



September 9, 2016

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

INNISFIL BEACH ROAD OVERPASS, SITE NO. 30-210
HIGHWAY 400 WIDENING
FROM 1 KM SOUTH OF HIGHWAY 89 TO JUNCTION OF HIGHWAY 11
MINISTRY OF TRANSPORTATION, ONTARIO
G.W.P. 06-20016

Submitted to:
AECOM
30 Leek Crescent, 4th Floor
Richmond Hill, Ontario
L4B 4N4



FINAL REPORT

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PART A

**PRELIMINARY FOUNDATION INVESTIGATION REPORT
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by AECOM (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 Innisfil Beach Road Overpass 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.

This report addresses the proposed replacement of the Innisfil Beach Road Overpass (MTO Structure Site No. 30-210) and the associated approach embankments only.

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 Innisfil Beach Road Overpass replacement is contained in Section 5.8 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.

2.0 SITE DESCRIPTION

The Innisfil Beach Road Overpass, which is part of the Highway 400-Innisfil Beach Road (Simcoe Road 21) Interchange, is located approximately 9.6 km north of Highway 89 Interchange, in the Town of Innisfil, in the County of Simcoe. The existing Innisfil Beach Road Overpass is an about 35 m wide by 28.5 m long single-span structure supported on spread footings.

The Innisfil Beach Road-Highway 400 Interchange is located in the Innisfil Heights strategic settlement employment area. The overall surface topography in the vicinity of the site is relatively flat and consists of rural farmland to the west of Highway 400 and an industrial and residential area to the east. The natural ground surface at the site ranges between approximately Elevations 303 m and 306 m. At this structure site, Highway 400 has been constructed on an approximately 5 m high embankment and has an existing grade at about Elevation 308 m. The Innisfil Beach Road surface is near the original ground surface, with the existing grade varying between about Elevations 302.5 m and 303 m.

3.0 INVESTIGATION PROCEDURES

3.1 Previous Borehole Investigation

Two boreholes were advanced at this site as part of a previous Golder geotechnical investigation in 2000 (MTO, 2002) for the widening or replacement of the existing Innisfil Beach Road Overpass structure, associated with the widening of Highway 400. Borehole B4-1 was advanced on the north side of Innisfil Beach Road, east of Highway 400, to a depth of about 6.2 m below ground surface; and Borehole B4-2 was advanced south of Innisfil Beach Road, west side of Highway 400, to a depth of about 10.8 m. The borehole locations are shown on Drawing 1.

Both boreholes were advanced using 108 mm diameter 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 (ASTM D1586).



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The water level in the open boreholes was observed during and following the drilling operations and a piezometer was installed in Borehole B4-2 to allow monitoring of the groundwater level at the site.

The borehole locations in MTM NAD83 northing and easting coordinates, ground surface elevations reference to Geodetic datum and drilled depths are summarized below.

Borehole Number	Location (MTM NAD83)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m)	Easting (m)		
BH4-1	4,905,036.5	290,509.0	302.7	6.2
BH4-2	4,904,989.7	290,437.7	305.6	10.8

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

As delineated in *The Physiography of Southern Ontario*¹, 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 Uplands. 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, as well as between about 1 km north of Highway 89 to about Essa Road. The Simcoe Lowlands covers the area from south of Line 13 to approximately 1 km north of Highway 89 and from about Essa Road to just north of Anne Street. The Simcoe Uplands extends from just north of Anne Street to beyond the northern limit of this project site.

The surficial soils in the western portion of the Peterborough Drumlin Field, which encompasses the Innisfil Beach Road site, consist primarily of sandy till deposits and sand to sand and gravel deposits. Deposits of silt, clay or peat may also be found in the low-lying areas between drumlins and eskers.

Along Highway 400, the Simcoe Lowlands include: the Holland River valley; the lowlands of the Lake Simcoe basin to the east; the lowlands of the Nottawasaga basin to the west, which includes Innisfil Creek and the Nottawasaga River to the south and west of the project limits, respectively. The Lake Simcoe and Nottawasaga basins are connected by a flat floored valley through Barrie which extends from the shores of Kempenfelt Bay west generally along Highway 90. The Simcoe Lowlands are generally characterized by deep deposits of deltaic or lacustrine silts, sands and clays associated with glacial Lake Algonquin.

The Simcoe Uplands consist of till plains and ancient shorelines. The till deposits range from clayey to silty and generally become more sandy and containing more boulders in the north. The low-lying areas of this region may also contain shallow deposits of sand and gravel associated with former glacial lake shorelines.

4.2 Subsurface Conditions

The Record of Borehole sheets and laboratory testing results from the previous investigation are presented in Appendix A. The interpreted stratigraphic profile and cross-sections are shown on Drawings 1 and 2.

¹ 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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The results of the in situ field tests (i.e. SPT 'N'-values) carried out during the previous 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.

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 fill and/or topsoil underlain by a glacial till deposit comprised of clayey silt with sand 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 Topsoil

A 0.2 m and 0.5 m thick layer of topsoil was encountered in Boreholes BH4-1 and BH4-2, respectively. The topsoil layer in Borehole B4-2 is fill material, having been spread over an underlying fill deposit.

4.2.2 Clayey Silt with Sand Fill

A 1.7 m thick deposit of fill was encountered at Elevation 305.1 m below the topsoil in Borehole B4-2. The fill consists of an upper 1.0 m thick layer of clayey silt with sand trace organics, underlain by a lower 0.7 m thick layer of silty sand.

The SPT 'N'-values measured within the fill deposit are 11 blows per 0.3 m of penetration and 27 blows per 0.3 m of penetration, suggesting a stiff consistency and indicating a compact relative density for the clayey silt with sand fill and silty sand fill, respectively.

The natural water content measured on a sample of the silty sand fill is about 8 per cent.

4.2.3 Clayey Silt with Sand Till

A 6.0 m and 8.6 m thick till deposit comprised of clayey silt with sand was encountered below the topsoil in Borehole BH4-1 and below the fill in Borehole B4-2 at Elevations 302.5 m and 303.4 m, respectively. A 0.6 m thick pocket of silty sand was encountered within the clayey silt with sand till deposit in Borehole B4-1 at Elevation 300.6 m. Silty sand till was also encountered in a split-spoon sample in Borehole B4-2 at about Elevation 299.5 m. Cobbles were inferred within the till deposit at depths between 5.5 m and 5.8 m in Borehole B4-1 and at a depth of 3.7 m in Borehole B4-2, corresponding to Elevations 296.9 m, 297.2 m and 301.9 m, respectively.

