



September 30, 2015

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

4TH LINE UNDERPASS, SITE NO. 30-212
HIGHWAY 400 WIDENING
FROM 1 KM SOUTH OF HIGHWAY 89 TO JUNCTION OF HIGHWAY 11
MINISTRY OF TRANSPORTATION, ONTARIO
W.O. 06-20016

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REPORT



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PART A

**PRELIMINARY FOUNDATION INVESTIGATION REPORT
4th LINE UNDERPASS – SITE NO. 30-212
HIGHWAY 400 WIDENING
FROM 1 KM SOUTH OF HIGHWAY 89 TO JUNCTION OF HIGHWAY 11
MINISTRY OF TRANSPORTATION, ONTARIO
W.O. 06-20016**



1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by AECOM (formerly URS Canada Inc.) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services in support of the preliminary design for the replacement of the 4th Line (Churchill Sideroad) Underpass in the Town of Innisfil. The proposed work is part of the preliminary and design-build ready design associated with the Highway 400 widening from 1 km south of Highway 89 to the junction of Highway 11 in Simcoe County, Ontario.

The terms of reference and scope of work for the foundation investigation are outlined in MTO's Request for Proposal, dated July 2013. Golder's scope of work for foundation engineering services associated with the 4th Line Underpass replacement is contained in Section 6.8 of AECOM's (previously URS Canada) Technical Proposal for this assignment. The work has been carried out in accordance with Golder's Supplementary Specialty Plan for foundation engineering services for this project, dated January 20, 2014.

This report addresses the proposed replacement of the 4th Line Underpass and the associated approach embankments (MTO Structure Site No. 30-212). The work for this structure site is being administrated as a design-build assignment. The Preliminary Foundation Investigation Report is intended for planning and preliminary purposes only and the Design-Builder shall satisfy himself as to the sufficiency of the subsurface information and supplement the information as needed to meet the requirements for detail design.

2.0 SITE DESCRIPTION

The 4th Line Underpass is located approximately 5.4 km south of the Innisfil Beach Road (Simcoe Road 21) interchange and approximately 4.2 km north of Highway 89 interchange, in the Town of Innisfil, in the County of Simcoe. The existing 4th Line Underpass is a single-span structure supported on spread footings.

The overall surface topography in the vicinity of the site is relatively flat and consists predominantly of rural farmland. The natural ground surface at the site ranges between approximately Elevations 284 m and 285 m. At this structure site, Highway 400 has been constructed near the original ground surface, with its grade at between approximately Elevations 286 m and 286.5 m, while 4th Line has been constructed on embankments up to about 8 m and 7.5 m high on the west and east sides of Highway 400, respectively, with the existing grade along 4th Line varying between approximately Elevations 291 m and 292.4 m.

3.0 INVESTIGATION PROCEDURES

3.1 Previous Borehole Investigation

Two boreholes were advanced at this site as part of a previous geotechnical investigation for the replacement of the existing 4th Line Underpass structure, associated with the widening of Highway 400. The investigation was completed by Golder in October 2000. Borehole B2-1 was advanced on the east side of Highway 400 to a depth of about 11 m, and Borehole B2-2 was advanced on the west side of Highway 400 to a depth of about 9.5 m, approximately the locations shown on Drawing 1.



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Both boreholes were advanced using solid stem augers and soil samples were obtained at intervals of depth of about 0.75 m and 1.5 m, using a 50 mm outer diameter split-spoon sampler driven by a manual hammer in accordance with the Standard Penetration Test (SPT) procedure.¹

The water level in the open boreholes was observed during and following the drilling operations and a piezometer was installed in Borehole B2-1 to allow monitoring of the groundwater level at the site.

3.2 Current Borehole Investigation

The field work at the site of the 4th Line Underpass was carried out between June 3 and 11, 2015, during which time a total of four boreholes were advanced to supplement the existing subsurface information. The Record of Borehole sheets are presented in Appendix A. The locations of these boreholes are shown in plan on Drawing 1.

The borehole investigation was carried out using a Diedrich D-90 truck-mounted drill rig and an Acker Renegade track-mounted drill rig supplied and operated by Walker Drilling Ltd. of Utopia, Ontario. The boreholes were advanced through the overburden using 210 mm outside diameter hollow stem augers. Soil samples were generally obtained at intervals of depth of about 0.75 m and 1.5 m, using a 50 mm outside diameter split-spoon sampler driven by an automatic hammer in accordance with the Standard Penetration Test (SPT) procedure. The boreholes were generally advanced at least 3 m into a “refusal” stratum, defined as a material for which the SPT ‘N’-values exceed 100 blows per 0.3 m of penetration. Borehole BH1 encountered auger refusal at a depth of 11.6 m within a loose clayey sand deposit and as such, an additional borehole, Borehole BH1B, was advanced about 2 m east of Borehole BH1 to confirm the stratigraphy and “refusal” conditions at the west abutment.

The groundwater conditions and water level in the open boreholes were observed during and immediately following the completion of drilling operations. A piezometer was installed in each of Boreholes BH1B and BH3 to allow monitoring of the groundwater level at this site. The piezometers consist of a 50 mm diameter PVC pipe, with a slotted screen sealed within the silt and sand fill material in Borehole BH1B and within the sand deposit and lower silty sand till deposit in Borehole BH3. The borehole and annulus surrounding the piezometer pipe above the screen and sand pack were backfilled with bentonite pellets to near the ground surface and the road pavement was reinstated using dry mix concrete and cold asphalt patch. The piezometer installation details and water level readings are noted on the Record of Borehole BH1B and BH3 in Appendix A. All other boreholes were backfilled upon completion of drilling in accordance with Ontario Regulation 903 (as amended), and the pavement was reinstated using dry mix concrete and cold asphalt patch.

The field work was observed by members of Golder’s engineering staff who located the boreholes, arranged for the clearance of underground services, observed the drilling, sampling and in situ testing operations, logged the boreholes and examined and cared for the soil samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to Golder’s Mississauga geotechnical laboratory where the samples underwent further visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Classification testing (water content, grain size distribution and Atterberg limits) was carried out on selected soil samples. The results of the laboratory testing are included in Appendix B.

¹ ASTM D1586 – Standard Test Method for Standard Penetration Test and Split Barrel Sampling of Soils.



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The as-drilled borehole locations were measured relative to the existing on-site features shown on the Digital Terrain Model (DTM) for the site, and the ground surface elevations were interpolated from the topographic data provided by AECOM. The borehole locations provided on the borehole records and shown on Drawing 1 are given using MTM NAD83 northing and easting coordinates, and the ground surface elevations are referenced to Geodetic datum. The borehole locations, ground surface elevations and drilled depths are summarized below.

Borehole Number	Location (MTM NAD83)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m)	Easting (m)		
BH1	4,899,650.6	291,433.3	291.8	11.6
BH1B	4,899,650.9	291,434.3	291.8	18.9
BH2	4,899,657.5	291,457.6	292.1	20.4
BH3	4,899,675.5	291,512.7	292.0	18.7

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

As delineated in *The Physiography of Southern Ontario*², this section of Highway 400 from 6 km south of Highway 89 to the junction of Highway 11 traverses, generally in a south–north direction, the following physiographic regions: the Peterborough Drumlin Field; the Simcoe Lowlands; and the Simcoe Highlands. Along Highway 400, the Peterborough Drumlin Field is present from the southern limit of the project site to south of Line 13 of the Township of Bradford-West Gwillimbury. The Simcoe Lowlands covers the area from south of Line 13 to approximately 1 km north of Highway 89. The Simcoe Highlands extends from about 1 km north of Highway 89 to beyond the northern limit of this project site.

The surficial soils in the Peterborough Drumlin Field, which encompasses the 4th Line site, consist primarily of sand to sand and gravel deposits, as well as sand till deposits. Deposits of silt, clay or peat may be found in the low-lying areas between drumlins.

Along Highway 400, the Simcoe Lowlands include the Holland River valley, the shores of Kempenfelt Bay, the Nottawasaga River and Innisfil Creek. The Holland River valley at the southern end of this project extends southwest from Cook Bay at the south end of Lake Simcoe, and was once a shallow extension of the lake. The floor of the valley is covered by extensive deposits of loose silts and soft clays overlying a till sheet. In localized areas, these silts and clays are overlain by a thin, poorly graded sand of deltaic origin. Because the valley is depressed and poorly drained, a surficial cover of peat has formed in many areas.

The surficial soils of the northern lobe of the Simcoe Lowlands consist primarily of sand, although silt, clay or peat may be found in low-lying areas.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions encountered in the boreholes advanced as part of the current investigation, together with the results of in situ and laboratory testing, are presented on the Record of

² Chapman, L. J. and Putnam, D. F., 1984. *The Physiography of Southern Ontario*, Ontario Geological Survey. Special Volume 2, Third Edition. Accompanied by Map P.2715, Scale 1:600,000. Ontario Ministry of Natural Resources.



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Borehole sheets and laboratory test summary figures provided in Appendices A and B, respectively. The Record of Borehole sheets and laboratory testing results from the previous investigation are presented in Appendix C. The interpreted stratigraphic profile and cross-sections are shown on Drawings 1 to 3.

The results of the in situ field tests (i.e. SPT 'N'-values) carried out during the current investigation as presented on the Record of Borehole sheets and in Section 4.2 are uncorrected. According to the Canadian Foundation Engineering Manual (*CFEM*, 2006), the energy delivered to the drill rod varies with the hammer release system, hammer type, anvil and operator characteristics. It should be noted that different hammer release systems were used during the previous and current investigations (i.e. manual versus automatic) and as such SPT 'N'-values measured during the previous investigation may be higher than the 'N'-values measured during the current investigation within the same deposit.

The stratigraphic boundaries shown on the Record of Borehole sheets and on the interpreted stratigraphic profile and cross-sections are inferred from observations of drilling progress and 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 general, the subsurface conditions at the site consist of a layer of asphalt and non-cohesive fill material associated with the existing 4th Line pavement structure and approach embankments, underlain by a silty clay till deposit which in turn is underlain by a silty sand till deposit and/or a silt and sand to sand deposit. The native silt and sand to sand deposit is underlain by a glacial till deposit comprised of silty sand or clayey silt which extends to the refusal condition.

A detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Asphalt

An approximately 100 mm to 200 mm thick layer of asphalt was encountered in Boreholes BH1, BH2 and BH3 which were advanced along 4th Line embankment platform.

4.2.2 Fill

Fill material was encountered below the asphalt in all boreholes advanced during the current investigation and immediately at the ground surface in the two boreholes advanced during the previous investigation. The non-cohesive fill material is variable in composition and includes the following: sand and gravel, trace silt; sand, some gravel to gravelly, some silt to silty; silty sand, trace to some gravel and clay; and silt and sand, trace to some gravel and clay. It should be noted that portions of the fill material also contain asphalt fragments, organics and clay pockets/clayey silt lenses. A cobble, inferred from auger grinding at a depth of about 7.6 m below existing ground surface, was encountered in Borehole BH2.

For boreholes advanced along 4th Line (i.e. Boreholes BH1, BH2 and BH3), the top of the fill deposit ranges from about Elevation 291.9 m to 291.7 m and the thickness of the fill varies from about 7.6 m to 7.9 m. In Boreholes B2-1 and B2-2, advanced east and west of Highway 400, respectively, at the toe of the 4th Line approach embankments, the top of the fill was encountered at about Elevation 286.2 m and 285.9 m. The fill deposit is about 1.5 m and 2.1 m thick in the respective boreholes.



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The SPT 'N'-values measured within the fill deposit generally range from 0 blows (i.e. weight of hammer) to 29 blows per 0.3 m of penetration, indicating a very loose to compact relative density. Two SPT 'N'-values measured near the bottom of the fill deposit in Boreholes BH2 and BH3 are 49 blow and 59 blows per 0.3 m of penetration, respectively, indicating a dense and very dense relative density.

The natural water content measured on samples from the current investigation range from about 4 per cent to 14 per cent.

The grain size distributions of three samples of the fill material obtained during the current investigation are shown on Figure B1 in Appendix B.

Atterberg limits tests carried out on two samples of the silty sand fill material, recovered from Boreholes BH2 and BH3, measured liquid limits of about 16 per cent and plastic limits of about 10 per cent and 12 per cent, corresponding to plastic indices of about 6 per cent and 4 per cent. The results of the Atterberg limits test are shown on the plasticity chart on Figure B2 in Appendix B and indicate that fines portion of the fill material is slightly plastic.

4.2.3 Topsoil / Organic Silt

An approximately 0.1 m to 0.8 m thick layer of topsoil/organic silt containing trace sand and rootlets was encountered in all boreholes. The top of this layer ranges between Elevations 284.7 m and 283.8 m.

The natural water content measured on a sample of topsoil from Borehole B2-1 is about 30 per cent.

The organic content measured on the sample of topsoil is about 7.3 per cent by weight.

4.2.4 Sandy Clayey Silt

A 0.9 m thick layer of sandy clayey silt, some gravel was encountered below the organic silt in Borehole BH2 where the top of the deposit is at Elevation 284.1 m.

One SPT 'N'-value measured within the sandy clayey silt is 17 blows per 0.3 m of penetration, suggesting a very stiff consistency.

The natural water content measured on a sample of clayey sand is about 9 per cent.

4.2.5 Silty Clay Till

An upper deposit of till comprised of silty clay containing trace to some sand and trace gravel was encountered underlying the topsoil/organic silt in Boreholes BH1, B2-1 and B2-2 and underlying the sandy clayey silt in Borehole BH2. The top of the silty clay till deposit ranges from Elevation 283.9 m to 283.0 m and the thickness of the deposit varies between about 0.8 m and 3.6 m.

The SPT 'N'-values measured within the cohesive till deposit range from 11 blows to 23 blows per 0.3 m of penetration, suggesting a stiff to very stiff consistency.

The natural water content measured on samples of the cohesive till deposit ranges from about 22 per cent to 27 per cent.



The result of a grain size distribution test completed on a sample of the silty clay till sample from Borehole B2-2 completed during the previous investigation is shown on Figure 1 in Appendix C.

An Atterberg limits test carried out on a sample of the silty clay till deposit obtained during the current investigation measured a liquid limit of about 40 per cent, a plastic limit of about 16 per cent and a corresponding plastic index of about 24 per cent. The result of the Atterberg limits test is shown on the plasticity chart on Figure B3 in Appendix B and indicates that the material is classified as silty clay of intermediate plasticity. Atterberg limits test carried out on two samples of cohesive till deposit obtained during the previous investigation measured liquid limits of about 35 per cent and 39 per cent and plastic limits of about 15 per cent and 19 per cent, corresponding to plastic indices of about 20 per cent, also indicating that the cohesive till deposit is comprised of silty clay of intermediate plasticity.

