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FOUNDATION
INVESTIGATION AND DESIGN REPORT
DWYER HILL ROAD UNDERPASS
STRUCTURE SITE 3-720
HIGHWAY 7 TWINNING FROM CARLETON PLACE TO 3 KM WEST OF JINKINSON ROAD
G.W.P. 251-99-00

Submitted to:

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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, plus 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 Sideroad, 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 proposed Dwyer Hill Road underpass structure.

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 abutment at the Hazeldean Road site, are outlined in Golder Associates' letter dated May 7, 2003.

2.0 SITE DESCRIPTION

The existing Dwyer Hill Road (Regional Road 3) – Highway 7 intersection is located approximately 8 km southwest of Highway 417, in West Carleton Township in the Regional Municipality of Ottawa-Carleton. The proposed underpass structure at this site is designated as MTO's Structure Site 3-720.

West Carleton Township is generally flat-lying; however, the topography at the Highway 7 – Dwyer Hill Road site is dominated by a southwest-northeast trending "ridge" that passes through the southwest, southeast and northeast quadrants of the intersection. The ground surface along the ridge is at about Elevation 139 m, some 5 m to 6.5 m higher than the general ground surface in the vicinity of the site, which varies from about Elevation 132.5 m to 134 m. The existing Highway 7 – Dwyer Hill Road intersection has been constructed in a cut through the ridge that is up to about 6 m deep. Outside of the ridge in the northwest quadrant, a commuter parking lot is present; it appears that some fill has been placed in the western portion of this parking lot to raise the grade to about Elevation 134 m.

To the west of the Dwyer Hill Road site, Highway 7 passes through a large, low-lying swamp. A small swamp, approximately 15 m wide, is also present near the north end of the proposed bridge, extending from about 30 m west of the existing Dwyer Hill Road corridor eastward across the local road.

3.0 INVESTIGATION PROCEDURES

A total of fourteen boreholes and one probehole were advanced as part of the subsurface investigation program for the proposed Dwyer Hill Road underpass structure, at the locations shown on Drawing 1. Two boreholes were advanced at each proposed foundation element, and one borehole was advanced within the limits of the north and south approach embankments. Where preliminary field investigation results suggested that the bedrock surface elevation varied by more than 0.3 m, as was the case in the vicinity of the proposed south abutment, an additional borehole or probehole was advanced to determine the bedrock surface elevation at the mid-point of the proposed foundation element. In addition, a supplementary borehole (Borehole 02-701A) was advanced adjacent to one of the north abutment boreholes, to conduct additional in situ vane shear testing in the silty clay deposit encountered at the site.

The boreholes were advanced, using hollow stem augers, to auger and/or sampler refusal which occurred at depths between 3.5 m and 5.9 m below the existing ground surface. Samples of the overburden were obtained at 0.75 m depth intervals using 50 mm outside diameter split-spoon samplers, in accordance with the Standard Penetration Test (SPT) procedure. In situ vane shear testing was conducted in the encountered silty clay deposit, using "N"- and "B"-sized vanes. In six of the boreholes advanced at the proposed foundation locations, the boreholes were cored about 3 m into the bedrock using NQ-size coring equipment. The water level in the open boreholes was observed throughout the drilling operations, and six 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 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.

Borehole Number	*Borehole Location*	*MTM NAD83 Northing (m)*	*MTM NAD83 Easting (m)*	*Ground Surface Elevation (m)*
02-701	North abutment	5,007,182.3	340,944.0	133.0 m
02-701A	North abutment	5,007,183.1	340,944.7	133.0 m
02-702	North abutment	5,007,180.0	340,948.3	132.8 m
02-703	Centre pier	5,007,156.7	340,981.7	134.0 m
02-704	South abutment	5,007,134.0	341,015.3	132.9 m
02-705	South abutment	5,007,131.3	341,019.4	132.9 m
02-706	North abutment	5,007,166.1	340,929.2	132.4 m
02-707	North abutment	5,007,163.2	340,932.9	132.5 m
02-708	Centre pier	5,007,141.0	340,967.0	134.0 m
02-709	South abutment	5,007,119.2	341,001.3	132.8 m
02-710	South abutment	5,007,116.4	341,005.6	132.7 m
02-714	South abutment	5,007,127.4	341,009.0	132.8 m
02-715	South abutment	5,007,124.1	341,012.8	132.8 m
02-720	North approach	5,007,190.5	340,914.9	132.8 m
02-721	South approach	5,007,112.7	341,037.4	133.3 m

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

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

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

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

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

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

4.2 Site Stratigraphy

As part of the subsurface investigation at this site, fourteen boreholes and one probehole 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 5. 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 bridge site is covered by either fill, associated with the existing Highway 7 embankment and commuter parking lot, or topsoil, outside of the highway and parking lot. In the northern and west-central (i.e. western portion of the commuter parking lot) areas of the site, the fill and topsoil overlie glaciofluvial or glaciolacustrine deposits consisting of loose to compact silty sand to sand and gravel underlain by a firm to very stiff silty clay layer. In the southern and east-central (i.e. eastern portion of the commuter parking lot) areas of the site, where the existing Highway 7 and Dwyer Hill Road corridors have been cut into the "ridge" that runs southwest-to-northeast across the site, a dense to very dense sand and silt till (including granular interlayers) is present immediately below the topsoil. This till is also present below the glaciofluvial or glaciolacustrine deposits encountered elsewhere at the site. The overburden soils are underlain by interlayered limestone and dolomitic limestone bedrock that was encountered between 3.6 m and 5.8 m depth (at about Elevation 127.9 m to 129.3 m).

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

4.2.1 Asphalt and Fill

Grade fills and/or road base fills were encountered below the asphalt surface in three boreholes located within the existing commuter parking lot and the existing Highway 7 alignment. Fill was also encountered in one borehole (Borehole 02-702) located near the treed area in the vicinity of the proposed north abutment.

Within the parking lot, Boreholes 02-703 and 02-708 encountered about 60 mm of asphalt overlying about 250 mm of crushed gravel, in turn overlying about 2.1 m to 2.2 m of brown or grey, sandy silt to silty sand fill, containing some gravel and trace wood fragments and organics; a grain size distribution test result for one sample of this fill material is shown on Figure 1. The base of this fill was encountered between Elevations 131.5 m and 131.7 m in the two boreholes. The Standard Penetration Test (SPT) "N" values measured within the sandy silt to silty sand fills ranged from 5 to 33 blows per 0.3 m of penetration, indicating that the fill was very loose to dense; on average, this fill was compact to dense.

Within the existing Highway 7 area (i.e. in the vicinity of the south approach embankment), Borehole 02-721 encountered approximately 150 mm of asphalt overlying about 0.8 m of granular road base and sub-base courses, in turn underlain by about 0.3 m of sandy silt fill. The base of this fill was encountered at about Elevation 132.1 m (approximately 1.2 m depth). The measured SPT "N" value within the sandy silt fill was 3 blows per 0.3 m of penetration, indicating that the material encountered in Borehole 02-721 has a very loose relative density.

Some fill was also encountered in Borehole 02-702, at the location of the proposed north abutment. At this location, the fill was approximately 0.8 m thick, with its base at about Elevation 132.0 m. The fill consists of sandy silt containing some gravel, and trace wood fragments and organics. An SPT "N" value of 4 blows per 0.3 m of penetration, measured partly within this fill, suggest that it has a loose state of packing.

4.2.2 Topsoil

Topsoil was encountered at ground surface in all of the boreholes located outside of the existing Highway 7 and commuter parking lot areas. The topsoil ranges in thickness from approximately 100 mm to 300 mm.

4.2.3 Silty Sand to Sand and Gravel

In the north and west-central portions of the site, in the vicinity of the proposed north abutment / approach and at the west end of the center pier, Boreholes 02-701, 702, 706 to 708 and 720 encountered a deposit ranging in composition from silty sand containing trace to some gravel, to sand containing trace to some silt and gravel, to sand and gravel containing trace silt. The results from six grain size distribution tests are shown on Figure 2. This deposit is interpreted to be of glaciofluvial or glaciolacustrine origin, in contrast to the southwest-northeast trending till ridge that is located to the south and east at this site. The silty sand to sand and gravel deposit is about 0.7 m to 3.0 m thick, with its base between Elevations 129.4 m and 130.9 m, as encountered in the boreholes.

The measured SPT "N" values within the silty sand to sand portions of the deposit range from 2 to 26 blows, but are typically between 2 and 10 blows per 0.3 m of penetration, indicating that these soils are generally of very loose to loose relative density. The measured SPT "N" value within the sand and gravel layer, encountered in one borehole, was 29 blows per 0.3 m of penetration, indicative of a compact relatively density.

4.2.4 Silty Clay

The silty sand to sand and gravel deposit, where encountered in the area of the north approach, north abutment and west end of the central pier, is underlain by a silty clay deposit. The deposit varies in thickness from about 0.2 m to 1.6 m, with the base of the deposit between approximately Elevations 128.8 m and 130.3 m.

The silty clay deposit contains sand seams, with trace quantities of gravel observed near the base; the result from one grain size distribution test is shown on Figure 3. The upper portion of the deposit is generally brown, while the lower portion is grey; the upper, brown portion of the silty clay is considered to represent a "weathered crust". Atterberg limits testing conducted on four samples measured plastic limits of 18 to 20 per cent, liquid limits of 35 to 44 per cent, and plasticity indices of 17 to 25 per cent; the results of this testing are presented on the borehole records and on Figure 4. Measured natural water contents within the upper, brown silty clay ranged from 26 to 36 per cent (i.e. several per cent lower than the corresponding liquid limit), while the measured water content for one sample of the lower grey silty clay was 45 per cent, slightly above the corresponding liquid limit of 44 per cent.

The measured SPT "N" values range from 1 to 6 blows per 0.3 m of penetration. In situ field vane testing in the grey silty clay measured undrained shear strengths ranging from 50 kPa to 80 kPa, and a sensitivity of about 6 to 16; the silty clay is therefore considered sensitive. Based on the results of the Standard Penetration testing, in situ vane shear strength testing and local experience, the upper brown portion of the deposit is generally stiff to very stiff, while the lower portion of the deposit is firm to stiff.

4.2.5 Sand and Silt Till (Including Granular Interlayers)

In the northern and west-central portions of the site, the surficial silty sand to sand and gravel and silty clay deposits are underlain by a deposit of sand and silt till, which in turn immediately overlies bedrock. In the east-central and southern portions of the site, the till deposit immediately underlies the existing fill or topsoil, extending down to bedrock. In three of the six boreholes in the southern portion of the site, interlayers consisting of sand and gravel to sandy silt were encountered within the till. Grain size distribution test results for one sample of the sand and silt till and two samples of the sand and gravel to sandy silt interlayer are presented on Figure 5.

The till deposit (including granular interlayers) as encountered in the east-central and southern areas of the site is associated with the southwest-northeast trending ridge through which the existing Highway 7 and Dwyer Hill Road have been cut. In this area, the till deposit and its granular interlayers have a total thickness that ranges from about 3.2 m to 4 m, extending from just below the topsoil or fill to the bedrock surface. This till and the interlayers are typically dense to very dense, with the majority of the measured SPT "N" values greater than 40 blows and an average SPT "N" value of about 75 blows per 0.3 m of penetration. The upper 0.5 m to 1 m of till in some of the boreholes was loose to compact, based on SPT "N" values of 6 to 29 blows per 0.3 m of penetration. A probable boulder was encountered within the till deposit in Borehole 02-704, based on the presence of a till seam in the core sample retrieved.

In boreholes located in the west-central and north portions of the site, the sand and silt till deposit underlies the surficial silty sand to sand and gravel and silty clay strata, and extends down to bedrock. The till at these locations varies from about 0.7 m to 2.2 m in thickness, with its surface encountered between Elevations 128.8 m and 130.3 m. The till in this portion of the site is less dense than that which comprises the "ridge". In Boreholes 02-706 and 02-707, where the till deposit thickens and its surface is between Elevations 129.6 m and 130.3 m, the measured SPT "N" values range from 6 to 28 blows per 0.3 m of penetration, indicative of a loose to compact stratum. In Boreholes 02-701, 02-702 and 02-708, where the till deposit thins and its surface is between Elevations 128.8 m and 129.3 m, the till is very loose based on measured SPT "N" values ranging from 2 to 4 blows per 0.3 m of penetration.

4.2.6 Interlayered Limestone and Dolomitic Limestone Bedrock

Interlayered limestone and dolomitic limestone bedrock underlies the overburden deposits at this site at a depth of 3.6 m to 5.8 m below the existing ground surface. The bedrock surface varies from approximately Elevation 127.9 m to 129.1 m across the site, generally rising toward the south. The following table summarizes the bedrock surface depth and elevation encountered at the borehole locations. It should be noted that bedrock was cored in six of the boreholes; the surface of the limestone bedrock was inferred in the seven remaining boreholes and the probehole by refusal to split-spoon sampler and/or auger advance.

Borehole Location	*Borehole Number*	*Ground Surface Elevation*	*Depth to Bedrock*	*Bedrock Surface Elevation*
North approach	02-720	132.8 m	4.7 m	128.1 m
North abutment	02-701	133.0 m	5.0 m	128.0 m (Cored)
	02-702	132.8 m	4.9 m	127.9 m
	02-706	132.4 m	4.4 m	128.0 m
	02-707	132.5 m	4.5 m	128.1 m (Cored)
Centre pier	02-703	134.0 m	5.5 m	128.5 m (Cored)
	02-708	134.0 m	5.8 m	128.2 m (Cored)

Borehole Location	*Borehole Number*	*Ground Surface Elevation*	*Depth to Bedrock*	*Bedrock Surface Elevation*
South abutment	02-704	132.9 m	4.1 m	128.8 m (Cored)
	02-705	132.9 m	4.1 m	128.8 m
	02-709	132.8 m	3.8 m	129.0 m
	02-710	132.7 m	3.6 m	129.1 m (Cored)
	02-714	132.8 m	3.9 m	128.9 m
	02-715	132.8 m	3.9 m	128.9 m
South approach	02-721	133.3 m	4.5 m	128.9 m

The interlayered limestone and dolomitic limestone bedrock at the site is a member of the Ottawa Formation. The bedrock is grey, slightly weathered to fresh, weak to medium strong, and thinly to thickly bedded. The Rock Quality Designation (RQD) values measured on recovered bedrock core samples ranged from 61 to 100 per cent, but were typically greater than 85 per cent, indicating that the rock is generally of good to excellent quality.

4.3 Groundwater Conditions

Six piezometers were installed within the boreholes advanced at this site. The water levels measured in the piezometers on June 6, 2003 varied from Elevation 131.7 m to 132.4 m, as summarized in the following table.