The SPT 'N'-values measured within the cohesive till deposit generally range from 120 blows per 0.3 m of penetration to 151 blows per 0.15 m of penetration, suggesting a hard consistency. An SPT 'N'-value of 48 blows per 0.3 m of penetration was measured at the top of the till deposit below the fill in Borehole B4-2



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suggesting a hard consistency. The SPT 'N'-values measured within the silty sand pocket and the zone of silty sand till are 103 blows per 0.15 m of penetration and 43 blows per 0.3 m of penetration, indicating a very dense and dense relative density, respectively.

The natural water content measured on samples of the clayey silt with sand till deposit ranges from about 6 per cent to 9 per cent.

The result of a grain size distribution test completed on a sample of the clayey silt with sand till from Borehole B4-1 is shown on Figure 1 in Appendix A.

Atterberg limits test carried out on three samples of clayey silt with sand till deposit measured liquid limits between about 13 per cent and 14 per cent, plastic limits between about 10 per cent and 11 per cent and plastic indices between about 3 per cent and 4 per cent, indicating that the till deposit is comprised of clayey silt of low plasticity.

4.3 Groundwater Conditions

In general the soil samples retrieved in the two boreholes were moist and both boreholes were dry upon completion of drilling.

A standpipe piezometer was installed in Borehole BH4-2 located on the south-west quadrant of the Highway 400-Innisfil Beach Road Interchange, and the groundwater level in the standpipe piezometer was measured at a depth of 7.7 m below ground surface, Elevation 297.9 m, on March 15, 2001.

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

This report was prepared by Ms. Madison Kennedy, B.A.Sc., a member of the geotechnical engineering group, and was reviewed by Mr. Christopher Ng, P.Eng., a senior geotechnical engineer and Associate of Golder. Mr. Jorge M. A. Costa, P.Eng., a Senior Consultant with Golder and Designated MTO Foundations Contact, conducted an independent quality control review of this report.

GOLDER ASSOCIATES LTD.

Madison C. Kennedy, B.A.Sc.
Geotechnical Engineering Group



Christopher Ng, P.Eng.
Senior Geotechnical Engineer, Associate



Jorge M. A. Costa, P.Eng.
Designated MTO Foundations Contact, Senior Consultant

MCK/CN/JMAC/mck

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PART B

**PRELIMINARY FOUNDATION DESIGN REPORT
INNISFIL BEACH ROAD OVERPASS – SITE NO. 30-210
HIGHWAY 400 WIDENING
FROM 1 KM SOUTH OF HIGHWAY 89 TO JUNCTION OF HIGHWAY 11
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6.0 DISCUSSION AND PRELIMINARY ENGINEERING RECOMMENDATIONS

This section of the report provides preliminary foundation design recommendations for the proposed replacement of the Highway 400-Innisfil Beach Road Overpass (MTO Structure Site No. 30-210). These preliminary recommendations are based on interpretation of the factual data obtained from the boreholes advanced during a 2000 subsurface investigation. The discussion and recommendations presented are intended to provide the designer with sufficient information to assess the feasible foundation alternatives and carry out the design of the structure foundations, as may be required. The foundation investigation report, discussion and recommendations are intended for the use of the Ministry of Transportation and shall not be used or relied upon for any other purpose or by any other parties, including the construction or design-build contractor. The contractor must make their own interpretation based on the factual data in Part A of the report. Where comments are made on construction, they are provided to highlight those aspects that could affect the future detail design of the project and for which special provisions may be required in the Contract Documents. Those requiring information on the aspects of construction must 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 Highway 400-Innisfil Beach Road Overpass in the Town of Innisfil. It is understood that the Innisfil Beach Road Overpass will consist of a single-span, pre-cast girder bridge with a 34 m span length and a 65.4 m wide section of girders to accommodate the northbound lanes (NBL) and southbound lanes (SBL).

Based on the General Arrangement (GA) Drawing provided by AECOM on May 11, 2016, the grade of the proposed Overpass is about Elevation 308 m. In comparison, the proposed grade for Innisfil Beach Road is between about Elevation 300.5 m to 301 m.

6.2 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the Canadian Highway Bridge Design Code (CHBDC 2014) and its Commentary, the proposed overpass structure and foundation system may be classified as having large traffic volumes and its performance as having potential impacts on other transportation corridors, hence having a “typical consequence level” associated with exceeding limits states design. In addition, given the limited level of foundation investigation completed to date as presented in Sections 3.0 and 4.0, in comparison to the degree of site understanding in Section 6.5 of *CHBDC (2014)*, the level of confidence for design is considered to be a “low degree of site and prediction model understanding.” Accordingly, the appropriate corresponding ULS and SLS consequence factor, Ψ , and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs} , from Tables 6.1 and 6.2 of the CHBDC have been used for design, as indicated in Sections 6.4 to 6.8 below.



6.3 Foundation Options

As part of the future widening of Highway 400 in Simcoe County, the existing Innisfil Beach Road Overpass will require replacement. According to the available information, the existing single-span NBL and SBL structure is supported on spread footings that are founded at approximately Elevation 300.5 m. Highway 400 is proposed to be widened by approximately 30 m to the west and 2 m to the east of the existing alignment, with the centreline re-positioned to the west of the existing highway, and maintaining its grade at approximately Elevation 308 m. The proposed Innisfil Beach Road grade will be lowered by about 2 m to accommodate the minimum vertical clearance requirements. Based on the proposed overpass geometry and the subsurface conditions at this site, both shallow foundation and deep foundation options have been considered for support of the abutments for the proposed structure. A summary of the advantages and disadvantages associated with each option is provided below, and a comparison of the foundation alternative for a replacement structure is presented in Table 1.

- **Shallow foundations – spread/strip footings:** Shallow foundations comprised of spread or strip footings, founded on the hard clayey silt with sand till deposit, are feasible for support of the new abutments, although this foundation type will preclude the use of integral abutments.
- **Deep foundations – driven steel H-piles or pipe (tube) piles:** Driven steel H-piles or steel pipe (tube) piles are feasible for support of the abutments, and would permit design of conventional abutments, semi-integral abutments (for H-piles and pipe piles) or integral abutments (for H-piles only). Relatively short pile lengths would be required for integral abutment design.
- **Deep foundations – drilled shaft (caissons):** Drilled shafts (caissons) are considered feasible for the support of the abutments; 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 temporary liners may 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, although steel H-pile foundations are preferred from a foundations perspective as they would permit integral abutments design.

