the depth (m); and

B is the pile diameter or width (m).

It is understood that an integral abutment design is being considered. Where the integral design includes the installation of 3 m long CSP liners (with the annular space between the pile and the liner filled with uniform grained, uncompacted sand), the upper portion of the H-pile will be generally free to flex and move laterally within the limits of the CSP. With this design, the passive lateral resistance over the length of the pile within the limits of the CSP liner should be based on the resistance provided by loose sand. The passive lateral resistance on the exterior of the CSP should be based on the resistance provided by the surrounding soil conditions.

The following values of n_h may be assumed in the structural analyses, using the interpreted stratigraphic conditions as shown on the profiles on Drawing 1 and 2.

Soil Unit	n_h (kPa/m)	Bulk Unit Weight (kN/m^3)	Effective Friction Angle (Degrees)	Undrained Shear Strength (kPa)	Shear Modulus (kPa)
Embankment fill (assuming engineered earth fill)	5,000	21	32°	-	7,000
Loose sand within CSP (if applicable)	2,200	19	28°	-	3,500
Very stiff clayey silt till/upper 1.5 m compact to dense sand and silt till	6,600	19	32°	-	10,000
Dense to very dense sand and silt till to hard clayey silt till (having SPT N-values greater than 100 blows per 0.3 m of penetration)	16,000	20	35°	-	25,000

Alternatively, the resistance to lateral loading in front of the piles may be calculated using non-linear resistance-displacement relationships (i.e. p-y curves) using commercially available software programs such as LPILE (Reese & Wang, 1997), or FLPIER (McVay et al., 1992). The deformation characteristics of the soil are based on established p-y curve models using basic soil parameters such as undrained shear strength (C_u), Bulk Unit Weight (ɣ), Angle of Internal Friction (Φ'), and Shear Modulus (G) which are provided above and can be used for the structural analysis, using the interpreted stratigraphic conditions as shown on the profiles on Drawing 1 and 2.

A maximum factored lateral resistance of 120 kN at ULS and a maximum lateral resistance of 50 kN at SLS (for 10 mm of horizontal deflection at pile cap level) is recommended for the HP 310x110 piles assuming the pile cap is at the ground level. Increased horizontal deflections can be anticipated if the pile cap is raised above the adjacent ground surface. These values are based on the "Assessed Horizontal Passive Resistance" (provided in Table C6.4 of the *Commentary* to the *CHBDC)*, and Geotechnical Reaction at SLS interpreted for the site conditions and pile size presented above. The structural capacity of the pile should be checked and verified by the structural engineer.

Group action for lateral loading should be considered where the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor, R (NAVFAC DM-7.2, 1982) as follows:

Pile Spacing in direction of Loading (d = Pile Diameter)	Subgrade Reaction Reduction Factor, R
8d	1.00
6d	0.70
4d	0.40
3d	0.25

The subgrade reaction reduction factor should be interpolated for pile spacings in between those provided in the above table.

6.5 Caisson Foundations

6.5.1 Founding Elevations

The abutments, pier and any associated wing walls may be supported on caissons founded within the very dense sand and silt till/hard clayey silt till (having SPT "N" values greater than 100 blows per 0.3 m of penetration). The surface of the "100-blow" soils was encountered between approximately Elevation 110.9 m to 107.9 m across the footprint of the proposed new structure. The following caisson founding elevations may be used, assuming a minimum 4 m long socket into the "100-blow" till deposit for a 1.2 m diameter caisson, or a 2 m long socket into the "100-blow" till deposit for a 1.5 m diameter caisson. The options for the larger caisson diameter and shallower socket depth or smaller caisson diameter and deeper socket depth are provided to allow the structural engineer to design the caissons to achieve the desired geotechnical resistance at the site.

Founding Element	Caisson Diameter	Estimated Design Caisson Founding Elevation	Socket Length
North Abutment	1.2 m	107 m (west limit) to 104 m (east limit)	4 m
	1.2 m	108 m (west limit) to 105 m (east limit)	3 m
	1.2 m or 1.5 m	109 m (west limit) to 106 m (east limit)	2 m
Pier	1.2 m	104.5 m	4 m
	1.2 m	105.5 m	3 m
	1.2 m or 1.5 m	106.5 m	2 m
South Abutment	1.2 m	105 m	4 m
	1.2 m	106 m	3 m
	1.2 m or 1.5 m	107 m	2 m

The fill soils consist of granular materials which may contain perched groundwater above the till deposit. It is anticipated that temporary liners will be required to support the granular soils and saturated cohesionless till soils during construction, especially if perched water conditions are present. If permanent liners are used by the Contractor, the lower 2 m or 4 m section of liner socketted into the till must be removed (i.e. raised) to ensure that an adequate length of socket is present to allow for a bond to develop between the soil and the outside of the concrete caisson. The performance of caissons will depend on the final cleaning and verification of the subgrade quality (very dense sandy silt to sand and silt till) at the base of the caissons. The Ontario Occupational Health and Safety Act (2012) outlines appropriate safety procedures and requirements that must be implemented prior to entry of personnel into the caissons for inspection of the base or alternatively, the inspections may be carried out remotely using visual recording equipment.

The caisson caps for the new foundations should be provided with a minimum of 1.2 m of soil cover to provide adequate protection against frost penetration (per OPSD 3090.101) unless the caps are positioned at the top of the pier columns.

6.5.2 Geotechnical Axial Resistance/Reaction

The caissons will derive their capacity from a combination of base resistance and shaft friction along the "socket" into the "100-blow" till deposit. For a 2 m, 3 m or 4 m long socket into 100 blow till and assuming a 1.2 m or 1.5 m diameter caisson (as appropriate), the recommended values for factored geotechnical axial resistance at ULS and the geotechnical axial reaction at SLS (for 25 mm of settlement) are provided below. These values assume the caisson base is properly cleaned and inspected.

Founding Element	Design Caisson Tip Elevation	Caisson Diameter	Length of Socket	Factored Geotechnical Axial Resistance at ULS	Geotechnical Reaction at SLS (for 25 mm settlement)
North Abutment	107 (west limit) to 104 (east limit)	1.2 m	4 m	7,600 kN	6,500 kN
	108 m (west limit) to 105 m (east limit)	1.2 m	3 m	6,600 kN	5,500 kN
	109 m (west limit) to 106 m (east limit)	1.2 m	2 m	5,600 kN	4,500 kN
		1.5 m		7,600 kN	6,500 kN
Pier	104.5 m	1.2 m	4 m	7,600 kN	6, 500 kN
	105.5 m	1.2 m	3 m	6,600 kN	5,500 kN
	106.5 m	1.2 m	2 m	5,600 kN	4,500 kN
		1.5 m		7,600 kN	6,500 kN
South Abutment	105 m	1.2 m	4 m	7,600 kN	6, 500 kN
	106 m	1.2 m	3 m	6,600 kN	5,500 kN
	107 m	1.2 m	2 m	5,600 kN	4,500 kN
		1.5 m		7,600 kN	6,500 kN

6.5.3 Resistance to Lateral Loads

The resistance to lateral loading developed by the soils in front of the caissons, and the reductions due to group effects, may be determined as per Section 6.4.3.

6.6 Bridge Wing Walls

It is understood that the proposed Holt Road structure may require retaining walls adjacent to the north and south abutments and their associated wing walls. It is assumed that new retaining walls, if required, would be constructed along the shoulders of Holt Road, and the foundations of such new retaining walls would "step up" from the abutment founding level to follow the ground surface profile of the front slope.

Various wall and foundation types have been assessed, taking into account the proposed retaining wall geometry and the subsurface conditions at the site. 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 2 following the text of this report.

- **Conventional concrete retaining walls supported on shallow foundations:** Cantilevered concrete walls supported on shallow foundations (concrete strip footings "stepped" within the embankment fill to follow the ground surface profile) are considered to be a feasible option for this site. It is assumed that the existing fill would be completely removed and replaced with suitable earth (granular) fill as described in .Section 6.8.1. It is anticipated that excavations for these wall foundations would be made in open cut as part of the overall removal of the existing fill below the proposed approach embankments (see Section 6.9.2) and the retaining wall footings would be founded on the granular fill pad, 'perched' within the approach embankment fill. If strip footings founded on competent native soil, or perched in the approach embankments on a compacted Granular 'A' pad are being considered, the recommendations and geotechnical resistances provided in Section 6.3 (Shallow Foundations) can be used for design. Concrete cantilever walls are ideal for semi-integral and conventional abutment design; however, such walls may not be practical for false abutments or for integral abutment design due to the conflicts with pile foundations and lower tolerance to lateral deflections compared to the RSS wall option.

- **Retained Soil System (RSS) walls:** RSS walls are geotechnically feasible for the proposed retaining walls at this site. It is anticipated that excavations for these walls would be made in open cut as part of the overall removal of the existing fill below the proposed approach embankments (see Section 6.9.2). The magnitude of settlement expected from the new approach embankment loading is expected to be able to be tolerated by the reinforced soil mass and should not impact the aesthetic appearance of the wall facing panels; however, joints can be provided in the facing panels to allow for differential settlement if necessary. Alternatively, a two-stage construction process could be considered, where the wall facing panels are installed after the primary settlement is essentially complete.

- **Concrete retaining walls supported on deep foundations:** Concrete retaining walls supported on pile or caisson foundations are an option but are not considered to be practical at this site given that a competent subgrade is present below the existing topsoil and fill materials and the existing topsoil/fill soils are to be removed and replaced with embankment fill. If deep foundations are being considered for support of the walls, the recommendations and geotechnical resistances provided in Sections 6.4 and 6.5 (for steel H-Piles and concrete caisson foundations, respectively) can be used for design.

Based on the above considerations, RSS walls are considered to be the most practicable and cost-effective option for the proposed wing walls/retaining walls at this site (especially for integral abutment design) and are preferred from a geotechnical/foundations perspective. Concrete retaining walls supported on shallow foundations are also considered to be a feasible and acceptable option from a geotechnical/foundations perspective.

Based on the GA drawing provided by URS it is understood that the wing walls for the proposed bridge structure are to be designed using RSS walls and design recommendations are provided below.

6.6.1 RSS Walls

A typical RSS wall has a front facing supported on a strip/block footing placed at shallow depth (preferably below frost depth) below the ground surface in front of the wall. For this site, it is recommended that the existing topsoil and fill material within the RSS wall footprint be subexcavated down to the native till soils prior to construction of both the facing footing and the RSS mass.

The facing footing and reinforced soil mass can be constructed immediately on top of the exposed subgrade or from the subexcavated soil can be replaced with compacted Ontario Provincial Standard Specification (OPSS). Prov1010 (Aggregates) Granular A or Granular B Type II engineered fill up to the design founding level prior to construction of the facing footing and/or reinforced soil mass. This compacted granular pad should extend at least 0.3 m beyond the outside edge of the facing footing, then outward/downward at 1 horizontal to 1 vertical (1H:1V).

Assuming that the facing footing is at least 0.6 m wide and the RSS wall acts as a unit and utilizes the full width of the reinforced soil mass, which has been taken as two-thirds of the height of the wall, the factored geotechnical axial resistances at ULS and the geotechnical reaction at SLS (for 25 mm of settlement) given below may be used for assessment of the reinforced mass and/or facing footing founded on the properly prepared compacted granular fill or on the compact to very dense / stiff to hard till deposit.

RSS Wall	Design Subexcavation Depth / Elevation	Exposed Wall "Face" Height	Factored Geotechnical Resistance at ULS	Geotechnical Reaction at SLS
Northeast Wingwall	2.3 m / 109.4 m	5 m	300 kPa	200 kPa
Norhwest Wingwall	2.9 m / 110.9 m	5 m	300 kPa	200 kPa
Southeast Wingwall	0.8 m / 110.1 m	4 m	300 kPa	200 kPa
Southwest Wingwall	1.8 m/ 109.9 m	4 m	300 kPa	200 kPa

The settlement of the new RSS walls is expected to be less than 25 mm assuming the existing fill is subexcavated and replaced with compacted engineered fill in accordance with SP 206S03 (Earth Excavation and Grading), and OPSS 501 (Compacting), with inspection and field density testing by qualified personnel during placement operations to confirm that appropriate materials are used and that adequate levels of compaction are achieved.

Global stability of the RSS walls is calculated to be greater than 1.5 for exposed wall face heights up to 5 m and assuming the walls are supported within the properly compacted embankment fill overlying the competent native soils. The internal stability of the reinforced earth wall should be assessed by the proprietary product supplier / designer.

The RSS walls should meet the following criteria (RSS Design Guidelines, MTO, 2008) when being selected from an MTO pre-approved DSM list, and the criteria should be included in the Contract Documents:

Criterion	Recommended Minimum Rating
Geometry	Vertical
Performance	High
Appearance	High

RSS Walls should be designed and constructed in accordance with SP 599S22 (Retained Soil System) and SP 599S23 (Retained Soil System – Facing Elements).

6.7 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment stems and on any wing walls/retaining walls will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the magnitude of the surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls. Seismic (earthquake) loading may also need to be taken into account in the design.

6.7.1 Static Considerations

The following recommendations are made concerning the design of the abutment walls and any associated wing walls or retaining walls. These design recommendations and parameters assume level backfill and ground surface behind the walls.

- Select, free-draining granular fill meeting the specifications of OPSS PROV 1010 (Aggregates) Granular ‘A’ or Granular ‘B’ Type II (but with less than 5 percent passing the 200 sieve) should be used as backfill behind the walls. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS 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 such sub-drains and frost taper should be in accordance with OPSD 3101.150 (Walls, Abutment, Backfill, Minimum Granular Requirement) and OPSD 3121.150 (Walls, Retaining, Backfill, Minimum Granular Requirements).

- A minimum compaction surcharge of 12 kPa should be included 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 501 *(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 1.2 m behind the back of the walls (for a restrained wall see 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 see Figure C6.20(b) of the *Commentary* to the *CHBDC*).

- For a restrained wall, the pressures are based on any existing and new approach embankment fill materials and the following parameters (unfactored) may be used:

	Earth Fill
Soil unit weight:	21 kN/m^3
Coefficients of static lateral earth pressure: Active, K_a At rest, K_o	 0.33 0.50

- For an unrestrained wall, where the pressures are based on SP110S13 (Aggregates) Granular A or Granular B Type II fill behind the wall, the following parameters (unfactored) may be assumed:

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

Where the wall support does not allow lateral yielding, at-rest earth pressures should be assumed for the geotechnical design. Where the wall support allows lateral yielding of the stem, active earth pressures should be used in the geotechnical design of the wall structure(s). 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*.

6.7.2 Seismic Considerations

Seismic loading may also need to be considered in accordance with Section 4.6.4 of *CHBDC (2006)*, as such loading can result in increased lateral earth pressures acting on the abutment stem and any associated wing walls/retaining walls.

According to Table C4.2 of the *Commentary to the CHBDC*, this site is located in Seismic Zone 1, and the site-specific zonal acceleration ratio (A) for the Durham area is 0.05. The site-specific peak ground acceleration (PGA) is 0.027g based on the NRC website; however, the more conservative *CHBDC* value has been used in the assessment. The Site Coefficient (S) may be taken as 1.2, consistent with Soil Profile Type II in accordance with Section 4.4.6 and Table 4.4 of *CHBDC* (2006). Based on experience, for the subsurface conditions at this site, a 20 per cent amplification of the ground motion may occur, resulting in an increase in the peak horizontal ground surface acceleration (PGA) from 0.05g to approximately 0.06g. In accordance with Section 4.4.5.1 of CHBDC (2006) and the MTO Bridge Office Policy Memo *"Clarification of What is Considered a Lifeline, Emergency or Other Bridge for Seismic Design Ontario"* (MTO, 2011), seismic analysis is not required for structures located in seismic Performance Zone 1 that are not classified as "lifeline" structures.

6.8 Approach Embankments

The new Highway 401/Holt Road Underpass structure will require placement of engineered fill for the construction of the approach embankments. The existing ground surface at the north and south abutments is at about Elevation 114 m and 111 m, respectively, and the proposed realigned Holt Road grade is at about Elevation 119 m at the abutment locations, resulting in embankments up to about 8 m high.

In accordance with MTO's standard practice, a minimum 2 m wide bench is recommended where embankment slopes are equal to or greater than 8 m high. The stability results for this site indicate that for embankments with side slopes inclined at 2H:1V up to about 8 m high without a mid-height bench have a factor of safety greater than 1.3 against global instability (as discussed in Section 6.8.2); Therefore, if adequate control of surface water in the form of berms or ditches can be provided at the crest of the slope and the slopes are adequately protected with vegetation as noted below, consideration could be given to removing the mid-height bench requirement as the majority of the slope length along both sides of the roadway embankment is less than 8 m high. To reduce erosion of the slopes due to surface water runoff, placement of topsoil and seeding (OPSS 804) is recommended as soon as practicable after construction of the embankments. Consideration may also be given to the use of armoured drainage channels to direct surface water flow from the Holt Road grade to the Highway 401 grade, if applicable

6.8.1 Subgrade Preparation and Embankment Construction

It is not known what the existing Underpass approach embankment materials consist of and the new approach embankments will likely be constructed while the existing underpass remains in use. Therefore, the existing approach embankment material will likely not be available for re-use in the new approach embankments but could be used elsewhere on site where staging permits.

