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**FOUNDATION
INVESTIGATION AND DESIGN REPORT
HAZELDEAN ROAD UNDERPASS
STRUCTURE SITE 3-721
HIGHWAY 7 TWINNING FROM HIGHWAY 417
TO 3 KM WEST OF JINKINSON ROAD
G.W.P. 256-99-00**

Submitted to:

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September 2003



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PART A

**FOUNDATION INVESTIGATION REPORT
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G.W.P. 256-99-00**

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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).

Foundation investigation services are required for the following components:

- **W.P. 256-99-00:** New structures at the Highway 417E-7W ramp and Hazeldean Road, including a high fill embankment along the Highway 417E-7W ramp, high mast light poles, and overhead signs.
- **W.P. 251-99-00 and 252-99-00:** Five new structures at Appleton Road, Ashton Station Road, Dwyer Hill Road, the Trans-Canada Trail, and Lavallee Creek.
- **W.P. 458-98-00:** Widening of two existing structures (the Carp River bridge and CN Rail overpass) into the existing Highway 417 median area, a 900 m long section of high fill embankment within the Highway 417 median in the vicinity of the CN Rail overpass, and overhead signs.

This report addresses the new Hazeldean Road (Regional Road 36) underpass structure and the high fill embankments along Hazeldean Road and the interchange ramps.

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. Scope changes (Scope Change No. 1) related to additional borehole investigation work at the abutments of several structures and the high fill embankment on the Highway 417E-7W ramp are outlined in Golder Associates' letters dated November 12, 2002 and November 18, 2002, respectively. Additional scope changes (Scope Change No. 2) related to additional borehole investigation work associated with overhead signs, high mast light pole foundations, the high fill embankments at the Hazeldean Road site, and additional investigation work for the south/east abutment at the Hazeldean Road site, are outlined in Golder Associates' letter dated May 7, 2003.

The work has been carried out in accordance with Golder Associates' Supplemental Quality Control Plan for Foundation Engineering Services, dated October 2002.

2.0 SITE DESCRIPTION

The proposed Hazeldean Road (Regional Road 36) underpass structure is located approximately 2.5 km southwest of Highway 417, in West Carleton Township in the Regional Municipality of Ottawa-Carleton. The proposed underpass structure is designated as MTO's Structure Site 3-721.

The terrain in the vicinity of the site is flat to gently undulating, with the natural ground surface varying from about Elevation 135 m to 132 m, generally declining toward the east. The existing Highway 7 grade at the proposed structure location is at about Elevation 134 to 135 m, slightly above the surrounding natural grade which is at about Elevation 133.5 m. Ditches between 1.5 m and 3 m in depth are present along both sides of Highway 7. Bedrock is exposed in the ditch to the west of the proposed structure site.

The site is poorly drained, as evidenced by the presence of up to about 300 mm of standing water and/or ice to the north of the existing Highway 7 corridor at the time of the drilling investigation, and the occurrence of surficial organic soil in areas of the site. To the north and south of the existing Highway 7 corridor, the site is forested.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out for the proposed Hazeldean Road underpass structure, in which a total of fourteen boreholes were advanced. In December 2002, ten boreholes (Boreholes 02-501 to 02-510) were advanced within the limits of the proposed foundation elements. Two boreholes (Boreholes 02-520 and 02-521) were advanced in January 2003 at the south (east) and north (west) approach embankments, respectively; and two additional boreholes (Boreholes 02-511 and 02-512) were advanced in June 2003 at the final location for the east abutment. Boreholes 02-501 to 02-509, 02-511, and 02-512 were advanced by hollow stem augers using a bombardier-mounted drill rig, supplied and operated by Marathon Drilling Ltd. of Ottawa, Ontario. Due to the presence of the highway drainage ditch at Borehole 02-510, and difficult access / heavy bush at Boreholes 02-520 and 02-521, these boreholes were advanced by Marathon Drilling Ltd.'s portable drilling equipment using either Pionjar drilling techniques, or a half-weight hammer in conjunction with casing or augers.

The boreholes were advanced to auger and/or sampler refusal which occurred at depths between 1.0 m and 4.4 m below the existing ground surface at the borehole locations. 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, except in Borehole 02-520 where Pionjar drilling techniques were used. In six of the ten boreholes advanced at the proposed foundation locations, the boreholes were advanced about 3 m into the bedrock by coring using NQ-size coring equipment, except in Borehole 02-510 (advanced by portable drilling equipment), in which BQ-size coring equipment was used. 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 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 to Golder Associates' laboratory in Mississauga for testing. Index and classification tests consisting of water content determinations, Atterberg Limits testing and grain size distribution analyses were carried out on selected soil 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.

<i>Borehole Number</i>	<i>Borehole Location</i>	<i>MTM NAD83 Northing (m)</i>	<i>MTM NAD83 Easting (m)</i>	<i>Ground Surface Elevation (m)</i>
02-501	West abutment	5,012,322.3	346,338.2	133.4
02-502	West abutment	5,012,321.3	346,343.0	133.6
02-503	Centre pier	5,012,314.3	346,382.4	134.1
02-504	East abutment	5,012,305.7	346,418.6	135.2
02-505	East abutment	5,012,306.3	346,423.7	134.9
02-506	West abutment	5,012,306.6	346,336.0	133.6
02-507	West abutment	5,012,302.0	346,340.4	133.5
02-508	Centre pier	5,012,296.2	346,380.0	134.2
02-509	East abutment	5,012,289.8	346,414.8	134.9
02-510	East abutment	5,012,287.1	346,421.1	132.0
02-511	East abutment	5,012,305.5	346,426.9	134.5
02-512	East abutment	5,012,284.5	346,427.7	133.5
02-520	East approach	5,012,296.5	346,443.6	133.5
02-521	West approach	5,012,313.9	346,316.8	133.6

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 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

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, fourteen boreholes were advanced within the limits of the foundation elements and immediate approach embankments for the proposed underpass structure. The borehole locations and ground surface elevations are shown on Drawing 1.

The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of in-situ and laboratory testing are given on the Record of Borehole sheets and Figures 1 to 3. 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 Highway 7 embankment fill at the proposed centre pier and east abutment locations, and between 200 mm and 600 mm of peat outside of the Highway 7 embankment, at the locations of the west abutment and the west and east approaches. The fill and peat overlie relatively thin overburden soils consisting of sand, over silty sand to sand and silt till. These surficial soils are, in turn, underlain by limestone bedrock that was encountered between about 1 m and 4.4 m depth (at about Elevation 130.0 m to 131.6 m).

A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections, and stratigraphic profiles and sections of this site are shown on Drawings 1 and 2.

4.2.1 Fill

Fill, associated with the construction of the existing Highway 7 embankment, was encountered in six of the boreholes advanced as part of this subsurface investigation. In Boreholes 02-503 and 02-508, drilled on the northwest shoulder of Highway 7 in the vicinity of the proposed centre pier, approximately 0.8 m of fill was encountered, with the base of the fill at Elevation 133.3 m and 133.4 m in the two boreholes. In Boreholes 02-504, 02-505, 02-509 and 02-5511 advanced in the vicinity of the proposed east abutment, between 2.4 m and 3.1 m of fill was encountered, with the base of the fill between Elevation 131.9 m and 132.8 m.

The existing Highway 7 embankment fill ranges in composition from silty sand containing some gravel to sand and gravel containing trace to some silt. Grain size distribution test results obtained from two samples of the existing embankment fill are shown on Figure 1. Cobbles were noted within the fill during augering in many of the boreholes, and trace organics and/or wood fibres were noted within some of the fill samples.

The Standard Penetration Test (SPT) "N" values measured within the existing embankment fill range from 3 to 20 blows per 0.3 m of penetration, indicating that the fill is loose to compact.

4.2.2 Topsoil and Peat

Topsoil or peat was encountered immediately below ground surface in those boreholes that were advanced outside of the existing Highway 7 embankment and its associated drainage ditches. Between 200 mm and 600 mm of peat was encountered in Boreholes 02-501, 02-502, 02-506 and 02-507, located in the area of the proposed west abutment, and Boreholes 02-521 and 02-520, located within the limits of the proposed west and east approach embankments, respectively. At the east abutment, about 400 mm of topsoil was encountered at ground surface in Borehole 02-512, and about 300 mm of topsoil was encountered below the existing Highway 7 embankment fill in Borehole 02-511.

4.2.3 Surficial Silty Sand to Sand and Gravel

Below the existing fill and topsoil or peat lies a silty sand to sand and gravel deposit with a total thickness of between 0.4 m and 1.8 m. This stratum was absent in Boreholes 02-509 and 02-511, advanced near the southeast shoulder of the existing highway, where existing fill directly overlies the silty sand to sand and silt till deposit. The base of the surficial sand to silty sand deposit was encountered between about Elevation 131.5 m and 132.5 m.

In five of the boreholes (Boreholes 02-503, 02-504, 02-505, 02-508 and 02-510) advanced near the limits of the existing Highway 7 shoulders, the upper 0.2 m to 0.7 m of this deposit consists of dark brown to black sand containing trace to some silt and organic matter; this material has been interpreted as alluvium, deposited in a former natural drainage channel at the site. Below this in Boreholes 02-503, 02-504 and 02-508, and from the base of the peat or topsoil in Boreholes 02-501, 02-502, 02-506, 02-507 and 02-512, the deposit consists of silty sand containing trace gravel, to sand containing trace to some silt and trace gravel, to sand and gravel containing trace silt. The results of four grain size distribution tests obtained for samples from this stratum are shown on Figure 2.

Measured Standard Penetration Test (SPT) "N" values in this deposit range from 6 to 30 blows per 0.3 m of penetration, with an average of 17 blows per 0.3 m of penetration. The measured SPT "N" values in the alluvium portion of the deposit were typically 8 and 9 blows per 0.3 m of penetration. The surficial deposit therefore varies from loose to compact, but is generally compact.

4.2.4 Silty Sand Till to Sand and Silt Till

The surficial sand deposit is underlain by a till deposit that grades in composition from silty sand to sand and silt, containing trace to some gravel and clay. Cobbles and boulders were noted or inferred within the till in some of the boreholes. Grain size distribution test results obtained on two samples of this till are shown on Figure 3 following the text of this report. The till varies from compact to very dense, based on measured SPT "N" values of 13 to greater than 100 blows per 0.3 m of penetration; however, this till deposit is typically dense to very dense, based on typical SPT "N" values of 30 to greater than 100 blows per 0.3 m of penetration.

The till deposit ranges in thickness from 0.3 m to 2.3 m, although it is absent in Borehole 02-503 where the surficial sand directly overlies limestone bedrock; the till deposit is typically 1 m to 2 m thick. The base of this till deposit was encountered between about Elevations 130.3 m and 131.6 m in the boreholes.

4.2.5 Limestone Bedrock

Limestone bedrock underlies the till deposit at this site. In the boreholes put down at the proposed bridge foundations, the surface of the bedrock was encountered between Elevations 130.0 m and 131.6 m. 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 eight of the boreholes; the surface of the limestone bedrock was inferred in five of the remaining boreholes by refusal to split-spoon sampler and/or auger advance. In Borehole 02-520, which was advanced using Pionjar drilling techniques, refusal occurred before bedrock was reached.

<i>Borehole Location</i>	<i>Borehole Number</i>	<i>Ground Surface Elevation</i>	<i>Depth to Bedrock</i>	<i>Bedrock Surface Elevation</i>
West approach	02-521	133.6 m	2.7 m	130.9 m
West abutment	02-501	133.4 m	3.4 m	130.0 m (Cored)
	02-502	133.6 m	3.3 m	130.3 m
	02-506	133.6 m	3.2 m	130.4 m
	02-507	133.5 m	3.2 m	130.4 m (Cored)
Centre pier	02-503	134.1 m	2.6 m	131.5 m (Cored)
	02-508	134.2 m	2.6 m	131.6 m (Cored)
East abutment	02-504	135.2 m	4.4 m	130.8 m (Cored)
	02-505	134.9 m	4.2 m	130.7 m
	02-509	134.9 m	4.3 m	130.6 m
	02-510	132.0 m	1.0 m	131.0 m (Cored)
	02-511	134.5 m	3.9 m	130.6 m (Cored)
	02-512	133.5 m	2.5 m	131.0 m (Cored)
East approach	02-520	133.5 m	N/A	N/A

The limestone bedrock at the site is a member of the Ottawa Formation. It is fresh, weak to medium strong, very thinly- to medium-bedded, and contains characteristic shale seams. Rock Quality Designation (RQD) values measured on recovered bedrock core samples ranged from 0 to 90 per cent (but typically from about 20 to 60 per cent) in the upper 1 m of the bedrock, and from 35 to 75 per cent in the lower 2 m of the recovered bedrock core; the typical RQD values indicate that the bedrock is of poor to fair quality. The discontinuities observed in the rock core are typically horizontal to sub-horizontal, associated with the bedding planes and stylolitic features, although some vertical to sub-vertical jointing was also observed.

