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FOUNDATION INVESTIGATION AND DESIGN REPORT

**GO BRT BRIDGE AT GLEN ERIN DRIVE
GO TRANSIT - BUS RAPID TRANSIT WEST FROM
WINSTON CHURCHILL BOULEVARD TO ERIN MILLS
PARKWAY
MISSISSAUGA, ONTARIO**

Submitted to:
Giffels Associates Limited/IBI Group
30 International Boulevard
Toronto, Ontario
M9W 5P3



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REPORT



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FOUNDATION REPORT GO BRT BRIDGE AT GLEN ERIN DRIVE

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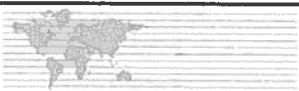
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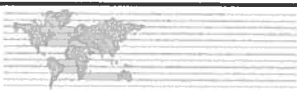
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**FOUNDATION REPORT
GO BRT BRIDGE AT GLEN ERIN DRIVE**

PART A

**FOUNDATION INVESTIGATION REPORT
GO BRT BRIDGE AT GLEN ERIN DRIVE
MISSISSAUGA, ONTARIO**



FOUNDATION REPORT GO BRT BRIDGE AT GLEN ERIN DRIVE

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Giffels Associates Limited/IBI Group (Giffels/IBI) on behalf of GO Transit (GO) to provide geotechnical engineering services for the detail design of the GO Bus Rapid Transit West (GO BRT) between Winston Churchill Boulevard and Erin Mills Parkway, in the City of Mississauga, Ontario. The proposed GO BRT alignment will run parallel to and will be about 50 m north of the existing Highway 403. In addition to the busway itself, the GO BRT project involves the bus stations at Winston Churchill Boulevard and at Erin Mills Parkway, ramps, five bridges, associated retaining walls, high mast lights and overhead signs.

This report addresses the geotechnical investigation carried out for the proposed GO BRT underpass structure at Glen Erin Drive. The purpose of this investigation is to establish the subsurface conditions and shallow groundwater conditions at the proposed structure site by means of a limited number of boreholes. Based on our interpretation of the data, engineering recommendations on the geotechnical aspects of design of the underpass structure, associated retaining walls and embankment construction are provided herein.

This report addresses only the geotechnical (physical) aspects of the subsurface conditions at this site. The geo-environmental (chemical) aspects, including consequences of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources, are outside the terms of reference for this report.

The factual data, interpretations and recommendations contained in this report pertain to a specific project as described in the report and are not applicable to any other project or site location. If the project is modified in concept, location or elevation, or if there are changes to site conditions, Golder should be given an opportunity to confirm that the recommendations are still valid.

This report should be read in conjunction with "Important Information and Limitations of This Report", following the text of this report. The reader's attention is specifically drawn to this information, as it is essential for the proper use and interpretation of this report.

The terms of reference for the project are presented in the Request for Proposal dated June 15, 2009, and the scope of work is outlined in Golder's proposal P9-1181-1045, dated June 25, 2009.

2.0 SITE DESCRIPTION

The site of the proposed GO BRT underpass structure at Glen Erin Drive is located north of Highway 403 between Winston Churchill Boulevard and Erin Mills Parkway in the City of Mississauga, Ontario (see key plan on Drawing 1). The GO BRT alignment will run parallel to and about 50 m north of Highway 403. Glen Erin Drive is currently a paved four-lane road which carries northbound and southbound traffic over the existing Highway 403.

North of Highway 403 in the area of the proposed GO BRT structure location, Glen Erin Drive is constructed on an earth embankment about 6 m high. The Glen Erin Drive road surface at the proposed GO BRT alignment is at about Elevation 174.0 m and the surrounding ground at the base of the embankment is at about Elevation 168.0 m. North of the proposed GO BRT alignment a hydro corridor runs in an east-west direction. The land use north of the hydro corridor is primarily residential.



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3.0 INVESTIGATION PROCEDURES

The fieldwork for the investigation associated with the proposed GO BRT underpass at Glen Erin Drive was carried out between January 27 and 29, 2010, at which time eight boreholes (Borehole 5001 to 5008) were advanced at the approximate locations shown on Drawing 1, following the text of this report.

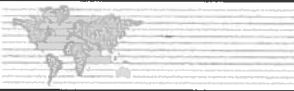
The field investigation was carried out using a track-mounted drill rig and a truck-mounted drill rig supplied and operated by Walker Drilling Ltd. of Utopia, Ontario. The boreholes were advanced using 200 mm outside diameter hollow stem augers to depths ranging from 12.5 m to 16.9 m below existing ground surface. Soil samples were obtained at 0.75 m and 1.5 m intervals of depth using 50 mm outside diameter split-spoon samplers driven by an automatic hammer in accordance with the Standard Penetration Test (SPT) procedure (ASTM D1586).

The groundwater conditions in the open boreholes were observed throughout the drilling operations and a standpipe piezometer was installed in Borehole 5002 to permit monitoring of the groundwater level at the site. The piezometer consists of 51 mm diameter PVC pipe with a slotted screen sealed at a selected depth within the borehole. A sand filter pack surrounds the screen, and the borehole and annulus surrounding the standpipe piezometer above the screen was backfilled to the ground surface with bentonite pellets. The piezometer installation details and water level readings are described on the Record of Borehole sheets following the text of this report. The open boreholes were backfilled with bentonite upon completion of drilling in accordance with Ontario Regulation (O.Reg.) 903 as amended by O.Reg. 372/07 of the Ontario Water Resources Act.

The field work was monitored on a full-time basis by a member of Golder's technical staff who arranged for service clearances and road occupancy permits, observed the drilling, sampling and in-situ testing operations, logged the boreholes and examined and cared for the soil samples. The soil samples were identified in the field, placed in labelled containers and transported to Golder's laboratory in Mississauga for further examination and classification testing (water contents, Atterberg limits and grain size distributions) on selected soil samples. All of the laboratory tests were carried out to MTO and/or ASTM Standards as appropriate.

The borehole locations were surveyed in the field by SCS Consulting Group Ltd. prior to the drilling operations. The as-drilled borehole locations (referenced to MTM NAD83 coordinate system) and ground surface elevations (referenced to geodetic datum) are summarized below.

Borehole	Northing (m)	Easting (m)	Ground Surface Elevation (m)
5001	4823283.9	288377.9	174.2
5002	4823294.4	288368.8	173.9
5003	4823303.9	288357.8	173.7
5004	4823309.8	288348.3	173.7
5005	4823325.0	288359.2	173.6
5006	4823315.4	288367.2	173.8
5007	4823305.6	288378.4	174.0
5008	4823298.8	288387.6	174.2



4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The study area for this investigation lies within the Trafalgar Moraine portion of the South Slope, as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984). A surficial till sheet, which generally follows the surface topography, is present throughout much of this area. The till is variable in composition ranging from a cohesive till comprised of clayey silt/silty clay to a cohesionless sand and silt till. The study area is underlain by Ordovician shales of the Queenston Formation (Ontario Geological Society, 1991). All major rivers in this area cut through the till deposit and into the shale; the valley walls formed within the shale often are almost perpendicular. In the vicinity of the Highway 407/403 interchange and the Erin Mills Parkway interchange, the depth to bedrock is fairly shallow; however, in vicinity of Winston Churchill Boulevard an infilled bedrock valley exists and the depth to the bedrock surface is much greater in this area. After this glacial valley was formed, it was infilled with clayey silt during the retreat of the glaciers.

4.2 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of in situ and laboratory testing are given on the Record of Borehole sheets following the text of this report; the results of laboratory testing are also presented on Figures 1 to 8.

The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests (SPTs). These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations. The interpreted soil stratigraphy based on the results of the boreholes is shown on Drawings 1 and 2.

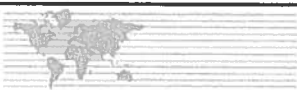
In summary, the subsoil conditions encountered at the site consist of fill (predominately comprised of silty clay to clayey silt) underlain by two till deposits: a clayey silt till underlain by a sand and silt till. The lower till deposit is underlain by residual soil/shale bedrock.

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

4.2.1 Topsoil

Topsoil was encountered at the existing ground surface in all boreholes except for Borehole 5001, 5003 and 5008. The topsoil was found to be between about 100 mm and 300mm thick at the borehole locations.

Standard Penetration Test (SPT) 'N'-values recorded at least partially within the topsoil range between 9 and 12 blows per 0.3 m of penetration, suggesting a stiff consistency. Drilling was performed during the winter months during which time the topsoil and near surface soils may have been frozen, therefore, the near surface SPT 'N'- values may not be representative of thawed conditions.



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4.2.2 Fill – Sand and Gravel and Silty Clay to Clayey Silt

Underlying the topsoil and immediately below ground surface in Boreholes 5003 and 5008, a layer of fill consisting of sand and gravel was encountered extending to depths between 0.6 m and 0.7 m below existing ground surface (extending to between Elevations 173.0 m and 173.6 m). Immediately below ground surface in Borehole 5001 and in all other boreholes drilled for this underpass the sand and gravel fill/topsoil is underlain by embankment fill consisting of silty clay to clayey silt. The cohesive fill extends to depths between 5.6 m and 7.2 m below ground surface (between Elevations 166.4 m and 168.1 m).

The measured SPT 'N'-values within the fill generally range from 5 to 25 blows per 0.3 m of penetration, suggesting a firm to very stiff consistency. Two measured SPT 'N'-values of 35 and 41 blows per 0.3 m of penetration were recorded in the fill, however the split spoon advancement was likely hindered by the presence of cobbles resulting in higher SPT 'N'-values. Cobbles and possibly boulders are anticipated to be present in the fill as inferred by the grinding of augers during drilling as noted on the Record of Borehole sheets, following the text of this report.

The silty clay to clayey silt fill contains some sand, trace to some gravel and contains organics, rootlets and cobble fragments. The results of grain size distribution tests completed on eight samples of the fill are provided on Figures 1A and 1B. Atterberg limits testing carried out on three samples of the silty clay portion of the fill measured liquid limits of 35 and 36 percent, plastic limits of 20 and 22 percent, and plasticity indices of 14 and 15 percent. The results of the Atterberg limits testing are shown on Figure 2 and indicate the fines of these samples to be silty clay of medium plasticity. Atterberg limits testing carried out on five samples of the clayey silt portion of the fill measured liquid limits ranging from 27 to 34 percent, plastic limits ranging from 18 to 19 percent, and plasticity indices ranging from 9 to 15 percent. The results of the Atterberg limits testing are also shown on Figure 2 and indicate that the fines are a clay of low plasticity. The measured water contents of samples of the fill range from 4 to 18 percent.

4.2.3 Clayey Silt Till

In all boreholes drilled for this structure, the fill is underlain by till deposits consisting of an upper stratum of clayey silt till, underlain by a stratum of sand and silt till (discussed in Section 4.2.4). The surface of the clayey silt till was encountered at depths between 5.6 m and 7.2 m below ground surface corresponding to between Elevation 168.1 m and 166.4 m. The thickness of this cohesive till deposit varies from about 1.3 m to 3.9 m.

The measured SPT 'N'-values in the clayey silt till ranged from 14 to 63 blows per 0.3 m of penetration, suggesting a stiff to hard consistency.

The clayey silt till deposit contains some sand and trace gravel. Cobbles/boulders have been inferred to be present within the till deposit at this site, based on grinding of the augers during the borehole drilling as noted on the Record of Borehole sheets. The results of grain size distribution tests completed on seven samples of clayey silt till are shown on Figure 3. Atterberg limits testing carried out on six samples of the clayey silt till measured liquid limits ranging from 27 to 30 percent, plastic limits ranging from 16 to 18 percent, and plasticity indices ranging from 10 to 13 percent. The results of the Atterberg limits testing are shown on Figure 4 and indicate that the fines are a clay of low plasticity. The measured water contents of samples of the till range from 10 to 17 percent.



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4.2.4 Sand and Silt Till

The clayey silt till in all boreholes drilled for this structure is underlain by a deposit of sand and silt till. The surface of the sand and silt till was encountered at depths ranging between 8.5 m and 11.1 m below ground surface (between Elevation 163.1 m and 165.5 m). The sand and silt till deposit ranges in thickness from 3 m to 5.3 m at the boreholes and was fully penetrated at Borehole 5002 and 5008.

The measured SPT 'N'-values in the sand and silt till range from 46 blows to greater than 100 blows per 0.3 m of penetration, suggesting a dense to very dense relative density.

The sand and silt till contains trace to some gravel and clay. Cobbles/boulders have been inferred to be present within the till deposit at this site based on grinding of the augers during the borehole drilling as noted on the Record of Borehole sheets. The results of grain size distribution tests completed on thirteen samples of sand and silt till are shown on Figures 5A and 5B. Atterberg limits tests were carried out on three samples of the sand and silt till and two of the samples indicate that the material is generally non-plastic, however, one test shows a liquid limit of 21 percent, a plastic limit of 15 percent and a plasticity index of 5 percent. The results of the Atterberg limits testing are shown on Figure 6 and indicate that sand and silt till is generally non-plastic or has a low plasticity. The measured water contents of samples of the sand and silt till range from 5 to 20 percent.

4.2.5 Clayey Silt Residual Soil

Residual soil, which is derived from weathering of the underlying shale bedrock was encountered underlying the sand and silt till deposit in Boreholes 5003 and 5007. The residual soil consists of clayey silt with a "till-like" structure and contains varying amounts of siltstone/limestone and shale fragments. The surface of the residual soil was encountered at depths of 13.7 m and 13.1 m below ground surface in Boreholes 5003 and 5007, respectively, corresponding to Elevation 160.0 m and 160.9 m. In Borehole 5007, the residual soil was found to be about 1.5 m thick, however, in Borehole 5003 the residual soil extends to the base of the borehole at a depth of 15.4 m (Elevation 158.3 m), indicating the deposit to be greater than 1.7 m thick.

The measured SPT 'N'-values in the clayey silt residual soil are 100 blows per 0.13 m and 0.15 m of penetration, suggesting a hard consistency.

A grain size distribution test was completed on one selected sample of the clayey silt residual soil and the results are provided on Figure 7. Atterberg limits testing carried out on one sample of the clayey silt residual soil indicate a liquid limit of 21 percent, a plastic limit of 15 percent and a plasticity index of 6 percent. The results of the Atterberg limits testing are shown on Figure 8 and indicate that the fines are a clay of a low plasticity. The measured water contents of two samples of the clayey silt residual soil are 9 and 14 percent.

4.2.6 Shale Bedrock

Shale bedrock was encountered in Boreholes 5001 and 5004 to 5007 and these boreholes were extended into the bedrock by augering and split spoon sampling. The depth to and elevation of the bedrock surface in these boreholes is summarized below.



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Borehole No.	Depth to Bedrock Surface	Bedrock Surface Elevation
5001	15.2 m	159.0 m
5004	14.3 m	159.4 m
5005	11.7 m	161.9 m
5006	13.3 m	160.5 m
5007	14.6 m	159.4 m

The bedrock samples obtained consist of reddish brown shale which, based on available bedrock geology maps, is understood to be part of the Queenston Formation.

4.2.7 Groundwater Conditions

Water levels were noted in the open boreholes during and after the drilling operations. A standpipe piezometer was installed in Borehole 5002 to permit monitoring of the groundwater level at the site. The standpipe piezometer was sealed within the sand and silt till deposit. Details of the installation are shown in the Record of Borehole sheet following the text of this report. The water levels recorded in the piezometer are summarized below.

Borehole No.	Ground Surface Elevation	Depth Below Ground Surface to Water Level	Ground Water Level Elevation	Date
5002	173.9 m	7.7 m	166.2 m	February 25, 2010
		7.8 m	166.1 m	March 3, 2010
		7.7 m	166.2 m	March 5, 2010
		7.5 m	166.4 m	March 12, 2010
		7.3 m	166.6 m	March 16, 2010
		7.4 m	166.5 m	April 1, 2010
		7.4 m	166.5 m	April 12, 2010

It is noted that the water level at about Elevation 166.6 m is approximately the same as the base of the fill to 2 m below the top of the native material at the site. The original ground surface of the native material(s) and original ground surface in the area are inferred to slope downward to the southeast based on the borehole information and available topographic mapping.