Prior to placing any embankment fill, all topsoil, organic matter and existing loose fill, not forming part of the existing approach embankments, should be stripped from below the approach embankment areas. Considering that the fill contains pockets of organics, as encountered in Boreholes 13-45 to 13-50 and HR-1 and HR-2, it is recommended that all existing fills be removed from within the approach embankment footprint where fill heights are in excess of 4.5 m and from within the footprint of any wing walls (i.e. Retaining Soil System Walls or concrete cantilever walls) to mitigate the potential for differential settlement. If existing fills are to remain below the embankments/ramps where fills are less than 4.5 m there should be a transition zone to avoid abrupt differential settlements that could be propagated to the road surface. A majority of the existing fill that will be excavated consists of granular soil that could be re-used as embankment fill to avoid transportation off-site. Prior to re-use any pockets of organics or clayey soils should be removed from the existing fill.

Any new embankment fill should be placed and compacted in accordance with SP 206S03 (Earth Excavation and Grading), and OPSS 501 (Compacting), with inspection and field density testing by qualified personnel during placement operations to confirm that appropriate materials are used and that adequate levels of compaction are achieved.

The use of suitable granular fill for the approach embankments is recommended rather than the use of cohesive fill, since the majority of settlement of granular fills would occur during construction whereas some settlement of cohesive fills, if used, would occur post-construction (refer to Section 6.8.3).

6.8.2 Embankment Stability

Static and seismic slope stability analyses have been performed for the Holt Road approach embankments, using the commercially available program *Slide (version 6.017),* produced by Rocscience Inc., to check that the target minimum factor of safety is achieved.

Static Stability Analysis

A target minimum factor of safety of 1.3 is normally adopted in the design of slopes under static conditions. This minimum factor of safety is considered appropriate for the proposed embankment construction on this project, considering the design requirements and the available field and laboratory testing data.

The following parameters have been used in the analysis, based on field and laboratory test data as well as accepted correlations (Bowles, 1984 and Kulhawy and Mayne, 1990):

Soil Deposit	Bulk Unit Weight (kN/m^3)	Effective Friction Angle (Degrees)	Undrained Shear Strength (kPa)
New embankment fill (granular fill)	21	32°	-
Existing fill	21	28° to 30°	-
Very stiff to hard clayey silt till/Compact to dense sand and silt till	19	32°	-
Very Dense Sand and Silt Till to Hard Clayey Silt Till (having SPT “N” values greater than 100 blows per 0.3 m of penetration)	20	35°	-

A groundwater level at Elevation 111 m (approximately as measured in the piezometers) was modelled in the analysis.

The stability analyses were completed for an overall 8 m high slope using the parameters outlined above. At the south approach embankment where existing topsoil/fills are less than 1.8 m thick, the analysis assumes that all existing fills (including topsoil, organics and rootlets) are completely stripped from below the approach embankment footprint prior to placing the new embankment fill. At the north approach embankment where fill is up to 4.4 m thick, the analysis conservatively assumes the existing fill containing trace organics is left in place and only the topsoil is stripped prior to placing engineered fill. The results of the static global stability analysis indicate that a minimum factor of safety greater than 1.3 is achieved for 8 m high slopes oriented no steeper than 2H:1V for both scenarios at the south abutment and north abutment locations. The results of the analysis at the south abutment and north approach embankments near the abutments are shown on Figure 1 and 2 respectively.

Short-term shallow sloughing (i.e. surficial failures) could occur on the 2H:1V slope faces, which could be mitigated in the long-term by providing well-vegetated slopes.

Seismic Stability Analysis

Under seismic conditions, the stability of the embankment slopes is assessed using conventional pseudo-static methods of slope stability analysis under the earthquake-induced peak ground acceleration. A calculated factor of safety of 1.0 is considered appropriate for global stability under seismic conditions. A seismic global stability analysis has been performed for the new embankment slopes, using the parameters summarized above.

The pseudo-static seismic slope stability analyses for a 2H:1V slope configuration indicate that the embankment slopes will have a factor of safety greater than 1.2 against deep-seated slope instability, using a peak ground acceleration of 0.06g. The result of the pseudo-static stability analysis at the north approach embankment is shown on Figure 3.

6.8.3 Approach Embankment Settlement

Settlement analysis for the anticipated foundation soil conditions below the new approach embankments was carried out using the commercially available computer program *Settle-3D* (version 2.015), produced by Rocscience, using estimated elastic deformation moduli as given below, based on correlations with the SPT "N" values and engineering judgement from experience with similar soils in this region of Ontario (Bowles, 1984; Kulhawy and Mayne, 1990; Peck et al., 1974).

Soil Conditions	Bulk Unit Weight (kN/m^3)	Elastic Modulus (MPa)
New embankment fill (granular fill)	21	Not applicable
Very stiff to hard clayey silt with sand till/Compact to dense sand and silt till	19	25 MPa
Very dense Sand and Silt Till to Hard Clayey Silt Till (having SPT "N" values greater than 100 blows per 0.3 m of penetration)	20	100 MPa

The settlement analysis assumes any existing fills are completely stripped and the new embankment fill is placed and compacted above the relatively undisturbed native soils.

Based on this assessment, the settlement of the foundation soils under the new up to 8 m high approach embankments is estimated to be less than 25 mm. This settlement is expected to occur relatively quickly during and immediately following construction of the approach embankments based on the nature of the fill and subgrade soils at the site.

The above estimates do not include compression of the new embankment fill itself, which would occur during and after the construction of the embankment depending on the type of materials used. The magnitude of fill compression typically ranges from 0.5 per cent to 1 per cent of the height of the embankment, assuming approximately 98 per cent compaction of the embankment fill is achieved, relative to the material's standard Proctor maximum dry density. In the case where granular fill is used for embankment construction, settlement of the fill itself is expected to occur essentially during embankment construction, whereas non-granular earth fill materials are expected to exhibit some additional settlement over time.

6.9 Construction Considerations

The following sections identify future construction considerations that may impact the detail design and/or require non-standard special provisions during construction.

6.9.1 Removal of Existing Bridge Foundations

The reconstruction and realignment of Holt Road will require removal of the existing bridge following construction of the proposed new bridge structure. It is recommended that foundation removals be limited to removal of the existing pile caps and wall stems only and cutting off the existing piles at the underside of the pile caps, with no extraction of existing timber or steel piles, or shallow concrete footings unless there is a specific conflict. The final ground surface in the area of and above concrete footings left in place should provide for adequate protection from frost penetration (i.e. 1.2 m minimum cover) to minimize the potential for upward jacking of the footings due to frost action.

Excavation for removal of the existing bridge foundations should be performed in accordance with the recommendations in Section 6.9.2, and backfilling to finished grade, or to the top of subgrade, should be done in accordance with the recommendations for embankment construction presented in Section 6.8.1.

6.9.2 Open Cut Excavation

The temporary excavations for removal of unsuitable soils prior to placement of engineered fill or construction of spread/strip footings/pile caps will extend to depths up to 4.4 m below existing grade through the existing loose to compact fill and compact sandy silt to very stiff clayey silt till deposit encountered at the north abutment, and to the very dense sandy silt to sand and silt till.

Where space permits, open-cut excavations into these materials 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 compact/very stiff portion of the near surface deposits are classified as Type 3 soil, while the lower very dense till deposit is classified as a Type 2 material, according to OHSA. Temporary excavations (i.e. those that are open for a relatively short time period) should be made with side slopes no steeper than 1H:1V.

Temporary cut slopes may be required within the existing approach embankments at the existing bridge structure which is likely to remain operational during construction of the new bridge. Considering the existing bridge abutments are supported on piles driven into the very dense till deposit, the temporary stepping of the existing side-slopes is not anticipated to impact operation of the existing bridge; however, temporary protection systems or excavation and placement of new embankment fill in stages may be required to maintain stability/prevent sloughing of the temporary cut slope near the existing bridge approach embankments and to allow for construction of the new abutment/retaining wall foundations.

6.9.3 Temporary Protection Systems

It is anticipated that a temporary protection system will be required along the outside and median edges of the Highway 401 westbound and eastbound lanes, to facilitate the construction of new footing(s) or pile cap(s). The temporary protection system should be designed and constructed in accordance with OPSS 539 (*Temporary Protection Systems).* The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS 539, provided any adjacent utilities can tolerate this magnitude of deformation.

6.9.4 Groundwater Control During Construction

Excavations for construction of the new abutment and pier foundations are expected to extend to or slightly below the groundwater level at the site, which has been measured at about Elevation 111 m in the vicinity of the north abutment. Groundwater seepage should be anticipated from the native till deposits (including cohesionless lenses or interlayers within the till) and perched water may be present within the cohesionless fill deposits above the till. However it is expected that such seepage volumes will be minor and could be controlled by pumping from properly filtered sumps within the foundation excavations. It is anticipated that a Permit to Take Water (PTTW) will not be required for control of the groundwater seepage at this site.

As discussed in Section 6.5, running or flowing water-bearing cohesionless soil strata could be encountered during caisson construction. If caisson foundations are adopted, temporary or permanent caisson liners may be required to support the soils during construction, and special methods such as the use of drilling mud and placement of concrete by tremie methods may be required near the bottom of the caissons to keep the hole open and minimize disturbance to the caisson base.

6.9.5 Subgrade Protection

The sand and silt till/hard clayey silt till (and any interlayers, if present) that will be exposed at the shallow foundations subgrade level will be susceptible to disturbance from construction traffic and/or ponded water. To limit this degradation, it is recommended that a 100 mm thick concrete working slab be placed on the subgrade within four hours after preparation, inspection and approval of the footing subgrade. This requirement can be addressed with a note on the Contract Documents and/or with an NSSP. An example NSSP for the concrete working slab is included in Appendix D.

6.9.6 Obstructions During Pile Driving/Caisson Installation

The soils at this site are glacially derived and as such should be expected to contain cobbles and boulders, which could affect the installation of deep foundations or protection systems. The presence of cobbles and boulders was inferred from auger grinding in the very dense sand and silt till/hard clayey silt till as noted on the Record of Boreholes 13-45, 13-46, 13-49 and 13-50. It is recommended that driving shoes be used on all steel H-piles or tube piles to facilitate driving into the very dense sand and silt till/hard clayey silt till. In addition, it is recommended that an NSSP be included in the Contract Documents to warn the Contractor of the possible presence of cobbles and/or boulders within the overburden soils and an example NSSP is presented in Appendix D.

7.0 CLOSURE

This Foundation Design Report was prepared by Mr. Billy Murphy and by Mr. Matthew Kelly, P.Eng., and reviewed Mr. Kevin Bentley, P.Eng., a geotechnical engineer and Associate with Golder. Mr. Jorge Costa, P.Eng., a Designated MTO Foundations Contact, and principal with Golder, conducted an independent review and quality control audit of this report.

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ASTM International

ASTM D1556 Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of Soils

Contract Design and Estimating and Documentation (CDED)

SP 105S21	Amendment to OPSS 501
SP 206S03	Earth Excavation, Grading
SP 599S22	Retained Soil System
SP 599S23	Retained Soil System – Facing Elements

Ontario Provincial Standard Specifications (OPSS)

OPSS 501	Construction Specification for Compacting
OPSS 539	Construction Specification for Temporary Protection Systems
OPSS 804	Construction Specification for Seed and Cover
OPSS 902	Construction Specification for Excavating and Backfilling Structures
OPSS 903	Construction Specification for Deep Foundations
OPSS PROV1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

Ontario Provincial Standard Drawings (OPSD)

OPSD 3000.201	Foundation Piles – Steel HP310 Oslo Point
OPSD 3001.100	Foundation Piles - Steel Tube Pile Driving Shoe
OPSD 3090.101	Foundation, Frost Penetration Depths for Southern Ontario
OPSD 3101.150	Walls, Abutment, Backfill, Minimum Granular Requirement
OPSD 3121.150	Walls, Retaining, Backfill, Minimum Granular Requirement

Table 1 – COMPARISON OF FOUNDATION ALTERNATIVES

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Spread/strip footings on the very stiff to hard clayey silt till/dense to very dense sand and silt till deposits	1a	• Appropriate geotechnical axial resistances readily available for support of pier, abutments and associated wing walls/retaining walls • Adjacent existing structure piers supported on shallow foundations, and appears to have performed satisfactory • Standard construction operation • Same foundations type as for pier	• Temporary excavations (to a depth of up to 4.4 m below the existing grade) may require temporary excavation support • Precludes use of integral abutments; potentially greater maintenance required at abutments • Lower, but adequate, geotechnical resistances available than for deep foundations • Stepped foundation between east and west limit of north abutment required to limit potential for differential settlement	• Less expensive than deep foundations although bridge maintenance costs may be higher due to non-integral abutment configuration	• Traffic disruption to Hwy 401 can be reduced if temporary protection systems are used • Relatively low risk of significant groundwater seepage for excavations • Founding elevations to consistent competent till ranges from Elev. 107.9 m to 110.5 m at north abutment • Excavations to footing founding level could extend to/below the groundwater level requiring dewatering • Differential settlement (up to 25mm) between abutments and between the east and west limit of the north abutment resulting in stepped foundation
Spread/strip footings perched on compacted granular pad in approach embankment fill (abutments only)	1b	• Feasible for support of abutments and associated wing walls/retaining walls • Abutment footings can be maintained higher than footings founded on till deposit and do not require subexcavation or temporary protection systems	• Precludes use of integral abutments; potentially greater maintenance required at abutments • Existing fill (up to 4.4 m thick) will need to be subexcavated from below approach embankment footprint and replaced with compacted granular pad • Potential for differential settlement between abutments and pier	• Less expensive than deep foundations, although bridge maintenance costs may be higher due to non-integral abutment configuration • Similar cost to footings on the underlying till	• Geotechnical resistance relies on quality of placement and compaction of engineered fill • Potential for differential settlements if existing fill is not stripped from below approach embankments and due to inconsistent strength of soil strata in near – surface tills • Excavation for fill replacement could extend below the groundwater level

Steel H-piles or tube piles driven to found within the very dense sand and silt till	2	• Subsurface conditions are appropriate for support of pier, abutments and associated wing walls/retaining walls • Limited temporary excavation for pile caps compared to deeper excavation and temporary excavation support requirements for shallow footings • Allows for integral abutment construction (steel H-piles) • Higher geotechnical axial resistance available compared to shallow foundations • Can be used in combination with shallow foundations for the central pier	• Potential for encountering obstructions (cobbles and/or boulders) during pile driving that could result in piles "hanging up" and not achieving a minimum pile embedment length (typically 5 m) for integral abutment design • Piles of varying length will likely be required to found within the very dense till deposit • If piles "hang up", pre-augering may be required • Requires pre-augered holes at North abutment to achieve minimum pile length for integral abutment design • Potential for traffic disruption due to requirement for large piling equipment • Tube piles not normally accepted by MTO for integral abutment design	• Lower relative cost compared with caisson option • Higher relative cost compared to shallow foundation options • Steel H-piles typically lower cost than tube piles • Additional cost for pre-augering holes at North Abutment	• Conventional construction methods for H-pile foundations • Potential for piles to "hang up" on cobbles/boulders or not penetrating sufficiently into the very dense till deposit and pre-augering may be required • Excavations for pile caps could extend to/below groundwater level requiring dewatering
Caissons founded within the very dense sand and silt till	3	• Subsurface conditions are appropriate for support of piers, abutments and wing walls/ retaining walls • Higher capacity than for steel H-piles, so reduced number of deep foundation elements compared to steel H-piles • Caissons can be designed to be continuous to act as columns above ground, thereby eliminating caisson/pile caps below grade and associated subexcavation requirements	• Temporary or permanent liners may be required through loose to compact granular fills and/or saturated cohesionless till soils • Precludes use of integral abutments • Large staging area required and will likely lead to traffic disruption	• Higher cost compared with shallow foundations or steel H-piles •	• Risk of loosening soils at base of caissons and potential need for temporary or permanent liners if water table is higher than expected • Difficulties augering through till soil if cobbles/ boulders are present as encountered in four boreholes and as should be anticipated to be present in the glacial till

Micropiles	N/A	• Subsurface conditions are appropriate for support of piers, abutments and wing walls/ retaining walls, however risks/consequences and increased costs make this option not preferrable • Can drill through cobbles and boulders easier than caissons or pre-drilled H-piles	• Lower axial capacity than steel piles or caissons requiring more elements to resist loads • Integral abutment design not possible if micropiles used • Requires excavation and possible shoring for construction of pile caps • Requires site specific micropile design	• Higher construction costs than for steel piles or caissons • Increased costs due to requirement for additional geotechnical investigation and load testing during consruction	• Higher risk of improper installation/construction than for micropiles founded in bedrock • Risk of changing soil conditions below depth of current borehole investigation • High risk of construction difficulties due to lack of experienced contractors available

TABLE 2 – COMPARISON OF RETAINING WALL TYPES AND FOUNDATION ALTERNATIVES

Wall Type and Foundation Option	Rank	Advantages	Disadvantages	Risks/Consequences	Relative Costs
Concrete retaining walls on shallow foundations	• 2	• Wall footings could be founded on granular pads perched within approach embankment fill. • Ideal in association with semi-integral abutment design.	• Less flexible and therefore less tolerant to lateral movements for integral abutment design. • May conflict with steel H-Piles for integral abutment design.	• Conventional excavation and construction techniques. • Relatively longer construction time for formwork and cast in place construction.	• Higher cost relative to RSS wall.
Retained soil system (RSS) walls	• 1	• More tolerant of post-construction settlements, although this is not anticipated to be a significant issue for this site. • Wall footing, facing and reinforced soil mass could be founded on granular pads perched within the approach embankment fill. • Ideal in association with False Abutment design for integral abutments (i.e. will not conflict with steel H-Piles).	• Proprietary design and construction required; although performance criteria has been provided.	• Conventional excavation and construction techniques.	• Lower cost than concrete retaining wall.
Concrete retaining walls supported on deep foundations	• 3	• Considering existing fills will be sub-excavated from below wall and embankment footprint, no advantages gained in supporting the wall on deep foundations.	• Not practical given competent soil conditions at relatively shallow depth.	• More specialized equipment and skilled labour required.	• Higher costs compared to concrete retaining walls on shallow foundations and RSS wall options.