4.2.6 Upper Silty Sand Till / Clayey Silt Till

A upper deposit of till comprised of silty sand, trace to some gravel and clay was encountered below the silty clay till deposit Boreholes BH1, BH1B and BH2, while a deposit of till comprised of clayey silt, trace to some sand and gravel to silty sand, trace clay and gravel was encountered below the silty clay till deposit in Borehole B2-2. Auger grinding was noted at a depth of about 13.1 m in Borehole BH2 during the drilling operations and auger refusal was encountered at a depth 11.6 m in Borehole BH1, suggesting the presence of cobbles and boulders within the upper till deposit. The top of the upper till deposit ranges from Elevation 281.6 m to 279.9 m and is approximately 2.8 m to 3.8 m thick.

The SPT 'N'-values measured within the non-cohesive portions of the upper till deposit generally range from 4 blows per 0.3 m of penetration to 100 blows per 0.23 m of penetration, indicating a loose to very dense relative density. The SPT 'N'-values measured within the cohesive portions of the upper till deposit range from 108 blows to 121 blows per 0.3 m of penetration, suggesting a hard consistency.

The natural water content measured on samples of the non-cohesive portions of the upper till deposit ranges from about 6 per cent to 9 per cent, while the natural water content measured on one sample of cohesive portions of the upper till deposit is about 8 per cent.

An Atterberg limits test carried out on a sample of the upper clayey silt till deposit obtained during the previous investigation measured a liquid limit of about 16 per cent, a plastic limit of about 9 per cent and a corresponding plastic index of about 7 per cent. The results of the Atterberg limits test indicate that the material is classified as a clayey silt of low plasticity.

4.2.7 Sand and Silt to Silty Sand to Sand

A deposit of non-cohesive soil comprised of silt and sand to silty sand containing trace gravel and clay, and sand containing some silt, trace gravel and clay, was encountered below the upper silty sand till in Boreholes BH1B and BH2, below the organic silt in Borehole BH3 and below the silty clay till in Borehole B2-1. The top of the deposit ranges from Elevation 284.0 m to 277.3 m and the thickness of the deposit varies between about 1.5 m and 3.7 m.



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The SPT 'N'-values measured within the non-cohesive deposit generally range from 23 blows to 64 blows per 0.3 m of penetration, indicating a compact to very dense relative density. A SPT 'N'-values as low as 9 blows per 0.3 m of penetration was measured in Boreholes B2-1, indicating a loose relative density.

The natural water content measured on samples of this deposit ranges from about 12 per cent and 17 per cent.

The grain size distributions of two samples of the sand deposit obtained during the current investigation are shown on Figure B4 in Appendix B. The results of a grain size distribution test completed on a sample of the silty sand to sand and silt deposit obtained during the previous investigation are shown on Figure 2 in Appendix C.

4.2.8 Lower Silty Sand Till / Clayey Silt Till

A lower deposit of till comprised of silty sand, trace gravel to gravelly, trace to some to clay to clayey silt, trace to some sand and gravel was encountered below the silty clay till in Borehole B2-2 and below the silt and sand to sand silt deposit in Boreholes BH1B, BH2, BH3 and B2-1. The top of the lower till deposit ranges from Elevation 280.3 m to 275.8 m. All boreholes, except for Borehole BH1, were terminated within the lower silty sand till / clayey silt till deposit after penetrating it for between 0.7 m and 7 m.

The SPT 'N'-values measured within the non-cohesive portions of the lower till deposit generally range from 40 blows per 0.3 m of penetration to 180 blows per 0.28 m of penetration, indicating a dense to very dense relative density. The SPT 'N'-values measured within the cohesive portions of the lower till deposit range from 108 blows per 0.15 m of penetration to 100 blows per 0.06 m of penetration, suggesting a hard consistency.

The natural water content measured on samples of the non-cohesive portions of the lower till deposit ranges from about 6 per cent to 9 per cent, while the natural water content measured on a sample of cohesive portions of the lower till deposit is about 7 per cent.

The grain size distributions of three samples of the non-cohesive lower till deposit obtained during the current investigation are shown on Figure B5 in Appendix B.

Atterberg limits tests carried out on two samples of the lower silty sand till deposit obtained during the current investigation and measured liquid limits of about 13 per cent and 14 per cent, plastic limits of about 9 per cent and 10 per cent and corresponding plastic indices of about 4 per cent. The results of the Atterberg limits test are shown on the plasticity chart on Figure B6 in Appendix B and indicate that fines portion of the lower till deposit are slightly plastic.

4.3 Groundwater Conditions

The water level encountered during drilling and observed in Boreholes BH1, BH2 and BH3 upon completion of drilling is between approximately Elevation 284.8 m and 278.0 m. However, the water level observed in the open boreholes during and/or on completion of drilling may not represent the longer-term, stabilized groundwater level at the site.

A standpipe piezometer was installed in each of Boreholes BH1B and BH3 on the west and east side of Highway 400, respectively, as part of the current investigation, and a standpipe piezometer was installed in Borehole B2-2 on the west side of the highway during the October 2000 investigation. The observed



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groundwater level in the standpipe piezometers is shown on the Record of Borehole sheets and summarized below:

Borehole	Depth to Water Level (m)	Groundwater Elevation (m)	Date of Measurement
B2-2	6.0	279.9	October 26, 2000
	1.9	284.0	January 19, 2001
	1.1	284.8	March 15, 2001
BH1B	6.3 ¹	285.5 ¹	July 6, 2015
BH3	7.2	284.8	June 11, 2015
	7.2	284.8	July 6, 2015

Note: 1. The water level indicator probe extended through an approximately 0.5 m thick layer of clayey slurry inside the standpipe piezometer before reaching the bottom of the screen at a depth of about 6.8 m below existing ground surface (Elev. 285.5 m).

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 and periods of precipitation.



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5.0 CLOSURE

Messrs. Qasim Cheema, B.A.Sc., and Jeremy Lebow, B.A.Sc., E.I.T., supervised the borehole investigation program. This report was prepared by Mr. Tomasz Zalucki, P.Eng., a geotechnical engineer, and was reviewed by Mr. Christopher Ng, P.Eng., a geotechnical engineer and Associate with Golder. Mr. Jorge M. A. Costa, P.Eng., a Principal with Golder and Designated MTO Foundations Contact, conducted an independent review of this report.

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PART B

**PRELIMINARY FOUNDATION DESIGN REPORT
4th LINE UNDERPASS – SITE NO. 30-212
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6.0 DISCUSSION AND PRELIMINARY ENGINEERING RECOMMENDATIONS

This section of the report provides preliminary foundation recommendations in support of the design-build ready design for the proposed replacement of the 4th Line Underpass at Highway 400 (MTO Structure Site No. 30-212). These preliminary recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current and previous subsurface investigations at this site. The interpretation and recommendations contained in this report are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the preliminary design of the structure foundations. Further investigation and design will be required during the design-build process.

Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project, and for which special provisions may be required during construction. Those requiring information on 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

Golder Associates Ltd. (Golder) has been retained by AECOM (formerly URS Canada Inc.) on behalf of the Ministry of Transportation, Ontario (MTO) to provide recommendations on foundation aspect for the preliminary design of the 4th Line (Churchill Sideroad) Underpass in Town of Innisfil. It is understood that the 4th Line Underpass will consist of a two-span, pre-cast girder bridge with 36 m span lengths.

Based on the General Arrangement (GA) Drawing provided by AECOM on August 5, 2015, the grade of the proposed Underpass varies between about Elevations 293.7 m and 293.9 m. In comparison, the proposed grade for Highway 400 is at about Elevation 286.0 m.

6.2 Foundation Options

As part of the future widening of Highway 400 in Simcoe County, the existing 4th Line Underpass and associated wingwalls will require replacement. According to the available information, the existing single-span structure is supported on spread footings that are founded at approximately Elevation 284.5 m. It is understood that the existing underpass is to be replaced with a two-span structure along the same alignment. Highway 400 is proposed to be widened by approximately 35 m to the west with the existing centreline re-aligned to the west of the existing highway but maintaining its grade at approximately Elevation 286.5 m. The 4th Line grade will be raised by about 1.5 m and 2 m at the east and west abutments, respectively, to accommodate the longer-span structure. It is understood at this time that staged construction of the replacement structure will not be required, as the existing structure is to be removed under a full closure of 4th Line. Based on the proposed span configuration for the replacement structure, there may be conflict at the proposed centre pier with the existing west abutment foundation. The existing spread footing at the west abutment should, therefore, be removed following removal of the existing structure and as such additional excavation for full removal of the footing may be required. 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 centre pier for the proposed structure. A summary of the advantages and disadvantages associated with each option is provided



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

- **Shallow foundations – spread/strip footings:** Shallow foundations comprised of spread or strip footings, founded on the dense to very dense silt and sand to sand deposit, the silty sand till deposits or “perched” within the embankment fill, are feasible for support of the new abutments and centre pier, although this foundation type will preclude the use of integral abutments. It is noted that based on observations during the current site investigation, the foundations for the existing structure appear to have performed satisfactorily to date. If the footings are founded on native soils, the proposed founding level will be below the groundwater level at the site, and groundwater control is expected to be required to enable shallow foundations to be constructed in “dry” conditions.
- **Deep foundations – driven steel H-piles or pipe piles:** Driven steel H-piles or steel pipe (tube) piles are feasible for support of the abutments and centre pier, and would permit design of conventional abutments, semi-integral abutments (for pipe piles) or integral abutments (for H-piles). At the locations of the abutments, the surface of the “100-blow” soil is between about Elevations 277 m and 278 m, and as such the minimum required pile length of 5 m for integral abutments will be achievable, assuming that the underside of the pile cap is above Elevation 283 m while still constructed below the depth of frost penetration. A “perched” pile cap would also minimize excavation and groundwater control requirements at the new abutments, notwithstanding the excavation that would have to be carried out for removal of the existing abutment foundations. Pile driving shoes are recommended to protect the pile tips from damage during driving into the very dense non-cohesive deposits.
- **Deep foundations – caissons:** Caissons are considered feasible for the support of the abutments and centre pier; however this option would preclude integral abutment design. This option would 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 that would be required. If caissons are adopted for support of the abutments and centre pier, they would extend into and through water-bearing non-cohesive soil deposits; temporary liners would be required during construction to control potential ground losses and/or disturbance of the caisson base.

Based on the above considerations, both shallow and deep foundation options are considered feasible for the support of the new abutments and centre pier, although pile foundations are preferred from a foundations perspective as they would permit integral abutments design. In addition, given that a spread footing would require excavations up to about 6.5 m deep into the native soils (below the existing Highway 400 grade) in order to achieve adequate bearing capacity for design, a pile foundation is also the preferred foundation at the centre pile.

6.3 Shallow Foundations

6.3.1 Founding Elevation

For support of the new abutments and centre pier, spread/strip footings should be founded on the dense silt and sand to sand/dense to very dense silty sand till deposits, or on compacted granular pads. Where spread/strip footings are to be founded on the native soil, the highest founding elevations recommended for preliminary design of footings are presented below.



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Foundation Element	Highest Founding Elevation (m)	Founding Soil
West Abutment	279.5	Very dense silty sand till
Centre Pier	280.0	Dense to very dense silty sand till
East Abutment	281.5	Dense sand

6.3.2 Geotechnical Axial Resistance and Reaction

The following factored geotechnical axial resistances at Ultimate Limit States (ULS) and geotechnical reactions at Serviceability Limit States (SLS, for 25 mm of settlement) may be used for preliminary design of spread/strip footing founded on the properly prepared sand/till deposits, or on a compacted Granular 'A' pad having a minimum thickness of 1 m.

Foundation Alternative	Factored Geotechnical Axial Resistance at ULS (kPa)	Geotechnical Reaction at SLS for 25 mm of Settlement (kPa)
Footing on properly prepared dense sand / dense to very dense silty sand till	750	400
Footing on minimum 1 m thick compacted Granular 'A' pad	750	350

Note: 1. The geotechnical resistance/ reaction values given above are estimated for a 3 m wide spread/strip footing.

The geotechnical resistances provided herein are given for loads will that be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the footing, inclination of the load should be taken into account in accordance with Section 6.7.2 of the *CHBDC (2006)*.

6.3.3 Resistance to Lateral Loads

Resistance to lateral forces/sliding resistance between cast-in-place concrete footings and the subsoils should be calculated in accordance with Section 6.7.5 of the *CHBDC (2006)*. The following presents the coefficient of friction, $\tan \phi'$, for the interface between the concrete footing and sand deposit or Granular 'A' pad.

Founding Material	Coefficient of Friction ($\tan \phi'$)
Cast-in-place concrete footing on native dense sand / dense to very dense sand silty sand till	0.55
Cast-in-place concrete footing on compacted Granular 'A' pad	0.60

The values presented above are unfactored values.



6.3.4 Frost Protection

The footings should be provided with a minimum 1.5 m of soil cover for frost protection as per OPSD 3090.101 (*Frost Penetration Depths for Southern Ontario*), as measured vertically and perpendicular from the face of the abutment slope to the edge of the underside of the footing.

If adequate soil cover cannot be provided for the footing, rigid styrofoam insulation could be installed to compensate for the lack of soil cover and provide protection from frost penetration.

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

6.4.1 Founding Elevation

The abutments, centre pier and associated wingwalls for the replacement structure may be supported on steel H-piles or pipe piles driven to found within the very dense ("100-blow") silty sand till deposit.

Based on the GA Drawing, integral abutments have been adopted for the design of the replacement structure, and the abutments will be "perched" within the 4th Line approach embankments, with the underside of the new pile caps at approximately Elevation 287.5 m. For the centre pier, the underside of the new pile cap is at Elevation 284.5 m. The following pile tip elevations are recommended for preliminary design, based on approximately 2 m of penetration into the "100-blow" lower silty sand till deposit.

Foundation Element	Approximate Surface Elevation of "100-Blow" Soil (m)	Estimated Design Tip Elevation (m)	Founding Soil at Tip Elevation
West Abutment	276.5	274.5	Silty sand till
Centre Pier	275.5	273.5	Silty sand till
East Abutment	278.0	276.0	Silty sand till

Based on the above elevations, the proposed piles are estimated to be approximately 13 m and 11.5 m long at the west and east abutment, respectively. At the centre pier the piles are estimated to be approximately 11 m long.

