GEOCRES No.
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FOUNDATION INVESTIGATION AND DESIGN REPORT ASHTON STATION ROAD UNDERPASS (STRUCTURE SITE NO. 3-719) HIGHWAY 7 TWINNING FROM CARLETON PLACE TO 3 KM WEST OF JINKINSON ROAD W.P. 251-99-00

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TO 3 KM WEST OF JINKINSON ROAD
W.P. 251-99-00

1.0 INTRODUCTION

Golder Associates Ltd. (Golder Associates) has been retained by Marshall Macklin Monaghan (MMM) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations associated with the twinning of Highway 7 from two to four lanes in the former West Carleton and Goulbourn Townships which are now part of the City of Ottawa, and in Beckwith Township in Lanark County. The sections of Highway 7 included in this assignment extend from Highway 417 westerly 7 km to 3 km west of Jinkinson Road (W.P. 256-99-00), and from 3 km west of Jinkinson Road westerly to Carleton Place (W.P. 251-99-00 and 252-99-00). Foundation investigation services are also required as part of this assignment for the widening of Highway 417 from the Highway 417-7 interchange easterly to Carp River (W.P. 458-98-00).

This report addresses the proposed Ashton Station Road underpass structure site.

The terms of reference for the original scope of work and Addenda 1 through 7 issued during the proposal period are outlined in the MTO's Request for Proposal (RFP) and in Golder Associates' Proposal No. P21-1301, dated July 2002. As part of Scope Change No. 1, additional borehole investigation work was carried out at the proposed abutment locations for the underpass structures under W.P. 256-99-00, W.P. 251-99-00, and W.P. 252-99-00.

2.0 SITE DESCRIPTION

The existing Ashton Station Road – Highway 7 intersection is located in the village of Ashton Station, approximately 7 km east of the Town of Carleton Place. Ashton Station Road is located at the boundary between the former West Carleton Township in the City of Ottawa, and the Township of Beckwith in Lanark County.

The proposed underpass structure, designated as MTO's Structure Site 3-719, will be located approximately 500 m south of the existing intersection, on the existing Ashton Station Road alignment. The terrain in the immediate vicinity of the site is relatively flat, with the natural ground surface varying from about Elevation 133.5 m to 132 m, declining from the north to the sound end of the proposed structure; a swampy area is present over the southern portion of the proposed structure location. The proposed interchange is situated in farmland.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at the Ashton Station Road site in December 2002, March 2003 and May 2003, at which time twelve boreholes (Boreholes 02-901 to 02-910, 02-920 and 02-921) were advanced within the limits of the proposed structure, at the locations shown on Drawing 1. Due to the presence of swampy ground outside of the existing Ashton Station Road embankment, four additional boreholes (Boreholes 02-930 to 02-933) were advanced by hand augering within the limits of the approach embankments, at the locations shown on Drawing 1, to determine the thickness of organic soils at these locations.

The boreholes were advanced through the overburden soils and into the shale bedrock at the site by hollow stem augers and/or NW casing, using a bombardier-mounted drill rig, supplied and operated by Marathon Drilling Ltd. of Ottawa, Ontario. Boreholes 02-901 to 02-910, 02-920 and 02-921 were advanced to refusal on the bedrock, and Boreholes 02-930 to 02-933 were terminated below the peat / organic soil layer. Samples of the overburden were obtained at 0.75 m intervals of depth using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure. In six of the boreholes (Boreholes 02-901, 02-903, 02-904, 02-907, 02-908, and 02-910), approximately 3 m of bedrock coring was carried out using NQ-size coring equipment.

The water level in the open boreholes was observed throughout the drilling operations, and a total of three piezometers were installed to monitor the groundwater level(s) at the site. The piezometers consist of 25 mm diameter PVC pipe with a slotted tip, placed within a 1.9 m to 2.4 m thick filter sand pack within the bedrock, then sealed above the filter sand pack to ground surface using bentonite pellets. Where no piezometers were installed, the boreholes were backfilled using bentonite pellets, mixed in places with soil cuttings, and a bentonite seal was placed immediately below the ground surface.

The field work was supervised on a full-time basis by members of Golder Associates' staff who located the boreholes in the field, directed the drilling, sampling, and in situ testing operations, and logged the boreholes. The soil and bedrock samples were identified in the field, placed in labelled containers and transported to Golder Associates' laboratory in Ottawa for further examination, and index and classification testing (water content determinations and grain size distribution analyses on selected samples.

The borehole locations and ground surface elevations were established by MMM surveyors or were determined by Golder Associates relative to points staked by MMM. The borehole locations, including MTM NAD83 northing and easting coordinates, and ground surface elevations referenced to geodetic datum are summarized in the following table and are shown on Drawing 1.

Borehole Number	*MTM NAD83 Northing (m)*	*MTM NAD83 Easting (m)*	*Ground Surface Elevation*
02-901	5,003,620.7	340,122.0	133.3 m
02-902	5,003,617.5	340,124.8	133.2 m
02-903	5,003,588.7	340,150.8	132.4 m
02-904	5,003,557.6	340,175.3	132.2 m
02-905	5,003,554.0	340,180.0	132.3 m
02-906	5,003,609.8	340,111.7	133.8 m
02-907	5,003,605.4	340,116.1	133.7 m
02-908	5,003,574.3	340,141.2	132.7 m
02-909	5,003,545.4	340,166.3	132.3 m
02-910	5,003,540.6	340,170.5	132.2 m
02-920	5,003,625.9	340,104.2	134.3 m
02-921	5,003,528.8	340,188.3	132.9 m
02-930	5,003,619.1	340,095.6	134.1 m
02-931	5,003,642.6	340,113.8	133.9 m
02-932	5,003,524.0	340,182.3	132.1 m
02-933	5,003,535.2	340,200.0	131.8 m

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

The study area for this assignment lies within two minor physiographic regions, as delineated in *The Physiography of Southern Ontario*[1], that lie within the major physiographic region of the Ottawa-St. Lawrence Lowland. The Highway 7 area between the Highway 417-7 interchange and Carleton Place is part of the Smiths Falls Limestone Plain, while the area along Highway 417 east of the Highway 417-7 interchange is part of the Ottawa Valley Clay Plain. Most of both physiographic regions is underlain by a series of sedimentary rocks, consisting of sandstones, dolostones, limestones and shales that are, in turn, underlain by igneous and metamorphic bedrock of the Precambrian Shield. The Shield rock generally outcrops to the north of the Ottawa River, and it is also present immediately below the overburden in a localized area between the Hazeldean Fault (approximately the location of the Carp River) and the Ottawa River.

The Smiths Falls Limestone Plain, in which the Ashton Station Road site is located, is characterized by shallow overburden deposits overlying limestone bedrock of the Ottawa Formation; this formation consists of grey limestone with some shaly partings and seams.[2] The shallow overburden soils are typically between 1 m and 3 m in thickness and are commonly comprised of sandy to gravelly till derived from the Precambrian Shield to the north, overlain by glaciofluvial sediments that consist of layered sands and gravels. Large areas of the plain are covered with peat and muck, due to poor drainage as a consequence of the relatively flat topography and shallow depth to bedrock.[1]

The Ottawa Valley Clay Plain region, present along Highway 417 from the Highway 417-7 interchange site eastward, is characterized by relatively thick deposits of sensitive marine clay, silt and silty clay that were deposited within the Champlain Sea basin. These deposits, known as the Champlain Sea clay or Leda clay, overlie relatively thin, commonly reworked glacial till and glaciofluvial deposits, that in turn overlie bedrock.[1] West of the Carp River valley along Highway 417, the upper bedrock consists of limestone of the Ottawa Formation, as described above. Within and immediately east of the Carp River valley, the upper bedrock consists of sandstones and dolostones that have been cut by igneous and metamorphic rocks, controlled by faulting in the vicinity of the Carp River.[2]

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

[2] Belanger, J.R. "Urban Geology of Canada's National Capital Area", in *Urban Geology of Canadian Cities*, Geological Association of Canada Special Paper 42, Ed. P.F. Karrow and O.L. White, 1998.

4.2 Site Stratigraphy

As part of the subsurface investigation at this site, a total of sixteen boreholes were advanced within the limits of the proposed underpass structure and its approach embankments. The borehole locations and ground surface elevations are shown on Drawing 1. The detailed subsurface soil, bedrock and groundwater conditions encountered in the boreholes and the results of in-situ and laboratory testing are given on the Record of Borehole sheets and Figures 1 and 2. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

In summary, the soils encountered immediately below ground surface at this site consist of existing fill (within the existing Ashton Station Road embankment), and peat or topsoil (outside of the existing embankment) overlying relatively thin overburden soil consisting predominantly of sand and silt glacial till, although thin layers of silty sand to sand and gravel were encountered overlying the glacial till in two of the boreholes at the site, and thin layers of clayey silt to silty clay were encountered overlying the glacial till in five of the boreholes in the southern portion of the site. These surficial soils are, in turn, underlain by bedrock that was encountered between 0.3 m and 2.7 m depth (about Elevation 130.2 m to 133.1 m) in the boreholes. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Fill

About 150 mm to 750 mm of sand to sand and gravel fill, associated with the existing Ashton Station Road embankment construction, was encountered in Boreholes 02-901, 02-902, 02-903, 02-906, 02-907, 02-920, and 02-921.

4.2.2 Topsoil / Peat

Between 100 mm and 300 mm of topsoil was encountered immediately below ground surface in Boreholes 02-908, 02-909, 02-930, and 02-931; these boreholes are located north or west of the swampy area at the site.

Between 300 mm and 750 mm of peat was encountered in the boreholes advanced outside of the existing Ashton Station Road embankment in the southern portion of the site, as summarized in the following table:

Borehole Number	*Peat Thickness*
02-904	600 mm
02-905	400 mm
02-910	300 mm
02-921	500 mm (Below existing fill)
02-932	750 mm
02-933	400 mm

As noted in the above table, approximately 500 mm of peat was encountered below the existing embankment fill in Borehole 02-921 (which was drilled at the south approach embankment).

4.2.3 Silty Sand to Sand and Gravel

Below the peat in Borehole 02-910, and below the topsoil in Borehole 02-931, is a layer of silty sand to sand and gravel. The result of a grain size distribution test result on one sample of this material – a sand containing trace silt – is shown on Figure 1. This surficial layer was fully penetrated in Borehole 02-910, where it was found to have a total thickness of 0.5 m.

4.2.4 Clayey Silt to Silty Clay

In five of the boreholes (Boreholes 02-904, 02-905, 02-921, 02-932, and 02-933), a clayey silt to silty clay layer was encountered below the existing fill or topsoil/peat. The clayey silt to silty clay was fully penetrated in three of the boreholes, where it was found to vary in thickness from 150 mm to 550 mm. One SPT "N" value of 2 blows per 0.3 m of penetration was measured in Borehole 02-921; based on local experience with correlations between SPT "N" values and shear strength, the clayey silt to silty clay is considered to have a firm to stiff consistency. The measured natural water content on one sample of this material was about 30 per cent.

4.2.5 Sand and Silt Till

The above-mentioned soils are underlain by a deposit of glacial till, that is comprised of sand and silt containing trace to some gravel and trace to some clay. The results of two grain size distribution tests are shown on Figure 2. This glacial till deposit ranges from 0.2 m to 1.0 m thick as encountered in the boreholes.

The glacial till varies from very loose to dense, based on measured SPT "N" values of 2 to 38 blows per 0.3 m of penetration; typically, however, this till deposit is compact.

4.2.6 Interlayered Shale, Limestone and Sandy Dolostone Bedrock

Interlayered shale, limestone and sandy dolostone bedrock underlies the sand and silt till deposit at this site. The surface of the bedrock was encountered between Elevation 130.2 m and 133.1 m (at about 0.3 m to 2.7 m depth), generally declining toward the south. The following table summarizes the bedrock surface depth and elevation as encountered at the borehole locations. It should be noted that bedrock was cored in six of the boreholes; in the remaining six boreholes, the upper shale portion of the bedrock was confirmed after penetration by augering and/or split-spoon sampling.

