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PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

SUNNIDALE ROAD UNDERPASS, SITE NO. 30-173
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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**PRELIMINARY FOUNDATION REPORT - HIGHWAY 400
SUNNIDALE ROAD UNDERPASS**

PART A

**PRELIMINARY FOUNDATION INVESTIGATION REPORT
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by URS Canada Inc. (now AECOM) 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 Sunnidale Road Underpass in the City of Barrie. 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 Sunnidale Road Underpass (MTO Structure Site No. 30-173) 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 Sunnidale Road Underpass replacement is contained in Section 5.8 of AECOM's 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 Highway 400/Sunnidale Road Underpass is located in Barrie, Ontario and the existing bridge structure is a single-span rigid frame supported on spread footings. The total length of the bridge is approximately 29 m measured along the centerline of Sunnidale Road between abutments, and the total deck width is 12 m measured between fasciae.

The overall surface topography in the vicinity of the site is relatively flat and consists of residential areas to the east and west of Highway 400. At this structure site, Highway 400 has been constructed in a cut up to about 8.5 m deep and has an existing grade between about Elevation 248.5 m and 249 m, rising toward the north. The Sunnidale Road grade rises westward, from Elevations 255 m to 257 m, over Highway 400.

3.0 INVESTIGATION PROCEDURES

3.1 Previous Borehole Investigation

Two boreholes were advanced at this site as part of a previous Golder's geotechnical investigation in 2001 (MTO, 2002) for the replacement of the existing Sunnidale Road Underpass structure, associated with the widening of Highway 400. Borehole B13-1 was advanced near the west abutment on Sunnidale Road, to a depth of about 12.4 m below ground surface; and Borehole B13-2 was advanced near the east abutment on Sunnidale Road, to a depth of about 13.4 m. The borehole locations are shown on Drawing 1.

Both boreholes were advanced using 108 mm diameter hollow 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).

The water level in the open boreholes was observed during and following the drilling operations and a piezometer was installed in Borehole B13-2 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.



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Borehole Number	Location (MTM NAD83)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m)	Easting (m)		
BH13-1	4,916,980.9	288,547.3	256.8	12.4
BH13-2	4,916,967.4	288,585.1	255.1	13.4

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

As delineated in *The Physiography of Southern Ontario*¹, the 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 Peterborough Drumlin Field, consist primarily of gravelly sand till or 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; and 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 borehole locations are shown on Drawing 1 and stratigraphic profile and cross-section are shown on Drawings 1 and 2.

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.

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 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 Sunnisdale Road approach embankments, underlain by a surficial deposit of silty sand underlain by a silty sand till deposit which in turn is underlain by a clayey silt till deposit and/or a silty sand to sand deposit which extends to the bottom of the borehole.

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

4.2.1 Asphalt

An approximately 200 mm thick layer of asphalt was encountered at ground surface in Boreholes B13-1 and B13-2.

4.2.2 Sand and Gravel Fill

A 7.1 m and 6.7 m thick deposit of fill comprised on sand and gravel, some cobbles was encountered below the asphalt in Boreholes B13-1 and B13-2 at Elevations 256.6 m and 254.9 m, respectively.

The SPT 'N'-values measured within the fill deposit range from 9 blows to 62 blows per 0.3 m of penetration, but are typically between 11 blows and 25 blows per 0.3 m of penetration, indicating a predominantly compact relative density, and dense to very dense zones.

4.2.3 Silty Sand

A 1.1 m and 0.7 m thick deposit of silty sand, trace to some gravel, trace clay and trace wood fragments and organics was encountered below the fill in Boreholes B13-1 and B13-2 at Elevations 249.5 m and 248.2 m, respectively.

The SPT 'N'-values measured within the non-cohesive silty sand deposit are 19 blows and 31 blows per 0.3 m of penetration, indicating a compact to dense relative density.

The natural water content measured on samples of the silty sand deposit ranges from about 8 per cent to 10 per cent.

4.2.4 Silty Sand Till

A 2.1 m and 1.5 m thick till deposit comprised of silty sand, trace to some gravel, trace clay was encountered below the silty sand deposit in Boreholes B13-1 and B13-2 at Elevations 248.4 m and 247.5 m, respectively.

In general, the SPT 'N'-values measured within the non-cohesive till deposit range from 84 blows per 0.3 m of penetration to 100 blows per 0.15 m of penetration, indicating a very dense relative density. However, it is noted



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that an SPT 'N'-value of 21 blows per 0.3 m of penetration was measured at the top of the till deposit in Borehole B13-2, indicating a compact relative density this zone of the till deposit.

The natural water content measured on two samples of the silty sand till deposit are about 7 per cent and 8 per cent.

The result of a grain size distribution test completed on a sample of the silty sand till from Borehole B13-1 is shown on Figure 1 in Appendix A.

4.2.5 Clayey Silt Till

A 1.9 m thick cohesive till deposit comprised of clayey silt, trace to some sand and gravel was encountered below the silty sand till deposit in Borehole B13-1 at Elevation 246.3 m and the borehole was terminated within the deposit.

The SPT 'N'-values measured within the cohesive till deposit generally are 102 blows per 0.3 m of penetration and 100 blows per 0.18 m of penetration, suggesting a hard consistency.

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

An Atterberg limits test carried out on a sample of clayey silt till deposit measured a liquid limit of about 14 per cent, a plastic limit of about 11 per cent and a corresponding plastic index of about 3 per cent. The results of Atterberg limits test indicate that the till deposit is comprised of clayey silt of low plasticity to a silt of slight plasticity.

4.2.6 Silty Sand to Sand

A 4.3 m thick deposit of non-cohesive silty sand to sand, some silt, trace gravel was encountered below the silty sand till deposit in Borehole B13-2 at Elevation 246.0 m and the borehole was terminated within this deposit.