It should be noted that groundwater levels are expected to fluctuate seasonally and are expected to be higher during wet periods of the year.



FOUNDATION REPORT GO BRT BRIDGE AT GLEN ERIN DRIVE

5.0 CLOSURE

The field technicians directing the drilling program were Messrs. Matthew Kelly, E.I.T., Ravinder Singh and Srinivasa Yellu of Golder. This Foundation Investigation Report was prepared by Ms. Sandra McGaghran, P.Eng., a Geotechnical Engineer with Golder and reviewed by Ms Anne Poschmann, P.Eng., a Principal and Geotechnical Engineer with Golder. Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project and Principal with Golder, conducted an independent quality control review of the report.

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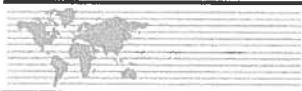
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SMM/ASP/JMAC/jl

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**FOUNDATION REPORT
GO BRT BRIDGE AT GLEN ERIN DRIVE**

PART B

**FOUNDATION DESIGN REPORT
GO BRT BRIDGE AT GLEN ERIN DRIVE
MISSISSAUGA, ONTARIO**



6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides recommendations for the foundation design aspects of the proposed GO BRT underpass structure at Glen Erin Drive in the City of Mississauga. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the design of the structure foundations and approach embankments. Where comments are made on construction, they are provided in order to highlight those aspects which could affect the design of the project, and for which special provisions or operational constraints may be required in the Contract Documents. Those requiring information on the aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.1 General

Currently there is an existing embankment that carries Glen Erin Drive over Highway 403 and the proposed GO BRT underpass will be constructed by cutting through the embankment beneath Glen Erin Drive. The underpass structure at Glen Erin Drive is proposed to be a single span bridge with a span length of 13.9 m. Based on the General Arrangement (GA) drawing provided by Giffels/IBI on November 18, 2009, the minimum proposed road grade of the GO BRT adjacent to the abutments will be at about Elevation 167.4 m and the road grade of Glen Erin Drive will be maintained at Elevation 174.0 m. It is understood that the underside of the abutment footings will extend to about 1.6 m below final grade, thereby providing about 0.6 m of cover over the 1.0 m thick footings. There will be some minor lowering of the ground surface adjacent to the proposed structure for the busway, however, there will be no raising or widening of the Glen Erin Drive approach embankments.

6.2 Foundation Options

Based on the proposed vertical elevation of the bridge structure and the subsurface soil and groundwater conditions encountered at the site, the following foundations options are considered feasible for the GO BRT underpass at Glen Erin Drive:

- **Spread Footings founded on the very stiff to hard clayey silt till:** Spread footings would need to be founded on the very stiff to hard clayey silt till at a founding level no higher than Elevation 166.0 m. For the proposed footing foundation level 1.6 m below final grade and a finished road grade at Elevation 167.4 m, the footings would be founded at about Elevation 165.8 m.
- **Caissons founded within the sand and silt till deposit or bedrock:** Caissons may extend into the sand and silt till to Elevation 163.0 m, or alternatively the caissons may be founded on the shale bedrock at about Elevation 158.5 m. It is noted that grinding of the augers was encountered in the majority of the boreholes, generally occurring between about Elevation 162 m and 165 m. The caisson excavation operations will therefore need to be able to deal with boulders and possibly boulder layers within the till deposits.

Driven steel H-piles are not considered appropriate for this site given the presence of hard/very dense strata at relatively shallow depth below the proposed founding level. Recommendations for each of the feasible



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foundation options are presented in the following sections of this report. From a geotechnical perspective, spread footings are preferred over caissons, due to the potential difficulty for caisson construction with augering through cobbles and boulders which are anticipated to be present within the till.

6.3 Spread Footings

The following sections provide recommendations for founding elevations, geotechnical resistances, resistance to lateral loads and protection from frost penetration for spread footings founded on the very stiff to hard clayey silt till.

6.3.1 Founding Elevations

Based on the GA drawing provided by Giffels/IBI, the minimum proposed road grade of GO BRT adjacent to the abutments is at about Elevation 167.4 m and the abutment spread footings are proposed to extend to Elevation 165.8 m. Based on the soil conditions encountered in the boreholes drilled close to the proposed abutments, fill materials generally extend to about Elevation 166.6 m, with the exception of Borehole 5003 at the west side of the north abutment where the fill extends to about Elevation 168.1 m. The spread footings will need to be founded at or below Elevation 166.0 m and be placed within the very stiff clayey silt till below the fill material.

6.3.2 Geotechnical Resistance

Given the above proposed road grade for the busway, the proposed footing founding level will be at a depth of about 1.6 m below the final grade. For this condition and considering that the groundwater table is at about Elevation 166.5 m, a factored geotechnical resistance at Ultimate Limit States (ULS) of 475 kPa and a geotechnical resistance at Serviceability Limit States (SLS) of 300 kPa (for 25 mm of settlement) may be used for design of 3 m wide spread footings placed on undisturbed very stiff to hard clayey silt till at or below Elevation 166 m or on very dense sand and silt till below Elevation 165 m.

The base of each footing excavation should be cleaned of loose / softened material. It is essential that the founding level for the footings be inspected by geotechnical personnel immediately prior to pouring concrete to confirm the adequacy of the foundation subgrade for the above noted geotechnical resistances. If the concrete for the footings cannot be poured immediately after excavation and inspection, it is recommended that mass concrete (100 mm thick of 20 MPa compressive strength concrete) be placed in the excavation to protect the integrity of the bearing stratum within three hours of excavating to the bearing stratum.

The geotechnical resistances provided above are given under the assumption that the loads will be applied perpendicular to the surface of the footing. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Section 6.7.4 of the Canadian Highway Bridge Design Code (*CHBDC*) and its Commentary.

6.3.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the cast-in-place concrete footings and the very stiff to hard clayey silt till should be calculated in accordance with Section 6.7.5 of the *Canadian Highway Bridge Design Code (CHBDC)*. A coefficient of friction, $\tan \phi'$ of 0.55 can be used for cast-in-place concrete footings placed on properly prepared undisturbed clayey silt till subgrade.



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6.3.4 Frost Protection

All footings should be provided with a minimum of 1.2 m of soil cover for frost protection purposes, (Ontario Provincial Standard Drawing (OPSD) 3090.101 – Foundation Frost Depths for Southern Ontario).

6.4 Caissons

Consideration could be given to the use of caissons socketted into the sand and silt till for the support of the foundation elements for the abutments. Consideration could also be given to extending the caissons to the shale bedrock.

Founding Stratum	Estimated Caisson Founding Elevation
Very Dense Sand and Silt Till	163.0 m
Shale Bedrock	158.5 m

The caisson excavation operations and equipment will need to be able to deal with boulders and possibly bouldery layers within the till deposits. Grinding of the augers was encountered in the majority of the boreholes, generally occurring between about Elevations 162 m and 165 m within the till deposits.

The performance of caissons will depend upon the final cleaning and verification of the quality of the bedrock at the base should the caissons extend into the shale bedrock. Each caisson excavation must be carefully cleaned to remove all loosened debris to ensure that the concrete is in intimate contact with the competent bearing stratum. The base of the caissons will need to be inspected immediately prior to placing concrete to ensure that the above procedures have been followed. The stabilized groundwater level is at about Elevation 166.4 m, and although groundwater inflow from the till is expected to be minimal, it is anticipated that there could be some groundwater seepage through the fractures in the bedrock, particularly in the upper portion of the bedrock. A temporary or permanent liner would therefore be required to support the excavation walls during construction, and to permit inspection and cleaning of the caisson base. The liner must be maintained tight to the sides of the bore to minimize seepage of water. In addition, the Ontario Occupational Health and Safety Act (2007) outlines appropriate safety procedures and requirements that must be implemented prior to entry of personnel into the caissons.

If there is water infiltration such that there is standing water within the caisson excavation prior to concrete placement, the concrete must be placed using tremie techniques. That is, the concrete must be discharged at the base of the caisson excavations, and flow upward to the ground surface. The tremie discharge should be maintained a minimum of 1 m below the surface of the wet concrete during placement.

6.4.1 Geotechnical Axial Resistance

Recommendations are provided below for factored geotechnical axial resistances at ULS and geotechnical resistances at SLS (for 25 mm of settlement) for caissons founded within the sand and silt till at Elevation 163.0 m at the north and south abutments. Recommendations are also given below for the factored geotechnical axial resistances at ULS for caissons founded on the shale bedrock at Elevation 158.5 m. For



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caissons founded on bedrock the geotechnical resistance at SLS for 25 mm of settlement will be greater than the factored geotechnical axial resistance at ULS, and as a result the SLS condition does not apply.

Founding Stratum	Caisson Diameter	Factored Geotechnical Resistance at ULS	Geotechnical Resistance at SLS
Very Dense Sand and Silt Till	0.9 m	1,900 kN	1,600 kN
	1.2 m	3,400 kN	2,900 kN
	1.5 m	5,400 kN	3,500 kN
Shale Bedrock	0.9 m	2,800 kN	N/A
	1.2 m	4,900 kN	
	1.5 m	7,700 kN	

6.4.2 Resistance to Lateral Loads

The resistance to lateral loading in front of a caisson may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction (k_h) is determined based on the equations given below (CFEM 1992 as noted in Section 6.8.7.1 of the *Commentary to the CHBDC, 2006*):

For cohesionless soils:

$$k_h = \frac{n_h z}{B} \quad \text{where}$$

k_h is the coefficient of horizontal subgrade reaction (kPa/m);
 n_h is the constant of subgrade reaction (kPa/m);
 z is the depth (m); and
 B is the caisson diameter / width (m).

For cohesive soils:

$$k_h = \frac{67s_u}{B} \quad \text{where}$$

k_h is the coefficient of horizontal subgrade reaction (kPa/m);
 s_u is the undrained shear strength of the soil (kPa); and
 B is the caisson diameter / width (m).

The following for the value of n_h and s_u may be assumed in the structural analyses, based on the stratigraphy conditions shown on Drawing 1.

Foundation Element	Soil Unit	n_h (kPa/m)	s_u (kPa)
North and South Abutment	Clayey Silt Till	-	175
	Sand and Silt Till	11,000	-



6.4.3 Frost Protection

The caisson caps should be provided with a minimum of 1.2 m of soil cover for frost protection (OPSD 3090.101) (Foundation Frost Depths for Southern Ontario).

6.5 Seismic Site Coefficient

The peak zonal acceleration ratio is 0.05g for the City of Mississauga, Ontario (CHBDC Table A3.1.1). The Site Coefficient (S) may be taken as 1.2, consistent with Soil Profile Type II in accordance with Section 4.4.6 and Table 4.4 of the CHBDC (2006).

6.6 Lateral Earth Pressures

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

The following recommendations are made concerning the design of the walls. It should be noted that 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 Special Provision 110S13 (Material Specification for Aggregates) Granular 'A' or Granular 'B' Type II but with less than 5 percent passing the 200 sieve should be used as backfill behind the walls. Transverse drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub drains and frost taper should be in accordance with OPSD 3101.150 (Abutment Walls, Backfill) and OPSD 3121.150 (Retaining Walls, Backfill).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with CHBDC Section 6.9.3 and Figure 6.6. Compaction equipment should be used in accordance with MTO's Special Provision 105S10 (Compaction). Other surcharge loadings should be accounted for in the design as required.
- The granular fill may be placed either in a zone with the width equal to at least 1.2 m behind the back of the walls (Case I on Figure C6.20(a) of the *Commentary* to the CHBDC), or within the wedge shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (see Case II on Figure C6.20(b) of the *Commentary* to the CHBDC).
- For Case I, the pressures are based on the existing embankment fill materials and the following parameters (unfactored) may be used:



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	Earth Fill
Soil unit weight:	20 kN/m ³
Coefficients of static lateral earth pressure:	
Active, K_a	0.33
At rest, K_o	0.50

- For Case II, the pressures are based on the granular fill as placed, and the following parameters (unfactored) may be assumed:

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

If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.9.1 and Table C6.6 of the *Commentary* to the CHBDC.

If the abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design. A restrained structure is typically a concrete box culvert or a rigid frame bridge structure where the rotational and/or horizontal movement is not sufficient to mobilize the active pressure condition. For these types of structures, an at-rest pressure plus any compaction surcharge should be included in the design of the structure.

6.6.1 Seismic Considerations

Seismic (earthquake) loading must also be taken into account in the design in accordance with Section 4.6 of the CHBDC, as significant seismic loading will result in increased lateral earth pressures acting on the abutment stem, wing walls and retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the applicable earthquake-induced dynamic earth pressure. The earthquake-induced dynamic pressure distribution is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e. an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:



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$$P = K \gamma' d + (K_{AE} - K) \gamma' H$$

where

K	is either the static active earth pressure coefficient (K_a) or the static at-rest earth pressure coefficient (K_o);
K_{AE}	is the seismic active earth pressure coefficient;
γ'	is the effective unit weight of the soil (kN/m^3)
	<ul style="list-style-type: none">• taken as soil unit weights given above for fill materials• taken as 21 kN/m^3 for the native materials
d	is the depth below the top of the wall (m); and
H	is the height of the wall above the toe (m).

According to Table C4.2 of the *Commentary* to the *CHBDC*, this site is located in Seismic Zone 1. The site specific zonal acceleration ratio for Mississauga is 0.05. For the thickness and competent overburden soils encountered at this site, an amplification factor equal to 1.2 for the ground motion is recommended for design. As such, the recommended ground surface acceleration will increase to $0.06g$.

The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of $A = 0.06$. These coefficients have been determined in accordance with Sections 4.6.4 and C4.6.4 of the *CHBDC* and its *Commentary*, and assume that the back of the wall is vertical and the ground surface behind the wall is flat.

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Case I	Case II	
	Earth Fill (Granular Material)	Granular 'A'	Granular 'B' Type II
Yielding wall	0.32	0.26	0.26
Non-yielding wall	0.36	0.30	0.30

Note : These *CHBDC* seismic K_{AE} values include the effect of wall friction ($\delta=\phi'/2$) and are less than the static values of K_a and K_o reported above for the very low zonal acceleration ratio for this site.

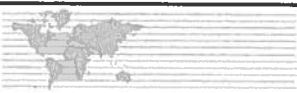
The above K_{AE} values for yielding walls are applicable provided that the wall can move up to $250A$ (mm), where A is the design zonal acceleration ratio of 0.06. This corresponds to displacements of up to 15 mm at this site.

6.7 Retained Soil System (RSS) Walls

It is understood that mechanically-reinforced soil retaining systems (retained soil system or RSS walls) are proposed as wing walls/retaining walls adjacent to each abutment to retain the Glen Erin Drive embankment, north and south of the GO BRT busway.

6.7.1 Founding Elevations

A typical RSS wall has a front facing supported on a strip footing placed at shallow depth below the ground surface in front of the wall. The fill within the existing embankment at Glen Erin Drive extends to about between Elevation 166.4 m to 168.1 m. The footing and the RSS mass should be founded below any topsoil, loose fill or unsuitable native soils. Consideration could be given to subexcavating the existing fill material within the RSS



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wall footprint, for at least the portions of the wing wall adjacent to the abutments. The following subexcavation elevations may be assumed for this case.

Wall Location	Subexcavation Elevation
Southwest Retaining Wall	166.5 m
Southeast Retaining Wall	166.6 m
Northwest Retaining Wall	168.0 m
Northeast Retaining Wall	166.4 m

The facing footing and reinforced soil mass can be constructed immediately on top of the subgrade exposed at the above-noted elevations. Alternatively, the subexcavated soil can be replaced with compacted OPSS 1010 Granular A or Granular B Type II fill prior to construction of the facing footing and reinforced soil mass. This compacted granular pad should extend at least 1.0 m beyond the outside edge of both sides of the facing footing, then outward/downward at 1 horizontal to 1 vertical (1H:1V).