As discussed in Section 4.2.8, cobbles are inferred to be present within the silty sand till deposit. In this regard, steel H-piles are preferred over steel pipe piles as pipe piles are considered to pose a higher risk of experiencing refusal on cobbles and/or boulders or being deflected away from the vertical/battered orientation during installation due to their larger end area. Piles should be reinforced at the tip with driving shoes and/or flange plates in accordance with OPSD 3000.100 (Steel H-Pile Driving Shoe) or OPSD 3001.100 (Steel Tube Pile Driving Shoe) Type II, as appropriate, to reduce the potential for damage to the piles during driving. In very dense deposits containing cobbles and/or boulders, as potentially encountered at this site, driving shoes (such as Titus Standard 'H' Bearing Pile Points) are preferred over flange plates.



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6.4.2 Geotechnical Axial Resistance/Reaction

The factored geotechnical axial resistance at ULS and the geotechnical reaction at SLS (for 25 mm of settlement) for driven steel H-piles and closed-end, concrete-filled 324 mm (12-¾ in.) diameter steel pipe piles having a minimum wall thickness of 9.5 mm (3/8 in.) are presented below.

Pile Type	Approximate Pile Length (m)	Factored Geotechnical Axial Resistance at ULS (kN)	Geotechnical Reaction at SLS for 25 mm of Settlement (kN)
HP 310x110 (Integral Abutment)	11.5 to 13	1,300	N/A ¹
HP 310x110 (Conventional or Semi-Integral Abutment)	11.5 to 13	1,450	N/A ¹
HP 310x110 (Centre Pier)	11	1,400	N/A ¹
324 mm OD Pipe Pile (Conventional or Semi-Integral Abutment)	11.5 to 13	1,200	N/A ¹
324 mm OD Pipe Pile (Centre Pier)	11	1,150	N/A ¹

Note: 1. The geotechnical reaction at SLS for 25 mm of settlement will be greater than the factored axial resistance at ULS and as such, the SLS condition does not apply.

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 Pile Dynamic Analysis (PDA) or Hiley method (MTO's Standard Drawing SS103-11, Pile Driving Control) during the final stages of driving to verify that the required ultimate capacity has been achieved. Relaxation of soil surrounding the pile tips and/or heaving of the pile tips as a result of driving of adjacent piles could lead to reduced pile capacities. In this regard, it is recommended that a minimum of 10 per cent of piles be re-tapped at each foundation element to confirm that relaxation/heave is not occurring. If a significant reduction in the pile driving resistance is noted during re-tapping, all of the piles may need to be re-tapped and/or re-driven.

The preliminary geotechnical resistances/reactions provided above will have to be re-evaluated and modified, as necessary, during the design-build assignment in consideration of additional subsurface investigation at the foundation elements.

Consideration should be given to including, Pile Driving Note 2 in the Contact Drawings in assessing the individual pile resistances as per Section 3.3 of MTO's Structural Manual (2014).

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 pile (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 mobilized,



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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 mobilization 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.

The resistance to lateral loading in front of a single pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h (kPa/m), is based on the following equations (CFEM, 1992 as referenced in the CHBDC Commentary, 2006):

For cohesionless soils:

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

where: n_h = coefficient related to soil density (kPa/m)
 z = depth (m)
 B = pile diameter or width (m)

For cohesive soils:

$$k_h = \frac{67s_u}{B}$$

where: s_u = undrained shear strength of the soil (kPa)
 B = pile diameter or width (m)

The values of n_h (Terzaghi, 1955 and Reese, 1975) and s_u to be incorporated into the calculations of the coefficient of horizontal subgrade reaction (k_h) within the fill materials and the native subsoils to be utilized for the structural analysis of the piles and casings at this site are summarized below.

Soil Unit	n_h (kPa/m)	s_u (kPa)
Very loose to compact silty sand fill to silt and sand fill / organic silt	2,500	-
Very stiff sandy clayey silt	-	150
Stiff to very stiff silty clay till	-	100
Compact to very dense upper silty sand till	7,500	-
Compact silt and sand to sand	5,000	-
Dense to very dense silt and sand to sand	11,000	-
Dense to very dense lower silty sand till	11,000	-

Where integral abutment design includes the installation of CSP liners (with the annular space between the pile and liner filled with uniform-grained, uncompacted sand), the upper portion of the H-piles installed inside the CSP will be free to flex and move laterally. With this design, the passive lateral resistance over the length of the CSP liner should be neglected.



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Group action for lateral loading should also be considered when 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 (NAVFAC, 1982) in the direction of loading by a reduction factor, R , 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.

In order to estimate the lateral capacity of a single pile at ULS and SLS, a detailed analysis of pile response to lateral loads should be carried out when the detailed pile design (such as loading, pile head fixity, cross-section properties) becomes available.

6.5 Caisson Foundations

6.5.1 Founding Elevations

Caissons founded within the very dense silty sand till lower deposit may be considered for support of the abutments and center pier for the proposed replacement structure. The following caisson founding elevations may be used for preliminary design purposes, assuming about 2 m penetration into “100-blow” soil:

Foundation Element	Approximate Surface Elevation of “100-Blow” Soil (m)	Estimated Design Tip Elevation (m)	Founding Soil at Tip Elevation
West Abutment	276.5	274.5	Silty sand till
Centre Pier	275.5	273.5	Silty sand till
East Abutment	278.0	276.0	Silty sand till

If caisson foundations are adopted, a temporary liner and/or drilling slurry will be required to support the overburden soils during construction and balance groundwater pressures to minimize disturbance to the side walls and to control base disturbance/basal heave. In addition, placement of concrete by tremie methods would be required.

6.5.2 Geotechnical Axial Resistance/Reaction

The following factored geotechnical axial resistance at ULS and the geotechnical reaction at SLS (for 25 mm of settlement) may be used for design of caisson foundations:



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Caisson Diameter (m)	Factored Geotechnical Axial Resistance at ULS (kN)	Geotechnical Reaction at SLS for 25 mm of Settlement (kN)
0.9	3,000	1,750
1.2	6,000	2,300

The preliminary geotechnical resistances/reactions provided above will need to be re-evaluated and modified, as necessary, during the design-build assignment in consideration of any additional subsurface investigation at the foundation elements.

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 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment stem walls and any associated wingwalls/retaining walls will depend on the type and method of placement of the backfill material, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls.

The following recommendations are made concerning the design of the abutment walls and associated retaining walls. These design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select, free draining granular fill meeting the specifications of OPSS.PROV 1010 (Aggregates) Granular 'A' or Granular 'B' Type II, but with less than 5 per cent passing the No. 200 sieve, should be used as backfill behind the walls. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 (Compacting). Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3121.150 (Retaining Walls, Backfill, Minimum Granular Requirement).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with CHBDC Section 6.9.3 and Figure 6.6. Other surcharge loadings should be accounted for in the design as required.
- For restrained walls, granular fill should be placed in a zone with the width equal to at least 1.5 m behind the back of the wall (in accordance with Figure C6.20(a) of the *Commentary* to the CHBDC). For unrestrained walls, fill should be placed 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 (in accordance with Figure C6.20(b) of the *Commentary* to the CHBDC). The pressures are based on the proposed embankment fill material and the following parameters (unfactored) may be used:



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Fill Type	Soil Unit Weight	Coefficients of Static Lateral Earth Pressure	
		At-Rest, K_o	Active, K_a
Granular 'A'	22 kN/m ³	0.43	0.27
Granular 'B' Type II	21 kN/m ³	0.43	0.27

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 Retained Soil System (RSS) Walls

6.7.1 Founding Elevations

If retaining walls are required adjacent to the abutments and wingwalls at this site, it is assumed that the walls would be constructed parallel to 4th Line, with the base of the wall “stepped up” into the slope as the retaining wall extends away from the abutment. Retained soil system (RSS) walls are considered to be a suitable option for the conditions at this site.

The front facing panels and the reinforced soil mass of the RSS wall should be founded below any existing topsoil or unsuitable fill soils. Typically, the front facing panels are supported on a granular levelling pad at a shallow depth below the ground surface in front of the wall. The levelling pad should consist of a minimum thickness of 0.5 m of compacted Granular 'A' material, which should extend at least 0.5 m beyond the outside edge of both sides of the facing footing, then outward/downward at a slope of 1H:1V.

6.7.2 Geotechnical Resistance/Reaction

For the RSS facing panels founded on a 0.6 m wide footing constructed on a compacted granular pad as described above, preliminary design may be completed based on a factored geotechnical resistance at ULS of 150 kPa, and a geotechnical reaction at SLS (for 25 mm of settlement) of 100 kPa.

Assuming that the RSS wall (up to approximately 8 m high) acts as a unit and uses the full width of the reinforced soil mass (which can be taken as approximately 0.8 times the wall height for preliminary design) and is founded on a compacted granular pad, a factored geotechnical resistance at ULS of 250 kPa and a geotechnical reaction at SLS of 150 kPa may be used for preliminary design. The preliminary geotechnical resistance/reaction values should be reviewed and revised if necessary during the design-build assignment after the RSS wall configuration and “step” elevations are confirmed, taking into account any additional subsurface information at that time.



6.7.3 Resistance to Lateral Loads/Sliding Resistance

Resistance to lateral forces/sliding resistance between the compacted fill of the RSS wall and the subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC* (2006). The coefficient of friction, $\tan \phi'$, between the compacted granular fill of the RSS wall and the properly prepared subgrade may be taken as 0.5.

6.7.4 Global Stability of RSS Walls

The native overburden at this site is generally comprised of stiff to very stiff cohesive deposits or dense to very dense non-cohesive materials. The factor of safety against global instability of RSS walls, if adopted, is greater than 1.5. However, this preliminary assessment of the global stability of the retaining walls should be reviewed and confirmed as part of the design-build assignment as the design is further refined.

In addition, the internal stability of a reinforced earth structure is to be assessed by the proprietary product designer to ensure the internal stability of the wall is adequate.

6.7.5 Settlement

At this preliminary stage, it is estimated that for reconstructed approach embankments along the existing 4th Line alignment, with a grade raise of about 1.5 m and 2.5 m at the east and west approach embankment, respectively, to accommodate the longer-span replacement structure, the settlement of the underlying soils is estimated to be about 25 mm. This settlement is expected to be completed essentially during or upon completion of construction. Therefore, it is anticipated that the settlement performance for RSS walls and facing panels will be acceptable.

6.7.6 Performance and Appearance

Given that the RSS walls adjacent to the abutments are for a 400-series highway, a high site performance rating and a high appearance rating is required in accordance with the *MTO RSS Design Guidelines*.

6.8 Approach Embankments

6.8.1 Subgrade Preparation and Embankment Construction

Based on the existing topographic information it appears that the existing 4th Line embankment side slopes are inclined at about 2 horizontal to 1 vertical (2H:1V). As a result of the proposed grade raises of the 4th Line embankments, the front slopes along the east and west side of Highway 400, as well as the side slope of the embankments for any required widening of the 4th Line embankment, the new front side slopes should be formed at a maximum inclination of 2H:1V. Where widening of the existing embankment occurs, benching the existing embankment side slopes should be carried out in accordance with OPSD 208.010 (Benching of Earth Slopes).

To reduce erosion of the embankment side slopes due to surface water runoff, placement of topsoil and seeding or pegged sod should be carried out as soon as practicable after construction of the embankments. The erosion protection should be in accordance with OPSS.PROV 804 (Seed and Cover).



If the grade raise to the 4th Line embankments results in an overall earth embankment height of 8 m or greater, a 2 m wide bench should be geometry as per OPSD 202.010 (Slope Flattening).

6.8.2 Approach Embankment Stability and Settlement

Based on observations at the time of Golder's 2015 field investigation, the existing 4th Line embankment side slopes have performed satisfactorily, with no visual evidence of instability or settlement. Given that the native soils are predominantly comprised of stiff to very stiff cohesive soil deposits and dense to very dense non-cohesive soil deposits at this site, stability issues are not anticipated within the limits of the reconstructed approach embankments. Settlement associated with an up to 2.5 m grade raise at the approach embankments is estimated to be about 25 mm. Given the predominantly granular nature of the underlying soils, the majority of the settlement is expected to occur during or immediately upon completion of construction.

6.9 Construction Considerations

The following sections identify future construction considerations that may impact the future design and construction.

6.9.1 Open-Cut Excavation

The construction of new spread/strip footings and/or pile caps "perched" with the approach embankments (at about Elevation 287.5 m) will require excavations up to about 4.5 m below the existing 4th Line grade and will be made through the existing embankment fill. The construction of new spread/strip footings or pile caps at the centre pier location will require excavations up to about 6.5 m or 1.5 m, respectively, below the existing Highway 400 grade and will be made through the existing embankment fill and native soil deposits. The existing fill material and native stiff to very stiff sandy clayey silt/silty clay till deposits, compact to dense sand deposit, and dense to very dense silty sand till deposits are classified as Type 3 soils, according to the Occupational Health and Safety Act (OHSA) and, as such, temporary open-cut excavations above the groundwater level should be made with side slopes no steeper than 1H:1V.

All excavations must be carried out in accordance with Ontario Regulation 213 (Ontario Occupational Health and Safety Act for Construction Projects) (as amended).

6.9.2 Temporary Protection Systems

Temporary protection systems may be required along the existing Highway 400 northbound and southbound lanes to facilitate the removal of the existing bridge foundations and construction of the east abutment and centre pier. Where required, temporary protection systems should be designed and constructed in accordance with OPSS.PROV 539 (Temporary Protection System), and the lateral movement should meet Performance Level 2 provided that any existing adjacent utilities can tolerate this magnitude of deformation.

The selection and design of the protection system will be the responsibility of the Contractor.



6.9.3 Control of Groundwater

At the abutments, whether “perched” spread/strip footings or “perched” pile caps for deep foundations are adopted, any excavations should be maintained above the groundwater level at the site. At the centre pier, however, excavations for the spread/strip footing or pile cap could extend about 5 m or 0.5 m, respectively, below the groundwater level at the site.

