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**FOUNDATION
INVESTIGATION AND DESIGN REPORT
APPLETON SIDEROAD (COUNTY ROAD 17)
UNDERPASS (STRUCTURE SITE 15-225)
HIGHWAY 7 TWINNING FROM 3 KM WEST OF
JINKINSON ROAD TO CARLETON PLACE
W.P. 252-99-00**

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PART A

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W.P. 252-99-00**

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder Associates) has been retained by Marshall Macklin Monaghan (MMM) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations associated with the twinning of Highway 7 from two to four lanes in 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.

Foundation investigation services are required for the following components:

- **W.P. 256-99-00:** New structures at the Highway 417E-7W ramp and Hazeldean Road, including a high fill embankment along the Highway 417E-7W ramp, high mast light poles, and overhead signs.
- **W.P. 251-99-00 and 252-99-00:** Five new structures at Appleton 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 new Appleton Sideroad (County Road 17) 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. Further 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 proposed Appleton Sideroad (County Road 17) underpass structure is located approximately 3.5 km east of Carleton Place in Beckwith Township, in Lanark County. It is noted that north of Highway 7, County Road 17 is called Appleton Sideroad, while south of Highway 7, County Road 17 is called Cemetery Road. The proposed underpass structure is designated as MTO's Structure Site 15-225.

The terrain in the vicinity of the site is relatively flat, with the natural ground surface varying from about Elevation 129 m to 130 m, generally declining slightly toward the north. The existing Highway 7 grade at the proposed structure location is at or slightly above Elevation 130 m, slightly above the surrounding natural grade. Ditches between 1.0 m and 1.5 m in depth are present along both sides of Highway 7.

The proposed site of the underpass structure is located about 30 m to 50 m east of the current intersection of County Road 17 and Highway 7. North of Highway 7, Appleton Sideroad will extend in-line with the alignment of the current intersection. South of Highway 7, Cemetery Road will be offset about 30 m to 50 m east of the existing alignment; removing the existing "S"-curve in the alignment of Cemetery Road, immediately south of Highway 7. An existing park-and-ride facility lies on the south side of Highway 7, in-line with Cemetery Road, and the "S" curve runs along the west side of that facility. As presently proposed, the underpass structure will be located along the northerly extension of the Cemetery Road alignment, some 30 m to 50 m east of the present intersection.

The existing lanes of Highway 7 will be maintained and will form the new westbound lanes. To the south of the existing lanes, the area of the proposed structure and the south approach is occupied by an existing park-and-ride facility and the entrance roadway to the parking lot. There is an existing retail fuel outlet in the north approach area which includes a small building, a canopy, and several fuel pumps, which will be demolished.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out for the proposed Appleton Sideroad underpass structure in which a total of fourteen boreholes were drilled in two phases, first in May 2003 and then subsequently in July 2003. For the first phase, ten boreholes (Boreholes 02-801 to 02-810) were advanced at the proposed foundation elements and two boreholes (Boreholes 02-820 and 02-821) were advanced at the north and south approach embankments, respectively. For the second phase of the investigation, two additional boreholes (Boreholes 02-820A and 02-820B) were advanced at the north approach embankment, adjacent to Borehole 02-820, to carry out supplementary in situ testing and sampling in the silty clay deposit. All of the boreholes were advanced using hollow stem augers by a bombardier-mounted drill rig, supplied and operated by Marathon Drilling Ltd. of Ottawa, Ontario.

With the exception of Borehole 02-820 within the footprint of the north approach embankment, all of the boreholes for the first phase of the investigation were advanced to auger refusal which occurred at depths between 5.4 m and 6.1 m below the existing ground surface at the borehole locations; Borehole 02-820 was terminated at about 6.7 m depth without encountering auger refusal. Samples of the overburden were obtained at 0.75 m intervals of depth using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure. In six of the ten boreholes advanced at the proposed foundation locations, the boreholes were advanced about 3 m into the bedrock by coring using NQ-size coring equipment. The water level in the open boreholes was observed throughout the drilling operations and a total of four piezometers were installed to monitor the groundwater level(s) at the site. For the second phase of the investigation, Borehole 02-820A included in situ vane testing in the silty clay (using a B-sized vane due to the very stiff consistency of the deposit) at approximately 0.3 m intervals to a depth of about 6.1 m. Borehole 02-820B was advanced directly (i.e., without sampling or in situ testing) to about 4.6 m depth, at which depth one relatively undisturbed 73 mm diameter Shelby tube sample of the silty clay was retrieved using a fixed-piston sampler.