Location	*Borehole No.*	*Water Level on June 6, 2003*	
		Depth	*Elevation*
North Approach	02-720	1.0 m	131.8 m
North Abutment	02-701	1.1 m	131.9 m
	02-707	0.7 m	131.8 m
Centre Pier	02-708	2.3 m	131.7 m
South Abutment	02-704	0.5 m	132.4 m
	02-710	0.4 m	132.3 m

In the four piezometers in the northern and central areas of the site, the water level was measured between Elevations 131.7 m and 131.9 m, about 0.7 m to 1.1 m below the natural ground surface and approximately 2.3 m below the existing commuter parking lot grade. In the southern portion of the site, closest to the till ridge, the water level is about 0.5 m to 0.8 m higher at Elevation 132.3 m to 132.4 m, or about 0.4 m to 0.5 m depth.

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
DWYER HILL ROAD UNDERPASS
STRUCTURE SITE 3-720
HIGHWAY 7 TWINNING FROM CARLETON PLACE
TO 3 KM WEST OF JINKINSON ROAD
G.W.P. 251-99-00

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides foundation design recommendations for the proposed Dwyer Hill 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 Dwyer Hill Road underpass structure will be two spans, with each span about 40 m to 45 m in length. Based on the information contained in Totten Sims Hubicki's *Update to the Preliminary Design Study (Highway Engineering),* dated June 2002, the future Highway 7 grade will be at about Elevation 133 m to 133.5 m, and the proposed Dwyer Hill Road grade at the structure site is about Elevation 141.8 m. The natural ground surface at the site varies from about Elevation 132.7 m in the northwest quadrant of the existing Highway 7 – Dwyer Hill Road intersection, to about Elevation 139 m on top of the till ridge. The immediate approach embankments will be approximately 9 m high relative to the surrounding natural grade.

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 general, dense to very dense sand and silt till soils were encountered at the proposed south approach and abutment, and at the east end of the proposed centre pier; these dense soils are associated with the till "ridge" that traverses the site from the southwest to the northeast. To the north and west of the ridge, at the west end of the proposed centre pier and at the north abutment and approach, generally loose silty sand to sand and gravel soils and variable thicknesses of relatively soft silty clay soils were encountered. Bedrock is present at approximately 3.6 m to 5.8 m depth, or about Elevation 127.9 m to 129.1 m. The groundwater level at the site varies from about Elevation 131.7 m to 132.4 m, declining to the north away from the till ridge.

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.

Deep foundations, such as driven steel H-piles in a conventional or integral abutment configuration, or caissons supported on or socketted into the limestone/dolomitic limestone bedrock, are considered to be the most appropriate foundation type for this site. Given the proposed Dwyer Hill Road grade of about Elevation 141.8 m, an assumed underside of pile cap at about Elevation 138.8 m, and the bedrock surface at the abutments locations at about Elevation 127.9 m to 129.1 m, it is estimated that the pile length will be approximately 9.5 m to 11 m; this satisfies the minimum pile length of 5 m required to impart sufficient flexibility of the piles to accommodate bridge deck deflections for an integral abutment structure, if adopted for the site.

The dense to very dense sand and silt till soils in the southern portion of the site are suitable for the support of the south abutment and any associated concrete retaining walls on spread footings. The soils at this location are also suitable for support of RSS walls, either as wingwalls or in front of the abutments. At the centre pier and north abutment locations, spread footings supported on the soil are not a feasible foundation option for the foundation elements or any associated retaining walls (including RSS walls), due to the generally loose nature of the granular soils, the presence of the compressible silty clay, and the high groundwater level. At the centre pier, in particular, the differing conditions encountered in Boreholes 02-703 and 02-708 would result in significant differential settlement for spread footings supported on the soil. For the same reason, spread footings supported on a compacted granular pad within the approach embankment fill are also unsuitable for the north abutment, since the embankment loading would induce consolidation of the silty clay stratum. The abutment footing settlement could be minimized by preloading the area; however, surcharging would probably be required to effect the consolidation given the depth of the compressible stratum. Since the pier would have to be supported on the bedrock, there is still potential for differential settlement between the pier and each abutment.

At the centre pier, as an alternative to "deep" foundations, consideration could be given to the use of a spread footing founded on the surface of the bedrock, which was encountered in the boreholes between Elevation 128.2 m and 128.5 m, or on a compacted Granular "A" pad following excavation to the bedrock surface. Excavations down to bedrock would extend to about 6 m depth, and groundwater control would be required.

Recommendations for spread footings, steel H-piles and caisson foundations for the pier, bridge abutments and associated retaining walls, as considered applicable, are presented in the following sections.

5.3 Spread Footings

5.3.1 Geotechnical Resistance for Spread Footings at South Abutment

The south bridge abutment and any associated concrete cantilever wing walls / retaining walls may be supported on spread footings placed on the properly prepared sand and silt till soils at or below Elevation 132 m, depending on final site grades and minimum frost protection requirements. Alternatively, spread footings for the south abutment may be placed on a compacted granular pad within the approach embankment fill.

For the south abutment and any associated wing walls / retaining walls, spread footings placed on the surface of the properly prepared sand and silt till, at or below Elevation 132 m, may be designed based on a factored geotechnical resistance at Ultimate Limit States (ULS) of 700 kPa. The geotechnical resistance at Serviceability States (SLS), for 25 mm of settlement, may be taken as 500 kPa. These geotechnical resistances assume a footing width of 3 m; the geotechnical resistances should be reviewed if there are significant changes in the foundation geometry.

For a spread footing placed within the south approach embankment on a compacted Granular "A" pad, 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 consistency and thickness of the underlying fill and loose/compact upper till materials; a value of 350 kPa may be assumed for design purposes. These values assume that the granular 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 Sections 6.7.2 and 6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC)* and its *Commentary*.

5.3.2 Geotechnical Resistance for Spread Footings at Centre Pier

Consideration could be given to supporting the centre pier on a spread footing founded on the limestone/dolomitic limestone bedrock. The surface of the bedrock was encountered in Boreholes 02-703 and 02-708 at Elevations 128.5 m and 128.2 m, respectively. Based on these borehole results, there is potential for some variability in the bedrock surface within the limits of the centre pier. Consideration could be given to a design founding level which involves partial bedrock excavation and partial mass concrete placement; however, given the magnitude of difference between the two boreholes, the following options for founding levels are put forward:

1. Founding level of Elevation 128.5 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 founding level. Provision should be made in the Contract Documents for such mass concrete placement to accommodate the variations in the bedrock surface. The benefit of this approach is that excavation into the medium strong bedrock is avoided.

2. Founding level of Elevation 128.2 m: In this case, excavation of the higher portions of the bedrock will be required within the foundation footprint. Based on the borehole results, subexcavation of up to about 0.3 m of bedrock will be required. It is noted that the bedrock is weak to medium 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 or line drilling and pre-shearing techniques.

At the centre pier, a spread footing placed on the surface of the properly prepared limestone/dolomitic limestone bedrock may be designed using a factored geotechnical resistance at ULS of 3,000 kPa. The geotechnical resistance at SLS for 25 mm of settlement will be greater than the factored resistance at ULS, since the limestone bedrock is considered to be an unyielding material; as such, ULS conditions will govern for this foundation type.

As an alternative, to minimize the column height and concrete quantities, the centre pier could be founded on a compacted Granular "A" pad following excavation down to bedrock. It is noted that this option is considered feasible only if the excavation is carried out with adequate groundwater control, and ensuring there is sufficient space available for the use of proper compaction equipment. Assuming a minimum cover of 1.8 m for frost protection purposes, the centre pier footing founded on a compacted Granular "A" pad may be designed using a factored geotechnical resistance at ULS of 900 kPa. The geotechnical resistance at SLS may be taken as 400 kPa.

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 Sections 6.7.2 and 6.7.4 of the *CHBDC* and its *Commentary*.

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 δ', between 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.

Subgrade	**Coefficient of Friction (tan δ')**
South abutment:	
Dense to very dense sand and silt till	0.5
Compacted Granular "A" pad	0.57
Centre pier:	
Compacted Granular "A" pad	0.57
Limestone / dolomitic limestone bedrock	0.7

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.

5.3.4 Frost Protection

A minimum of 1.8 m of soil cover, or equivalent, is required above the spread footings for frost protection purposes.

5.4 Steel H-Pile Foundations

Steel H-piles driven to found on the limestone/dolomitic limestone bedrock may be used for support of the abutments. It is assumed that the abutment pile caps will be "perched" within the approach embankment fill in order to minimize the abutment wall height. Based on the proposed Dwyer Hill Road grade at about Elevation 141.8 m and the assumed pile cap base at Elevation 138.8 m, the pile length will be approximately 9.5 to 11 m without socketting into bedrock. If necessary, for additional pile length and/or to resist seismic forces, the piles could be placed within the bedrock. The limestone/dolomitic limestone bedrock is weak to medium strong, however, and this would require socket formation using coring or churn drilling to advance the hole. If socketting is carried out, it is noted that the native soils at the site are cohesionless and water-bearing and, as such, will flow into the auger hole if left unsupported during coring / churn drilling. The use of a temporary liner or casing, possibly in conjunction with drilling mud, will be required in order to carry out such installations with minimal loss of ground.

Steel H-piles founded on the bedrock may also be used for the support of the centre pier. Based on the current site grade (i.e. the existing commuter parking lot grade) and the frost protection requirements, piles founded on the bedrock surface would be approximately 3.7 m to 4.1 m in length.

5.4.1 Axial Geotechnical Resistance

For HP 310 x 110 piles driven to found on or socketted nominally (less than 1 m) into the limestone/dolomitic limestone bedrock, a factored axial resistance at ULS of 2,000 kN may be assumed for design. In the case of driven H-piles founded on the bedrock, 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.

There will be negative skin friction induced on the north abutment piles as the silty clay soils around the piles consolidate under the embankment loading. The unfactored downdrag load acting on a single pile over the length of pile within the native soils is estimated to be 300 kN (Briaud and Tucker, 1994). There would also be downdrag load imparted on the length of pile within the embankment fill; however, it is assumed that liners will be used through the embankment fill to allow sufficient pile flexibility for integral abutment foundations; the use of liners will isolate the pile from the effects of downdrag in the embankment fill. For structural design of the piles, the factored downdrag load should be added to the factored permanent (dead) loads, in accordance with Section 6.8.4 of the *CHBDC* and its *Commentary*. In this regard, a load factor of 1.25 is applied to the unfactored downdrag load. Consideration could be given to preloading at the north abutment to avoid the downdrag load; however, given the high ULS capacity for the piloes, preloading may not be necessary.

Cobbles and boulders should be expected within the glacially-derived soils at the site, particularly at the south abutment and the eastern end of the centre pier where thicker till deposits are present. In addition, given the potential for relatively short and battered piles at the pier and the hardness of the bedrock, the piles should be equipped with suitable driving points to ensure adequate seating of the piles on the bedrock.

For this site, the piles will essentially be driven to practical refusal on the bedrock. The drawings should incorporate the appropriate note stating that the piles should be equipped with rock points and driven to bedrock. 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 is met 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 equations given below.

For cohesionless soils:

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

where

n_h is the constant of subgrade reaction
z is the depth (m)
B is the pile diameter (m)

For cohesive soils:

$$k_h = \frac{k_{s1}}{5B}$$

where

B is the pile diameter (m) and
k_{s1} is the constant of horizontal subgrade reaction, as given below

The following ranges for the value of n_h and k_{s1} may be assumed in the structural analysis:

Soil Unit	n_h	k_{s1}
Embankment fill, including backfill around piles and CSPs (assumed to be compacted granular fill), above Elevation 132.5 m (i.e. above groundwater level)	5 to 10 MPa/m	–
Backfill around piles and CSPs below Elevation 132.5 m (i.e. below groundwater level)	5 to 8 MPa/m	
South abutment:		
Sand and silt till above Elev. 132.5 m	5 to 7 MPa/m	–
Sand and silt till / interlayers below Elev. 132.5 m	9 to 12 MPa/m	–

Soil Unit	n_h	k_{s1}
Centre pier:		
Sandy silt fill above Elev. 131.5 m	2 to 7 MPa/m	–
Sand and silt till below Elev. 131.5 m (east half of pier)	9 to 12 MPa/m	–
Silty clay between Elev. 131.5 m and 129 m (west half of pier)	–	15 to 20 MPa/m
Sand and silt till below Elev. 129 m (west half of pier)	1 to 3 MPa/m	–
North abutment:		
Silty sand to sand above Elev. 132 m	2 to 5 MPa/m	–
Silty sand to sand and gravel between Elev. 132 m and 130 m	1 to 3 MPa/m	–
Silty clay between Elev. 130 m and 129 m (east half of pier)	–	15 to 20 MPa/m
Sand and silt till below Elev. 130 m (west half of pier) or below Elev. 129 m (east half of pier)	1 to 3 MPa/m	–

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

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

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 Caisson Foundations

Caissons founded on or socketted into the limestone/dolomitic limestone bedrock may be used for support of the abutments and any associated concrete wing walls / retaining walls. It is assumed that the abutment pile caps will be "perched" within the approach embankment fill in order to minimize the abutment wall height. Based on the proposed Dwyer Hill Road grade at Elevation 141.8 m and the assumed pile cap base at Elevation 138.8 m, the length of caissons used for abutment support will be approximately 9.5 m to 11 m.

The use of caissons founded on or socketted into the bedrock is also considered appropriate for support of the centre pier, and for any retaining walls or wing walls associated with the proposed abutments. For a proposed Highway 7 grade similar to current ground surface elevations, and assuming 1.8 m of soil cover over the base of the pile cap for frost protection purposes, caissons for the centre pier would be approximately 3.7 m to 4.1 m in length. Assuming retaining wall footings would be close to existing ground surface, caissons for retaining wall support would be about 2.5 m to 3 m long in the vicinity of the north abutment, and about 2 m to 2.5 m long in the vicinity of the south abutment.

It is noted that the native soils at the site are cohesionless and water-bearing; these soils will flow into the auger hole during drilled shaft installation if left unsupported. The use of a temporary liner or casing, possibly in conjunction with drilling mud, will be required in order to advance the caissons with minimal loss of ground.

As discussed in Section 5.4, the limestone/dolomitic limestone bedrock at the site is moderately strong. If socketting of the caissons into the bedrock is required, the sockets will have to be advanced by rock coring or churn drilling.

5.5.1 Axial Geotechnical Resistance

Caissons founded on the surface of the limestone/dolomitic 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 4 MPa should be used. Serviceability Limit States resistances do not apply to caissons founded on the bedrock, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS. This capacity will have to be reviewed once the caisson configuration is established so that group effects for axial loading may be taken into account.

5.5.2 Resistance to Lateral Loads

The resistance to lateral loading developed by the soils in front of the caissons, 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 anticipated consolidation settlement of the silty clay stratum due to the embankment loading varies from less than 50 mm up to about 150 mm depending on the thickness of the silty clay underlying the embankment and RSS wall. This range is essentially due to the variable thickness of the silty clay stratum, which ranges from about 0.2 m to 1.6 m in the boreholes advanced in the vicinity of the north abutment and approach embankment. Given this magnitude and variability of settlement, RSS walls are not a feasible option for retaining walls or wing walls associated with the north abutment, unless preloading, possibly in conjunction with surcharging, is carried out prior to construction of the walls.