The SPT 'N'-values measured within the non-cohesive till deposit range from 69 blows to 129 blows per 0.3 m of penetration, and 100 blows per 0.15 m of penetration, indicating a very dense relative density.

The natural water content measured on samples of the silty sand to sand deposit ranges from about 17 per cent to 19 per cent.

The result of a grain size distribution test completed on a sample of the silty sand to sand deposit from Borehole B13-2 is shown on Figure 2 in Appendix A.

4.3 Groundwater Conditions

Borehole B13-1 was dry upon completion of drilling. A standpipe piezometer was installed in Borehole B13-2 located near the east abutment, and the groundwater level in the standpipe piezometer was measured at a depth of 9 m below ground surface, corresponding to Elevation 246.1 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.



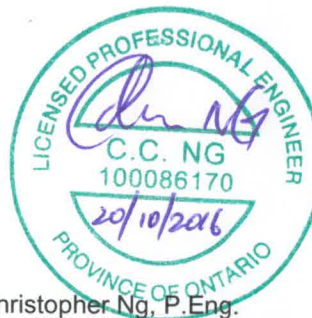
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5.0 CLOSURE

This report was prepared by Ms. Marzieh Kamranzadeh, M.Sc., EIT, 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.

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PART B

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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-Sunnidale Road Underpass (MTO Structure Site No. 30-173). These preliminary recommendations are based on interpretation of the factual data obtained from the boreholes advanced during a 2001 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 on behalf of the Ministry of Transportation, Ontario (MTO) to provide recommendations on foundation aspect for the preliminary design of the Highway 400-Sunnidale Road Underpass in the City of Barrie. It is understood that the Sunnidale Road Underpass will consist of a two-span, pre-cast girder bridge with 35 m and 36.5 m span lengths. Further, a grade raise of up to 1.8 m is proposed at the approach embankments adjacent to the abutments.

Based on the General Arrangement (GA) Drawing provided by AECOM on July 21, 2016, the grade of the proposed Underpass varies between Elevations 256.0 m and 259.2 m. In comparison, the proposed grade for Highway 400 is at about Elevation 248.5 m.

6.2 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the 2014 Canadian Highway Bridge Design Code (2014 CHBDC) and its Commentary, the proposed underpass 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.



6.3 Foundation Options

As part of the future widening of Highway 400 in Simcoe County, the existing Sunnidale Road Underpass will require replacement. According to the available information, the existing single-span structure is supported on spread footings that are founded at approximately Elevation 247.4 m. Highway 400 is proposed to be widened by approximately 20 m to the west and 21 m to the east of the existing alignment. Based on the proposed underpass geometry and the subsurface conditions at this site, both shallow foundation and deep foundation options have been considered for support of the abutments and pier 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 new (replacement) structure is presented in Table 1.

- **Shallow foundations – spread/strip footings:** Shallow foundations comprised of spread or strip footings, founded on the compact to very dense silty sand till deposit, are feasible for support of the new abutments and centre pier, 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 centre pier, and would permit design of conventional abutments, semi-integral abutments (for H-piles and pipe piles) or integral abutments (for H-piles only).
- **Deep foundations – drilled shaft (caissons):** Drilled shafts (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 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 and center pier, although steel H-pile foundations are preferred from a foundations perspective for all foundations elements, and they would permit integral abutment 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 compact to very dense silty sand till, 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
West Abutment	247.0	Very Dense Silty Sand Till
Centre Pier	247.0	Very Dense Silty Sand Till
East Abutment	247.0	Very Dense Silty Sand Till



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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 silty 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 compact to very dense silty 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 very dense silty sand till	0.45
Cast-in-place concrete footing on compacted Granular 'A' pad	0.60

6.4.4 Frost Protection

All 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 till and/or very dense silty sand to sand deposits.



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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 Sunnidale Road embankments. The following pile tip elevations could be considered for preliminary design. It should be noted that the higher tip elevation is based on approximately 3 m of penetration into the “100-blow” hard clayey silt till and very dense silty sand to sand deposit, while the lower tip elevation is for where higher factored geotechnical resistances are required for design. However, it should be noted that the lower tip elevation extends below the depth of available subsurface information.

Foundation Element	Proposed Underside of Pile Cap (m)	Estimated Design Tip Elevation (m)	Founding Soil at Tip Elevation
West Abutment	254.6	242.0 or 237.0	Hard Clayey Silt Till (Inferred)
Centre Pier	247.4	242.0 or 237.0	Hard Clayey Silt Till / Very Dense Silty Sand to Sand (Inferred)
East Abutment	252.2	242.0 or 237.0	Very Dense Silty Sand to Sand (Inferred)

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	Estimated Design Tip Elevation (m)	Factored Ultimate Geotechnical Axial Resistance (at ULS) (kN)	Factored Serviceability Geotechnical Resistance (at SLS) for 25 mm of Settlement ¹ (kN)
HP 310x110	242.0	700 (Abutments) 650 (Centre Pier)	N/A
	237.0	1,150 (Abutments) 1,100 (Centre Pier)	N/A
324 mm OD Pipe Pile	242.0	600 (Abutments) 600 (Centre Pier)	N/A
	237.0	1,000 (Abutments) 950 (Centre Pier)	N/A

Note: 1. 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.

Pile installation should be in accordance with OPSS 903 (Deep Foundations). Pile driving shoes, as per OPSD 3000.100 for H-Pile or OPSD 3001.100 for pipe (tube) piles, as applicable, are recommended to protect the pile tips from damage during driving into the dense till deposit. 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



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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 till and very dense silty sand to sand deposits 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 m penetration into "100-blow" soil below the depth of frost penetration:

Foundation Element	Proposed Underside of Pile Cap (m)	Estimated Design Tip Elevation (m)	Founding Soil at Tip Elevation
West Abutment	254.6	242.0	Hard Clayey Silt Till
Centre Pier	247.4	242.0	Very Dense Silty sand to Sand (Inferred)
East Abutment	252.2	242.0	Very Dense Silty sand to Sand

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.