MINISTRY OF TRANSPORTATION, ONTARIO

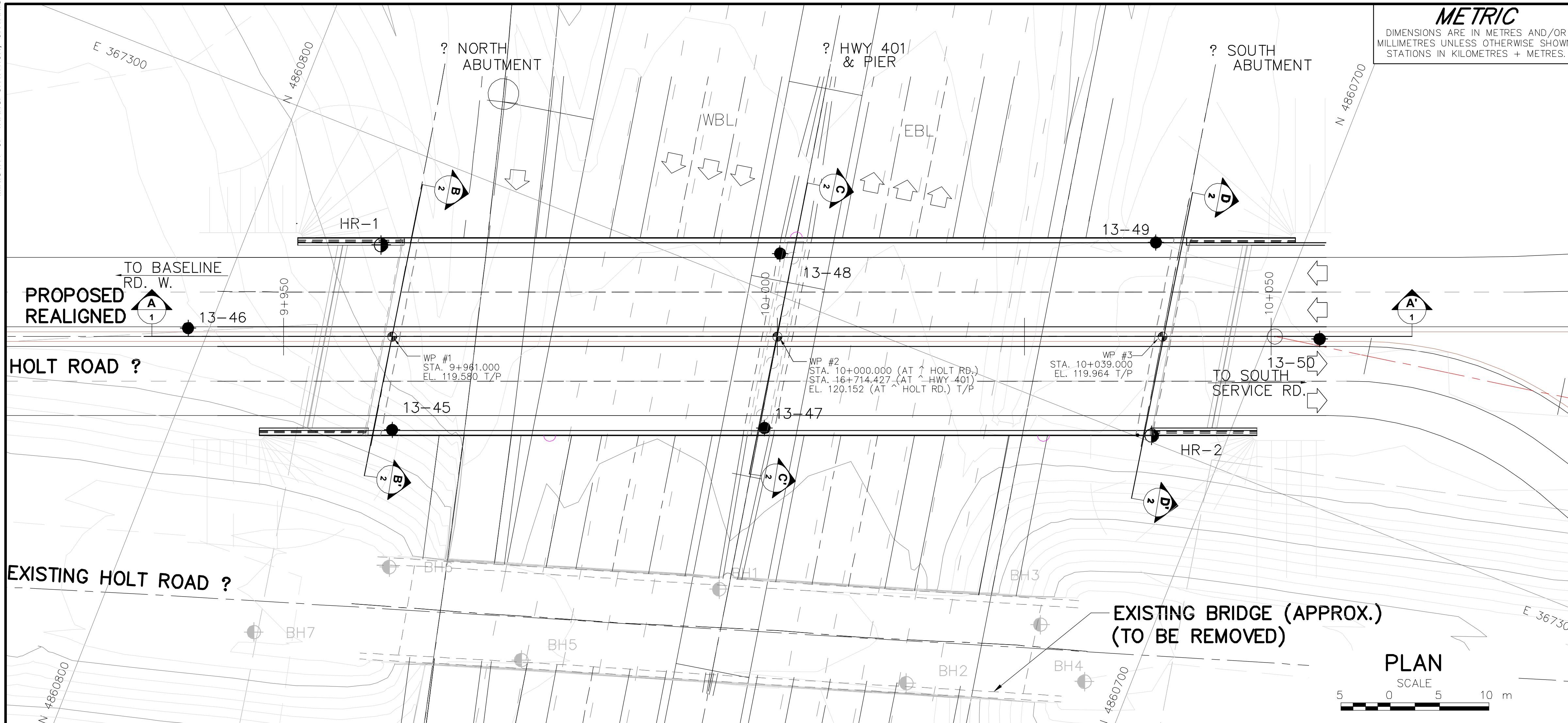
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 2101-08-00

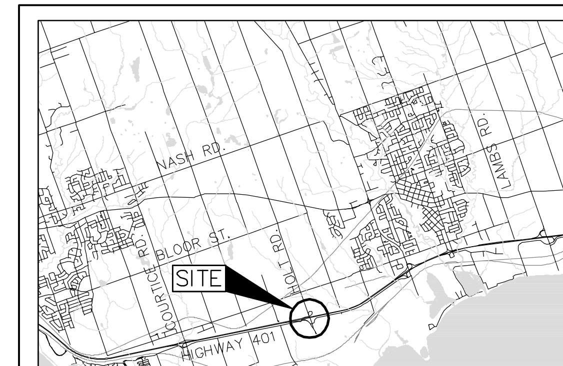
HIGHWAY 401
HOLT ROAD INTERCHANGE STRUCTURE
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



PLAN
SCALE
5 0 5 10 m



KEY PLAN
SCALE
2 0 2 4 km

LEGEND

- Borehole – Current Investigation
- Borehole – Preliminary Investigation (Golder 2012)
- Borehole – Previous Investigation (1961)
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL measured in piezometer
- WL upon completion of drilling September 9, 2013

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
13-45	113.8	4860779.0	367273.0
13-46	114.1	4860802.0	367275.0
13-47	112.3	4860744.0	367287.0
13-48	111.8	4860749.0	367304.0
13-49	110.9	4860714.0	367319.0
13-50	110.9	4860695.0	367316.0
HR-1	111.7	4860786.9	367290.0
HR-2	111.7	4860707.2	367300.7

NOTES

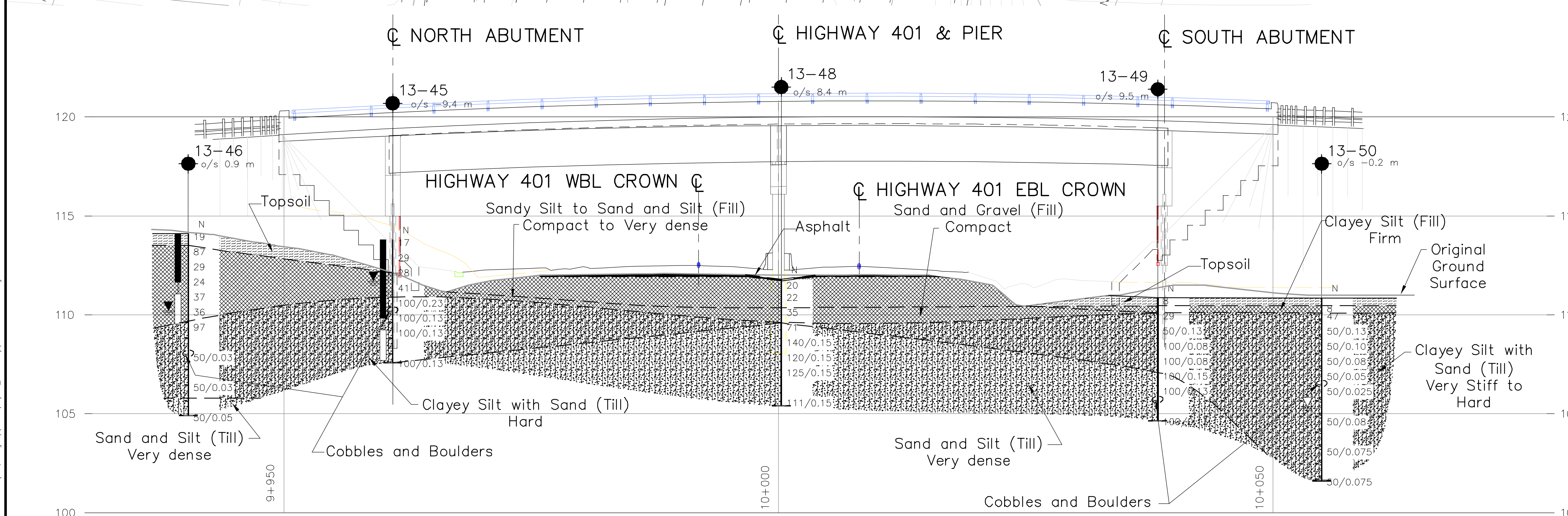
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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 plan provided in digital format by URS, drawing file no. 120921-X-Design_Holt Preferred_ACAD 2007.dwg, received January 3, 2013 and 01-HoltRd_GA_Dec 20 2013.dwg, received December 20, 2013.



CENTRELINE PROFILE ALONG REALIGNED HOLT ROAD
HORIZONTAL SCALE 5 0 5 10 m
VERTICAL SCALE 2.5 0 2.5 5 m

NO.	DATE	BY	REVISION

Geocres No. 30M15-154

HWY. 401	PROJECT NO. 09-1111-0019		DIST.
SUBM'D. MWK	CHKD. MWK	DATE: Dec. 2013	SITE: 21-159
DRAWN: JFC	CHKD. KJB	APPD. JMAC	DWG. 1

PLOT DATE: January 6, 2014
FILENAME: T:\Projects\2009\09-1111-0019 (URS,Darlington)\-AB- (Hwy401_Holt Road)\0911110019AB001.dwg

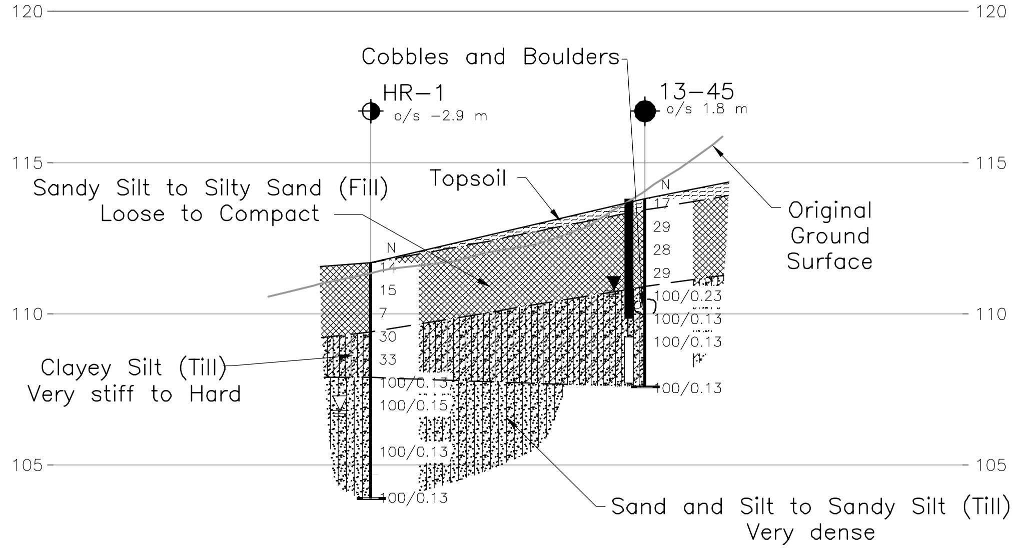
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 2101-08-00

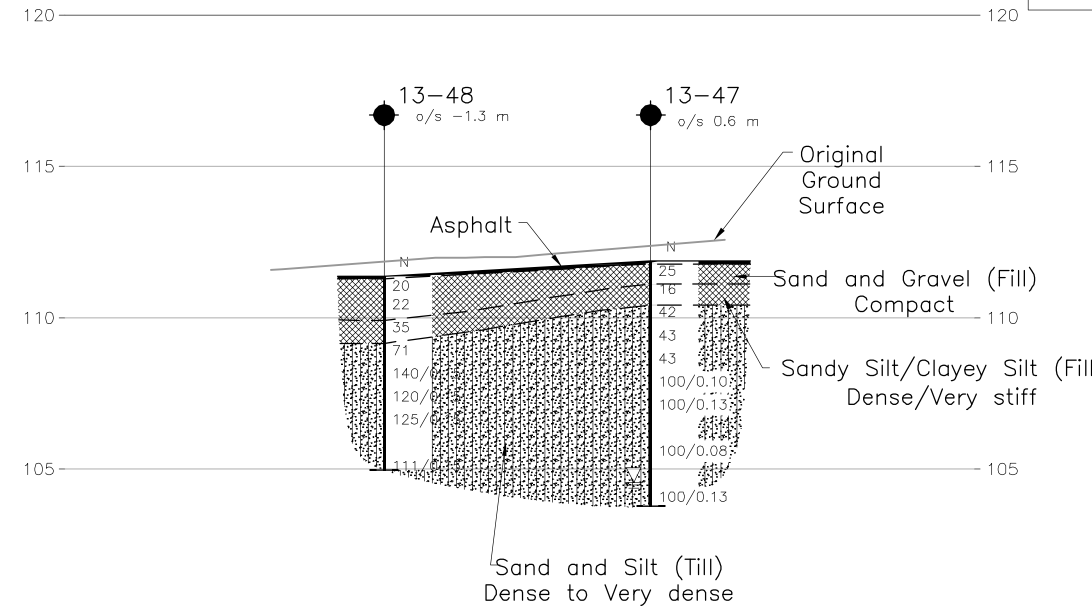
HIGHWAY 401
HOLT ROAD INTERCHANGE STRUCTURE
SOIL STRATA

SHEET

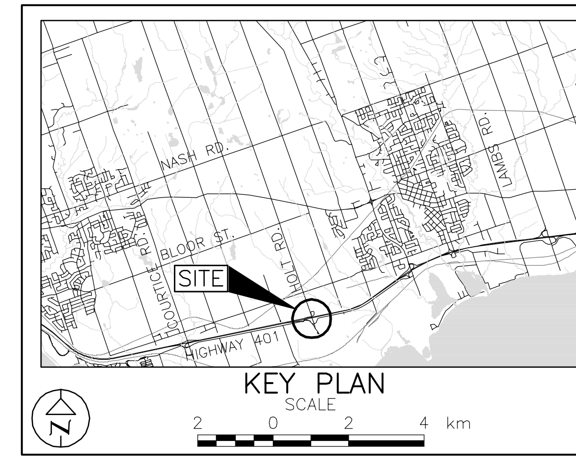
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



B-B' 1
CROSS-SECTION (NORTH ABUTMENT)
STATION 9+961
HORIZONTAL SCALE 5 0 5 10 m
2.5 0 2.5 5 m VERTICAL SCALE



C-C' 1
CROSS-SECTION (PIER)
STATION 10+000
HORIZONTAL SCALE 5 0 5 10 m
2.5 0 2.5 5 m VERTICAL SCALE

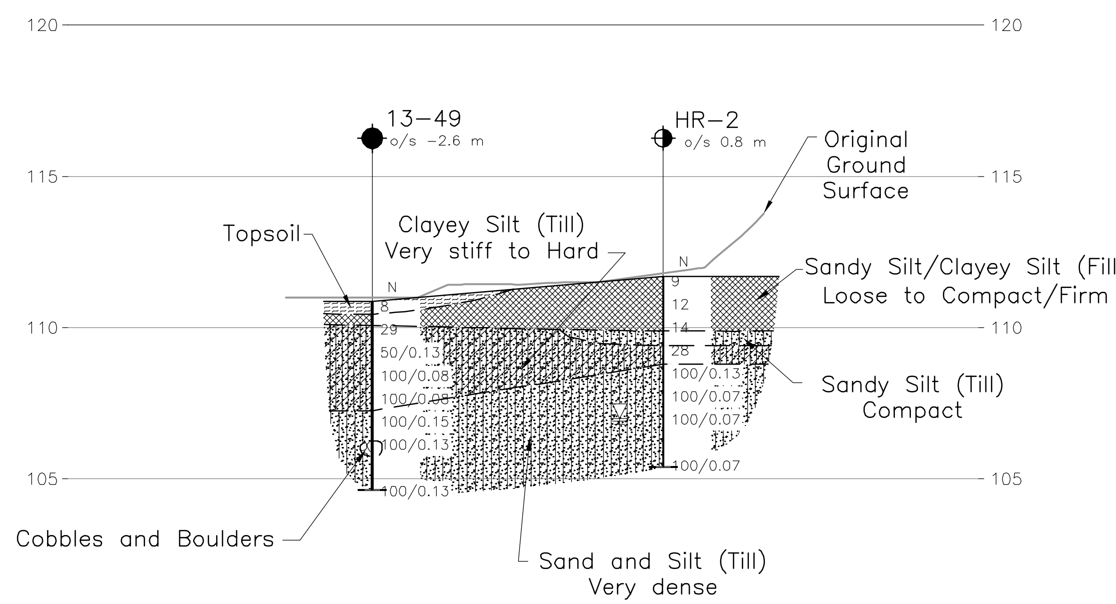


LEGEND

- Borehole – Current Investigation
- Borehole – Preliminary Investigation (Golder 2012)
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL measured in piezometer
- WL upon completion of drilling September 9, 2013

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
13-45	113.8	4860779.0	367273.0
13-47	112.3	4860744.0	367287.0
13-48	111.8	4860749.0	367304.0
13-49	110.9	4860714.0	367319.0
BH HR-1	111.7	4860786.9	367290.0
BH HR-2	111.7	4860707.2	367300.7



D-D' 1
CROSS-SECTION (SOUTH ABUTMENT)
STATION 10+039
HORIZONTAL SCALE 10 0 10 20 m
2 0 2 4 m VERTICAL SCALE

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.