The use of RSS walls is considered appropriate for retaining walls or wing walls associated with the proposed south abutment. Depending upon where the walls are used at the south abutment (i.e. in front of the abutments in a false abutment configuration, or on the approach embankment side slopes), RSS walls at this site would be between about 7 m and 9 m high.

For a typical RSS wall, the front facing panels are supported on a strip footing placed at shallow depth below the ground surface in front of the wall, below any topsoil, loose fill or unsuitable native soils. Because of the presence of existing fill behind the south abutment (associated with the existing Highway 7 embankment), allowance should be made for provision of a 0.6 m thick granular pad below the facing footing.

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 below any topsoil/organics on the properly prepared sand and silt till deposit:

Wall Height	***Assumed Footing Width***	***Factored Geotechnical Resistance at ULS***
7 m	4.7 m	450 kPa
9 m	6 m	600 kPa

The geotechnical resistance at SLS, for 25 mm of settlement, may be taken as 300 kPa. The majority of the settlement of the RSS walls will occur during construction since the founding soils at the south abutment and approach embankment are essentially granular (i.e. sand and silt till and cohesionless soil interlayers).

The resistance to lateral forces / sliding resistance between the compacted granular fill (assumed to be Granular "A") and the subgrade soils should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, tan δ', may be taken as 0.5 for compacted Granular "A" placed on the properly prepared, dense to very dense sand and silt till. 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 greater than 1.1 against liquefaction is obtained for the soils in the vicinity of the south abutment, for magnitude 6.2 earthquake events. Pseudo-static stability analysis indicates that the ground surface acceleration due to the design earthquake event would not result in global instability of an RSS wall at the south abutment.

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. 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 a given depth (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 Approach Embankment Design and Construction

The construction of the Dwyer Hill Road underpass will require placement of up to about 9.5 m of fill within the limits of the approach embankments. At the south approach, approximately 1.2 m of fill (associated with the existing Highway 7 corridor) is present, overlying dense to very dense sand and silt till. At the north approach, loose to compact sands and silts overlie a firm to very stiff silty clay layer that varies from 0.2 m to 1.6 m in thickness as encountered in Boreholes 02-701, 02-702, 02-706, 02-707 and 02-720.

5.8.1 Subgrade Preparation and Embankment Construction

Any topsoil, organic matter and loosened soils should be stripped from below the approach embankment areas, and all subgrade soils should be proof-rolled prior to fill placement. The approach embankment subgrades should be inspected by qualified personnel prior to the placement of structural fill. Embankment fill should be placed in regular lifts with a 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 approach embankment height is greater than 8 m, a mid-height berm at least 2 m in width is required for maintenance purposes. 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 Approach Embankment Stability

Limit equilibrium slope stability analyses were performed using the commercially available program SLOPE/W, produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis, to check that a minimum factor of safety of 1.3 is achieved for the proposed approach embankment height and geometry under static conditions. This minimum factor of safety is considered appropriate for the embankments at this site considering the design requirements and the available field and laboratory testing data.

With appropriate subgrade preparation and proper placement and compaction of embankment fill materials, the 8 m to 9.5 m high approach embankments with side slopes maintained at 2 horizontal to 1 vertical (2H:1V) will have a factor of safety of greater than 1.3 against deep-seated slope instability. Static slope stability analyses for this embankment configuration were carried out using the following parameters, based on field and laboratory test data and accepted correlations:

Soil Deposit	*Bulk Unit Weight*	*Effective Friction Angle*	*Undrained Shear Strength*
Embankment Fill	20 – 22 kN/m^3	32° to 35°	–
Surficial Silty Sand to Sand and Gravel	19 – 20 kN/m^3	30° to 32°	–
Silty Clay	18 kN/m^3	–	50 kPa
Sandy Silt Till	19 – 21 kN/m^3	32° to 35°	–

The liquefaction potential of the soils below the embankment under seismic loading has been considered using the empirical method outlined in Section C.4.6.2 of the *CHBDC Commentary*, which correlates the cyclic resistance ratio of the soils with their normalized penetration resistance and fines content. Based on this assessment, a factor of safety of less than 1.1 against liquefaction is obtained for magnitude 6.2 earthquake events for areas under the north approach embankment toes, where the surficial sand has low fines content, low SPT "N" values representative of a very loose to 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 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 Approach Embankment Settlement

Settlement of the immediate approach embankments will occur due to compression of the surficial sand to silty sand and sand and silt till strata, consolidation of the firm to very stiff silty clay deposit that was encountered at the north abutment / approach embankment, and compression of the new embankment fill itself. 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 embankment foundation soils were carried out using the commercially available computer program Unisettle. The immediate compression of the very loose to compact surficial 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. Provided that proper subgrade preparation is carried out, the settlement of the existing granular soils at the site is expected to be less than 50 mm at the north approach, and less than 25 mm at the south approach, as a result of construction of the 8 m to 9.5 m high approach embankments. This compression is expected to occur rapidly (i.e. during or shortly after construction).

At the north abutment / approach embankment, the boreholes encountered between 0.2 m and 1.6 m of firm to very stiff silty clay. The consolidation settlement of this silty clay deposit was modelled by estimating consolidation parameters from correlations with the vane shear strength and Atterberg limits test results. The following parameters were used in the analyses:

Soil Unit	Preconsolidation Pressure P_c'	Initial Void Ratio e_o	Recompression Index C_r	Compression Index C_c
Silty Clay Deposit	185 kPa	1.1	0.05	2.0

The consolidation settlement of the foundation soils at the north approach is expected to be between 50 mm and 150 mm as a result of construction of the 8 m to 9.5 m high embankment; the magnitude of the settlement will vary with the thickness of the underlying silty clay stratum. It is expected that about one-half of this settlement would occur relatively quickly (possibly within two months), with the remainder occurring over the following two to five years. The observed settlement of the embankment surface, following paving of the roadway, may therefore be on the order of 25 mm to 75 mm. Based on this, it is recommended that consideration be given to constructing the approach embankments as early as possible in the contract to maximize the amount of settlement that occurs prior to paving of Dwyer Hill Road.

5.9 Design and Construction Considerations

5.9.1 Excavation

At the south abutment, excavation for construction of spread footings or pile caps (if not perched within the embankment) would extend through at least 1.8 m of loose to very dense sand and silt till. At the centre pier, if adopted, excavations for construction of spread footings founded on the bedrock would extend through about 5.5 m to 6 m of soil, including existing fill, generally loose surficial sand to silty sand, stiff silty clay, and very loose to loose till. The soils are generally water-bearing, as discussed in Section 5.9.2.

Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act (OHSA) for Construction Activities. The water-bearing surficial sands and silty sands are classified as Type 3 soil, and the water-bearing sand and silt till soils are classified as Type 2 soil, according to the OHSA. Temporary excavations (i.e. those which are only open for a relatively short period) through these overburden soils should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V) assuming that the overburden soils are dewatered. Shallower side slopes will be required if full dewatering cannot be achieved, as is likely to occur immediately above the bedrock surface.

5.9.2 Groundwater and Surface Water Control

The groundwater level at the site is approximately 1 m below ground surface in the vicinity of the north approach and north abutment, 2.3 m below ground surface at the center pier (raised parking lot grade), and 0.5 m below ground surface at the south abutment. Excavations to expose the subgrade soils and construct footings or pile caps will require groundwater control. Given the available space, it is likely that open-cut excavations could be carried out in conjunction with a shallow eductor system to lower the groundwater level within the overburden, supplemented by pumping from sumps at the base of the excavations. As noted in Section 5.9.1, shallow open-cut side slopes (approximately 3H:1V) will be required where full dewatering cannot be achieved prior to excavation.

Surface water should be directed away from the excavation works, and consideration should be given to scheduling construction to avoid foundation / subgrade excavation in the spring when water levels are likely to be highest.

As noted in Section 5.5, if drilled shafts are adopted at this site, the use of a temporary liner (possibly in conjunction with drilling mud) will be required within the overburden to support the auger holes during installation and 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. The presence of cobbles and/or boulders was generally inferred from the augers grinding during borehole advance, and from cobbles recovered from auger flights. A boulder was cored in one of the boreholes.

The presence of such obstructions will affect the installation of driven steel H-piles or drilled shaft foundations. A Non-Standard Special Provision should be included in the Contract Documents to ensure that the Contractor is equipped to handle the presence of cobbles or boulders during the installation of driven steel H-piles or drilled shaft foundations.

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MT/LCC/ASP/FJH/lcc
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TABLE 1
COMPARISON OF FOUNDATION ALTERNATIVES
DWYER HILL ROAD UNDERPASS STRUCTURE

Foundation Option	*Feasibility*	*Advantages*	*Disadvantages*	*Relative Costs*	*Risks/Consequences*
Spread footings on overburden soil (for underpass structure and concrete retaining walls)	• Feasible only at south abutment and associated retaining walls • Loose sands and relatively soft silty clay at pier and north abutment preclude use of shallow spread footings	• Minimal excavation required	• Not feasible at north abutment or centre pier due to differential consolidation settlement in silty clay stratum • Potential differential settlement between spread footing on soil and other foundations on bedrock • Local dewatering required	• Less expensive compared to deep foundation options.	• Potential for differential settlement between foundation elements, since other foundations will be extended to bedrock
Spread footings founded within approach embankment fill (for abutments and concrete retaining walls)	• Feasible only at south abutment unless preloading/ surcharging of the north abutment is carried out to minimize consolidation settlement	• Minimal groundwater control required • Minimizes abutment wall height	• Not feasible at north abutment due to differential consolidation settlement in silty clay stratum • Potential differential settlement between spread footing within embankment fill and other foundations on bedrock	• Probably least expensive foundation construction costs	• Potential for differential settlement between foundation elements, due to other foundations on bedrock
Spread footings founded on bedrock (for underpass structure)	• Feasible at centre pier, as alternative to deep foundations • Could also be feasible at abutments but would result in very high abutment wall	• High bearing resistance • Dowelling of footing into bedrock provides a more economic alternative to socketting steel H-piles or drilled shafts into the medium strong bedrock to resist lateral loading / seismic forces, if required • Differential settlement between centre pier and abutments founded on bedrock will be minimal	• Requires excavation down to about 6 m depth with significant groundwater control • May not be possible to fully dewater soils immediately overlying bedrock; could require shallower excavation sideslopes (3H:1V instead of 1H:1V in wet areas)	• Probably most expensive spread footing option owing to costs associated with dewatering and excavation	• Potential for difficulties with dewatering that could affect the construction schedule

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

Foundation Option	*Feasibility*	*Advantages*	*Disadvantages*	*Relative Costs*	*Risks/Consequences*
Spread footings founded on compacted Granular "A" pad following excavation to bedrock (centre pier)	• Feasible at centre pier if open-cut excavation allows use of proper compaction equipment	• Reduction in concrete quantities	• No reduction in excavation or groundwater control requirements • Could be difficult to compact granular fill in reduced space, depending on excavation configuration	• May be less than spread footing for centre pier founded on bedrock	• Potential for differential settlement between abutments and centre pier, if abutment foundations supported on bedrock • Potential for difficulties with dewatering that could affect the construction schedule
Steel H-pile foundations driven to found on or drilled to socket nominally into bedrock (for underpass structure and concrete retaining walls)	• Feasible for support of all foundation elements and retaining walls	• High bearing resistance • Negligible settlement • Site conditions appropriate for use of integral abutments	• If lateral / seismic loading conditions merit, pile toe may have to be socketted into medium strong bedrock, which would require coring or churn drilling • If sockets required, temporary liner necessary • Possibility of encountering cobbles or boulders during pile driving	• May be less expensive than drilled shaft option with rock sockets, owing to potentially smaller socket diameter	• If required for pile toe fixity, socketting into the medium strong bedrock could be difficult and time-consuming
Caissons founded or socketted nominally into bedrock (for underpass structure or concrete retaining walls)	• Feasible for support of all foundation elements and retaining walls	• High bearing resistance • Negligible settlement	• Temporary liners required to minimize disturbance to surrounding soils • Possibility of encountering cobbles or boulders during drilled shaft installation • If rock socket required, coring or churn drilling will be required to form socket in medium strong bedrock	• May be more expensive than steel H-pile option if rock sockets are necessary, owing to potentially larger socket diameter	• If required for pile toe fixity, socketting into the medium strong bedrock could be difficult and time-consuming
Retained Soil System (RSS) walls	• Feasible only at south abutment (dense to very dense soils) • Not feasible at north abutment (variable thicknesses of compressible silty clay and loose sands / silts	• Minimal excavation and groundwater control required for construction • New approach embankments are being constructed for this new underpass structure, so no	• Only suitable at south abutment; very loose to loose, water-bearing granular soils and variable thicknesses of compressible silty clay at north abutment preclude the use of RSS walls for wing walls / retaining walls at that location	• Generally less expensive than concrete retaining wall foundations	• Potentially more settlement than for concrete wing walls / 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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d:

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

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

Piezo-Cone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils

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

IV. SOIL TESTS

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

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

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 stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1+\sigma_2+\sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

w	water content
w_l	liquid limit
w_p	plastic limit
I_p	plasticity index $= (w_l - w_p)$
w_s	shrinkage limit
I_L	liquidity index $= (w - w_p)/I_p$
I_C	consistency index $= (w_l - w)/I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index $= (e_{max} - e)/(e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation pressure
OCR	over-consolidation ratio $= \sigma'_p/\sigma'_{vo}$

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = tan δ
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ 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
FO - Foliation/Schistosity
CL - Cleavage
SH - Shear Plane/Zone
VN - Vein
F - Fault
CO - Contact
J - Joint
FR - Fracture
MF - Mechanical Fracture
∥ - Parallel To
⊥ - Perpendicular To

P - Polished
S - Slickensided
SM - Smooth
R - Ridged/Rough
ST - Stepped
PL - Planar
FL - Flexured
UE - Uneven
W - Wavy
C - Curved

Golder Associates

Foundation Design

RECORD OF BOREHOLE No 02-701

1 OF 1

METRIC

PROJECT 021-1155-7

W.P. 251-99-00

DIST

HWY 7

DATUM Geodetic

LOCATION N 5007182.3 ;E 340944.0

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

DATE MAY 20, 2003

ORIGINATED BY J.S.

COMPILED BY M.I.C.

CHECKED BY L.C.C.