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:



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Drilled Shaft Diameter (m)	Factored Ultimate Geotechnical Axial Resistance (at ULS) (kN)	Factored Serviceability Geotechnical Resistance (at SLS) for 25 mm of Settlement ¹ (kN)
0.9	1,600	N/A
1.2	2,800	N/A

Note: 1. 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 and perpendicular from 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, 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).
- 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



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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 Sunnidale Road embankment side slopes are inclined at about 2 horizontal to 1 vertical (2H:1V). For the proposed widening of the Highway 400 embankments, the new side slopes 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 OPSD 208.010 (Benching of Earth Slopes) to integrate the new fill into the existing slope fill.

It is understood that a 1.8 m grade raise of the existing/widened portion of the approach embankments is proposed. As indicated on OPSD 202.010 (Slope Flattening), a minimum 2 m wide bench should be incorporated into the approach embankment slopes where the slopes are equal to or greater than 8 m high, such that the uninterrupted slope height does not exceed 8 m.

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).

6.8.2 Embankment Stability and Factored Settlement

Limit equilibrium slope stability analyses for the embankment was carried out using the commercially available program Slide (version 6.0), developed by Rocscience Inc., employing the Morgenstern-Price method of analysis. For all analyses, the Factors of Safety (FoS) of numerous potential failure surfaces were computed for the critical embankment cross-section in order to establish the minimum FoS. The FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. For the purpose of the stability analysis, the FoS is equal to the inverse of the product of the consequence factor, Ψ , and the geotechnical resistance factor, ϕ_{gu} . (i.e. $FoS = 1/(\Psi \cdot \phi_{gu})$). Accordingly, a target minimum FoS of 1.7 has been used for the design of the embankment slopes for temporary and permanent conditions, respectively, as per Table 6.2 of CHBDC (2014).



The stability analyses assume that all organics and other deleterious materials are removed prior to constructing the approach embankments. Based on the results of the analysis for deep-seated global failure surfaces, the minimum FoS is equal to or greater than 1.7 and as such, stability issues are not anticipated within the limits of the approach embankment widening.

Settlement analyses were carried out using the commercially available program Settle3D (version 3.0), developed by Rocscience Inc. The factored settlement associated with the up to about 1.8 m of grade raise of the approach embankments is estimated to be about 25 mm. Given that the native subgrade deposits are primarily non-cohesive and the clayey silt till deposit is heavily over-consolidated, it is expected that the majority of the settlement will occur during and shortly after reconstruction and raising of the embankments.

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 of up to about 7.5 m below the existing Sunnidale Road grade and will be made through the existing embankment fill, native compact to dense silty sand, as well as the compact to very dense silty sand till deposit. The existing fill material and native dense silty sand and compact to very dense silty sand till deposits are classified as a Type 3 soil, 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 to facilitate the removal of the existing bridge foundations and construction of the abutments 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

Given that the depth to the groundwater level in the standpipe piezometer in Borehole BH13-2 was measured to be below the proposed bottom of the excavations for footings and pile caps, it is anticipated that control of groundwater will not be required.



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, the borehole at the east abutment indicated that the groundwater level was encountered at a depth of about 9 m below ground surface (about Elevation 246.1 m), hence running or flowing soil from the native non-cohesive deposits 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 and centre pier foundations to a sufficient depth below the ground surface to determine the presence and extent of cobbles and boulders that may be present with the native overburden 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 at the west abutment 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 deposits 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 2001 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 hard clayey silt till and silty sand to sand deposits will need to be reassessed during detail design.



PRELIMINARY FOUNDATION REPORT - HIGHWAY 400 SUNNIDALE ROAD UNDERPASS

7.0 CLOSURE

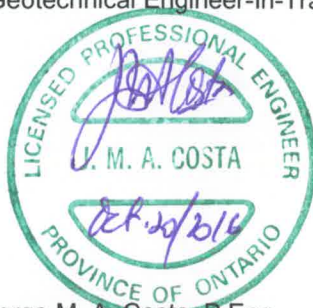
This report was prepared by Ms. Marzieh Kamranzadeh, M.Sc., a member of the geotechnical engineering group, and was reviewed by Mr. Christopher Ng, P.Eng., a senior geotechnical engineer and Associate of Golder. Mr. Jorge M. A. Costa, P.Eng., a Senior Consultant with Golder and Designated MTO Foundations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

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MK/CN/JMAC/mck/mk

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PRELIMINARY FOUNDATION REPORT - HIGHWAY 400 SUNNIDALE ROAD UNDERPASS

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Ministry of Transportation, Ontario. 2002. *Preliminary Foundation Investigation and Design Report Sunnidale Road Underpass, Structure Site 30-173; Highway 400 Widening from 1 km South of Highway 89 to Highway 11, G.W.P. 30-95-00*, GEOCREs No. 31D00-477, prepared by Golder Associates Ltd.

ASTM International:

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

Commercial Software:

Slide (Version 6.0) by Rocscience Inc.

Settle^{3D} (Version 3.0) by Rocscience Inc.