The soils at the base of the excavation at the pier location consist of water-bearing, relatively permeable sand and silty sand till. At this preliminary stage, it is anticipated that an active dewatering system (beyond pumping from sumps within the excavation) will be required to lower the groundwater level. It is recommended that the groundwater level be lowered to 0.5 m to 1 m below the footing/pile cap founding level. At this preliminary stage, an accurate prediction of the groundwater pumping volumes cannot be made, as the flow rate would be dependent on whether the contractor includes an interlocking sheetpile cut-off wall and the duration for which the foundation excavation is open. However, it is considered that groundwater pumping volumes could exceed 50 m³/day during initial drawdown stages and/or during periods of heavy precipitation. For this pumping volume, a Permit to Take Water (PTTW) would be required.

At this preliminary stage, it is anticipated that the zone of influence for dewatering operations would be relatively localized at the structure site. Assuming the dewatering system is properly constructed and operated such that there is no loss of fine soil particles, the dewatering operations are not expected to cause excessive settlement in the sand and silty sand till deposits. However, the potential for settlement impacts on the structure foundations and any adjacent utilities should be re-assessed in the design-build assignment.

6.9.4 Obstructions

It should be noted that obstructions (inferred as cobbles and boulders) were encountered within the till deposits in the area of the proposed west abutment. The presence of such obstructions could affect excavation works, installation of temporary protection systems as well as construction of deep foundation. As such, it is recommended that additional site investigation be carried out during the design-build phase to determine the extent of such obstructions.

6.9.5 Ground and Groundwater Control for Caisson Construction

As discussed in Section 6.5, running or flowing of water-bearing non-cohesive soil deposits could occur during or after drilling of caissons (if adopted), and basal heave could occur at the caisson base. If caisson foundations are adopted, temporary caisson liners with a balancing head of water and/or drilling slurry will be required to support the overburden soils and balance groundwater pressures during construction. In addition, placement of concrete by tremie methods would be required.

6.9.6 Protection of Subgrade

The non-cohesive soils that will be exposed within the excavations at the abutments and centre pier will be susceptible to disturbance from construction traffic and/or precipitation and ponded water. To limit the effects of this disturbance, a concrete working slab should be placed on the subgrade within four hours after preparation,



inspection and approval of the subgrade. The minimum thickness of the concrete working slab should be 100 mm and the concrete should have a minimum 28-day compressive strength of 20 MPa.

6.10 Recommendations for Future Work During Detail Design

During the design-build phase, it is recommended that additional site investigation and field testing be carried out to determine the extent of cobbles and boulders, should deep foundation be adopted for the 4th Line Underpass. The scope and results of this investigation must be reviewed at that time to determine if they meet the then-current MTO requirements for the structure type and span configuration under consideration, and if additional investigation and analysis is necessary.



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7.0 CLOSURE

This Foundation Design Report was prepared by Mr. Tomasz Zalucki, P.Eng., a geotechnical engineer, and reviewed by Mr. Christopher Ng, P.Eng., a geotechnical engineer and Associate with Golder. Mr. Jorge M. A. Costa, P.Eng., a Principal with Golder and Designated MTO Foundations Contact, conducted an independent review of this report.

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ASTM International:

ASTM D1586 Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils

Ministry of Transportation Ontario:

Drawing SS103-11 Pile Driving Control

Ontario Occupational Health and Safety Act:

Ontario Regulation 213 Construction Projects (as amended)



PRELIMINARY FOUNDATION REPORT - 4TH LINE UNDERPASS

Ontario Provincial Standard Specifications (OPSS)

OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS 903	Construction Specification for Deep Foundations
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material

Ontario Provincial Standard Drawings (OPSD)

OPSD 202.010	Slope Flattening Using Surplus Excavated Material on Earth or Rock Embankment
OPSD 208.010	Benching of Earth Slopes
OPSD 3000.100	Foundation, Piles, Steel H-Pile, Driving Shoe
OPSD 3001.100	Foundation, Piles, Steel Tube Piles, Driving Shoe
OPSD 3090.101	Foundation, Frost Penetration Depths for Southern Ontario
OPSD 3121.150	Walls, Retaining, Backfill, Minimum Granular Requirement

Ontario Water Resources Act:

Ontario Regulation 903	Wells (as amended)
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TABLES



PRELIMINARY FOUNDATION REPORT - 4TH LINE UNDERPASS

TABLE 1 – COMPARISON OF REPLACEMENT STRUCTURE FOUNDATION ALTERNATIVES

Foundation Option	Feasibility	Advantages	Disadvantages	Estimated Costs	Risk / Consequences
Spread/strip footings	<ul style="list-style-type: none"> Feasible for support of new abutments “perched” within the approach embankments; and centre pier. 	<ul style="list-style-type: none"> Conventional excavation and construction techniques. Lower cost compared to deep foundations 	<ul style="list-style-type: none"> Excavation at the centre pier will extend up to about 5 m below the groundwater level, and groundwater control will be required. Requires larger footing excavation and disposal of a larger volume of soil compared to the excavation for a pile cap. Does not allow for integral abutment construction. 	<ul style="list-style-type: none"> Estimated cost is approximately \$600/m³ for construction of shallow foundations. 	<ul style="list-style-type: none"> Softening / loosening of subgrade due to groundwater would require a concrete to be placed working slab immediately after excavation to design depth, inspection and approval of subgrade. Subgrade should be protected from freezing.
Steel H-piles or pipe piles	<ul style="list-style-type: none"> Feasible for support of new abutments with pile cap below the Highway 400 grade or “perched” within the approach embankments; and centre pier. 	<ul style="list-style-type: none"> Conventional construction methods for H-pile or steel pipe pile foundations. Abutment pile caps could be maintained higher than spread footings, potentially reducing depth of excavation and protection system requirements. Steel H-piles allow for integral abutment configuration; and pipe piles allow for semi-integral abutment configuration. Higher geotechnical resistance than for 	<ul style="list-style-type: none"> Temporary excavation support may still be required to facilitate removal of existing abutments. Piles may refuse above design tip elevation due to very dense overburden soils, especially pipe piles which have a larger displacement base. Excavation for the pile cap at the centre pier will extend about 2.5 m below the groundwater level, and groundwater control will be required. Pipe piles not readily 	<ul style="list-style-type: none"> Estimated cost is approximately \$250/m length for pile installation and \$600/m³ for pile cap construction. 	<ul style="list-style-type: none"> Minor potential for pile damage / deflection if cobbles and boulders are encountered during pile driving. Slightly greater risk in this regard for pipe piles as compared with H-piles if boulders are encountered during pile driving.



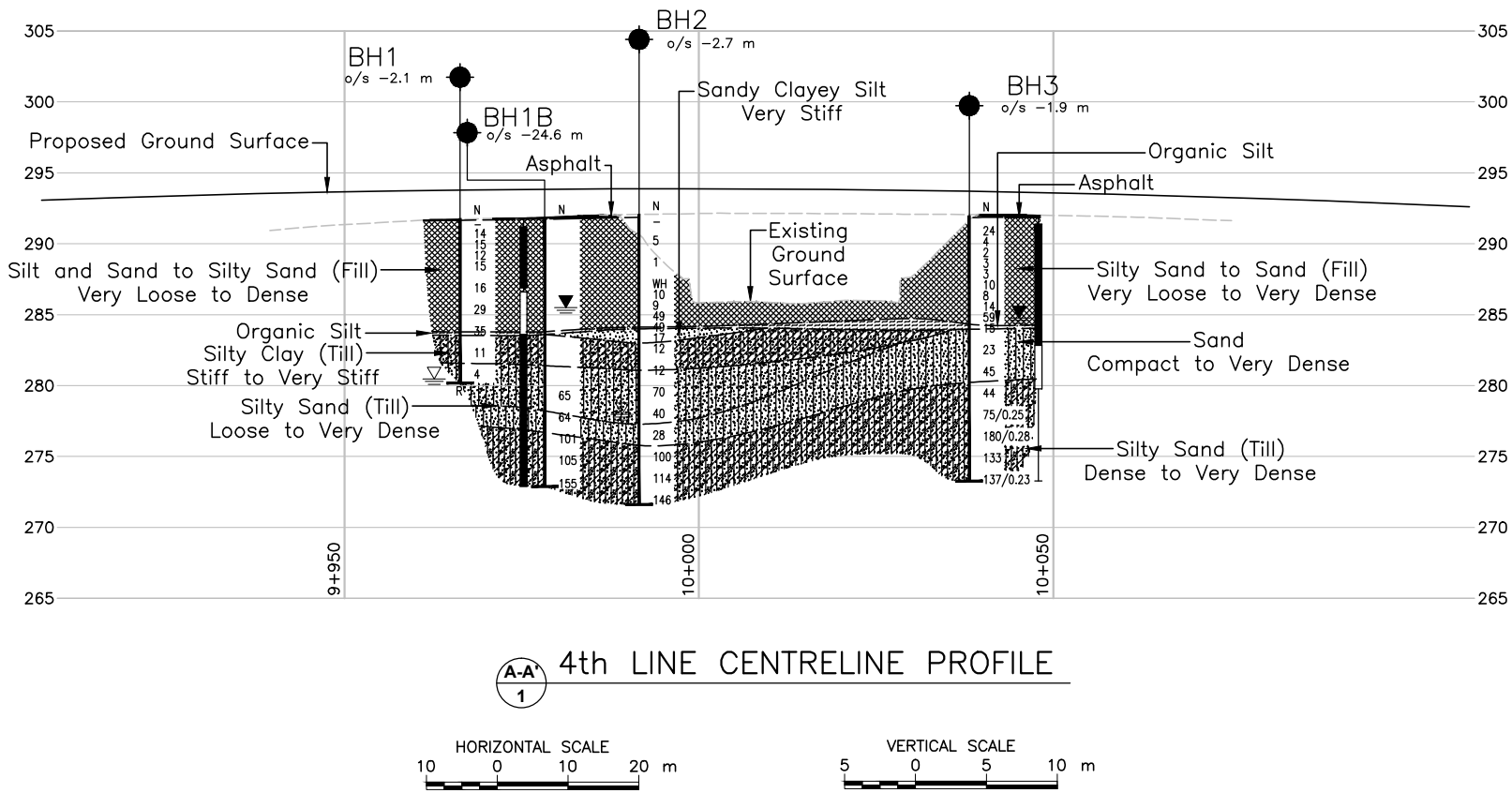
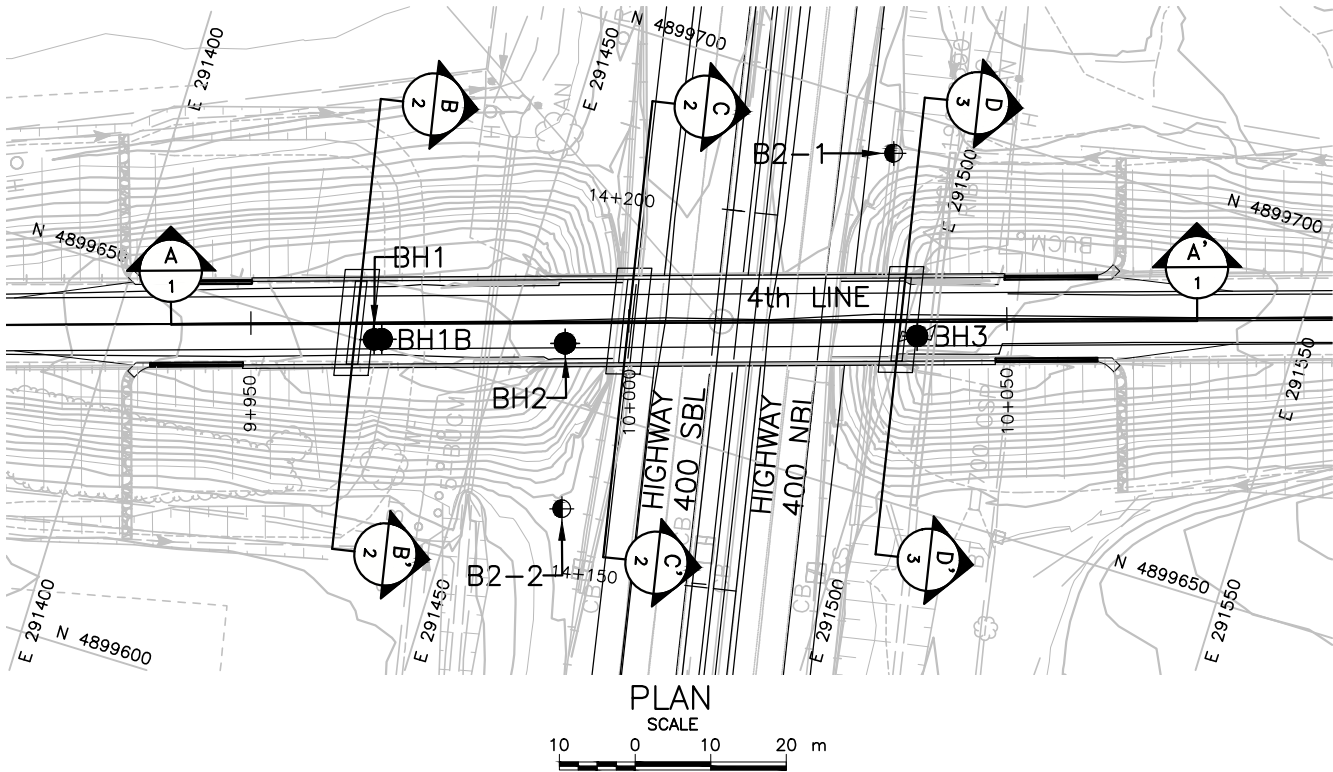
PRELIMINARY FOUNDATION REPORT - 4TH LINE UNDERPASS

TABLE 1 – COMPARISON OF REPLACEMENT STRUCTURE FOUNDATION ALTERNATIVES

Foundation Option	Feasibility	Advantages	Disadvantages	Estimated Costs	Risk / Consequences
		shallow foundations.	accepted for integral abutment construction.		
Caissons	<ul style="list-style-type: none"> Feasible but not recommended for support of abutments and centre pier. 	<ul style="list-style-type: none"> Abutment pile caps could be maintained higher than spread footings, or H-pile caps, potentially reducing depth of excavation and protection system requirements, or caps can be constructed at level of underside of structure. Higher capacity than for driven piles, so reduced number of deep foundation elements compared to piles. 	<ul style="list-style-type: none"> Caissons would extend below the groundwater level at the site into water-bearing non-cohesive soils, with potential for loss of ground or base disturbance. Temporary liners would be required, plus special measures such as use of drilling mud and tremie placement of concrete; likely not possible to inspect caisson base. Precludes use of integral abutments. More expensive compared to shallow foundations. 	<ul style="list-style-type: none"> Estimated cost is approximately \$1,000/m length for caisson installation and \$600/m³ for pile cap construction; the cost may be higher to account for temporary/pavement liners. 	<ul style="list-style-type: none"> Risk of loosening and leaving in place disturbing founding soils at base of caissons.