SOIL PROFILE		SAMPLES			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GR	SA	SI	CL
133.0	GROUND SURFACE							
0.0	Topsoil							
0.2	SAND, trace silt and gravel Loose to compact Brown Moist	1	SS	5	1	95	(4)	
		2	SS	8				
		3	SS	15				
130.3 2.7	Silty CLAY with sand seams Stiff to very stiff Brown							
129.8 3.2	Silty CLAY, trace sand Firm to stiff Grey	4	SS	2				
128.7 4.3	SAND and SILT, some gravel and clay (TILL) Loose Grey Wet	5	SS	4				
		6	SS	4	12	46	30	12
128.0 5.0	Interlayered LIMESTONE and DOLOMITIC LIMESTONE (BEDROCK) Fresh Weak to medium strong Thinly to thickly bedded Grey Bedrock cored from 5.0m to 8.4m depth. For details refer to Record of Drillhole 02-701							
124.6 8.4	END OF BOREHOLE							

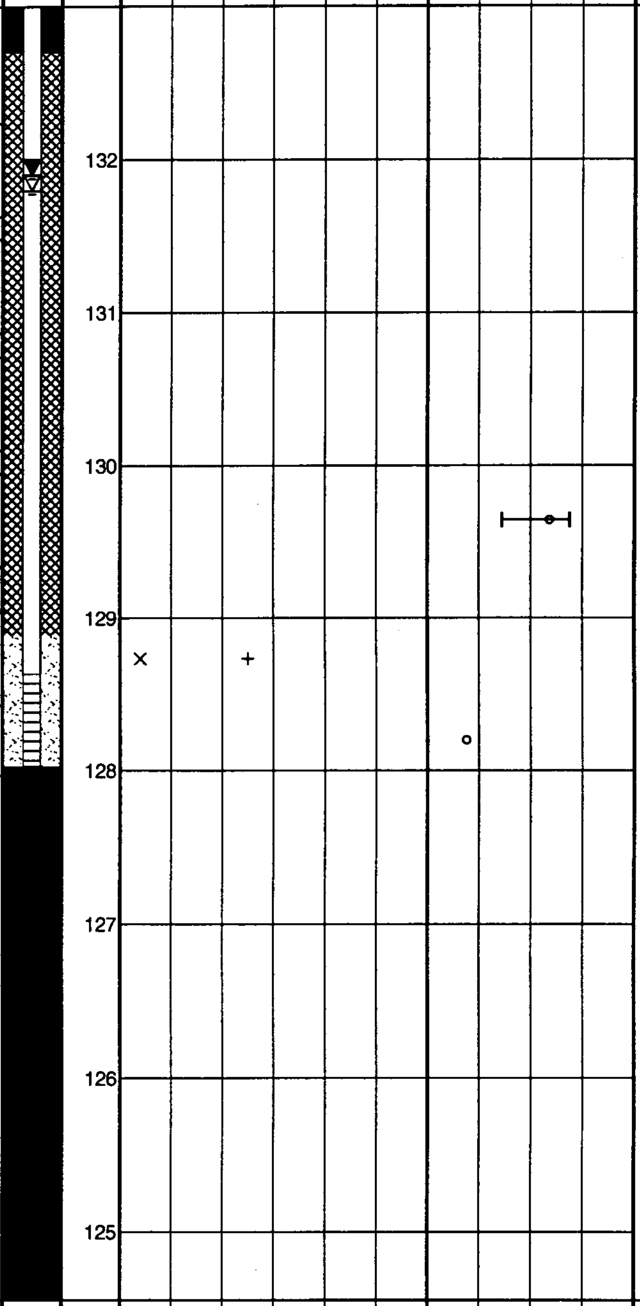
STRAT PLOT | GROUND WATER CONDITIONS | ELEVATION SCALE | UNIT WEIGHT γ kN/m³

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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

PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L

WATER CONTENT (%) 15 30 45



Notes:

1. Water encountered during overburden drilling at about 1.2m depth (Elev. 131.8m).

2. Water level in piezometer measured at 1.1m depth (Elev. 131.9m) on June 6, 2003.

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

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

MISS_MTO 021-1155-7.GPJ ON_MOT.GDT 18/8/04

PROJECT: 021-1155-7

RECORD OF DRILLHOLE: 02-701

SHEET 2 OF 2

LOCATION: N 5007182.3 ;E 340944.0

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: MAY 20, 2003

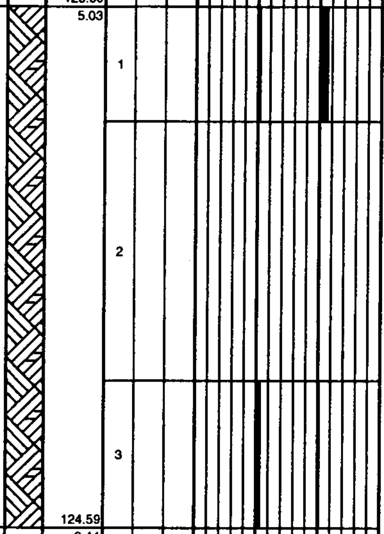
DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Marathon

DATUM: Geodetic

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE % 80 60 40 20	RECOVERY SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT. INDEX PER 0.3 5 10 15 20	DISCONTINUITY DATA DIP w.r.t. CORE AXIS 0 30 60 90	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}	DIAMETRAL POINT LOAD INDEX (MPa) 2 4 6	NOTES WATER LEVELS INSTRUMENTATION
		.		128.00												
	Rotary Drill NQ Core	Interlayered LIMESTONE and DOLOMITIC LIMESTONE (BEDROCK) Fresh Weak to medium strong Thinly to thickly bedded Grey		5.03	1											
6																
					2											
7																
8					3											
				124.59 8.44												
		END OF BOREHOLE														
9																
10																
11																
12																
13																
14																
15																



DEPTH SCALE

1 : 50

Golder Associates

LOGGED: J.S.

CHECKED: L.C.C.

RECORD OF BOREHOLE No 02-701A 1 OF 1 **METRIC**

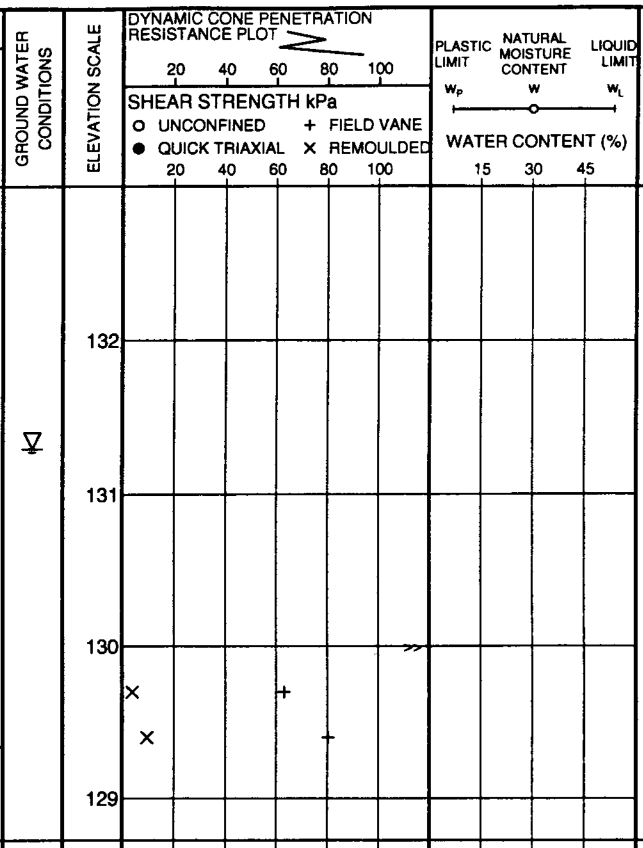
PROJECT 021-1155-7
W.P. 251-99-00
LOCATION N 5007183.1 ;E 340944.7
ORIGINATED BY PAH
DIST HWY 7
BOREHOLE TYPE CME 55 Bombardier, 108mm I.D. Hollow Stem Auger
COMPILED BY JDR
DATUM Geodetic
DATE JULY 18,2003
CHECKED BY L.C.C.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
133.0	GROUND SURFACE								
0.0	SAND, trace silt and gravel Brown Moist to wet								
130.1 2.9	Silty CLAY, with sand seams Very stiff Grey brown								
129.8 3.2	Silty CLAY, with sand seams Stiff Grey								
129.2 3.8	SAND and SILT, some gravel and clay (TILL) Loose Grey Wet	1	SS	4					
128.7 4.3	END OF BOREHOLE								

Notes:

1. Water level in open borehole at 1.7m depth (Elevation 131.3m) during drilling

2. Shelby tube sample obtained in grey silty clay between 3.1m and 3.6m depth, in adjacent borehole.



REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

RECORD OF BOREHOLE No 02-702

1 OF 1

METRIC

PROJECT 021-1155-7
W.P. 251-99-00
DIST HWY 7
DATUM Geodetic
LOCATION N 5007180.0 ;E 340948.3
BOREHOLE TYPE CME 55 Bombardier, 108mm I.D. Hollow Stem Auger
DATE MAY 15, 2003
ORIGINATED BY J.S.
COMPILED BY M.I.C.
CHECKED BY L.C.C.

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 15 30 45

UNIT WEIGHT γ kN/m³

SOIL PROFILE			SAMPLES					REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	GR	SA	SI	CL
132.8	GROUND SURFACE										
0.0	Topsoil										
	Sandy silt, some gravel, trace fragments and organics (FILL)										
132.0 0.9	SAND, trace silt and gravel Loose Brown Moist to wet		1	SS	4	∇	132				
			2	SS	5		131	0	92	7	1
130.7 2.1	SAND and GRAVEL Compact Brown Wet		3	SS	29		130	35	57	(8)	
129.9 2.9	Silty CLAY, with sand seams Stiff to very stiff Grey - brown		4	SS	6						
129.5 3.4	Silty CLAY, with sand seams Firm to stiff Grey										
129.0 3.8	SAND and SILT, some gravel and clay (TILL) Very loose Grey Wet		5	SS	3		129				
			6	SS	2		128				
127.9 4.9	END OF BOREHOLE Auger refusal - Probable bedrock Note: Water encountered in open borehole at about 0.9m depth (Elev. 129.9m)										

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

PROJECT 021-1155-7

RECORD OF BOREHOLE No 02-703 1 OF 1 **METRIC**

W.P. 251-99-00

LOCATION N 5007156.7 ;E 340981.7

ORIGINATED BY J.S.

DIST HWY 7

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

COMPILED BY M.I.C.

DATUM Geodetic

DATE MAY 21, 2003

CHECKED BY L.C.C.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 15 30 45	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						GR	SA	SI	CL
134.0	GROUND SURFACE													
0.0	ASPHALT													
0.1	Sandy silt to silty sand, some gravel, trace organics and wood fragments (FILL) Loose to dense Brown to grey Moist													
			1	SS	33		133				4	47	41	8
			2	SS	5		132							
131.7 2.3	SAND and SILT, some gravel, trace clay, with cobbles (TILL) Compact to very dense Brown to Grey Moist to wet													
			3	SS	29		131							
			4	SS	58				○					
						∇								
			5	SS	112		130							
			6	SS	110/0.15									
							129							
128.5	Cobbles cored from 5.3m to 5.5m													
5.5	Interlayered LIMESTONE and DOLOMITIC LIMESTONE (BEDROCK) Slightly weathered, becoming fresh below 6.3m depth Weak to medium strong Thinly to thickly bedded Grey						128							
							127							
	Bedrock cored from 5.5m to 8.8m depth. For details refer to Record of Drillhole 02-703						126							
125.2 8.8	END OF BOREHOLE Note: Water encountered during overburden drilling at about 3.8m depth (Elev. 130.2m).													

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

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

PROJECT: 021-1155-7

RECORD OF DRILLHOLE: 02-703

SHEET 2 OF 2

LOCATION: N 5007156.7 ;E 340981.7

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: MAY 21, 2003
DRILL RIG: CME 55 Bombardier
DRILLING CONTRACTOR: Marathon

DATUM: Geodetic

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE % 80 60 40 20	RECOVERY SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT. INDEX PER 0.3 5 10 15 20	DISCONTINUITY DATA DIP w.r.t. CORE AXIS 0 30 60 90	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}	DIAMETRAL POINT LOAD INDEX (MPa) 2 4 6	NOTES WATER LEVELS INSTRUMENTATION
				128.50												
	Rotary Drill NQ Core	Interlayered LIMESTONE and DOLOMITIC LIMESTONE (BEDROCK) Slightly weathered, becoming fresh below 6.3m depth Weak to medium strong Thinly to thickly bedded Grey	▨	5.49	2											
6 7			▨		3											
8			▨	125.21	4											
9 10 11 12 13 14 15		END OF BOREHOLE		8.78												

MISS_ROCK 021-1155-7 ROCK.GPJ GLDR_CAN.GDT 18/8/04 JFC

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: J.S.
CHECKED: L.C.C.

RECORD OF BOREHOLE No 02-704

1 OF 1

METRIC

PROJECT 021-1155-7			ORIGINATED BY J.S.
W.P. 251-99-00		LOCATION N 5007134.0 ;E 341015.3	
DIST	HWY 7	BOREHOLE TYPE CME 55 Bombardier, 108mm I.D. Hollow Stem Auger	COMPILED BY M.I.C.
DATUM Geodetic		DATE MAY 12, 2003	CHECKED BY L.C.C.

SOIL PROFILE		SAMPLES			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
132.9	GROUND SURFACE							
0.0	Topsoil							
132.6 0.3	SAND and SILT, some organics, trace clay (TILL) Compact to dense Brown Moist to wet	1	SS	11				
		2	SS	40				
131.7 1.3	SAND, trace to some silt and gravel, to silty SAND, trace gravel Dense to very dense Brown to grey brown Wet	3	SS	44	1	90	(9)	
		4	SS	54/.25*				
130.0 2.9	SAND and SILT, some gravel, trace clay (TILL) Very dense Grey Wet	5	SS	100/.10				
129.4 3.5	Probable limestone boulder - Till seam at 4.1m depth							
128.8 4.1	Interlayered LIMESTONE and DOLOMITIC LIMESTONE (BEDROCK) Fresh Weak to medium strong Medium to thickly bedded Grey Boulder / bedrock cored from 3.5m to 6.9m depth. For details refer to Record of Drillhole 02-704							
126.0 6.9	END OF BOREHOLE							

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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

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

WATER CONTENT (%) 15 30 45

UNIT WEIGHT γ kN/m³

Notes:

* Spoon bouncing after 54 blows.

1. Water encountered during overburden drilling at about 1.1m depth (Elev. 131.8m).

2. Water level in piezometer measured at 0.5m depth (Elev. 132.4m) on June 6, 2003

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

PROJECT: 021-1155-7

RECORD OF DRILLHOLE: 02-704

SHEET 2 OF 2

LOCATION: N 5007134.0 ;E 341015.3

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: MAY 12, 2003
DRILL RIG: CME 55 Bombardier
DRILLING CONTRACTOR: Marathon

DATUM: Geodetic

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
			129.40												
4	Rotary Drill NQ Core	Probable limestone boulder Till seam at 4.1m depth Slightly weathered Grey	3.50	1											
5		Interlayered LIMESTONE and DOLOMITIC LIMESTONE (BEDROCK) Fresh Weak to medium strong Medium to thickly bedded Grey	128.82 4.08	2											
6				3											
7		END OF BOREHOLE	126.00 6.90												
8															
9															
10															
11															
12															
13															

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: J.S.
CHECKED: L.C.C.