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



PRELIMINARY FOUNDATION REPORT - HIGHWAY 400 SUNNIDALE ROAD UNDERPASS

Ontario Provincial Standard Drawings (OPSD)

OPSD 202.010	Slope Flattening Using Surplus Excavated Material on Earth or Rock Embankments
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



PRELIMINARY FOUNDATION REPORT - HIGHWAY 400 SUNNIDALE ROAD UNDERPASS

TABLES



PRELIMINARY FOUNDATION REPORT - HIGHWAY 400 SUNNIDALE ROAD 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 the support of new abutments 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"> Requires larger footing excavation and disposal of a larger volume of soil compared to the excavation for a pile cap (excavation to about 4.5 m depth). Do not allow for integral abutment construction. Likely requires temporary protection system to allow for excavation/footing construction at centre pier. 	<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/very dense native overburden, 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 SUNNIDALE ROAD UNDERPASS

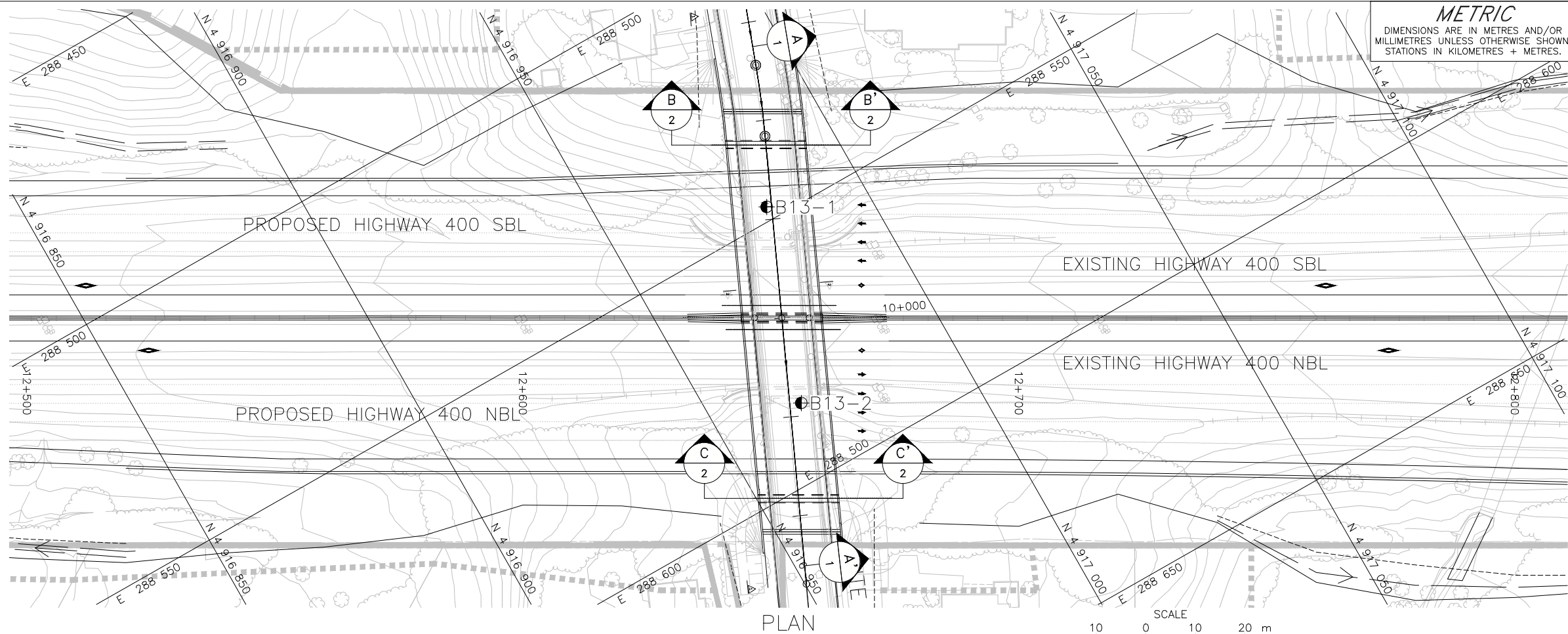
TABLE 1 – COMPARISON OF REPLACEMENT STRUCTURE FOUNDATION ALTERNATIVES

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>	

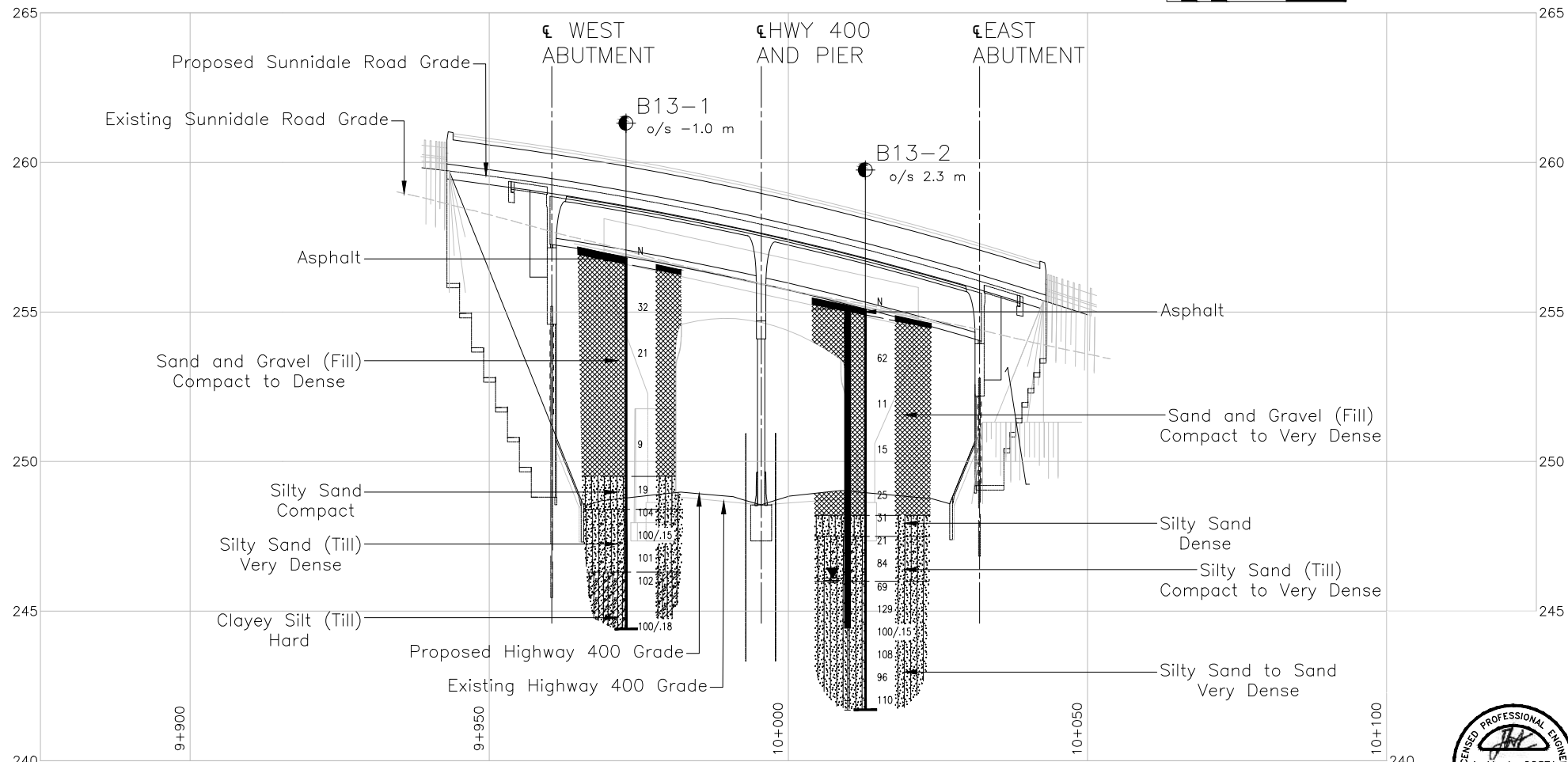
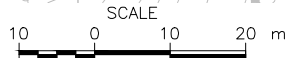


**PRELIMINARY FOUNDATION REPORT - HIGHWAY 400
SUNNIDALE ROAD UNDERPASS**

DRAWINGS

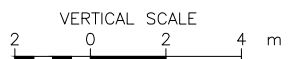


PLAN



A-A
1

SUNNIDALE ROAD
CENTRELINE PROFILE