A description of some 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

Up to 300 mm of standing water was present during the December 2002 field investigation in the forested area to the northwest of the existing Highway 7 corridor. In the boreholes where water was noted on completion of drilling, the water level was at depths ranging from about 0.2 m above ground surface to 2.6 m below ground surface (about Elevation 132.2 m to 133.6 m). The water level was generally higher (about Elevation 133.4 m to 133.6 m) in the boreholes located to the northwest of the existing highway, and generally lower (about Elevation 131.5 m to 133.1 m) in those boreholes located adjacent to or within the existing drainage ditches.

Three piezometers were installed within the overburden soil deposits at this site. The water level measured in the piezometers on January 8, 2003 varied from Elevation 133.1 m to 132.4 m, as summarized in the following table:

<i>Borehole No.</i>	<i>Borehole Location</i>	<i>Water Level on Jan 8, 2003</i>	
		<i>Elevation</i>	<i>Depth</i>
02-501	North abutment	133.1 m	0.3 m
02-508	Centre pier	132.4 m	1.8 m
02-505	South abutment	132.3 m	2.6 m

The lower water levels encountered in Boreholes 02-505 and 02-508 are considered to be a result of these boreholes' location adjacent to the existing highway drainage ditches.

Based on the observations during drilling and the measurements made in the piezometers, it is considered that the groundwater table is at about Elevation 133.1 m to 133.6 m and that it is influenced by the drainage ditches. It should be noted that groundwater levels are expected to fluctuate seasonally and are expected to rise during wet periods of the year.

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PART B

**FOUNDATION DESIGN REPORT
HAZELDEAN ROAD UNDERPASS
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HIGHWAY 7 TWINNING FROM HIGHWAY 417
TO 3 KM WEST OF JINKINSON ROAD
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5.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides foundation design recommendations for the proposed Hazeldean 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 the proposed Hazeldean Road underpass structure will be two spans, with each span about 45 m in length. Three alternative integral or semi-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).
- Semi-integral abutments supported on spread footings.
- 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.

5.2 Bridge and Retaining Wall Foundation Options

In the immediate vicinity of the proposed two-span underpass structure, the natural ground surface is at about Elevation 133.5 m, and the existing Highway 7 grade is at about Elevation 135 m. It is understood that the proposed Hazeldean Road grade will be at about Elevation 142.7 m to 143 m within the proposed structure and approach embankment limits. The approach embankments will be approximately 9.5 m high relative to the existing natural grade at the site.

The native soils at the site consist of topsoil and peat overlying generally compact surficial sands and silty sands, in turn underlain by a generally dense to very dense silty sand till to sand and silt till deposit. These overburden soils are underlain by weak to medium strong limestone bedrock, the surface of which was encountered in the boreholes between Elevations 130.0 m and 131.6 m, about

1 m to 4.5 m below the existing grade at the site. The limestone 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 for the abutments and associated wing walls or retaining walls may be placed on a compacted Granular "A" pad within the approach embankment fill. The overburden soils at the site are suitable for the support of RSS walls, either as wingwalls or in front of the abutments.

Since integral abutments are under consideration, steel H-piles can also be considered for support of the abutments. Given the proposed underside of pile cap at about Elevation 137 m and the limestone bedrock surface at the abutment locations at about Elevation 130 m to 131 m, it is estimated that the pile length will be approximately 6 m to 7 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. Steel H-piles are likely not practicable at the centre pier, owing to the relatively shallow depth to bedrock and the frost protection requirements.

As an alternative to spread footings or steel H-pile foundations, drilled shaft foundations resting on or socketted into the limestone bedrock could be used for support of the abutments and centre pier (although drilled shafts may not be practical at the centre pier due to the shallow bedrock depth). This option 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 drilled shaft foundations for the pier, bridge abutments and associated retaining walls, as applicable, 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.

5.3 Spread Footings

5.3.1 Geotechnical Resistance for Spread Footings on Bedrock

The bridge pier, abutments and any associated concrete cantilever wing walls / retaining walls may be supported on spread footings placed on the properly prepared limestone bedrock. The surface of the bedrock was encountered in the boreholes at the proposed foundation elements between Elevations 130 m and 131.6 m, as summarized in the following table.

<i>Foundation Element</i>	<i>Borehole Numbers</i>	<i>Depth to Bedrock</i>	<i>Bedrock Surface Elevation</i>
West abutment	02-501 and 02-506	3.2 m to 3.4 m	130.0 m to 130.4 m
Centre pier	02-503, 02-508	2.6 m	131.5 m to 131.6 m
East abutment	02-511 and 02-512	2.9 m to 3.5 m (<1.0 m in ditch)	130.6 m to 131.0 m

Based on the borehole results, there is some variability in the bedrock surface within the limits of each foundation element. In addition, the upper portion of the bedrock is, in local areas, highly fractured (RQD values of less than 30 per cent, as encountered in Boreholes 02-503, 02-504, 02-507 and 02-508), 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:

1. The following founding elevations may be assumed:

West abutment:	Elevation 130.5 m
Centre pier:	Elevation 131.6 m
East abutment:	Elevation 131.0 m

In this case, 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 to medium strong bedrock is avoided.

2. Alternatively, the following design founding levels may be assumed:

West abutment:	Elevation 129.7 m
Centre pier:	Elevation 131.2 m
East abutment:	Elevation 130.4 m

In this case, excavation of the higher portions of the bedrock will be required within the foundation footprints. Based on the borehole results, subexcavation of up to about 0.8 m of bedrock will be required in some foundation areas. It is noted that the bedrock is weak to moderately strong (corresponding to unconfined compressive strengths in the range of 5 MPa to 50 MPa), making excavation relatively difficult particularly where only small depths are needed. Bedrock excavation could be carried out using hoe ramming techniques; however, line drilling and pre-shearing techniques, if properly executed and inspected, should provide better control over the configuration of the founding surface.

3. As a third option, 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 limestone bedrock or on mass concrete may be designed based on a factored geotechnical resistance at Ultimate Limit States (ULS) of 2,000 kPa; this factored geotechnical resistance at ULS has been determined taking into

consideration the RQD values measured in the upper portion of the bedrock in Boreholes 02-502 and 02-508, which are located at the centre pier. 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 limestone bedrock is considered to be an unyielding material; as such, ULS conditions will govern for this foundation type.

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 curve for cohesive soils.

5.3.2 Geotechnical Resistance for "Perched" Footings

Spread footings for the west and east abutments may be placed on a compacted Granular "A" pad within the approach embankment fill. A factored geotechnical resistance at ULS of 900 kPa may be assumed for design. The geotechnical resistance at SLS will depend on the thickness of Granular "A" and the relative density and thickness of the underlying fill and loose / compact surficial soils; a value of 350 kPa may be assumed for design purposes. These values assume that the Granular "A" pad has a thickness of at least one footing width.

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.

5.3.3 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, $\tan \phi'$, for cast-in-place concrete footings and the 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.

<i>Subgrade</i>	<i>Coefficient of Friction ($\tan \phi'$)</i>
Limestone bedrock	0.7
Compacted Granular "A" pad	0.5

If necessary, the sliding resistance at the centre pier 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, where the rock mass is essentially as strong as or stronger than concrete, 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.

The use of a shear key excavated into the bedrock could be considered as an alternative to dowelling. However, as noted in Table 1, subexcavation of the medium strong limestone bedrock will be difficult and time-consuming particularly as compared with dowelling.

5.3.4 Frost Protection

For spread footings founded on the properly prepared limestone bedrock at this site, frost susceptibility is not an issue.

5.4 Steel H-Pile Foundations

Steel H-piles driven to found on the limestone bedrock may be used for support of the abutments. The surface of the limestone bedrock was encountered in the boreholes between Elevation 130.0 m and 131.0 m at the proposed north and south abutment locations, as noted below:

<i>Foundation Element</i>	<i>Borehole Numbers</i>	<i>Depth to Bedrock</i>	<i>Bedrock Surface Elevation</i>
West abutment	02-501 and 02-506	3.2 m to 3.4 m	130.0 m to 130.4 m
East abutment	02-511 and 02-512	2.9 m to 3.5 m (<1.0 m in ditch)	130.6 m to 131.0 m

It is understood that the abutment pile caps will be “perched” within the approach embankment fill such that the base of the pile caps will be at about Elevation 137 m. The piles will therefore be driven through the fill as placed and will be approximately 6 m to 7 m long without socketting into bedrock.

If necessary, to resist seismic forces, the pile toes could be placed within the bedrock. The limestone bedrock is weak to moderately strong (corresponding to compressive strengths of up to about 50 MPa), however, and this would 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.

5.4.1 Axial Geotechnical Resistance

For HP 310 x 110 piles driven to found on or socketted nominally into the limestone 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 will 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 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 OPSD 3301.00. The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile. All of these factors must be taken into consideration in establishing the driving criteria to ensure that the piles are not overdriven and to avoid possible damage to the piles. In this regard, it is a generally accepted practice to reduce the hammer energy after abrupt peaking occurs on the bedrock surface, and then to gradually increase the energy over a series of blows to seat the pile.

5.4.2 Resistance to Lateral Loads

Lateral loading could be resisted fully or partially by the use of battered steel H-piles. 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, k_h , is based on the following equation for granular soils:

$$k_h = \frac{n_h z}{B} \quad \text{where} \quad \begin{array}{l} n_h \text{ is the constant of horizontal subgrade reaction, as given below;} \\ z \text{ is the depth (m); and} \\ B \text{ is the pile diameter (m).} \end{array}$$

The following ranges for the value of n_h may be assumed in the structural analysis, using the stratigraphic sections provided on Drawings 1 and 2. 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 and the pier.

<i>Soil Unit</i>	<i>n_h</i>
Embankment fill (assumed to be compacted granular fill) and existing surficial soil above Elevation 133.5 m	5 to 15 MPa/m
Surficial soils below Elevation 133.5 m (i.e. below the highest groundwater level measured in the vicinity of the proposed structure)	2 to 8 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:

<i>Pile Spacing in Direction of Loading $d = \text{Pile Diameter}$</i>	<i>Reduction Factor</i>
8d	1.0
6d	0.7
4d	0.4
3d	0.25

5.4.3 Frost Protection

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

5.5 Drilled Shaft Foundations

Drilled shafts founded on or socketted into the limestone bedrock may be used for support of the abutments. It is assumed that the abutment pile caps would be “perched” within the approach embankment fill in order to minimize the abutment wall height. Based on an assumed pile cap base at Elevation 137 m and the bedrock surface between Elevations 130 m and 131 m, the length of drilled shafts used for abutment support will be approximately 6 m to 7 m. The use of drilled shaft foundations is not considered appropriate for support of the centre pier or of any concrete wing walls / retaining walls associated with the structure, owing to the shallow depth to bedrock and the frost protection depth required for the pile caps.

As discussed in Section 5.4, the limestone bedrock at the site is weak to medium strong (corresponding to unconfined compressive strengths up to about 50 MPa). Formation of socket holes in the bedrock is feasible; however, it will be necessary to use rock coring or churn drilling techniques to advance the holes. It is noted that the stronger layers would 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 cohesionless and water-bearing; these soils will flow into the auger hole during caisson 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.

5.5.1 Axial Geotechnical Resistance

Drilled shafts founded on the surface of the limestone bedrock, or socketted nominally (less than 1 m) into the bedrock, should be designed based on end-bearing resistance and a factored geotechnical resistance at ULS of 3 MPa should be used. Serviceability Limit States resistances do not apply to drilled shafts founded on the 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 drilled shaft construction, to ensure that all loose and/or fractured rock has been removed from the foundation areas.

5.5.2 Resistance to Lateral Loads

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

5.5.3 Frost Protection

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

5.6 Retained Soil System (RSS) Walls

A mechanically-reinforced soil retaining wall system (retained soil system or RSS wall) consists of granular fill placed and compacted in layers, and reinforced with metal or fabric strips or grids. A facing material, typically pre-cast concrete panels mechanically fastened to the reinforcing strips or grids, is used to form the face of the reinforced soil structure and to prevent the loss of fill material.