Since the height of the RSS wall decreases with increasing distance away from the bridge abutments, consideration could be given to partial subexcavation of the fill to 1.5 m below the proposed underside of the facing footing for the RSS wall and providing a 1.5 m thick granular pad constructed of compacted OPSS 1010 Granular A or Granular B Type II fill. This compacted granular pad should extend at least 1.0 m beyond the outside edge of both sides of the facing footing, then outward/downward at 1 horizontal to 1 vertical (1H:1V). The compacted Granular A or Granular B Type II as well as the reinforced soil mass should be keyed into the existing embankment by benching into the embankment fill, as per OPSD 208.010.

6.7.2 Global Stability

The static and seismic global slope stability of RSS walls adjacent to the GO BRT structure has been analyzed using the commercially available program SLIDE, produced by Rocscience Inc., employing the Morgenstern-Price method of analysis. For all analyses, the factor of safety of numerous potential failure surfaces was computed in order to establish the minimum Factor of Safety. A target Factor of Safety of 1.3 against deep-seated global instability of the RSS walls is normally used for the design under static conditions; under seismic conditions, a target Factor of Safety of 1.1 is used. These Factors of Safety are considered appropriate for the RSS walls at this site, considering the design requirements and the field data available.

The soil parameters used in the analysis, as given in the following table, were estimated from empirical correlations using the results of in situ Standard Penetration Tests (SPTs) and geotechnical classification testing. The groundwater table was taken at Elevation 166.5 m in the analyses.

Soil Type	Unit Weight (kN/m ³)	Undrained Shear Strength (kPa)	Cohesion, c' (kPa)	Angle of Internal Friction, ϕ' (degrees)
Existing Embankment Fill	20	—	—	30
Very Stiff to Hard Clayey Silt Till	21	150 kPa	—	32
Very Dense Sand and Silt Till	21	—	—	35



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Three RSS wall sections were analyzed for the different wall heights as shown on the drawings provided by Giffles/IBI, dated November 18, 2009 as shown on Figures 9, 10 and 11. The analysis was carried out using a minimum of 0.8 m of cover over the facing footing and a 2H:1V embankment side slope above the wall. If the actual wall configuration differs from this, further stability analyses should be carried out with the actual proposed wall configuration since the results are sensitive to the buried depth of wall and the presence of the 2H:1V embankment side slope at the base of the wall.

Based on the analysis results, the following provides the minimum reinforced width of RSS wall required to obtain a Factor of Safety of 1.3 or greater against deep-seated global instability. The results indicate that the length of reinforcing strips/reinforced soil mass width must be greater than would normally be used (typically a minimum of 0.7 times the height of the wall) for the 4.5 m and 3 m high walls. This means that the contract drawings will need to specify the reinforced soil mass width. It is noted that for the 6.0 m high wall, the ratio of minimum reinforced width to wall height to satisfy a Factor of Safety of 1.3 is actually less than 0.7; however, the analysis for the normal 0.7 length to height ratio is presented for the stability analysis.

Wall Height	Minimum Required Reinforced Width	Ratio of Wall Height to Minimum Reinforced Width
3.0 m	2.7 m	0.90
4.5 m	3.5 m	0.78
6.0 m	4.2 m	0.70

Under seismic loading conditions, using a seismic coefficient taken as 50 percent of the site-specific design peak horizontal ground acceleration equal to 0.03g, the Factor of Safety is greater than 1.1. The results of example seismic slope stability analyses for the three cases are shown on Figures 12, 13 and 14.

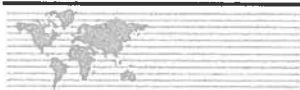
6.7.3 Geotechnical Resistance

Assuming that the RSS wall acts as a unit and utilizes the full width of the reinforced soil mass, as specified in Section 6.7.2 the factored geotechnical resistances at ULS and the geotechnical resistances at SLS (for 25 mm of settlement) given below may be used for assessment of the reinforced mass founded on the properly prepared compacted granular fill or on the very stiff to hard clayey silt till deposit are given below.

Wall Height	Minimum Reinforced Width	Factored Geotechnical Resistance at ULS	Geotechnical Resistance at SLS
3.0 m	2.7 m	175 kPa	125 kPa
4.5 m	3.5 m	250 kPa	150 kPa
6.0 m	4.2 m	350 kPa	225 kPa

6.7.4 Resistance to Lateral Loads

The resistance to lateral forces / sliding resistance between the compacted backfill and the subgrade should be calculated in accordance with Section 6.7.5 of the CHBDC. The coefficient of friction, $\tan \phi'$, between the compacted granular fills of the RSS wall and the properly prepared subgrade may be taken as 0.55.



6.8 Construction Considerations

6.8.1 Open Cut Excavation

Depending on the foundation option adopted, excavation to about Elevation 165.8 m for the bridge foundations (including RSS wall foundations) will extend to depths of up to 8 m below the existing grade along Glen Erin Drive. The foundation excavations will extend through the existing embankment fill and into the clayey silt till deposit. Where space permits, open-cut excavations into these materials should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. The existing fill materials are classified as Type 3 soil and the till is classified as Type 2 soil, according to the OHSA. Temporary excavations (i.e. those which are open for a relatively short time period) should be made with side slopes no steeper than 1 horizontal to 1 vertical. All excavations must be carried out in accordance with the latest edition of the OHSA.

6.8.2 Temporary Excavation Protection

It is expected that temporary excavation support will be required to maintain traffic lanes on Glen Erin Drive during construction of the new abutments and retaining walls. The following soil parameters may be used for the design of the temporary support system (in accordance with OPSS 539 – Construction Specification for Temporary Protection Systems) assuming the system will be retaining the existing embankment fill; surcharge loads must be included in the design:

Unit Weight of Soil, $\gamma = 20 \text{ kN/m}^3$ and

Earth Pressure Coefficient, $K = 0.36$

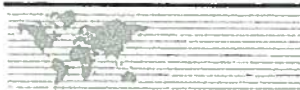
6.8.3 Control of Groundwater and Surface Water

The groundwater level was measured in one standpipe piezometer at the site at Elevation 166.6 m, which is about 7.3 m below the existing ground surface along Glen Erin Drive (about Elevation 174 m) and between about 1.0 m to 1.5 m below the general ground surface at the base of the Glen Erin Drive embankment. Therefore, it is expected that for spread footings founded on the very stiff to hard silty clay till at about Elevation 165.8 m, the excavations will be about 0.6 m below the groundwater level. Considering the fine-grained nature of the clayey silt till it is anticipated that the groundwater inflow to the excavations will be minimal and that the water inflow can be handled by pumping from filtered sump pumps, placed at the base of the excavation and outside the footprint of the foundations.

6.8.4 Obstructions

The fill materials, clayey silt till and sand and silt till soils at the site contain cobbles and boulders as was inferred from grinding of the augers during borehole drilling (see Record of Borehole sheets for depths and elevations).

Conventional excavation equipment should be suitable for the majority of the shallow excavation work at the site. The presence of cobbles and boulders within the fill and till deposits may, however, affect the excavation works, the installation of soldier piles for the temporary excavation protection and the installation of caissons if that foundation option is adopted. It is recommended that this be identified in the Contract Document to warn the Contractor of these obstructions and to ensure that the Contractor is equipped to handle such obstructions.



FOUNDATION REPORT GO BRT BRIDGE AT GLEN ERIN DRIVE

7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Sandra McGaghran, P.Eng., a Geotechnical Engineer with Golder and reviewed by Ms. Anne Poschmann, P.Eng., a Principal and Geotechnical Engineer with Golder. Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project and Principal with Golder, conducted an independent quality control review of the report.

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SMM/ASP/JMAC/jl

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Ontario Provincial Standard Specifications (OPSS)

OPSS 539 Construction Specification for Temporary Protection Systems

OPSS 1010 Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

OPSD 208.010 Benching of Earth Slopes

Ontario Provincial Standard Drawings (OPSD)

OPSD 208.010 Benching of Earth Slopes

OPSD 3090.101 Foundation Frost Depths for Southern Ontario

OPSD 3101.150 Abutment Walls, Backfill – Minimum Granular Requirements

OPSD 3121.150 Retaining Walls, Backfill – Minimum Granular Requirements

MTO Special Provisions (SP)

SP 105S10 Amendment to OPSS 501 – Construction Specification for Compaction

SP 110S13 Amendment to OPSS 1010 – Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material



IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

Standard of Care: Golder Associates Ltd. (Golder) has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently practising under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report. No other warranty, expressed or implied is made.

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The information, recommendations and opinions expressed in this report are for the sole benefit of the Client. No other party may use or rely on this report or any portion thereof without Golder's express written consent. If the report was prepared to be included for a specific permit application process, then upon the reasonable request of the client, Golder may authorize in writing the use of this report by the regulatory agency as an Approved User for the specific and identified purpose of the applicable permit review process. Any other use of this report by others is prohibited and is without responsibility to Golder. The report, all plans, data, drawings and other documents as well as all electronic media prepared by Golder are considered its professional work product and shall remain the copyright property of Golder, who authorizes only the Client and Approved Users to make copies of the report, but only in such quantities as are reasonably necessary for the use of the report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make available the report or any portion thereof to any other party without the express written permission of Golder. The Client acknowledges that electronic media is susceptible to unauthorized modification, deterioration and incompatibility and therefore the Client can not rely upon the electronic media versions of Golder's report or other work products.

The report is of a summary nature and is not intended to stand alone without reference to the instructions given to Golder by the Client, communications between Golder and the Client, and to any other reports prepared by Golder for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. Golder can not be responsible for use of portions of the report without reference to the entire report.

Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

Soil, Rock and Groundwater Conditions: Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, Golder does not warrant or guarantee the exactness of the descriptions.



IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that Golder interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report. The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

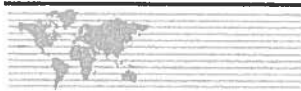
Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

Follow-Up and Construction Services: All details of the design were not known at the time of submission of Golder's report. Golder should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Golder's report.

During construction, Golder should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Golder's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in Golder's report. Adequate field review, observation and testing during construction are necessary for Golder to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, Golder's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

Changed Conditions and Drainage: Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that Golder be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that Golder be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. Golder takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.



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
F	factor of safety
V	volume
W	weight

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 ($\sigma' = \sigma - \mu$)
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
μ	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

(a) Index Properties (continued)

w	water content
w_l	liquid limit
w_p	plastic limit
I_p	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_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation pressure
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

T_p, T_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

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

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



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

III. SOIL DESCRIPTION

(a) Cohesionless 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

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.)

(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

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.

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.

PROJECT 09-1181-1045

RECORD OF BOREHOLE No 5001

1 OF 2 **METRIC**

G.W.P.

LOCATION N 4823283.9 ; E 288377.9

ORIGINATED BY RY

DIST HWY 403

BOREHOLE TYPE 200 mm Outside Diameter Hollow Stem Augers

COMPILED BY AH

DATUM Geodetic

DATE January 28, 2010

CHECKED BY SMM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED 20 40 60 80 100	+ FIELD VANE 20 40 60 80 100						
174.2	GROUND SURFACE														
0.0	Clayey silt, some sand, trace to some gravel, containing shale fragments and rootlets (FILL) Firm to hard Brown and grey Moist		1	SS	19		174								
			2	SS	15		173								
			3	SS	8		172								
			4	SS	7		171								
			5	SS	6		170								
			6	SS	19		169								
			7	SS	16		168								
	Augers grinding between 5.8 m and 6.1 m depth		8	SS	15		167								
167.0							166								
7.2	CLAYEY SILT, some sand, trace to some gravel (TILL) Hard Brown Moist		9	SS	41		165								
			10	SS	35		164								
			11A	SS	63		163								
163.1			11B				162								
11.1	SAND and SILT, trace to some clay, some gravel (TILL) Very dense Grey Moist		12	SS	100/10		161								
			13	SS	100/10		160								

MIS-MTO 001 0911811045.GPJ GAL-MISS.GDT 13/4/10 JFC

Continued Next Page

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



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

PROJECT 09-1181-1045		RECORD OF BOREHOLE No 5002		1 OF 2 METRIC	
G.W.P.		LOCATION N 4823294.4 ; E 288368.8		ORIGINATED BY VZ	
DIST HWY 403		BOREHOLE TYPE 200 mm Outside Diameter Hollow Stem Augers		COMPILED BY AH	
DATUM Geodetic		DATE January 27, 2010		CHECKED BY SMM	

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 (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40						60	80
173.9	GROUND SURFACE															
8.9	TOPSOIL		1	SS	10											
	Clayey silt, some sand, trace to some gravel, containing shale fragments and topsoil (FILL) Firm to stiff Grey, reddish brown and black Moist		2	SS	6											
			3	SS	8											
			4	SS	6											
			5	SS	11											
	Augers grinding at 4.3 m depth		6	SS	12											
			7	SS	13											
166.7	CLAYEY SILT, some sand, trace to some gravel (TILL) Very stiff Brown Moist		8	SS	26											
165.4	SAND and SILT, trace to some gravel and clay (TILL) Very dense Brown becoming grey between 10.7 m and 12.2 m depth Moist Augers grinding between 8.5 m and 11.6 m depth		9	SS	68/15											
			10	SS	50/1											
	Augers grinding at 13.6 m depth		11	SS	50/05											
160.1	END OF BOREHOLE See Page 2 for Notes		12	SS	50/05											

MIS-MTO 001 0911811045.GPJ GAL-MISS.GDT 13/4/10 JFC

Continued Next Page

+³, X³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

MIS-MTO 001 0911811045.GPJ GAL-MISS.GDT 13/4/10 JFC

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

PROJECT 09-1181-1045

RECORD OF BOREHOLE No 5003

1 OF 2 **METRIC**

G.W.P.