MISS_ROCK 021-1155-7 ROCK.GPJ GLDR_CAN.GDT 18/8/04 JFC

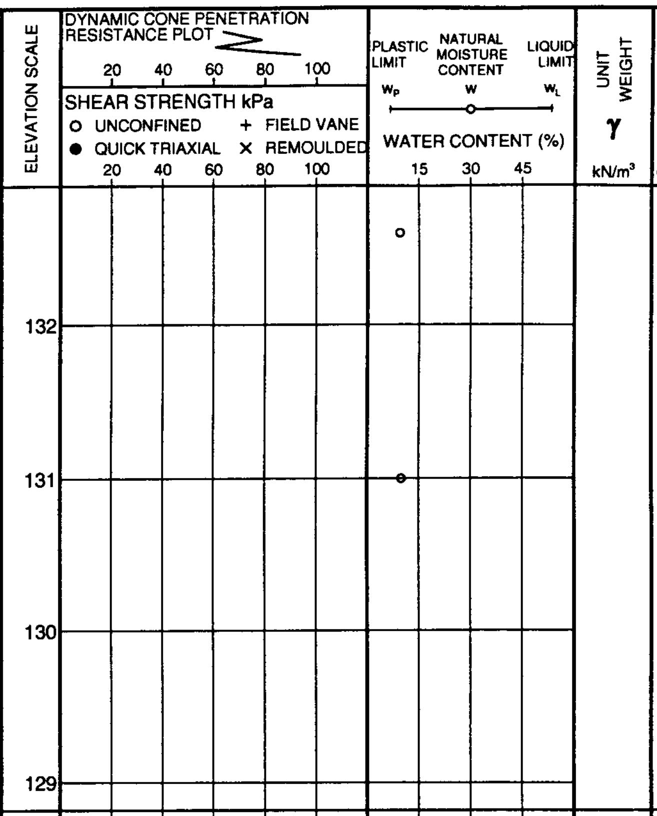
RECORD OF BOREHOLE No 02-705

1 OF 1

METRIC

PROJECT 021-1155-7			ORIGINATED BY S.I.
W.P. 251-99-00	LOCATION N 5007131.3 ;E 341019.4		
DIST	HWY 7	BOREHOLE TYPE CME 55 Bombardier, 108mm I.D. Hollow Stem Auger	COMPILED BY M.I.C.
DATUM Geodetic	DATE MAY 13, 2003		CHECKED BY L.C.C.

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
132.9	GROUND SURFACE					
0.0	Topsoil					
0.1	SAND and SILT, some gravel, trace clay with cobbles and boulders (TILL) Loose Brown Moist	1	SS	7		
132.0 0.9	Silty SAND, trace gravel Very dense Brown Moist	2	SS	82		
131.3 1.7	Sandy SILT, trace gravel and clay Very dense Grey Stratified Moist	3	SS	160	∇	0 26 64 10
130.7 2.2	SAND and GRAVEL, trace silt, with cobbles Very dense Grey Wet	4	SS	137/0.1		
		5	SS	175		
128.8 4.1	END OF BOREHOLE Auger refusal - Probable bedrock Note: Water encountered in open borehole at about 2.1m depth (Elev. 131.8m)					



+³, X³: Numbers refer to Sensitivity

O ³% STRAIN AT FAILURE

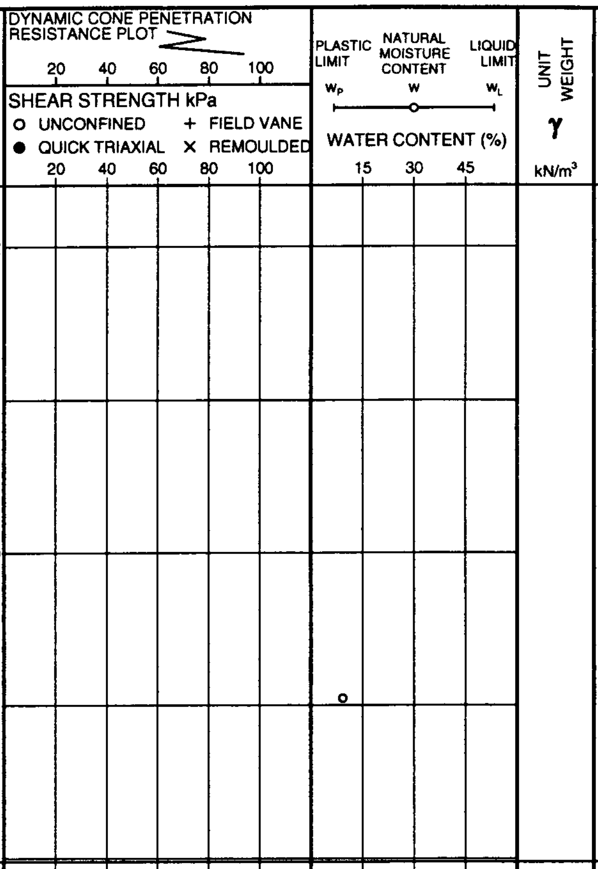
RECORD OF BOREHOLE No 02-706

1 OF 1

METRIC

PROJECT 021-1155-7
W.P. 251-99-00
LOCATION N 5007166.1 ;E 340929.2
ORIGINATED BY J.S.
DIST ______ HWY 7
BOREHOLE TYPE CME 55 Bombardier, 108mm I.D. Hollow Stem Auger
COMPILED BY M.I.C.
DATUM Geodetic
DATE MAY 15, 2003
CHECKED BY L.C.C.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES			GR	SA	SI	CL
132.4	GROUND SURFACE									
0.0 132.2	Topsoil									
0.2	SAND, trace silt and gravel, to silty SAND, trace gravel Loose Brown Moist to wet					132				
		1	SS	4			0	96	3	1
						131				
		2	SS	9						
130.3	Silty CLAY Stiff to very stiff Grey - brown									
130.1 2.3	SAND and SILT, some gravel and clay (TILL) Loose to compact Brown to grey Wet	3	SS	21		130				
		4	SS	12		129				
		5	SS	6						
128.0						128				
4.4	END OF BOREHOLE Auger refusal - Probable bedrock Note: Water encountered in open borehole at about 1.1m depth (Elev. 131.3m)									



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

Golder Associates

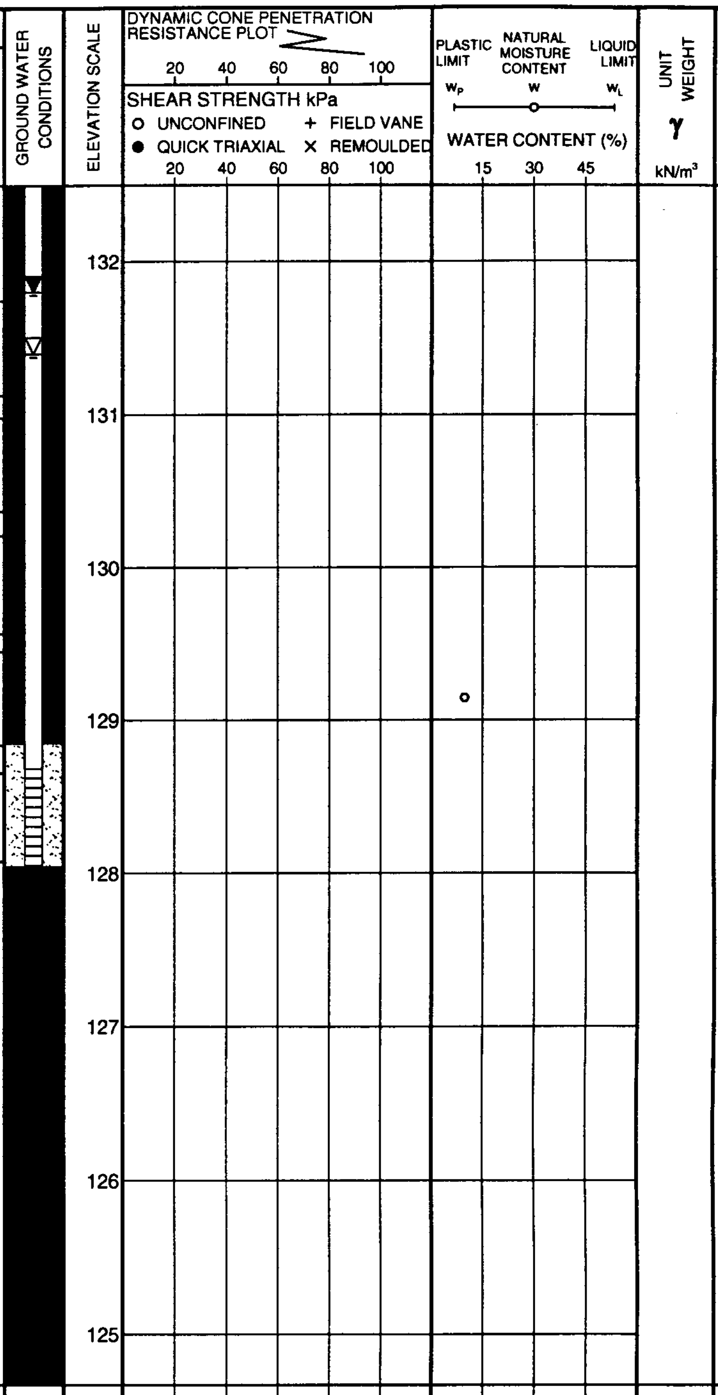
RECORD OF BOREHOLE No 02-707

1 OF 1

METRIC

PROJECT 021-1155-7
W.P. 251-99-00
DIST HWY 7
DATUM Geodetic
LOCATION N 5007163.2 ;E 340932.9
BOREHOLE TYPE CME 55 Bombardier, 108mm I.D. Hollow Stem Auger
DATE MAY 15, 2003
ORIGINATED BY J.S.
COMPILED BY M.I.C.
CHECKED BY L.C.C.

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
132.5	GROUND SURFACE							
0.0	Topsoil							
0.1	SAND, trace silt and gravel, to silty SAND, trace gravel Very loose to compact Yellow-brown to brown Moist to wet	1 2	SS SS	3 16	0	96	3	1
130.6	SAND and GRAVEL, trace silt Compact Grey - brown							
130.2 2.3	Silty CLAY Stiff to very stiff Grey - brown SAND and SILT, some gravel and clay (TILL) Loose to compact Brown to grey Wet	3 4 5	SS SS SS	9 11 28				
128.1 4.5	Interlayered LIMESTONE and DOLOMITIC LIMESTONE (BEDROCK) Fresh Weak to medium strong Thinly to thickly bedded Grey Bedrock cored from 4.5m to 7.8m depth. For details refer to Record of Drillhole 02-707							
124.7 7.8	END OF BOREHOLE							



Notes:

1. Water encountered during overburden drilling at 1.1m depth (Elev. 131.4m)

2. Water level in piezometer measured at 0.7m depth (Elev. 131.8m) on June 6, 2003

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

PROJECT: 021-1155-7

RECORD OF DRILLHOLE: 02-707

SHEET 2 OF 2

LOCATION: N 5007163.2 ;E 340932.9

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: MAY 15, 2003

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Marathon

DATUM: Geodetic

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
				128.10 4.45												
5	Rotary Drill NQ Core	Interlayered LIMESTONE and DOLOMITIC LIMESTONE (BEDROCK) Fresh Weak to medium strong Thinly to thickly bedded Grey			1											
6					2											
7					3											
8		END OF BOREHOLE		124.72 7.83												
9																
10																
11																
12																
13																
14																

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: J.S.
CHECKED: L.C.C.

RECORD OF BOREHOLE No 02-708

1 OF 2 **METRIC**

PROJECT 021-1155-7

W.P. 251-99-00

DIST ____ HWY 7

DATUM Geodetic

LOCATION N 5007141.0 ;E 340967.0

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

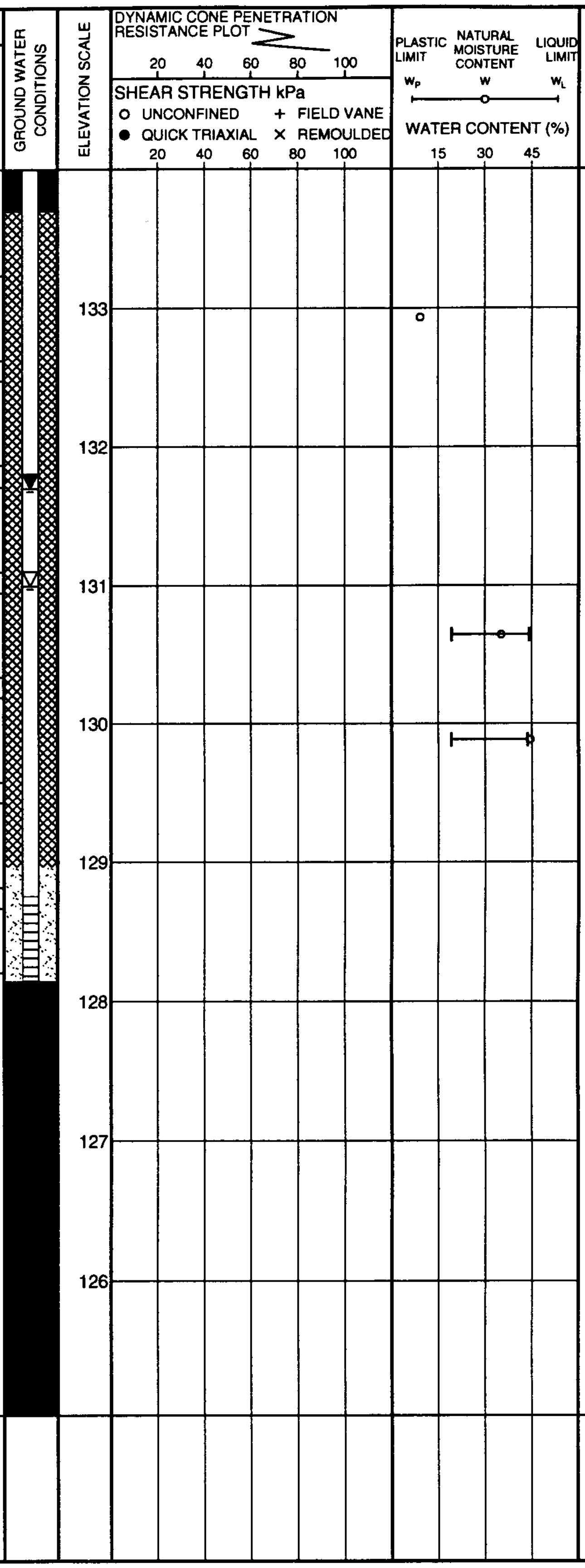
DATE MAY 21, 2003

ORIGINATED BY J.S.

COMPILED BY M.I.C.

CHECKED BY L.C.C.