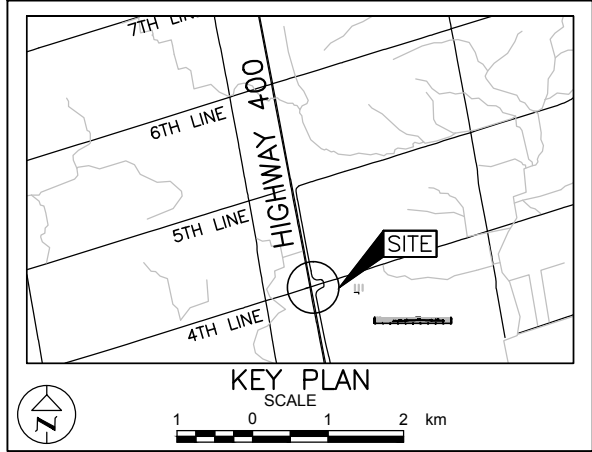
DRAWINGS



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

CONT No.
WO No. 06-20016

4TH LINE UNDERPASS
HIGHWAY 400 WIDENING
BOREHOLE LOCATIONS AND SOIL STRATA



LEGEND

- Borehole - Current Investigation
- Borehole - Previous Investigation
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL upon completion of drilling
- WL in piezometer
- R Refusal

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
B2-1	286.2	4899694.3	291491.8
B2-2	285.9	4899636.5	291463.6
BH1	291.8	4899650.6	291433.3
BH1B	291.8	4899650.9	291434.3
BH2	292.1	4899657.5	291457.6
BH3	292.0	4899672.1	291501.8

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 design configuration as shown elsewhere in the design drawings.

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

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

REFERENCE

Base plans provided in digital format by AECOM, drawing file nos. X-Base_4th Line.dwg and X-Design_4th Line_Interim.dwg, received June 22, 2015. Surface data provided in digital format by AECOM, drawing file no. X-Design_4th Line_Interim.dwg, received June 22, 2015.

NO.	DATE	BY	REVISION

Geocres No. 31D-616

HWY. 400	PROJECT NO. 14-1111-0002	DIST. .
SUBM'D. JIL	CHKD. CN	DATE: Sep. 2015
DRAWN: MR	CHKD. MCK	APPD. JMAC

SITE: 30-212
DWG. 1

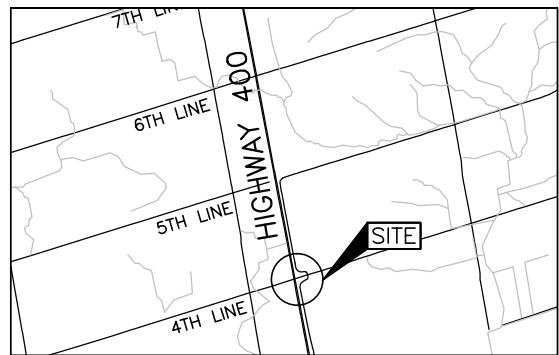


METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No. .
WO No. 06-20016

4TH LINE UNDERPASS
HIGHWAY 400 WIDENING
SOIL STRATA

SHEET



KEY PLAN
SCALE
1 0 1 2 km

LEGEND

- Borehole - Current Investigation
- ⊙ Borehole - Previous Investigation
- ⬮ Seal
- ⬮ Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- ≡ WL upon completion of drilling
- ≡ WL in piezometer
- R Refusal

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
B2-2	285.9	4899636.5	291463.6
BH1	291.8	4899650.6	291433.3
BH1B	291.8	4899650.9	291434.3
BH2	292.1	4899657.5	291457.6

NOTES

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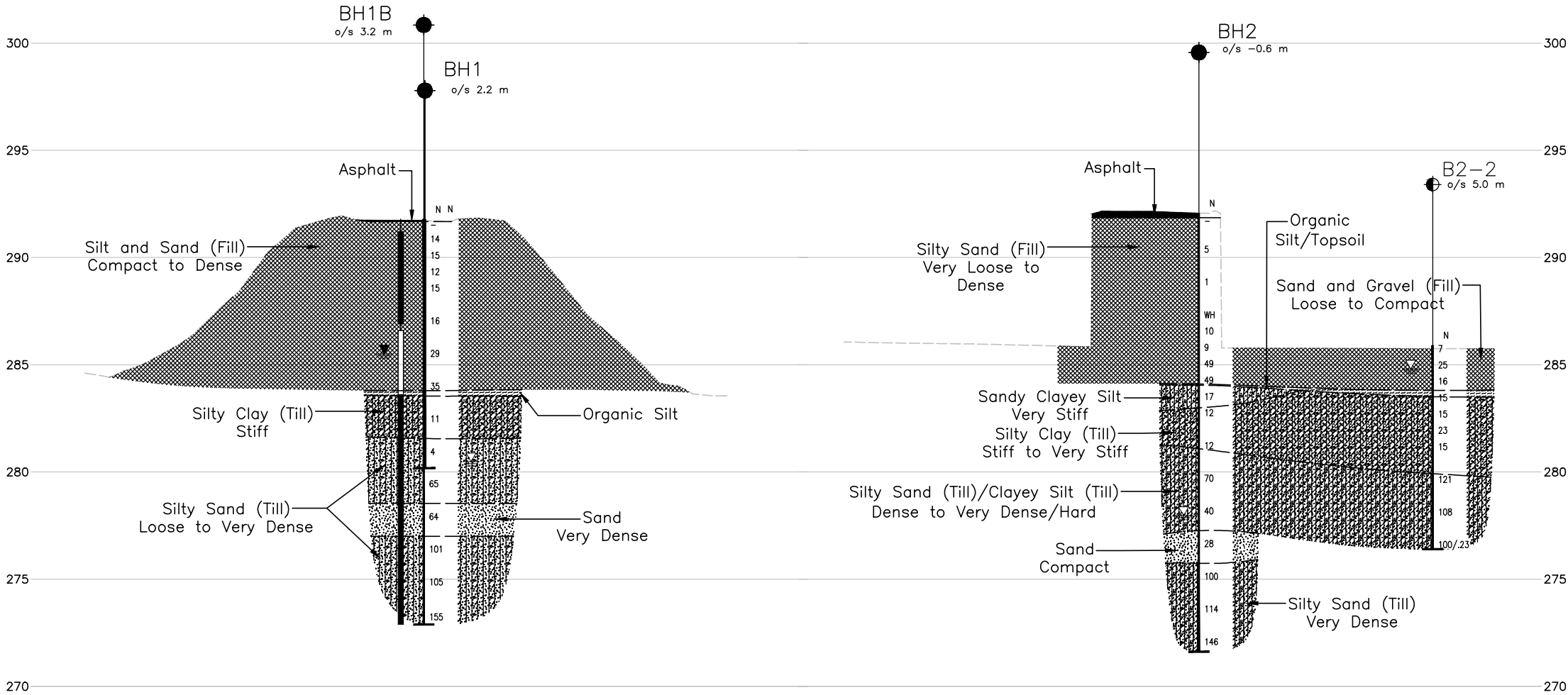
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REFERENCE

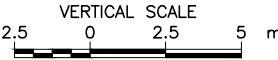
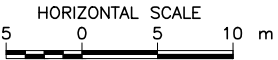
Base plans provided in digital format by AECOM, drawing file nos. X-Base_4th Line.dwg and X-Design_4th Line_Interim.dwg, received June 22, 2015. Surface data provided in digital format by AECOM, drawing file no. X-Design_4th Line_Interim.dwg, received June 22, 2015.

NO.	DATE	BY	REVISION
Geocres No. 31D-616			
HWY. 400	PROJECT NO. 14-1111-0002		DIST. .
SUBM'D. JIL	CHKD. CN	DATE: Sep. 2015	SITE: 30-212
DRAWN: MR	CHKD. MCK	APPD. JMAC	DWG. 2



B-B
1
4th LINE WEST ABUTMENT

C-C
1
4th LINE CENTRE PIER

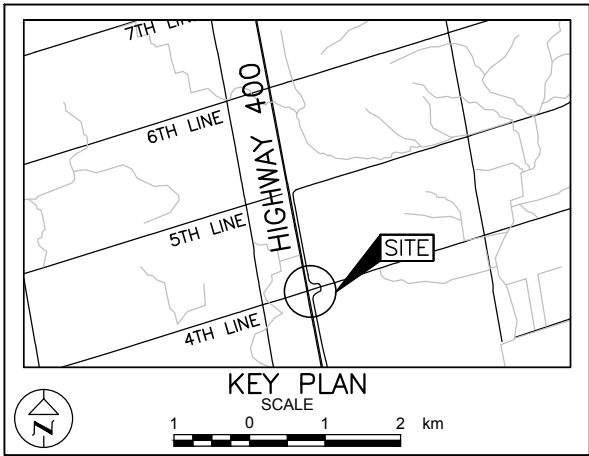


METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No. .
WO No. 06-20016

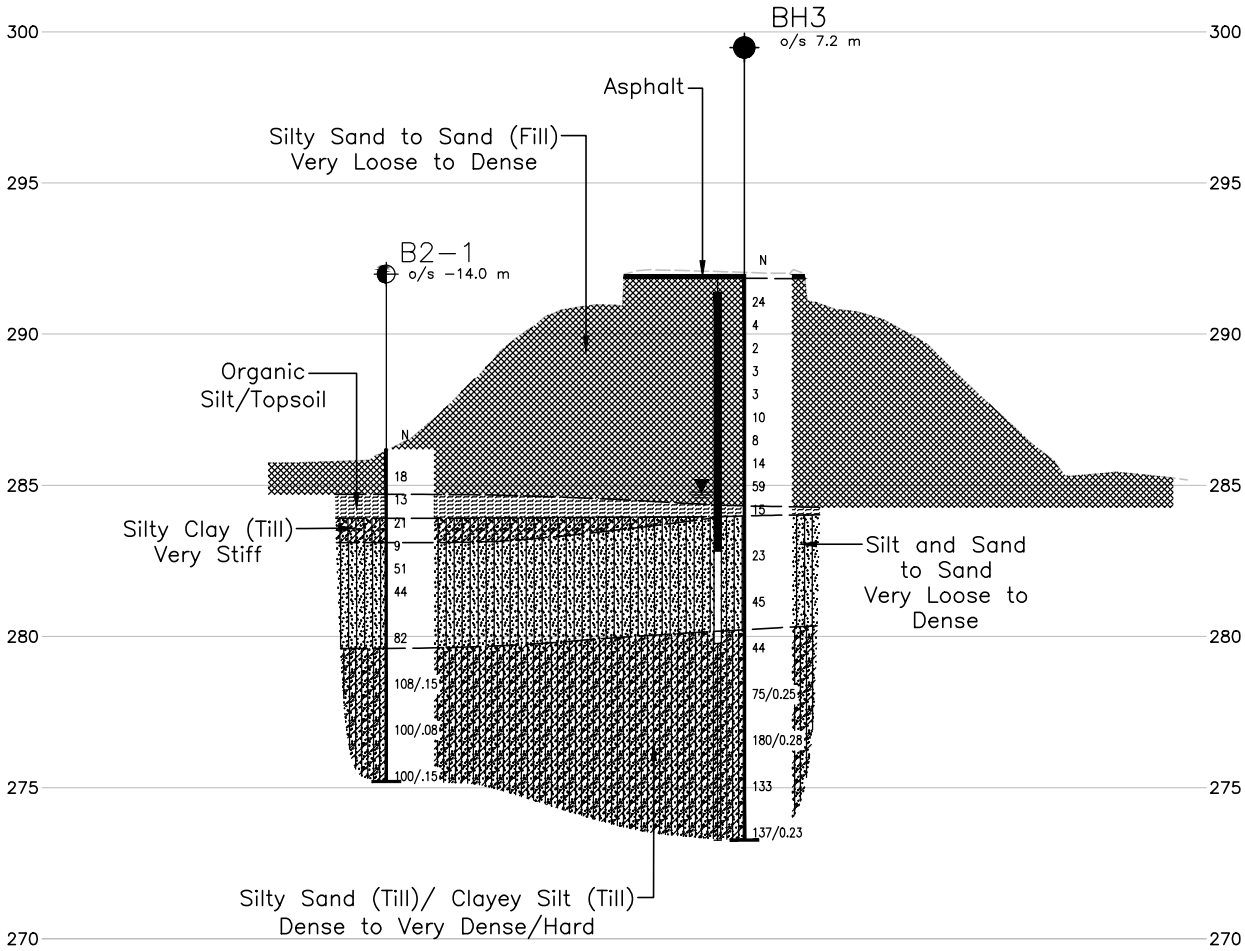
4TH LINE UNDERPASS
HIGHWAY 400 WIDENING
SOIL STRATA

SHEET



LEGEND			
	Borehole	-	Current Investigation
	Borehole	-	Previous Investigation
	Seal		
	Piezometer		
N	Standard Penetration Test Value		
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)		
	WL upon completion of drilling		
	WL in piezometer		
R	Refusal		

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
B2-1	286.2	4899694.3	291491.8
BH3	292.0	4899672.1	291501.8



D-D' 4th LINE EAST ABUTMENT
1



NOTES

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REFERENCE

Base plans provided in digital format by AECOM, drawing file nos. X-Base_4th Line.dwg and X-Design_4th Line_Interim.dwg, received June 22, 2015. Surface data provided in digital format by AECOM, drawing file no. X-Design_4th Line_Interim.dwg, received June 22, 2015.