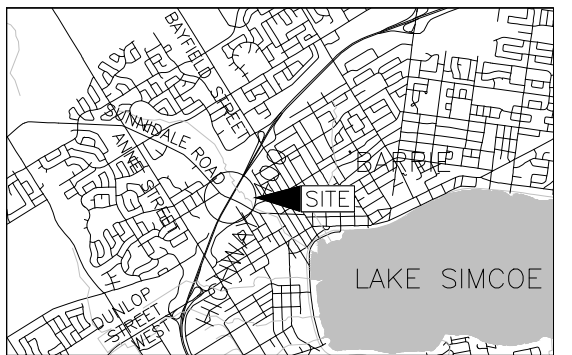
METRIC
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MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 06-20016

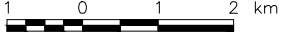
SUNNIDALE ROAD UNDERPASS
HIGHWAY 400 WIDENING
BOREHOLE LOCATIONS
AND SOIL STRATA



SHEET



KEY PLAN
SCALE



LEGEND

- Borehole - Previous Investigation (Geocres No. 31D-477)
- 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
B13-1	256.8	4916980.9	288547.3
B13-2	255.1	4916967.4	288585.1

NOTES

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

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

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

REFERENCE

General arrangement, designs, base plans, profile and surface data provided in digital format by AECOM, drawing file nos. "01_Sunnidale Road Underpass_GA(2).dwg", received June 23, 2016, "X-Base_All.dwg", received January 27, 2016, "X-Design_4th Line_Interim.dwg", received June 22, 2015, and "X-Surfaces.dwg", received April 14, 2015.



NO.	DATE	BY	REVISION
Geocres No. 31D-665			
HWY. 400		PROJECT NO. 14-1111-0002	
SUBM'D. BM	CHKD. CN	DATE: 7/21/2016	SITE: 30-173
DRAWN: MR	CHKD. MK	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

SUNNIDALE ROAD UNDERPASS
HIGHWAY 400 WIDENING

SOIL STRATA

SHEET



KEY PLAN
SCALE
1 0 1 2 km

LEGEND

- Borehole - Previous Investigation (Geocres No. 31D-477)
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
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BOREHOLE CO-ORDINATES