LOCATION N 4823303.9 ; E 288357.8

ORIGINATED BY MWK

DIST HWY 403

BOREHOLE TYPE 200 mm Outside Diameter Hollow Stem Augers

COMPILED BY AH

DATUM Geodetic

DATE January 28, 2010

CHECKED BY SMM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × REMOULDED					
173.7	GROUND SURFACE						20 40 60 80 100							
0.0	Sand and gravel, trace silt (FILL) Compact Brown Moist		1	SS	10				○					
173.1	Clayey silt to silty clay, trace to some sand and gravel, containing rootlets (FILL) Firm to very stiff Reddish brown and grey Moist		2	SS	10					○				
0.6			3	SS	5					○				
			4	SS	6					○				
			5	SS	9					○				
			6	SS	13					○				
			7	SS	23					○				
168.1														
5.6	CLAYEY SILT, some sand, trace gravel (TILL) Very stiff to hard Brown Moist		8	SS	20					○				
			9	SS	31					○				
165.1														
8.6	SAND and SILT, trace to some gravel and clay, containing cobbles (TILL) Very dense Brown becoming grey between 9.8 m and 10.7 m Moist		10	SS	122					○				
	Augers grinding between 10.1 m and 10.4 m depth		11	SS	68					○				
			12	SS	191/28					○				
160.0														
13.7	CLAYEY SILT, trace to some sand, containing shale fragments (RESIDUAL SOIL) Hard Reddish brown Moist		13	SS	100/13					○				

Continued Next Page

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

MIS-MTO 001 0911811045.GPJ GAL-MISS.GDT 13/4/10 JFC

PROJECT 09-1181-1045

RECORD OF BOREHOLE No 5003

2 OF 2 **METRIC**

G.W.P. LOCATION N 4823303.9 ; E 288357.8

ORIGINATED BY MWK

DIST HWY 403 BOREHOLE TYPE 200 mm Outside Diameter Hollow Stem Augers

COMPILED BY AH

DATUM Geodetic DATE January 28, 2010

CHECKED BY SMM

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						
	— CONTINUED FROM PREVIOUS PAGE —							20 40 60 80 100						
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED						
								20 40 60 80 100						

+³, X³: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

PROJECT 09-1181-1045		RECORD OF BOREHOLE No 5004		1 OF 2 METRIC	
G.W.P.		LOCATION N 4823309.8 ; E 288348.3		ORIGINATED BY RS	
DIST HWY 403		BOREHOLE TYPE 200 mm Outside Diameter Hollow Stem Augers		COMPILED BY AH	
DATUM Geodetic		DATE January 27, 2010		CHECKED BY SMM	

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						
						20	40	60	80	100				
173.7	GROUND SURFACE													
0.0	TOPSOIL													
173.2	Stiff Dark brown Moist		1	SS	10									
0.7	Sand and gravel, trace silt (FILL) Compact Brown Moist		2	SS	9									
	Clayey silt, trace to some sand and gravel (FILL) Firm to very stiff Grey to reddish brown Moist		3	SS	6									
			4	SS	9									
			5	SS	7									
	Augers grinding between 3.7 m to 4.0 m depth		6	SS	12									
			7	SS	25									
168.1														
5.6	CLAYEY SILT, some sand, trace gravel, containing cobbles (TILL) Stiff to very stiff Reddish brown Moist		8	SS	14									
			9	SS	26									
164.6	Augers grinding at 8.8 m depth													
9.1	SAND and SILT, trace to some gravel, trace clay (TILL) Very dense Brown becoming grey between 9.9 m and 10.7 m depth Moist		10	SS	60/13									
			11	SS	69									
			12	SS	46									
			13	SS	114/13									
159.4														
14.3	SHALE (BEDROCK) Reddish brown													

MIS-MTO 001 0911811045.GPJ GAL-MISS.GDT 13/4/10 JFC

Continued Next Page

+³, X³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

PROJECT 09-1181-1045

RECORD OF BOREHOLE No 5004

2 OF 2 **METRIC**

G.W.P. LOCATION N 4823309.8 ; E 288348.3


ORIGINATED BY RS

DIST HWY 403 BOREHOLE TYPE 200 mm Outside Diameter Hollow Stem Augers

COMPILED BY AH

DATUM Geodetic DATE January 27, 2010

CHECKED BY SMM

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									
							20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%) 10 20 30					
	— CONTINUED FROM PREVIOUS PAGE —																
	SHALE (BEDROCK) Reddish brown		14	SS	66/13		158										
156.8																	
16.9	END OF BOREHOLE NOTE: 1. Open borehole dry upon completion of drilling.		15	SS	55/13		157										

MIS-MTO 001 0911811045.GPJ GAL-MISS.GDT 13/4/10 JFC

PROJECT 09-1181-1045

RECORD OF BOREHOLE No 5005

1 OF 1 **METRIC**

G.W.P. LOCATION N 4823325.0 ; E 288359.2

ORIGINATED BY MWK

DIST HWY 403 BOREHOLE TYPE 200 mm Outside Diameter Hollow Stem Augers

COMPILED BY AH

DATUM Geodetic DATE January 28, 2010

CHECKED BY SMM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL						
173.6	GROUND SURFACE						20	40	60	80	100					
8.9	TOPSOIL															
	Clayey silt, some sand, some gravel, containing shale and limestone fragments (FILL) Firm to very stiff Reddish brown and grey Moist		1	SS	9								○			
			2	SS	5								○			
			3	SS	7								○			
			4	SS	8								○			
			5	SS	25								○			
	Augers grinding between 4.3 m and 4.6 m depth		6	SS	13								○	┌───┐	23 11 41 26	
			7	SS	18								○			
			8	SS	18								○			
166.4																
7.2	CLAYEY SILT, some sand, trace to some gravel (TILL) Very stiff Brown Moist		9	SS	27								○	┌───┐	8 21 49 22	
164.9																
8.7	SAND and SILT, some gravel, trace clay (TILL) Very dense Brown to grey Moist		10	SS	194/25								○			
			11	SS	100/13								○		18 42 35 5	
161.9																
11.7	SHALE (BEDROCK) Reddish brown															
161.1			12	SS	100/15											
12.5	END OF BOREHOLE															
	NOTE: 1. Borehole open to 11.4 m depth; water level in borehole at 5.5 m depth (Elev. 168.1 m) upon completion of drilling.															

MIS-MTO 001 0911811045.GPJ GAL-MISS.GDT 13/4/10 JFC

PROJECT 09-1181-1045

RECORD OF BOREHOLE No 5006

1 OF 1 **METRIC**

G.W.P.

LOCATION

N 4823315.4 ; E 288367.2

ORIGINATED BY MWK

DIST

HWY 403

BOREHOLE TYPE

200 mm Outside Diameter Hollow Stem Augers

COMPILED BY AH

DATUM Geodetic

DATE

January 29, 2010

CHECKED BY SMM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)				
								○ UNCONFINED + FIELD VANE								○				
								● QUICK TRIAXIAL × REMOULDED												
173.8	GROUND SURFACE						20	40	60	80	100	10	20	30						
173.8	TOPSOIL																			
	Silty clay, trace to some sand, some gravel (FILL) Firm to very stiff Reddish brown and grey Moist		1	SS	12															
			2	SS	8															
			3	SS	6															
			4	SS	9															
			5	SS	12															
			6	SS	13															
			7	SS	24															
			8	SS	17															
166.6	CLAYEY SILT, some sand, trace gravel (TILL) Very stiff Reddish brown Moist		9	SS	25															
165.1	SAND and SILT, trace to some gravel and clay (TILL) Very dense Grey Moist		10	SS	54															
			11	SS	164/25															
			12	SS	100/08															
160.5	SHALES (BEDROCK) Reddish brown		13	SS	100/0.1															
160.0	END OF BOREHOLE																			
13.8	NOTE: 1. Water level in open borehole at 7.0 m depth (Elev. 166.8 m) upon completion of drilling.																			

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

MIS-MTO 001 0911811045.GPJ GAL-MISS.GDT 13/4/10 JFC

PROJECT 09-1181-1045 **RECORD OF BOREHOLE No 5007** 1 OF 2 **METRIC**
 G.W.P. _____ LOCATION N 4823305.6 ; E 288378.4 ORIGINATED BY RY
 DIST _____ HWY 403 BOREHOLE TYPE 200 mm Outside Diameter Hollow Stem Augers COMPILED BY AH
 DATUM Geodetic DATE January 29, 2010 CHECKED BY SMM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL							× REMOULDED	
174.0	GROUND SURFACE						20	40	60	80	100							
0.0	TOPSOIL						20	40	60	80	100							
173.7																		
0.3	Sand and gravel, trace to some silt (FILL) Compact Brown Moist		1	SS	10								○					
172.6			2	SS	11								○					
1.4	Clayey silt, some sand, some gravel, containing shale fragments and rootlets (FILL) Firm to hard Reddish brown and grey Moist		3	SS	5								○	100	3 14 53 30			
			4	SS	6								○					
			5	SS	9								○					
			6	SS	13								○					
			7	SS	11								○					
			8	SS	35								○					
166.8	Augers grinding between 6.7 m and 7.3 m depth																	
7.2	CLAYEY SILT, some sand, some gravel (TILL) Hard Reddish brown Moist		9	SS	30								○	100	2 20 55 23			
165.3																		
8.7	SAND and SILT, trace to some gravel and clay, containing silt and clayey silt pockets, containing cobbles (TILL) Very dense Brown becoming grey at 10.5 m depth Moist Augers grinding at 9.1 m depth		10	SS	100								○		11 46 37 6			
			11	SS	100/0.1								○		4 31 53 12			
	Augers grinding at 11.9 m depth		12	SS	100/0.05								○					
160.9																		
13.1	CLAYEY SILT, some sand, some gravel, containing shale fragments (RESIDUAL SOIL) Hard Reddish brown Moist		13	SS	100/13								○					
159.4																		
14.6	SHALE (BEDROCK) Reddish brown																	

MIS-MTO 001 0911811045.GPJ GAL-MISS.GDT 13/4/10 JFC

Continued Next Page

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

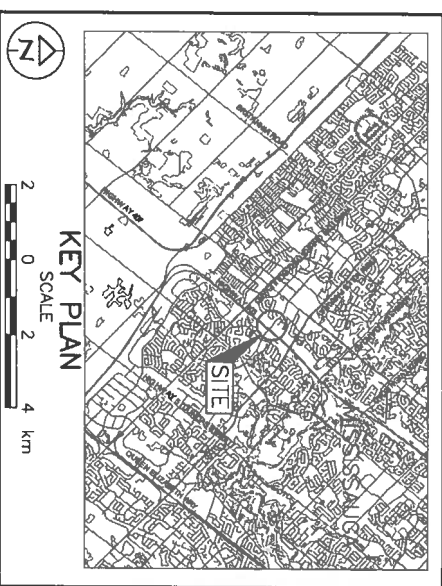
PROJECT <u>09-1181-1045</u>		RECORD OF BOREHOLE No 5007				2 OF 2 METRIC										
G.W.P. _____		LOCATION <u>N 4823305.6 ; E 288378.4</u>				ORIGINATED BY <u>RY</u>										
DIST _____ HWY <u>403</u>		BOREHOLE TYPE <u>200 mm Outside Diameter Hollow Stem Augers</u>				COMPILED BY <u>AH</u>										
DATUM <u>Geodetic</u>		DATE <u>January 29, 2010</u>				CHECKED BY <u>SMM</u>										
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 Y kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								
	-- CONTINUED FROM PREVIOUS PAGE --						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100				WATER CONTENT (%) 10 20 30					
158.9 15.1	END OF BOREHOLE NOTE: 1. Borehole open and dry upon completion of drilling.	✓	14	SS	1007.00											

PROJECT 09-1181-1045		RECORD OF BOREHOLE No 5008		1 OF 1 METRIC						
G.W.P. _____		LOCATION N 4823298.8 :E 288387.6		ORIGINATED BY RY						
DIST _____ HWY 403		BOREHOLE TYPE 200 mm Outside Diameter Hollow Stem Augers		COMPILED BY AH						
DATUM Geodetic		DATE January 29, 2010		CHECKED BY SMM						
SOIL PROFILE			SAMPLES		DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE						
174.2	GROUND SURFACE									
0.0	Sand and gravel, containing topsoil and pockets of clayey silt (FILL)		1	SS	21					
173.6	Compact Brown Moist		2	SS	20					
0.6	Silty clay, trace to some sand, some gravel, containing shale and limestone fragments (FILL)		3	SS	8					
	Firm to hard Reddish brown and grey Moist		4	SS	6					
			5	SS	7					
			6	SS	41					
			7	SS	13					
	Augers grinding at 4.9 m depth		8	SS	15					
167.0	CLAYEY SILT, some sand, some gravel (TILL)		9	SS	30					
7.2	Hard Reddish brown Moist		10	SS	127					
165.5	SAND and SILT, some gravel, trace to some clay, containing cobbles (TILL)		11	SS	100/1					
8.7	Very dense Brown becoming grey at 12.0 m depth Moist to wet		12	SS	131					
	Augers grinding at 11.3 m depth									
161.4	END OF BOREHOLE									
12.8	NOTE: 1. Borehole open and dry upon completion of drilling.									

CONT No.
WP No.

GO TRANSIT BUS RAPID TRANSIT W.
GLEN ERIN DRIVE
BOREHOLE LOCATION AND SOIL STRATA

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



LEGEND

- Borehole - Current Investigation
- N Standard Penetration Test Value
- 18 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL upon completion of drilling

BOREHOLE CO-ORDINATES		
No.	ELEVATION	EASTING
5001	174.2	4823283.9
5002	173.9	4823294.4
5003	173.7	4823303.9
5004	173.7	4823309.8
5005	173.6	4823325.0
5006	173.8	4823315.4
5007	174.0	4823305.6
5008	174.2	4823298.8

NOTES

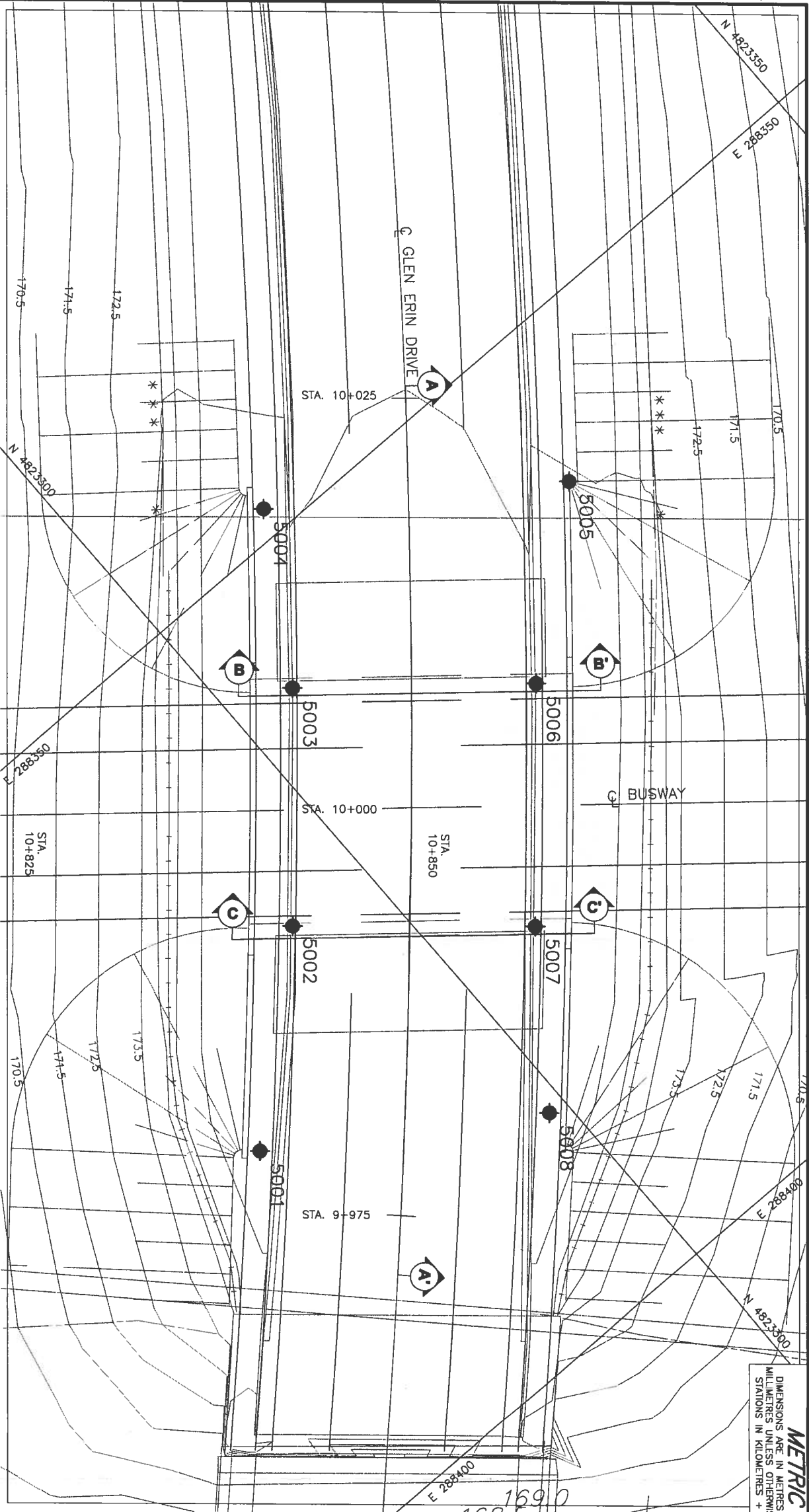
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contract Documents.