The use of RSS walls is considered appropriate for the proposed wing walls / retaining walls at the proposed Hazeldean Road underpass. Depending upon where the walls are used (i.e. in front of the abutments in a false abutment configuration, or on the approach embankment side slopes), it is estimated that RSS walls at this site would be between about 7 m and 9 m high. Typically, RSS walls are founded at least 0.3 m below the existing ground surface in front of the wall, below any topsoil and/or peat. 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 surficial sand to silty sand deposit:

<i>Wall Height</i>	<i>Assumed Footing Width</i>	<i>Factored Geotechnical Resistance at ULS</i>
7 m	4.7 m	250 kPa
9 m	6 m	300 kPa

The settlement of the walls will occur as a result of the loading due to the embankment itself, since the walls are incorporated into the embankment. The geotechnical resistance at SLS, for 25 mm of settlement resulting from the combined wall and embankment loading, may be taken as 250 kPa. The majority of the settlement of the RSS walls will occur during construction since the founding soils are essentially granular (i.e. silty sand to sand and gravel, and cohesionless till), overlying bedrock at a shallow depth.

The resistance to lateral forces / sliding resistance between the compacted Granular "A" and the subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, $\tan \delta'$, between the compacted granular of the RSS wall and the loose to compact surficial sand to silty sand may be taken as 0.45. This represents an unfactored value; 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. In this regard, the internal stability must also be checked for seismic loading. 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.

The liquefaction potential of the soils below and adjacent to the RSS wall 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 less than 1.1 against liquefaction is obtained for magnitude 6.2 earthquake events, for localized areas in front of the RSS wall (i.e. areas of sand with low fines content, low "N" values representative of a loose state of compaction, and low confining stresses). Due to the shallow depth of liquefiable

soil relative to the reinforced area, the yield acceleration required to reduce the factor of safety against instability to 1.0 is higher than the maximum acceleration for this site. Based on the simplified Newmark method and correlations by Makdisi and Seed (1978), it is estimated that the potential deformation of the RSS wall due to liquefaction during the design earthquake event will be nominal (less than about 50 mm).

Given the above results, there is no strict requirement for liquefaction mitigation measures associated with the RSS wall construction. However, if these walls are adopted as part of a "false abutment" configuration, consideration could be given to improvement of the soils immediately in front of the RSS wall to lessen the RSS wall deformation as a result of liquefaction. If deformations of less than 50 mm are required, soil improvement methods could include densification, removal and recompaction, grouting, or permanent drainage so that the pore water pressure rise necessary to trigger liquefaction is controlled. Given the subsurface conditions, removal and recompaction of the liquefiable soil layer is considered to be the most cost effective soil improvement approach for this site, if such improvement is deemed necessary.

5.7 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 compacted in loose lifts not greater than 200 mm in thickness to 95 per cent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. 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 OPSS 501.06; heavy compaction equipment should not be used within a lateral distance behind the structure equal to the current height of the fill above the base of the structure. 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/m ³	21 kN/m ³
Coefficients of static lateral earth pressure:		
Active, K_a	0.27	0.31
At rest, K_o	0.43	0.47

- 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}

	Case I	Case II	
		Granular A	Granular B Type II
Yielding wall	0.40	0.32	0.36
Non-yielding wall	0.80	0.63	0.71

- 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 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:

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

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).

5.8 Embankment Design and Construction

The construction of the Hazeldean Road underpass will require placement of up to about 9.5 m of fill within the limits of the approach embankments. Based on the borehole results, the embankment subgrade soils consist of loose to compact sand to silty sand.

As part of Scope Change 2 for this assignment, the stability of and settlement associated with high fill embankments (equal to or greater than 5 m in height) have also been assessed. Based on the *Update to the Preliminary Design Study* for Highway 7 between Carleton Place and Highway 417, prepared by Totten Sims Hubicki and dated June 2002, a total length of about 1.5 km of high fill embankments will be constructed at the Highway 7 – Hazeldean Road interchange, as summarized in the following table:

<i>Area</i>	<i>Station</i>	<i>Embankment Height</i>	<i>High Embankment Length</i>
Hazeldean Road	9+730 to 10+300	5 m to 9 m	570 m
Hazeldean N – Highway 7W Ramp	19+720 to 19+970 (Approx. 21+280 to 21+040 on new alignment)	5 m to 7 m	250 m
Hazeldean S – Highway 7W Ramp	20+180 to 20+500	5 m to 10 m	320 m
Highway 7W – Hazeldean N/S Ramp	21+100 to 21+370	5 m to 8.5 m	270 m
Highway 7E – Hazeldean N/S Ramp	22+650 to 22+710	5 m to 7 m	60 m

As part of the pavement investigation work carried out at the Hazeldean Road site by Jacques, Whitford and Associates Limited, shallow auger holes were advanced at 25 m to 50 m spacings along the proposed Hazeldean Road and ramp alignments. These auger holes were typically advanced to 1.5 m depth or to auger refusal (on probable bedrock), whichever was encountered first. The results of the pavement investigation boreholes indicate that the depth to bedrock is typically between 1 m and 2 m below the natural ground surface at the site. The overburden deposits consist of topsoil and peat overlying sand, silt and gravel layers that are, in turn, underlain by a silty sand to sand and silt till stratum. No Standard Penetration Test (SPT) “N” values were obtained as part of the foundation investigation; however, the foundation boreholes advanced at the structure location indicate that the surficial sand, silt and gravel layers have a loose to compact relative density, and that the till deposit has a compact to very dense relative density.

5.8.1 Subgrade Preparation and Embankment Construction

Any topsoil, organic matter and softened / loosened soils should be stripped from below the embankment areas, and all subgrade soils should be proof-rolled prior to fill placement. Embankment fill should be placed in regular lifts with loose thickness not exceeding 300 mm, and be compacted to at least 95 per cent of the material’s Standard Proctor maximum dry density. The final lift prior to placement of the granular subbase and base courses should be compacted to 100 per cent of the Standard Proctor maximum dry density. 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 where seepage is present.

5.8.2 Embankment Stability

With appropriate subgrade preparation and proper placement and compaction of embankment fill materials, the 8 m to 9.5 m high immediate 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 5 m to 10 m high fill embankments along Hazeldean Road and the ramps will also have a factor of safety of greater than 1.3 against deep-seated slope instability, provided that the embankment side slopes are maintained at 2H:1V.

Static slope stability analyses for the above embankment configurations were carried out using the following parameters, derived from field and laboratory testing and accepted correlations, with the commercially available program SLOPE/W, produced by Geo-Slope International Ltd.

<i>Soil Deposit</i>	<i>Bulk Unit Weight</i>	<i>Effective Friction Angle</i>	<i>Undrained Shear Strength</i>
Embankment Fill	20 – 22 kN/m ³	32° to 35°	–
Silty Sand to Sand and Gravel	19 – 20 kN/m ³	30° to 32°	–
Silty Sand Till to Sand and Silt Till	21 kN/m ³	32° to 35°	–

The liquefaction potential of the soils below the immediate approach embankments and other high fill embankment areas 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 less than 1.1 against liquefaction is obtained for magnitude 6.2 earthquake events, for localized areas under the embankment toe (i.e. areas of sand with low fines content, low “N” values representative of a loose state of compaction, and low confining stresses under less than about 2 m of embankment fill). Pseudo-static methods of embankment stability analysis indicate that a yield acceleration of approximately 0.1g results in a factor of safety against side slope instability of 1.0. Based on this yield acceleration and the correlation proposed by Makdisi and Seed, it is estimated that between 50 mm and 300 mm of deformation of the embankment could result under the design earthquake event. Localized failures at the embankment toe, resulting in steepening of the embankment side slopes, could occur. Since deep-seated global instability is not anticipated under the design earthquake event, localized embankment toe failures would be mainly a maintenance issue. This should be considered in the life cycle costing when assessing the relative costs of the works.

5.8.3 Embankment Settlement

Settlement of the immediate approach embankments and other high fill embankments at the site will occur due to compression of the new embankment fill itself, as well as compression of the relatively thin, cohesionless overburden soils. 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.

Settlement analyses for the foundation soils were carried out using the commercially available computer program Unisettle. The immediate compression of the loose to compact silty sand to sand and gravel, and the loose to very dense sand and silt 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:

<i>Soil Unit</i>	<i>Bulk Unit Weight</i>	<i>Elastic Modulus</i>
Embankment fill (range of parameters assumed for earth fill and granular fill)	20 – 22 kN/m ³	–
Loose to compact silty sand to sand and gravel	19 – 20 kN/m ³	10 MPa to 40 MPa
Compact to very dense silty sand till to sand and silt till	21 kN/m ³	50 MPa to 200 MPa

Provided that proper subgrade preparation is carried out, the settlement of the cohesionless foundation soils for the immediate approach embankments is expected to be less than 25 mm at the west approach, and less than 15 mm at the east approach, as a result of construction of the 9.5 m high approach embankments. This compression is expected to occur during construction.

Since the distribution of loose soils (relative to compact to very dense soils) was not determined in the pavement investigation, it is not possible to accurately predict the settlement profile along each of the high fill embankments. Instead, settlement analyses were conducted for the range of embankment heights (5 m to 10 m), using the ranges of parameters provided above, and assuming a total overburden thickness ranging from less than 1 m up to about 4 m. For the 5 m to 10 m high fill embankments along Hazeldean Road and the interchange ramps, the anticipated settlement of the silty sand to sand and gravel, and silty sand till to sand and silt till soils ranges from less than 10 mm to about 50 mm. The larger predicted settlements correspond to higher fill heights and "looser" soils. The majority of the predicted settlement will occur during construction.

5.9 Design and Construction Considerations

5.9.1 Excavation

Excavations to expose the bedrock surface (to allow construction of spread footings, or to permit socket formation within the bedrock) would extend to about 2.5 m to 4.5 m depth below the existing ground surface. The excavations will typically extend through about 1 m of existing embankment fill (where present), then 0.5 m to 1.5 m of loose to compact silty sand to sand and gravel, followed by about 0.5 m to 2 m of compact to very dense till. The groundwater level at the site is relatively shallow, generally at or less than 1 m below the ground surface, or about 2.5 m to 3.5 m above the bedrock 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 existing fill, water-bearing silty sand to sand and gravel, and water-bearing till 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 fully dewatered. If full dewatering is not achieved, as discussed in Section 5.9.2, shallower side slopes of 3H:1V will be required.

Depending on construction staging and the proximity of the foundation excavations to the operating portions of Highway 7, temporary roadway protection may be required to permit construction of the new Hazeldean Road underpass structure. In particular, the proposed east abutment and centre pier are located within 10 m of the existing Highway 7, and if full dewatering cannot be achieved, open-cut excavations with portions of their side slopes oriented at 3H:1V may impact on the operation of the roadway. Consideration could be given to the use of a soldier pile and lagging system using rakers to provide lateral support. It may be necessary to socket the soldier piles into the bedrock to provide sufficient lateral resistance at the pile toe once the excavation is advanced. The temporary excavation support system should be designed and constructed in accordance with MTO's Special Provision 539S01. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in SP 539S01.

5.9.2 Groundwater and Surface Water Control

The groundwater level at the site is generally at or less than about 1 m below the natural ground surface. Excavations to expose the bedrock surface, either for founding of spread footings or to enable formation of sockets or a trench within the bedrock to provide toe support to piles, will require groundwater control. The shallow depth to bedrock at the site will dictate the type of groundwater control system that may be used. A shallow eductor system could be used to lower the groundwater level within the overburden. This system would have to be supplemented with

pumping from sumps formed within the bedrock at the base of the excavations in order to fully dewater the site and permit the use of 1H:1V side slopes. Alternatively, shallower side slopes (3H:1V) could be using in conjunction with sumping alone.

It is noted that the area to the north of the existing Highway 7 alignment, in the vicinity of the proposed north abutment and approach embankment, was flooded by approximately 300 mm of standing water / ice at the time of the subsurface investigation. Consideration should be given to scheduling the construction work to avoid foundation excavation in the spring.

As noted in Section 5.5, if drilled shafts 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.

5.9.3 Obstructions


The native soils at the site are glacially-derived and, as such, are expected to contain cobbles and boulders. Indeed, the presence of cobbles and/or boulders was inferred from grinding of the augers during borehole advance, and numerous cobbles were recovered during augering.

The presence of such obstructions will affect the installation of driven steel H-piles or drilled shaft foundations, and will also affect the installation of soldier piles and soil or rock anchors (tie-backs) if temporary roadway protection measures are adopted at the site. Ultimately, provision will have to be made in the Contract Documents to ensure that the Contractor is equipped to handle such obstructions.

GOLDER ASSOCIATES LTD.