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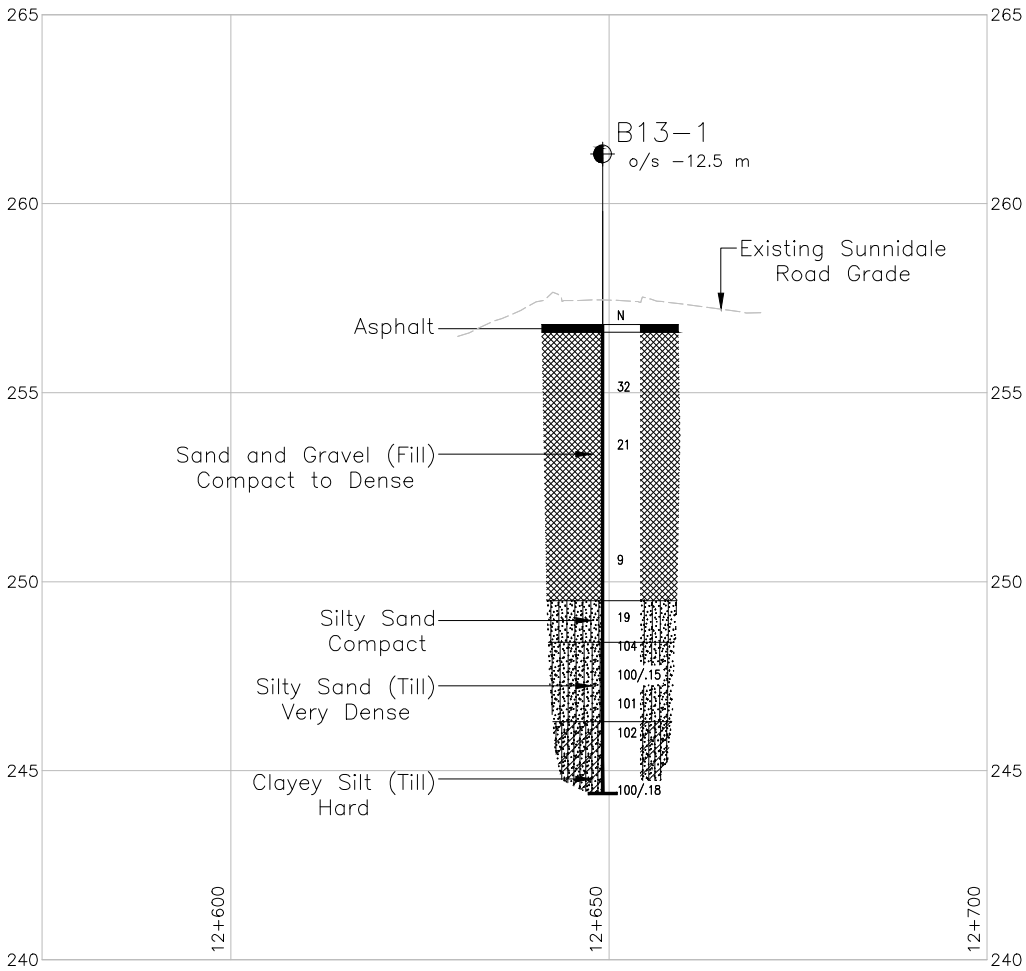
REFERENCE

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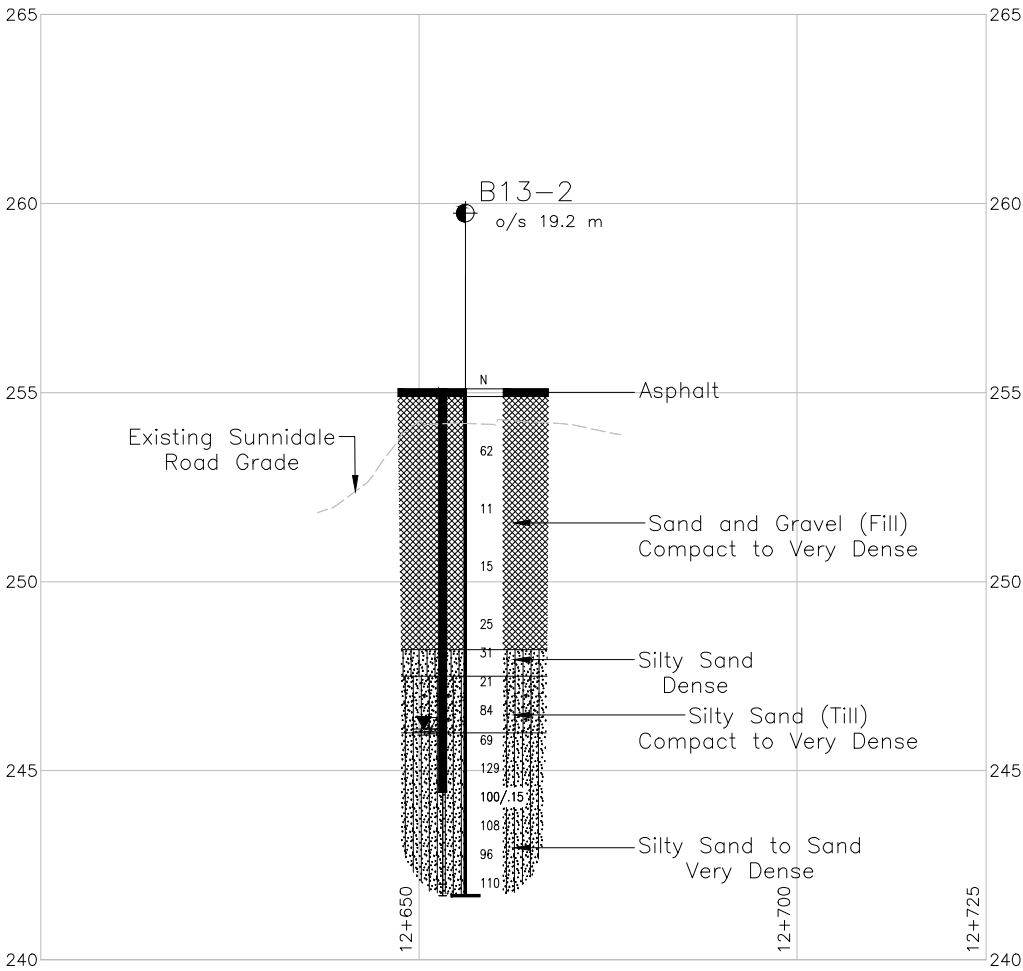
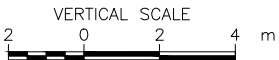
NO.	DATE	BY	REVISION
Geocres No. 31D-665			
HWY. 400		PROJECT NO. 14-1111-0002	DIST. .
SUBM'D. BM	CHKD. CN	DATE: 7/21/2016	SITE: 30-173
DRAWN: MR	CHKD. MK	APPD. JMAC	DWG. 2