Borehole Location	*Borehole Number*	*Ground Surface Elevation*	*Depth to Bedrock*	*Bedrock Surface Elevation*
North approach	02-920	134.3 m	1.2 m	133.1 m
North abutment	02-901	133.3 m	0.5 m	132.8 m (Cored)
	02-902	133.2 m	0.3 m	132.9 m
	02-906	133.8 m	1.0 m	132.8 m
	02-907	133.7 m	0.9 m	132.8 m (Cored)
Centre pier	02-903	132.4 m	0.7 m	131.7 m (Cored)
	02-908	132.7 m	0.8 m	131.9 m (Cored)
South abutment	02-904	132.2 m	1.5 m	130.7 m (Cored)
	02-905	132.3 m	1.6 m	130.7 m
	02-909	132.3 m	1.3 m	131.0 m
	02-910	132.2 m	1.0 m	131.2 m (Cored)
South approach	02-921	132.9 m	2.7 m	130.2 m

The upper 0.8 m to 2.3 m of the bedrock is generally comprised of dark grey or black shale, which contains limestone interbeds. The shale itself is weak, and the limestone interbeds are medium strong. This portion of the bedrock is typically very thinly to thinly-bedded, although portions of the formation are medium-bedded. The shale varies from fresh to moderately weathered.

Below the shale, the bedrock generally consists of grey sandy dolostone and dark grey-green dolomitic limestone, in places containing shale seams and layers. These portions of the bedrock formation are fresh to slightly weathered, weak to medium strong, and thinly to medium-bedded.

The Rock Quality Designation (RQD) values measured on the bedrock core samples recovered from Boreholes 02-901, 02-903, 02-904, 02-907, 02-908 and 02-910 range from 0 to 25 per cent in shale and sandy dolostone portions of the bedrock; these RQD values are indicative of very poor to poor quality rock. The RQD values measured within the dolomitic limestone portions of the bedrock range from about 25 to 90 per cent, indicative of poor to excellent quality rock, although the upper portion of the dolomitic limestone encountered in Borehole 02-904 was highly fractured and had an RQD of 0 per cent. The discontinuities observed in the rock core are typically horizontal to sub-horizontal, associated with the bedding planes and, where present within the dolomitic limestone, stylolitic features, although some vertical to sub-vertical jointing was also observed.

A description of the terms used in the description of the bedrock samples from this site is provided on the *Lithological and Geotechnical Rock Description Terminology* sheet which precedes the Record of Borehole sheets included with this report.

4.3 Groundwater Conditions

The water levels measured in the three piezometers installed at the site are summarized in the following table:

Borehole No.	***Groundwater Elevation Measured in Piezometer***				
	21 Mar '03	***26 Mar '03***	***15 Apr '03***	***29 Apr '03***	***06 Jun '03***
02-901	133.3 m *	132.8 m	132.9 m	132.7 m	132.7 m
02-903	132.4 m *	132.6 m	132.6 m	132.6 m	132.4 m
02-910	–	–	–	132.3 m	132.2 m

* **NOTE:** The water in the piezometers was frozen at ground surface on March 21, 2003.

It is noted that swampy ground conditions are present in the south abutment and south approach embankment areas. The water levels in these areas are typically at or above the ground surface. Based on the above water level measurements, the groundwater level associated with the overburden and bedrock at the site is at about Elevation 132.5 m to 133 m during spring conditions, generally declining from the north end to the south end of the proposed structure location. The groundwater level should be expected to be similar to or slightly higher than this during periods of high snow melt or precipitation (i.e. in the early spring or fall), and to be between 0.3 m and 0.5 m lower than this during drier seasons.

5.0 CLOSURE

This Foundation Investigation Report was prepared by Ms. Lisa Coyne, P.Eng., an Associate and Senior Engineer with Golder. Mr. Fintan Heffernan, a Designated MTO Contact for Golder, conducted an independent review of the report.

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PART B

FOUNDATION DESIGN REPORT
ASHTON STATION ROAD UNDERPASS
HIGHWAY 7 TWINNING FROM CARLETON PLACE
TO 3 KM WEST OF JINKINSON ROAD
W.P. 251-99-00

6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation design recommendations for the proposed two-span Ashton Station Road underpass structure. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation at this site. The interpretation and recommendations provided are intended only to provide the designers with sufficient information to assess the feasible foundation alternatives and to design the proposed structure foundations. As such, where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

It is understood that two alternative integral abutment configurations, which eliminate the requirement for expansion joints, were considered during the preliminary structural design stage, as follows:

- Perched, pile-supported abutments with abutment foreslopes oriented at 2 horizontal to 1 vertical (2H:1V).
- Perched, pile-supported abutments with a mechanically-reinforced soil retaining wall system (retained soil system or RSS walls) in a false abutment configuration. It is understood that this option would allow a reduction of up to about 10 m in the total span length required for the more conventional configuration incorporating a 2H:1V abutment foreslope, with an accompanying reduction in the construction cost.

6.2 Bridge and Retaining Wall Foundation Options

At the proposed underpass structure site, the natural ground surface varies from about Elevation 132 m to 133.5 m. It is understood that the proposed Ashton Station Road grade at the structure location is at about Elevation 141.5 m to 142 m. The approach embankments will, therefore, be approximately 8 m to 10 m above the existing natural grade.

The native soils at the site consist of existing fill and topsoil/peat overlying a generally compact sand and silt till stratum; up to about 0.6 m of firm to stiff clayey silt to silty clay was encountered atop the glacial till in the boreholes advanced in the southern portion of the structure, in the vicinity of the south abutment and south approach embankment. These overburden soils are underlain by weak shale bedrock and weak to medium strong sandy dolostone and dolomitic limestone bedrock. The bedrock surface was encountered in the boreholes between about Elevations 130.2 m and 133.1 m, about 0.3 m to 2.7 m below the existing ground surface.

The bedrock is suitable for support of the proposed centre pier, abutments and associated retaining walls, such as concrete cantilever retaining walls, on shallow foundations. Alternatively, spread footings placed on a compacted Granular "A" pad within the approach embankment fill may be considered for the abutments. The overburden soils at the site are suitable for the support of RSS walls, either as wingwalls or in front of the abutments, following subexcavation of the peat and surficial silty clay stratum.

Since integral abutments are under consideration, steel H-piles can also be considered for support of the abutments; steel H-piles are not suitable for use at the centre pier, owing to the shallow depth to bedrock. Based on the proposed Ashton Station Road grade, it is expected that the pile cap underside will be at about Elevation 138 m. Based on these levels, it is estimated that the pile length would be about 5.1 m to 5.2 m at the north abutment (where the bedrock surface was encountered at about Elevation 132.8 m to 132.9 m), and about 6.8 m to 7.3 m long at the south abutment (where the bedrock surface was encountered between about Elevations 130.7 m and 131.2 m). This satisfies the minimum pile length of 5 m required to impart sufficient flexibility of the piles to accommodate bridge deck deflections for an integral abutment structure. If the pile caps are maintained lower and/or for pile toe fixity for seismic design considerations, the steel H-piles could be socketted into the bedrock.

As an alternative to spread footings or steel H-pile foundations, caisson foundations resting on or socketted into the bedrock could be used for support of the abutments; due to the shallow depth of overburden soils, caisson foundations are not feasible at the centre pier. This option, if adopted for support of the abutments, has the advantage of minimizing the groundwater control that would be required to advance spread footing excavations to bedrock.

Recommendations for spread footings, steel H-pile and caisson foundations are presented in the following sections. A summary comparison of the advantages, disadvantages, relative costs and risks associated with each of the foundation options is presented in Table 1 following the text of this report. Based on this comparison, and in consideration of the shallow bedrock depth at this site, the preferred alternative from a foundations perspective is the use of spread footings supported on the bedrock for the centre pier, and spread footings either supported on the bedrock or perched on a compacted Granular "A" pad within the approach embankment fill for the abutments.

6.3 Spread Footings

6.3.1 Geotechnical Resistance for Spread Footings on Bedrock

The centre pier, abutments and any associated concrete cantilever wing walls / retaining walls may be supported on spread footings placed on the properly prepared shale bedrock. The surface of the bedrock was encountered in the boreholes between Elevations 130.7 m and 132.9 m, as summarized in the following table.

Foundation Element	*Borehole Numbers*	*Depth to Bedrock*	*Bedrock Surface Elevation*
North abutment	02-901, 02-902, 02-906, 02-907	0.3 m – 1.0 m	132.8 m – 132.9 m
Centre pier	02-903, 02-908	0.7 m – 0.8 m	131.7 m – 131.9 m
South abutment	02-904, 02-905, 02-909, 02-910	1.0 m – 1.6 m	130.7 m – 131.2 m

Based on the borehole results, there is some slight variability in the bedrock surface within the limits of each foundation element. In addition, the upper shale portion of the bedrock is, in local areas, highly weathered and quite fractured (RQD values of 0 to 25 per cent), and subexcavation of any loose, fractured bedrock will be required prior to construction of the footing. MTO's Special Provision SP902S01 should be included in the Contract Documents requiring inspection and approval of the foundation area by the Quality Verification Engineer prior to footing construction, to ensure that all loose and/or fractured rock has been removed from the foundation areas prior to construction of the spread footings.

For design, the following options for founding levels may be considered:

	Design Founding Elevation		
Foundation Element	*Case 1*	*Case 2*	*Case 3*
North Abutment	133.0 m	132.5 m	132.8 m
Centre Pier	132.0 m	131.5 m	131.8 m
South Abutment	131.3 m	130.4 m	131.0 m

1. For Case 1, the bedrock surface would have to be exposed and cleaned, and then mass concrete would be placed to raise the grade to the founding level. Provision should be made in the Contract Documents for mass concrete placement to accommodate variations in the bedrock surface. The benefit of this approach is that excavation into the weak shale bedrock, which contains interlayers of medium strong limestone, would be avoided.

2. For Case 2, excavation of the higher portions of the bedrock will be required within the foundation footprints. Based on the borehole results, subexcavation of typically 0.3 m (but up to 0.9 m at the south abutment) would be required. It is noted that the upper shaley portion of the bedrock is generally weak, but that it does contain limestone interlayers which are medium strong (corresponding to unconfined compressive strengths in the range of 25 MPa to 50 MPa); although these medium strong layers are relatively thin, hoe ramming techniques are expected to be necessary to penetrate them.

3. As a third option (Case 3), an intermediate founding level may be assumed for design. In this case, a combination of bedrock subexcavation and mass concrete placement will be required.

Spread footings placed on the surface of the properly prepared shale bedrock or on mass concrete may be designed based on a factored geotechnical resistance at Ultimate Limit States (ULS) of 1,000 kPa; this factored geotechnical resistance at ULS has been determined taking into consideration the fractured nature of the upper portion of the bedrock. The geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement will be greater than the factored geotechnical resistance at ULS, since the bedrock is considered to be an unyielding material; as such, ULS conditions will govern for this foundation type.

The geotechnical resistances provided herein are given under the assumption that the loads will be applied perpendicular to the surface of the footings. 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*, using the curve for cohesive soils.

6.3.2 Geotechnical Resistance for "Perched" Footings – Abutments and Retaining Walls in Approach Embankments

Spread footings for the north and south abutments may be placed on a compacted Granular "A" pad constructed within the approach embankment fill. A factored geotechnical resistance at ULS of 900 kPa may be used for design, assuming that the subgrade is properly prepared prior to fill placement (in accordance with Section 5.8.1) and that the Granular "A" pad is placed in regular lifts, and compacted to 100 per cent of the material's Standard Proctor maximum dry density. Assuming that the topsoil, peat and, where present, the surficial clayey silt to silty clay soils are removed from under the approach embankment, a geotechnical resistance at SLS of 350 kPa may be assumed for design.

The geotechnical resistances provided above are given under the assumption that the loads will be applied perpendicular to the surface of the footings. 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*, using the curves for non-cohesive soils.

6.3.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footings and the subgrade should be calculated in accordance with Section 6.7.5 of the CHBDC. The coefficient of friction, tan φ', for cast-in-place concrete footings placed on undisturbed, properly prepared subgrade may be taken as given in the following table. These represent unfactored values; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

Subgrade	*Coefficient of Friction (tan φ')*
Shale bedrock	0.50
Compacted Granular "A" pad	0.57

If necessary, the sliding resistance of spread footings founded on the bedrock can be supplemented by dowelling into the bedrock. The horizontal resistance of the dowels is dependent on the strength of the bedrock, grout and steel. For this site, the design of the dowels in the rock may be handled in the same way as the dowel embedment into the concrete. This assumes that the unconfined compressive strength of the grout will be similar to that of the concrete. The dowels should have a minimum embedded length within the bedrock of 1 m, and the structural strength of the dowel and compressive strength of the grout should not be exceeded. If dowelling into bedrock is adopted at this site, a Special Provision should be included in the Contract Documents to specify the installation, materials and testing of the dowels.