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

REFERENCE

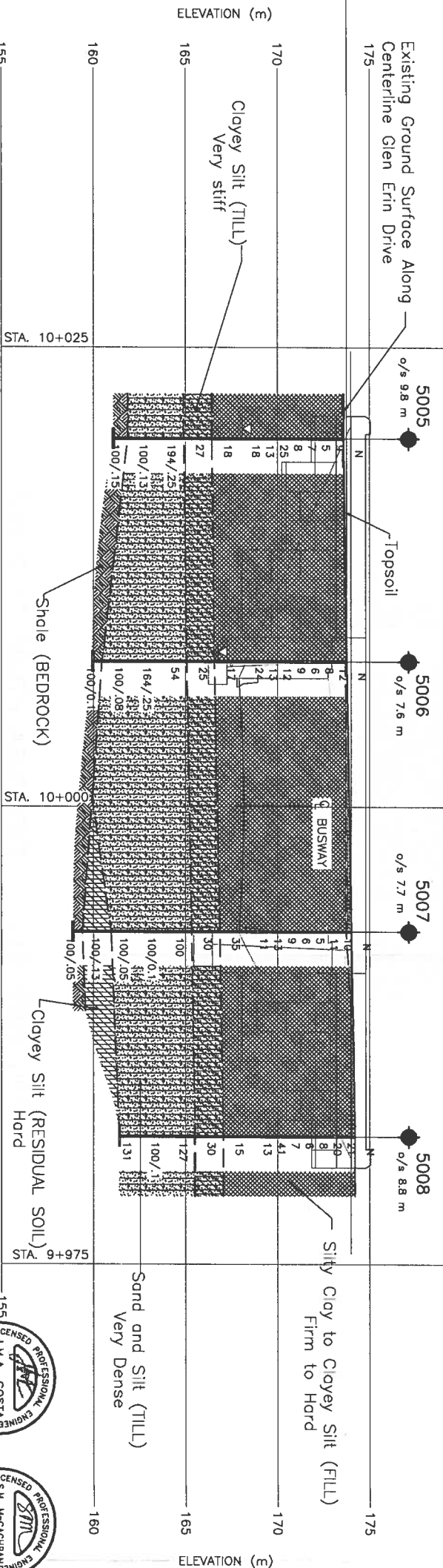
Base plan provided in digital format by Giffels/IB Group, drawing file no. 069770-B005.dwg, drawn dated November, 16, 2009.

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



PLAN

SCALE 3 0 3 6 m

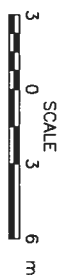


CENTRELINE PROFILE A-A' - GLEN ERIN DRIVE

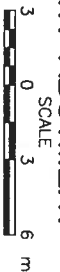
SCALE 3 0 3 6 m

Licensed Professional Engineer
J.M.A. COSTA
April 13, 2010
PROVINCE OF ONTARIO

Licensed Professional Engineer
S.M. McLaughlin
April 13, 2010
PROVINCE OF ONTARIO



A horizontal scale bar labeled "SCALE" with markings at 3, 0, 3, and 6 m.



SCALE
3 0 3 6 m

CONT No.
WP No.

GLEN ERIN DRIVE
SOIL STRATA

Goldier Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



LEGEND

- | | Seal | Piezometer | Standard Penetration Test Value | Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 i/blow) | WL in piezometer, measured on April 12, 2010 | WL upon completion of drilling |
|----|------|------------|---------------------------------|---|--|--------------------------------|
| 16 | | | | | | |

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
5002	173.9	4823294.4	288366.8
5003	173.7	4823303.9	288357.8
5006	173.8	4823315.4	288367.2
5007	174.0	4823305.6	288378.4

NOTES

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

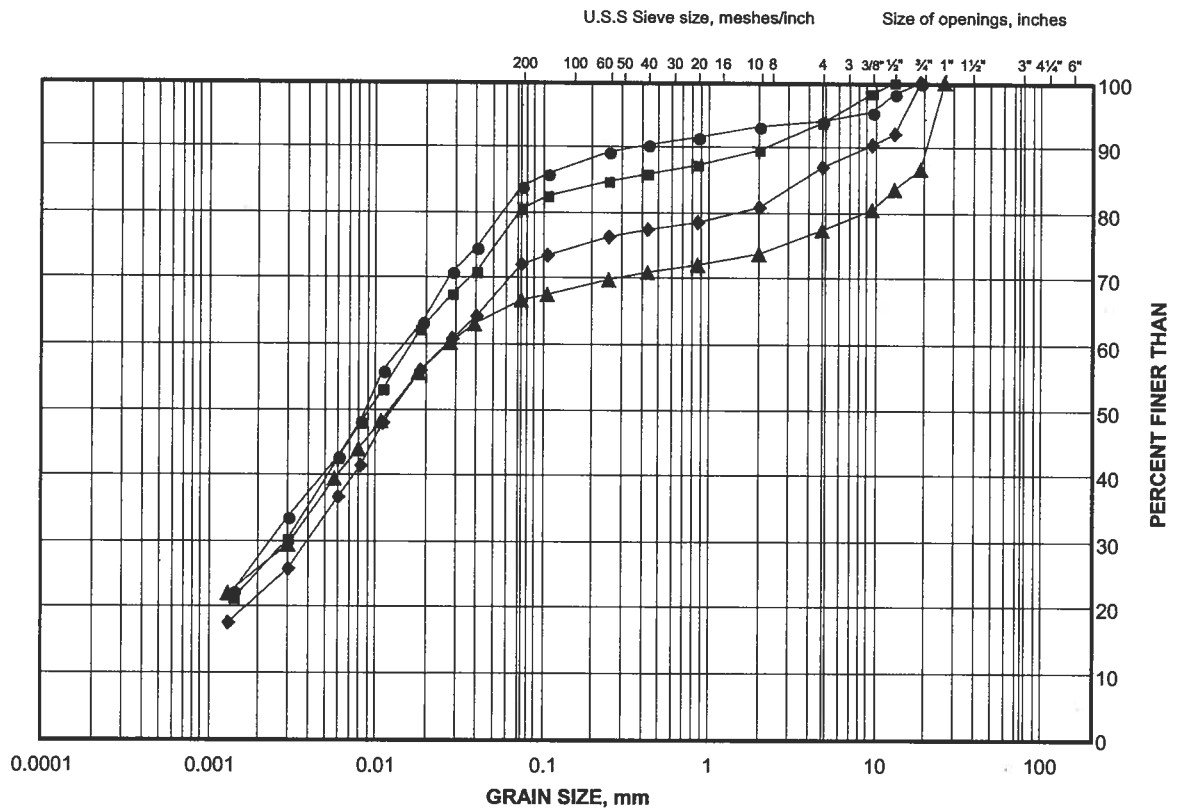


Georges No.		PROJECT NO.09-1181-1045-1		DIST.	
IMV.		CHKD. SMM	DATE: Mar. 2010	SITE:	
SUBRD. AH		CHKD. SMM	APPD. JMKG	DWG. 2	
DATE	BY	REVISION			

GRAIN SIZE DISTRIBUTION

Silty Clay Fill to Clayey Silt Fill

FIGURE 1A



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	5004	3	171.8
■	5002	4	171.3
◆	5001	5	170.8
▲	5005	6	169.6

Project Number: 09-1181-1045-1

Checked By: *SM*

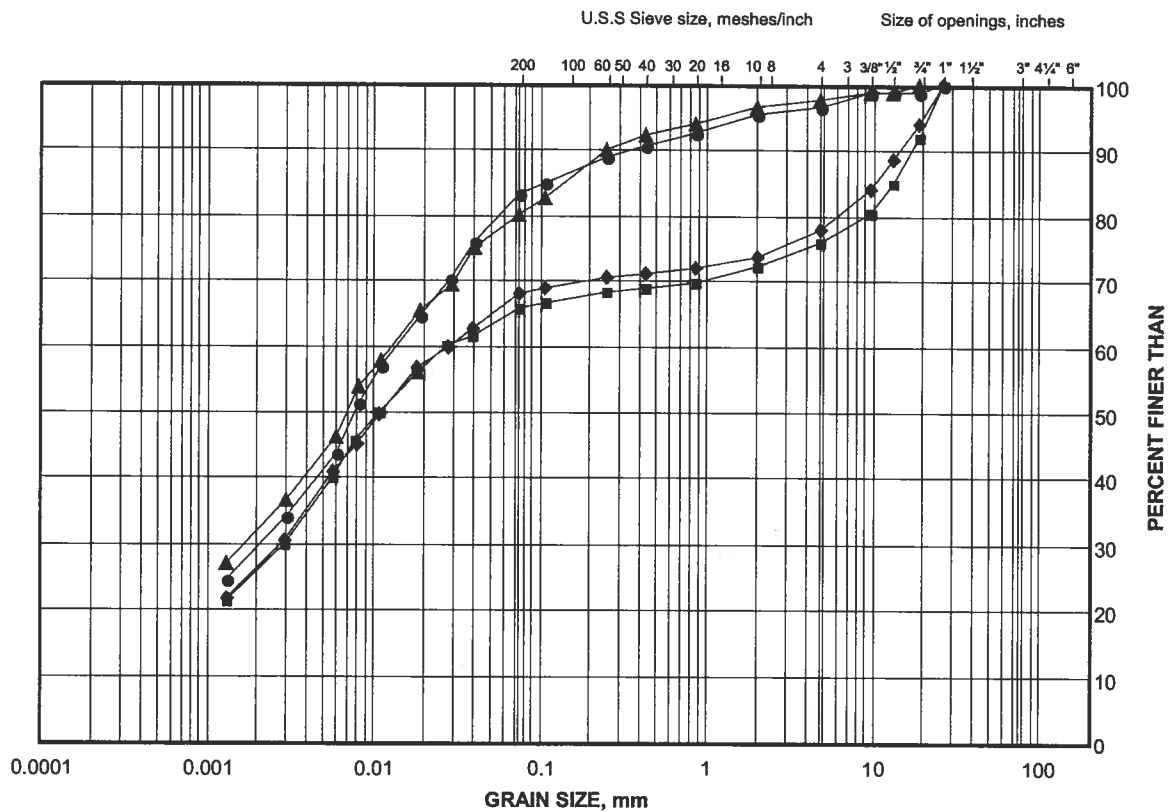
Golder Associates

Date: 16-Mar-10

GRAIN SIZE DISTRIBUTION

Silty Clay Fill to Clayey Silt Fill

FIGURE 1B



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

LEGEND

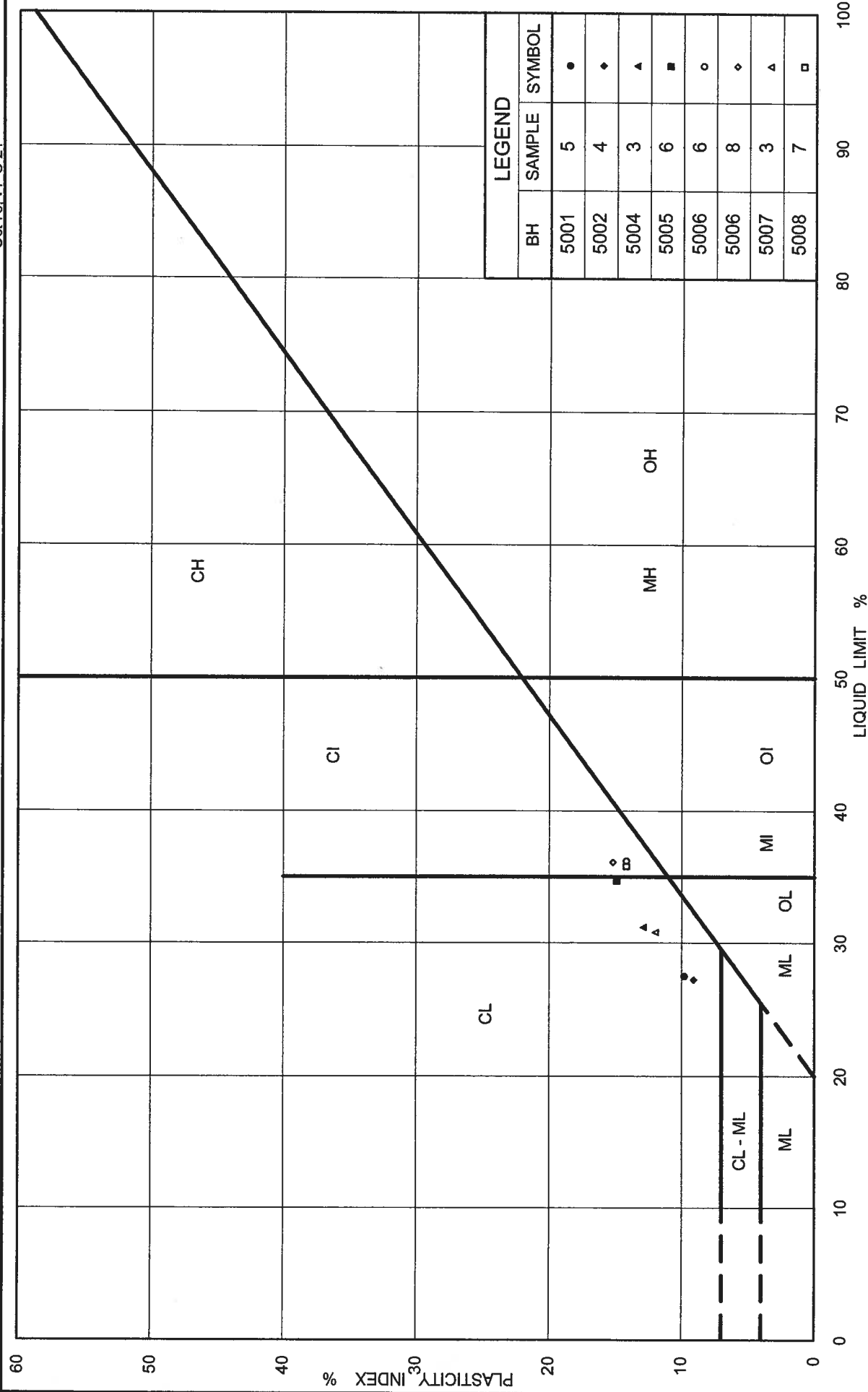
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	5007	3	172.2
■	5006	6	169.8
◆	5008	7	169.4
▲	5006	8	167.4