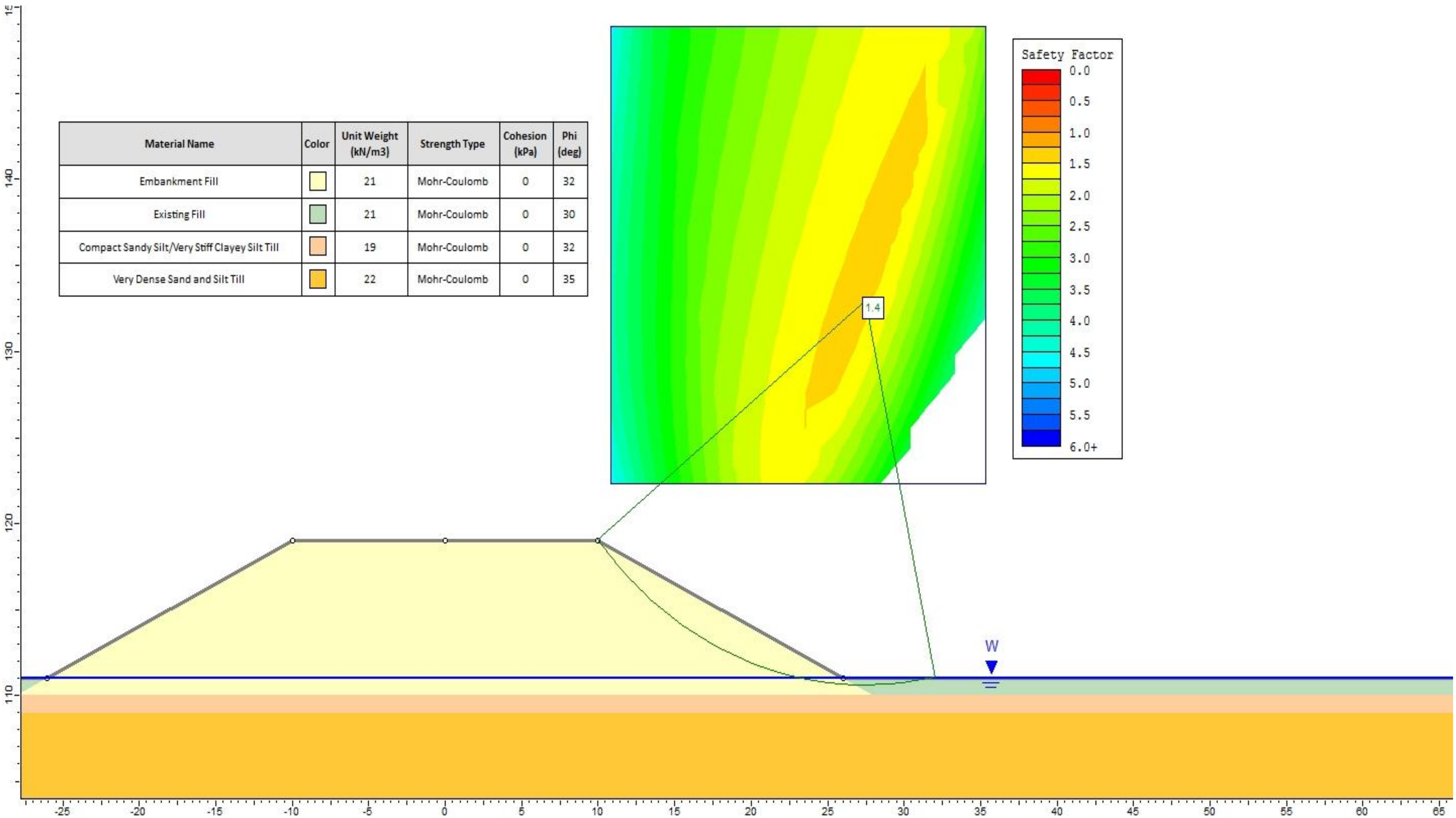
NO.	DATE	BY	REVISION

Geocres No. 30M15-154

HWY. 401	PROJECT NO. 09-1111-0019		DIST.
SUBM'D. MWK	CHKD. MWK	DATE: Dec. 2013	SITE: 21-159
DRAWN: JFC	CHKD. KJB	APPD. JMAC	DWG. 2

STATIC GLOBAL STABILITY
HWY 401/HOLT ROAD INTERCHANGE, SOUTH APPROACH EMBANKMENT

Figure 1

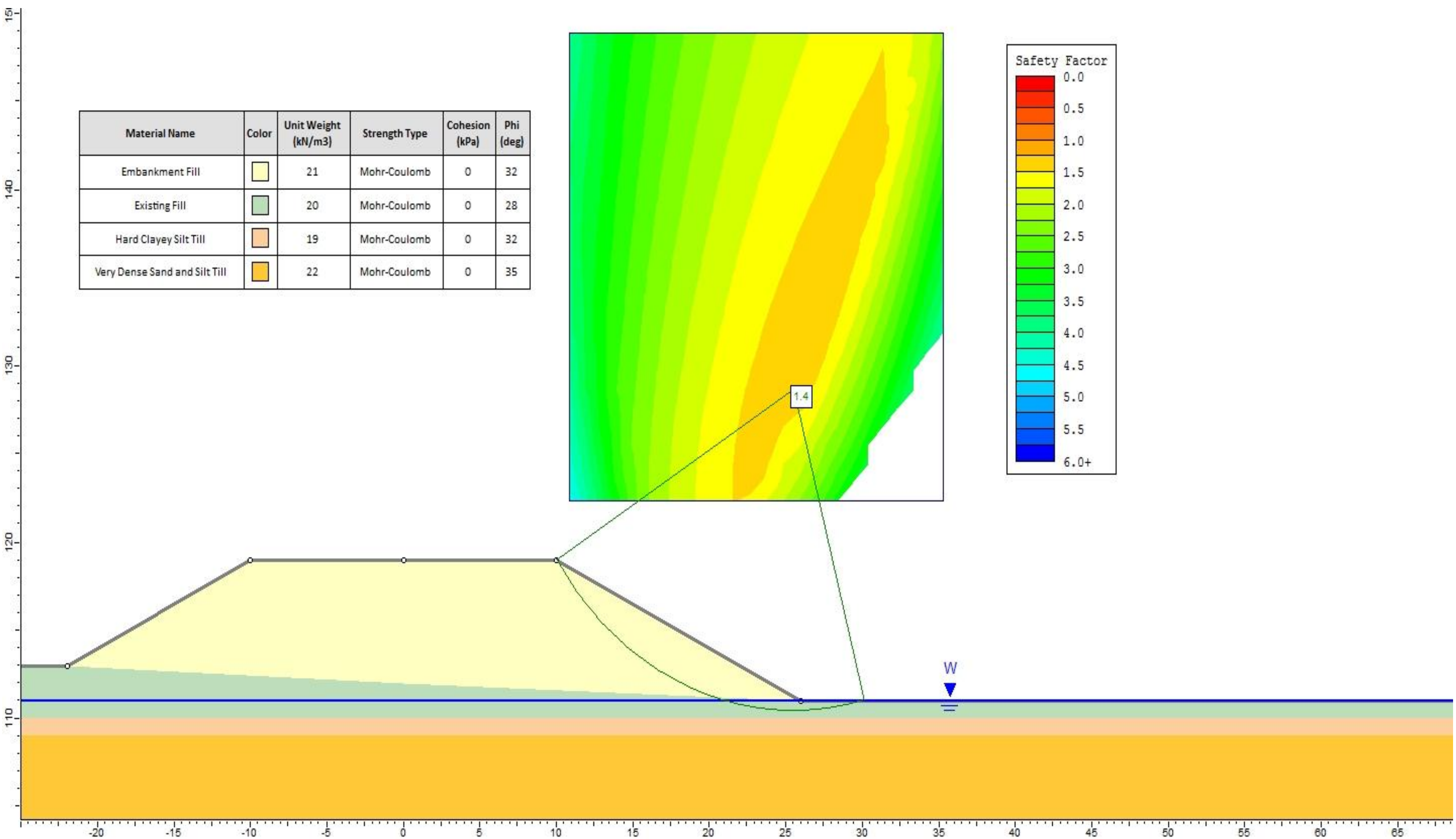


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Embankment Fill		21	Mohr-Coulomb	0	32
Existing Fill		21	Mohr-Coulomb	0	30
Compact Sandy Silt/Very Stiff Clayey Silt Till		19	Mohr-Coulomb	0	32
Very Dense Sand and Silt Till		22	Mohr-Coulomb	0	35

Date: September 2013
Project No: 09-1111-0019

Analysis By: BM **Reviewed By:** KJB

Golder Associates

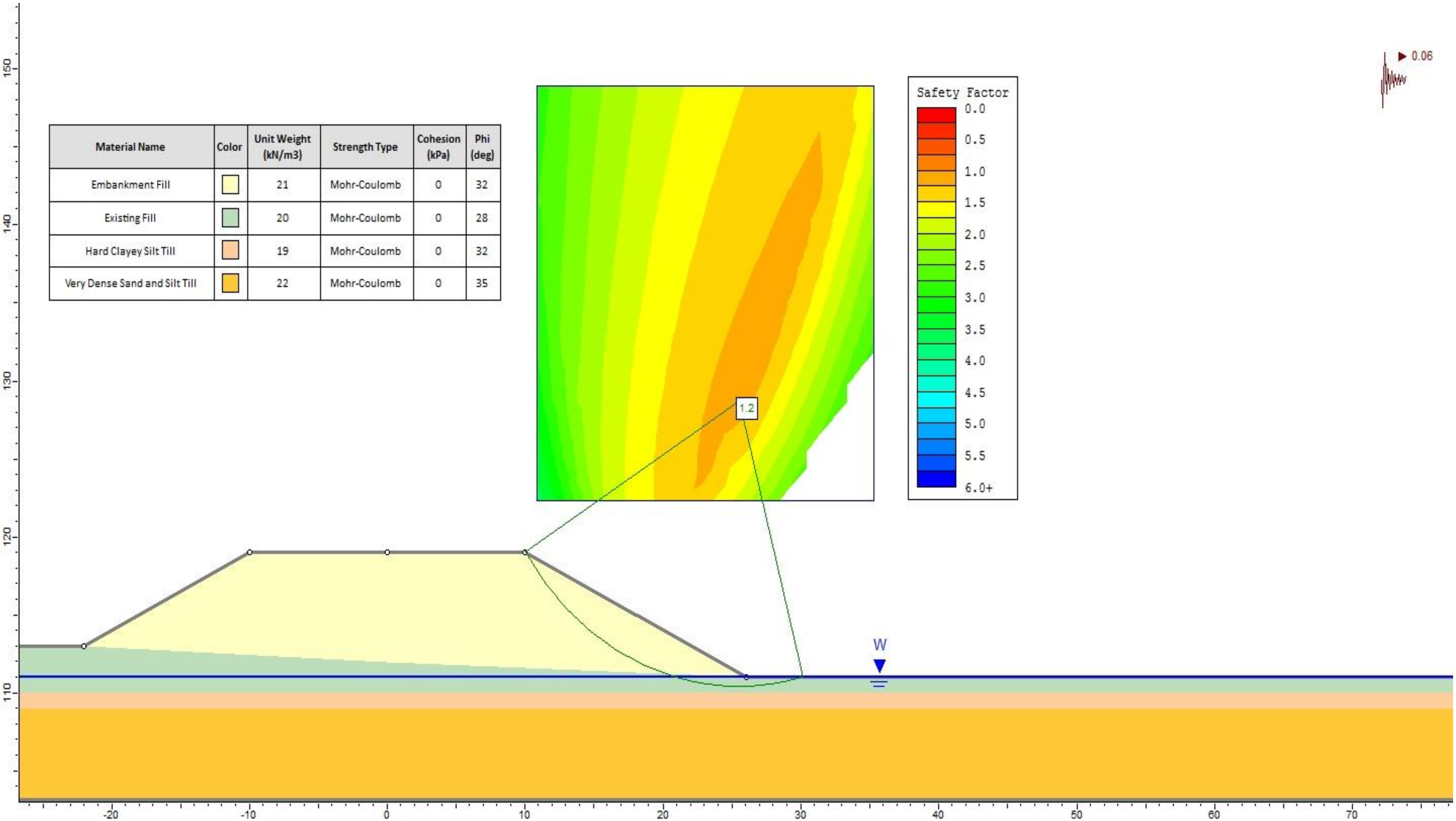


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Embankment Fill		21	Mohr-Coulomb	0	32
Existing Fill		20	Mohr-Coulomb	0	28
Hard Clayey Silt Till		19	Mohr-Coulomb	0	32
Very Dense Sand and Silt Till		22	Mohr-Coulomb	0	35

SEISMIC GLOBAL STABILITY
HWY 401/HOLT ROAD INTERCHANGE, NORTH APPROACH EMBANKMENT

Figure 3

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Embankment Fill		21	Mohr-Coulomb	0	32
Existing Fill		20	Mohr-Coulomb	0	28
Hard Clayey Silt Till		19	Mohr-Coulomb	0	32
Very Dense Sand and Silt Till		22	Mohr-Coulomb	0	35



Date: September 2013
Project No: 09-1111-0019

Analysis By: BM **Reviewed By:** KJB

APPENDIX A

Borehole Records

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

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

Notes:
1. $\tau = c' + \sigma' \tan \phi'$
2. shear strength = (compressive strength)/2

Golder Associates

Foundation Design

RECORD OF BOREHOLE No 13-45

SHEET 1 OF 1 **METRIC**

PROJECT 09-1111-0019
G.W.P. 2101-08-00
LOCATION N 4860779.0 ;E 367273.0
ORIGINATED BY JLC
DIST HWY 401
BOREHOLE TYPE 120 mm O.D. Continuous Flight Solid Stem Power Auger
COMPILED BY BM
DATUM Geodetic
DATE May 28, 2013
CHECKED BY MWK

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
113.8	GROUND SURFACE							
0.0	TOPSOIL	1	SS	17				
113.4 / 0.4	Sand and silt, trace to some gravel, some clay, trace organics (FILL) Compact Grey to dark brown Moist	2	SS	29				
	Pockets of organics below a depth of 1.8 m (Elev. 112.0 m)	3	SS	28	8	42	37	13
		4	SS	41	15	37	33	15
110.9 / 2.9	CLAYEY SILT with SAND, trace gravel (TILL) Hard Brown to grey Moist	5	SS	100/0.23				
	Auger grinding on possible cobbles and boulders below 3.0 m depth	6	SS	100/0.13				
		7	SS	100/0.13	3	40	42	15
107.6 / 6.2	END OF BOREHOLE	8	SS	100/0.13				

NOTES:

1. Borehole caved at a depth of 5.8 m below ground surface (Elev. 108.0 m) upon completion of drilling.

2. Water level at 3.0 m below ground surface (Elev. 110.8 m) upon completion of drilling.

3. Water level measurements in Piezometer:

Date	Depth (m)	Elev. (m)
05/29/13	3.0	110.8
09/10/13	2.1	111.7

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (113, 112, 111, 110, 109, 108); 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 (%)

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

GTA-MTO 001 09-1111-0019.GPJ GAL-GTA.GDT 12/23/13

Golder Associates

RECORD OF BOREHOLE No 13-46

SHEET 1 OF 1 **METRIC**

PROJECT 09-1111-0019
G.W.P. 2101-08-00
LOCATION N 4860802.0 ;E 367275.0
ORIGINATED BY JLC
DIST HWY 401
BOREHOLE TYPE 120 mm O.D. Continuous Flight Solid Stem Power Auger
COMPILED BY BM
DATUM Geodetic
DATE June 11, 2013
CHECKED BY MWK

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
114.1	GROUND SURFACE						
0.0	TOPSOIL Compact Dark brown Moist	1	SS	19	114		
113.5 0.6	Sand and silt, some gravel, some clay, trace organics (FILL) Compact to very dense Grey to black Moist	2	SS	87	113	○	
		3	SS	29	112	○	
		4	SS	24			
	Pockets of wood fibres/rootlets below Elev. 111.0 m	5	SS	37	111	○	16 38 32 14
		6	SS	36	110		
109.7 4.4	CLAYEY SILT with SAND, trace to some gravel (TILL) Hard Brown to grey Moist - - - - - - - - - - - - - - - Auger grinding on possible cobbles and boulders below 5.1 m depth	7	SS	97	109	○	10 39 36 15
		~~8~~	~~SS~~	~~50/0.03~~	108		
		~~9~~	~~SS~~	~~50/0.03~~	107	○ ├─┤	
105.8 8.3	SAND and SILT, trace to some gravel, some clay (TILL) Very dense Grey Moist				106		
104.9 9.2	END OF BOREHOLE	~~10~~	~~SS~~	~~50/0.05~~	105		

Column headings also printed: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; 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.

NOTES:

1. Borehole caved at a depth of 8.7 m below ground surface (Elev. 105.4 m) upon completion of drilling.

2. Water level in caved borehole at a depth of 6.2 m below ground surface (Elev. 107.9 m) upon completion of drilling.

3. Water level measurements in Piezometer:

Date	Depth (m)	Elev. (m)
09/10/13	3.9	110.2

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

Golder Associates

RECORD OF BOREHOLE No 13-47

SHEET 1 OF 1 **METRIC**

PROJECT 09-1111-0019			
G.W.P. 2101-08-00	LOCATION N 4860744.0 ;E 367287.0		ORIGINATED BY JLC
DIST	HWY 401	BOREHOLE TYPE 120 mm O.D. Continuous Flight Solid Stem Power Auger	COMPILED BY BM
DATUM Geodetic	DATE June 9, 2013		CHECKED BY MWK

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
112.3	GROUND SURFACE						
0.0	ASPHALT						
0.1	Sand and gravel, some silt (FILL) Compact Brown Moist	1	SS	25			
111.5 / 0.8	Clayey silt, with sand, trace to some gravel, trace organics (FILL) Very stiff Brown Moist	2	SS	16			9 43 32 16
110.9 / 1.5	SAND and SILT, some gravel, some clay (TILL) Dense to very dense Grey Moist	3	SS	42			
		4	SS	43			
		5	SS	43			
		6	SS	100/0.10			15 38 31 16
		7	SS	100/0.13			
		8	SS	100/0.08			
					∇ (105.0)		
		9	SS	100/0.13			
104.2 / 8.1	END OF BOREHOLE						

Chart headings: 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; ELEVATION SCALE 112 111 110 109 108 107 106 105.