6.3.4 Frost Protection

Spread footings founded on the shale bedrock or "perched" within the approach embankment fill should be provided with a minimum of 1.8 m of soil cover for frost protection.

6.4 Steel H-Pile Foundations

Steel H-piles driven to found on the shale bedrock may be used for support of the abutments. The surface of the bedrock was encountered in the boreholes between Elevation 130.7 m and 132.9 m in the vicinity of the proposed abutments, as noted below:

Foundation Element	*Borehole Numbers*	*Depth to Bedrock*	*Bedrock Surface Elevation*
North abutment	02-901, 02-902, 02-906, 02-907	0.3 m – 1.0 m	132.8 m – 132.9 m
South abutment	02-904, 02-905, 02-909, 02-910	1.0 m – 1.6 m	130.7 m – 131.2 m

If the pile caps are "perched" within the approach embankment fill with the pile cap base at about Elevation 138 m, the piles would be about 5.1 m to 5.2 m long at the north abutment, and 6.8 m to 7.3 m long at the south abutment, without socketting into bedrock.

If necessary to resist seismic forces, or to ensure a minimum pile length of 5 m if the north abutment pile cap is maintained lower, the pile toes could be socketted into the bedrock. Although the shale bedrock is weak, the interbedded limestone bedrock is medium strong (corresponding to compressive strengths of up to about 50 MPa); the presence of these medium strong interlayers would likely require socket formation using coring or churn drilling. Alternatively, consideration could be given to open excavation to the bedrock surface and trenching / excavating into the bedrock to provide a preformed slot into which the piles could be subsequently driven. It should be noted that groundwater control measures would be required in order to complete such excavation.

6.4.1 Axial Geotechnical Resistance

For HP 310 x 110 piles driven to found on or socketted within the shale bedrock, a factored axial resistance at ULS of 2,000 kN may be assumed for design. This value represents a structural limitation for the pile rather than a geotechnical limitation. The geotechnical resistance at SLS for 25 mm of settlement will be greater than the factored axial resistance at ULS, since the limestone bedrock is considered to be an unyielding material; as such, ULS conditions will govern for this foundation type.

For this site, the piles would essentially be driven to practical refusal on the bedrock, unless socketting is required to resist seismic forces. It is assumed that the piles would be driven after construction of the approach embankment to the base of the pile cap level. For these driven piles, consideration must be given to the presence of cobbles and boulders within the glacially-derived soils at the site. Driven piles should be equipped with flange reinforcement (driving shoes) as per SS103-12. If battered piles are under consideration, however, the piles should be equipped with suitable driving points (such as Titus Ejector or equivalent) instead of driving shoes in order to make adequate seating of the pile more certain given the relatively short pile length and the hardness of the bedrock.

6.4.2 Resistance to Lateral Loads

Lateral loading could be resisted fully or partially by the use of battered steel H-piles; where battered piles are adopted, the pile batter should be limited to 3V:1H or steeper.

If vertical piles are used, the resistance to lateral loading will have to be derived from the soil in front of the piles. Where integral abutments are under consideration, there will also be a requirement for the piles to move sufficiently to accommodate the bridge deck deflections.

The resistance to lateral loading in front of the pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, n_h, is based on the following equation for granular soils:

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

where n_h is the constant of horizontal subgrade reaction, as given below;
z is the depth (m); and
B is the pile diameter (m).

The following ranges for the value of n_h may be assumed in the structural analysis. The range in values reflects the variability in the subsurface conditions as well as the two extremes of design: the requirement for flexibility in the case of integral abutments. and the requirement for lateral support in the case of non-integral abutments (or the centre pier, if applicable).

Soil Unit	n_h
North abutment:	
Embankment fill (assumed to be compacted granular fill), above approximately Elevation 133 m	10 MPa/m
South abutment:	
Embankment fill (assumed to be compacted granular fill), above approximately Elevation 132 m	10 MPa/m
Surficial soils between about Elevations 132 m and 131 m (i.e. below groundwater level)	4 to 6 MPa/m

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor as follows:

Pile Spacing in Direction of Loading d = Pile Diameter	***Reduction Factor***
8d	1.0
6d	0.7
4d	0.4
3d	0.25

6.4.3 Frost Protection

The pile caps should be provided with a minimum of 1.8 m of soil cover for frost protection.

6.5 Caisson Foundations

Caissons socketted nominally into the shale bedrock may be used for support of the abutments; it is recommended that a nominal socket depth of about 0.3 m be adopted, to ensure that the caisson is founded below any broken or moderately weathered shale. The surface of the bedrock was encountered in the boreholes between Elevation 130.7 m and 132.9 m in the vicinity of the proposed abutments, as noted below:

Foundation Element	***Borehole Numbers***	***Depth to Bedrock***	***Bedrock Surface Elevation***
North abutment	02-901, 02-902, 02-906, 02-907	0.3 m – 1.0 m	132.8 m – 132.9 m
South abutment	02-904, 02-905, 02-909, 02-910	1.0 m – 1.6 m	130.7 m – 131.2 m

If the abutment pile caps are "perched" within the approach embankment fill with the pile cap base at about Elevation 138 m, the caissons would be about 5.4 m to 5.5 m long at the north abutment, and 7.1 m to 7.6 m long at the south abutment, including a nominal 0.3 m deep socket. The use of caisson foundations is not appropriate for support of the centre pier or of any concrete retaining walls associated with the structure, owing to the shallow depth to bedrock.

As discussed in Section 5.4, the shale bedrock is generally weak, and its limestone interlayers are medium strong (corresponding to unconfined compressive strengths up to about 50 MPa). Formation of deeper socket holes in the bedrock is feasible; however, it is expected to be necessary to use rock coring or churn drilling techniques to advance the holes through the limestone interlayers. It is noted that these stronger layers could make churn drilling slow, and the more thickly-bedded portions of the bedrock may be difficult to remove by coring operations, particularly where large diameter sockets are required.

In addition, the overburden soils at the site are generally cohesionless and water-bearing; these soils will flow into the auger hole during drilled shaft installation if left unsupported. Consequently, a temporary liner or the use of drilling slurry will be required to support the holes through the overburden during drilling, installation and concrete placement. It should be noted that the design of slurry, if used, would have to accommodate a wide range in soil gradations and states of compaction; further, the use of slurry would not allow inspection of the bedrock at the base of the drilled shafts.

6.5.1 Axial Geotechnical Resistance

Caissons socketted nominally into the shale bedrock should be designed based on end-bearing resistance and a factored geotechnical resistance at ULS of 3 MPa should be used. If higher geotechnical resistances are required, the caissons could be socketted 3 m into the bedrock, and designed based on a factored geotechnical resistance at ULS of 6 MPa. Serviceability Limit States resistances do not apply to drilled shafts founded on the shale or dolomitic limestone bedrock, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

MTO's Special Provision SP902S01 should be included in the Contract Documents requiring inspection and approval of the foundation areas by the Quality Verification Engineer prior to caisson installation and concreting, to ensure that all loose and/or fractured rock has been removed from the foundation areas.

6.5.2 Resistance to Lateral Loads

The resistance to lateral loading developed by the soils in front of vertical caissons, and the reductions due to group effects, may be determined as per Section 5.4.2.

6.5.3 Frost Protection

The pile caps should be provided with a minimum of 1.8 m of soil cover for frost protection.

6.6 Retained Soil System (RSS) Walls

Retained soil system (RSS) walls could be adopted at the Ashton Station Road site in conjunction with integral abutments. It is expected that RSS walls at this site would be about 4 m to 8 m high.

The RSS walls should be founded below any topsoil or peat. In order to minimize the total and differential settlement of RSS walls at the south abutment and approach embankment, where up to about 0.6 m of clayey silt to silty clay was encountered, it is recommended that this cohesive material be subexcavated so that the RSS wall is placed directly on the sand and silt till. In addition, the clayey silt to silty clay stratum should be removed from below the strip footing that supports the RSS wall facing panels. The silty clay subexcavation for the RSS mass and facing panel area at the south abutment should extend down to Elevation 131.3 m. The subexcavated soils should be replaced with compacted Granular "A" fill.

Assuming that the RSS wall acts as a unit and utilizes the full width of the reinforced soil mass, which is taken as two-thirds of the height of the wall, the following factored geotechnical resistances at ULS may be used for design of RSS walls founded on the properly prepared sand and silt till stratum, or on compacted Granular "A" fill.

Wall Height	***Assumed Footing Width***	***Factored Geotechnical Resistance at ULS***
4 m	2.7 m	170 kPa
8 m	5.4 m	350 kPa

Assuming the clayey silt to silty clay stratum is subexcavated, the settlement of the founding soils as a result of these magnitudes of loading is expected to be between 5 mm and 15 mm (as discussed further in Section 5.8); therefore the ULS conditions will govern for design of RSS walls at this site. The majority of the settlement of the RSS walls would occur during construction since the founding soils are cohesionless, overlying bedrock at a shallow depth.

The resistance to lateral forces / sliding resistance between the compacted granular fill (assumed to be Granular "A") and the subgrade soils should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, tan δ', between the compacted Granular "A" and the properly prepared, loose surficial sand or generally compact sand and silt till subgrade may be taken as 0.55. Where the RSS mass is placed on a compacted Granular "A" pad, the coefficient of friction between the RSS mass and granular pad may be taken as 0.57. These represent unfactored values; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

The internal stability of the mechanically-reinforced soil walls should be checked by the RSS supplier / designer. The factor of safety related to global stability under static loading for properly designed and constructed RSS walls at this site is greater than 1.3.

6.7 Lateral Earth Pressures for Design

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, on the nature of the soils behind the backfill, on the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and on 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 Ontario Provincial Standard Specifications (OPSS) Granular 'A' or Granular 'B' but with less than 5 per cent passing the 200 sieve should be used as backfill behind the walls. This fill should be placed and compacted in accordance with MTO's Special Provision SP105S10. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3501.00 and 3504.00.

- 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 6.9.3. Compaction equipment should be used in accordance with MTO's Special Provision SP105S10. Other surcharge loadings should be accounted for in the design, as required.

- The granular fill may be placed either in a zone with width equal to at least 1.8 m behind the back of the wall stem (Case I in Figure C6.9.1(l) 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 (Case II in Figure C6.9.1(l) of the *Commentary to the CHBDC)*.

- For Case I, the pressures are based on the proposed embankment fill materials and the following parameters (unfactored) may be used assuming the use of Select Subgrade material:

Soil unit weight:	20 kN/m^3
Coefficients of static lateral earth pressure:	
Active, K_a	0.35
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/m³	21 kN/m³
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. If the abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design.

- Seismic loading will result in increased lateral earth pressures acting on the abutment stem and retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure. According to the National Building Code of Canada, this site is located in Seismic Zone 4. The site-specific zonal acceleration ratio for Ottawa is 0.18. Based on experience, for the subsurface conditions at this site, a 10 to 20 per cent amplification of the ground motion will occur, resulting in an increase in the ground surface acceleration from 0.18g to between 0.2g and 0.22g. The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of A = 0.22.

- In accordance with Sections 4.6.4 and C.4.6.4 of the *CHBDC* and its *Commentary*, for structures which do not allow lateral yielding, the horizontal seismic coefficient, k_h, used in the calculation of the seismic active pressure coefficient, is taken as 1.5 times the zonal acceleration ratio (i.e. $k_h = 0.33$). For structures which allow lateral yielding, k_h is taken as 0.5 times the zonal acceleration ratio (i.e. $k_h = 0.11$). The seismic active earth pressure coefficient is also dependent on the vertical component of the earthquake acceleration, k_v. Three discrete values of vertical acceleration are typically selected for analysis, corresponding to $k_v = +2/3\ k_h$, $k_v = 0$, and $k_v = -2/3\ k_h$.

- The following seismic active pressure coefficients (K_{AE}) for the two backfill cases (Case I and Case II) may be used in design; these coefficients reflect the maximum K_{AE} obtained using the k_h and three values of k_v as described above. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat.

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

Wall Type	*Case I*	*Case II*
Yielding wall	0.40	0.32
Non-yielding wall	0.80	0.63

- 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 0.22. This corresponds to outward displacements of up to 55 mm at this site.