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LCC/ASP/FJH/lcc

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TABLE 1
COMPARISON OF FOUNDATION ALTERNATIVES
HAZELDEAN ROAD UNDERPASS STRUCTURE

<i>Foundation Option</i>	<i>Feasibility</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Relative Foundation Costs</i>	<i>Risks/Consequences</i>
Spread footings founded on bedrock – high founding elevation with placement of mass concrete	<ul style="list-style-type: none"> • Feasible at centre pier • Feasible at abutments but would result in very high abutment wall 	<ul style="list-style-type: none"> • Dowelling of footing into bedrock provides a more economical alternative to socketting H-piles or drilled shafts into the medium strong bedrock to resist lateral loading, if required • Differential settlement between centre pier and abutments founded on bedrock will be minimal 	<ul style="list-style-type: none"> • Requires excavation down to 2.5 m to 4.5 m depth with 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) 	<ul style="list-style-type: none"> • Less expensive than spread footings supported on bedrock using the lower founding elevation, but more expensive than “perched” footings due to groundwater control costs 	<ul style="list-style-type: none"> • Potential for difficulties with groundwater control that could affect the construction schedule
Spread footings founded on bedrock – low founding elevation with up to 0.8 m of subexcavation of bedrock, OR subexcavation of shear key into bedrock	<ul style="list-style-type: none"> • Feasible at centre pier • Feasible at abutments but would result in very high abutment wall 	<ul style="list-style-type: none"> • Dowelling of footing into bedrock provides a more economical alternative to socketting H-piles or drilled shafts into bedrock to resist lateral loading / seismic forces, if required • Differential settlement between centre pier and abutments founded on bedrock will be minimal 	<ul style="list-style-type: none"> • Subexcavation of medium strong bedrock expected to be difficult and time-consuming, especially compared to surface preparation requirements for higher founding option or to dowelling for resistance of lateral loading • Requires excavation down to typically 2.5 m to 4.5 m depth with 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) 	<ul style="list-style-type: none"> • Probably most expensive spread footing option, owing to costs associated with bedrock subexcavation as well as groundwater control 	<ul style="list-style-type: none"> • Potential for difficulties with groundwater control that could affect the construction schedule • Potential for difficulties with bedrock subexcavation
Spread footings founded within approach embankment fill (for abutments and concrete retaining walls)	<ul style="list-style-type: none"> • Feasible at both abutments 	<ul style="list-style-type: none"> • Minimal groundwater control required • Minimizes abutment wall height 	<ul style="list-style-type: none"> • Minor differential settlement between abutments founded within embankment fill and centre pier on bedrock 	<ul style="list-style-type: none"> • Probably least expensive foundation construction costs 	<ul style="list-style-type: none"> • Minor differential settlement between foundation elements, since centre pier will be supported on bedrock

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

<i>Foundation Option</i>	<i>Feasibility</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Relative Foundation Costs</i>	<i>Risks/Consequences</i>
Steel H-pile foundations driven to found on or drilled to socket nominally into bedrock	<ul style="list-style-type: none"> • 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 	<ul style="list-style-type: none"> • 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 	<ul style="list-style-type: none"> • If lateral / seismic loading conditions merit, socketting of pile toe into medium strong bedrock would require coring or churn drilling, with use of temporary liner to support overburden soils • Possibility of encountering cobbles/boulders during pile driving 	<ul style="list-style-type: none"> • Less expensive than drilled shaft option with rock sockets, owing to potentially smaller socket diameter 	<ul style="list-style-type: none"> • If required for pile toe fixity, socketting into the medium strong bedrock expected to be difficult and time-consuming
Steel H-pile foundations placed in trench within bedrock	<ul style="list-style-type: none"> • Feasible for support of abutments • Not feasible for support of centre pier due to shallow depth to bedrock 	<ul style="list-style-type: none"> • Negligible settlement, particularly as compared to abutment footings "perched" within approach embankments 	<ul style="list-style-type: none"> • Would require open excavation up to 4.5 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 bedrock expected to be difficult 	<ul style="list-style-type: none"> • Expected to be more expensive than socketting option due to groundwater control and bedrock excavation costs 	<ul style="list-style-type: none"> • 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	<ul style="list-style-type: none"> • Feasible for support of centre pier and abutments 	<ul style="list-style-type: none"> • 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 	<ul style="list-style-type: none"> • Temporary liners required to minimize disturbance to surrounding soils • Possibility of encountering cobbles or boulders during drilled shaft installation • Socketting into bedrock would require coring or churn drilling; churn drilling expected to be relatively slow, and may be difficult with large diameter core holes 	<ul style="list-style-type: none"> • May be more expensive than steel H-pile option if rock sockets are necessary, owing to potentially larger socket diameter 	<ul style="list-style-type: none"> • If required for pile toe fixity, socketting into the medium strong bedrock expected to be difficult and time-consuming

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

<i>Foundation Option</i>	<i>Feasibility</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Relative Foundation Costs</i>	<i>Risks/Consequences</i>
Retained Soil System (RSS) walls	<ul style="list-style-type: none">• Soils at both abutments are suitable for support of RSS walls	<ul style="list-style-type: none">• Minimal excavation and groundwater control required for construction	<ul style="list-style-type: none">• Some settlement will occur	<ul style="list-style-type: none">• Generally less expensive than concrete retaining wall foundations	<ul style="list-style-type: none">• More settlement than for concrete retaining walls supported on steel H-piles or drilled shafts founded on the 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

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

(b) Cohesive Soils

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

Dynamic Cone Penetration Resistance; N_4 :

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

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

IV. SOIL TESTS

w water content
w_p plastic limit
w_l liquid limit
C consolidation (oedometer) test
CHEM chemical analysis (refer to text)
CID consolidated isotropically drained triaxial test¹
CIU consolidated isotropically undrained triaxial test with porewater pressure measurement¹
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
SO₄ 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.

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LIST OF SYMBOLS

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

I. GENERAL

π	= 3.1416
$\ln x$	natural logarithm of x
$\log_{10} x$ or $\log x$	logarithm of x to base 10
g	acceleration due to gravity
t	time
F	factor of safety
V	volume
W	weight

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta \sigma$
ϵ	linear strain
ϵ_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stresses (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 (con't.)

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)

(c) 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

(d) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (overconsolidated range)
C_s	swelling index
C_α	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	Overconsolidation ratio $= \sigma'_p / \sigma'_{vo}$

(e) Shear Strength

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

Notes: 1. $\tau = c' + \sigma' \tan \phi'$

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
- Parallel To	
⊥ - Perpendicular To	

PROJECT 021-1155-2				RECORD OF BOREHOLE No 02-501				1 OF 1				METRIC					
W.P. 256-99-00				LOCATION N 5012322.3 ; E 346338.2				ORIGINATED BY D.B.									
DIST _____ HWY 7				BOREHOLE TYPE CME 55 Bombardier, 108mm I.D. Hollow Stem Auger				COMPILED BY M.I.C.									
DATUM Geodetic				DATE Dec. 16, 2002				CHECKED BY L.C.C.									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
133.4	Ground Surface																
0.0	Peat																
0.2	Sand, trace silt Loose Grey Wet						133										
132.3			1	SS	30												
1.1	Sand and Silt, some gravel, trace clay with cobbles and boulders (Till) Compact to very dense Grey Wet						132										
			2	SS	13												
							131										
			3	SS	141/ 0.28												
130.0			4	SS	46/0.23		130										
3.4	LIMESTONE (BEDROCK) with shale seams Fresh Weak to medium strong Thinly to medium - bedded Grey Bedrock cored between 3.4 m and 6.5 m depth For bedrock coring details refer to Record of Drillhole 02-501						129										
							128										
126.9							127										
6.5	End of Borehole Notes: 1. Water level on completion of drilling at 0.1m above ground surface. 2. Water level in piezometer at 0.3m depth (Elev. 133.1m) on Jan. 8, 2003. * Sampler bouncing after 46 blows.																

MISS_MTO 021-1155-2.GPJ ON MOT.GDT 5/9/03

PROJECT: 021-1155-2

RECORD OF DRILLHOLE: 02-501

SHEET 2 OF 2

LOCATION: N 5012322.3 ; E 346338.2

DRILLING DATE: Dec. 16, 2002

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55 Bombardier, 200mm I.D. Hollow Stem Auger

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH % RETURN	FR/FX-FRACTURE-FAULT												SM-SMOOTH				FL-FLEXURED				BC-BROKEN CORE				DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
								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													
RECOVERY		R.Q.D. %		FRACT. INDEX PER 0.3		DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K, cm/sec																									
TOTAL CORE %	SOLID CORE %	%	%	%	%	%	%	%	%	%	%	%	%	%	%	%	%	%	%	%	%												
100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100												
10 ⁻⁸	10 ⁻⁶	10 ⁻⁴	10 ⁻²	10 ⁻⁸	10 ⁻⁶	10 ⁻⁴	10 ⁻²	10 ⁻⁸	10 ⁻⁶	10 ⁻⁴	10 ⁻²	10 ⁻⁸	10 ⁻⁶	10 ⁻⁴	10 ⁻²	10 ⁻⁸	10 ⁻⁶	10 ⁻⁴	10 ⁻²	10 ⁻⁸	10 ⁻⁶												
																						2											

4	Refer to previous page	130.00																								
5	Rotary Drill NO Core	LIMESTONE (BEDROCK) with shale seams Fresh Weak to medium strong Thinly to medium - bedded Grey		3.43																						
6				1	100																					
7																										
8																										
9																										
10																										
11																										
12																										
13																										
		End of Borehole		126.95																						
				6.48																						

MISS ROCK ROCK1155-2.GPJ GLDR CAN.GDT 5/9/03 JFC

DEPTH SCALE




1 : 50



LOGGED: D.B.

CHECKED: M.I.C.

PROJECT 021-1155-2		RECORD OF BOREHOLE No 02-502		1 OF 1	METRIC
W.P. 256-99-00		LOCATION N 5012321.3 E 346343.0		ORIGINATED BY D.B.	
DIST HWY 7		BOREHOLE TYPE CME 55 Bombardier, 108mm I.D. Hollow Stem Auger		COMPILED BY M.I.C.	
DATUM Geodetic		DATE Dec. 17, 2002		CHECKED BY L.C.C.	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X REMOULDED					WATER CONTENT (%) W _p — W — W _L				
133.6	Ground Surface																
0.0	Peat Wet																
133.2																	
0.4	Sand, trace gravel Loose Grey Wet		1	SS	6											1 97 (2)	
131.8			2	SS	17												
1.8	Sand and Silt, some gravel, trace clay, with cobbles and boulders (Till) Dense to very dense Grey Wet																
			3	SS	53												
130.3			4	SS	20/0.20												
3.3	End of Borehole Refusal to sampler and auger penetration Note: Water level in open borehole at 0.1 m depth (Elev. 133.5m) on completion of drilling.																

MISS_MTO 021-1155-2.GPJ ON MOT.GDT 5/9/03

MISS_MTO 021-1155-2.GPJ ON MOT.GDT 5/9/03

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

PROJECT: 021-1155-2

RECORD OF DRILLHOLE: 02-503

SHEET 2 OF 2

LOCATION: N 5012314.3 ; E 346382.4

DRILLING DATE: Dec. 16, 2002

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55 Bombardier, 200mm I.D. Hollow Stem Auger

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH	COLOUR % RETURN	FR/FX-FRACTURE-FAULT				SM-SMOOTH				FL-FLEXURED				BC-BROKEN CORE				DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
									CL-CLEAVAGE		J-JOINT		R-ROUGH		UE-UNEVEN		MB-MECH. BREAK		B-BEDDING							
									SH-SHEAR		P-POLISHED		ST-STEPPED		W-WAVY											
									VN-VEIN		S-SLICKENSIDED		PL-PLANAR		C-CURVED											
		Refer to previous page		131.50																						
3		LIMESTONE (BEDROCK) with shale seams		2.59																						
4	Rotary Drill NQ Core	Fresh Weak to medium strong Very thinly to thinly - bedded Grey			1	100																				
5					2	100																				
6		End of Borehole		128.42																						
7				5.67																						
8																										
9																										
10																										
11																										
12																										

MISS. ROCK ROCK1155-2 GPJ GLDR. CAN GDT 5/9/03 JFC

DEPTH SCALE

1 : 50



LOGGED: P.A.H.

CHECKED: M.I.C.