B-B WEST ABUTMENT CROSS-SECTION



C-C EAST ABUTMENT CROSS-SECTION





APPENDIX A

**Record of Boreholes and Laboratory Test Results – Golder 2001
Investigation (GEOCRES No. 31D-477)**

PROJECT 001-1143F		RECORD OF BOREHOLE No B13-1		1 OF 1		METRIC							
W.P. 30-95-00		LOCATION N 4916980.9; E 288547.3		ORIGINATED BY PKS									
DIST SW HWY 400		BOREHOLE TYPE 108mm ID HOLLOW STEM AUGERS		COMPILED BY LCC									
DATUM Geodetic		DATE Feb. 8/2001		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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					
256.8	GROUND SURFACE						20 40 60 80 100						
0.0	Asphalt												
0.2	Sand and Gravel, some cobbles (Fill) Compact to dense Brown Moist												
			1	SS	32								
			2	SS	21								
			3	SS	9								
249.5													
7.3	Silty Sand, trace gravel Compact Brown Moist		4	SS	19								
248.4													
8.4	Silty Sand, trace clay, trace gravel (Till) Very dense Brown Moist		5	SS	104								
			6	SS	100/15								4 57 30 9
			7	SS	101								
246.3													
10.5	Clayey Silt, trace to some sand and gravel (Till) Hard Brown Moist		8	SS	102								
244.4			9	SS	100/18								
12.4	END OF BOREHOLE												
	Notes: 1. Borehole dry on completion of drilling. 2. Borehole backfilled with bentonite and surface cold patch on sand bedding.												

ON_MOT 0011143F.GPJ ON_MOT_GDT 14/1/02

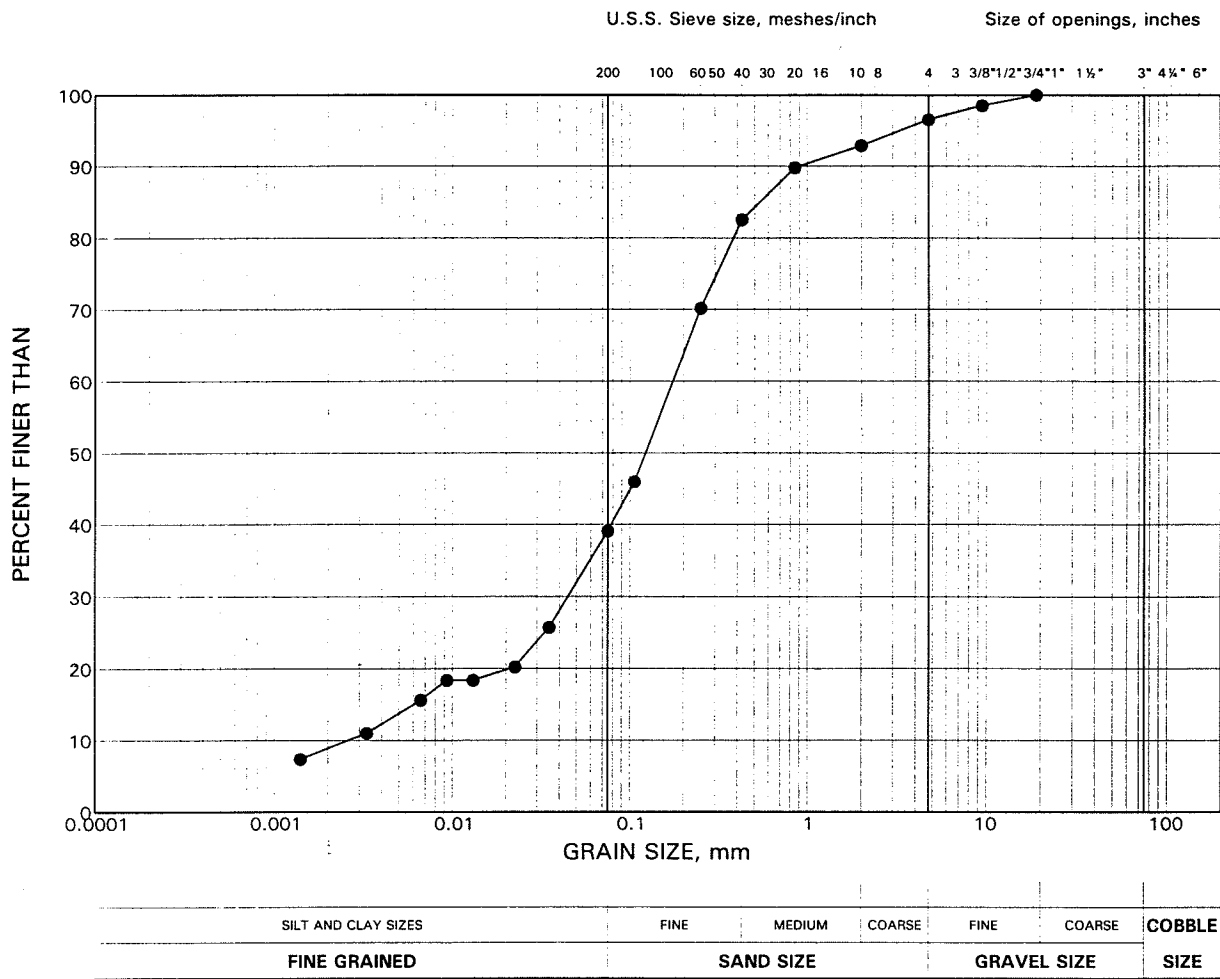
PROJECT 001-1143F				RECORD OF BOREHOLE No B13-2				1 OF 1		METRIC				
W.P. 30-95-00				LOCATION N 4916967.4; E 288585.1				ORIGINATED BY PKS						
DIST SW HWY 400				BOREHOLE TYPE 108mm ID HOLLOW STEM AUGERS				COMPILED BY LCC						
DATUM Geodetic				DATE Feb.6-7/2001				CHECKED BY ASP						
SOIL PROFILE			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						
255.1	GROUND SURFACE													
0.0	Asphalt													
0.2	Sand and Gravel (Fill) Compact to very dense Brown Moist													
			1	SS	62									
			2	SS	11									
			3	SS	15									
			4	SS	25									
248.2														
6.9	Silty Sand, trace clay, some gravel, trace wood and organics Dense Brown Moist		5	SS	31									
247.5														
7.6	Silty Sand, trace clay, trace to some gravel (Till) Compact to very dense Brown Moist		6	SS	21									
			7	SS	84									
246.0														
9.1	Silty Sand to Sand, some silt, trace gravel Very dense Wet Brown		8	SS	69									0 85 15 0
			9	SS	129									
			10	SS	100/15									
			11	SS	108									
			12	SS	96									
	Thin silty clay layers present in Sample 13.		13	SS	110									
241.7														
13.4	END OF BOREHOLE													
	Notes: 1. Water level on completion of drilling at 11m depth (Elev.244.1m). 2. Water level in piezometer measured at 9m depth (Elev.246.1m) on March 15, 2001.													