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 laboratory 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. Laboratory oedometer consolidation testing was carried out on one sample of the silty clay deposit, and also included a sustained load test to assess the secondary compression behaviour of the deposit. Three samples of the bedrock were also submitted to SGS Lakefield Research Limited of Lakefield, Ontario, for chemical ("whole rock") analysis.

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

<i>Borehole Number</i>	<i>Borehole Location</i>	<i>MTM NAD83 Northing (m)</i>	<i>MTM NAD83 Easting (m)</i>	<i>Ground Surface Elevation (m)</i>
02-801	North abutment	5001259.2	336943.7	129.5 m
02-802	North abutment	5001252.6	336946.0	129.3 m
02-803	Centre pier	5001225.8	336976.3	129.5 m
02-804	South abutment	5001200.5	337000.7	130.3 m
02-805	South abutment	5001196.7	337003.4	130.4 m
02-806	North abutment	5001242.9	336927.4	129.4 m
02-807	North abutment	5001238.9	336930.4	129.3 m
02-808	Centre pier	5001211.6	336959.1	129.5 m
02-809	South abutment	5001185.9	336984.5	129.8 m
02-810	South abutment	5001182.5	336987.4	129.8 m
02-820	North approach	5001260.2	336915.7	129.7 m
02-820A	North approach	5001259.5	336915.0	129.7 m
02-820B	North approach	5001258.8	336914.3	129.7 m
02-821	South approach	5001173.5	337008.2	130.1 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 is characterized by shallow overburden deposits overlying limestone bedrock of the Ottawa Formation; this formation consists of grey limestone with some shaly partings and seams.² The shallow overburden soils are typically between 1 m and 3 m in thickness and are commonly comprised of sandy to gravelly till derived from the Precambrian Shield to the north, overlain by glaciofluvial sediments that consist of layered sands and gravels. Large areas of the plain are covered with peat and muck, due to poor drainage as a consequence of the relatively flat topography and shallow depth to bedrock.¹

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

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

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

4.2 Site Stratigraphy

As part of the subsurface investigation at this site, twelve boreholes were advanced within the limits of the foundation elements and immediate approach embankments for the proposed underpass structure. The borehole locations and ground surface elevations are shown on Drawing 1.

The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of in-situ and laboratory testing are given on the Record of Borehole sheets and Figures 1 to 8. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

In summary, the soils encountered immediately below ground surface at this site generally consist of existing surficial fill materials associated with the existing Highway 7, park-and-ride lot and retail fuel outlet, and topsoil, overlying about 0.7 m to 1.0 m of layered clayey silt and silty clay, over an approximately 3.6 m to 4.6 m thick deposit of silty clay to clay. The upper 2 m to 3 m of this clay deposit have been weathered to a grey brown crustal zone while the underlying portions of the deposit are grey in colour. Below a depth of about 5.2 m to 6.3 m, the silty clay to clay is underlain by about 0.1 m to 0.8 m of sandy silt till. These soils are, in turn, underlain by interlayered limestone, dolomitic limestone, and sandstone bedrock that was encountered between about 5.4 m and 6.1 m depth (at about Elevation 123.2 m to 124.4 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

Fill materials were encountered in all of the boreholes with the exception of Borehole 02-807. However, the variation in type and thickness of the fill materials reflects the different land uses across the site, associated with the retail fuel outlet, the existing lanes of Highway 7, and the park-and-ride lot.

In the area of the proposed north abutment, located in the landscaped area between the retail fuel outlet and the existing lanes of Highway 7, Boreholes 02-801, 02-802, and 02-806 encountered 150 mm to 300 mm of topsoil fill at ground surface. No fill material was encountered in Borehole 02-807 in that same area.

At the proposed pier, essentially within the ditch on the south side of the existing Highway 7 lanes, Boreholes 02-803 and 02-808 encountered about 0.5 m of fill material at ground surface, consisting of both topsoil fill as well as sand and gravel.

In the area of the proposed south abutment, Boreholes 02-804 and 02-805 penetrated the pavement surface in the park-and-ride lot and encountered approximately 90 mm of asphalt overlying crushed stone granular base to about 0.5 m depth, overlying about 0.7 m to 1.0 m of mixed crushed stone and silty clay fill material, with cobbles, and trace amounts of plastic and organic matter. The result of grain size distribution testing carried out on one sample of this material from Borehole 02-804 is provided on Figure 1. Two Standard Penetration Test (SPT) "N" values measured within this fill of 14 blows per 0.3 m of penetration, indicate that the fill is compact. Boreholes 02-809 and 02-810 were put down within the landscaped area to the west of the park-and-ride lot and encountered about 150 mm of topsoil fill overlying sand and gravel fill to about 0.6 m depth.