PROJECT <u>021-1155-2</u>		RECORD OF BOREHOLE No 02-504		1 OF 1	METRIC
W.P. <u>256-99-00</u>	LOCATION <u>N 5012305.8 : E 346418.6</u>	ORIGINATED BY <u>P.A.H./D.B.</u>			
DIST <u> </u> HWY <u>7</u>	BOREHOLE TYPE <u>CME 55 Bombardier, 108mm I.D. Hollow Stem Auger</u>	COMPILED BY <u>M.I.C.</u>			
DATUM <u>Geodetic</u>	DATE <u>Dec. 12, 13, 2002</u>	CHECKED BY <u>L.C.C.</u>			

[illegible]

MISS_MTO 021-1155-2.GPJ ON_MOT.GDT 5/9/03

PROJECT: 021-1155-2

RECORD OF DRILLHOLE: 02-504

SHEET 2 OF 2

LOCATION: N 5012305.8 ;E 346418.6

DRILLING DATE: Dec. 12, 13, 2002

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55 Bombardier, 200mm I.D. Hollow Stem Auger

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH	COLOUR % RETURN	FR/FX-FRACTURE-FAULT				SM-SMOOTH		FL-FLEXURED		BC-BROKEN CORE		DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
									CL-CLEAVAGE	J-JOINT	R-ROUGH	UE-UNEVEN	MB-MECH. BREAK	FL-FLEXURED	UE-UNEVEN	MB-MECH. BREAK				
									SH-SHEAR	P-POLISHED	ST-STEPPED	W-WAVY	B-BEDDING	FL-FLEXURED	UE-UNEVEN	MB-MECH. BREAK				
									VN-VEIN	S-SLICKENSIDED	PL-PLANAR	C-CURVED								
									RECOVERY		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			
									TOTAL CORE %	SOLID CORE %							10 ⁻⁶	10 ⁻⁵	10 ⁻⁴	
		Refer to previous page		130.80					100	100	100	100	100	100	100	100	100	100	100	
5		LIMESTONE (BEDROCK) with shale seams		4.40	1															
		Fresh																		
		Weak to medium strong																		
		Thinly - bedded																		
		Grey																		
6					2															
7					3															
		End of Borehole		127.71																
		Note:		7.49																
8		Vertical joint split the core sample recovered from Run No.1, resulting in RQD = 0%.																		
9																				
10																				
11																				
12																				
13																				
14																				

MISS ROCK ROCK1155-2.GPJ GLDR CAN.GDT 5/9/03 JFC

DEPTH SCALE




1 : 50



LOGGED: P.A.H.

CHECKED: M.I.C.

PROJECT 021-1155-2				RECORD OF BOREHOLE No 02-505				1 OF 1		METRIC							
W.P. 256-99-00				LOCATION N 5012306.3 ; E 346423.7				ORIGINATED BY D.B.									
DIST _____ HWY 7				BOREHOLE TYPE CME 55 Bombardier, 108mm I.D. Hollow Stem Auger				COMPILED BY M.I.C.									
DATUM Geodetic				DATE Dec. 13, 2002				CHECKED BY L.C.C.									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
134.9	Ground Surface																
0.0	Silty sand, some gravel, trace to some organics, with cobbles and boulders (Fill) Very loose to loose Grey Moist to wet		1	SS	6												
			2	SS	5												
			3	SS	3												
132.2	Sand, some silt, trace clay and organic matter (Alluvium) Very loose Dark brown to black Wet		4	SS	31												
2.7			5	SS	100/0.1												
131.7	Sand and Silt, some gravel, trace clay with cobbles and boulders (Till) Compact to very dense Grey Wet																
3.2																	
130.7	End of Borehole Refusal to auger penetration																
4.2	Note: Water Level in piezometer at 2.6 m depth (Elev. 132.3m) on Jan. 8, 2003.																

PROJECT <u>021-1155-2</u>			RECORD OF BOREHOLE No 02-506			1 OF 1			METRIC																		
W.P. <u>256-99-00</u>			LOCATION <u>N 5012306.6 , E 346336.0</u>			ORIGINATED BY <u>D.B.</u>																					
DIST <u> </u> HWY <u>7</u>			BOREHOLE TYPE <u>CME 55 Bombardier, 108mm I.D. Hollow Stem Auger</u>			COMPILED BY <u>M.I.C.</u>																					
DATUM <u>Geodetic</u>			DATE <u>Dec. 17, 2002</u>			CHECKED BY <u>L.C.C.</u>																					
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			ELEVATION SCALE			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			WATER CONTENT (%)			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES																						
133.6 0.0	Ground Surface Peat Wet																										
133.0 0.6	Sand, trace silt Compact Wet																										
132.5 1.1	Silty Sand, some gravel, trace clay with cobbles and boulders, some sand seams (Till) Compact to very dense Grey Wet		1	SS	17																						
			2	SS	28																						
			3	SS	90/0.20																						
130.4 3.2	End of Borehole Auger Refusal Notes: 1. Water noted during drilling at approximately ground surface. 2. Refusal to split-spoon sampling and augering occurred at 2.6m depth; presence of boulder inferred. 3. Borehole moved 1.0m and augered to refusal at 3.2m depth (Elev. 130.4m).																										

MISS_MTO 021-1155-2.GPJ ON MOT.GDT 5/9/03

PROJECT 021-1155-2

RECORD OF BOREHOLE No 02-507

1 OF 1

METRIC

W.P. 256-99-00

LOCATION N 5012302.0, E 346340.4

ORIGINATED BY D.B.

DIST _____ HWY 7

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

COMPILED BY M.I.C.

DATUM Geodetic

DATE Dec. 17, 2002

CHECKED BY L.C.C.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
133.5	Ground Surface													
0.0	Peat Wet													
132.9							133							
0.6	Sand, trace silt Loose Wet													
132.5														
1.0	Sand and Silt, some gravel, trace clay, with cobbles and boulders, some sand seams (Till) Compact to very dense Grey Wet		1	SS	17									1 94 (5)
							132							
			2	SS	13									
			3	SS	89		131							
130.4			4	SS	39/0.10									
3.2	LIMESTONE (BEDROCK) with shale seams Fresh Weak to medium strong Thinly-bedded Grey Bedrock cored between 3.2 m and 6.2 m depth For bedrock coring details refer to Record of Drillhole 02-507						130							
							129							
							128							
127.3														
6.2	End of Borehole Notes: 1. Water level in open borehole at 0.1 m depth (Elev. 133.4m) on completion of drilling. * Sampler bouncing after 39 blows.													

MISS MTO 021-1155-2.GPJ ON MOT.GDT 5/9/03

PROJECT: 021-1155-2

RECORD OF DRILLHOLE: 02-507

SHEET 2 OF 2

LOCATION: N 5012302.0 ; E 346340.4

DRILLING DATE: Dec. 17, 2002

DATUM: Geodetic

INCLINATION: -90°

AZIMUTH: —

DRILL RIG: CME 55 Bombardier, 200mm I.D. Hollow Stem Auger

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	CORRELATION														NOTES		
				ELEV.		RUN No.	PENETRATION RATE (m/min)	FLUSH % RETURN	RECOVERY			R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY		DIAMETRAL POINT LOAD INDEX (MPa)	WATER LEVELS INSTRUMENTATION	
				DEPTH (m)	ELEV. (m)				TOTAL CORE %	SOLID CORE %	TYPE AND SURFACE DESCRIPTION			K _f cm/sec	K _s cm/sec					
		Refer to previous page		130.40																
		LIMESTONE (BEDROCK) with shale seams		3.15																
4		Fresh			1		100													
5		Weak to medium strong																		
6		Thinly-bedded			2		100													
6		Grey																		
		End of Borehole		127.32																
				6.23																
7																				
8																				
9																				
10																				
11																				
12																				
13																				

DEPTH SCALE

1 : 50



LOGGED: D.B.

CHECKED: M.I.C.

MISS ROCK ROCK1155-2.GPJ GLDR CAN GDT 5/9/03 JFC

PROJECT 021-1155-2			RECORD OF BOREHOLE No 02-508			1 OF 1			METRIC														
W.P. 256-99-00			LOCATION N 5012296.2, E 346380.0			ORIGINATED BY P.A.H.																	
DIST _____ HWY 7			BOREHOLE TYPE CME 55 Bombardier, 108mm I.D. Hollow Stem Auger			COMPILED BY M.I.C.																	
DATUM Geodetic			DATE Dec. 16, 2002			CHECKED BY L.C.C.																	
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			SHEAR STRENGTH kPa			WATER CONTENT (%)			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	20 40 60 80 100	W _p	W	W _L	20 40 60 80 100	10 20 30	γ	GR SA SI CL							
134.2 0.0	Ground Surface Silty Sand, some organic matter, some cobbles (Fill) Dark brown to black Moist		1	AS	-		134																
133.4 0.8	Sand, trace organics (Alluvium) Loose Dark brown to black Moist		2	SS	9		133																
132.7 1.5	Sand, trace silt Compact Grey- brown to grey Wet		3	SS	29		132									3 86 9 2							
131.9 2.3	Sand and Silt, some gravel, trace clay (Till) Compact Grey		4	SS	*		131																
131.6 2.6	Wet LIMESTONE (BEDROCK) with shale seams Fresh Weak to medium strong Very thinly to thinly- bedded Grey Bedrock cored between 2.6 m and 5.1 m depth For bedrock coring details refer to Record of Drillhole 02-508					130.15	130																
129.1 5.1	End of Borehole Notes: 1. Difficulties with coring equipment and freezing temperatures resulted in coring of only 2.5m of rock. 2. Water Level in piezometer at 1.8 m depth (Elev. 132.4m) on Jan. 8, 2003. * Sampler bouncing after 13 blows.																						

MISS_MTO 021-1155-2.GPJ ON MOT.GDT 5/9/03

PROJECT: 021-1155-2

RECORD OF DRILLHOLE: 02-508

SHEET 2 OF 2

LOCATION: N 5012296.2 ; E 346380.0

DRILLING DATE: Dec. 16, 2002

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55 Bombardier, 200mm I.D. Hollow Stem Auger

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH	COLOUR % RETURN	FR/FX-FRACTURE-F-FAULT				SM-SMOOTH				FL-FLEXURED				BC-BROKEN CORE				NOTES WATER LEVELS INSTRUMENTATION
									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										
									RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY				DIAMETRAL POINT LOAD INDEX (MPa)						
									TOTAL CORE %	SOLID CORE %			DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	10 ⁻⁴	10 ⁻³	10 ⁻²	2	4	6					
		Refer to previous page		131.80					100	100	100	100	100												
3	Rotary Drill NQ Core	LIMESTONE (BEDROCK) with shale seams		2.59																					
		Fresh Weak to medium strong Very thinly to thinly - bedded Grey		1	100																				
4					2				100																
5		End of Borehole		129.10																					
		Note: Low RQD's are a result of problems with coring equipment (inner tube did not lock properly within core barrel).		5.09																					
6																									
7																									
8																									
9																									
10																									
11																									
12																									

MISS ROCK ROCK1155-2.GPJ GLDR CAN GDT 5/9/03 JFC

DEPTH SCALE

1 : 50



LOGGED: P.A.H.

CHECKED: M.I.C.

PROJECT 021-1155-2		RECORD OF BOREHOLE No 02-509		1 OF 1	METRIC
W.P. 256-99-00		LOCATION N 5012289.8 E 346414.8		ORIGINATED BY D.B.	
DIST _____ HWY 7		BOREHOLE TYPE CME 55 Bombardier, 108mm I.D. Hollow Stem Auger		COMPILED BY M.I.C.	
DATUM Geodetic		DATE Dec. 13, 2002		CHECKED BY L.C.C.	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								20 40 60 80 100					W _p	W	W _L		
134.9	Ground Surface																
0.0	Silty Sand, some gravel and cobbles, trace wood and organics (Fill) Loose to compact Grey-brown Moist		1	SS	4		134										
			2	SS	4		133										
			3	SS	20		132										
131.9																	
3.1	Sand and Silt, some gravel, trace clay, with cobbles and boulders (Till) Very dense Grey Wet		4	SS	52												
			5	SS	100/0.10		131										
130.6																	
4.3	End of Borehole Refusal to auger penetration Note: Water noted during drilling at about 1.8m depth (Elev. 133.1m).																

PROJECT <u>021-1155-2</u>			RECORD OF BOREHOLE No 02-510			1 OF 1			METRIC							
W.P. <u>256-99-00</u>			LOCATION <u>N 5012287.1, E 346421.1</u>			ORIGINATED BY <u>D.B.</u>										
DIST <u> </u> HWY <u>7</u>			BOREHOLE TYPE <u>Portable rig, 108mm Dia Hollow Stem Auger</u>			COMPILED BY <u>M.I.C.</u>										
DATUM <u>Geodetic</u>			DATE <u>Dec. 13, 2002</u>			CHECKED BY <u>L.C.C.</u>										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)	
								20	40	60	80	100	PLASTIC LIMIT w_p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w_L	
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED								
								20	40	60	80	100	10	20	30	
132.0	Ground Surface															
0.0	Sand, trace organics (Alluvium)		1	SS	21											
131.7	Compact Grey															
0.3	Wet Sand and Silt, some gravel and cobbles (Till)															
131.0	Compact Grey															
1.0	Wet LIMESTONE (BEDROCK) with shale seams															
	Fresh															
	Weak to medium strong															
	Thinly-bedded															
	Grey															
	Bedrock cored between 1.0 m and 3.9 m depth															
	For bedrock coring details refer to Record of Drillhole 02-510															
128.1	End of Borehole															
3.9	Notes:															
	1. Borehole advanced using portable drilling equipment with a half-weight hammer. The SPT "N" values have been adjusted on these logs to reflect the values that would be obtained using a standard - weight hammer.															
	2. Water level in ditch at 0.2 m above ground surface (Elev. 132.2m) during drilling.															