- The earthquake-induced dynamic pressure distribution, which is to be added to the static earth 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:

$$\sigma_h(d) = K_a \, \gamma \, d + (K_{AE} - K_a) \, \gamma \, (H\text{-}d)$$

where
$\sigma_h(d)$ is the lateral earth pressure at depth, d, (kPa)
K_a is the static active earth pressure coefficient;
K_{AE} is the seismic active earth pressure coefficient;
γ is the unit weight of the backfill soil (kN/m^3), as given previously;
d is the depth below the top of the wall (m); and
H is the total height of the wall (m).

6.8 Embankment Design and Construction

The construction of the new Ashton Station Road will require placement of between 8 m and 10 m of fill within the limits of the approach embankments. Based on the borehole results, the embankment subgrade soils will generally consist of compact sand and silt till; however, localized layers of loose silty sand to sand and gravel are expected atop the till and, at the south approach embankment, a firm to stiff clayey silt to silty clay stratum is present.

6.8.1 Subgrade Preparation and Embankment Construction

Any topsoil, peat or organic matter, the clayey silt to silty clay where present, and any softened / loosened soils should be stripped within the limits of the approach embankment footprints, and all subgrade soils should be proof-rolled prior to fill placement. The embankment fill should be placed and compacted in accordance with MTO's Special Provision SP105S10. Inspection and field density testing should be carried out by qualified personnel during placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

Where the embankment height is greater than 8 m, a mid-height berm at least 2 m in width is required. To reduce surface water erosion on the embankment side slopes, placement of topsoil and seeding or pegged sod is recommended. It is noted that ditching alongside the embankment may extend below the existing groundwater level at the site. The cuts should be inspected after completion to check for evidence of water seepage which could affect the surficial stability. It is recommended that remedial measures, such as a granular blanket, be placed in the ditches where seepage is present.

6.8.2 Embankment Stability

With appropriate subgrade preparation and proper placement and compaction of earth or granular embankment fill materials, the 8 m to 10 m high approach embankments with side slopes maintained at 2 horizontal to 1 vertical (2H:1V) will have a factor of safety of greater than 1.3 against deep-seated slope instability. The static slope stability analyses for this embankment configuration were carried out using the following parameters:

Soil Deposit	Bulk Unit Weight	Effective Friction Angle	Undrained Shear Strength
Earth or Granular Embankment Fill	20 – 22 kN/m^3	32°	–
Surficial Silty Sand to Sand and Gravel	19 – 20 kN/m^3	30°	–
Sand and Silt Till	21 kN/m^3	32°	–

The liquefaction potential of the soils below the embankment under seismic loading has been considered using the empirical method outlined in Section C.4.6.2 of the *CHBDC Commentary*, which correlates the cyclic resistance ratio of the soils with their normalized penetration resistance and fines content. Based on this assessment, a factor of safety of greater than 1.1 against liquefaction for an earthquake of magnitude 7.5 is obtained for the surficial sand and glacial till soils below the water table. Although the site soils are not considered to be liquefiable, there will still be some deformation of the soils under seismic loading conditions. Pseudo-static methods of slope stability analysis indicate a yield acceleration of approximately 0.2g is required to reduce the factor of safety against slope instability to 1.0. Using this result and the simplified Newmark method, embankment deformations as a result of the design earthquake event are anticipated to be less than 25 mm.

6.8.3 Embankment Settlement

Settlement of the approach embankments will occur due to compression of the new embankment fill itself, and due to compression of the relatively thin, cohesionless overburden soils; in addition, consolidation settlement will occur within the clayey silt to silty clay stratum that is present at the south end of the structure site.

Provided that the embankment material consists of select subgrade material or clean earth fill, the settlement of the embankment fill itself is expected to be less than 25 mm. The use of granular fill for the new embankment construction would reduce this magnitude, since the majority of settlement of granular fills will occur during construction.

The compression of the generally loose surficial sand and compact glacial till strata was modelled using elastic deformation moduli, based on correlations with the measured SPT "N" values; these values are summarized in the following table:

Soil Unit	*Bulk Unit Weight*	*Elastic Modulus*
Embankment fill (range of parameters assumed for earth fill and granular fill)	20 – 22 kN/m^3	–
Generally loose silty sand to sand and gravel	19 – 20 kN/m^3	10 MPa to 20 MPa
Generally compact sand and silt till	21 kN/m^3	20 MPa to 50 MPa

Provided that proper subgrade preparation is carried out, the settlement of the cohesionless foundation soils for the immediate approach embankments is expected to range from about 5 mm to 10 mm at the north approach, and from about 5 mm to 15 mm at the south approach, as a result of construction of the 8 m to 10 m high approach embankments. This settlement is expected to occur mainly during construction.

6.9 Design and Construction Considerations

6.9.1 Excavation

Excavations to expose the bedrock surface (to allow construction of spread footings, or to permit socket formation within the bedrock) would extend to depths of about 0.3 m to 3 m below the existing ground surface. The excavations will typically extend through the existing Ashton Station Road embankment fill, peat, loose surficial sands (where present), and generally compact sand and silt glacial till. The groundwater level at the site is typically at or less than 0.5 m below ground surface.

Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act (OHSA) for Construction Activities. The water-bearing surficial sands and cohesionless till soils are classified as Type 3 soil, according to the OHSA. Temporary excavations (i.e. those which are only open for a relatively short period) through these overburden soils should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V) assuming that the overburden soils are dewatered. If full dewatering of the soils comprising the excavation side slopes is not achieved, as discussed in Section 5.9.2, shallower side slopes of 3H:1V will be required.

It is not anticipated that temporary roadway protection will be required to permit construction of the new Ashton Station Road underpass structure.

6.9.2 Groundwater and Surface Water Control

The groundwater level at the site is typically at or less than 0.5 m below the ground surface. Excavations to expose the bedrock surface, either for founding of spread footings or to enable formation of a trench within the bedrock to provide toe support to piles, will require groundwater control. Pumping from sumps formed within the bedrock at the base of the excavations should be sufficient to dewater the excavation; however, in order to permit the use of 1H:1V side slopes, it will be necessary to control the groundwater outside of the excavation. Owing to the shallow depth to bedrock, an eductor system is not feasible, but it may be practicable to construct sumps surrounding the excavations to dewater the soils comprising the side slopes. Otherwise, excavation side slopes oriented at 3H:1V will be required.

As noted in Section 5.5, if caisson foundations are adopted at this site, the use of a temporary liner or drilling slurry will be required within the overburden to support the auger holes during pile or concrete placement.

7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Lisa Coyne, P.Eng., an Associate and Senior Engineer with Golder. Mr. Fintan Heffernan, a Designated MTO Contact for Golder, conducted an independent review of the report.

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TABLE 1
COMPARISON OF FOUNDATION ALTERNATIVES
ASHTON STATION ROAD UNDERPASS STRUCTURE

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Foundation Costs	Risks and/or Consequences
Spread footings founded on bedrock – high founding elevation with placement of mass concrete	• Feasible at centre pier • Feasible at abutments but would result in high abutment wall	• Minimizes differential settlement	• Excavation would require groundwater control • May not be possible to fully dewater soils immediately above bedrock; could require shallower excavation side slopes (3H:1V in wet areas instead of 1H:1V)	• Less expensive than spread footings supported on bedrock using the lower founding elevation, but more expensive than "perched" footings due to groundwater control costs	• Potential for difficulties with groundwater control that could affect the construction schedule
Spread footings founded on bedrock – low founding elevation with typically 0.3 m to 0.5 m (but up to 0.9 m) of subexcavation of bedrock	• Feasible at centre pier • Feasible at abutments but would result in high abutment wall	• Minimizes differential settlement	• Subexcavation of medium strong interlyaers in shale bedrock expected to be more difficult and time-consuming, especially compared to surface preparation requirements for higher founding option or to dowelling for resistance of lateral loading • Excavation requires groundwater control • May not be possible to fully dewater soils immediately over bedrock; could require shallower excavation side slopes (3H:1V in wet areas instead of 1H:1V)	• Probably most expensive spread footing option, owing to costs associated with bedrock subexcavation as well as groundwater control	• Potential for difficulties with groundwater control that could affect the construction schedule • Potential for difficulties with bedrock subexcavation where medium strong limestone interlayers encountered
Spread footings founded within approach embankment fill (for abutments and concrete retaining walls)	• Feasible at both abutments	• Avoids major groundwater control measures • Minimizes abutment wall height	• Minor differential settlement between abutments founded within embankment fill and centre pier footing on bedrock	• Probably least expensive foundation construction costs	• Minor differential settlement between foundation elements, since centre pier would be supported on bedrock

TABLE 1 (Continued)
COMPARISON OF FOUNDATION ALTERNATIVES
ASHTON STATION ROAD UNDERPASS STRUCTURE

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Foundation Costs	Risks and/or Consequences
Steel H-pile foundations driven to found on or drilled to socket nominally into bedrock	• Feasible for support of abutments; site conditions appropriate for use of integral abutments • Not feasible for support of centre pier due to shallow depth to bedrock	• Negligible settlement, particularly as compared to abutment footings "perched" within approach embankments • Potentially less groundwater control required than for open excavation to construct spread footings, depending on pile cap level	• If lateral / seismic loading conditions merit, socketting of pile toe into bedrock expected to require coring or churn drilling to penetrate medium strong limestone interlayers, with use of temporary liner to support overburden soils	• Less expensive than caisson option with rock sockets, owing to potentially smaller socket diameter	• If required for pile toe fixity, socketting into the shale bedrock expected to be made more difficult and time-consuming by presence of medium strong limestone interlayers
Steel H-pile foundations placed in trench within bedrock	• Feasible for support of abutments • Not feasible for support of centre pier due to shallow depth to bedrock	• Negligible settlement, particularly as compared to abutment footings "perched" within approach embankments	• Would require open excavation up to 3 m depth with groundwater control; may be difficult to fully dewater soils and 3H:1V excavation side slopes could be required • Subexcavation of medium strong limestone interlayers in shale bedrock expected to be difficult	• Expected to be more expensive than socketting option due to groundwater control and bedrock excavation costs	• Potential for difficulties with groundwater control that could affect the construction schedule • Potential for difficulties with bedrock subexcavation
Drilled shafts founded on or socketted nominally into bedrock	• Feasible for support of abutments • Not feasible for support of centre pier due to shallow depth to bedrock	• Negligible settlement, particularly as compared to abutment footings "perched" within approach embankments • Potentially less groundwater control required than for open excavation to construct spread footings depending on pile cap level	• Temporary liners required to minimize disturbance to surrounding soils • Socketting into bedrock expected to require coring or churn drilling to penetrate medium strong limestone interlayers in shale	• May be more expensive than steel H-pile option if rock sockets are necessary, owing to potentially larger socket diameter	• If required for pile toe fixity, socketting into the bedrock expected to be difficult and time-consuming due to presence of medium strong limestone interlayers

TABLE 1 (Continued)
COMPARISON OF FOUNDATION ALTERNATIVES
ASHTON STATION ROAD UNDERPASS STRUCTURE

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Foundation Costs	Risks and/or Consequences
Retained Soil System (RSS) walls	• Soils at both abutments are suitable for support of RSS walls	• Minimal excavation and groundwater control required for construction	• Some settlement will occur, particularly at south abutment / approach embankment where clayey silt to silty clay stratum is present	• Generally less expensive than concrete retaining wall foundations	• More settlement than for concrete retaining walls supported on bedrock

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

II. PENETRATION RESISTANCE

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

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

PH: Sampler advanced by hydraulic pressure
PM: Sampler advanced by manual pressure
WH: Sampler advanced by static weight of hammer
WR: Sampler advanced by weight of sampler and rod

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils

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

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

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

S:\FINALDATA\ABBREV\2000\LOFA-D00.DOC

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. General

π	3.1416
in 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 ($\sigma' = \sigma$-u)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_3	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1+\sigma_2+\sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

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

(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

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

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

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of weathering.

Faintly weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable.

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock texture and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	> 2 m
Thickly bedded	0.6 m to 2m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	< 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	> 3 m
Wide	1 - 3 m
Moderately close	0.3 - 1 m
Close	50 - 300 mm
Very close	< 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	> 60 mm
Coarse Grained	2 - 60 mm
Medium Grained	60 microns - 2 mm
Fine Grained	2 - 60 microns
Very Fine Grained	< 2 microns

Note: * Grains >60 microns diameter are visible to the naked eye.