Borehole 02-821, put down within the footprint of the future south approach embankment and within the landscaped area to the south of the park-and-ride lot, encountered about 120 mm of topsoil fill overlying sand and gravel fill to about 0.9 m depth.

Borehole 02-820, put down within the footprint of the future north approach embankment and within the paved area around the pumps of the existing retail fuel outlet, encountered about 90 mm of asphalt overlying about 60 mm of crushed stone base, over about 0.7 m of sand and gravel subbase.

4.2.2 Topsoil

Topsoil was encountered in all of the boreholes put down at the proposed foundation elements (i.e., Boreholes 02-801 to 02-810). In Borehole 02-807, the topsoil layer exists at ground surface. In the remaining foundation boreholes, the topsoil layer is buried beneath the surficial fill material. Topsoil also exists beneath the fill material in Borehole 02-821 at the south approach embankment. Where present, the topsoil ranges in thickness from about 90 mm to 390 mm, averaging 220 mm.

4.2.3 Layered Clayey Silt and Silty Clay

The fill material and/or topsoil are underlain at all borehole locations by layered clayey silt and silty clay with sand seams. This deposit ranges from about 0.7 m to 1.0 m in thickness. The results of grain size distribution testing carried out on two samples of this material (from Boreholes 02-808 and 02-809) are provided on Figure 2.

Measured Standard Penetration Test (SPT) "N" values in this deposit range from 4 to 9 blows per 0.3 m of penetration. Based on past experience in this area, this range of "N" values indicates a generally very stiff consistency.

4.2.4 Silty Clay to Clay

The layered clayey silt and silty clay are underlain by a deposit of silty clay to clay which extends to depths ranging from 5.2 m to 6.3 m below ground surface (Elevation 123.5 m to 124.6 m).

The upper 2.0 m to 2.9 m of this deposit have been weathered to a grey brown crust. The results of grain size distribution testing carried out on one selected sample of this material are provided on Figure 3. Measured SPT "N" values in this portion of the deposit range from 6 to 15 blows per 0.3 m of penetration, generally decreasing with depth. The results of in situ vane testing carried out within the weathered crust in Borehole 02-820A indicate undrained shear strengths in excess of 130 kPa. These in situ test results indicate the weathered crust to have a very stiff consistency. The results of Atterberg limit testing on selected samples of the weathered crust indicate plasticity index values ranging from 28 to 33 per cent and liquid limit values ranging from 51 to 57 per cent. These results are summarized on the plasticity chart on Figure 4 and indicate this material to be a clay of high plasticity. The measured natural water content of samples of the weathered crust ranges from 33 to 42 per cent.

The silty clay to clay below the depth of weathering is grey in colour. The results of grain size distribution testing carried out on one selected sample of this material are provided on Figure 5. Measured SPT "N" values in this deposit range from 4 to 8 blows per 0.3 m of penetration. The results of in situ vane testing carried out in Borehole 02-820A indicate undrained shear strengths generally in excess of 130 kPa, with one measured value of about 115 kPa. These results indicate the grey silty clay to clay deposit to have a generally very stiff consistency. The results of Atterberg limit testing on selected samples indicate plasticity index values ranging from 23 to 31 per cent and liquid limit values ranging from 45 to 53 per cent. These results are summarized on the plasticity chart on Figure 6 and indicate this material to be of intermediate to high plasticity. The measured natural water contents of samples of the grey silty clay to clay range from 29 to 41 per cent.

Oedometer consolidation testing was carried out on one thin-walled Shelby tube sample of the deposit. The results of that testing are provided on Figure 7 and are summarized in the table below.

Borehole/ Sample Number	Sample Depth/Elev. (m)	Unit Weight (kN/m ³)	σ_p' (kPa)	σ_{vo}' (kPa)	Cc	Cr	e_o	OCR	Cv (cm ² /s)
02-820B / 1	5.0 / 124.7	19.4	700	40	0.37	0.012	0.79	18	0.01

NOTES:

- σ_p' - Apparent preconsolidation pressure
- σ_{vo}' - Computed existing vertical effective stress
- Cc - Compression index
- Cr - Recompression index
- e_o - Initial void ratio
- OCR - Overconsolidation ratio
- Cv - Coefficient of consolidation

Following completion of this test, another specimen was taken from the same Shelby tube sample and loaded to a stress of 280 kPa (the approximate estimated average final stress level for the deposit) in the oedometer consolidation test machine. That stress level was maintained for a period of approximately 20 days, well past the end of primary consolidation, to assess the secondary compression behaviour of the sample. The results of that testing are summarized on Figure 7c and indicate a secondary compression index value of 0.0038

4.2.5 Silty Sand Till to Sandy Silt Till

The silty clay deposit is underlain by glacial till, which ranges in thickness from 0.1 m to 0.8 m at those locations where the deposit was fully penetrated and/or before auger refusal was encountered. The results of grain size distribution carried out on one sample of the glacial till matrix recovered from Borehole 02-808 are provided on Figure 8. From these results as well as observations of the drilling resistance, the glacial till is considered to consist of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of silty sand to sandy silt with a trace of clay.