6.4 Shallow Foundations

6.4.1 Founding Elevation

For the support of the new abutments spread/strip footings should be founded on the hard clayey silt with sand till deposit, 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:

Foundation Element	Highest Founding Elevation (m)	Founding Soil
North and South Abutment	299.0	Hard Clayey Silt with Sand Till

It should be noted that the highest founding elevation takes into consideration the proposed Innisfil Beach Road grade and the depth of frost penetration.



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6.4.2 Factored Geotechnical Axial Resistances

The following factored ultimate and serviceability geotechnical resistances (at ULS and SLS for 25 mm of settlement, respectively) may be used for preliminary design of spread/strip footing founded on the properly prepared clayey silt with sand till, or on a compacted Granular 'A' pad having a minimum thickness of 1 m:

Foundation Alternative	Factored Ultimate Geotechnical Axial Resistance ¹ (at ULS) (kPa)	Factored Serviceability Geotechnical Resistance ¹ (at SLS) for 25 mm of Settlement (kPa)
Footing on properly prepared hard clayey silt with sand till	700	450
Footing on minimum 1 m thick compacted Granular 'A' pad	750	350

Note: 1. The factored geotechnical resistances given above are estimated for a 3 m wide spread/strip footing.

The preliminary factored geotechnical resistances provided above are given for loads that will 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 6.10.4 of the *CHBDC (2014)*.

6.4.3 Resistance to Lateral Loads

Resistance to lateral forces/sliding resistance between cast-in-place concrete footings and the founding soils should be calculated in accordance with Section 6.10.5 of the *CHBDC (2014)*. The following presents the coefficient of friction, $\tan \phi'$, for the interface between the concrete footing and till deposit or Granular 'A' pad:

Founding Material	Coefficient of Friction ($\tan \phi'$)
Cast-in-place concrete footing on native hard clayey silt with sand till	0.45
Cast-in-place concrete footing on compacted Granular 'A' pad	0.60

6.4.4 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 from and perpendicular to 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.



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

6.5.1 Founding Elevation

The abutments for the replacement structure may be supported on steel H-piles or pipe piles driven to found within the hard clayey silt with sand till deposit.

Based on the GA Drawing, integral abutments are proposed to be adopted for the design of the replacement structure with the abutments “perched” within the Innisfil Beach Road embankments, with the underside of the new pile caps at approximately Elevation 305 m. The following pile tip elevations are recommended for preliminary design, based on approximately 3 m of penetration into the “100-blow” hard clayey silt with sand till deposit.

Foundation Element	Proposed Innisfil Beach Road Grade Elevation (m)	Estimated Design Tip Elevation (m)	Founding Soil at Tip Elevation
North and South Abutment	300.5	295.0	Clayey silt with sand till

Based on the above elevations, the proposed piles are estimated to be approximately 10 m long at the east and west abutment.

As discussed in Section 4.2.3, cobbles are inferred to be present within the clayey silt with 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 hard till deposit 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.

6.5.2 Factored Geotechnical Axial Resistances

The factored ultimate and serviceability geotechnical axial resistances (at ULS and SLS for 25 mm of settlement, respectively) 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 Length of Driven Pile (m)	Factored Ultimate Geotechnical Axial Resistance (at ULS)¹ (kN)	Factored Serviceability Geotechnical Resistance (at SLS) for 25 mm of Settlement² (kN)
HP 310x110	5.5	1,050	N/A
324 mm OD Pipe Pile	5.5	900	N/A

Note: 1. The settlement at the factored ultimate geotechnical axial resistance is estimated to be less than 25 mm.

2. The factored serviceability geotechnical resistance (at SLS) for 25 mm of settlement will be greater than the factored ultimate geotechnical axial resistance (at ULS) and as such, the SLS condition does not apply.



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Pile installation should be in accordance with OPSS 903 (Deep Foundations). The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile; the criteria must therefore be established at the time of construction after the piling equipment is known. The pile capacity should then be verified in the field by the use of the Hiley formula (MTO's Standard Drawing SS103-11, *Pile Driving Control*) and/or Pile Dynamic Analyzer (PDA) testing during pile installation on selected piles to confirm the design capacity.

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

6.5.3 Frost Protection

All pile caps 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 from and perpendicular to the face of the abutment slope to the edge of the underside of the pile cap.

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

6.6 Drilled Shaft (Caisson) Foundations

6.6.1 Founding Elevations

Drilled shafts (caissons) founded within the hard clayey silt with sand till deposit may be considered for support of the abutments for the proposed replacement structure. The following drilled shaft founding elevations may be used for preliminary design purposes, assuming about 3 shaft diameters into "100-blow" soil below the depth of frost penetration:

Foundation Element	Proposed Innisfil Beach Road Grade Elevation (m)	Drilled Shaft Diameter (m)	Estimated Design Tip Elevation (m)	Founding Soil at Tip Elevation
North and South Abutment	300.5	0.9	296.0	Clayey silt with sand till
		1.2	295.0	Clayey silt with sand till

If drilled shaft foundations are adopted, a temporary liner should be utilized to support the overburden soils during construction 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.



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6.6.2 Geotechnical Axial Resistance/Reaction

The following factored ultimate and serviceability geotechnical axial resistances (at ULS and SLS for 25 mm of settlement, respectively) may be used for design of drilled shaft (caisson) foundations:

Drilled Shaft Diameter (m)	Factored Ultimate Geotechnical Axial Resistance (at ULS) ¹ (kN)	Factored Serviceability Geotechnical Resistance (at SLS) for 25 mm of Settlement ² (kN)
0.9	2,800	N/A
1.2	4,500	N/A

Note: 1. The settlement at the factored ultimate geotechnical axial resistance is estimated to be less than 25 mm.

2. The factored serviceability geotechnical resistance (at SLS) for 25 mm of settlement will be greater than the factored ultimate geotechnical axial resistance (at ULS) and as such, the SLS condition does not apply.

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

6.6.3 Frost Protection

All pile caps 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 from and perpendicular to the face of the abutment slope to the edge of the underside of the pile cap.

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

6.7 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 (Walls, Retaining, Backfill, Minimum Granular Requirement).



PRELIMINARY FOUNDATION REPORT - HIGHWAY 400 INNISFIL BEACH ROAD OVERPASS

- 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 (2014)* Section 6.12.3 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 2014*). 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 2014*). The pressures are based on the proposed embankment fill material and the following parameters (unfactored) may be used:

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 C6.12.1 and Table C6.6 of the *Commentary to the CHBDC (2014)*.