MISS_MTO 021-1155-2.GPJ ON MOT.GDT 5/9/03

PROJECT: 021-1155-2

RECORD OF DRILLHOLE: 02-510

SHEET 1 OF 1

LOCATION: N 5012287.1 ; E 346421.1

DRILLING DATE: Dec. 13, 2002

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: Portable rig, 108mm Dia Hollow Stem Auger

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH % RETURN	FR/FX-FRACTURE-FAULT				SM-SMOOTH		FL-FLEXURED		BC-BROKEN CORE		DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
								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					
RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY													
TOTAL CORE %	SOLID CORE %			TYPE AND SURFACE DESCRIPTION		K, cm/sec													
		Refer to previous page		131.00															
		LIMESTONE (BEDROCK) with shale seams		1.02															
2	Rotary Drill BQ Core	Fresh Weak to medium strong Thinly - bedded Grey			1		100												
3					2		100												
4					3		100												
		End of Borehole		128.16															
5				3.66															
6																			
7																			
8																			
9																			
10																			
11																			

DEPTH SCALE



1 : 50



LOGGED: P.A.H.

CHECKED: M.I.C.

MISS ROCK ROCK1155-2.GPJ GLDR CAN.GDT 5/9/03 JFC

PROJECT 021-1155-2		RECORD OF BOREHOLE No 02-511		1 OF 1		METRIC															
W.P. 256-99-00		LOCATION N 5012305.5, E 346426.9		ORIGINATED BY J.S.																	
DIST _____ HWY 7		BOREHOLE TYPE CME 55 with 108mm I.D. Hollow Stem Auger		COMPILED BY J.D.R.																	
DATUM Geodetic		DATE Jun. 19, 2003		CHECKED BY L.C.C.																	
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X REMOULDED					WATER CONTENT (%) W _p — W — W _L			γ			GR SA SI CL		
134.5 0.0	Ground Surface Sand and gravel (FILL) Compact Brown Moist						134														
133.4 1.1	Sand and gravel, some silt, trace organics, occasional cobbles (FILL) Loose to compact Brown Moist		1	SS	16		133														
			2	SS	12																
131.9 2.6	Topsoil		3	SS	7		132														
131.6 2.9	Sand and Silt, some gravel, trace clay (TILL) Very dense Grey Wet		4	SS	73		131														
130.6 3.9	LIMESTONE (BEDROCK) with shale seams Fresh Medium strong Very thinly to thinly bedded Grey Borehole corred between 3.9m and 7.3m depth. For details, refer to Record of Drillhole 02-511.		5	SS	50/0.08		130														
						129															
						128															
127.2 7.3	End of Borehole Notes: 1. Water encountered at about 3.0 m below ground surface (Elev. 131.5m) during overburden drilling.																				

MISS_MTO 021-1155-2.GPJ ON MOT.GDT 5/9/03

PROJECT: 021-1155-2

RECORD OF DRILLHOLE: 02-511

SHEET 2 OF 2

LOCATION: N 5012305.5 ; E 346426.9

DRILLING DATE: Jun. 19, 2003

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55 with 108mm I.D. Hollow Stem Auger

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	COLOUR FLUSH % RETURN	FR/FX-FRACTURE-F-FAULT			SM-SMOOTH			FL-FLEXURED			BC-BROKEN CORE			DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
								CL-CLEAVAGE			J-JOINT			UE-UNEVEN			MB-MECH. BREAK				
								SH-SHEAR			P-POLISHED			W-WAVY			B-BEDDING				
								VN-VEIN			S-SLICKENSIDED			PL-PLANAR			C-CURVED				
RECOVERY		R.Q.D. %		FRACT. INDEX PER 0.3		DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K, cm/sec		TYPE AND SURFACE DESCRIPTION											
TOTAL CORE %	SOLID CORE %																				
100	100	100	100	100	100	100	100	100	100	100	100	100	100								
10 ⁻⁶	10 ⁻⁵	10 ⁻⁴	10 ⁻³	2	4	6	8														
4		Refer to previous page		130.60																	
4		LIMESTONE (BEDROCK) with shale seams Fresh Medium strong Very thinly to thinly bedded Grey		3.90			100														
5							100														
6							100														
7							100														
7		End of Borehole		127.22																	
8				7.28																	
9																					
10																					
11																					
12																					
13																					

MISS. ROCK ROCK1155-2.GPJ GLDR CAN.GDT 5/9/03 JFC

DEPTH SCALE

1 : 50



LOGGED: J.S.

CHECKED: L.C.C.

PROJECT 021-1155-2

RECORD OF BOREHOLE No 02-512

1 OF 1

METRIC

W.P. 256-99-00

LOCATION N 5012284.5, E 346427.7

ORIGINATED BY J.S.

DIST HWY 7

BOREHOLE TYPE CME 55 with 108mm I.D. Hollow Stem Auger

COMPILED BY J.D.R.

DATUM Geodetic

DATE Jun. 20, 2003

CHECKED BY L.C.C.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								20 40 60 80 100										
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x REMOULDED										
133.5	Ground Surface																	
0.0	Topsoil																	
133.1																		
0.4	Sand Compact Brown						133											
132.7	Moist																	
0.8	Sand and Gravel																	
132.3	Compact Brown		1	SS	26													
1.2	Wet																	
	Sand and Silt, trace organic, trace clay (TILL) Very dense Grey Wet		2	SS	56		132											
131.0			3	SS	50/0.06		131											
2.5	LIMESTONE (BEDROCK) with shale seams																	
	Fresh Medium strong Thinly bedded Grey																	
	Borehole corrd between 2.5m and 6.0m depth. For details, refer to Record of Drillhole 02-512.						130											
							129											
							128											
127.5																		
6.0	End of Borehole																	
	Notes: 1. Water encountered at about 1.1 m below ground surface (Elev. 132.4m) during overburden drilling.																	

MISS_MTO 021-1155-2.GPJ ON MOT GDT 5/9/03

PROJECT <u>021-1155-2</u>		RECORD OF BOREHOLE No 02-520		1 OF 1	METRIC
W.P. <u>256-99-00</u>		LOCATION <u>N 5012296.5 ; E 346443.6</u>		ORIGINATED BY <u>D.B.</u>	
DIST <u> </u> HWY <u>7</u>		BOREHOLE TYPE <u>Electric core drill, 50mm Dia. Sampler</u>		COMPILED BY <u>M.I.C.</u>	
DATUM <u>Geodetic</u>		DATE <u>Jan. 8, 2003</u>		CHECKED BY <u>L.C.C.</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	W _p	W	W _L		GR	SA	SI	CL
133.5	Ground Surface																			
0.0	Topsoil and Peat																			
133.2																				
0.3	Sand, trace silt Brown Moist																			
132.7			1	SS	-															
0.9	Sand and silt, some gravel (Till)																			
132.4	Grey-brown																			
1.1	End of Borehole Refusal to Pionjar Penetration Note: Borehole dry on completion of drilling.																			

MISS_MTO 021-1155-2.GPJ ON_MOT.GDT 5/9/03

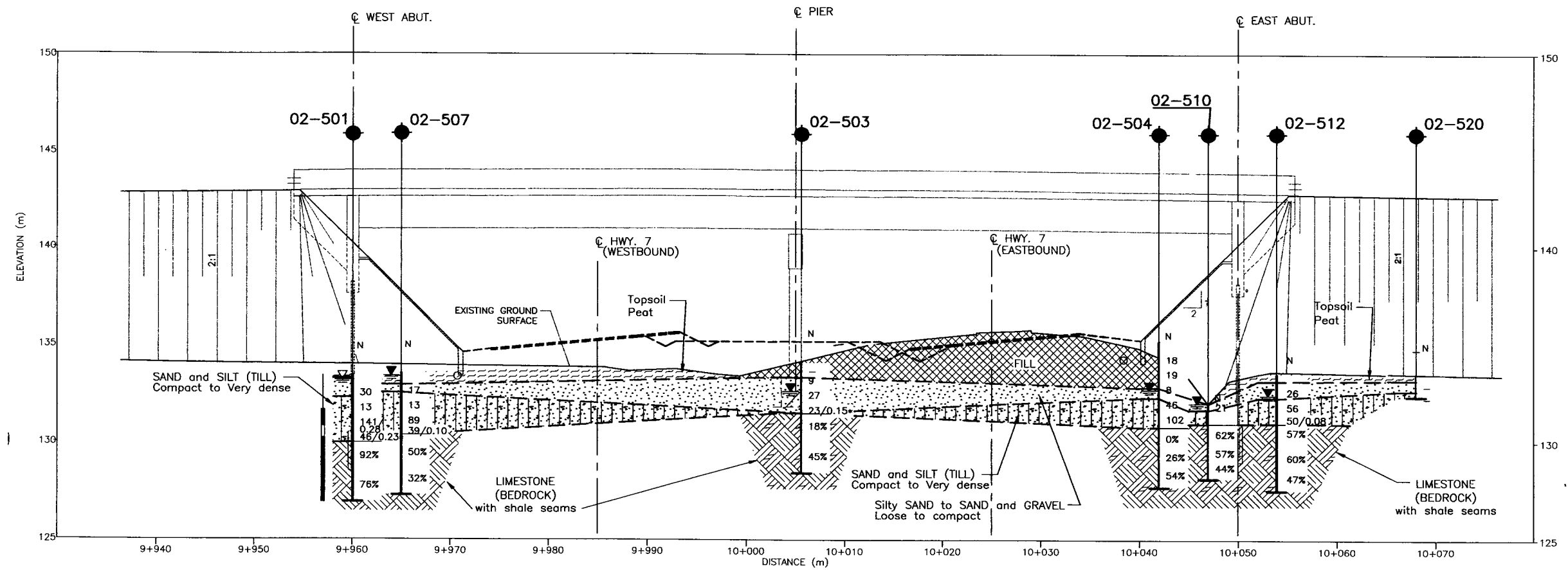
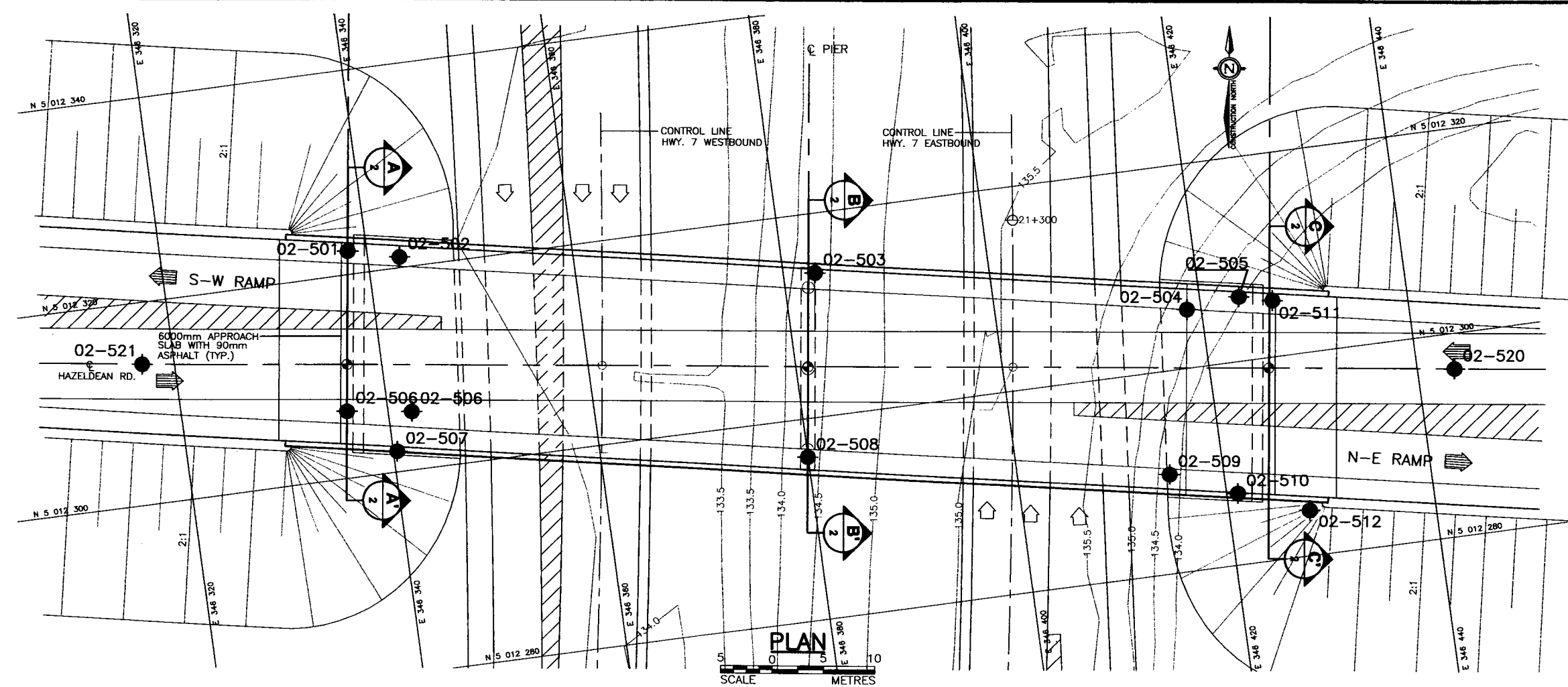
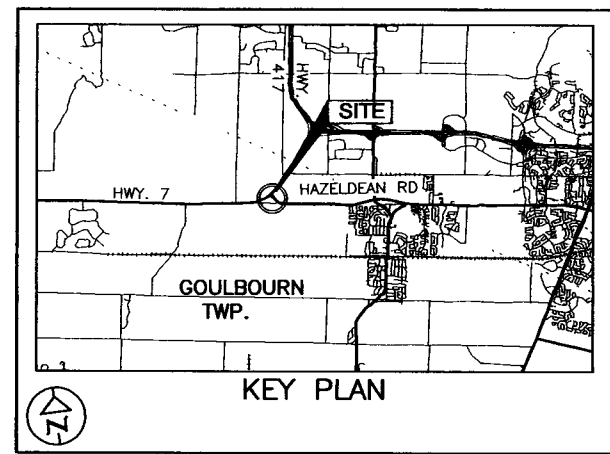
PROJECT 021-1155-2			RECORD OF BOREHOLE No 02-521			1 OF 1			METRIC										
W.P. 256-99-00			LOCATION N 5012313.9 E 346316.8			ORIGINATED BY D.B.													
DIST _____ HWY 7			BOREHOLE TYPE Electric core drill, with Half-Weight Sampler Hammer			COMPILED BY M.I.C.													
DATUM Geodetic			DATE Jan. 8, 2003			CHECKED BY L.C.C.													
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X REMOULDED					WATER CONTENT (%) W _p — W — W _L			UNIT WEIGHT γ	GR SA SI CL		
133.6 0.0	Ground Surface Peat							20 40 60 80 100											
133.1 0.5	Sand, trace silt Loose to compact Brown Wet		1	SS	8		133												
132.0 1.6	Sand and Silt, some gravel, trace clay (Till) Dense to very dense Grey Wet		2	SS	15		132												
			3	SS	48														
130.9 2.7	End of Borehole Refusal to Sampler Penetration Notes: 1. Borehole advanced using portable drilling equipment with a half-weight hammer. The SPT "N" values have been adjusted on these logs to reflect the values that would be obtained using a standard - weight hammer. 2. Water level at ground surface on completion of drilling.		4	SS	90		131												