Due to the limited thickness of this deposit, only limited standard penetration testing could be carried out before sampler refusal was encountered on the bedrock surface. However three measured SPT "N" values of between 17 and 25 blows and one measurement of 58 blows per 0.3 m of penetration indicate the deposit to be compact to very dense, although the one higher SPT "N" value may reflect the presence of cobbles in the deposit, rather than the state of packing of the matrix.

The base of this till deposit was encountered between about Elevations 123.2 m and 124.5 m in the boreholes (at depths below ground surface ranging from 5.4 to 6.1 m), except in Borehole 02-820 which was terminated in the till at about 6.7 m depth.

4.2.6 Limestone and Dolomitic Limestone Bedrock

Interlayered limestone and dolomitic limestone bedrock, locally containing a bed of sandstone bedrock, underlies the till deposit at this site. In the boreholes put down at the proposed bridge foundations, the surface of the bedrock was encountered between Elevation 123.2 m and 124.2 m. The following table summarizes the bedrock surface depth and elevation as encountered at the borehole locations. It should be noted that bedrock was cored in six of the boreholes; the surface of the limestone bedrock was inferred in the six remaining boreholes by refusal to auger advance.

<i>Borehole Location</i>	<i>Borehole Number</i>	<i>Ground Surface Elevation</i>	<i>Depth to Bedrock</i>	<i>Bedrock Surface Elevation</i>
North approach	02-820	129.7 m	Bedrock surface not encountered	
North abutment	02-801	129.5 m	5.7 m	123.8 m (Cored)
	02-802	129.3 m	6.1 m	123.2 m
	02-806	129.4 m	5.6 m	123.8 m
	02-807	129.3 m	5.4 m	123.9 m (Cored)
Centre pier	02-803	129.5 m	6.1 m	123.4 m (Cored)
	02-808	129.5 m	6.0 m	123.5 m (Cored)
South abutment	02-804	130.3 m	6.1 m	124.2 m (Cored)
	02-805	130.4 m	6.1 m	124.3 m
	02-809	129.8 m	5.6 m	124.2 m
	02-810	129.8 m	5.6 m	124.2 m (Cored)
South approach	02-821	130.1 m	5.7 m	124.4 m

Overall, the natural bedrock sequence is inferred to consist of dolomitic limestone bedrock overlying a layer of sandstone (about 120 mm to 600 mm thick), overlying limestone and dolomitic limestone. Using the sandstone layer as a marker bed, (absent from the boreholes at the south abutment, present at the bedrock surface at the centre pier, and located beneath the upper limestone layer at the north abutment) the stratigraphic sequence is inferred to be dipping to the north.

Three samples of bedrock core from borehole 02-801 were submitted to SGS Lakefield Research Limited of Lakefield, Ontario, for chemical ("whole rock") analysis. The results of that testing are provided in Appendix A, with the sample designations, sample depths, and SiO₂ contents being as follows:

<i>Sample Identification</i>	<i>Depth Interval (m)</i>	<i>Description</i>	<i>SiO₂ Content</i>
02-801A	6.4 – 6.6	Upper dolomitic limestone	61.6 %
02-801B	7.5 – 7.7	Sandstone	75.9 %
02-801C	8.3 – 8.4	Lower domomitic limestone	3.1 %

The results indicate relatively high silica contents (from the measured SiO₂ contents) for both the upper dolomitic limestone as well as the sandstone layer.

The interlayered limestone and dolomitic limestone bedrock at the site is a member of the Rockcliffe Formation; it is generally medium strong and thinly- to medium-bedded. Rock Quality Designation (RQD) values measured on recovered bedrock core samples typically ranged from 32 to 92 per cent, although isolated intervals were observed to have RQD values of 0 per cent. The discontinuities observed in the rock core are typically horizontal to sub-horizontal, associated with the bedding planes, although some vertical to sub-vertical jointing was also observed. A description of some of the terms used in the description of the bedrock samples from this site is provided on the *Lithological and Geotechnical Rock Description Terminology* sheet which precedes the Record of Borehole sheets included with this report.