NO.	DATE	BY	REVISION
Geocres No. 31D-616			
HWY. 400	PROJECT NO. 14-1111-0002		DIST. .
SUBM'D. JIL	CHKD. CN	DATE: Sep. 2015	SITE: 30-212
DRAWN: MR	CHKD. MCK	APPD. JMAC	DWG. 3



APPENDIX A

Record of Boreholes – Golder 2015 Investigation



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
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

Notes: 1
2

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



LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d :

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Non-Cohesive Soils

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

(b) Cohesive Soils Consistency

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

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO ₄	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

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

V. MINOR SOIL CONSTITUENTS

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

PROJECT <u>14-1111-0002</u>		RECORD OF BOREHOLE No BH1		SHEET 1 OF 2		METRIC	
W.O. <u>06-20016</u>		LOCATION <u>N 4899650.6 ; E 291433.3</u>		ORIGINATED BY <u>JIL</u>			
DIST <u>Central</u> HWY <u>400</u>		BOREHOLE TYPE <u>210 mm O.D. Continuous Flight Hollow Stem Augers</u>		COMPILED BY <u>ZR/JFC</u>			
DATUM <u>Geodetic</u>		DATE <u>June 4, 2015</u>		CHECKED BY <u>JIL/CN</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W _p	W	W _L		GR	SA	SI	CL
								○ UNCONFINED + FIELD VANE	20	40	60	80	100	● QUICK TRIAXIAL × REMOULDED	WATER CONTENT (%)					
291.8	GROUND SURFACE						20	40	60	80	100		10	20	30					
0.0	ASPHALT (100 mm)																			
	Silt and sand, trace to some gravel, trace to some clay, some clayey silt lenses (FILL) Compact to dense Brown to grey Dry to moist		1	AS	-															
			2	SS	14															
			3	SS	15															
			4	SS	12															
			5	SS	15															
			6	SS	16															
			7	SS	29															
			8A	SS	35															
283.8	ORGANIC SILT, trace sand, trace rootlets Black Moist		8B																	
8.2	SILTY CLAY, some sand (TILL) Stiff Grey Moist to wet		9	SS	11															
			10	SS	4															
281.6	Silty SAND, some clay, trace gravel (TILL) Loose Grey Moist to wet																			
280.2	END OF BOREHOLE AUGER REFUSAL (Inferred Boulder)																			
11.6	NOTES: 1. Water level in open borehole measured at a depth of 11.3 m below ground surface (Elev. 280.5 m) upon completion of drilling. 2. An additional borehole was advanced 2.0 m east of BH1 to obtain split spoon samples; see Record of Borehole BH1B for details.																			

Continued Next Page

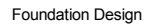
+³, ×³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

GTA-MTO 001 S:\CLIENTS\MTOWHY_400_BARRIER02_DATA\GINT\1411110002.GPJ GAL-GTA.GDT 9/30/15



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

GTA-MTO 001 S:\CLIENTS\IMTO\HWY 400 BARRIER\02 DATA\GINT\1411110002.GPJ GAL-GTA.GDT 9/30/15



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

PROJECT		14-1111-0002		RECORD OF BOREHOLE No BH1B		SHEET 2 OF 2		METRIC							
W.O.		06-20016		LOCATION		N 4899650.9 ; E 291434.3		ORIGINATED BY							
DIST		Central HWY 400		BOREHOLE TYPE		210 mm O.D. Continuous Flight Hollow Stem Augers		COMPILED BY							
DATUM		Geodetic		DATE		June 4, 2015		CHECKED BY							
JIL/CN															
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS		ELEVATION SCALE		DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES										
	--- CONTINUED FROM PREVIOUS PAGE ---														
	Silty SAND, trace gravel to gravelly, some clay (TILL) Very dense Grey Moist		3	SS	101										
			4	SS	105										
			5	SS	155										
272.9															
18.9	END OF BOREHOLE														
	NOTES:														
	1. Water level measurements in piezometer:														
	Date Depth (m) Elev. (m)														
	07/06/15 6.3 285.5														
	2. Water level meter passed through an approximately 0.5 m thick clayey slurry before reaching the bottom of the standpipe piezometer at a depth of 6.8 m below ground surface (Elev. 285.0 m) on July 6, 2015.														

PROJECT 14-1111-0002		RECORD OF BOREHOLE No BH2		SHEET 1 OF 2	METRIC
W.O. 06-20016	LOCATION N 4899657.5; E 291457.6	ORIGINATED BY JIL			
DIST Central HWY 400	BOREHOLE TYPE 210 mm O.D. Continuous Flight Hollow Stem Augers	COMPILED BY ZR/JFC			
DATUM Geodetic	DATE June 3 and 4, 2015	CHECKED BY JIL/CN			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL LIMIT MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	20 40 60 80 100	20 40 60 80 100	W _p W W _L						
292.1	GROUND SURFACE																
0.0	ASPHALT (200 mm)																
0.2	Silty sand, trace to some gravel, trace to some clay (FILL) Very loose to dense Brown to grey Moist to wet		1	AS	-												
			2	SS	5												
			3	SS	1												
			4	SS	WH												
			5	SS	10												
			6	SS	9												
			7	SS	49												
284.1	Auger grinding at a depth of about 7.6 m on inferred cobble.		8A	SS	49												
8.0	ORGANIC SILT, trace rootlets Black Moist		8B	SS	49												
	Sandy CLAYEY SILT, some gravel Very stiff Grey Moist		9	SS	17												
283.0	SILTY CLAY, trace gravel, trace sand, some sand lenses (TILL) Stiff Grey to brown Moist		10	SS	12												
9.1																	
			11A	SS	12												
281.1	Silty SAND, trace to some gravel, trace to some clay (TILL) Compact to very dense Brown to grey Moist		11B	SS	12												
11.0			12	SS	70												
	Auger grinding at a depth of about 13.1 m on inferred cobble.		13	SS	40												
277.3																	
14.8																	

Continued Next Page

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

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PROJECT <u>14-1111-0002</u>		RECORD OF BOREHOLE No BH2		SHEET 2 OF 2		METRIC	
W.O. <u>06-20016</u>		LOCATION <u>N 4899657.5; E 291457.6</u>		ORIGINATED BY <u>JIL</u>			
DIST <u>Central</u> HWY <u>400</u>		BOREHOLE TYPE <u>210 mm O.D. Continuous Flight Hollow Stem Augers</u>		COMPILED BY <u>ZR/JFC</u>			
DATUM <u>Geodetic</u>		DATE <u>June 3 and 4, 2015</u>		CHECKED BY <u>JIL/CN</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL	
								20	40	60	80	100	W _p	W	W _L						
								○ UNCONFINED													
	--- CONTINUED FROM PREVIOUS PAGE ---																				
275.8	SAND, some silt, trace gravel Compact Grey Wet		14	SS	28																
16.3	Silty SAND, trace to some gravel, trace to some clay (TILL) Very dense Brown to grey Moist		15	SS	100																
			16	SS	114																
271.7			17	SS	146																
20.4	END OF BOREHOLE																				
	NOTE: 1. Water level in open borehole measured at a depth of 14.1 m below ground surface (Elev. 278.0 m) upon completion of drilling.																				


PROJECT 14-1111-0002		RECORD OF BOREHOLE No BH3		SHEET 1 OF 2		METRIC	
W.O. 06-20016		LOCATION N 4899672.1 ; E 291501.8		ORIGINATED BY QC			
DIST Central HWY 400		BOREHOLE TYPE 210 mm O.D. Continuous Flight Hollow Stem Augers		COMPILED BY ZR/JFC			
DATUM Geodetic		DATE June 11, 2015		CHECKED BY JIL/CN			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL LIMIT MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	20	40	60	80	100	W _p	W		W _L			
292.0	GROUND SURFACE																			
0.0	ASPHALT (130 mm)		1	AS																
0.1	Sand, some gravel to gravelly, some silt to silty, some asphalt fragments (FILL) Loose to compact Brown Moist		2	SS	24															
			3	SS	4															
289.9			4	SS	2															
2.1	Silty sand, trace to some gravel, trace to some clay, trace organics, some asphalt fragments (FILL) Very loose to very dense Brown to grey Moist		5	SS	3															
			6	SS	3															
			7	SS	10															
			8	SS	8															
			9	SS	14															
	Trace organics at a depth of 6.9 m.		10	SS	59															
284.3			11A	SS	15															
284.0	ORGANIC SILT, trace sand Black Moist		11B																	
8.0	SAND, some silt, trace gravel, trace clay, some silty clay pockets Compact to dense Brown to grey Wet		12	SS	23															
			13	SS	45															
			14	SS	44															
280.3	Silty SAND, trace to some gravel, trace to some clay (TILL) Dense to very dense Grey Moist		15	SS	75/0.25															
11.7																				

Continued Next Page

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

GTA-MTO 001 S:\CLIENTS\MTOWHY_400_BARRIER2_DATA\GINT\1411110002.GPJ GAL-GTA.GDT 9/30/15

PROJECT		RECORD OF BOREHOLE No BH3				SHEET 2 OF 2		METRIC																	
W.O. 06-20016		LOCATION N 4899672.1 ; E 291501.8				ORIGINATED BY QC																			
DIST Central HWY 400		BOREHOLE TYPE 210 mm O.D.Continuous Flight Hollow Stem Augers				COMPILED BY ZR/JFC																			
DATUM Geodetic		DATE June 11, 2015				CHECKED BY JIL/CN																			
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL									
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa									WATER CONTENT (%)								
	--- CONTINUED FROM PREVIOUS PAGE ---						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					10 20 30													
273.3	Silty SAND, trace to some gravel, trace to some clay (TILL) Dense to very dense Grey Moist		16	SS	180/0.28																				
						276																			
			17	SS	133																				
						275										7 58 24 11									
						274																			
18.7	END OF BOREHOLE		18	SS	137/0.24																				
NOTE: 1. Water level measurements in piezometer: <table border="1" style="display: inline-table; margin-left: 40px;"> <thead> <tr> <th>Date</th> <th>Depth (m)</th> <th>Elev. (m)</th> </tr> </thead> <tbody> <tr> <td>06/11/15</td> <td>7.2</td> <td>284.8</td> </tr> <tr> <td>07/06/15</td> <td>7.2</td> <td>284.8</td> </tr> </tbody> </table>		Date	Depth (m)	Elev. (m)	06/11/15	7.2	284.8	07/06/15	7.2	284.8															
Date	Depth (m)	Elev. (m)																							
06/11/15	7.2	284.8																							
07/06/15	7.2	284.8																							



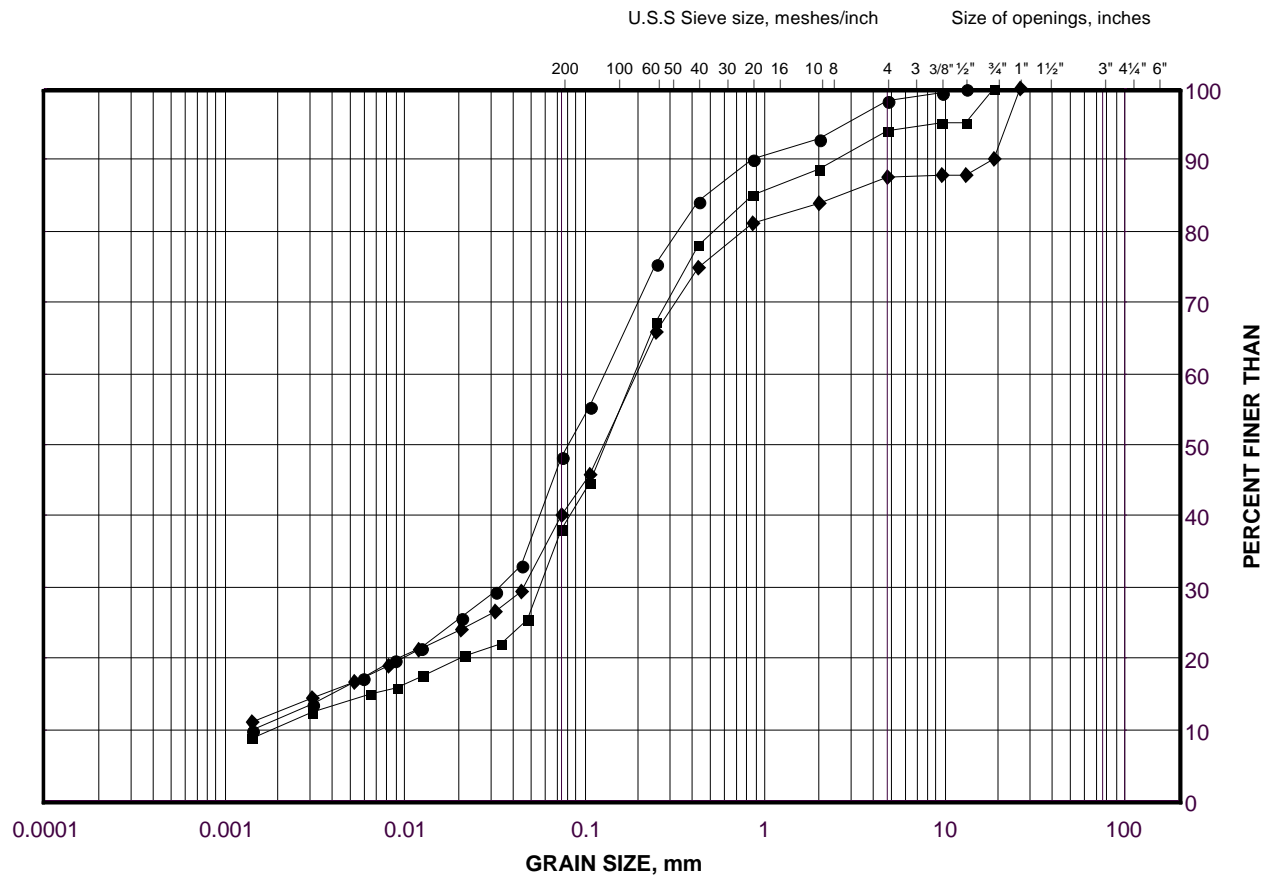
APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION

Silty Sand to Silt and Sand (Fill)