CORE CONDITION

Total Core Recovery

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varies from 0% for completely broken core to 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to (W.R.T.) Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.

Description and Notes

An abbreviated description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

B - Bedding	P - Polished
FO - Foliation/Schistosity	S - Slickensided
CL - Cleavage	SM - Smooth
SH - Shear Plane/Zone	R - Ridged/Rough
VN - Vein	ST - Stepped
F - Fault	PL - Planar
CO - Contact	FL - Flexured
J - Joint	UE - Uneven
FR - Fracture	W - Wavy
MF - Mechanical Fracture	C - Curved
II - Parallel To	
$\perp$ - Perpendicular To	

Form...G.A.-R-3

Golder Associates

RECORD OF BOREHOLE No 02-901 1 OF 1 **METRIC**

PROJECT 021-1155-6			
W.P. 251-99-00	LOCATION N 5003620.7 ;E 340122.0		ORIGINATED BY JS
DIST ___ HWY 7	BOREHOLE TYPE CME-55 Bombardier, NW Casing		COMPILED BY JR
DATUM Geodetic	DATE December 17, 2002		CHECKED BY MIC/LCC

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
133.3	Ground Surface				
0.0	Sand and gravel, trace rootlets (FILL) Compact	1	SS	16	133
132.8 / 0.5	Sand and Silt, some clay, trace gravel (TILL) Compact Brown				
	Interbedded Shale, Limestone and Sandy Dolostone (BEDROCK)				132
	Bedrock cored between 0.5 m and 4.2 m depth. For bedrock coring details, refer to Record of Drillhole 02-901.				131
					130
129.1 / 4.2	End of Borehole				

Column headings also shown: STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa — O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED (20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (10, 20, 30); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Notes:

1. Borehole dry on completion of overburden drilling operations.

2. Water level in piezometer measured as follows:

Mar 21 '03: Frozen at 0.0 m depth

Mar 26 '03: 0.5 m depth (Elev. 132.8 m)

Apr 15 '03: 0.4 m depth (Elev. 132.9 m)

Apr 29 '03: 0.6 m depth (Elev. 132.7 m)

Jun 06 '03: 0.6 m depth (Elev. 132.7 m)

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

MISS_MTO 021-1155-6 MTO.GPJ ON_MOT.GDT 18/8/04

RECORD OF DRILLHOLE: 02-901

PROJECT: 021-1155 (5050)

SHEET 2 OF 2

LOCATION: N 5003620.7 ;E 340122.0

DRILLING DATE: December 18, 2002

DATUM: Geodetic

INCLINATION: -90°   AZIMUTH: ---

DRILL RIG: CME-55 Bombardier

DRILLING CONTRACTOR: Marathon Drilling Ltd.

FR/FX-FRACTURE, F-FAULT, SM-SMOOTH, FL-FLEXURED, BC-BROKEN CORE
CL-CLEAVAGE, J-JOINT, R-ROUGH, UE-UNEVEN, MB-MECH. BREAK
SH-SHEAR, P-POLISHED, ST-STEPPED, W-WAVY, B-BEDDING
VN-VEIN, S-SLICKENSIDED, PL-PLANAR, C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	NOTES WATER LEVELS INSTRUMENTATION
	Rotary Drill / NW Casing	Continued from Record of Borehole	132.84		
		Shale (BEDROCK) Highly weathered to weathered Weak Very thinly to thinly bedded Black	0.46	1	
1	Rotary Drill / NQ Core	Shale (BEDROCK) containing limestone interbeds Fresh Weak to medium strong Very thinly to medium-bedded Black and grey	132.08 1.22	2	
2					
3		Sandy Dolostone (BEDROCK) Fresh to slightly weathered Weak to medium strong Thinly to medium-bedded Grey	130.59 2.71	3	SAND
4					SCREEN
		End of Borehole	129.06 4.24		
5					
6					
7					
8					
9					
10					

Other columns: SYMBOLIC LOG; PENETRATION RATE (m/min); COLOUR % RETURN; FLUSH; RECOVERY (TOTAL CORE %, SOLID CORE %); R.Q.D. %; FRACT. INDEX PER 0.3; DISCONTINUITY DATA (DIP w.r.t. CORE AXIS; TYPE AND SURFACE DESCRIPTION); HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3}); DIAMETRAL POINT LOAD INDEX (MPa)

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: J.S.
CHECKED: M.I.C.

Golder Associates

RECORD OF BOREHOLE No 02-902 1 OF 1 **METRIC**

PROJECT 021-1155-6

W.P. 251-99-00 LOCATION N 5003617.5 ;E 340124.8 ORIGINATED BY JS

DIST HWY 7 BOREHOLE TYPE CME-55 Bombardier, 108 mm I.D. Hollow Stem Augers COMPILED BY JR

DATUM Geodetic DATE December 19, 2002 CHECKED BY MIC/LCC

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
133.2	Ground Surface									
0.0	Sand and gravel, trace rootlets (FILL)	1	SS	40		133				
0.3	Sand and Silt, some clay, trace gravel (TILL) Loose Brown									
132.1 1.1	Shale (BEDROCK) Black									
	End of Borehole Refusal to Auger Advance									

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

Golder Associates

RECORD OF BOREHOLE No 02-903 1 OF 1 **METRIC**

PROJECT 021-1155-6
W.P. 251-99-00
DIST HWY 7
DATUM Geodetic
LOCATION N 5003588.7 ;E 340150.8
BOREHOLE TYPE CME-55 Bombardier, NW Casing
DATE December 17, 2002
ORIGINATED BY JS
COMPILED BY JR
CHECKED BY MIC/LCC

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
132.4	Ground Surface				
0.0	Sand and gravel, trace rootlets (FILL)	1	SS	38	132
0.2	Brown Sand and Silt, some clay, trace to some gravel (TILL) Dense Brown				
131.7 / 0.7	Interbedded Shale, Dolostone and Sandy Dolostone (BEDROCK) Bedrock cored between 0.7 m and 3.8 m depth. For bedrock coring details, refer to Record of Drillhole 02-903.				131, 130, 129
128.6 / 3.8	End of Borehole				

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (10, 20, 30); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Notes:

1. Water level at 0.1 m depth on completion of overburden drilling.

2. Water level in piezometer measured as follows:

Mar 21 '03: 0.0 m depth (Elev. 132.4 m)

Mar 26 '03: 0.2 m above ground surface (at Elev. 132.6 m)

Apr 15 '03: 0.2 m above ground surface (at Elev. 132.6 m)

Apr 29 '03: 0.2 m above ground surface (at Elev. 132.6 m)

Jun 06 '03: 0.0 m depth (Elev. 132.4 m)

MISS_MTO 021-1155-6 MTO.GPJ ON_MOT.GDT 18/8/04

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

PROJECT: 021-1155 (5050)

RECORD OF DRILLHOLE: 02-903

LOCATION: N 5003588.7 ;E 340150.8

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: December 19, 2002

DRILL RIG: CME-55 Bombardier

DRILLING CONTRACTOR: Marathon Drilling Ltd.

DATUM: Geodetic

FR/FX-FRACTURE, F-FAULT, SM-SMOOTH, FL-FLEXURED, BC-BROKEN CORE, CL-CLEAVAGE, J-JOINT, R-ROUGH, UE-UNEVEN, MB-MECH. BREAK, SH-SHEAR, P-POLISHED, ST-STEPPED, W-WAVY, B-BEDDING, VN-VEIN, S-SLICKENSIDED, PL-PLANAR, C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Record of Borehole		131.73		
	Rotary Drill NQ Core	Sandy Dolostone (BEDROCK) Fresh to slightly weathered Weak to medium strong Thinly to medium-bedded Grey		0.67	1	
1					2	
2		Interbedded Shale and Dolostone (BEDROCK) Fresh to slightly weathered Weak to medium strong Very thinly to medium-bedded Black and grey		130.45 1.95	3	SAND
3					4	SCREEN
		End of Borehole		128.59 3.81		

Other column headings: PENETRATION RATE (m/min); FLUSH; COLOUR % RETURN; RECOVERY (TOTAL CORE %, SOLID CORE %); R.Q.D. %; FRACT. INDEX PER 0.3; DISCONTINUITY DATA (DIP w.r.t. CORE AXIS; TYPE AND SURFACE DESCRIPTION); HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}); DIAMETRAL POINT LOAD INDEX (MPa)

Depth scale: 1 2 3 4 5 6 7 8 9 10

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: J.S.

CHECKED: M.I.C.

Golder Associates

PROJECT 021-1155-6	RECORD OF BOREHOLE No 02-904	1 OF 1	METRIC
W.P. 251-99-00	LOCATION N 5003557.6 ;E 340175.3		ORIGINATED BY HEC
DIST ___ HWY 7	BOREHOLE TYPE CME-55 Bombardier, 108 mm I.D. Hollow Stem Augers		COMPILED BY JR
DATUM Geodetic	DATE March 14, 2003		CHECKED BY LCC

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)
132.2	Ground Surface						
0.0	Peat					132	
131.6 / 0.8	Silty Clay, trace sand, Very soft, Grey, Moist to wet	1	SS	1			
		2	SS	2		131	~10
130.7 / 1.5	Sand and Silt, trace clay, trace gravel (TILL), Loose, Grey, Wet	3	SS	36			
	Interbedded Shale, Limestone and Sandy Dolostone (BEDROCK) Bedrock cored between 1.5 m and 5.5 m depth. For bedrock coring details, refer to Record of Drillhole 02-904.	4	SS	20/.15		130	
						129	
						128	
						127	
126.7 / 5.5	End of Borehole Note: Water encountered during drilling at about 0.6 m depth (Elev. 131.6 m).						

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

MISS_MTO 021-1155-6 MTO.GPJ ON_MOT.GDT 18/8/04

PROJECT: 021-1155 (5050)

RECORD OF DRILLHOLE: 02-904

SHEET 2 OF 2

LOCATION: N 5003557.6 ;E 340175.3

DRILLING DATE: March 14, 2003

DATUM: Geodetic

DRILL RIG: CME-55 Bombardier

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR: Marathon Drilling Ltd.

FR/FX-FRACTURE F-FAULT SM-SMOOTH FL-FLEXURED BC-BROKEN CORE
CL-CLEAVAGE J-JOINT R-ROUGH UE-UNEVEN MB-MECH. BREAK
SH-SHEAR P-POLISHED ST-STEPPED W-WAVY B-BEDDING
VN-VEIN S-SLICKENSIDED PL-PLANAR C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	RECOVERY: TOTAL CORE %	RECOVERY: SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Record of Borehole		130.71												
2	Rotary Drill, Hollow Stem Augers	Shale (BEDROCK) Highly weathered to weathered Weak Very thinly to thinly bedded Black		1.49												
3	Rotary Drill, NQ Core	Sandy Dolostone (BEDROCK) containing shale seams Fresh Thinly bedded Medium strong Grey		129.91 2.29	1											
4, 5	Rotary Drill, NQ Core	Dolomitic Limestone (BEDROCK) Fresh Weak to medium strong Thinly to medium-bedded Dark grey-green		128.57 3.63	2, 3											
		End of Borehole		126.66 5.54												

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: J.S.

CHECKED: M.I.C.

MISS_ROCK 021-1155-6 RCK.GPJ GLDR_CAN.GDT 18/8/04 JR

Golder Associates

RECORD OF BOREHOLE No 02-905 1 OF 1 **METRIC**

PROJECT 021-1155-6

W.P. 251-99-00 LOCATION N 5003554.0 ;E 340180.0 ORIGINATED BY HEC

DIST HWY 7 BOREHOLE TYPE CME-55 Bombardier, 108 mm I.D. Hollow Stem Augers COMPILED BY JR

DATUM Geodetic DATE March 14, 2003 CHECKED BY LCC

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
132.3	Ground Surface										
0.0	Ice										
	Peat						132				
131.7											
131.4 / 0.9	Silty Clay, trace sand Firm Grey Moist to wet		1	SS	14				o (~10)		14 32 30 14
130.7 / 1.6	Sand and Silt, trace clay, trace gravel (TILL) Very loose to compact Grey Wet		2	SS	100/.10		131				
130.2 / 2.1	Shale (BEDROCK) Black										
	End of Borehole Refusal to Sampler and Auger Advance										

Note:

Water level in open borehole at 0.3 m depth (Elev. 132.0 m) on completion of drilling.