4.3 Groundwater Conditions

Four piezometers were installed within the overburden soil deposits at this site. The water levels measured in the piezometers ranged from about Elevation 129.2 m to 128.7 m, generally declining from north to south, as summarized in the following table:

Borehole No.	Borehole Location	June 6, 2003		Aug. 1, 2003	
		Depth	Elevation	Depth	Elev.
02-801	North abutment	0.3 m	129.2 m	0.3 m	129.2 m
02-803	Centre pier	0.4 m	129.1 m	0.5 m	129.0 m
02-809	South abutment	0.6 m	129.2 m	1.0 m	128.8 m
02-821	South approach	1.0 m	129.1 m	1.4 m	128.7 m

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

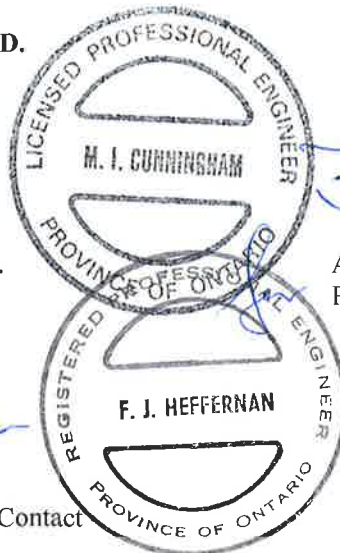
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PART B

**FOUNDATION DESIGN REPORT
APPLETON SIDEROAD (COUNTY ROAD 17)
UNDERPASS (STRUCTURE SITE 15-225)
HIGHWAY 7 TWINNING FROM 3 KM WEST OF
JINKINSON ROAD TO CARLETON PLACE
W.P. 252-99-00**

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides foundation design recommendations for the proposed Appleton Sideroad 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 from the Structural Planning Report prepared by Totten Sims Hubicki (TSH), dated August 2002, that the proposed Appleton Sideroad underpass structure will be two spans, with a central pier to be located within the future median, south of the existing Highway 7 alignment. Three alternative integral or semi-integral abutment configurations, which eliminate the requirement for expansion joints, were considered during the preliminary structural design stage, as follows:

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

5.2 Bridge and Retaining Wall Foundation Options

In the immediate vicinity of the proposed Appleton Sideroad underpass structure, the existing ground surface is at about Elevation 129 m to 130 m, and the existing Highway 7 grade is slightly above Elevation 130 m. Based on the information contained in Totten Sims Hubicki's *Update to the Preliminary Design Study (Highway Engineering)*, dated June 2002, the proposed Appleton Sideroad grade at the structure site is at about Elevation 138 m, but that will need to be confirmed by the designer responsible for the detailed design of this structure. The south approach embankment will be approximately 8 m high relative to the existing natural grade at the site, while the north approach embankment will be about 9 m to 10 m high.

In summary, the soils encountered immediately below ground surface at this site generally consist of existing surficial fill materials, overlying topsoil, overlying about 0.7 m to 1.0 m of layered clayey silt and silty clay, and then by an approximately 3.6 m to 4.6 m thick deposit of silty clay to clay overlying a thin deposit of sandy silt till. These overburden soils are underlain by medium strong, interlayered limestone, sandstone, and dolomitic limestone bedrock, the surface of which was encountered in the boreholes between Elevations 123.2 m and 124.4 m, about 5.4 m to 6.1 m below the existing grade at the site.

The overburden soils are not considered suitable for support of the proposed pier, abutments and associated retaining walls, such as concrete cantilever retaining walls, on shallow foundations placed on the overburden soils. The pier will need to be supported on deep foundations bearing on the bedrock. The use of shallow foundations "perched" on compacted Granular 'A' pad within the approach embankment fill is feasible for the abutment if the embankment fills are constructed well in advance of the abutment construction to allow the consolidation of the overburden soils to occur.

The overburden soils at the site are also not considered to be suitable for the support of RSS walls, either as wingwalls or in front of the abutments, due to the potential for consolidation settlement and longer-term secondary compression of the silty clay deposit unless adequate preloading can be carried out to reduce the differential settlement to within tolerable limits.

Since integral abutments are under consideration, steel H-piles are considered to be the most appropriate foundation type for support of the abutments (and as assumed in the *Structural Planning Report* prepared by TSH). Given the proposed Appleton Sideroad grade of about Elevation 138 m, an assumed underside of pile cap at about Elevation 135 m for the abutments, and the bedrock surface at the abutment locations at about Elevation 123.2 m to 124.3 m, it is estimated that the pile length will be approximately 11 m to 12 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.