NOTE:

1. Water level in open borehole at a depth of 7.3 m below ground surface (Elev. 105.0 m) upon completion of drilling.

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

Golder Associates

RECORD OF BOREHOLE No 13-48

SHEET 1 OF 1

METRIC

PROJECT 09-1111-0019
G.W.P. 2101-08-00
DIST
HWY 401
DATUM Geodetic
LOCATION N 4860749.0 ;E 367304.0
BOREHOLE TYPE 120 mm O.D. Continuous Flight Solid Stem Power Auger
DATE June 9, 2013
ORIGINATED BY JLC
COMPILED BY BM
CHECKED BY MWK

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
111.8	GROUND SURFACE						
0.0 / 0.1	ASPHALT						
	Sand and gravel, trace silt (FILL) Compact Brown Moist	1	SS	20			
		2	SS	22		111	
110.4 / 1.5	Sandy silt, some gravel, trace clay, trace organics (FILL) Dense Brown to grey Moist	3	SS	35		110	
109.6 / 2.2	SAND and SILT, trace to some gravel, some clay (TILL) Very dense Grey Moist	4	SS	71		109	22 35 30 13
		5	SS	140/0.15			
		6	SS	120/0.15		108	5 44 36 15
		7	SS	125/0.15		107	
		8	SS	111/0.15		106	
105.4 / 6.4	END OF BOREHOLE						

NOTE:

1. Open borehole dry on completion of drilling.

Chart headings: 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

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

RECORD OF BOREHOLE No 13-49

SHEET 1 OF 1

METRIC

PROJECT 09-1111-0019

G.W.P. 2101-08-00 LOCATION N 4860714.0 ;E 367319.0 ORIGINATED BY JLC

DIST HWY 401 BOREHOLE TYPE 120 mm O.D. Continuous Flight Solid Stem Power Auger COMPILED BY BM

DATUM Geodetic DATE May 27, 2013 CHECKED BY MWK

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
111.0	GROUND SURFACE					
0.0	TOPSOIL					
110.6	Loose Dark brown Moist	1	SS	8		
110.2 0.8	Clayey silt, some sand, trace gravel, trace organics (FILL) Firm Dark brown Moist	2	SS	29	110	18 44 30 8
	CLAYEY SILT with SAND, trace to some gravel (TILL) Very stiff to hard Brown to grey Moist	3	SS	50/0.13		
		4	SS	100/0.08	109	
		5	SS	100/0.08	108	
107.4 3.6	SAND and SILT, trace to some gravel, trace to some clay (TILL) Very dense Grey Moist	6	SS	100/0.15	107	7 42 39 12
		7	SS	100/0.13		
	Auger grinding on probable cobbles or boulders at 4.9 m (Elev. 106.0 m)				106	
104.8		8	SS	100/0.13	105	
6.2	END OF BOREHOLE NOTE: 1. Open borehole dry on completion of drilling.					

Column headings also printed: STRAT PLOT; GROUND WATER CONDITIONS; 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

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

Golder Associates

Foundation Design

RECORD OF BOREHOLE No 13-50

SHEET 1 OF 1

METRIC

PROJECT 09-1111-0019

G.W.P. 2101-08-00

LOCATION N 4860695.0 ;E 367316.0

ORIGINATED BY JLC

DIST HWY 401

BOREHOLE TYPE 120 mm O.D. Continuous Flight Solid Stem Power Auger

COMPILED BY BM

DATUM Geodetic

DATE May 27, 2013

CHECKED BY MWK

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%) w	GR	SA	SI	CL
111.0	GROUND SURFACE										
0.0	TOPSOIL										
110.6	Loose, Brown, Moist	1	SS	8							
110.2 / 0.8	Clayey silt, some sand, trace gravel, trace organics (FILL), Dark brown, Moist	2	SS	47		110	○	5	37	41	17
	CLAYEY SILT with SAND, trace to some gravel, occasional silt seams (TILL), Hard, Brown to grey, Moist	3	SS	50/0.13			○	2	39	44	15
						109					
		4	SS	50/0.10			○				
		5	SS	50/0.08		108	○	10	42	35	13
	---------------	6	SS	50/0.05		107					
	Auger grinding on inferred cobbles and boulders below 4.0 m depth	7	SS	50/0.03			○ H				
						106					
					∇						
		8	SS	50/0.05		105	○ H				
						104					
		9	SS	50/0.08		103	○				
101.8		10	SS	50/0.08		102	○				
9.2	END OF BOREHOLE										

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

NOTES:

1. Water level at a depth of 5.5 m below ground surface (Elev. 105.5 m) during drilling.

2. Borehole caved to a depth of 6.1 m below ground surface (Elev. 104.9 m) upon completion of drilling.

$+^3, \times^3$: Numbers refer to Sensitivity

$\circ^{3\%}$ STRAIN AT FAILURE

GTA-MTO 001 09-1111-0019.GPJ GAL-GTA.GDT 12/23/13

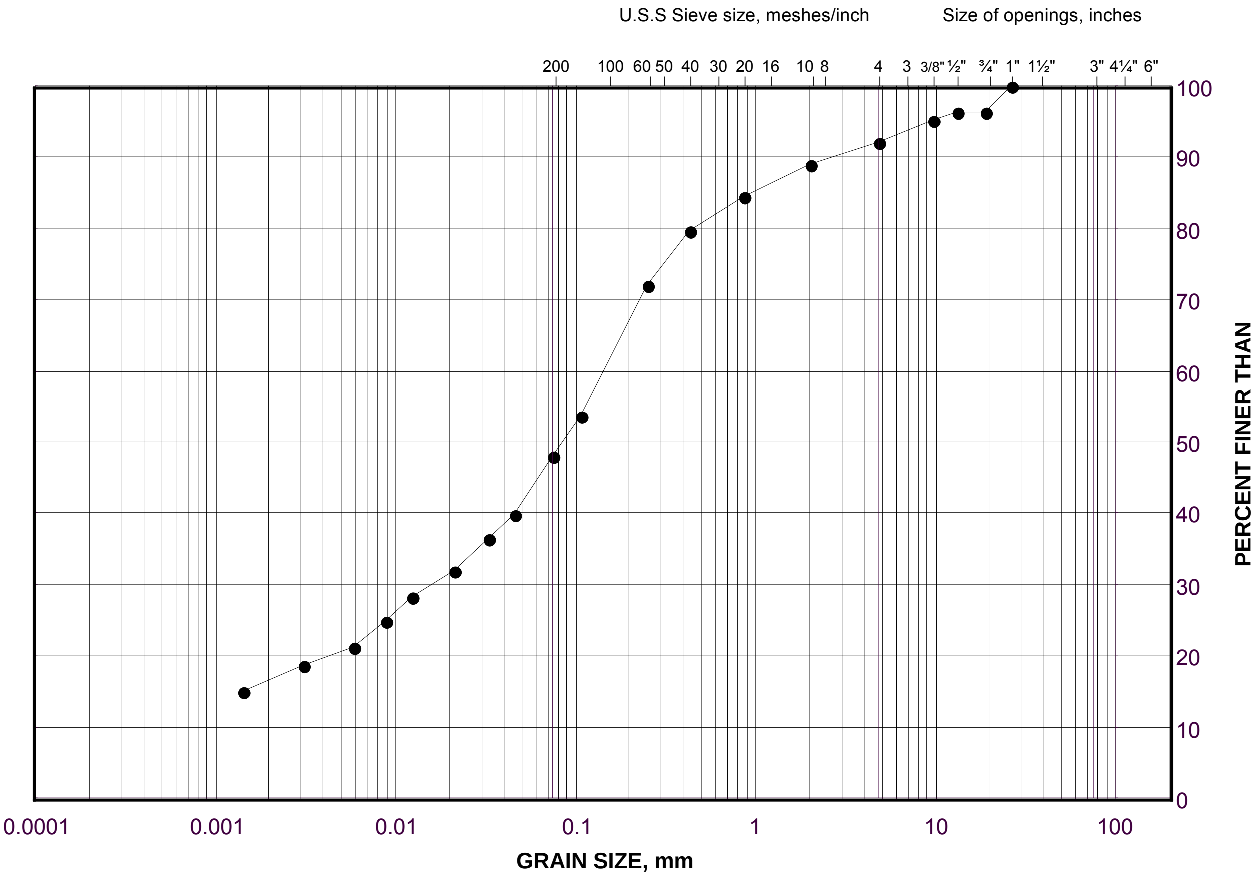
APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION

Clayey Silt with Sand (FILL)

FIGURE B1



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

LEGEND

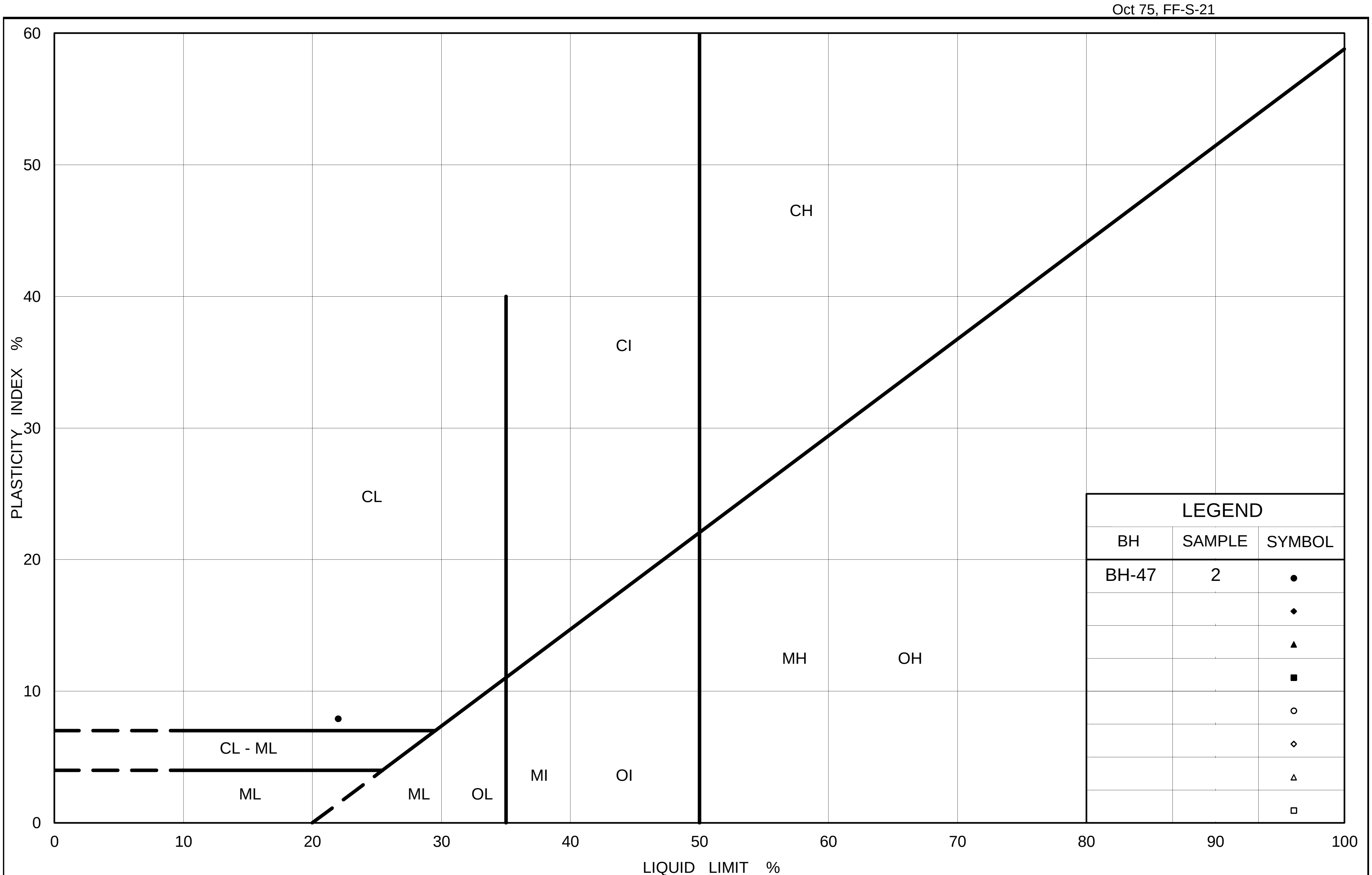
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	BH-47	2	111.3

Project Number: 09-1111-0019

Checked By: KJB

Golder Associates

Date: 01-Oct-13



Ministry of Transportation

Ontario

PLASTICITY CHART
Clayey Silt with Sand (FILL)

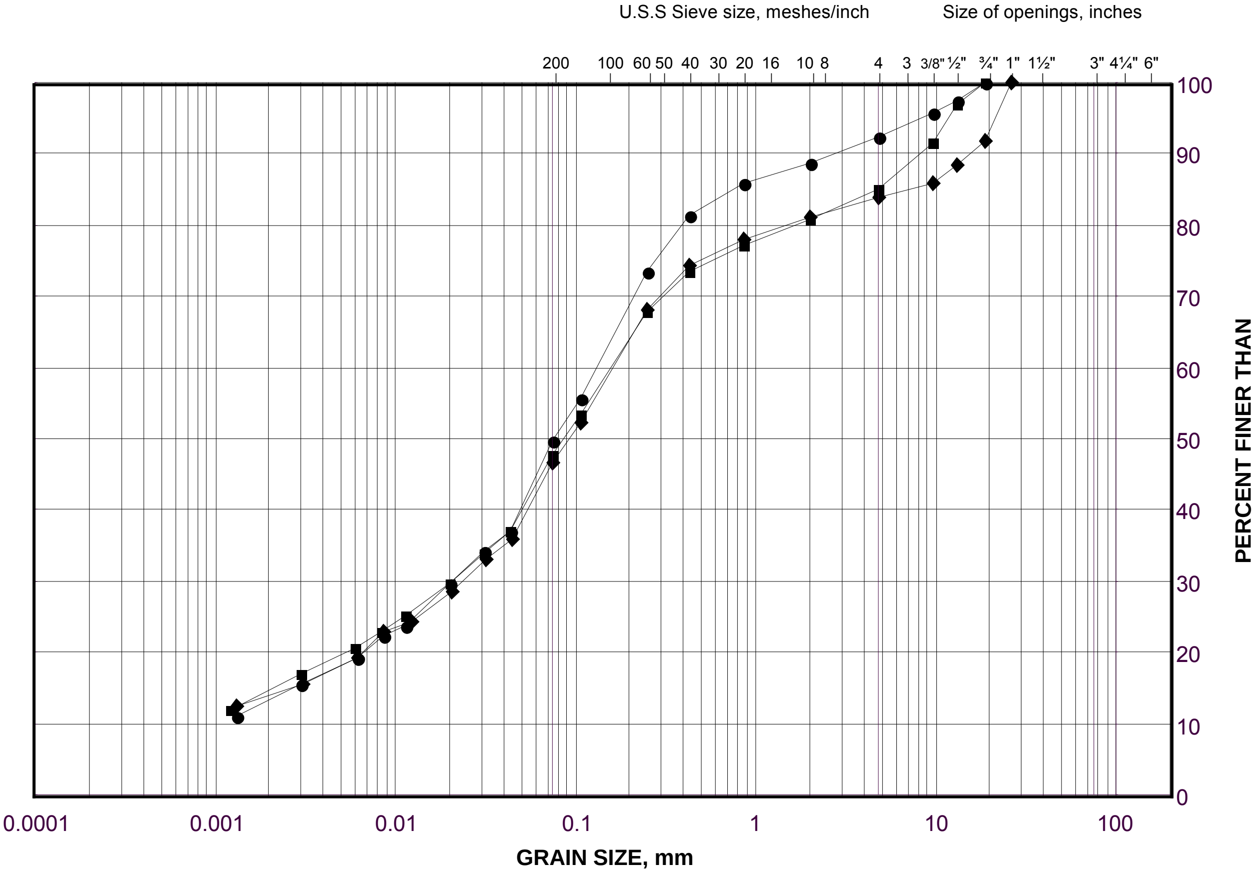
Figure No. B2

Project No. 09-1111-0019

Checked By: KJB

GRAIN SIZE DISTRIBUTION
Sand and Silt (FILL)

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)
●	BH-45	3	112.1
■	BH-45	4	111.3
◆	BH-46	5	110.9