6.8 Approach Embankments

6.8.1 Subgrade Preparation and Embankment Construction

Based on the existing topographic information, the existing Highway 400 embankment side slopes at the Innisfil Beach Road approach embankment are inclined at about 2 horizontal to 1 vertical (2H:1V). For the proposed grade lowering of Innisfil Beach Road and widening of the Highway 400 embankments, the new side slope should also be constructed 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 OPSP 208.010 (Benching of Earth Slopes) to integrate the new fill into the existing slope fill.

6.8.2 Approach Embankment Stability and Factored Settlement

Given the acceptable/satisfactory performance of the existing embankments and that the native soils are predominantly comprised of a hard till deposit, stability issues are not anticipated within the limits of the approach embankments widening. The factored settlement associated with the widening is estimated to be about 50 mm. Given that the native till deposit is heavily over-consolidated, it is expected that the majority of the settlement will occur during and shortly after construction of the embankment.

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).



6.9 Construction Considerations

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

6.9.1 Open-Cut Excavations

The construction of new spread/strip footings and/or pile caps will require excavations to depths up to about 4.5 m below the existing Innisfil Beach Road grade and will be made through the native hard clayey silt with sand till deposit. The native hard clayey silt with sand till deposit is classified as a Type 1 soil, according to the Occupational Health and Safety Act (OHSA) and, as such, temporary open-cut excavations above the groundwater level may be made with side slopes within 1.2 m from the excavation bottom and no steeper than 1H:1V. However, localized sloughing of the cut slopes/walls may occur where the excavation intercepts granular/loose/ wet interlayers or pockets.

6.9.2 Temporary Protection Systems

Temporary protection systems will be required to facilitate the removal of the existing bridge foundations and construction of the abutments. 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.

6.9.3 Obstructions

It should be noted that obstructions (inferred as cobbles) were encountered within the till deposit in the area of the proposed abutments. 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 detail design to determine the extent of such obstructions.

6.9.4 Control of Ground and Groundwater for Drilled Shaft (Caisson) Construction

As noted in Section 6.6.1, although one borehole was noted to be dry upon completion of drilling and the other borehole indicated the groundwater level was encountered at a depth of about 7 m below ground surface (about Elevation 298 m) running or flowing soil from the native till deposit could occur during or after drilling the drilled shafts (caissons) and heave could occur at the caisson base. If drilled shaft foundations are adopted, temporary liners should be used to support the overburden soils. Balancing groundwater pressures during construction by utilizing a head of water or bentonite drilling slurry inside the temporary liner may be required, and should be assessed at the detail design stage. In addition, placement of concrete by tremie methods would be required.

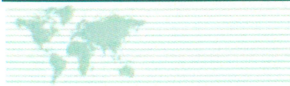


6.10 Recommendations for Future Work During Detail Design

During detail design, it is recommended that additional site investigation and field testing be carried out at/within the footprint of the abutment foundations to a sufficient depth below the ground surface to determine the presence and extent of cobbles and boulders and to allow for design of deep foundations, both to a lower tip elevation and higher factored geotechnical resistances if required.

In addition, although the till deposit has been classified as cohesive based on the limited laboratory testing data, the test results suggest that the till deposit is borderline material ranging between a cohesive and non-cohesive soil of varying composition. As such, it is recommended that sufficient laboratory testing be carried out on samples of the native till deposit to allow for the confirmation of the soil classification of the deposit, which will aid in the assessment of soil behaviour during detail design.

Further, it should be noted that the 2000 investigation was carried out using manual hammers during split-spoon sampling and as such, the “N”-values as presented on the Record of Boreholes are anticipated to be higher than those that would be obtained if automatic hammers were used. As a result, the factored geotechnical resistances and the practical limits for which driven piles can penetrate into the till deposit will need to be reassessed during detail design.



PRELIMINARY FOUNDATION REPORT - HIGHWAY 400 INNISFIL BEACH ROAD OVERPASS

7.0 CLOSURE

This report was prepared by Ms. Madison Kennedy, B.A.Sc., a member of the geotechnical engineering group, and was reviewed by Mr. Christopher Ng, P.Eng., a senior geotechnical engineer and Associate of Golder. Mr. Jorge M. A. Costa, P.Eng., a Senior Consultant with Golder and Designated MTO Foundations Contact, conducted an independent quality control review of this report.

GOLDER ASSOCIATES LTD.

Madison C. Kennedy, B.A.Sc.
Geotechnical Engineering Group



Christopher Ng, P.Eng.
Senior Geotechnical Engineer, Associate



Jorge M. A. Costa, P.Eng.
Designated MTO Foundations Contact, Senior Consultant

MCK/CN/JMAC/mck

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PRELIMINARY FOUNDATION REPORT - HIGHWAY 400 INNISFIL BEACH ROAD OVERPASS

REFERENCES

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

Canadian Standards Association (CSA), 2014. *Canadian Highway Bridge Design Code and Commentary on CAN/CSA S6-14*. CSA Special Publication, S6.1-14.

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

Ministry of Transportation, Ontario. 2002. *Preliminary Foundation Investigation and Design Report Innisfil Beach Road Overpass, Structure Site 30-210; Highway 400 Widening from 1 km South of Highway 89 to Highway 11, G.W.P. 30-95-00*, GEOCRE No. 31D00-468, prepared by Golder Associates Ltd.

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)

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 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



TABLES



PRELIMINARY FOUNDATION REPORT - HIGHWAY 400 INNISFIL BEACH ROAD OVERPASS

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 the support of new abutments 	<ul style="list-style-type: none"> Conventional excavation and construction techniques. Lower cost compared to deep foundations 	<ul style="list-style-type: none"> Requires larger footing excavation and disposal of a larger volume of soil compared to the excavation for a pile cap (excavation to about 3.5 m depth) Does not allow for integral abutment construction. Likely requires temporary protection system to allow for excavation/footing construction. 	<ul style="list-style-type: none"> Estimated cost is approximately \$600/m³ for construction of shallow foundations. 	<ul style="list-style-type: none"> Footing subgrade must be protected from frost penetration
Steel H-piles or pipe piles	<ul style="list-style-type: none"> Feasible for the support of new abutments with pile cap “perched” within the approach embankments 	<ul style="list-style-type: none"> Conventional construction methods for H-pile or steel pipe pile foundations. Abutment pile caps would be maintained higher than spread footings, thus reducing or eliminating the depth of excavation and protection system requirements. Steel H-piles allow for integral abutment configuration. 	<ul style="list-style-type: none"> Piles may refuse above design tip elevation due to the hard till deposit, especially pipe piles which have a larger displacement base. Pipe piles not readily accepted for integral abutment construction; allow for semi-integral abutment configuration. 	<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.
Drilled Shaft (Caissons)	<ul style="list-style-type: none"> Feasible but not recommended for the support of abutments 	<ul style="list-style-type: none"> Abutment pile caps could be constructed at the underside of the bridge or maintained higher than spread footings, or H-pile 	<ul style="list-style-type: none"> Temporary liners may be required, plus special measures such as tremie placement of concrete; likely not 	<ul style="list-style-type: none"> Estimated cost is approximately \$1,000/m length for caisson installation and \$600/m³ for pile cap construction; 	<ul style="list-style-type: none"> Risk of loosening and leaving in place disturbing founding soils at base of caissons.