As an alternative to steel H-pile foundations, caisson foundations resting on or socketted into the limestone bedrock could be used for support of the abutments and centre pier. However, this would preclude use of integral abutments at this site.

Recommendations for the geotechnical design of the pier, bridge abutment and associated retaining wall foundations are presented in the following sections. A summary comparison of the advantages, disadvantages, relative costs, and risks associated with the foundation options is presented in Table 1 following the text of this report.

5.3 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 Appleton Sideroad grade at about Elevation 138 m and the assumed pile cap base at Elevation 135 m, the pile length will be approximately 11 m to 12 m without socketting into bedrock. For the center pier, it is estimated that the piles would be about 5 m long.

If necessary to resist seismic forces, the piles could be placed/socketted within the bedrock. The limestone / dolomitic limestone bedrock is generally medium strong, however, and this would require socket formation using coring or churn drilling to advance the hole. Further, at least the upper portions of the limestone / dolomitic limestone as well as the sandstone layer present at the centre pier and north abutment have high silica contents, are considered to be rather abrasive, and could result in relatively high equipment wear.

5.3.1 Axial Geotechnical Resistance

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

In the case of the driven piles, it is assumed that the piles would be driven after construction of the approach embankment to the base of pile cap level. Embankment fill materials which are within the area of the piling and placed in advance of pile driving should be free of materials greater than 75 mm in size. Consideration must also be given to the presence of cobbles and boulders within the thin layer of glacial till which underlies the silty clay deposit at this site. In this regard, vertically driven piles should be equipped with flange reinforcement (driving shoes) as per OPSD 3301.00. Any battered piles should be equipped with suitable driving points (such as Titus Ejector or equivalent) to ensure adequate seating of the piles on the bedrock.

Pile installation should be in accordance with SP903S01. 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.

It should be noted that the construction of the approach embankments will raise the effective stress level in the grey silty clay leading to some compression of the deposit. As discussed subsequently in Section 5.7.3 of this report, the embankment subgrade settlements are estimated at about 30 mm (from both primary consolidation and secondary compression). The elastic shortening of the piles would be expected to be significantly less, at about 5 mm under service loads, and therefore the differential settlements would be sufficient to generate downdrag forces.

In calculating the magnitudes of the downdrag force, the methods described in both the Canadian Foundation Engineering Manual as well as the US Transportation Research Board report titled "Design and Construction Manual For Downdrag on Uncoated and Bitumen-Coated piles" [Briaud and Tucker (1994)] were considered. Considering the larger predicted settlement of the silty clay deposit versus the elastic shortening of the pile, the neutral plane used in those analyses was assumed to be at the underside of the silty clay deposit.

Based on the above, the unfactored downdrag load acting on a single HP 310 x 110 pile over the length of pile within the native soils is estimated to be 400 kN. 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. The structural capacity of the piles must be checked for the factored dead and downdrag loads in accordance with Section 6.8.4 of the CHBDC.

Alternatively, in view of the results of the oedometer consolidation testing which indicate a relatively high coefficient of consolidation for this deposit and therefore a relatively rapid predicted settlement (with about 90 per cent of the consolidation settlement expected to occur within a period of about 3 to 6 months, as discussed subsequently in Section 5.7.3 of this report), consideration could be given to eliminating the downdrag forces by preloading the foundation soils and allowing the settlements to occur prior to driving the piles. For this option, it is considered that the embankment would be constructed to its full design geometry plus an additional surcharge (of about 1 to 1.5 m), allowed to remain in place for six months while the settlements are monitored and until the settlements are largely completed, the surcharge removed, and the piles installed after re-excavating to the base of the pile cap. The surcharge is required in order to avoid/reduce future post-construction settlements and downdrag forces that could occur due to further consolidation, secondary compression, or future higher stress levels such as might occur due to groundwater level fluctuations or moisture content and weight gain of the embankment fill.

5.3.2 Resistance to Lateral Loads

Lateral loading could be resisted fully or partially by the use of battered steel H-piles. In the case of battered piles, precautions during driving are necessary in some situations (such as for specific soil/bedrock conditions/pile lengths and where the batter is shallower than 6 vertical to 1 horizontal) to ensure that the piles do not deflect along the bedrock surface even with relatively flat lying bedrock.

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, as described by Terzaghi (1955) and the Canadian Foundation Engineering Manual (3rd Edition).