Project Number: 09-1181-1045-1

Checked By: gml

Golder Associates

Date: 16-Mar-10



PLASTICITY CHART Silty Clay Fill to Clayey Silt Fill

Figure No. 2

Project No. 09-1181-1045-1

Checked By: *SM*

Ministry of Transportation

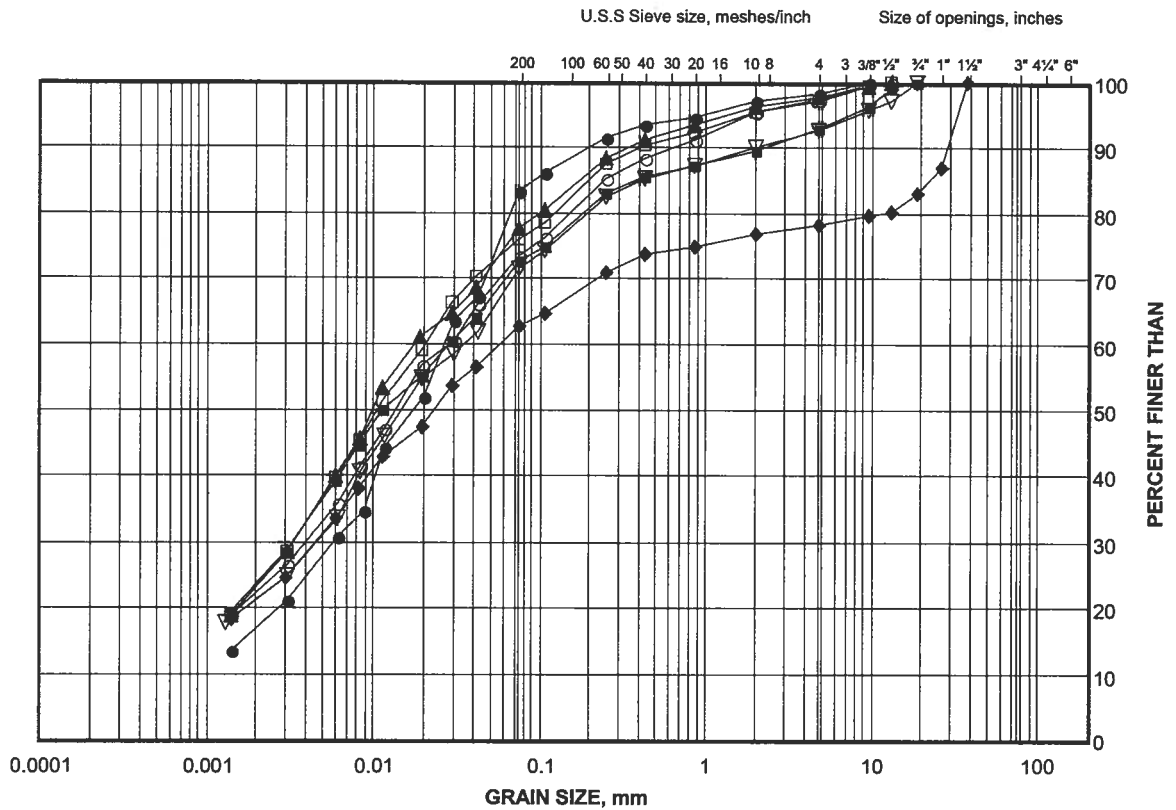


Ontario

GRAIN SIZE DISTRIBUTION

Clayey Silt Till

FIGURE 3



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

LEGEND

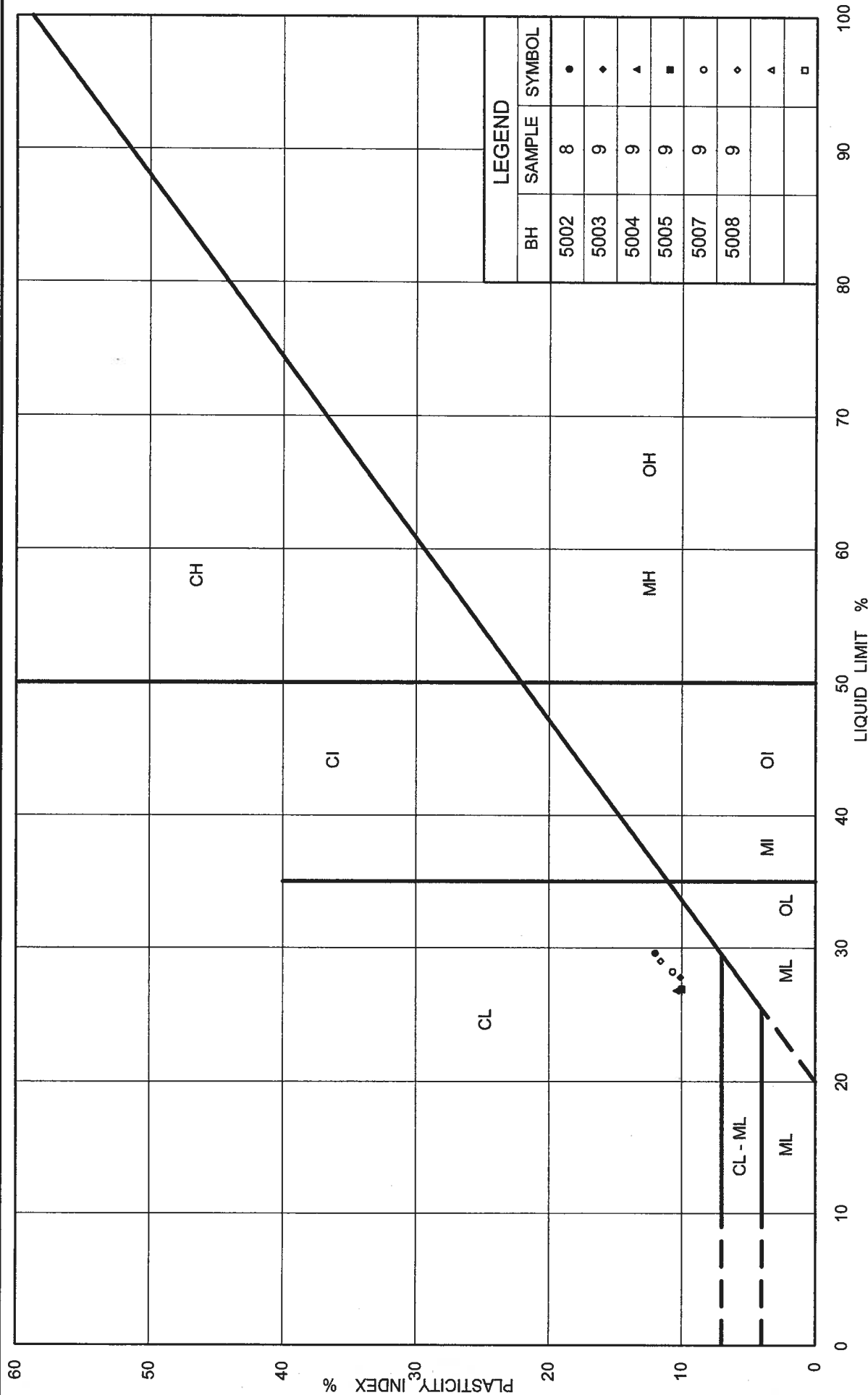
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	5001	11A	163.3
■	5002	8	166.0
◆	5008	9	166.3
▲	5007	9	166.1
▽	5005	9	165.7
○	5004	9	165.8
□	5003	9	165.8

Project Number: 09-1181-1045-1

Checked By: SM

Golder Associates

Date: 03-Mar-10



PLASTICITY CHART Clayey Silt Till

Figure No. 4

Project No. 09-1181-1045-1

Checked By: *SM*

Ministry of Transportation

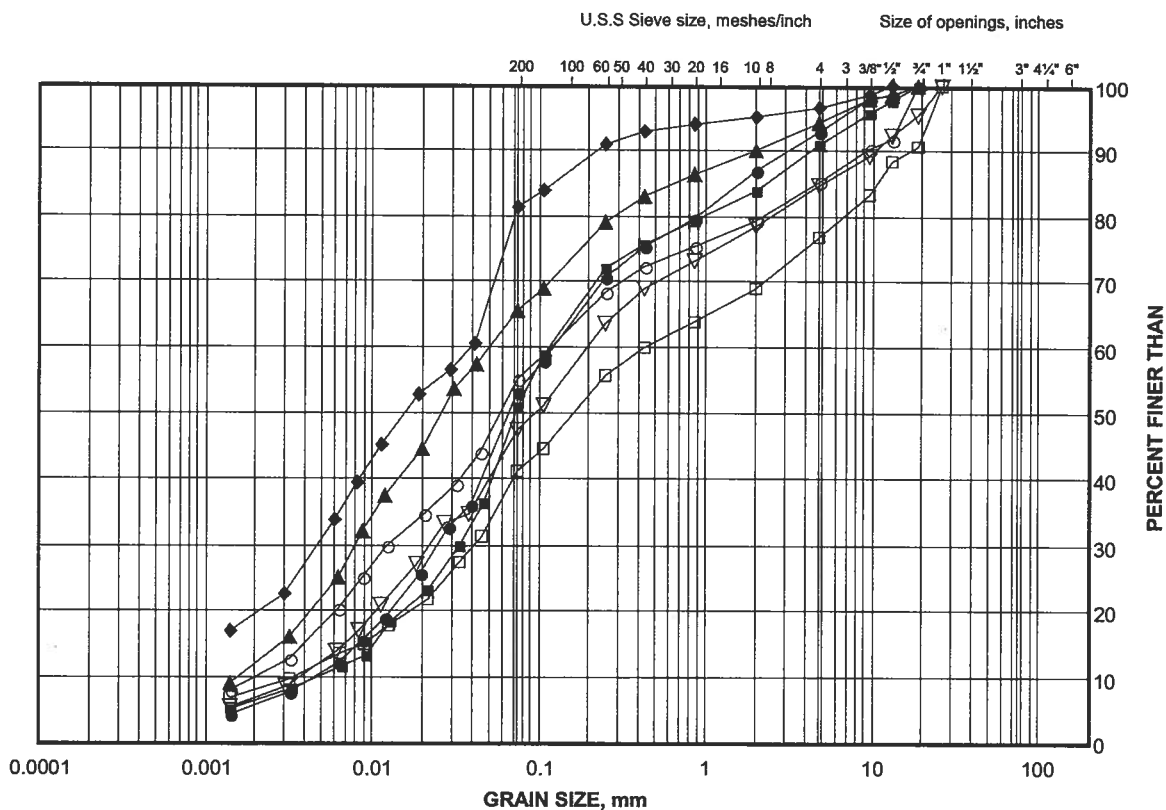


Ontario

GRAIN SIZE DISTRIBUTION

Sand and Silt Till

FIGURE 5A



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	5004	10	164.4
■	5003	10	164.3
◆	5004	12	161.2
▲	5003	12	161.3
▽	5001	12	161.9
○	5001	13	160.4
□	5002	9	164.7

Project Number: 09-1181-1045-1

Checked By: *SMC*

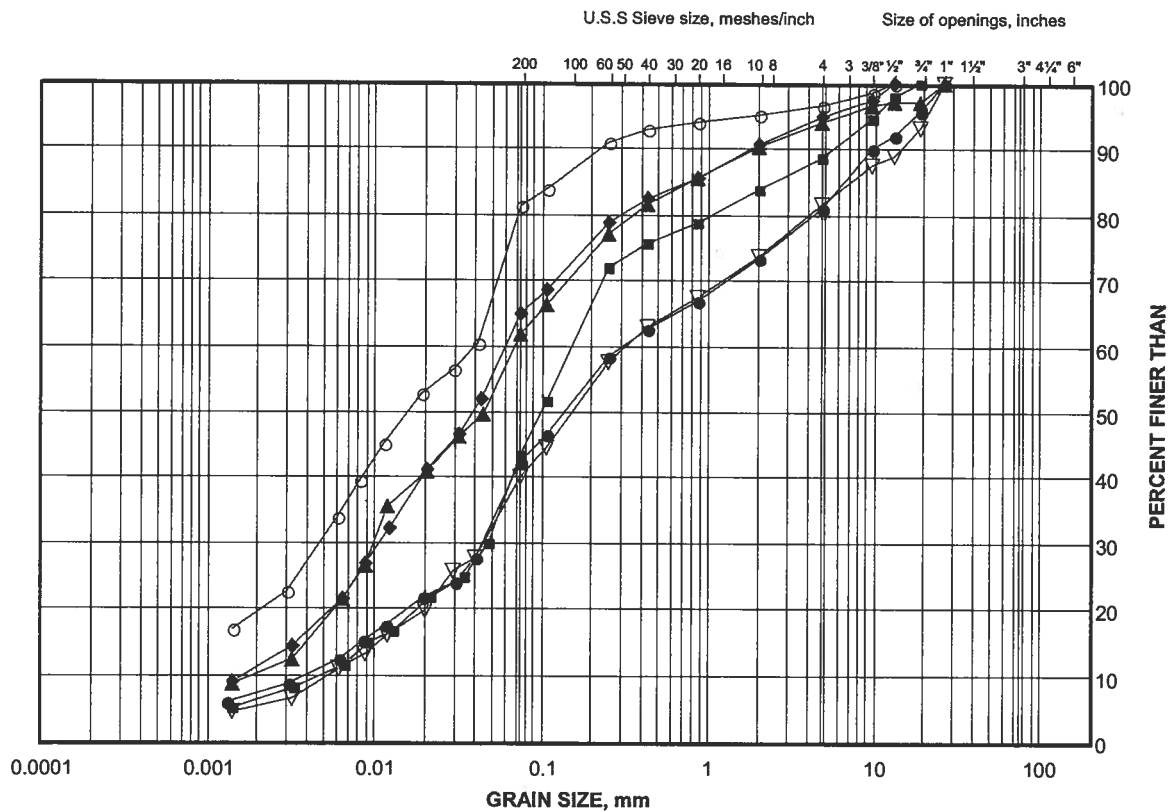
Golder Associates

Date: 03-Mar-10

GRAIN SIZE DISTRIBUTION

Sand and Silt Till

FIGURE 5B



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

LEGEND

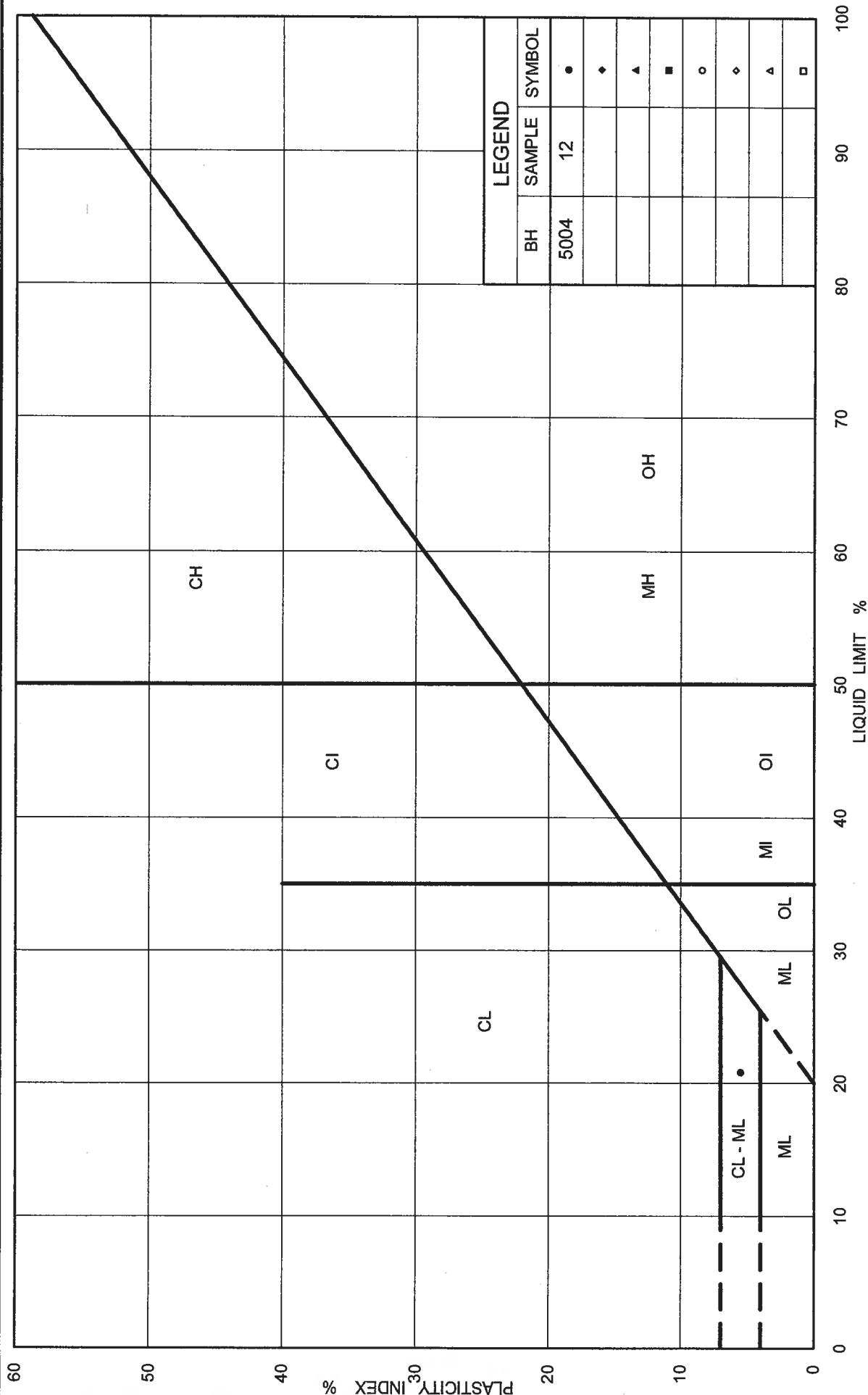
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	5008	10	164.8
■	5007	10	164.6
◆	5007	11	163.2
▲	5006	11	162.9
▽	5005	11	162.9
○	5004	12	161.2