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
134.0	GROUND SURFACE					
0.0 133.8	ASPHALT					
	Gravel (FILL)					
0.3	Sandy silt to silty sand, some gravel, trace organics (FILL) Compact Brown and grey Moist	1	SS	12		
		2	SS	22		
131.6 2.4	SAND, trace silt, trace to some gravel Compact Brown Moist to wet	3	SS	26		
130.9 3.1	Silty CLAY, with sand seams Stiff to very stiff Grey - brown Wet	4	SS	4		
130.0 4.0	Silty CLAY, with sand seams Firm to stiff Grey Wet	5	SS	0		
129.3 4.7	SAND and SILT, some gravel and clay (TILL) Very loose Grey Wet	6	SS	3		
		7	SS	2		
128.2 5.9	Interlayered LIMESTONE and DOLOMITIC LIMESTONE (BEDROCK) Slightly weathered, becoming fresh below 7.3m depth Weak to medium strong Thinly to thickly bedded Grey Bedrock cored from 5.9m to 9.0m depth. For details refer to Record of Drillhole 02-708					
125.0 9.0						



Continued Next Page

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

PROJECT 021-1155-7

RECORD OF BOREHOLE No 02-708 2 OF 2 **METRIC**

W.P. 251-99-00

LOCATION N 5007141.0 ;E 340967.0

ORIGINATED BY J.S.

DIST

HWY 7

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

COMPILED BY M.I.C.

DATUM Geodetic

DATE MAY 21, 2003

CHECKED BY L.C.C.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 15 30 45	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION *— CONTINUED FROM PREVIOUS PAGE —*	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	END OF BOREHOLE Notes: 1. Water encountered during overburden drilling at about 3.0m depth (Elev. 131.0m). 2. Water level in piezometer measured at 2.3m depth (Elev. 131.7m) on June 6, 2003.										

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

PROJECT: 021-1155-7

RECORD OF DRILLHOLE: 02-708

SHEET 3 OF 3

LOCATION: N 5007141.0 ;E 340967.0

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: MAY 21, 2003
DRILL RIG: CME 55 Bombardier
DRILLING CONTRACTOR: Marathon

DATUM: Geodetic

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE % 80 60 40 20	RECOVERY SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT. INDEX PER 0.3 5 10 15 20	DISCONTINUITY DATA DIP w.r.t. CORE AXIS 0 30 60 90	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}	DIAMETRAL POINT LOAD INDEX (MPa) 2 4 6	NOTES WATER LEVELS INSTRUMENTATION
				128.20 5.85												
6	Rotary Drill NQ Core	Interlayered LIMESTONE and DOLOMITIC LIMESTONE (BEDROCK) Slightly weathered, becoming fresh below 7.3m depth Weak to medium strong Thinly to thickly bedded Grey			1											
7					2											
8					3											
9		END OF BOREHOLE		125.09 8.96												
10																
11																
12																
13																
14																
15																

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: J.S.
CHECKED: L.C.C.

RECORD OF BOREHOLE No 02-709

1 OF 1

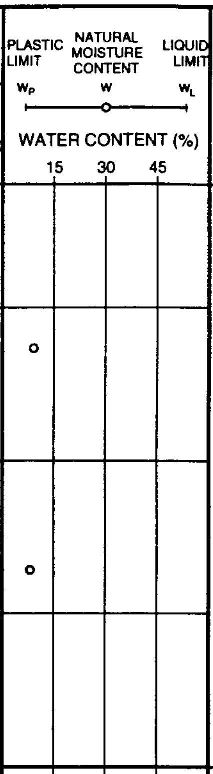
METRIC

PROJECT 021-1155-7
W.P. 251-99-00
DIST
HWY 7
DATUM Geodetic
LOCATION N 5007119.2 ;E 341001.3
BOREHOLE TYPE CME 55 Bombardier, 108mm I.D. Hollow Stem Auger
DATE MAY 13, 2003
ORIGINATED BY J.S.
COMPILED BY M.I.C.
CHECKED BY L.C.C.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES		
132.8	GROUND SURFACE					
0.0	Topsoil					
0.1	SAND and SILT, some gravel, trace clay, occ. cobbles (TILL) Compact to very dense Brown to grey Moist to wet	1	SS	11		
		2	SS	45		132
		3	SS	38		131
		4	SS	150	∇	130
		5	SS	73/0.15		
129.0 3.8	END OF BOREHOLE Auger refusal - Probable bedrock Note: Water encountered in open borehole at about 2.9m depth (Elev. 129.9m)					129

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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



UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)
GR SA SI CL

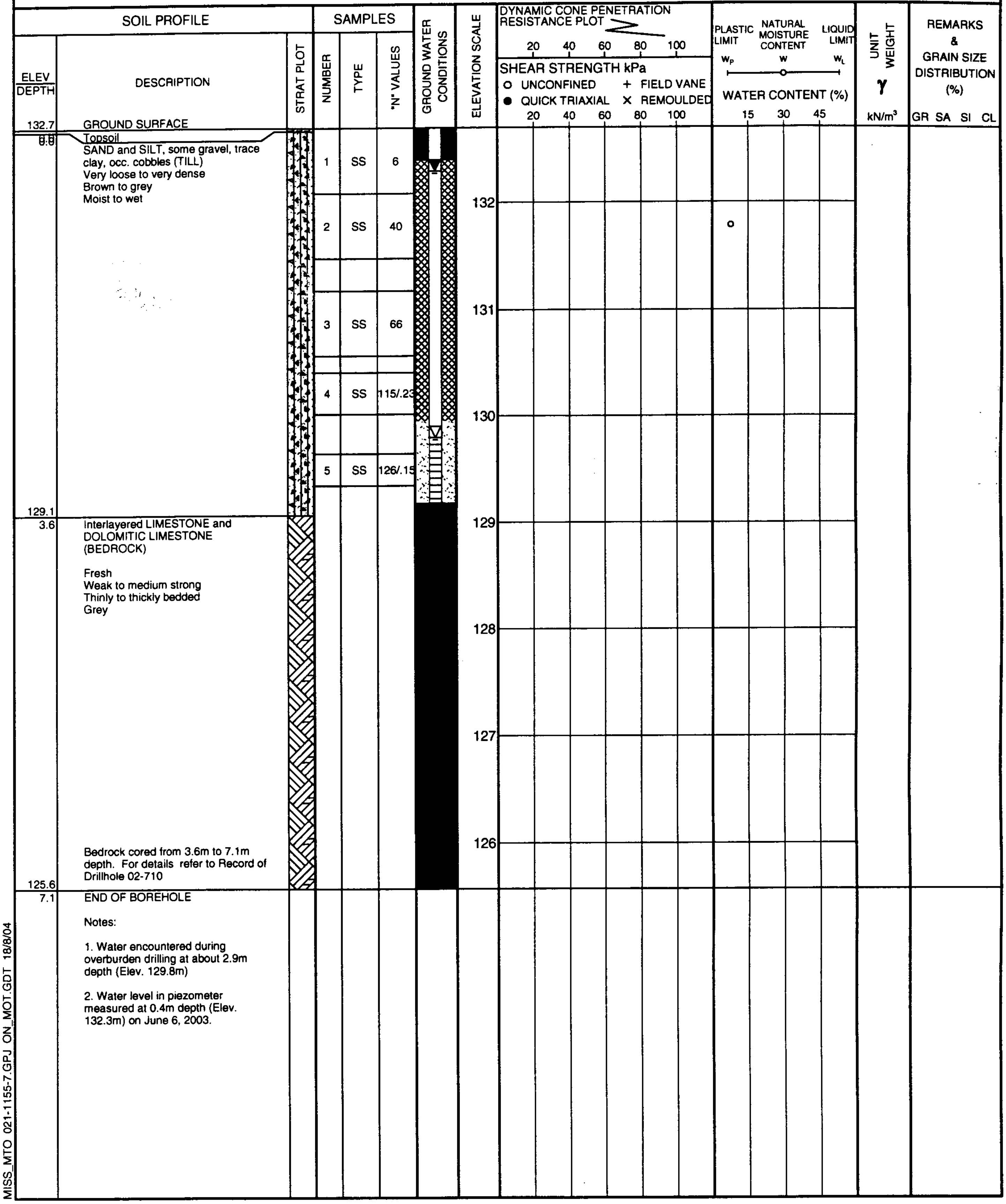
+³, ×³: Numbers refer to Sensitivity
○³% STRAIN AT FAILURE

RECORD OF BOREHOLE No 02-710

1 OF 1

METRIC

PROJECT 021-1155-7
W.P. 251-99-00
DIST ____ HWY 7
DATUM Geodetic
LOCATION N 5007116.4 ;E 341005.6
BOREHOLE TYPE CME 55 Bombardier, 108mm I.D. Hollow Stem Auger
DATE MAY 12, 2003
ORIGINATED BY J.S.
COMPILED BY M.I.C.
CHECKED BY L.C.C.



ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	"N" VALUES
132.7	GROUND SURFACE			
0.0	Topsoil			
	SAND and SILT, some gravel, trace clay, occ. cobbles (TILL) Very loose to very dense Brown to grey Moist to wet	1	SS	6
		2	SS	40
		3	SS	66
		4	SS	115/.23
		5	SS	126/.15
129.1 3.6	Interlayered LIMESTONE and DOLOMITIC LIMESTONE (BEDROCK) Fresh Weak to medium strong Thinly to thickly bedded Grey Bedrock cored from 3.6m to 7.1m depth. For details refer to Record of Drillhole 02-710			
125.6 7.1	END OF BOREHOLE			

Notes:

1. Water encountered during overburden drilling at about 2.9m depth (Elev. 129.8m)

2. Water level in piezometer measured at 0.4m depth (Elev. 132.3m) on June 6, 2003.

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

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

PROJECT: 021-1155-7

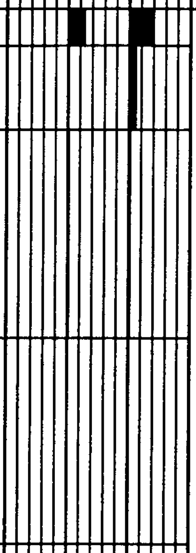
RECORD OF DRILLHOLE: 02-710

SHEET 2 OF 2

LOCATION: N 5007116.4 ;E 341005.6	DRILLING DATE: MAY 12, 2003	DATUM: Geodetic
INCLINATION: -90° AZIMUTH: ---	DRILL RIG: CME 55 Bombardier	
	DRILLING CONTRACTOR: Marathon	

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH / COLOUR % RETURN	RECOVERY: TOTAL CORE % 80 60 40 20	RECOVERY: SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT. INDEX PER 0.3 5 10 15 20	DISCONTINUITY DATA: DIP w.r.t. CORE AXIS 0 30 60 90	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}	DIAMETRAL POINT LOAD INDEX (MPa) 2 4 6	NOTES WATER LEVELS INSTRUMENTATION
		.		129.10												
	Rotary Drill NQ Core	Interlayered LIMESTONE and DOLOMITIC LIMESTONE (BEDROCK) Fresh Weak to medium strong Medium to thickly bedded Grey		3.63	1											
4					2											
5		15mm thick calcite seam at 5.2m depth			3											
6					4											
7				125.60												
		END OF BOREHOLE		7.13												
8																
9																
10																
11																
12																
13																



DEPTH SCALE
1 : 50

Golder Associates

LOGGED: J.S.
CHECKED: L.C.C.

RECORD OF BOREHOLE No 02-714

1 OF 1 **METRIC**

PROJECT 021-1155-7

W.P. 251-99-00

DIST ______ HWY 7

DATUM Geodetic

LOCATION N 5007127.4 ;E 341009.0

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

DATE MAY 13, 2003

ORIGINATED BY S.I.

COMPILED BY M.I.C.

CHECKED BY L.C.C.

SOIL PROFILE			SAMPLES								
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 15 30 45	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
132.8	GROUND SURFACE										
0.0	Topsoil										
0.1	SAND and SILT, trace to some gravel and clay (TILL)										
							132				
							131				
							130				
128.9							129				
3.9	END OF BOREHOLE Auger refusal - Probable bedrock Note: Probehole advanced to refusal without sampling.										

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

RECORD OF BOREHOLE No 02-715

1 OF 1

METRIC

PROJECT 021-1155-7

W.P. 251-99-00

DIST ______ HWY 7

DATUM Geodetic

LOCATION N 5007124.1 ;E 341012.8

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

DATE MAY 13, 2003

ORIGINATED BY S.I.

COMPILED BY M.I.C.

CHECKED BY L.C.C.

SOIL PROFILE			SAMPLES									
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 15 30 45	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
132.8	GROUND SURFACE										
0.0	Topsoil										
0.1	SAND and SILT, some gravel, trace clay with cobbles and boulders (TILL) Loose to very dense Brown Moist		1	SS	7						
							132				
			2	SS	62						
							131				
			3	SS	47						
130.2 2.6	Silty SAND, trace gravel Dense Brown Moist to wet		4	SS	44						
							130				
						∇					
129.6 3.2	Sandy SILT, occ. sand seams Very dense Brown Wet		5	SS	60						
129.3 3.5	SAND and SILT, some gravel, trace clay with cobbles and boulders (TILL) Dense Grey Wet						129				
128.9 3.9	END OF BOREHOLE Auger refusal - Probable bedrock Note: Water encountered in open borehole at about 3.0m depth (Elev. 129.8m)										

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

RECORD OF BOREHOLE No 02-720

1 OF 1

METRIC

PROJECT 021-1155-7

W.P. 251-99-00

DIST ____ HWY 7

DATUM Geodetic

LOCATION N 5007190.5 ;E 340914.9

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

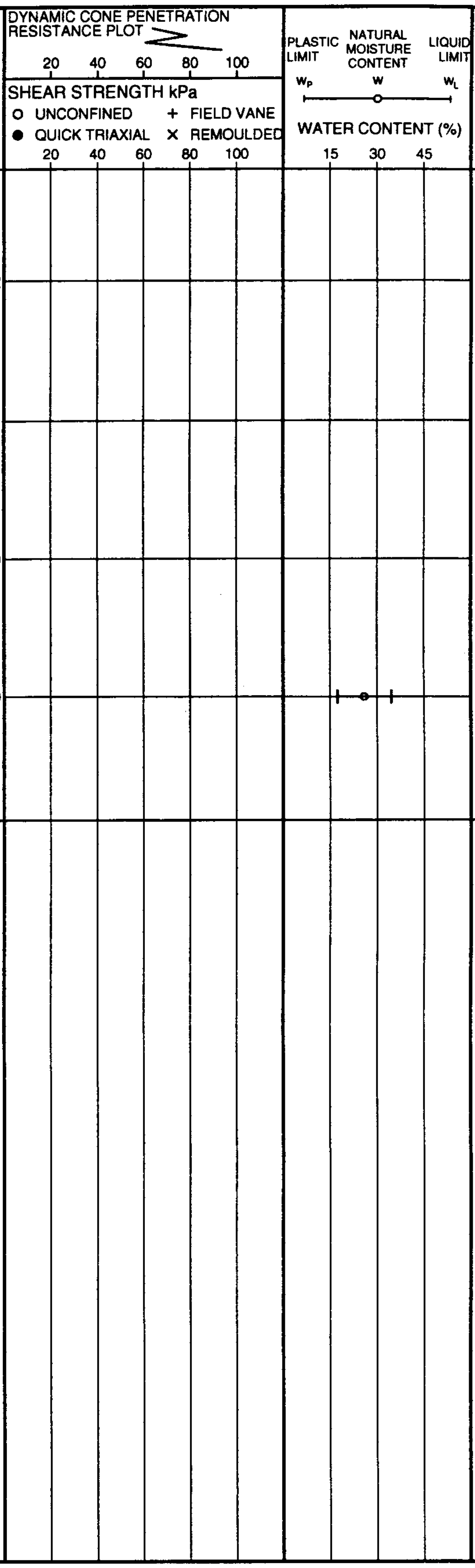
DATE MAY 14, 2003

ORIGINATED BY J.S.