PRELIMINARY FOUNDATION REPORT - HIGHWAY 400 INNISFIL BEACH ROAD OVERPASS

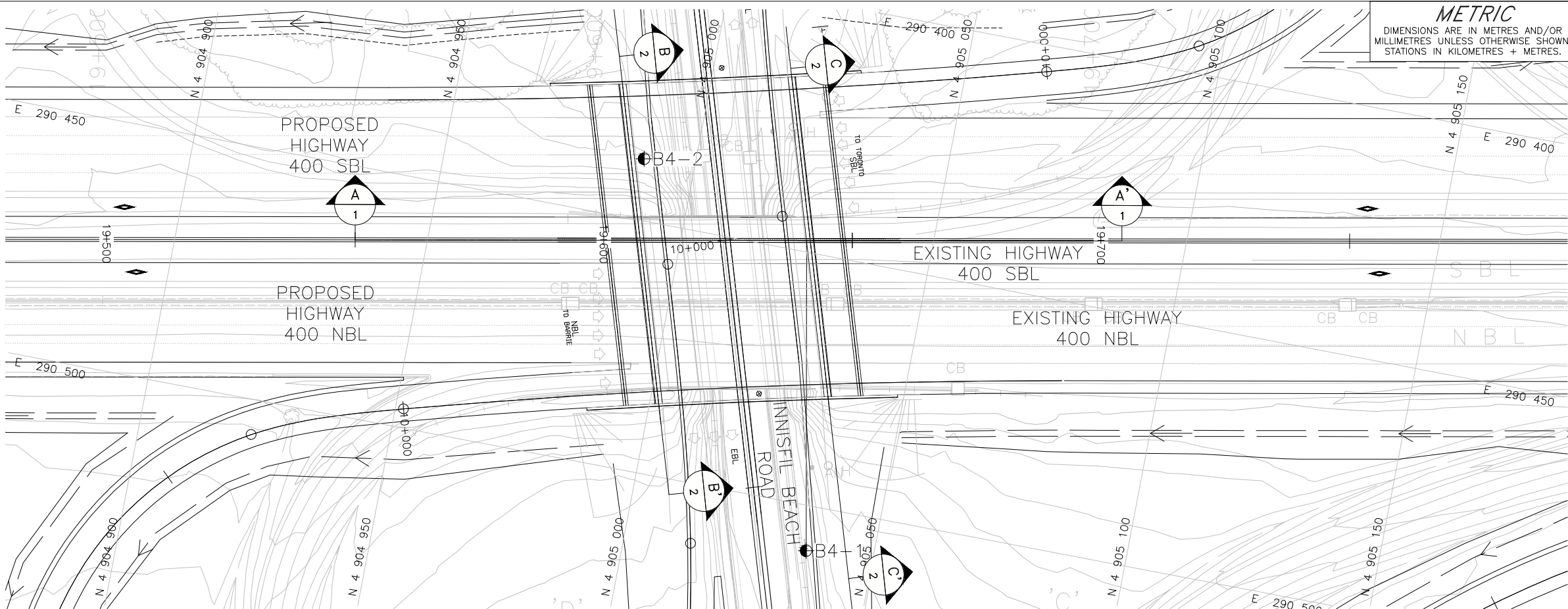
TABLE 1 – COMPARISON OF REPLACEMENT STRUCTURE FOUNDATION ALTERNATIVES

Foundation Option	Feasibility	Advantages	Disadvantages	Estimated Costs	Risk / Consequences
		<p>caps, reducing depth of excavation and protection system requirements, or caps can be constructed at level of underside of structure.</p> <ul style="list-style-type: none">• Higher capacity than for driven piles, so reduced number of deep foundation elements compared to piles.	<p>possible to inspect caisson base.</p> <ul style="list-style-type: none">• Precludes use of integral abutments.• More expensive compared to shallow foundations.	<p>the cost may be higher to account for the use of a temporary liner.</p>	