Project Number: 09-1111-0019

Checked By: KJB

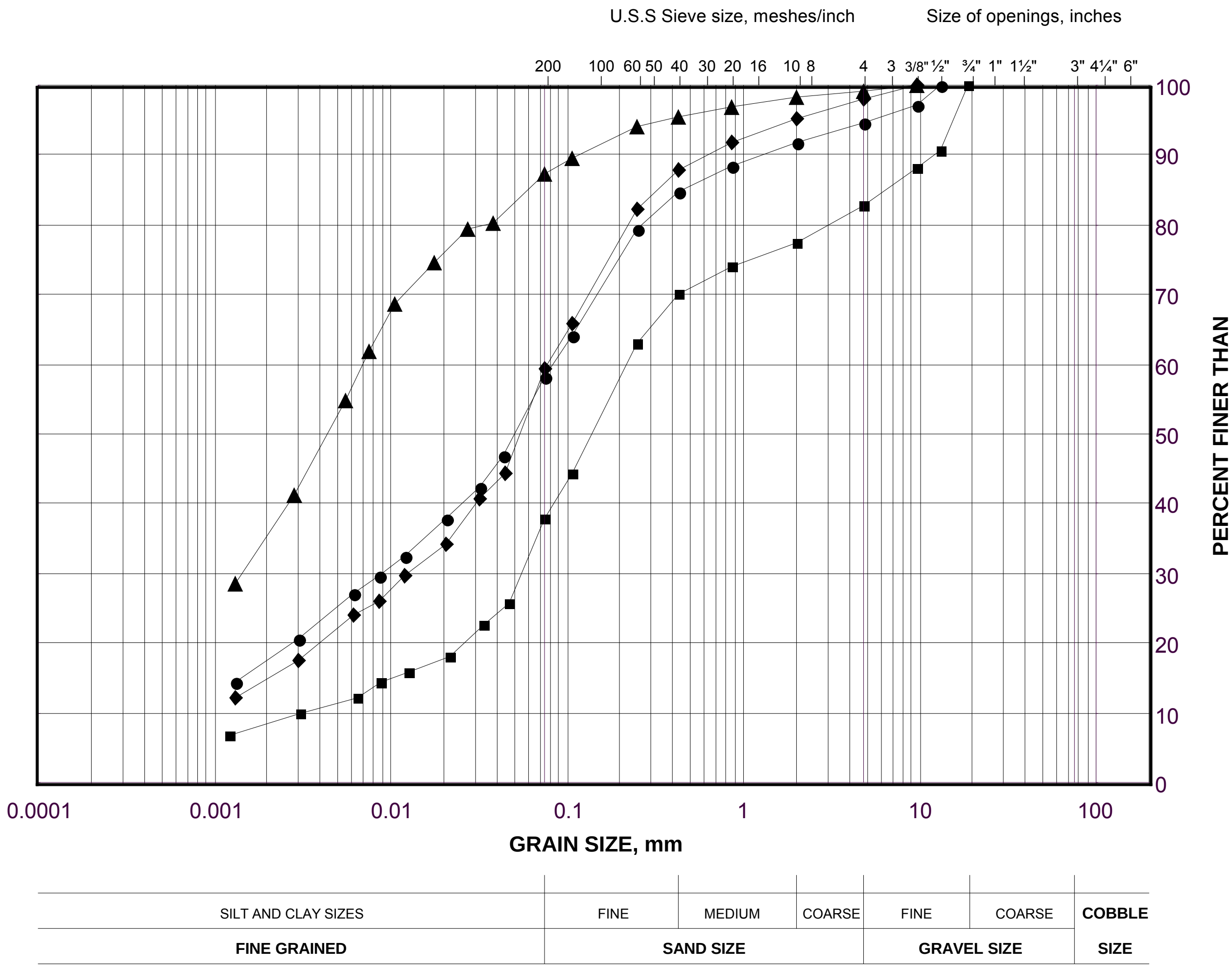
Golder Associates

Date: 01-Oct-13

GRAIN SIZE DISTRIBUTION

Clayey Silt (TILL)

FIGURE B4A



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-50	2	109.8
■	13-49	2	109.9
◆	13-50	3	109.3
▲	HR-2	4	109.1

Project Number: 09-1111-0019

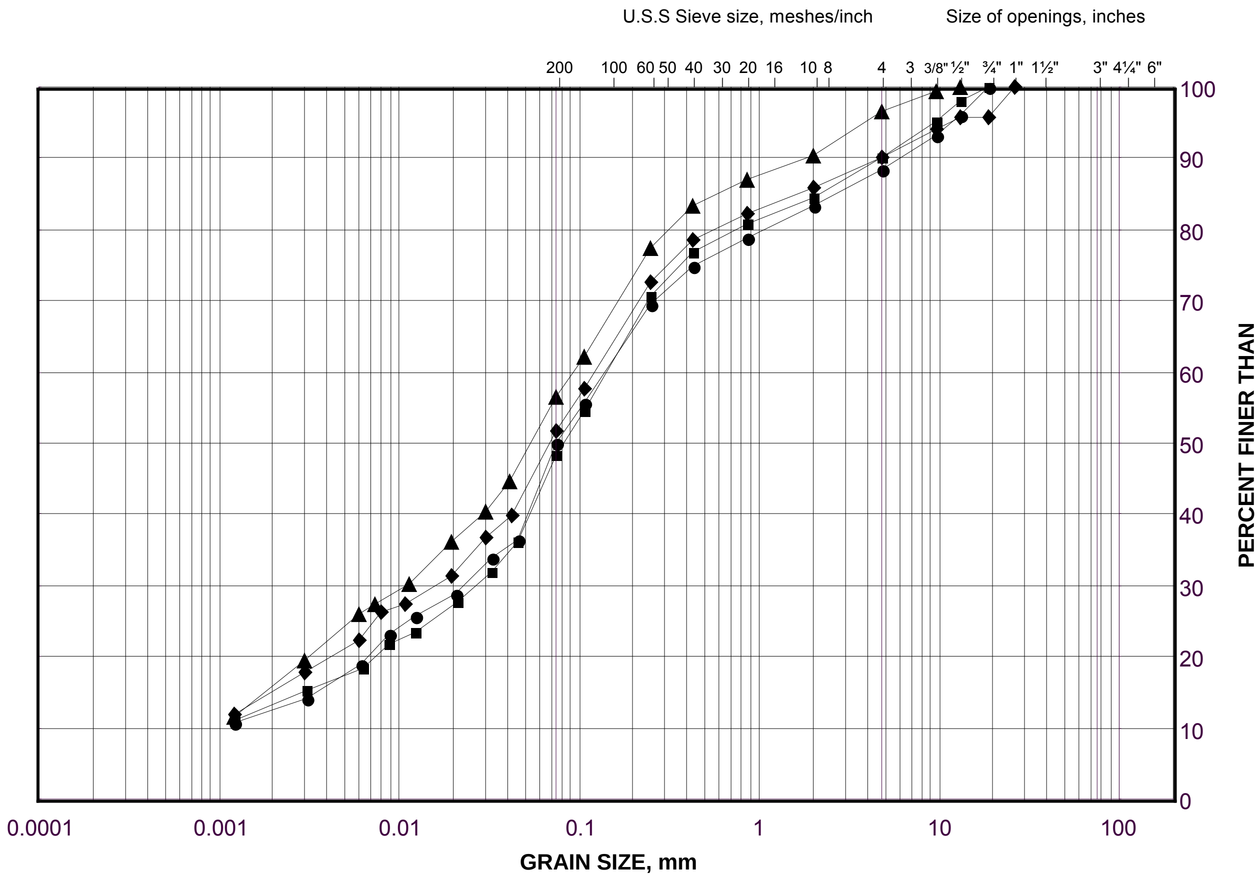
Checked By: MWK

Golder Associates

Date: 12-Dec-13

GRAIN SIZE DISTRIBUTION
Clayey Silt (TILL)

FIGURE B4B



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

LEGEND

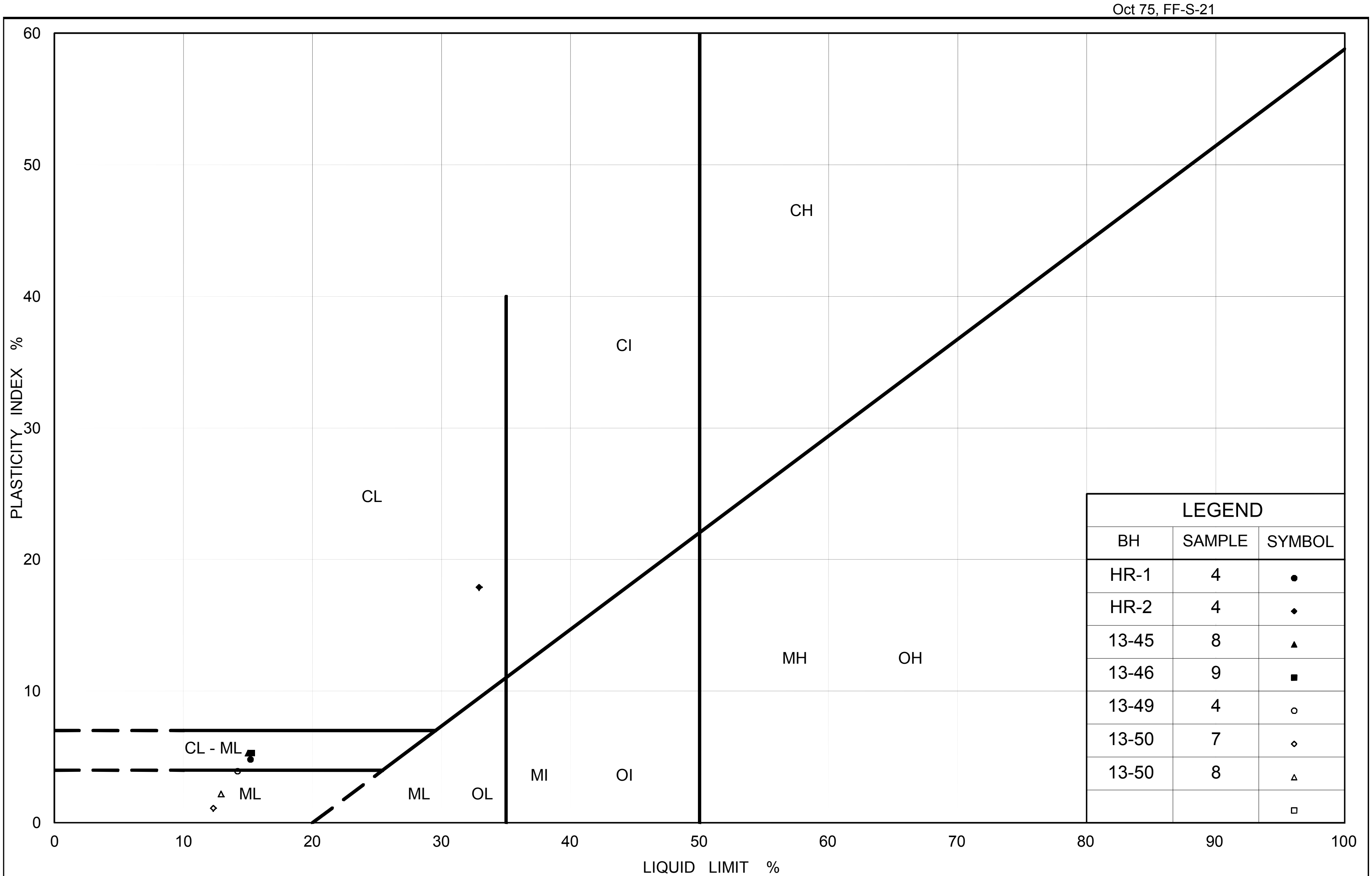
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	HR-1	4	109.1
■	13-50	5	107.9
◆	13-46	7	109.3
▲	13-45	7	109.2

Project Number: 09-1111-0019

Checked By: MWK

Golder Associates

Date: 12-Dec-13



Ministry of Transportation

Ontario

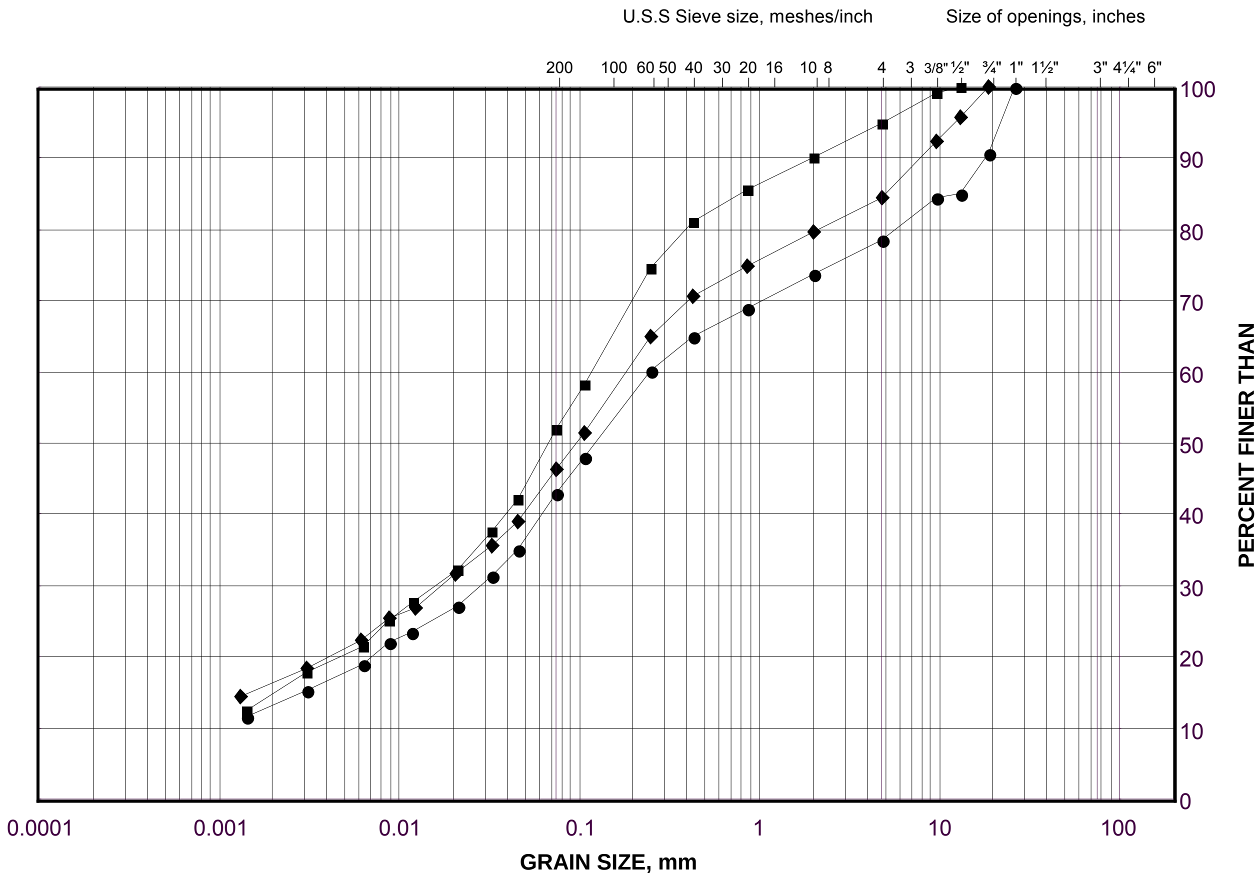
PLASTICITY CHART
Clayey Silt (TILL)

Figure No. B5

Project No. 09-1111-0019

Checked By: MWK

GRAIN SIZE DISTRIBUTION Sand and Silt (TILL)	FIGURE B6A



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-48	4	109.2
■	13-48	6	107.9
◆	13-47	6	108.4

Project Number: 09-1111-0019

Checked By: MWK

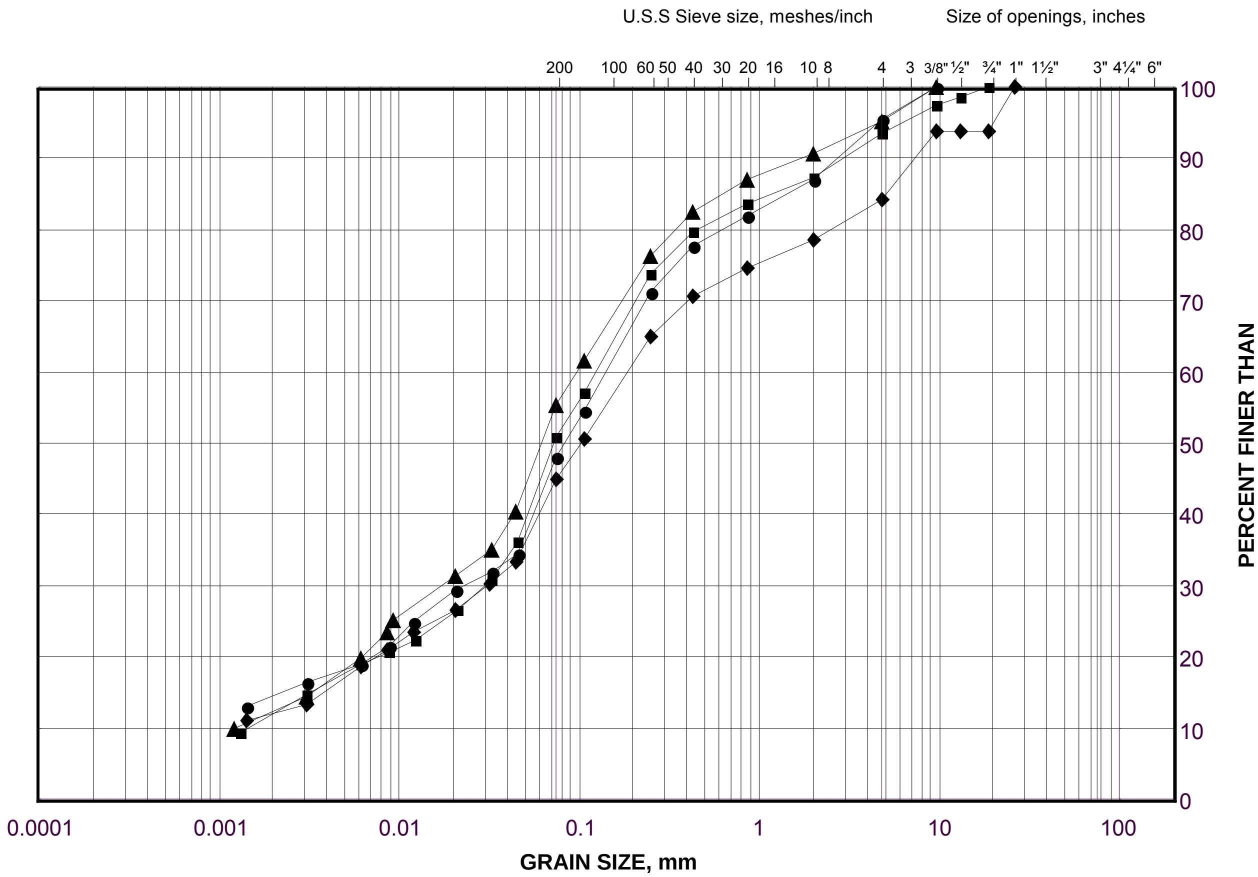
Golder Associates

Date: 12-Dec-13

GRAIN SIZE DISTRIBUTION

Sand and Silt (TILL)