Project Number: 09-1181-1045-1

Checked By: *SMC*

Golder Associates

Date: 16-Mar-10



PLASTICITY CHART Sand and Silt Till

Figure No. 6

Project No. 09-1181-1045-1

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Ministry of Transportation

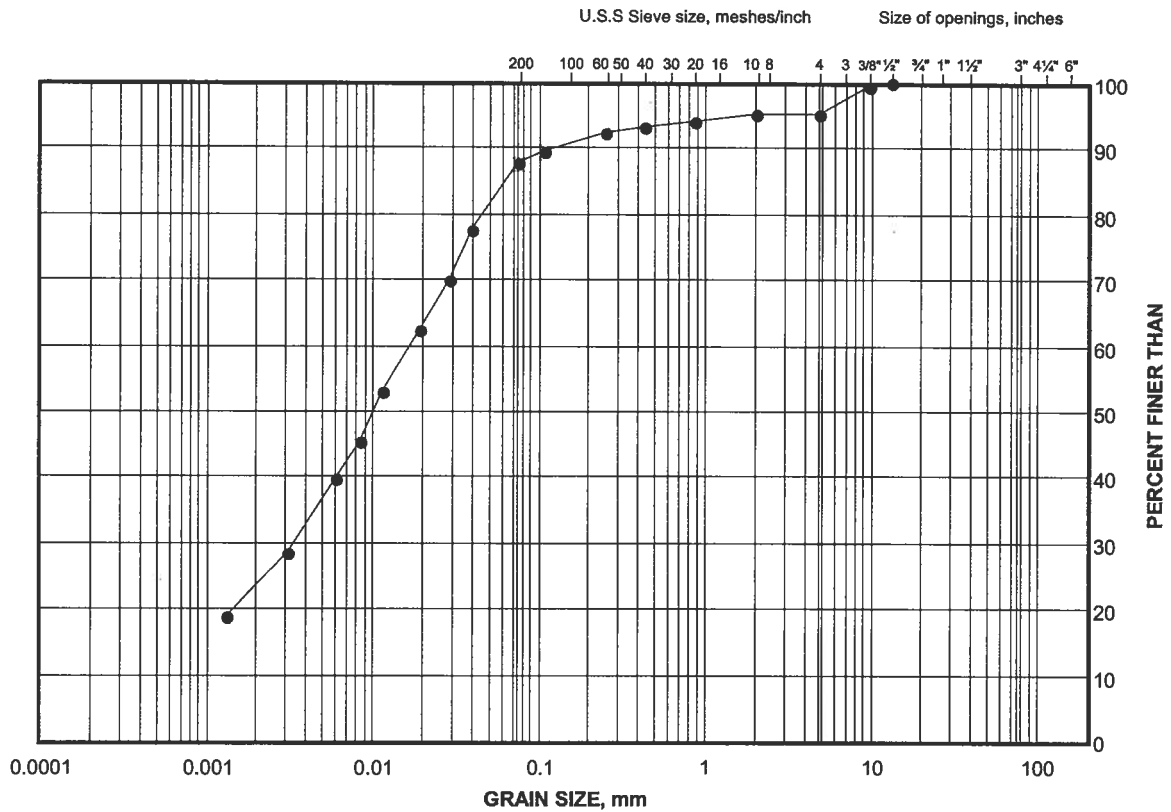


Ontario

GRAIN SIZE DISTRIBUTION

Clayey Silt (Residual Soil)

FIGURE 7



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

LEGEND

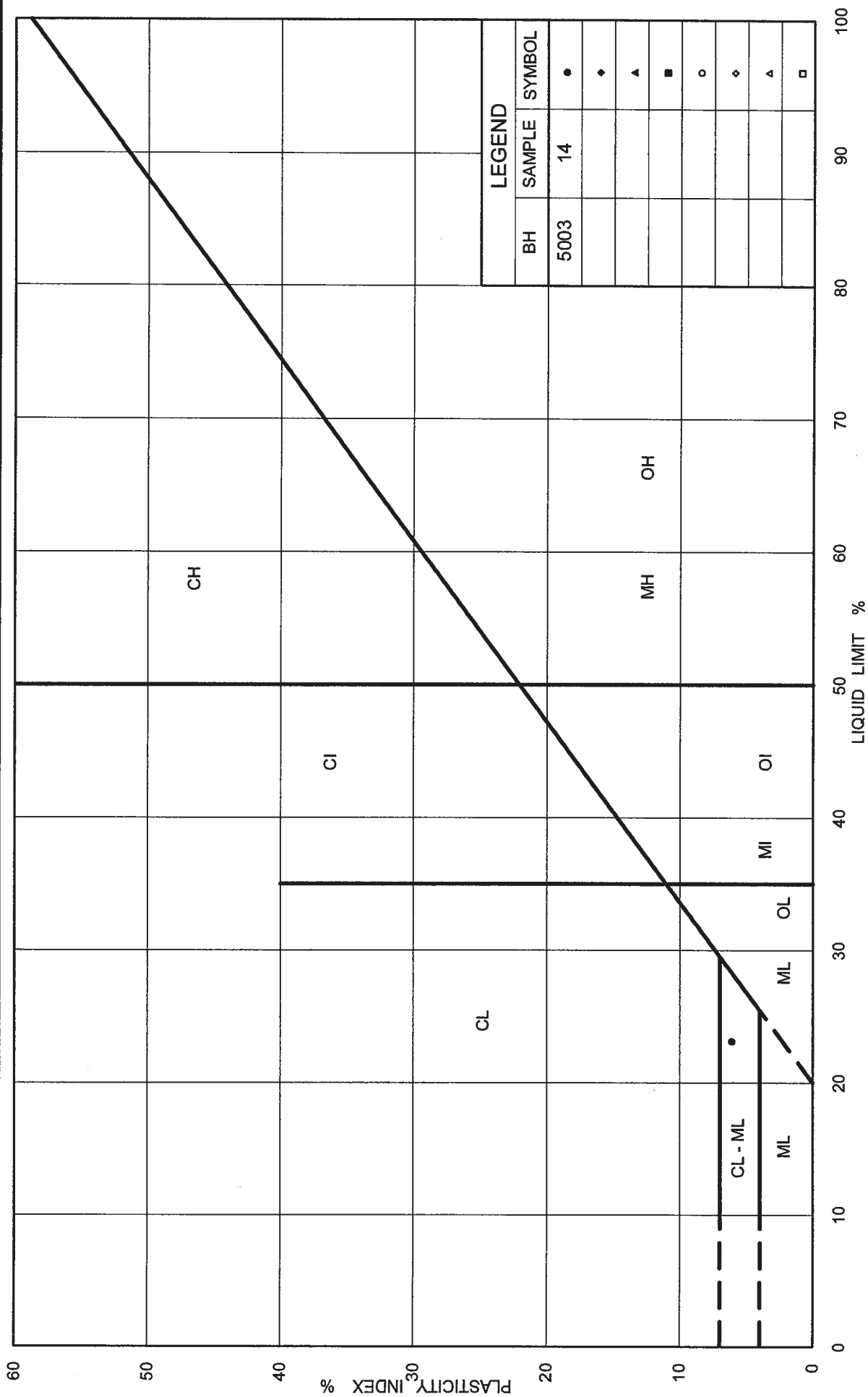
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	5003	14	158.3

Project Number: 09-1181-1045-1

Checked By: *SM*

Golder Associates

Date: 16-Mar-10



Ministry of Transportation

Ontario

PLASTICITY CHART

Clayey Silt (Residual Soil)

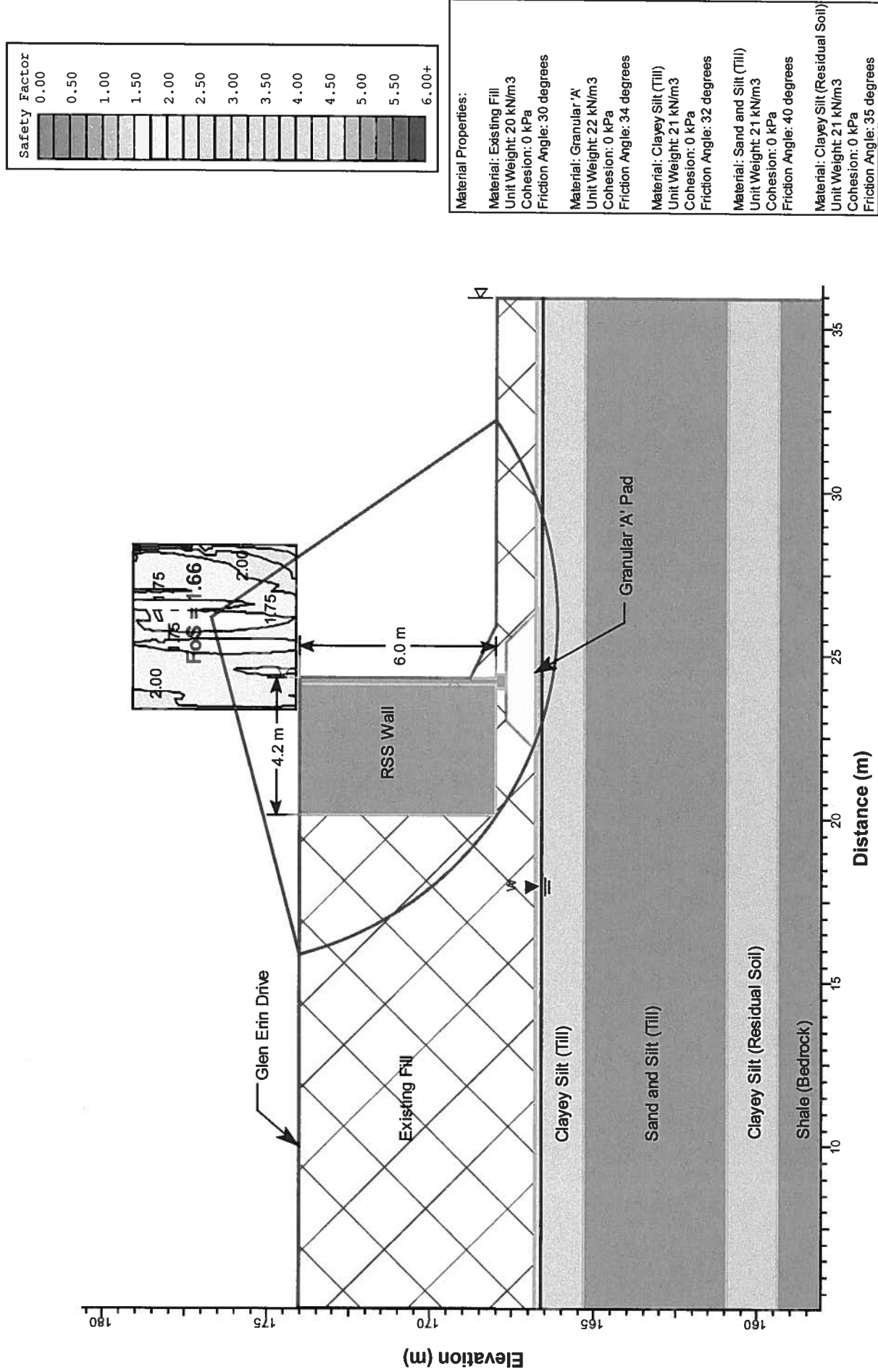
Figure No. 8

Project No. 09-1181-1045-1

Checked By: *SM*

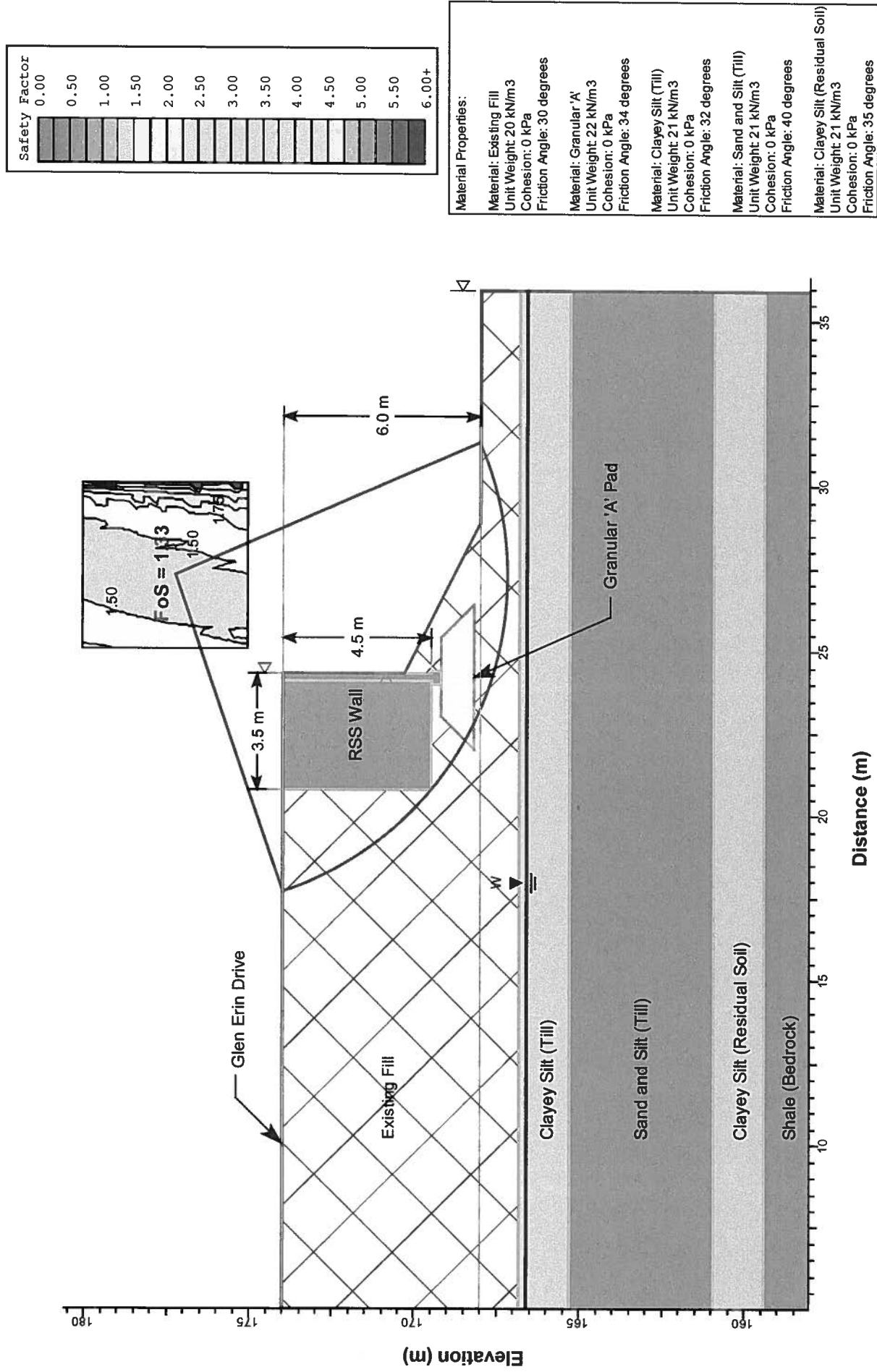
Glen Erin Bridge – RSS Wall Global Stability –Adjacent to the Abutment