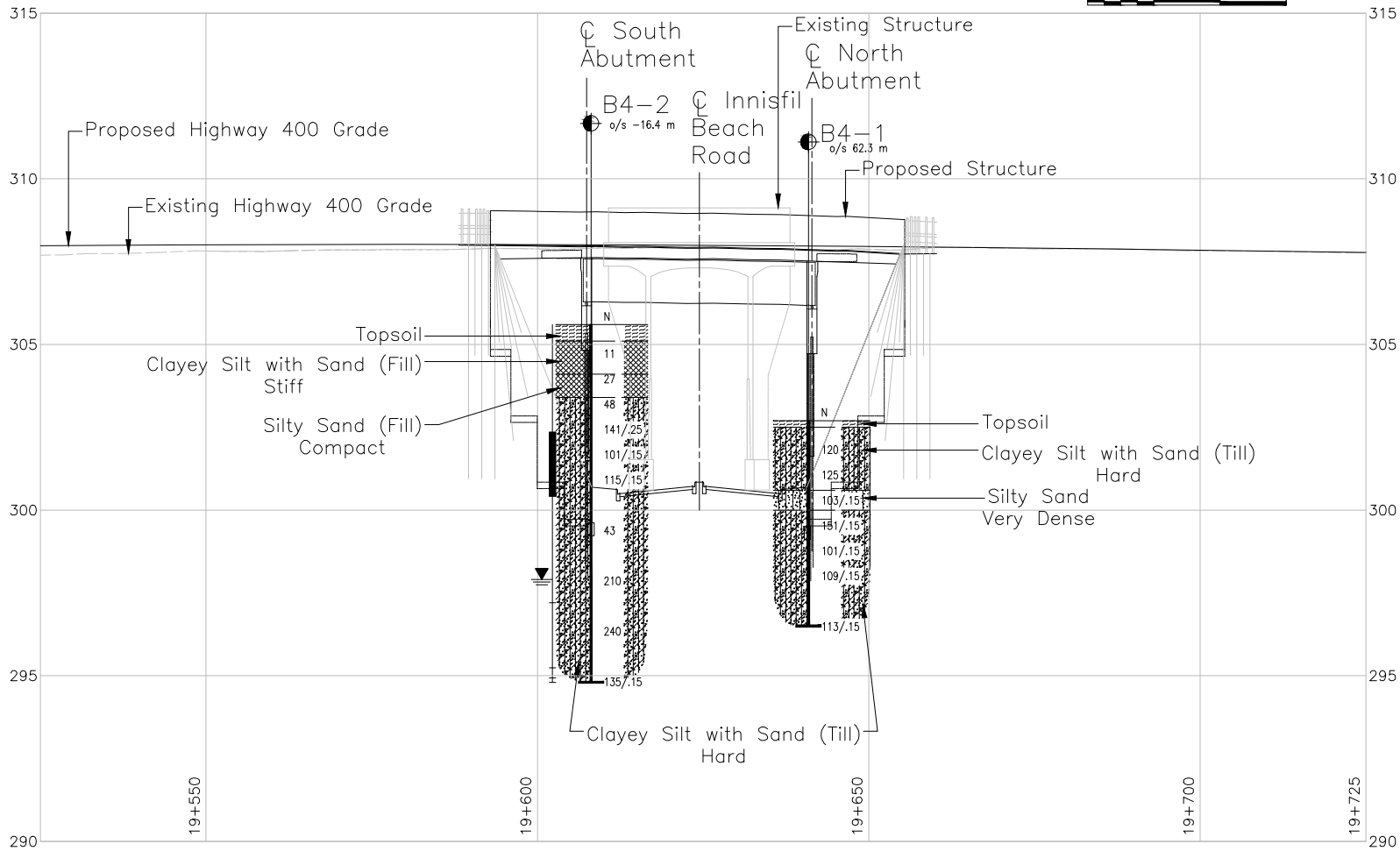


PRELIMINARY FOUNDATION REPORT - HIGHWAY 400 INNISFIL BEACH ROAD OVERPASS

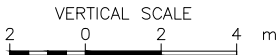
DRAWINGS



PLAN

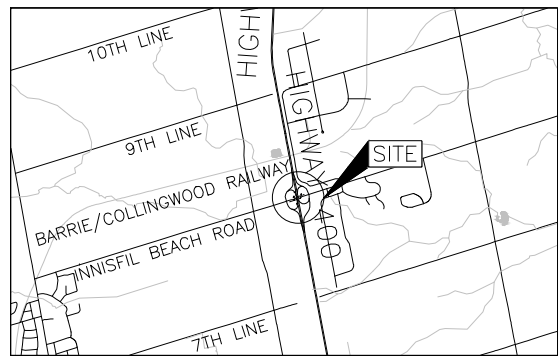


A-A' HIGHWAY 400 CENTRELINER PROFILE



CONT No.
GWP No. 06-20016

INNISFIL BEACH ROAD
HIGHWAY 400 WIDENING
BOREHOLE LOCATIONS AND SOIL STRATA



KEY PLAN
SCALE
1 0 1 2 km

LEGEND

- Borehole - Previous Investigation (Geocress No. 31D00-468)
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL in piezometer (Mar. 15, 2001)

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
B4-1	302.7	4905036.5	290509.0
B4-2	305.6	4904989.7	290437.7

NOTES

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

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

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

REFERENCE

Design plans, base plans, profile and surface data provided in digital format by AECOM, drawing file nos. "Innisfil Beach Road_Overpass_GA.dgn", "2_3-Innisfil Beach Rd_BC Rail.dwg", with associated reference files, received May 11, 2016, "X-Base_All.dwg", received January 27, 2016 and "X-Design_4th Line_Interim.dwg", received June 22, 2015.



NO.	DATE	BY	REVISION
Geocres No. 31D-655			
HWY. 400		PROJECT NO. 14-1111-0002	DIST. .
SUBM'D. MCK	CHKD. MCK	DATE: 5/25/2016	SITE: 30-210
DRAWN: MR	CHKD. CN	APPD. JMAC	DWG. 1

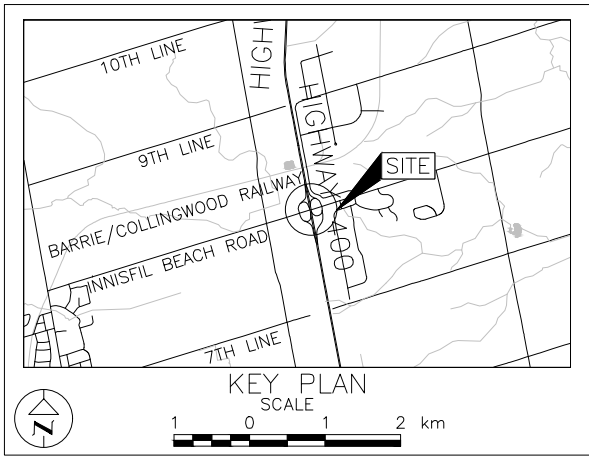
METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No. .
GWP No. 06-20016