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

GRAIN SIZE DISTRIBUTION TEST RESULT

Silty Sand Till

FIGURE 1



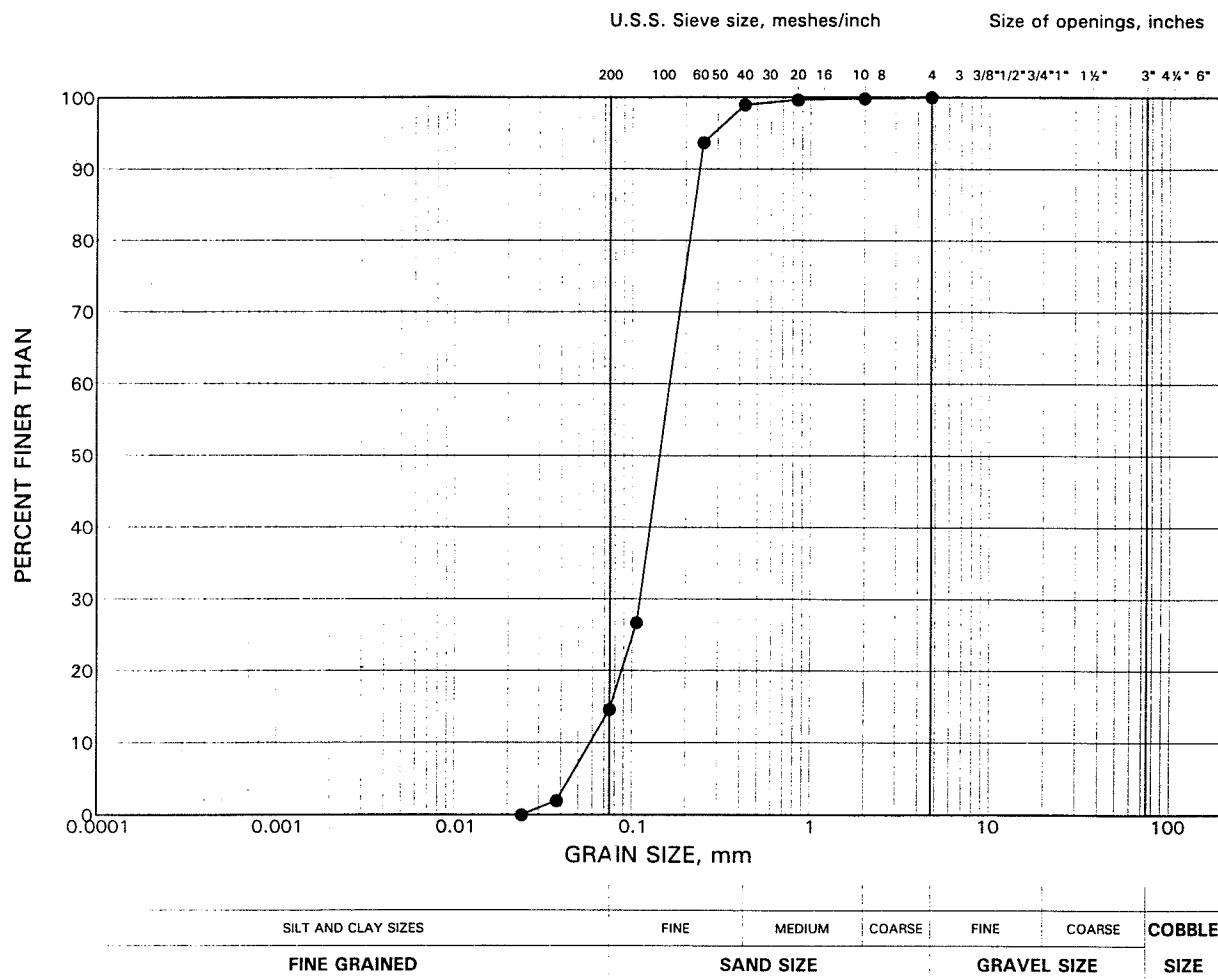
LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
•	B13-1	6	247.5

GRAIN SIZE DISTRIBUTION TEST RESULT

Sand, some silt

FIGURE 2



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
•	B13-2	8	245.6

At Golder Associates we strive to be the most respected global company providing consulting, design, and construction services in earth, environment, and related areas of energy. Employee owned since our formation in 1960, our focus, unique culture and operating environment offer opportunities and the freedom to excel, which attracts the leading specialists in our fields. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees who operate from offices located throughout Africa, Asia, Australasia, Europe, North America, and South America.

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