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

Golder Associates

RECORD OF BOREHOLE No 02-906

METRIC

PROJECT 021-1155-6
W.P. 251-99-00
DIST ____ HWY 7
DATUM Geodetic

LOCATION N 5003609.8 ;E 340111.7
BOREHOLE TYPE CME-55 Bombardier, 108 mm I.D. Hollow Stem Augers
DATE December 17, 2002

ORIGINATED BY JS
COMPILED BY JR
CHECKED BY MIC/LCC

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
133.8	Ground Surface										
0.0	Sand and gravel (FILL)										
	Sand (FILL)										
0.4	Sand and Silt, some clay, some gravel (TILL) Compact Brown Wet		1	SS	20		133				
132.8 / 1.3	Shale (BEDROCK) Black										
	End of borehole Refusal to Auger Advance										

Note:

Water encountered during drilling at about 0.6 m depth (Elev. 133.2 m).

MISS_MTO 021-1155-6 MTO.GPJ ON_MOT.GDT 18/8/04

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

Golder Associates

RECORD OF BOREHOLE No 02-907

METRIC

PROJECT 021-1155-6

W.P. 251-99-00

DIST ____ HWY 7

DATUM Geodetic

LOCATION N 5003605.4 ;E 340116.1

BOREHOLE TYPE CME-55 Bombardier, 108 mm I.D. Hollow Stem Augers

DATE December 17, 2002

ORIGINATED BY JS

COMPILED BY JR

CHECKED BY MIC/LCC

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
133.7	Ground Surface				
0.0	Sand and gravel, trace rootlets (FILL)				
0.4	Sand (FILL)				
	Sand and Silt, some clay, some gravel (TILL) Compact to dense Dark brown Moist to wet	1	SS	89	133
132.8 0.9	Interbedded Shale, Limestone and Sandy Dolostone (BEDROCK). Bedrock cored between 1.2 m and 4.4 m depth. For bedrock coring details, refer to Record of Drillhole 02-907.				132 131 130
129.3 4.4	End of Borehole Note: Water encountered during overburden drilling at about 0.9 m depth (Elev. 132.8m).				

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED; 20 40 60 80 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

PROJECT: 021-1155 (5050)

RECORD OF DRILLHOLE: 02-907

SHEET 2 OF 2

LOCATION: N 5003605.4 ;E 340116.1

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: December 19, 2002

DRILL RIG: CME-55 Bombardier

DRILLING CONTRACTOR: Marathon Drilling Ltd.

DATUM: Geodetic

FR/FX-FRACTURE, F-FAULT, SM-SMOOTH, FL-FLEXURED, BC-BROKEN CORE, CL-CLEAVAGE, J-JOINT, R-ROUGH, UE-UNEVEN, MB-MECH. BREAK, SH-SHEAR, P-POLISHED, ST-STEPPED, W-WAVY, B-BEDDING, VN-VEIN, S-SLICKENSIDED, PL-PLANAR, C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	COLOUR % RETURN	FLUSH	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
	Rotary Drill / Hollow Stem Augers	Continued from Record of Borehole		132.79 / 0.91													
1–3	Rotary Drill / NQ Core	Shale (BEDROCK) containing limestone interbeds Fresh Weak to medium strong Very thinly to medium-bedded Black and grey			1												
3–4		Sandy Dolostone (BEDROCK) Fresh to slightly weathered Weak to medium strong Thinly to medium-bedded Grey		130.71 / 2.99	2												
		End of Borehole		129.28 / 4.42													

Depth scale: 1, 2, 3, 4, 5, 6, 7, 8, 9, 10

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: J.S.

CHECKED: M.I.C.

Golder Associates

RECORD OF BOREHOLE No 02-908

METRIC

PROJECT 021-1155-6
W.P. 251-99-00
DIST HWY 7
DATUM Geodetic

LOCATION N 5003574.3 ;E 340141.2
BOREHOLE TYPE CME-55 Bombardier, 108 mm I.D. Hollow Stem Augers
DATE December 17, 2002

ORIGINATED BY JS
COMPILED BY JR
CHECKED BY MIC/LCC

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
132.7	Ground Surface										
0.0	Topsoil										
0.1	Sand and Silt, some clay, some gravel (TILL) Brown						132				
131.9 0.8	Interbedded Shale, Limestone and Sandy Dolostone (BEDROCK). Bedrock cored between 0.8 m and 3.8 m depth. For bedrock coring details, refer to Record of Drillhole 02-908.						131 130 129				
128.9 3.8	End of Borehole										

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

PROJECT: 021-1155 (5050)

RECORD OF DRILLHOLE: 02-908

LOCATION: N 5003574.3 ;E 340141.2

DRILLING DATE: December 19, 2002

DATUM: Geodetic

INCLINATION: -90°   AZIMUTH: ---

DRILL RIG: CME-55 Bombardier

DRILLING CONTRACTOR: Marathon Drilling Ltd.

FR/FX-FRACTURE, F-FAULT, SM-SMOOTH, FL-FLEXURED, BC-BROKEN CORE, CL-CLEAVAGE, J-JOINT, R-ROUGH, UE-UNEVEN, MB-MECH. BREAK, SH-SHEAR, P-POLISHED, ST-STEPPED, W-WAVY, B-BEDDING, VN-VEIN, S-SLICKENSIDED, PL-PLANAR, C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Record of Borehole		131.91												
1	Rotary Drill, NQ Core	Shale (BEDROCK) containing limestone interbeds Fresh Weak to medium strong Very thinly to medium-bedded Black and grey		130.39												
2		Sandy Dolostone (BEDROCK) Fresh to slightly weathered Weak to medium strong Thinly to medium-bedded Grey		131.12 1.58												
3		Dolomitic Limestone (BEDROCK) Fresh Weak to medium strong Thinly to medium-bedded Dark grey-green		129.93 2.77												
		Shale (BEDROCK) Highly weathered to weathered Weak Very thinly to thinly bedded Black		129.35 3.35												
4		End of Borehole		128.86 3.84												
5																
6																
7																
8																
9																
10																

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: J.S.
CHECKED: M.I.C.

Golder Associates

RECORD OF BOREHOLE No 02-909 1 OF 1 **METRIC**

PROJECT 021-1155-6

W.P. 251-99-00 LOCATION N 5003545.4 ;E 340166.3 ORIGINATED BY JS

DIST ____ HWY 7 BOREHOLE TYPE CME-55 Bombardier, 108 mm I.D. Hollow Stem Augers COMPILED BY JR

DATUM Geodetic DATE December 17, 2002 CHECKED BY MIC/LCC

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
132.3	Ground Surface										
0.0 / 132.0	Topsoil						132				
0.3	Sand and Silt, some clay, some gravel (TILL) Compact Brown Wet		1	SS	10				o (≈10)		4 40 39 17
131.0	Shale (BEDROCK) Black		2	SS	25/.15		131				
130.6 / 1.7	End of Borehole Auger and Sampler Refusal										

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

Golder Associates

RECORD OF BOREHOLE No 02-910

METRIC

PROJECT 021-1155-6
W.P. 251-99-00
DIST ____ HWY 7
DATUM Geodetic

LOCATION N 5003540.6 ;E 340170.5
BOREHOLE TYPE CME-55 Bombardier, NW Casing
DATE December 17, 2002

ORIGINATED BY JS
COMPILED BY JR
CHECKED BY MIC/LCC

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES
132.2	Ground Surface			
0.0	Peat			
131.9	Sand and Gravel			
	Silty Sand, Brown			
131.4	Sand and Silt, some clay, some gravel (TILL), Very loose, Brown to grey, Wet	1	SS	2
131.2 / 1.0	Interbedded Shale, Limestone and Sandy Dolostone (BEDROCK). Bedrock cored between 1.7 m and 5.0 m depth. For bedrock coring details, refer to Record of Drillhole 02-910.	2	SS	86/.25
127.2 / 5.0	End of Borehole			

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (132, 131, 130, 129, 128); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED; 20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (10, 20, 30); UNIT WEIGHT γ kN/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Notes:

1. Water encountered during drilling at approximately 0.5 m depth (Elev. 131.7 m).

2. Water level in piezometer measured as follows:

Apr 30 '03: 0.1 m above ground surface (at Elev. 132.3 m)

Jun 06 '03: 0.0 m depth (Elev. 132.2 m)

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

RECORD OF DRILLHOLE: 02-910

PROJECT: 021-1155 (5050)
LOCATION: N 5003540.6 ;E 340170.5
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: December 19, 2002
DRILL RIG: CME-55 Bombardier
DRILLING CONTRACTOR: Marathon Drilling Ltd.

DATUM: Geodetic

FR/FX-FRACTURE, F-FAULT, SM-SMOOTH, FL-FLEXURED, BC-BROKEN CORE
CL-CLEAVAGE, J-JOINT, R-ROUGH, UE-UNEVEN, MB-MECH. BREAK
SH-SHEAR, P-POLISHED, ST-STEPPED, W-WAVY, B-BEDDING
VN-VEIN, S-SLICKENSIDED, PL-PLANAR, C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Record of Borehole	131.16		
	Rotary Drill, NQ Core	Shale (BEDROCK) Highly weathered to weathered Weak Very thinly to thinly bedded Black	1.04		
			130.37	1	
2	Rotary Drill, NQ Core	Sandy Dolostone (BEDROCK) Fresh to slightly weathered Weak to medium strong Thinly to medium-bedded Grey	1.83	2	
3			128.73		
4		Dolomitic Limestone (BEDROCK) Fresh Weak to medium strong Thinly to medium-bedded Dark grey-green	3.47	3	SAND
				4	SCREEN
5			127.17		
		End of Borehole	5.03		
6					
7					
8					
9					
10					
11					

Other column headings: SYMBOLIC LOG; PENETRATION RATE (m/min); FLUSH; COLOUR % RETURN; RECOVERY (TOTAL CORE % 80 60 40 20; SOLID CORE % 80 60 40 20); R.Q.D. % 80 60 40 20; FRACT. INDEX PER 0.3 (5 10 15 20); DISCONTINUITY DATA (DIP w.r.t. CORE AXIS 0 30 60 90; TYPE AND SURFACE DESCRIPTION); HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3}); DIAMETRAL POINT LOAD INDEX (MPa) 2 4 6

MISS_ROCK 021-1155-6 RCK.GPJ GLDR_CAN.GDT 18/8/04 JR

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: J.S.
CHECKED: M.I.C.

Golder Associates

RECORD OF BOREHOLE No 02-920

METRIC

PROJECT 021-1155-6			ORIGINATED BY JS
W.P. 251-99-00	LOCATION N 5003625.9 ;E 340104.2		COMPILED BY JR
DIST HWY 7	BOREHOLE TYPE CME-55 Bombardier, 108 mm I.D. Hollow Stem Augers		CHECKED BY MIC/LCC
DATUM Geodetic	DATE May 29, 2003		

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED (20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
134.3	Ground Surface										
0.0	Asphalt										
0.1	Sand and gravel (FILL)		1	SS	92/.19		134				
133.5 / 0.8	Sand and Silt, some clay, some gravel (TILL)		2	SS	35						
133.1 / 1.2	Dense Brown Moist		3	SS	115/.15		133				
132.5	Shale (BEDROCK)		4	SS	100/.08						
1.8	Black										
	End of Borehole Auger Refusal										

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

Golder Associates

RECORD OF BOREHOLE No 02-921

METRIC

PROJECT 021-1155-6

W.P. 251-99-00

LOCATION N 5003528.8 ;E 340188.3

ORIGINATED BY JS

DIST HWY 7

BOREHOLE TYPE CME-55 Bombardier, 108 mm I.D. Hollow Stem Augers

COMPILED BY JR

DATUM Geodetic

DATE May 29, 2003

CHECKED BY MIC/LCC

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	SAMPLE TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
132.9	Ground Surface								
0.0	Asphalt								
0.1	Sand and gravel (FILL)								
132.0 / 0.9	Peat	1	SS	5		132	213.0		
131.5 / 1.4	Silty Clay, trace sand, Firm to stiff, Grey-brown	2	SS	2		131			
131.0 / 1.9	Sand and Silt, some clay, some gravel (TILL), Compact, Grey, Wet	3	SS	19					
130.2	Shale (BEDROCK), Black					130			
129.9 / 3.0	End of Borehole, Auger Refusal								

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30

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

Golder Associates

RECORD OF BOREHOLE No 02-930 1 OF 1 **METRIC**

PROJECT 021-1155-6			
W.P. 251-99-00	LOCATION N 5003619.1 ;E 340095.6	ORIGINATED BY HEC	
DIST HWY 7	BOREHOLE TYPE Hand auger	COMPILED BY JR	
DATUM Geodetic	DATE May 28, 2003	CHECKED BY LCC	

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED (20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
134.1	Ground Surface										
0.0	Topsoil		1	CS	-		134				
0.3	Sand and Silt, some clay, some gravel (TILL)		2	CS	-						
	End of Borehole Hand Auger Refusal Note: Borehole dry on completion of hand augering.										