FIGURE B1



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

LEGEND

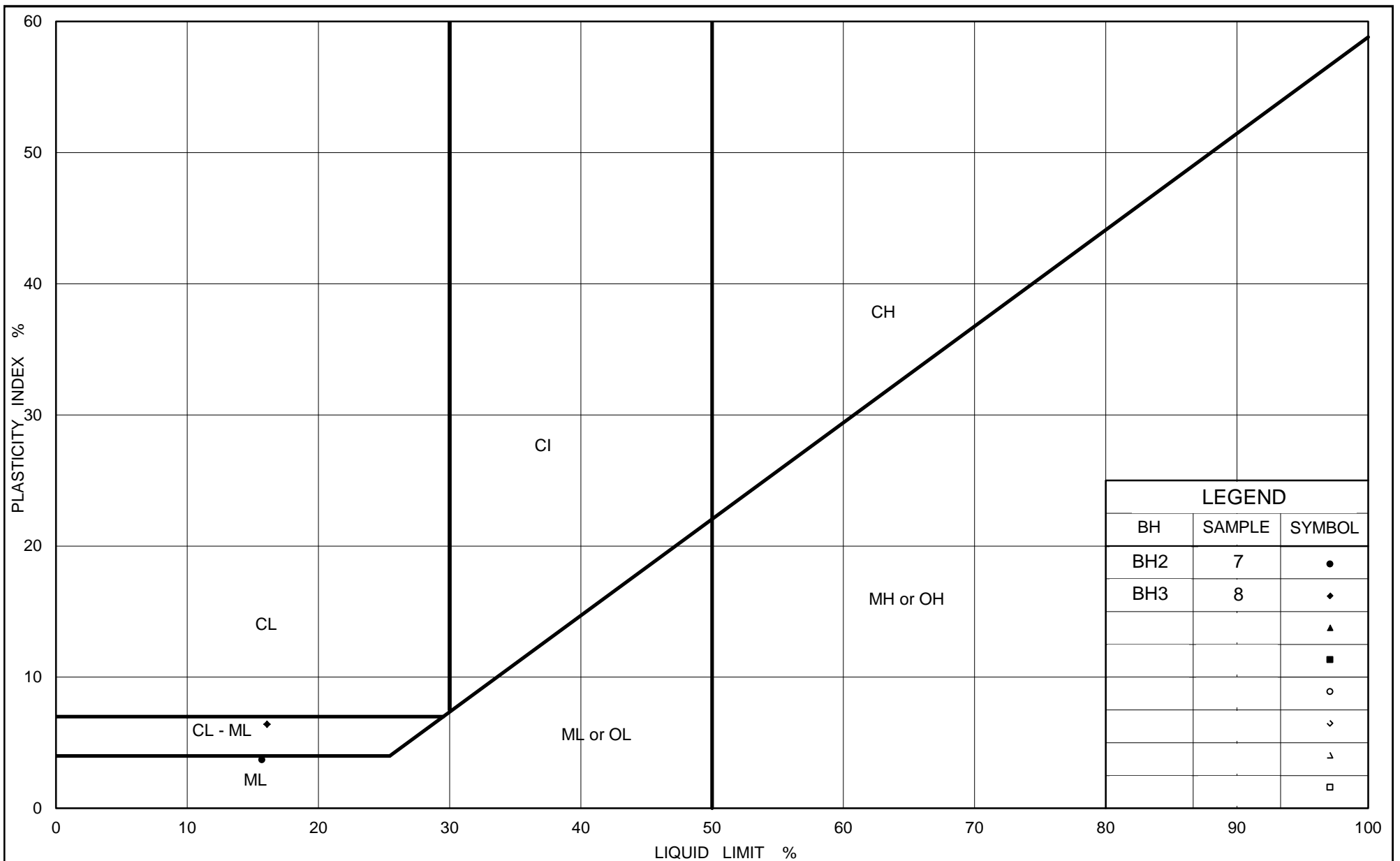
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	BH1	2	209.7
■	BH2	7	284.9
◆	BH3	8	286.3

Project Number: 14-1111-0002

Checked By: CN

Golder Associates

Date: 10-Jul-15

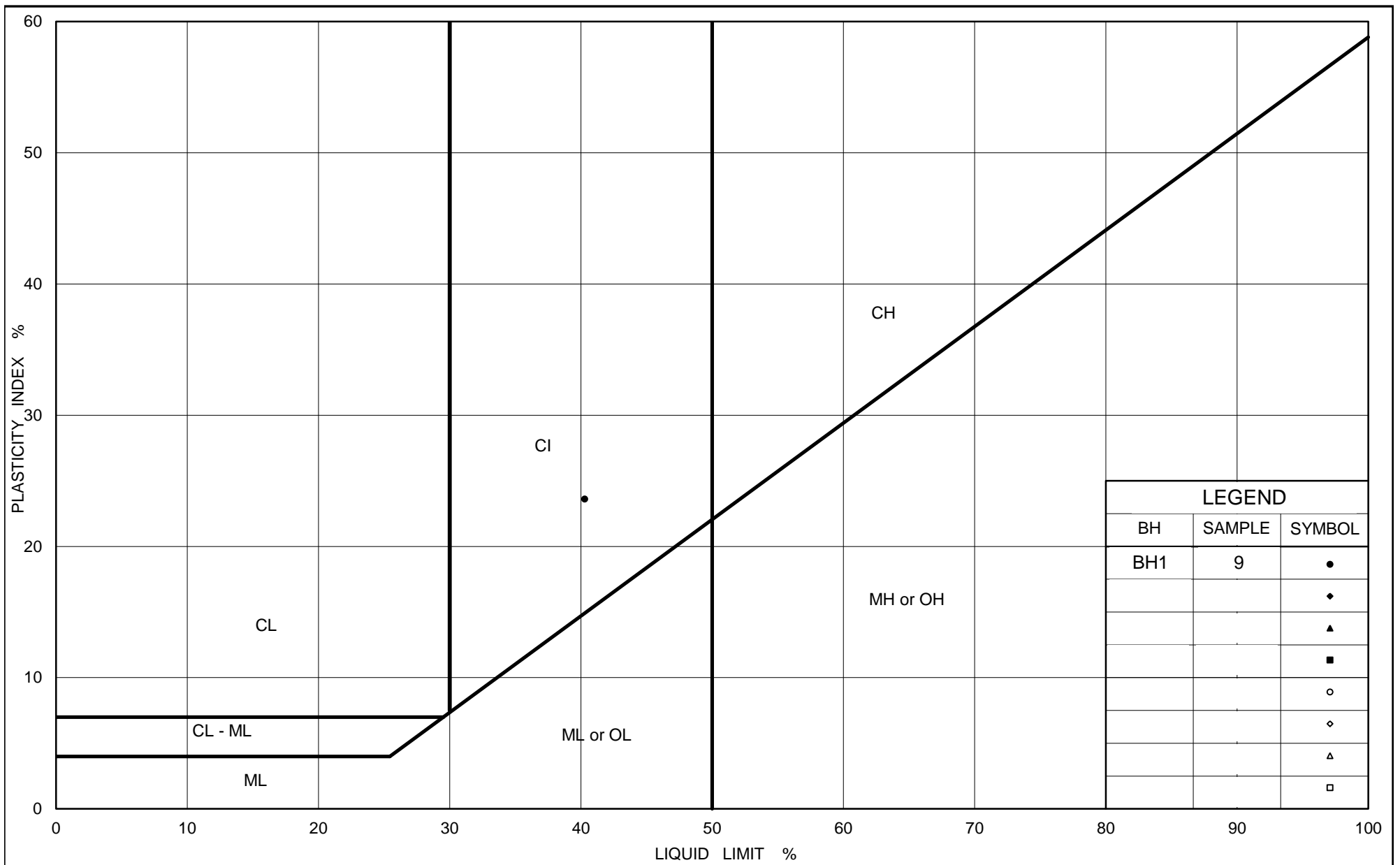


PLASTICITY CHART Silty Sand (Fill)

Figure No. B2

Project No. 14-1111-0002

Checked By: CN



PLASTICITY CHART Silty Clay (Till)

Figure No. B3

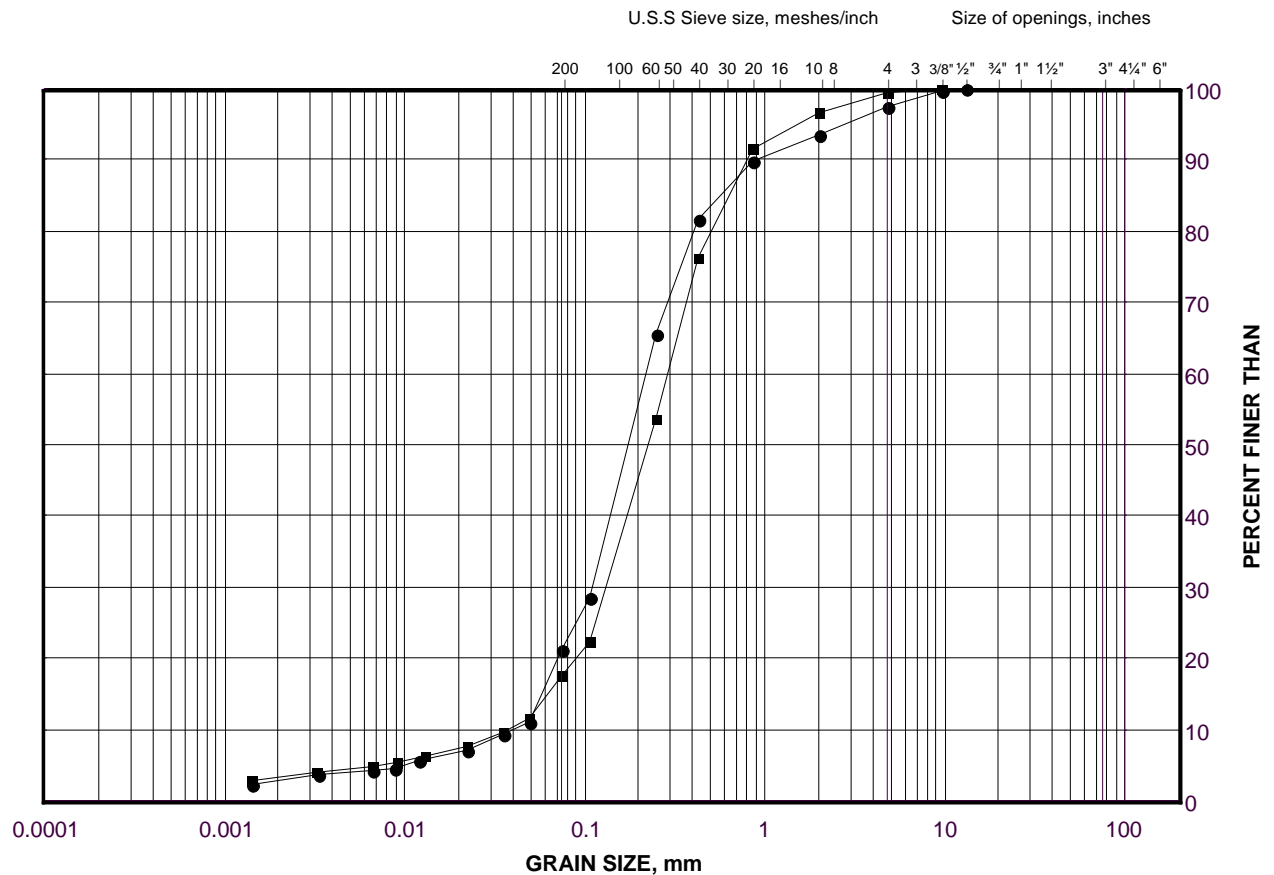
Project No. 14-1111-0002

Checked By: CN

GRAIN SIZE DISTRIBUTION

Sand

FIGURE B4



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	BH3	12	282.5
■	BH1B	2	277.8

Project Number: 14-1111-0002

Checked By: CN

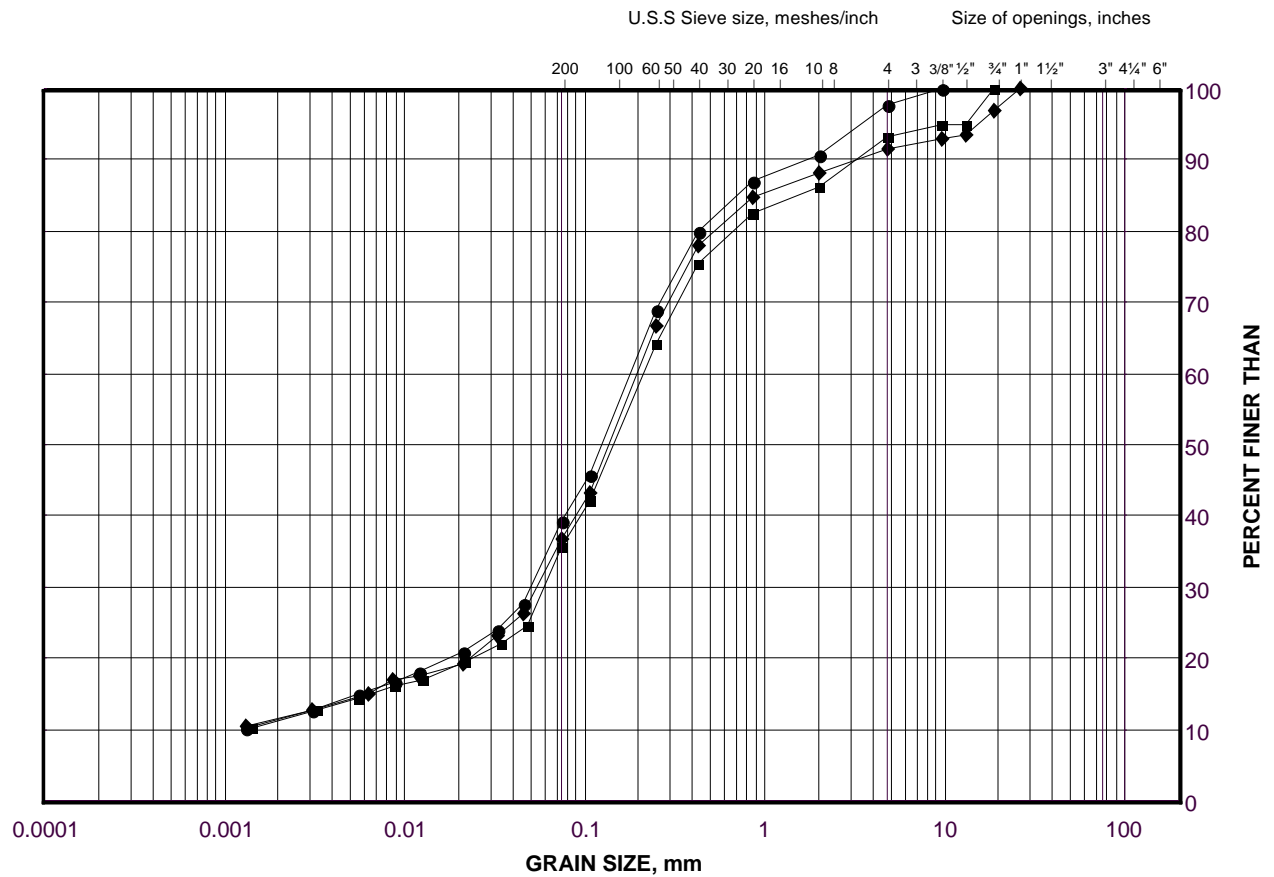
Golder Associates

Date: 02-Jul-15

GRAIN SIZE DISTRIBUTION

Lower Silty Sand (Till)