FIGURE B6B



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	HR-2	5	108.6
■	13-49	6	107.0
◆	HR-1	6	107.8
▲	HR-1	8	105.5

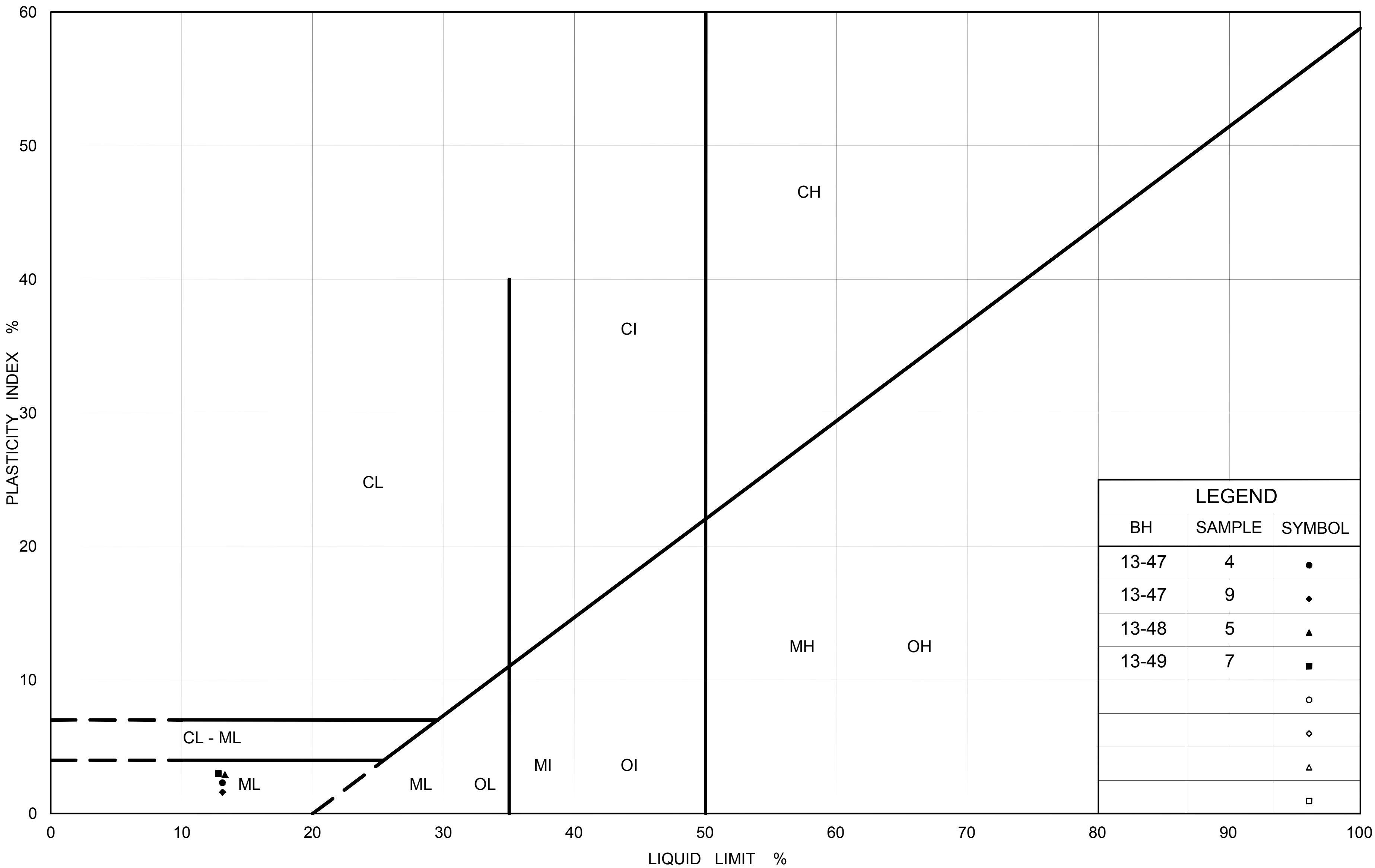
Project Number: 09-1111-0019

Checked By: MWK

Golder Associates

Date: 12-Dec-13

Oct 75, FF-S-21



LEGEND

BH	SAMPLE	SYMBOL
13-47	4	●
13-47	9	◆
13-48	5	▲
13-49	7	■
		○
		◇
		△
		□

Ministry of Transportation

Ontario

PLASTICITY CHART
Sand and Silt (TILL)

Figure No. B7

Project No. 09-1111-0019

Checked By: MWK

APPENDIX C

Borehole Logs – Previous Investigation

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS AND RESEARCH SECTION

W.P. 118-58 BORE HOLE NO. 1

JOB 61-F-15 STATION See drawing

DATUM 370.0' COMPILED BY B.K.

BORING DATE Mar. 2/61 CHECKED BY V.K.

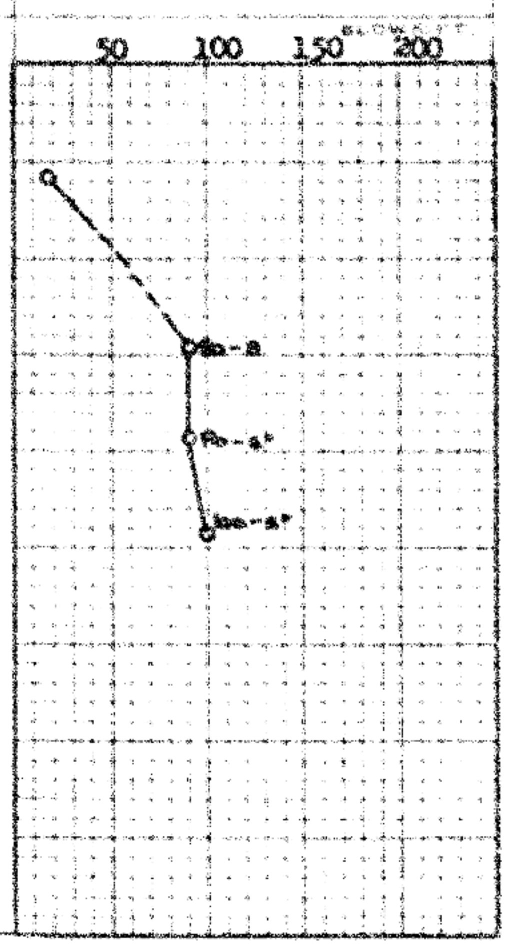
2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
2" SHELBY
CASING

LEGEND

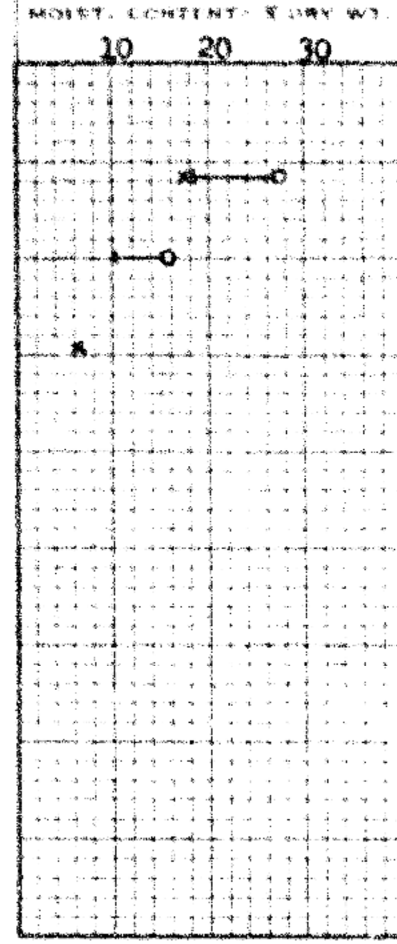
1/2 UNCONFINED COMPRESSION (Qu) O
VANE TEST (C) AND SENSITIVITY (S) +
NATURAL MOISTURE AND LIQUIDITY INDEX
LIQUID LIMIT
PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	SAMPLE	NATURAL UNIT WT. P.C.F.
	Groundlevel	370.0	0		
	Granular Fill				
		365.0			
	Dark brown clayey silt (Topsoil)	363.0		S1	-
	Light brown pebbly silty sand (till)		10	T2 S3	=
				S4	145.8
		351.0	20	S5	-
	Grey pebbly silty sand (till)			S6	-
		345.5			
	End of borehole		30		
			40		

STRENGTH AND PENETRATION RESISTANCE P.S.F.



CONSISTENCY



DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS AND RESEARCH SECTION

W.P. 118-58
BORE HOLE NO. 2
JOB 61-F-15
STATION See Drawing
DATUM 369.0'
COMPILED BY B.K.
BORING DATE Mar. 2/61
CHECKED BY V.K.

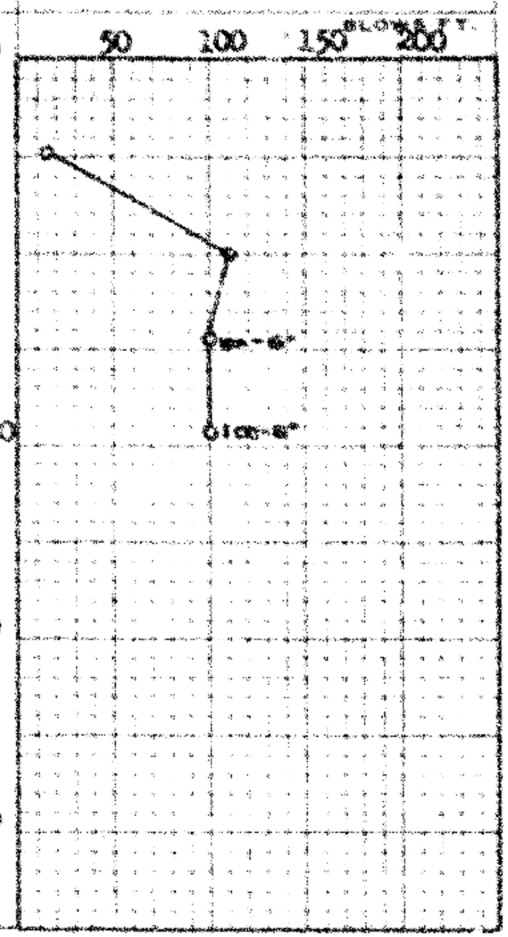
2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
2" SHELBY
CASING

LEGEND

1/2 UNCONFINED COMPRESSION (Qu) C
VANE TEST (C) AND SENSITIVITY (S) +S
NATURAL MOISTURE AND LIQUIDITY INDEX X
LIQUID LIMIT
PLASTIC LIMIT

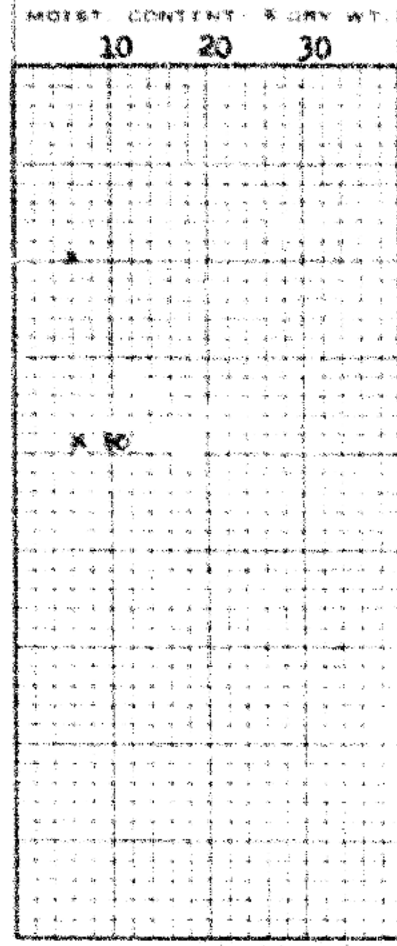
SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
	Groundlevel	369.0	0
	Granular Fill		
		365.0	
	Dark brown clayey silt (topsoil)	362.0	
	Light brown pebbly silty sand (Till)		10
		351.0	
	Grey pebbly silty sand (till)		20
		346.0	
	End of borehole.		30
			40

STRENGTH AND PENETRATION RESISTANCE P.S.F.



CONSISTENCY

MOIST. CONTENT % DRY WT.



SAMPLE	NATURAL UNIT WT. P.C.F.
S1	-
S2	-
S3	-
S4	-

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS AND RESEARCH SECTION

W.P. 118-58
JOB 61-F-15
DATUM 367.0'
BORING DATE Mar. 2/61

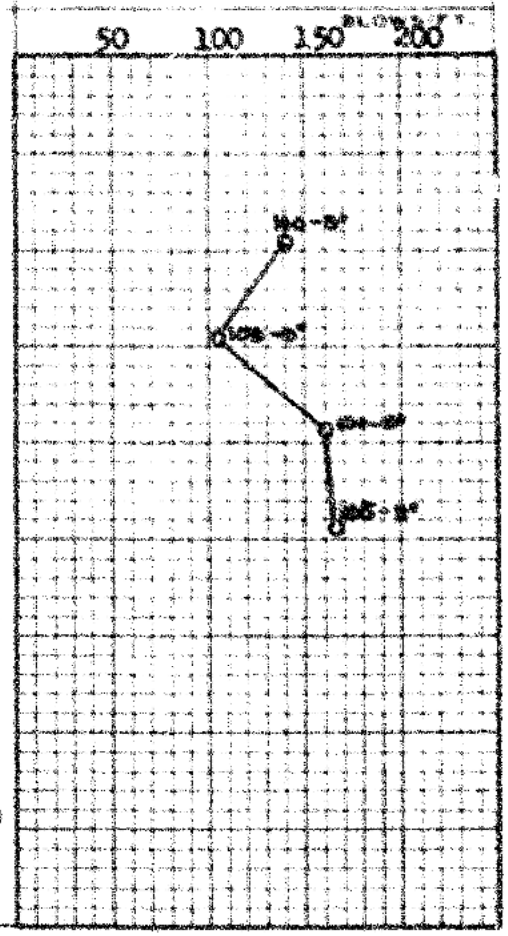
BORE HOLE NO. 3
STATION See Drawing
COMPILED BY B.K.
CHECKED BY V.K.

LEGEND

2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
2" SHELBY
CASING

1/2 UNCONFINED COMPRESSION (Qu) O
VANE TEST (C) AND SENSITIVITY (S) +S
NATURAL MOISTURE AND LIQUIDITY INDEX
LIQUID LIMIT
PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE P.S.F.	CONSISTENCY MOIST. CONTENT - % DRY WT.	SAMPLE	NATURAL UNIT WT. P.C.F.
	⊥ Groundlevel	367.0	0				
	Granular Fill	366.0					
	Dark brown silty clay (topsoil)					S1	-
		361.5					
	Light brown pebbly silty sand (till)		10			S2	-
						S3	-
		351.0					
	Grey pebbly silty sand till		20			S4	-
						S5	-
		342.0					
	End of borehole		30				
			40				



DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS AND RESEARCH SECTION

W.P. 118-58
BORE HOLE NO. 4
JOB 61-F-15
STATION See Drawing
DATUM 367.0'
COMPILED BY B.K.
BORING DATE Mar. 2/61
CHECKED BY V.K.