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

Golder Associates

PROJECT 021-1155-6	RECORD OF BOREHOLE No 02-931	1 OF 1	METRIC
W.P. 251-99-00	LOCATION N 5003642.6 ;E 340113.8		ORIGINATED BY HEC
DIST ___ HWY 7	BOREHOLE TYPE Hand auger		COMPILED BY JR
DATUM Geodetic	DATE May 28, 2003		CHECKED BY LCC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X REMOULDED (20 40 60 80 100)	PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						GR	SA	SI	CL
133.9	Ground Surface													
0.0	Topsoil													
	Sand, trace silt		1	CS	-						0	93	5	2
133.4	Red-brown													
0.5	End of Borehole						133							
	Note:													
	Borehole dry on completion of hand augering.													

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

RECORD OF BOREHOLE No 02-932

METRIC

PROJECT 021-1155-6			
W.P. 251-99-00	LOCATION N 5003524.0 ;E 340182.3		ORIGINATED BY HEC
DIST	HWY 7	BOREHOLE TYPE Hand auger	COMPILED BY JR
DATUM Geodetic	DATE May 28, 2003		CHECKED BY LCC

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L / WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
132.1	Ground Surface										
0.0	Peat		1	CS	-		132		461		
131.4	Clayey Silt, trace to some organics, Dark grey-brown		2	CS	-				106.4		
1.0	Silty Clay, Grey-brown										
	End of Borehole										

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

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

Golder Associates

RECORD OF BOREHOLE No 02-933

METRIC

PROJECT 021-1155-6
W.P. 251-99-00
LOCATION N 5003535.2 ;E 340200.0
ORIGINATED BY HEC
DIST
HWY 7
BOREHOLE TYPE Hand auger
COMPILED BY JR
DATUM Geodetic
DATE May 28, 2003
CHECKED BY LCC

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
131.8	Ground Surface										
0.0	Peat										
131.4	Silty Clay Grey-brown		1	CS	-						
0.6	End of Borehole						131				

SOIL PROFILE; SAMPLES

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X REMOULDED; 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%): 10 20 30

MISS_MTO 021-1155-6 MTO.GPJ ON_MOT.GDT 18/8/04

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

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

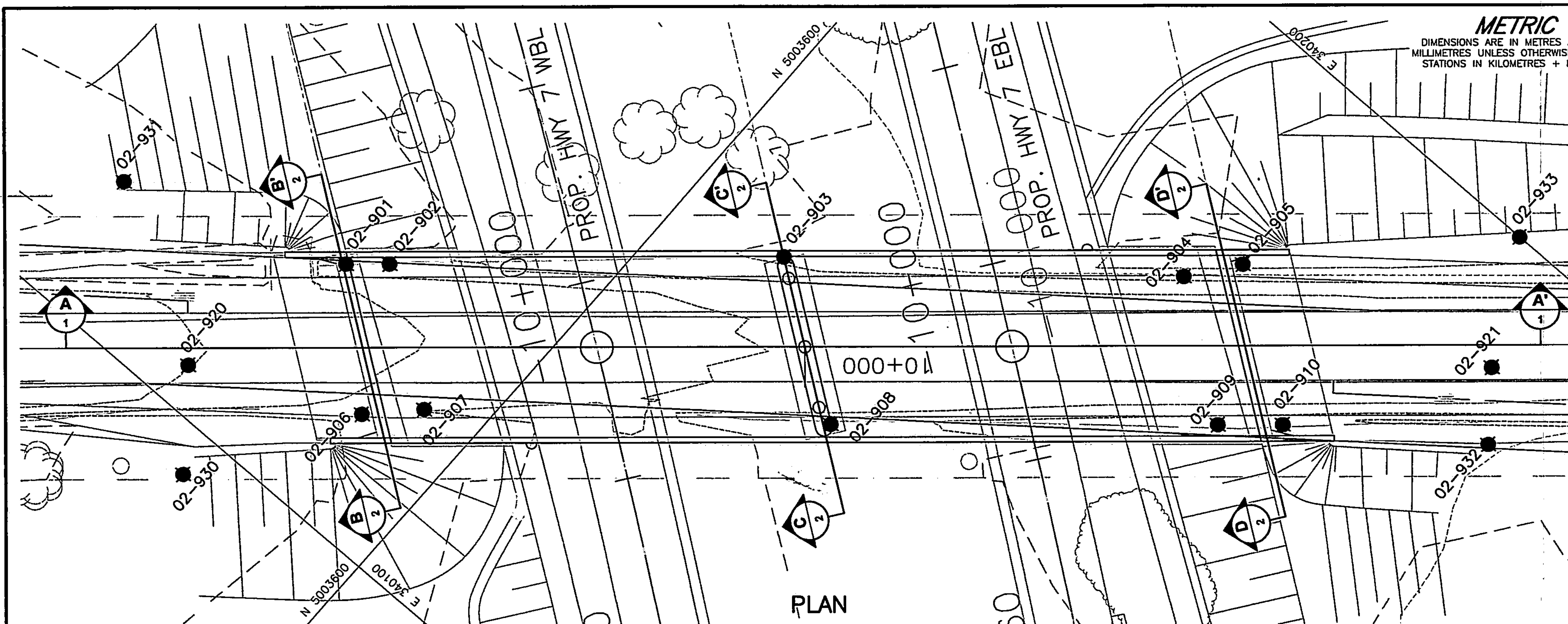
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 251-99-00

HIGHWAY 7 TWINNING
ASHTON STATION ROAD UNDERPASS
BOREHOLE LOCATIONS AND SOIL STRATA

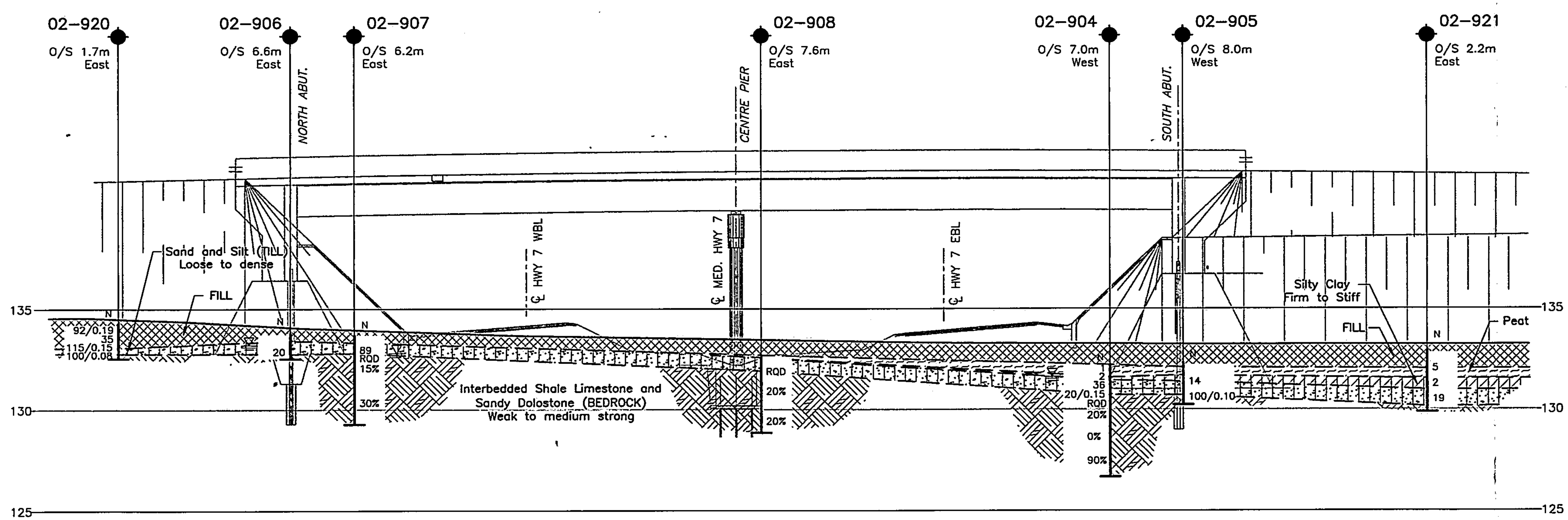
SHEET

Golder Associates
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



PLAN

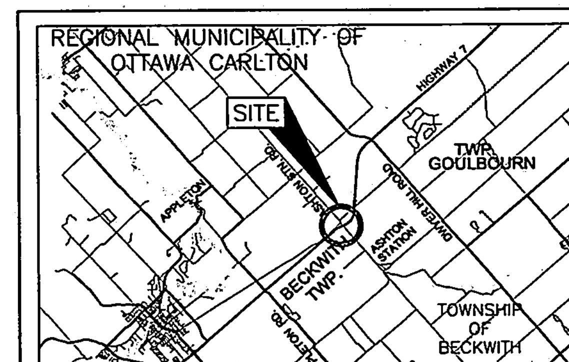
SCALE
5 0 5 10 m



PROFILE ALONG ℄ ASHTON STATION ROAD (A-A' / 1)

HORT. SCALE 5 0 5 10 m

VERT. SCALE 2.5 0 2.5 5 m



KEY PLAN
SCALE
3 0 3 km

LEGEND

- Borehole – Current Investigation
- Seal
- Piezometer
- N — Standard Penetration Test Value
- 16 — Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% — Rock Quality Designation (RQD)
- WL in piezometer, May 2003
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES NORTHING	CO-ORDINATES EASTING
02-901	133.3	5003620.7	340122.0
02-902	133.2	5003617.5	340124.8
02-903	132.4	5003588.7	340150.8
02-904	132.2	5003557.6	340175.3
02-905	132.3	5003554.0	340180.0
02-906	133.8	5003609.8	340111.7
02-907	133.7	5003605.4	340116.1
02-908	132.7	5003574.3	340141.2
02-909	132.3	5003545.4	340166.3
02-910	132.2	5003540.6	340170.5
02-920	134.3	5003625.9	340104.2
02-921	132.9	5003528.8	340188.3
02-930	134.1	5003619.1	340095.6
02-931	133.9	5003642.6	340113.8
02-932	132.1	5003524.0	340182.3
02-933	131.8	5003535.2	340200.0

NOTES

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

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

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

REFERENCE

Electronic general arrangement drawing file received from MTO on July 12, 2005.

NO.	DATE	BY	REVISION

Geocres No.