FIGURE B5



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

LEGEND

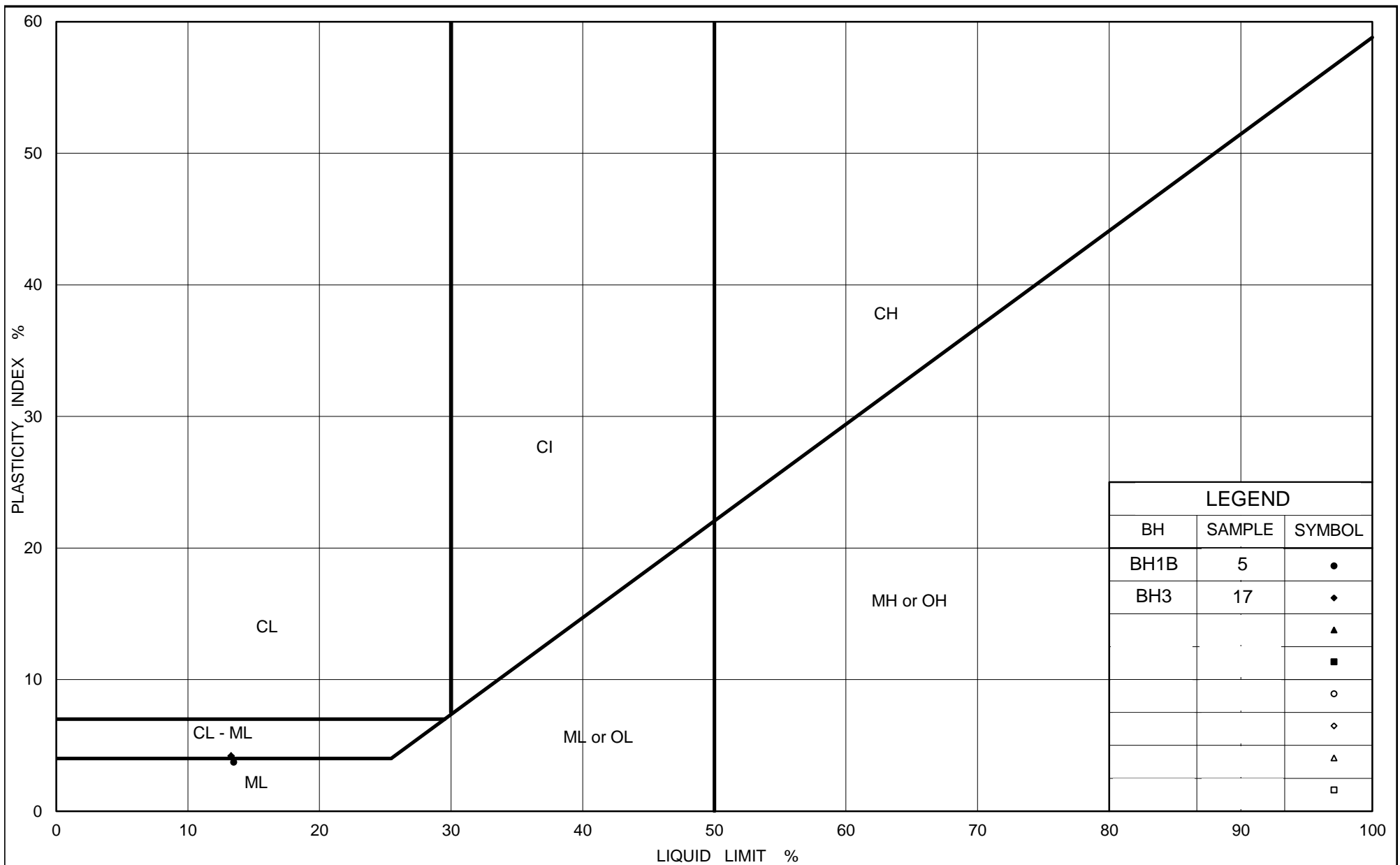
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	BH2	16	273.5
■	BH3	17	275.0
◆	BH1B	5	273.2

Project Number: 14-1111-0002

Checked By: CN

Golder Associates

Date: 10-Jul-15



PLASTICITY CHART
Lower Silty Sand (Till)

Figure No. B6

Project No. 14-1111-0002

Checked By: CN



APPENDIX C

Record of Boreholes and Laboratory Test Results – Golder 2000 Investigation

PROJECT 001-1143F			RECORD OF BOREHOLE No B2-1			1 OF 1			METRIC											
W.P. 30-95-00			LOCATION N 4899694.3; E 291491.8			ORIGINATED BY AZ														
DIST SW HWY 400			BOREHOLE TYPE 108mm DIAMETER SOLID STEM AUGERS			COMPILED BY LCC														
DATUM Geodetic			DATE Oct.23/2000			CHECKED BY ASP														
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES				ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED			WATER CONTENT (%) W _p W W _L			γ	GR SA SI CL			
286.2	GROUND SURFACE																			
0.0	Silty Sand, some organics, trace clay and gravel (Fill) Compact Moist Brown/black		1	SS	18				286											
284.7									285											
1.5	Topsoil		2	SS	13															
283.9									284											
2.3	Silty Clay, trace sand and gravel (Till) Very stiff Brown Moist		3	SS	21															
283.1									283											
3.1	Silty Sand to Sand and Silt, trace clay and gravel Loose to very dense Brown Moist to wet		4	SS	9															
			5	SS	51				282											
			6	SS	44				281											
									280											
279.6			7	SS	82															
6.6	Clayey Silt, some sand, trace gravel. (Till) Hard Grey Moist		8	SS	108/15				279											
									278											
									277											
			9	SS	100/06															
									276											
275.2			10	SS	100/15															
11.0	END OF BOREHOLE																			
	Note: Water level in open borehole at 2.1m depth (Elev.284.1m) on completion of drilling operations.																			

ON_MOT_0011143F.GPJ_ON_MOT.GDT 14/1/02

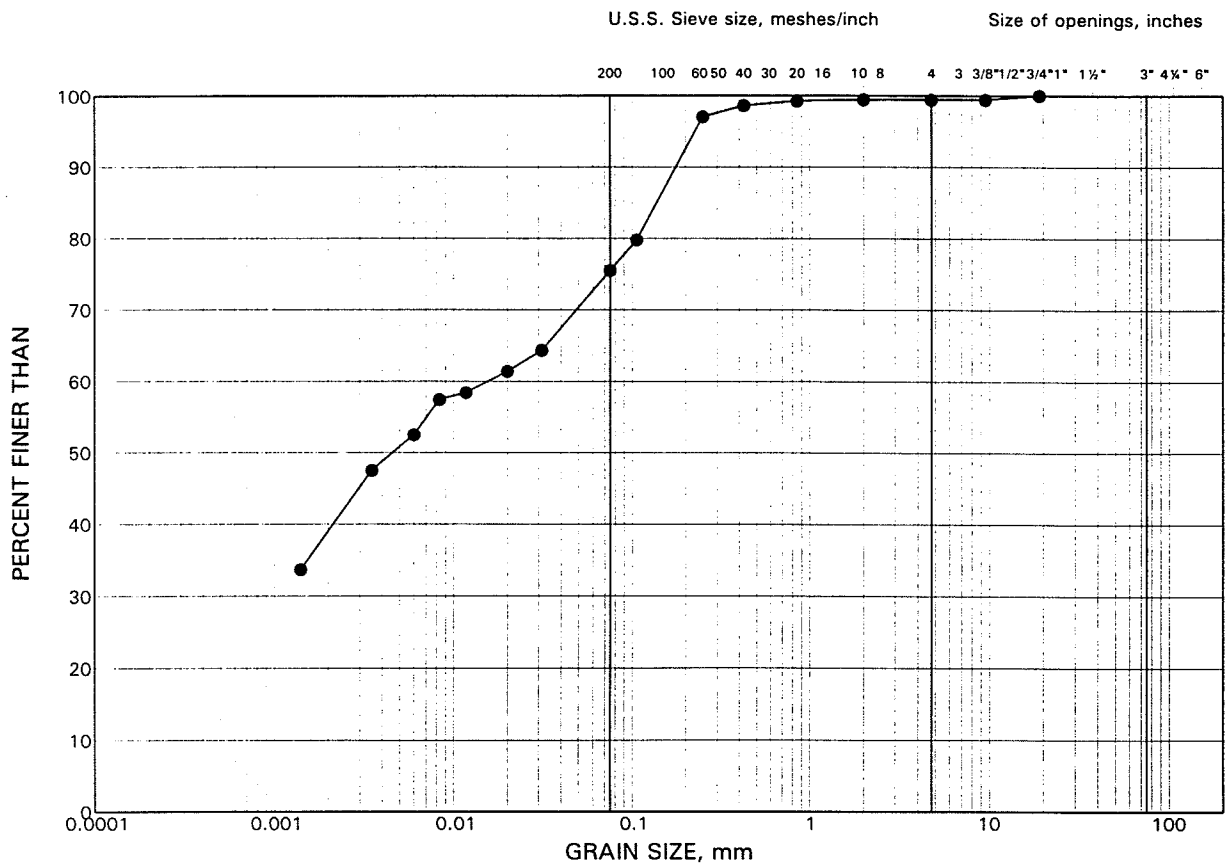
PROJECT 001-1143F			RECORD OF BOREHOLE No B2-2			1 OF 1			METRIC						
W.P. 30-95-00			LOCATION N 4899636.5; E 291463.6			ORIGINATED BY PKS									
DIST SW HWY 400			BOREHOLE TYPE 108mm DIAMETER SOLID STEM AUGERS			COMPILED BY LCC									
DATUM Geodetic			DATE Oct.26/2000			CHECKED BY ASP									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED							
285.9	GROUND SURFACE														
0.0	Sand and Gravel, trace silt, trace clay pockets and asphalt pieces (Fill) Loose to compact Moist Brown		1	SS	7										
			2	SS	25										
			3	SS	16										
283.8	Topsoil														
283.5															
2.4	Silty Clay, trace sand and gravel (Till) Stiff to very stiff Moist Grey		4	SS	15										
			5	SS	15										
			6	SS	23										
			7	SS	15										
279.9															
6.0	Clayey Silt, trace to some sand and gravel (Till) Hard Moist Grey		8	SS	121										
			9	SS	108										
277.1															
8.8	Silty Sand, trace clay and gravel (Till) Very dense Moist Grey														
276.4			10	SS	100/23										
9.5	END OF BOREHOLE														
	Notes: 1. Water level in open borehole at 6m depth (Elev.279.9m) on completion of drilling operations. 2. Water level in piezometer at 1.9m depth (Elev.284.0m) on January 19, 2001, and at 1.1m depth (Elev.284.8m) on March 15, 2001.														

ON_MOT_0011143F.GPJ ON_MOT.GDT 14/1/02

GRAIN SIZE DISTRIBUTION TEST RESULT

Silty Clay Till

FIGURE 1



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

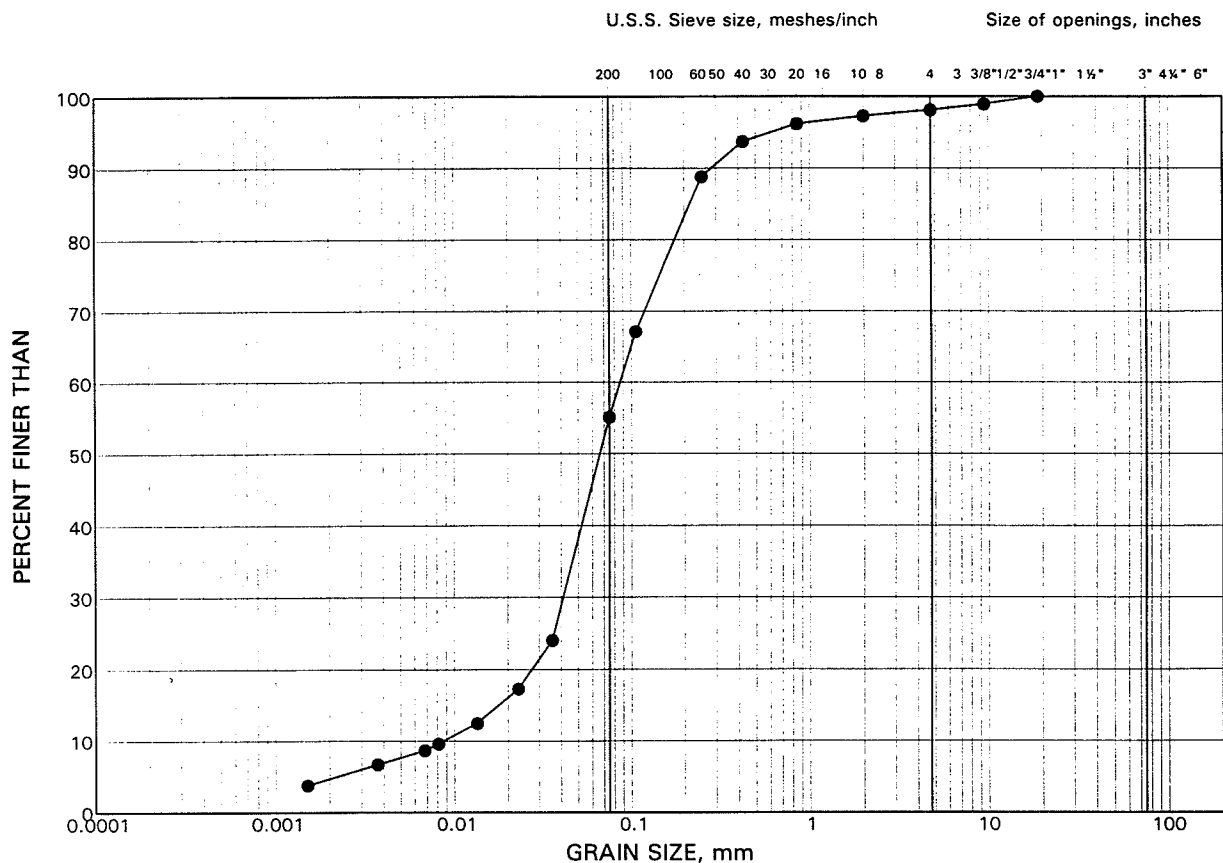
LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
•	B2-2	5	282.5

GRAIN SIZE DISTRIBUTION TEST RESULT

Sand and Silt

FIGURE 2



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
•	B2-1	5	282.1

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