INNISFIL BEACH ROAD
HIGHWAY 400 WIDENING

SOIL STRATA

SHEET



- LEGEND**
- Borehole - Previous Investigation (Geocress No. 31D00-468)
 - Seal
 - Piezometer
 - N Standard Penetration Test Value
 - 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
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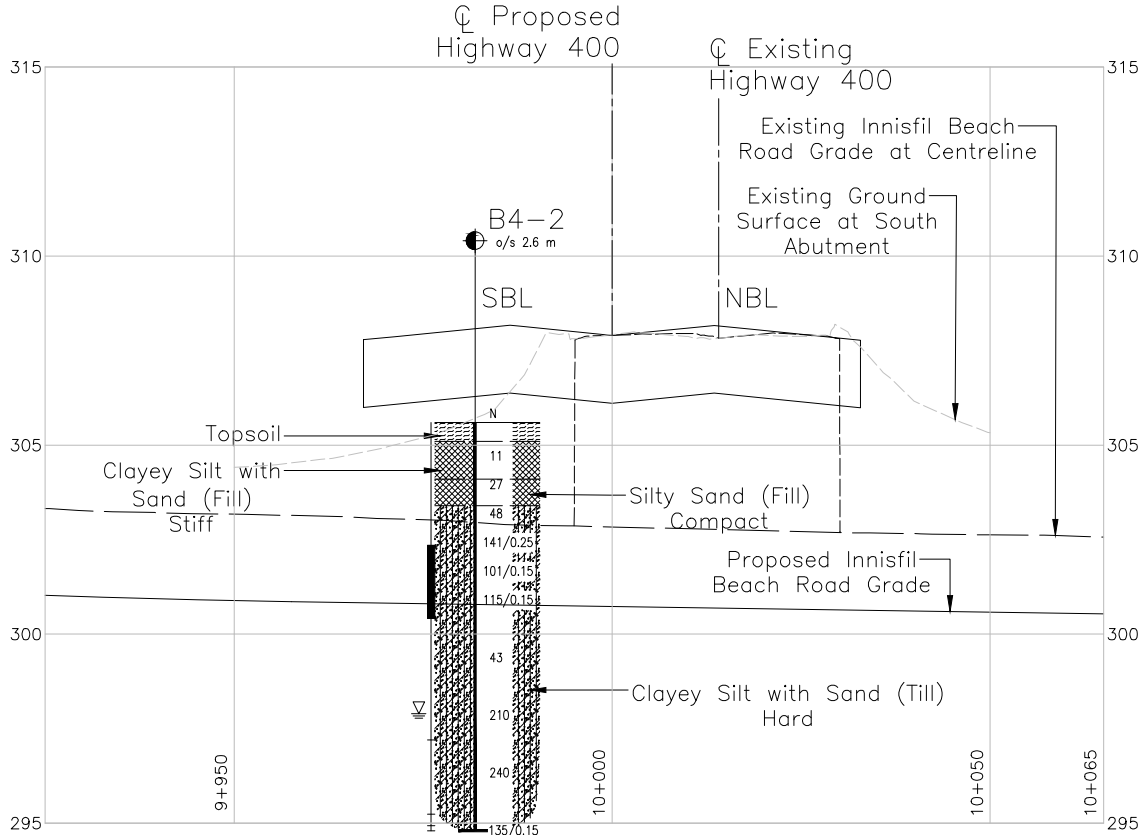
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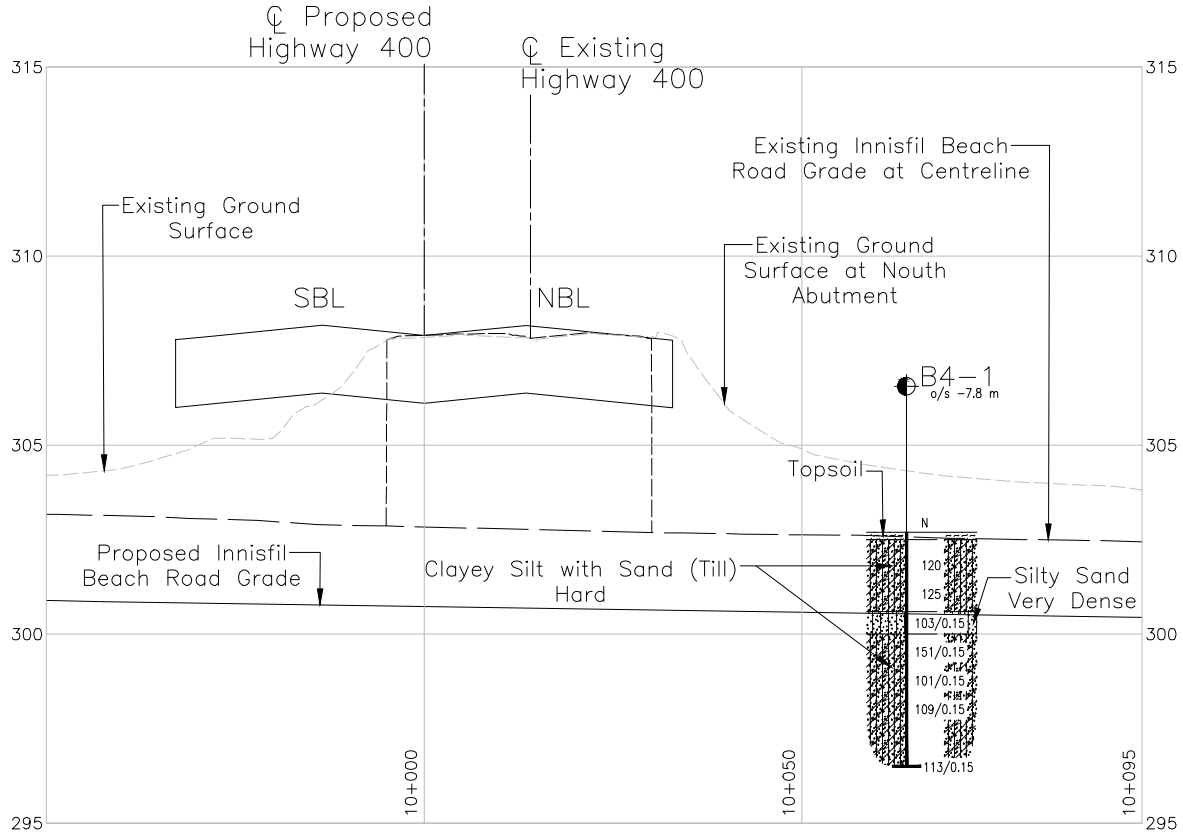
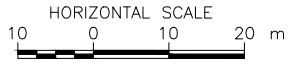
Design plans, base plans, profile and surface data provided in digital format by AECOM, drawing file nos. "Innisfil Beach Road_Overpass_GA.dgn", "2_3-Innisfil Beach Rd_BC Rail.dwg", with associated reference files, received May 11, 2016, "X-Base_All.dwg", received January 27, 2016 and "X-Design_4th Line_Interim.dwg", received June 22, 2015.



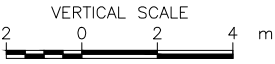
NO.	DATE	BY	REVISION
Geocres No. 31D-655			
HWY. 400		PROJECT NO. 14-1111-0002	DIST. .
SUBM'D. MCK	CHKD. MCK	DATE: 5/25/2016	SITE: 30-210
DRAWN: MR	CHKD. CN	APPD. JMAC	DWG. 2



**B-B SOUTH ABUTMENT AREA
CROSS-SECTION**



**C-C NORTH ABUTMENT AREA
CROSS-SECTION**





APPENDIX A

**Record of Boreholes and Laboratory Test Results – Golder 2000
Investigation (GEOCRES No. 31D00-468)**



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 $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

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

Notes: 1
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 001-1143F				RECORD OF BOREHOLE No B4-1				1 OF 1		METRIC					
W.P. 30-95-00				LOCATION N 4905036.5; E 290509.0				ORIGINATED BY AZ							
DIST SW HWY 400				BOREHOLE TYPE 108mm DIAMETER SOLID STEM AUGERS				COMPILED BY LCC							
DATUM Geodetic				DATE Oct.24/2000				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 X REMOULDED							
302.7	GROUND SURFACE														
0.0	Topsil														
0.2	Clayey Silt with sand, some gravel (Till) Hard Brown Moist		1	SS	120										
			2	SS	125										
300.6	Silty Sand, some gravel, trace clay Very dense Brown Dry		3	SS	103/15										
300.0	Clayey Silt with sand, some gravel (Till) Hard Brown Moist		4	SS	151/15										
2.7			5	SS	101/15										
			6	SS	109/15										
	Cobbles at 5.5m and 5.8m depth														
296.5			7	SS	103/15										
6.2	END OF BOREHOLE														
Notes: 1. Refusal to auger advance was encountered at 1.4m depth. Borehole was relocated 1m west and drilling continued. 2. Borehole dry on completion of drilling operations.															