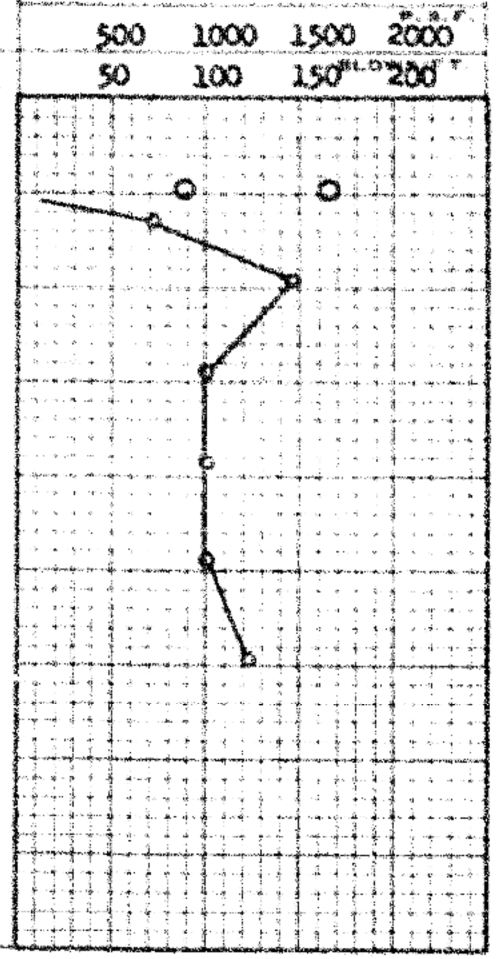
2" DIA SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA CONE
2" SHELBY
CASING

LEGEND

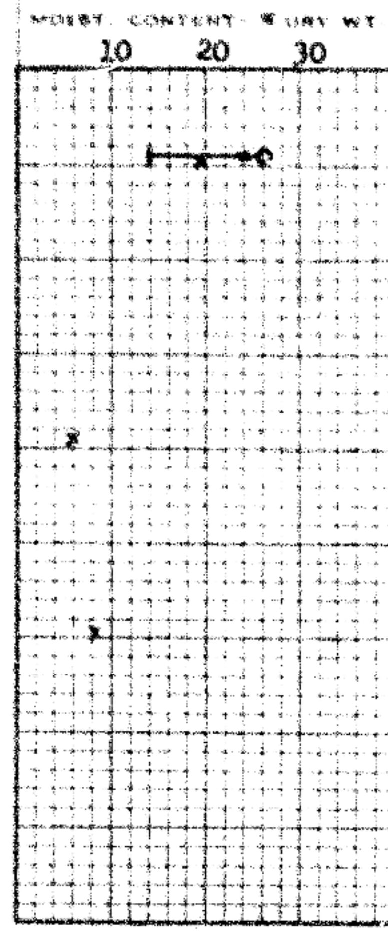
1/2 UNCONFINED COMPRESSION (Qu)
VANE TEST (C) AND SENSITIVITY (S)
NATURAL MOISTURE AND LIQUIDITY INDEX
LIQUID LIMIT
PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
	Groundlevel	367.0	0
	Granular fill	366.0	
	Dark brown clayey silt (topsoil)	362.0	
	Brown gravelly silty sand	351.0	10
	Grey gravelly silty sand till	337.0	20, 30
	End of borehole		40

STRENGTH AND PENETRATION RESISTANCE



CONSISTENCY



SAMPLE	NATURAL UNIT WT. P.C.F.
T1	123.0 131.0
S2	-
S3	-
S4	-
S5	-
S6	-
S7	-

FORM OB-MC-126 (REV. 1959) - 58-5491

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS AND RESEARCH SECTION

W.P. 118-58 BORE HOLE NO. 5

JOB 61-F-15 STATION See Drawing

DATUM 369.0' COMPILED BY B.K.

BORING DATE Mar. 3/61 CHECKED BY V.K.

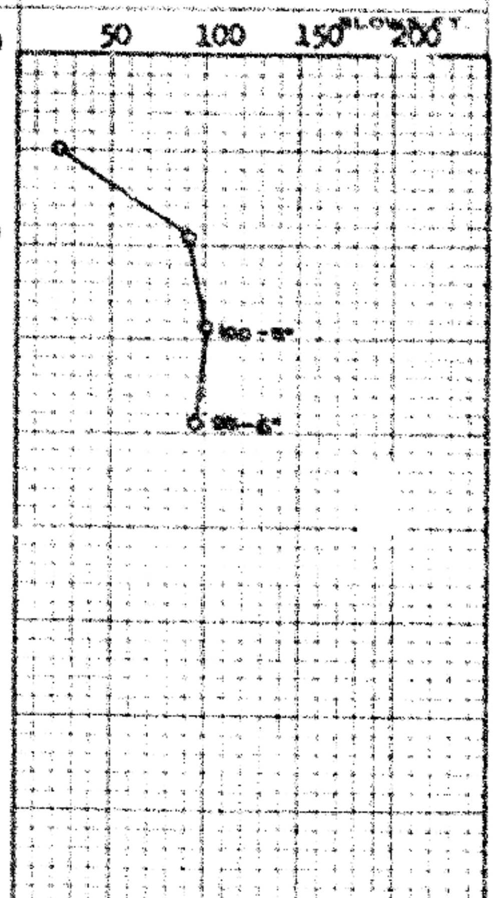
2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
2" SHELBY
CASING

LEGEND

1/2 UNCONFINED COMPRESSION (Qu)
VANE TEST (C) AND SENSITIVITY (S)
NATURAL MOISTURE AND LIQUIDITY INDEX
LIQUID LIMIT
PLASTIC LIMIT

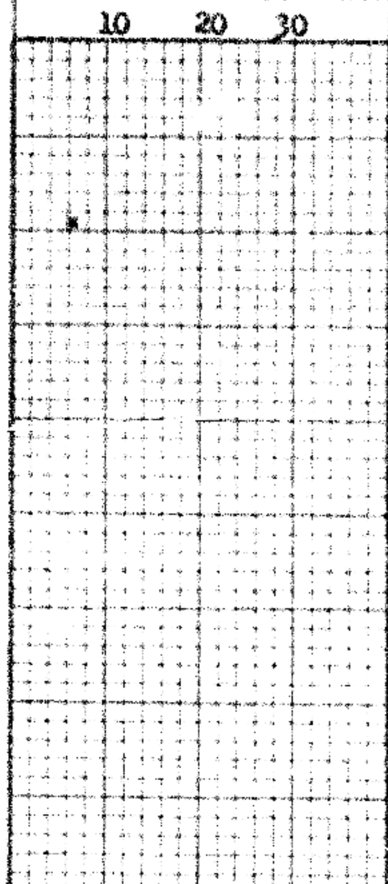
SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	SAMPLE	NATURAL UNIT WT. P.C.F.
	Groundlevel	369.0	0		
	Granular Fill				
		364.5			
	Dark brown clayey silt (topsoil)	362.5		S1	-
	Light brown pebbly silty sand (till)		10	S2	-
				S3	-
		349.5	20	S4	-
	End of borehole		30		
			40		

STRENGTH AND PENETRATION RESISTANCE P.S.F.



CONSISTENCY

MOIST. CONTENT % DRY WT.



DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS AND RESEARCH SECTION

W.P. 118-58
JOB 61-F-15
DATUM 366.0'
BORING DATE Mar. 3/61

BORE HOLE NO. 6
STATION See Drawing
COMPILED BY B.K.
CHECKED BY V.K.

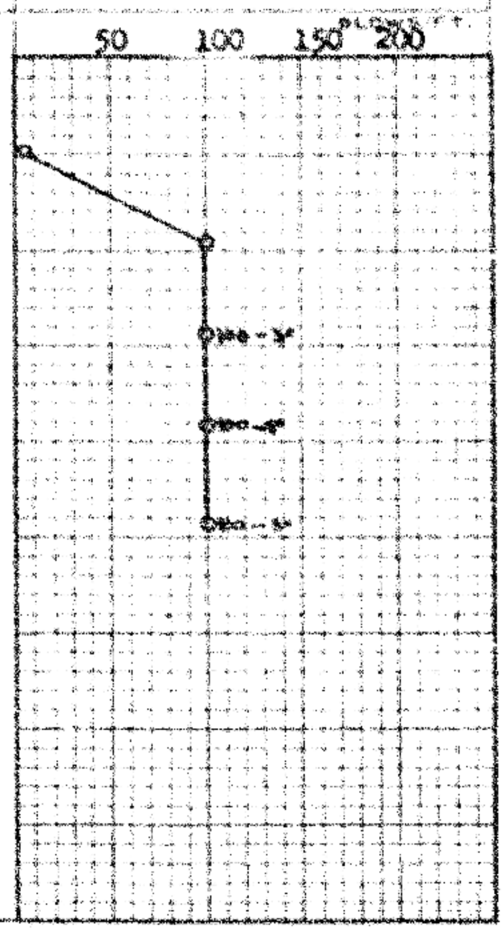
2" DIA SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA CONE
2" SHELBY
CASING

LEGEND

UNCONFINED COMPRESSION (Qu)
VANE TEST (C) AND SENSITIVITY (S)
NATURAL MOISTURE AND LIQUIDITY INDEX
LIQUID LIMIT
PLASTIC LIMIT

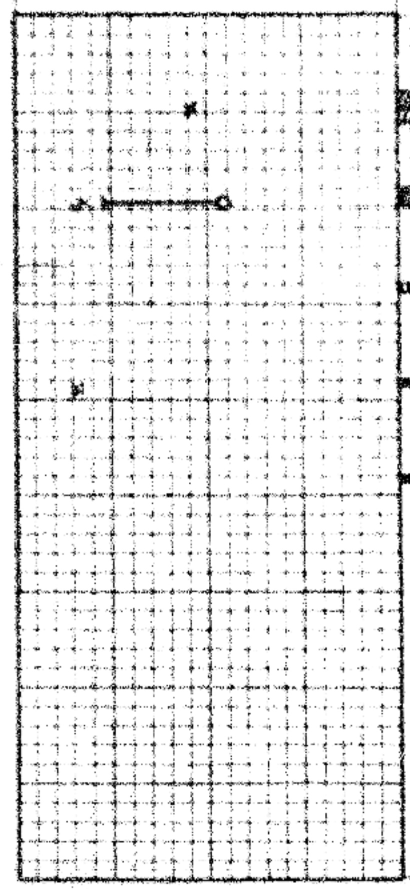
SYMBOL	DESCRIPTION	ELEV FEET	DEPTH FEET	SAMPLE	NATURAL UNIT WT. P.C.F.
	Groundlevel	366.0	0		
	Granular fill	363.5			
	Dark brown clayey silt (topsoil)	361.0		S1	-
	Light brown pebbly silty sand (till)		10	S2	-
		353.0		S3	-
	Grey pebbly silty sand (till)		20	S4	-
		341.7		S5	-
	End of borehole		30		
			40		

STRENGTH AND PENETRATION RESISTANCE P.S.F.



CONSISTENCY

MOIST. CONTENT % DRY WT.



DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS AND RESEARCH SECTION

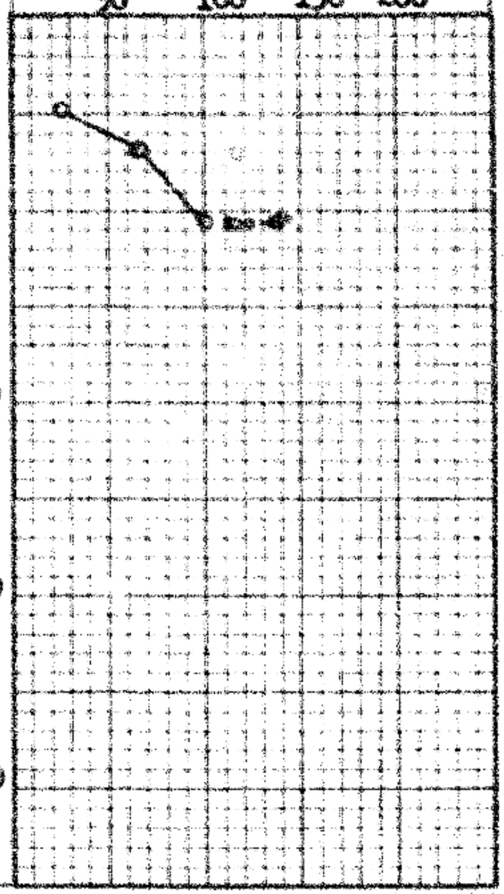
W.P. 118-58	BORE HOLE NO. 7
JOB 61-F-15	STATION See Drawing
DATUM 365.5'	COMPILED BY B.K.
BORING DATE Mar. 3/61	CHECKED BY V.K.

LEGEND

2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
2" SHELBY
CASING

1/2 UNCONFINED COMPRESSION (Qu) — O
VANE TEST (C) AND SENSITIVITY (S) — +S
NATURAL MOISTURE AND LIQUIDITY INDEX — LI X
LIQUID LIMIT
PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE P.S.F. (BLOWS/FT. 50 100 150 200)	CONSISTENCY MOIST. CONTENT - % DRY WT.	SAMPLE	NATURAL UNIT WT. P.C.F.
	Groundlevel	365.5	0				
	Granular fill	364.5					
	Dark brown clayey silt (topsoil)	363.5					
	Light brown pebbly silty sand (till)					S1	-
						S2	-
		355.0	10			S3	-
	End of borehole						
			20				
			30				
			40				



APPENDIX D

Non-Standard Special Provisions

OBSTRUCTIONS / PRE-AUGERING – Item No.

Non-Standard Special Provision

The very dense/hard sand and silt till and clayey silt till deposits are inferred to contain cobbles and boulders. Consideration of the very dense/hard relative density/consistency of the till deposits and the potential presence of these obstructions must be made in the selection of appropriate equipment and procedures for sub-excavation, caisson drilling and pre-drilling and pile driving for steel H-pile or pipe pile foundations. Pre-augering is anticipated to be required to install the piles to the required tip elevations in the west half of the north abutment.

For deep foundations comprised of steel H-Piles, pre-augering for the piles located within the west half of the north abutment should be carried out to at least Elevation 108 m. The diameter of the auger hole should be slightly larger than the minimum width/diameter of the pile to create an open hole for driving the steel piles below the socket design Elevation. Pre-augering for the piles within the east half of the north abutment and at the south abutment is not anticipated to be required

Following pre-augering, the steel piles should be inserted into the pre-augered hole and driven to the design capacity provided on the Contract Drawings. After driving is complete, the annulus around the pile should be backfilled with concrete (minimum compressive strength of 20 MPa) for the lower 1 m length of the pre-augered hole and backfilled with OPSS. Prov1010 Granular B Type II to the bottom of the CSP liner.

BASIS OF PAYMENT

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION

CSP FOR INTEGRAL ABUTMENTS – Item No

Special Provision

SCOPE

This specification covers the requirements for the installation of the corrugated steel pipes (CSPs) at the integral abutments.

SUBMISSION AND DESIGN REQUIREMENTS

All submissions shall bear the seal and signature of an Engineer.

At least two weeks prior to commencement of installation of the abutment piles, the Contractor shall submit to the Contract Administrator, for information purposes only, three (3) sets of the working drawings.

The Contractor shall have a copy of the submitted working drawings on site at all times. Working drawings shall include at least the following:

1. Layout and elevations of the CSPs;
2. Location of reference points, and location of the centroid of each pile with respect to the reference points;
3. Construction sequence and details;
4. Source of the sand fill, and description of placing methods and equipment;
5. Location and details of all temporary bracing and spacers for the piles and CSPs;
6. Method for preventing water and debris from entering the CSP prior to placing sand; and
7. Method for preventing concrete from abutment pours from entering the CSPs during placement.

The Contractor shall be responsible for the complete detailed design of all temporary bracing, including spacers required to maintain the piles, CSP spacing and abutment stems in their specified positions through all stages of construction until the CSPs have been backfilled. All temporary bracing shall be removed.

MATERIAL

Corrugated Steel Pipe

CSP shall be in accordance with OPSS 1801, and shall be from a supplier listed under DSM#4.60.80. The CSP shall be of the diameter and wall thickness specified on the Contract drawings, and shall be galvanized in accordance with CSA G164-M.

CSPs shall be supplied in the lengths and with the end treatments, either square or skew, as specified on the Contract drawings; field cutting and splicing of CSPs will not be permitted. Cut ends shall be neat and free of burrs. The planes defined by the end treatments of each CSP shall be parallel to each other.

Handling and storage of CSPs shall be in accordance with the manufacturer's recommendations. Damaged CSPs shall be rejected. Localized areas of damaged galvanizing on otherwise acceptable CSPs shall be repaired with two coats of zinc-rich paint.

Sand Fill

The sand fill for backfilling the CSP shall meet the gradation requirements of Table 1 below:

Table 1 – Sand Fill Gradation Requirements

MTO Sieve Designation		Percentage Passing by Mass
2 mm	#10	100%
600 mm	#30	80% to 100%
425 mm	#40	40% to 80%
250 mm	#60	5% to 25%
150 mm	#100	0% to 6%

CONSTRUCTION

The sequence of construction shall be in accordance with the working drawings and as follows, unless otherwise approved:

1. Construct levelling pad and place CSPs and spacers.
2. Install piles by driving to design criteria.
3. Place loose sand into CSP.
4. Remove temporary spacers.

The CSP shall be positioned such that the piles are centrally positioned within the CSP. Temporary blocking and bracing shall be used to hold the CSP in position.

The Contractor shall ensure the full perimeters of the tops of all CSPs at each abutment are at the elevation and orientation shown on the working drawings.
The CSP at each pile shall be constructed to the following tolerances:

Criteria	Tolerance
Maximum deviation of CSP from pile centroid	+/- 50 mm
Maximum deviation of any point on the top perimeter of the CSP from the specified elevation	+/- 10 mm

The sand fill shall be placed dry of optimum and free-flowing, completely filling the volume between the CSP and pile. No additional compaction effort other than the action of placing the sand itself shall be applied to the sand fill.

The placing of the sand fill shall be carried out in a manner such as to not damage and displace the CSP.

BASIS OF PAYMENT

Payment at the contract price for the above tender item shall include all labour, equipment and material required to do the work.

END OF SECTION

CONCRETE WORKING SLAB – Item No.

Special Provision

The subgrade for the Highway 401-Holt Road structure foundations will be susceptible to disturbance and softening/loosening from construction traffic and ponded water. Within four hours following inspection and approval of the prepared subgrade, a concrete working slab with a minimum thickness of 100 mm shall be placed on the foundation subgrade.

The concrete shall have a compressive strength of at least 20 MPa, and be placed in accordance with OPSS 904.

BASIS OF PAYMENT

Payment at the contract price for the above tender item shall include full compensation for all labour and materials to complete the work.

END OF SECTION

At Golder Associates we strive to be the most respected global group of companies specializing in ground engineering and environmental services. Employee owned since our formation in 1960, we have created a unique culture with pride in ownership, resulting in long-term organizational stability. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees now operating from offices located throughout Africa, Asia, Australasia, Europe, North America and South America.

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

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