HWY. 7	PROJECT NO. 021-1155		DIST.
SUBM'D. MIC	CHKD. LCC	DATE: JULY 2005	SITE: 3-719
DRAWN: JDR/MSM	CHKD. MIC	APPD. LCC	DWG. 1

MINISTRY OF TRANSPORTATION, ONTARIO

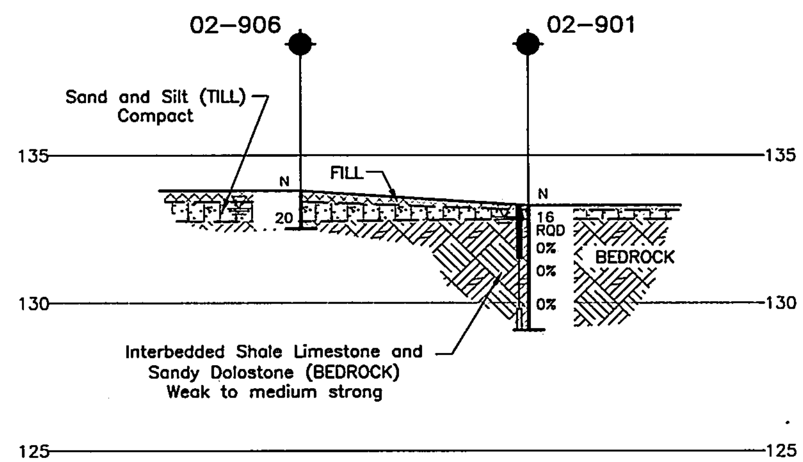
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 251-99-00

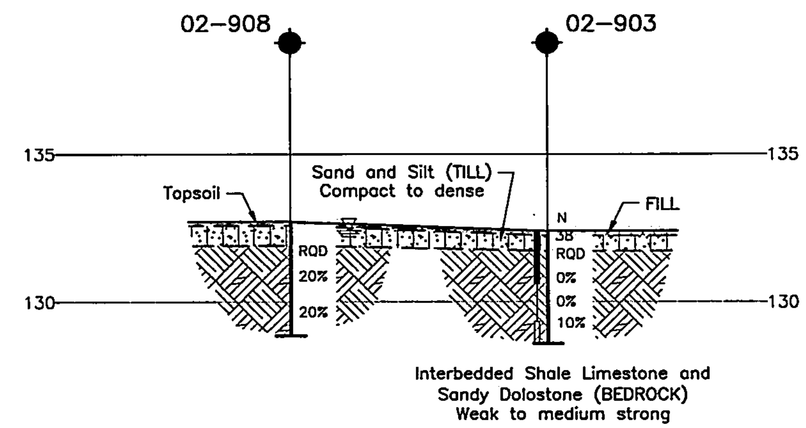
HIGHWAY 7 TWINNING
ASHTON STATION ROAD UNDERPASS
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

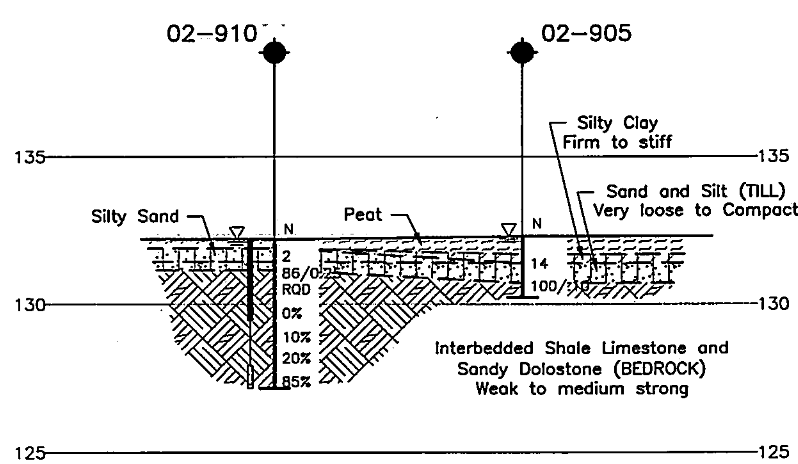
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



B-B' / 2 SECTION AT NORTH ABUTMENT



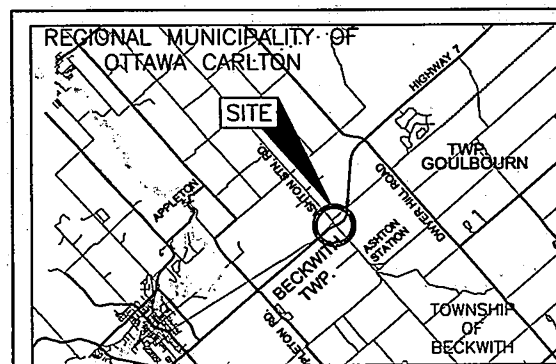
C-C' / 2 SECTION AT CENTRE PIER



D-D' / 2 SECTION AT SOUTH ABUTMENT

HORT. SCALE 5 0 5 10 m

VERT. SCALE 2.5 0 2.5 5 m



KEY PLAN
SCALE 3 0 3 km

LEGEND

- Borehole – Current Investigation
- Seal
- Piezometer
- N — Standard Penetration Test Value
- 16 — Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% — Rock Quality Designation (RQD)
- WL in piezometer, May 2003
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES NORTHING	EASTING
02-901	133.3	5003620.7	340122.0
02-902	133.2	5003617.5	340124.8
02-903	132.4	5003588.7	340150.8
02-904	132.2	5003557.6	340175.3
02-905	132.3	5003554.0	340180.0
02-906	133.8	5003609.8	340111.7
02-907	133.7	5003605.4	340116.1
02-908	132.7	5003574.3	340141.2
02-909	132.3	5003545.4	340166.3
02-910	132.2	5003540.6	340170.5
02-920	134.3	5003625.9	340104.2
02-921	132.9	5003528.8	340188.3
02-930	134.1	5003619.1	340095.6
02-931	133.9	5003642.6	340113.8
02-932	132.1	5003524.0	340182.3
02-933	131.8	5003535.2	340200.0

NOTES

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

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

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

REFERENCE

Electronic general arrangement drawing file received from MTO on July 12, 2005.

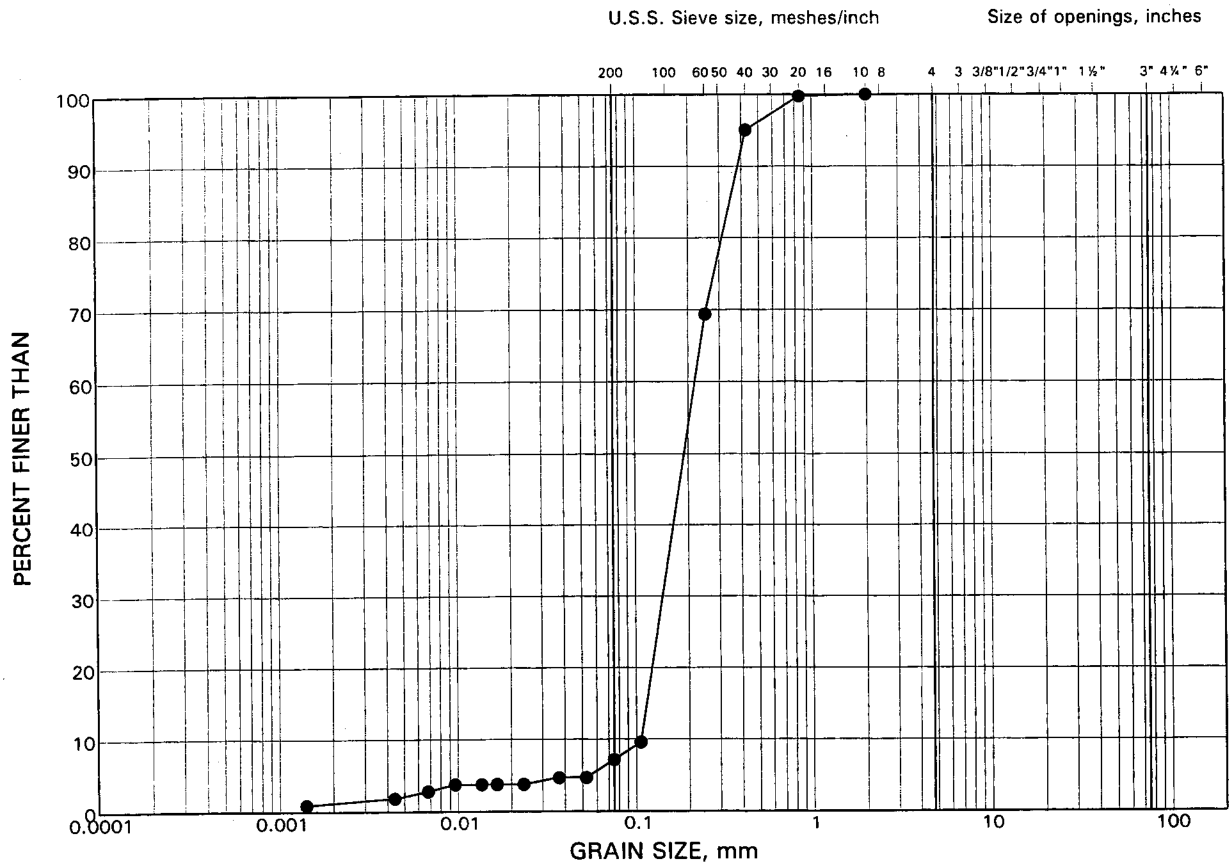
NO.	DATE	BY	REVISION

Geocres No.			
HWY. 7		PROJECT NO. 021-1155	DIST.
SUBM'D. MIC	CHKD. LCC	DATE: JULY 2005	SITE: 3-719
DRAWN: JDR/MSM	CHKD. MIC	APPD. LCC	DWG. 2

PLOT DATE: July 14, 2005

GRAIN SIZE DISTRIBUTION TEST RESULT
Sand

FIGURE 1



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	02-931	1	133.5

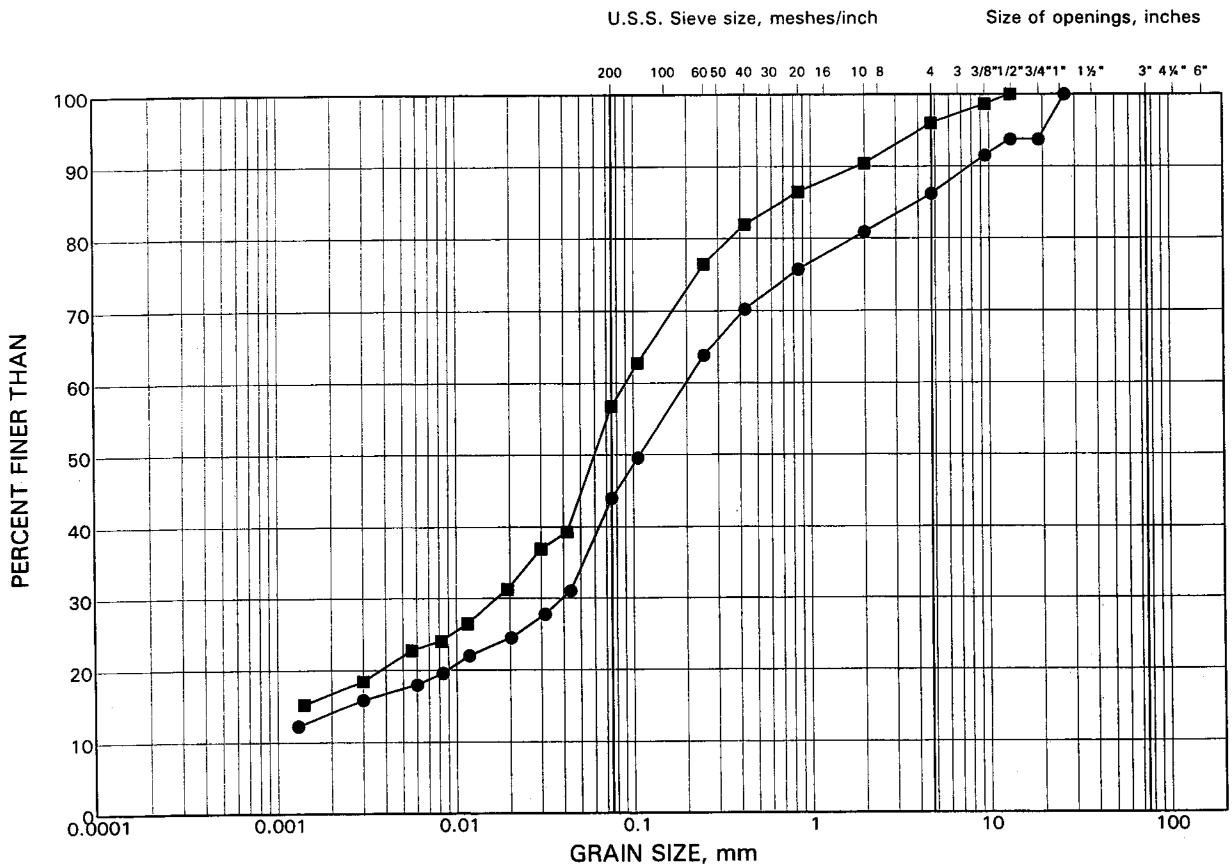
Date 8/17/2004
Project 021-1155

Golder Associates

Prepared by LG
Checked by

GRAIN SIZE DISTRIBUTION TEST RESULTS
Sand and Silt Till

FIGURE 2



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	02-905	1	131.3
■	02-909	1	131.3

Date 8/17/2004
Project 021-1155

Golder Associates

Prepared by LG
Checked by