COMPILED BY M.I.C.

CHECKED BY L.C.C.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
132.8	GROUND SURFACE							
0.0	Dark Brown Topsoil							
132.5	Sandy SILT Loose Brown	1	SS	4				
0.5	SAND, trace silt and gravel, trace shells to SILTY SAND, trace gravel Loose Brown Wet	2	SS	4	0	92	6	2
		3	SS	2	0	93	5	2
		4	SS	8				
129.8 3.1	Silty SAND, occ. sand and clay seams Loose Brown Wet	5	SS	9				
129.4 129.1 3.7	Silty CLAY Stiff to very stiff Grey - brown	6	SS	5				
128.8 4.0	Silty CLAY Firm to stiff Grey							
	SAND and SILT, some gravel and clay (TILL) Compact Grey Wet	7	SS	27/0.25				
128.1 4.7	END OF BOREHOLE Auger refusal							



ELEVATION SCALE: 132, 131, 130, 129

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Notes:

1. Water encountered in open borehole at about 0.6m depth (Elev. 132.2m).

2. Water level in piezometer measured at 1.0m depth (Elev. 131.8m) on June 6, 2003

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

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

Golder Associates

RECORD OF BOREHOLE No 02-721

1 OF 1

METRIC

PROJECT 021-1155-7
W.P. 251-99-00
DIST ____ HWY 7
DATUM Geodetic
LOCATION N 5007112.7 ;E 341037.4
BOREHOLE TYPE CME 55 Bombardier, 108mm I.D. Hollow Stem Auger
DATE MAY 30, 2003
ORIGINATED BY J.S.
COMPILED BY M.I.C.
CHECKED BY L.C.C.

SOIL PROFILE ELEV DEPTH	SOIL PROFILE DESCRIPTION	SOIL PROFILE STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 15 30 45	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
133.3	GROUND SURFACE										
0.0	ASPHALT										
0.2	GRAVEL, trace to some sand (FILL)						133				
132.4 0.9	Sandy silt, some gravel, trace organic (FILL) Very loose Brown and gey Wet		1	SS	3	∇					
132.1 1.2	SAND and SILT, some gravel, trace clay, occ.cobbles (TILL) Compact to very dense Brown to grey Wet						132				
			2	SS	63				○		
			3	SS	83		131				
			4	SS	111		130				
			5	SS	78/0.08		129				
128.9 4.5	END OF BOREHOLE Auger refusal - Probable bedrock Note: Water encountered in open borehole at about 0.9m depth (Elev. 132.4m)										

MISS_MTO 021-1155-7.GPJ ON_MOT.GDT 18/8/04

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

MINISTRY OF TRANSPORTATION, ONTARIO

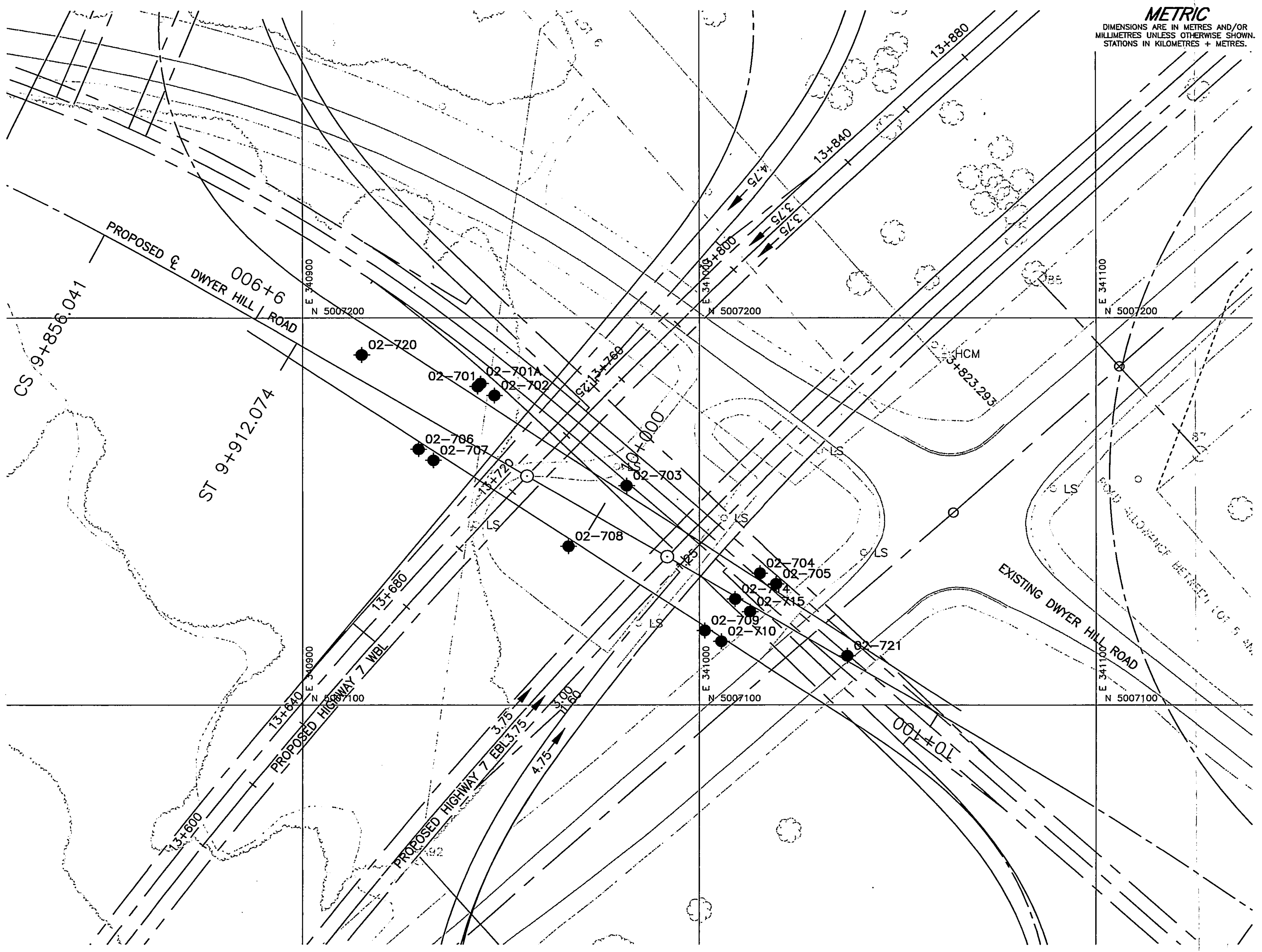
CONT No.
WP No. 251-99-00

SHEET

HIGHWAY 7 TWINNING
DWYER HILL ROAD UNDERPASS
BOREHOLE LOCATIONS

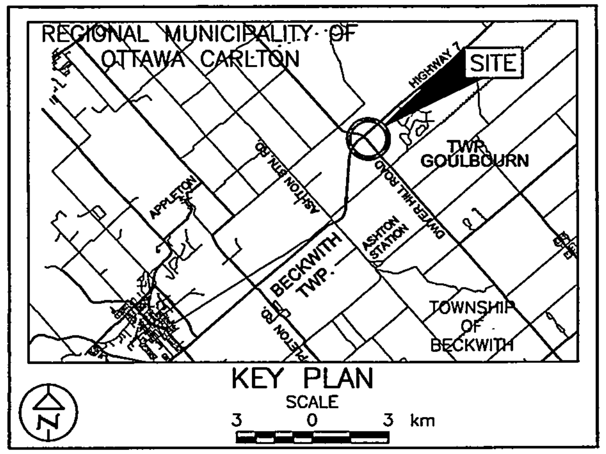
Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA


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

PLAN

SCALE
10 0 10 20 m



KEY PLAN

SCALE
3 0 3 km

LEGEND

● Borehole – Current Investigation

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
02-701	133.0	5007182.3	340944.0
02-701A	133.0	5007183.1	340944.7
02-702	132.8	5007180.0	340948.3
02-703	134.0	5007156.7	340981.7
02-704	132.9	5007134.0	341015.3
02-705	132.9	5007131.3	341019.4
02-706	132.4	5007166.1	340929.2
02-707	132.5	5007163.2	340932.9
02-708	134.0	5007141.0	340967.0
02-709	132.8	5007119.2	341001.3
02-710	132.7	5007116.4	341005.6
02-714	132.8	5007127.4	341009.0
02-715	132.8	5007124.1	341012.8
02-720	132.8	5007190.5	340914.9
02-721	133.3	5007112.7	341037.4

NOTES

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

NO.	DATE	BY	REVISION

Geocres No.

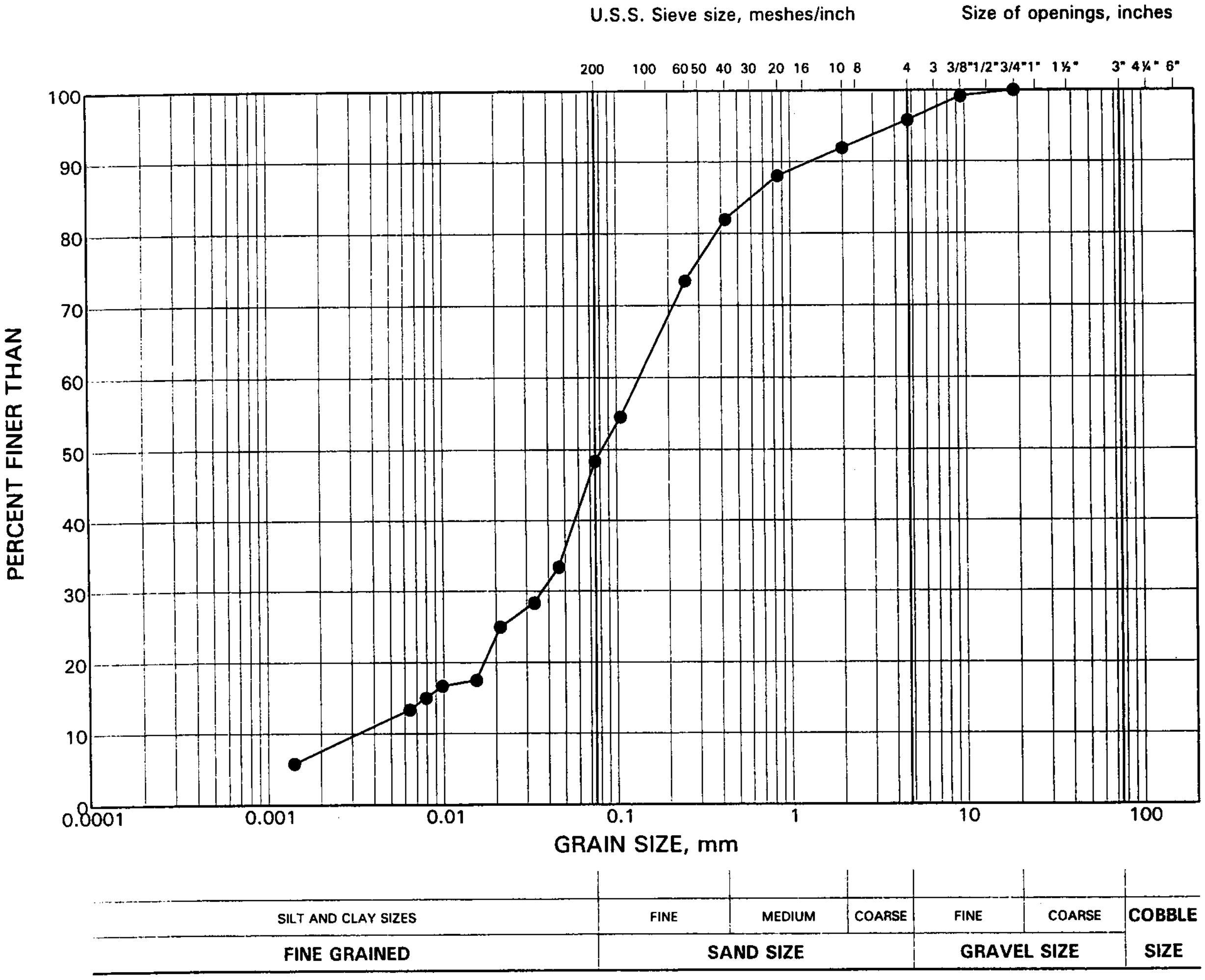
HWY. 7		PROJECT NO. 021-1155	DIST.
SUBM'D. MIC	CHKD. LCC	DATE: AUG., 2004	SITE: 3-720
DRAWN: JDR	CHKD. MIC	APPD. LCC	DWG. 1

PLOT DATE: August 18, 2004
FILENAME: T:\Projects\2002\021-1155\REPORT-7\0211155-7-001.dwg