Figure 9



Glen Erin Bridge – RSS Wall Global Stability – 4 m North/South of the Abutment

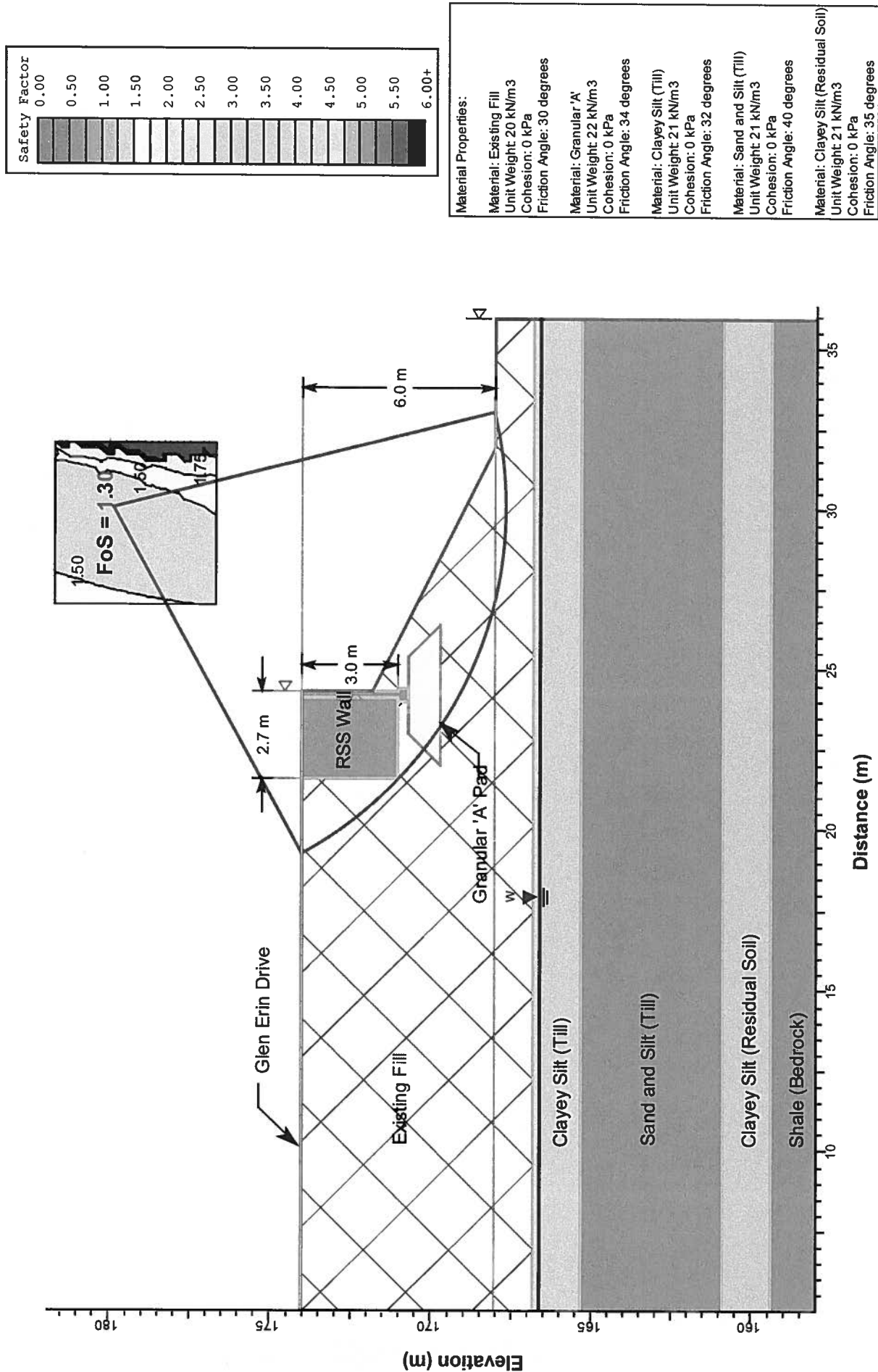
Figure 10





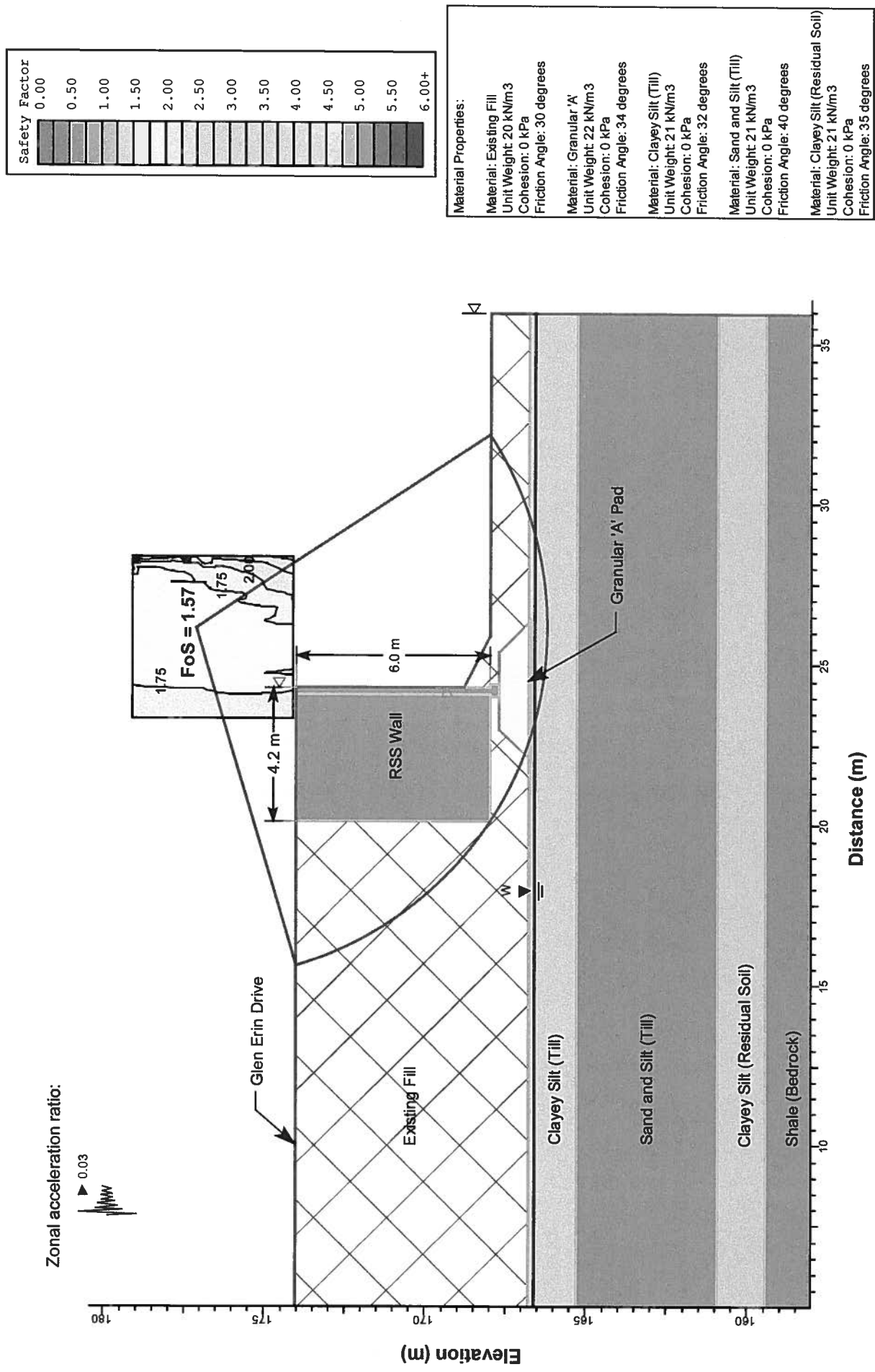
Glen Erin Bridge – RSS Wall Global Stability – 8 m North/South of the Abutment

Figure 11



Glen Erin Bridge – RSS Wall Seismic Global Stability – Adjacent to the Abutment

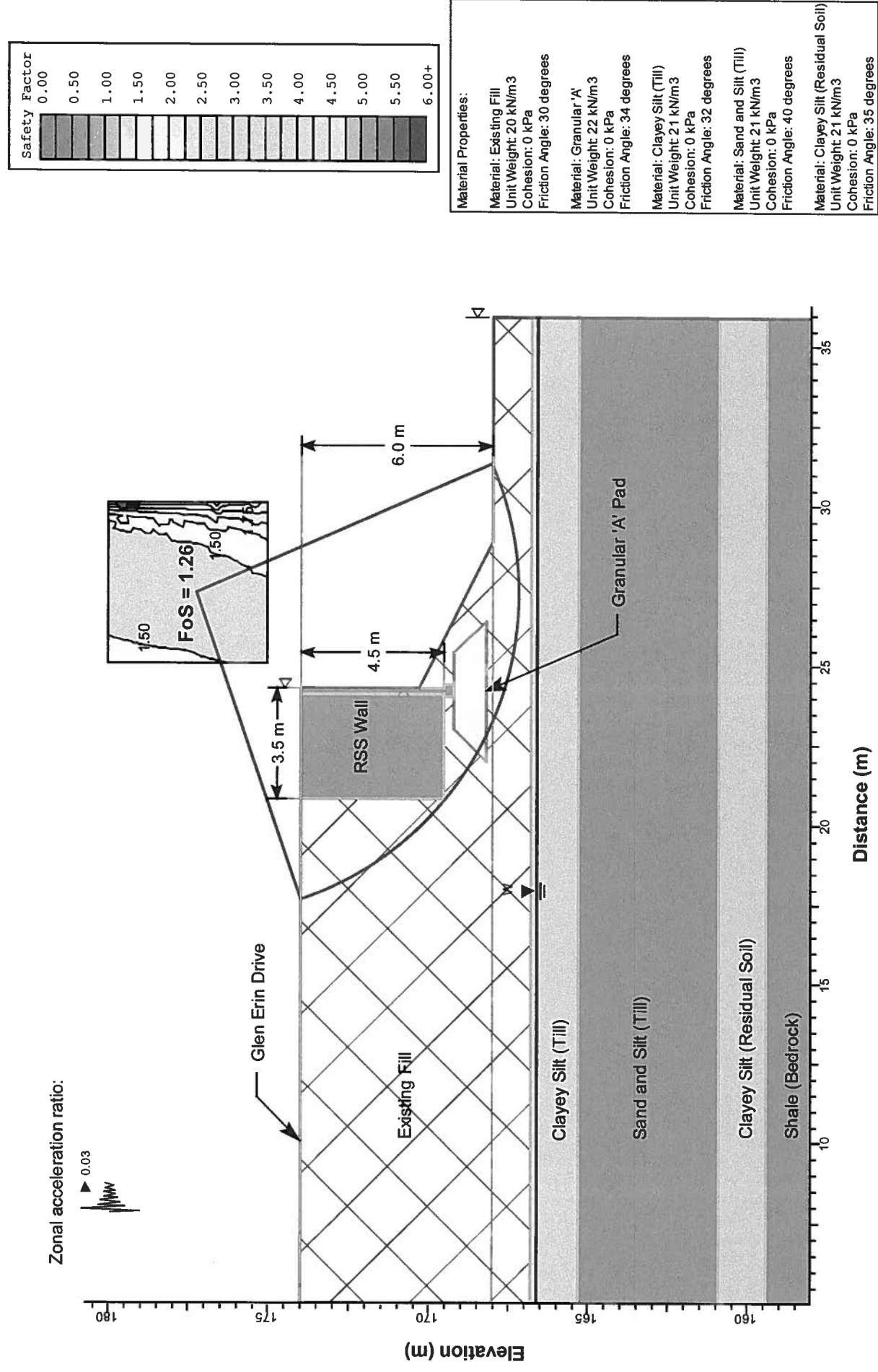
Figure 12





Glen Erin Bridge – RSS Wall Seismic Global Stability – 4 m North/South of the Abutment

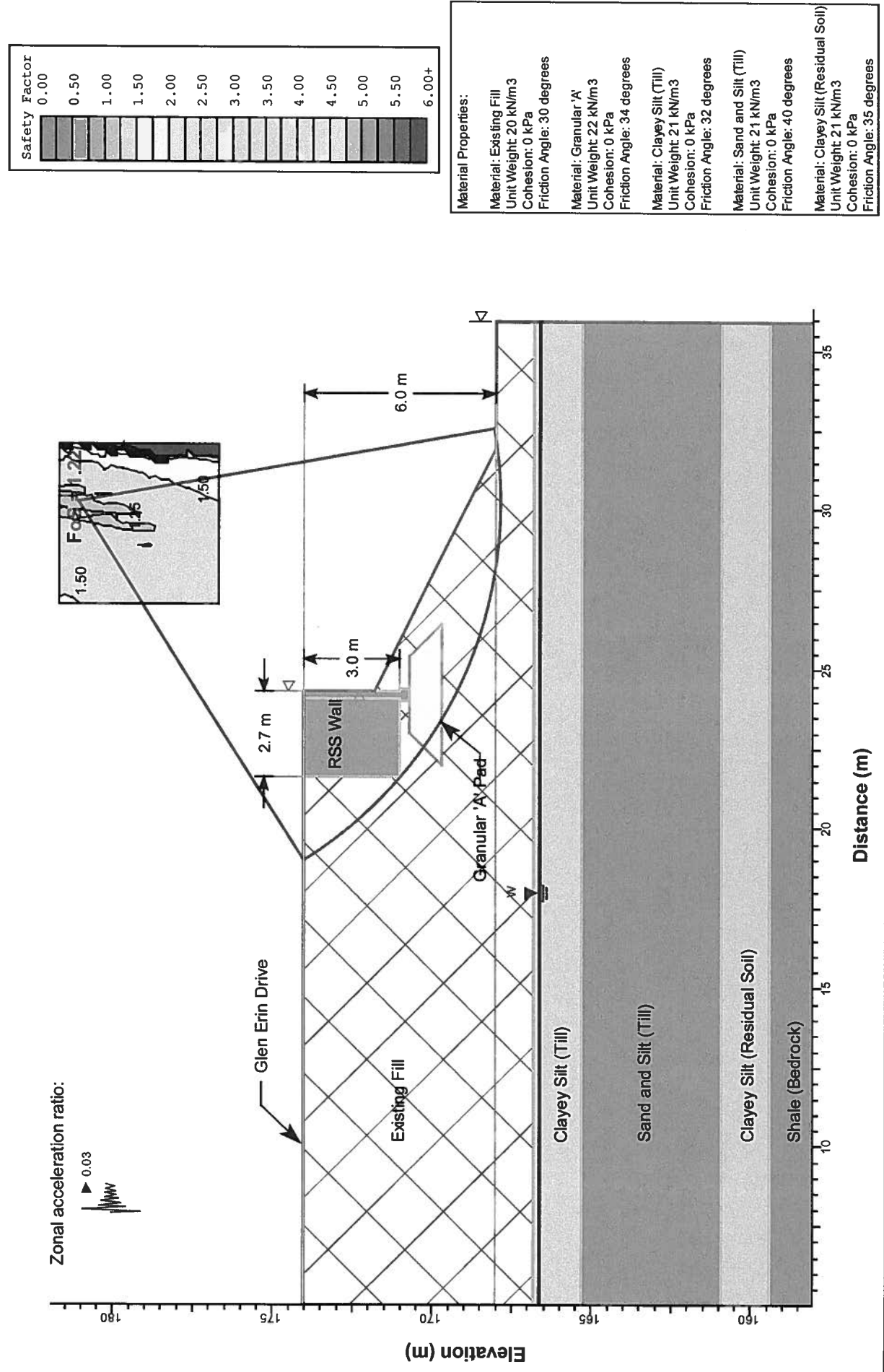
Figure 13





Glen Erin Bridge – RSS Wall Seismic Global Stability – 8 m North/South of the Abutment

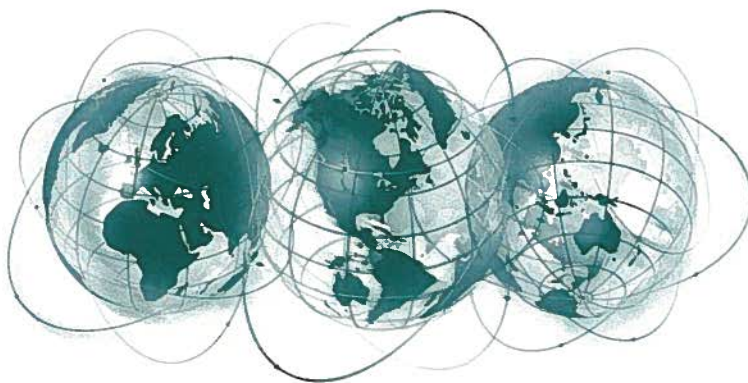
Figure 14



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

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