For cohesionless soils:

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

n_h is the constant of horizontal subgrade reaction, as given below
 z is the depth (m)
 B is the pile diameter/width (m)

For cohesive soils:

$$k_h = \frac{k_{s1}}{5B} \quad \text{where}$$

B is the pile diameter/width (m)
 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. The range in values reflects the variability in the subsurface conditions as well as the two extremes of design: the requirement for flexibility in the case of integral abutments and the requirement for lateral support in the case of non-integral abutments and the pier.

<i>Soil Unit</i>	<i>n_h</i>	<i>k_{sl}</i>
Embankment fill (assumed to be compacted granular fill) above approximately Elev. 129 m	5 to 15 MPa/m	—
Layered clayey silt and silty clay deposit and weathered clay crust between Elev. 129 m (approximately native subgrade level) and Elev. 126 m – approximately 3 m thick	—	45 to 50 MPa/m
Grey silty clay below Elevation 126 m: South abutment: Approximately 1.8 m thick between Elev. 126 m and bedrock surface at about Elev. 124.2 m Centre pier: Approximately 2.5 m thick between Elev. 126 m and bedrock surface at about Elev. 123.5 m North abutment: About 2.1 m to 2.8 m thick between Elev. 126 m and bedrock surface at about Elev. 123.2 m to 123.9 m (refer to Records of Boreholes)	—	30 to 50 MPa/m

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

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

5.3.3 Frost Protection

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

5.4 Caisson Foundations

Caissons founded on or socketted into the limestone / dolomitic limestone bedrock (including the sandstone bed within the bedrock) may be used for support of the abutments and pier. 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 Appleton Sideroad grade at Elevation 138 m and the assumed pile cap base at Elevation 135 m, the length of caissons used for abutment support will be approximately 11 m to 12 m, not including the rock socket length, if provided. For the center pier, it is estimated that the caissons would be about 5 m long, not including the length of the rock socket, if provided.

It is noted that the native marine (Champlain Sea) clay at this site is a sensitive soil. The disturbed clay could "flow" into the auger hole during caisson installation if left unsupported. The use of a temporary liner or casing will be required in order to advance the caissons with minimal loss of ground.

The 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. Further, at least the upper portions of the limestone / dolomitic limestone as well as the sandstone layer present at the centre pier and north abutment have high silica contents, are therefore considered to be rather abrasive, and could result in relatively high equipment wear.

5.4.1 Axial Geotechnical Resistance

Caissons founded on the surface of the limestone bedrock, or socketted nominally (less than 1 m) into the bedrock, should be designed based on end-bearing resistance and a factored geotechnical resistance at ULS of 4 MPa should be used. Serviceability Limit States resistances do not apply to caissons founded on the limestone bedrock, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

It should be noted that the construction of the approach embankments will raise the effective stress level in the grey silty clay deposit at depth close to its estimated preconsolidation pressure. That stress increase will lead to some compression of the deposit and resulting downdrag forces on caissons supporting the abutments and associated retaining walls. The unfactored downdrag load acting on a single 1.5 m diameter caisson over its length is estimated to be 3,700 kN. The structural capacity of the caissons must be checked for the factored dead and downdrag loads in accordance with Section 6.8.4 of the *CHBDC*. The assumptions and methods used in assessing that downdrag force are the same as those described in Section 5.3.1 of this report. The guidelines provided in that section of the report for reducing or eliminating the downdrag forces are equally applicable to this foundation option.

5.4.2 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 5.3.2.

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 Geotechnical Resistance for "Perched" Footings

Spread footings for the abutments may be placed on a compacted Granular "A" pad constructed within the approach embankment fill. Based on the existing and proposed grades at this site, the Granular "A" pad constructed from existing grade to the underside of footing is expected to be at least 4 m in thickness. A factored geotechnical resistance at ULS of 750 kPa may be used for design, assuming that the subgrade is properly prepared prior to fill placement and that the Granular "A" pad is placed in regular lifts, and compacted to 100 per cent of the material's Standard Proctor maximum dry density. The geotechnical resistance at SLS will be governed by the consolidation settlement of the overburden soils under the embankment loading. As discussed in Section 5.8.3, primary consolidation settlement is estimated at about 25 mm. Preloading of the abutment and approach area would be required for a duration of at least six months in order to minimize the long-term settlement. With this preloading, a geotechnical resistance at SLS of 350 kPa may be assumed for design.

The geotechnical resistances given above may be used for design of retaining wall footings on the Granular "A" pad, assuming a footing width of at least 2 m and granular pad thickness equal to at least one footing width.