GRAIN SIZE DISTRIBUTION TEST RESULT

Fill

FIGURE 1

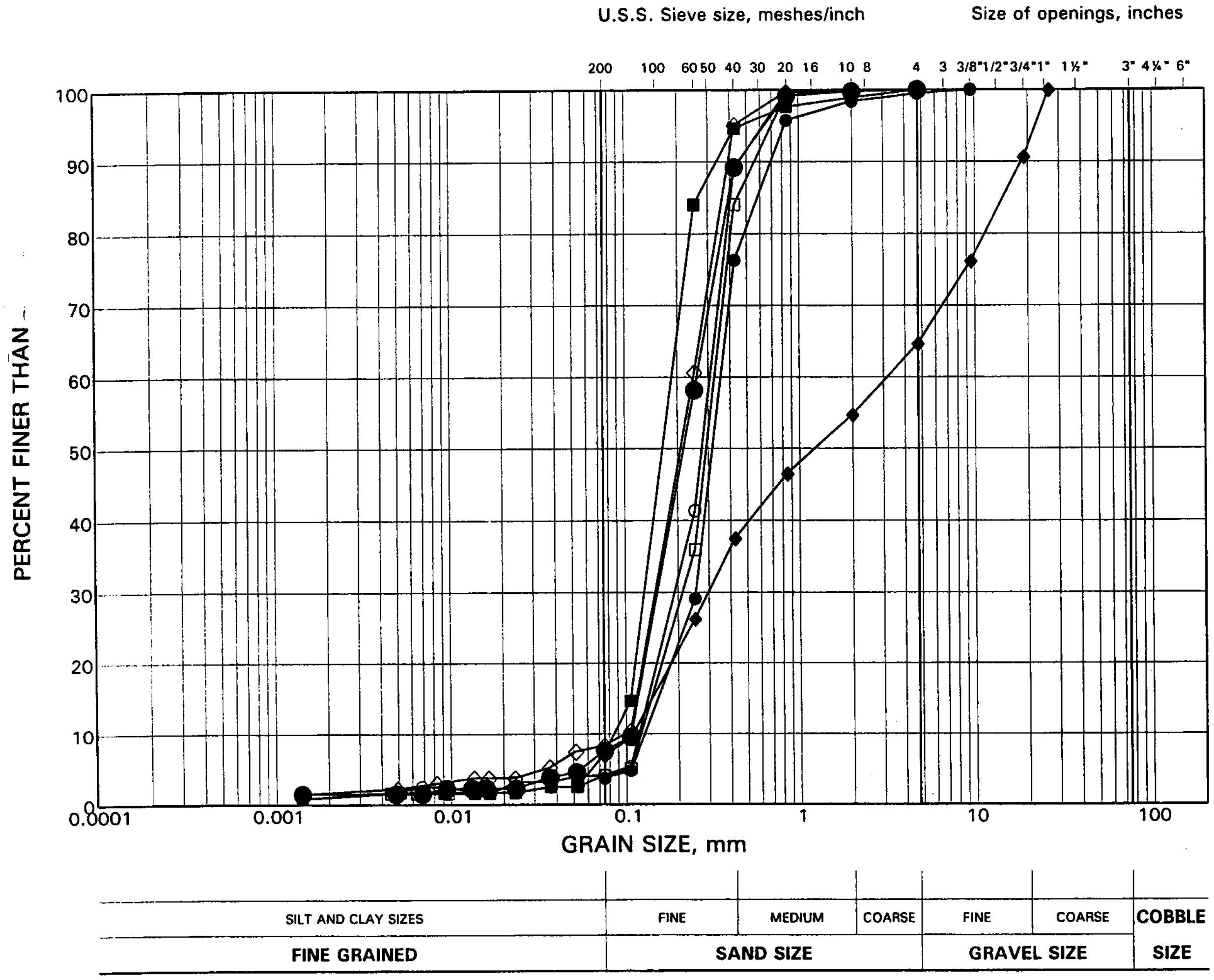


LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	02-703	1	133.0

GRAIN SIZE DISTRIBUTION TEST RESULTS
Surficial Silty Sand to Sand and Gravel

FIGURE 2

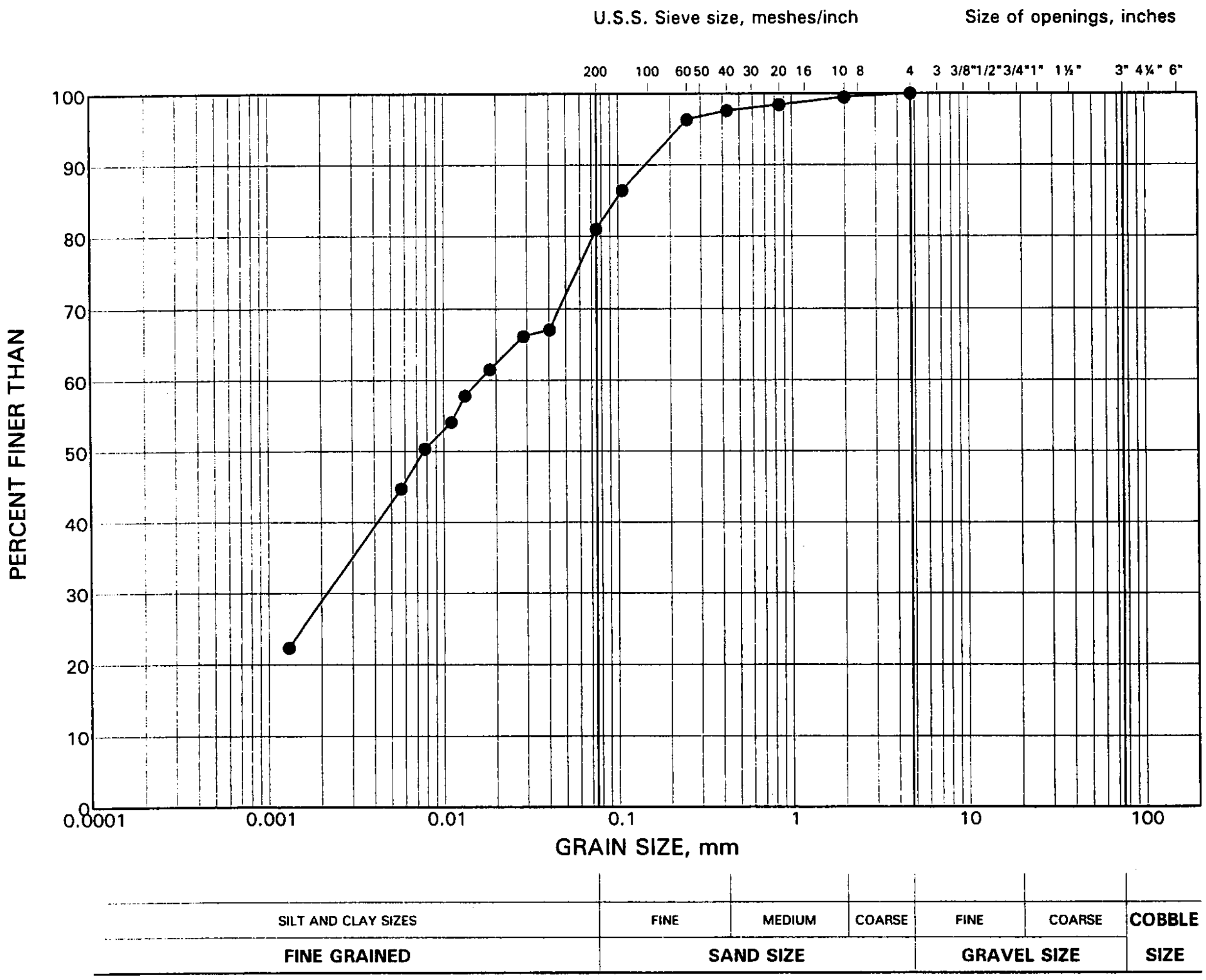


LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
•	02-701	1	132.0
■	02-702	2	131.2
◆	02-702	3	130.5
○	02-706	1	131.4
□	02-707	1	131.5
◇	02-720	2	131.8
●	02-720	3	131.3

GRAIN SIZE DISTRIBUTION TEST RESULT
Silty Clay

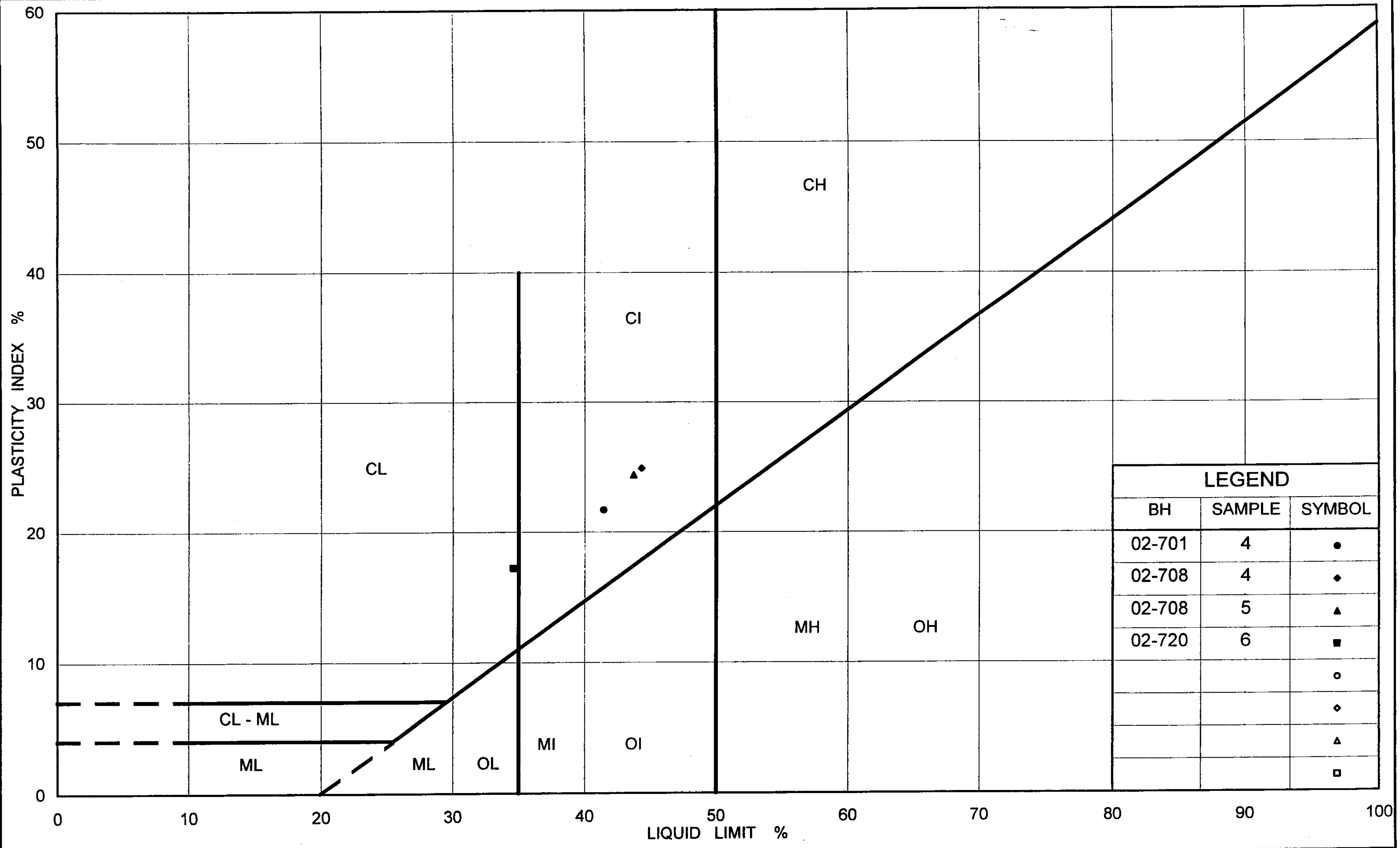
FIGURE 3



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	02-701	3	130.2

Oct 75, FF-S-21



LEGEND		
BH	SAMPLE	SYMBOL
02-701	4	●
02-708	4	◆
02-708	5	▲
02-720	6	■
		○
		◇
		△
		□

Ministry of Transportation

Ontario

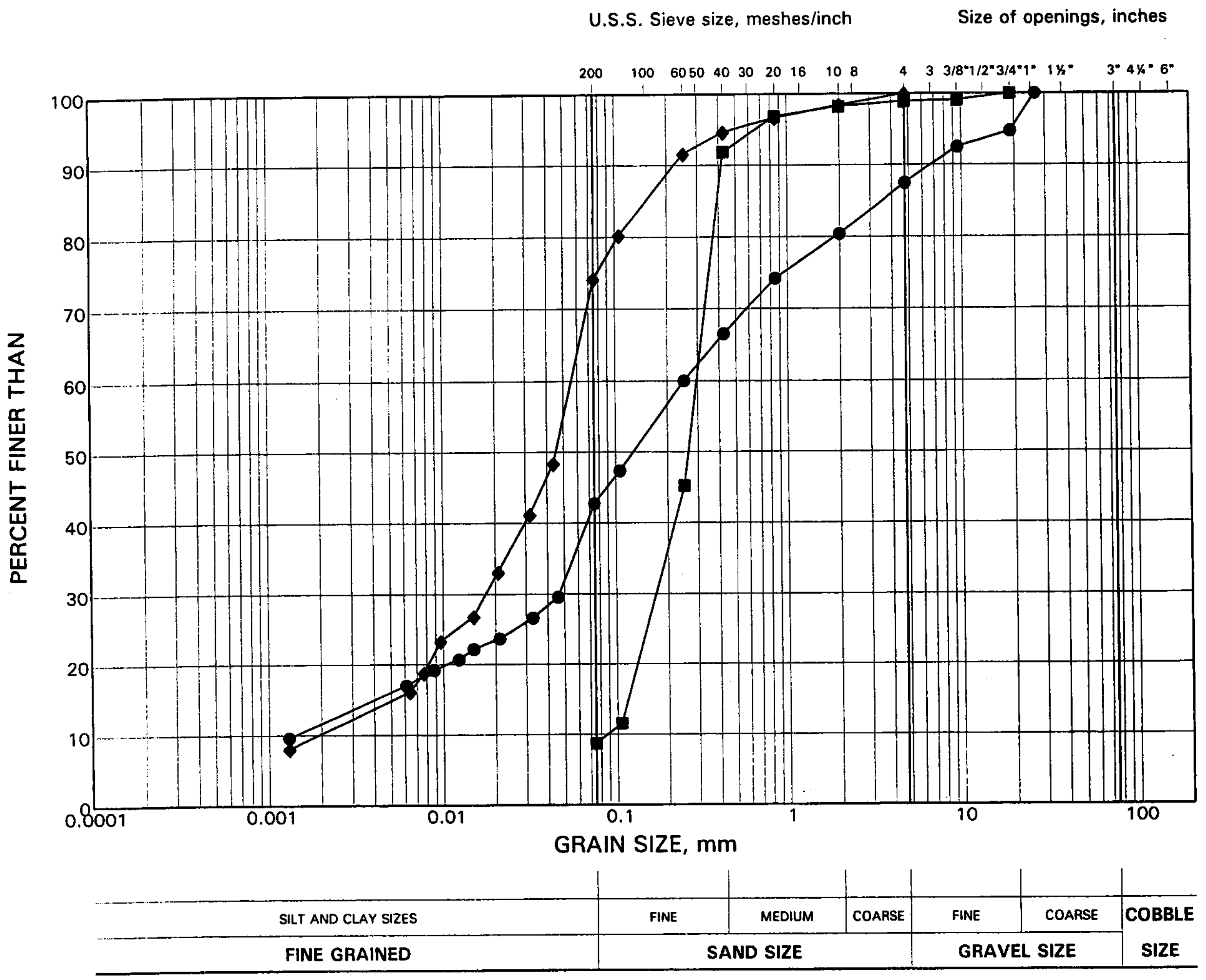
PLASTICITY CHART
Silty Clay

FIG No. 4

Project No. 021-1155

GRAIN SIZE DISTRIBUTION TEST RESULTS
Sand and Silt Till (Including Interlayer)

FIGURE 5



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	02-701	6	128.3
■	02-704	3	131.1
◆	02-705	3	131.2