MISS MTO 021-1155-2.GPJ ON MOT.GDT 5/9/03



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN



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, measured on Jan. 08, 2003
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
02-501	133.4	5 012 322.3	346 338.2
02-502	133.6	5 012 321.3	346 343.0
02-503	134.1	5 012 314.3	346 382.4
02-504	135.2	5 012 305.7	346 418.6
02-505	134.9	5 012 306.3	346 423.7
02-506	133.6	5 012 306.6	346 336.0
02-507	133.5	5 012 302.0	346 340.4
02-508	134.2	5 012 296.2	346 380.0
02-509	134.9	5 012 289.8	346 414.8
02-510	132.0	5 012 287.1	346 421.1
02-511	134.5	5 012 305.5	346 426.9
02-512	133.5	5 012 284.5	346 427.7
02-520	133.5	5 012 296.5	346 443.6
02-521	133.6	5 012 313.9	346 316.8

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

REFERENCE
General arrangement drawing provided in digital format (file: GA.DWG) by Marshall Macklin Monaghan, on April 24, 2003

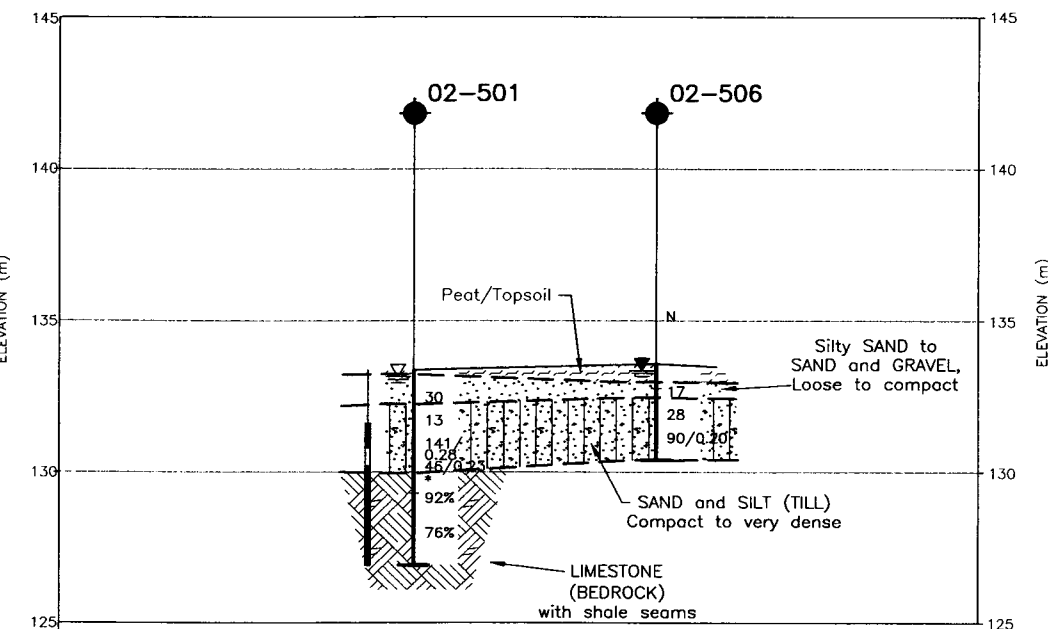
NO.	DATE	BY	REVISION

Geocres No. _____ PROJECT NO. 021-1155-2 DIST. 42

HWY. 7	CHKD. LCC	DATE: SEPT. 2003	SITE: 3-721
DRAWN: JDR	CHKD. LCC	APPD. ASP	DWG. 1

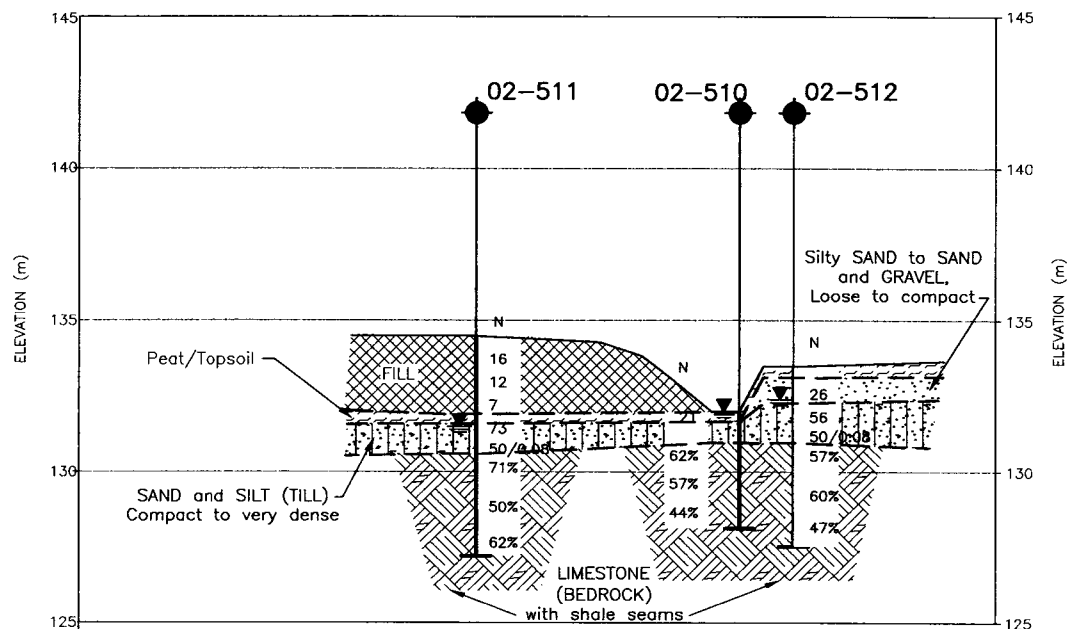
PROFILE

HORZ. SCALE 5 0 5 10
VERT. SCALE 2.5 0 2.5 5



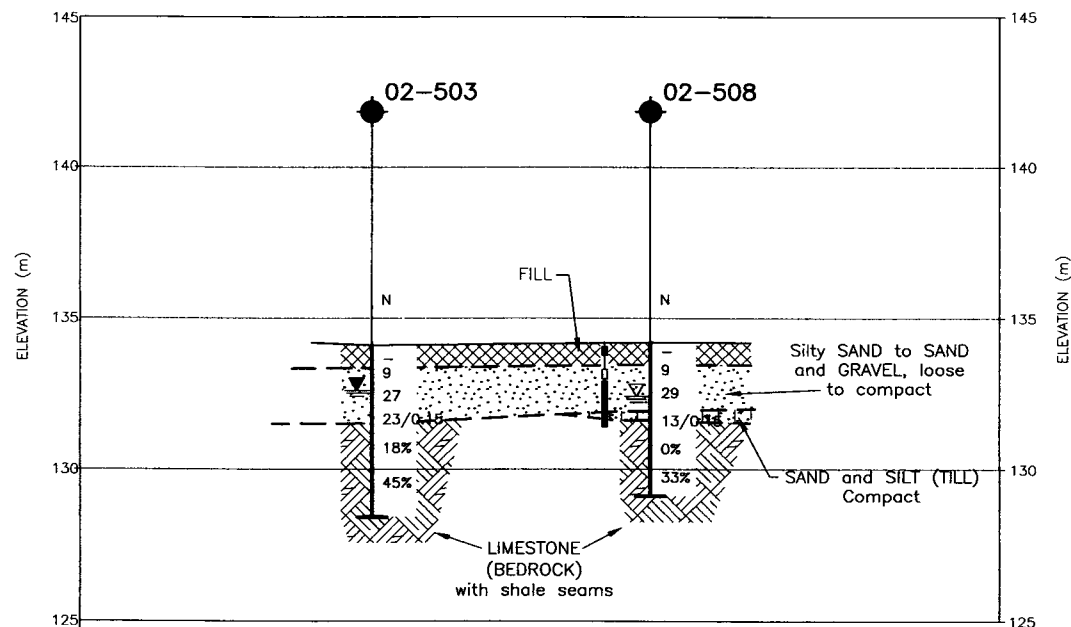
A-A'
1 WEST ABUTMENT SECTION

HORZ. SCALE 5 0 5 10
VERT. SCALE 2.5 0 2.5 5



C-C'
1 EAST ABUTMENT SECTION

HORZ. SCALE 5 0 5 10
VERT. SCALE 2.5 0 2.5 5



B-B'
1 CENTRE PIER SECTION

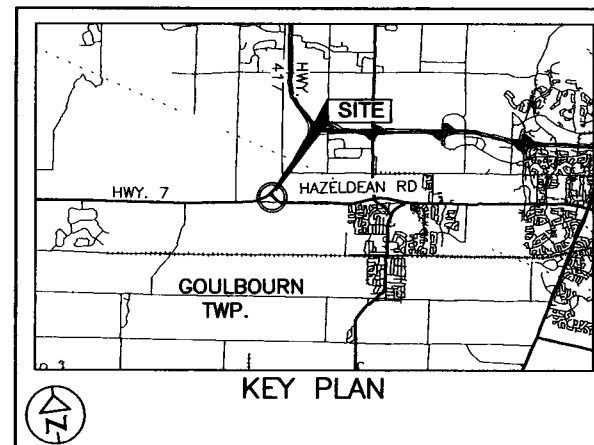
HORZ. SCALE 5 0 5 10
VERT. SCALE 2.5 0 2.5 5

DIST. 42	HWY. 7
CONT No.	
WP No.142-78-00	
HAZELDEAN ROAD UNDERPASS	SHEET
SOIL STRATA	



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METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN



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, measured on Jan. 08, 2003
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
02-501	133.4	5 012 322.3	346 338.2
02-502	133.6	5 012 321.3	346 343.0
02-503	134.1	5 012 314.3	346 382.4
02-504	135.2	5 012 305.7	346 418.6
02-505	134.9	5 012 306.3	346 423.7
02-506	133.6	5 012 306.6	346 336.0
02-507	133.5	5 012 302.0	346 340.4
02-508	134.2	5 012 296.2	346 380.0
02-509	134.9	5 012 289.8	346 414.8
02-510	132.0	5 012 287.1	346 421.1
02-511	134.5	5 012 305.5	346 426.9
02-512	133.5	5 012 284.5	346 427.7
02-520	133.5	5 012 296.5	346 443.6
02-521	133.6	5 012 313.9	346 316.8

NOTES

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

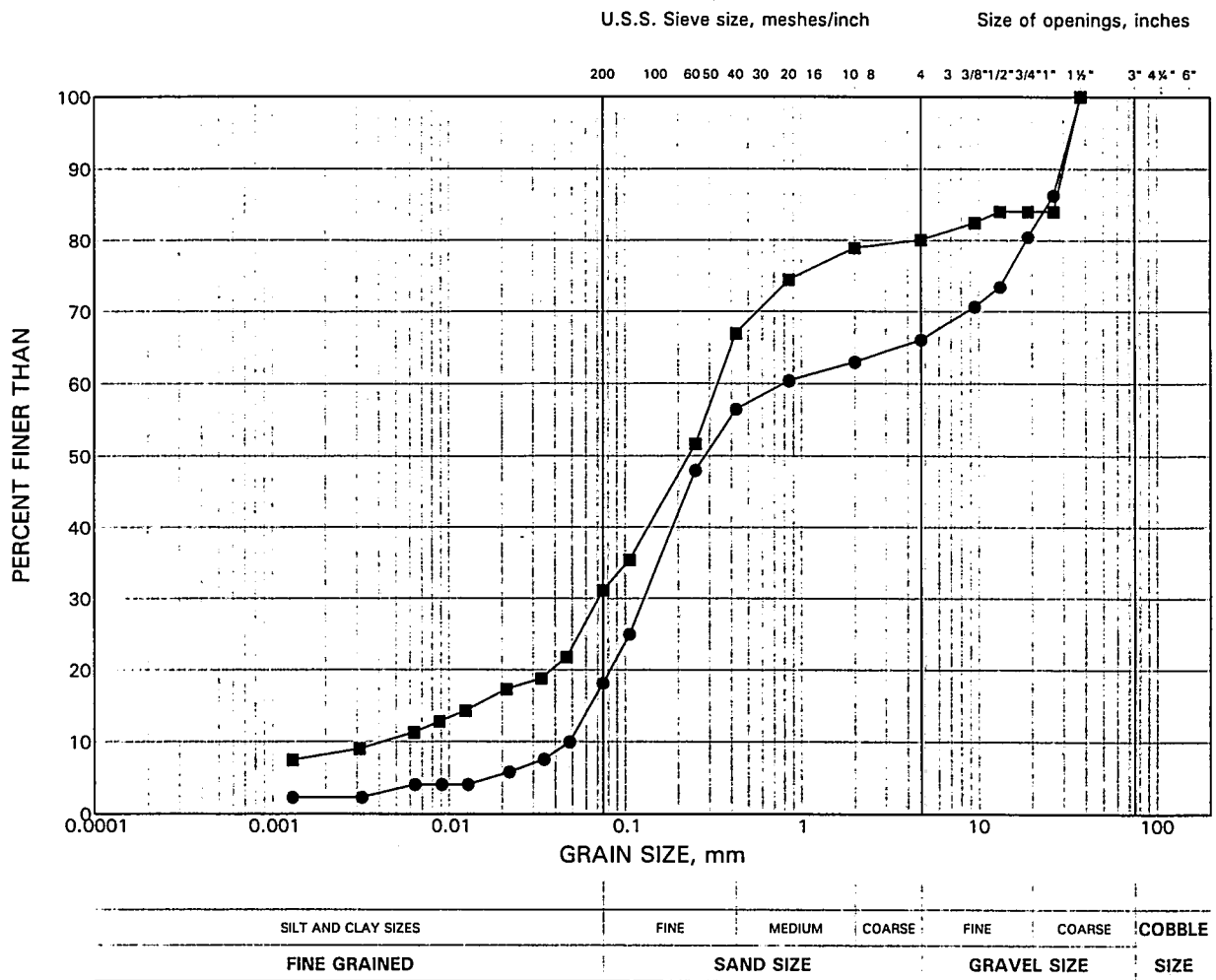
NO.	DATE	BY	REVISION

Geocres No.	PROJECT NO. 021-1155-2	DIST. 42
HWY. 7		
SUBM'D. LCC	CHKD. LCC	DATE: SEP 2003
DRAWN: JDR	CHKD. LCC	APPD. ASP
		SITE: 3-721
		DWG. 2

GRAIN SIZE DISTRIBUTION TEST RESULTS

Road Embankment Fill

FIGURE 1



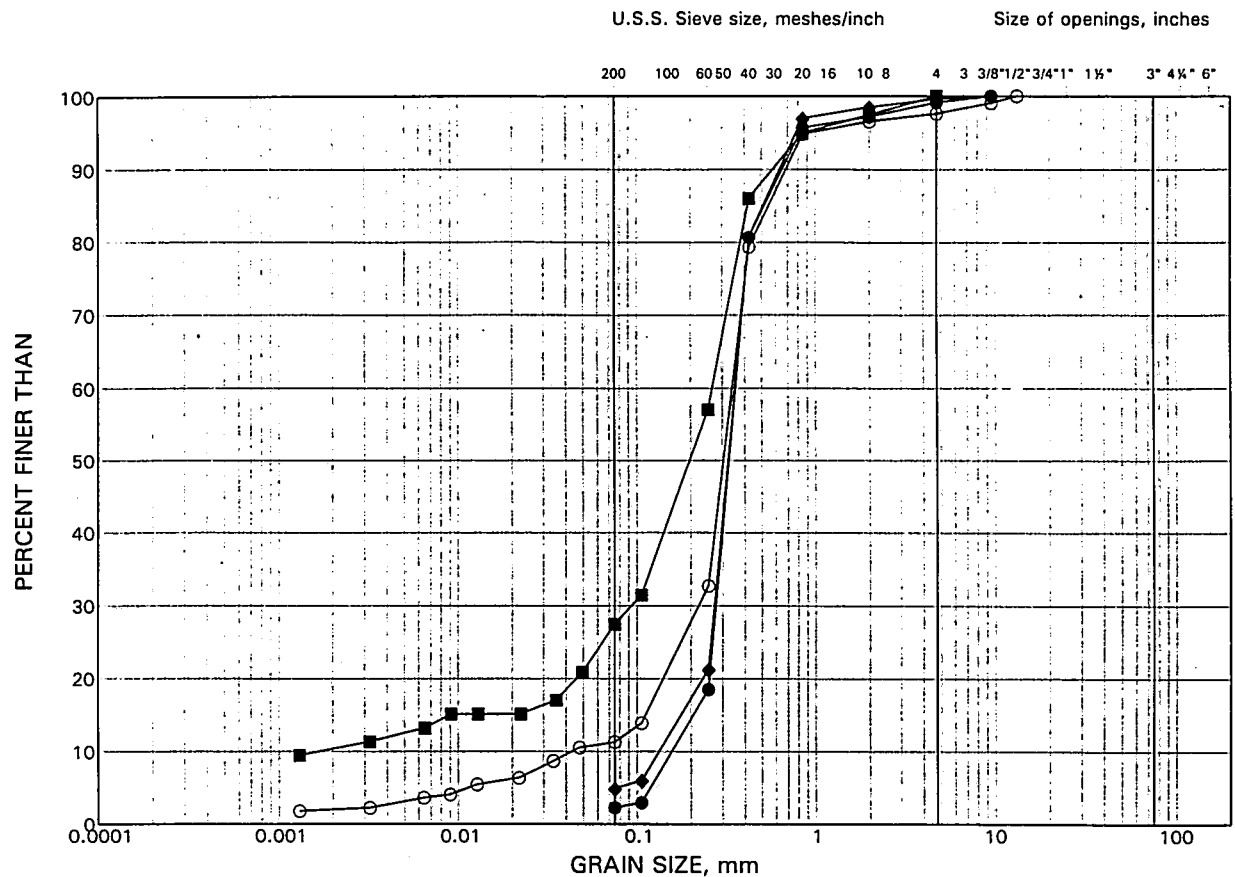
LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	02-504	1	134.1
■	02-505	2	133.2

GRAIN SIZE DISTRIBUTION TEST RESULTS

Surficial Sand to Silty Sand

FIGURE 2



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

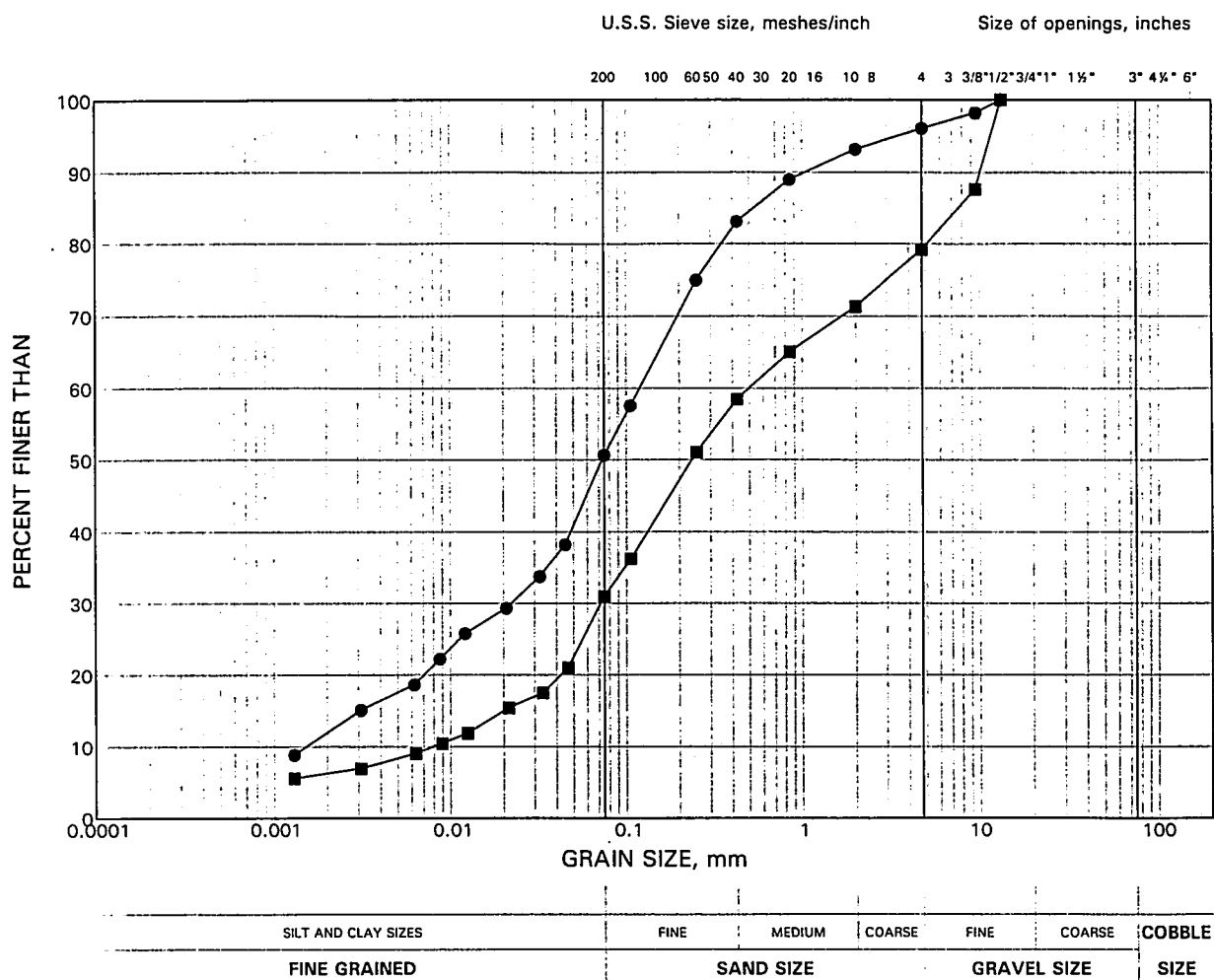
LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	02-502	1	132.5
■	02-505	3B	132.1
◆	02-507	1	132.5
○	02-508	3	132.4

GRAIN SIZE DISTRIBUTION TEST RESULTS

Silty Sand Till to Sand and Silt Till

FIGURE 3



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	02-504	5	131.2
■	02-506	2	131.7