ON_MOT 0011143F.GPJ ON_MOT.GDT 14/1/02

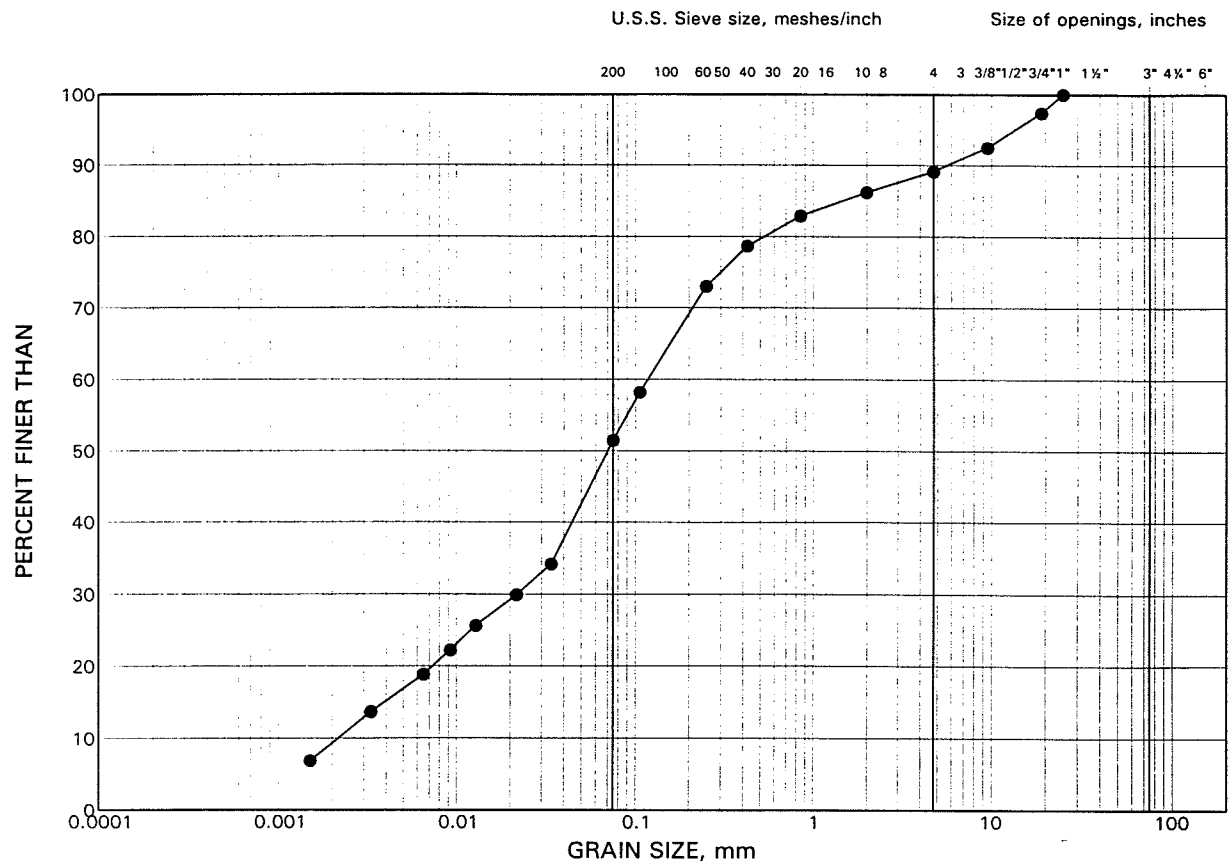
PROJECT 001-1143F				RECORD OF BOREHOLE No B4-2				1 OF 1		METRIC				
W.P. 30-95-00				LOCATION N 4904989.7; E 290437.7				ORIGINATED BY AZ						
DIST SW HWY 400				BOREHOLE TYPE 106mm DIAMETER SOLID STEM AUGERS				COMPILED BY LCC						
DATUM Geodetic				DATE Oct.24/2000				CHECKED BY ASP						
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DYNAMIC CONE PENETRATION RESISTANCE PLOT					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							
305.6	GROUND SURFACE						<div style="display: flex; justify-content: space-between;"> <div> 20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED </div> <div> 20 40 60 80 100 WATER CONTENT (%) </div> </div>							
0.0	Topsoil													
305.1	0.5	Clayey Silt with sand, some organics (Fill) Stiff Dark brown Moist				1	SS	11						
304.1	1.5	Silty Sand, trace gravel, clay and organics (Fill) Compact Dark brown Moist				2	SS	27						
303.4	2.2	Clayey Silt with sand, trace gravel (Till) Hard Brown becoming grey at 7.3m depth Moist				3	SS	48						
						4	SS	141/25						
		Cobbles at 3.7m depth				5	SS	101/15						
						6	SS	115/15						
		Silty sand, trace clay and gravel (Till) encountered in Sample 7.				7	SS	43						
						8	SS	210						
						9	SS	240						
						10	SS	185/15						
294.8	10.8	END OF BOREHOLE												
Notes: 1. Borehole dry on completion of drilling operations. 2. Water level in piezometer measured at 7.7m depth (Elev.297.9m) on March 15, 2001.														

ON_MOT_0011143F.GPJ ON_MOT.GDT 14/1/02

GRAIN SIZE DISTRIBUTION TEST RESULT

Clayey Silt 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)
•	B4-1	2	300.9

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Africa	+ 27 11 254 4800
Asia	+ 86 21 6258 5522
Australasia	+ 61 3 8862 3500
Europe	+ 356 21 42 30 20
North America	+ 1 800 275 3281
South America	+ 55 21 3095 9500

solutions@golder.com
www.golder.com

Golder Associates Ltd.
6925 Century Avenue, Suite #100
Mississauga, Ontario, L5N 7K2
Canada
T: +1 (905) 567 4444