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

5.6 Site Coefficient

For seismic design purposes, the Site Coefficient, S , for this site in accordance with Section 4.4.6 of the CHBDC may be taken as 1.0, consistent with Soil Profile Type I.

5.7 Lateral Earth Pressures for Design

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

The following recommendations are made concerning the design of the walls. It should be noted that these design recommendations and parameters assume level backfill and ground surface

behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular 'A' or Granular 'B' but with less than 5 per cent passing the 200 sieve should be used as backfill behind the walls. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 per cent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3501.00 and 3504.00.
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with *CHBDC* Section 6.9.3 and Figure 6.9.3. Compaction equipment should be used in accordance with OPSS 501.06; heavy compaction equipment should not be used within a lateral distance behind the structure equal to the current height of the fill above the base of the structure. Other surcharge loadings should be accounted for in the design, as required.
- The granular fill may be placed either in a zone with width equal to at least 1.8 m behind the back of the wall stem (Case I in Figure C6.9.1(l) of the *Commentary to the CHBDC*) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II in Figure C6.9.1(l) of the *Commentary to the CHBDC*).
- For Case I, the pressures are based on the proposed embankment fill materials and the following parameters (unfactored) may be used assuming the use of Select Subgrade material:

Soil unit weight:	20 kN/m ³
Coefficients of static lateral earth pressure:	
Active, K_a	0.35
At rest, K_o	0.50

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

	Granular 'A'	Granular 'B'
		Type II
Soil unit weight:	22 kN/m ³	21 kN/m ³
Coefficients of static lateral earth pressure:		
Active, K_a	0.27	0.31
At rest, K_o	0.43	0.47

- If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design.

- Seismic loading will result in increased lateral earth pressures acting on the abutment stem and retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure. According to the National Building Code of Canada, this site is located in Seismic Zone 4. The site-specific zonal acceleration ratio for Ottawa is 0.18. Based on experience, for the subsurface conditions at this site, a 50 per cent amplification of the ground motion will occur, resulting in an increase in the ground surface acceleration from 0.18g to about 0.27g. The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of $A = 0.27$.
- 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. $k_h = 0.41$). For structures which allow lateral yielding, k_h is taken as 0.5 times the zonal acceleration ratio (i.e. $k_h = 0.14$). 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	0.43	0.34	0.38
Non-yielding wall	1.5	0.9	1.1

- The above K_{AE} values for yielding walls are applicable provided that the wall can move up to 250A (mm), where A is the design zonal acceleration ratio of 0.22. This corresponds to displacements of up to 55 mm at this site.
- The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e. an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

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

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

d is the depth below the top of the wall (m); and
H is the total height of the wall (m).

5.8 Approach Embankment Design and Construction

The construction of the Appleton Sideroad underpass will require placement of up to about 9 m to 10 m of fill within the limits of the approach embankments. Based on the borehole results, the embankment subgrade soils, following removal of the surficial fill materials and underlying topsoil, will consist of about 1 m of stiff layered clayey silt and silty clay overlying about 4.5 m of silty clay to clay.

5.8.1 Subgrade Preparation and Approach Embankment Construction

Any topsoil, organic matter and softened / loosened soils should be stripped from below the approach embankment areas, and all subgrade soils should be proof-rolled prior to fill placement. To remove the buried topsoil at this site, the existing fill materials will also need to be removed.

Embankment fill should be placed in regular lifts with loose thickness not exceeding 300 mm, and be compacted to at least 95 per cent of the material's **Standard Proctor maximum dry density**. The final lift prior to placement of the granular subbase and base courses should be compacted to 100 per cent of the Standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified personnel during placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

Where the 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.

5.8.2 Approach Embankment Stability

Static slope stability analyses for this embankment configuration were carried out using the commercially available program SLOPE-W produced by Geo-Slope International Ltd. The soil parameters given in the following table were used in the analysis. The undrained shear strength of the silty clay deposits was measured to be typically greater than 130 kPa. The value of 100 kPa, therefore, as used in the analysis reflects the lower bound based on experience and correlations with water content/Atterberg Limits testing. The results of the analysis indicate the 9 m to 10 m high approach embankments with side slopes maintained at 2 horizontal to 1 vertical (2H:1V) will have a factor of safety of greater than 1.3 against deep-seated slope instability assuming appropriate subgrade preparation and proper placement and compaction of embankment fill materials.

<i>Soil Deposit</i>	<i>Bulk Unit Weight</i>	<i>Effective Friction Angle</i>	<i>Undrained Shear Strength</i>
Embankment Fill	20 – 22 kN/m ³	32° to 35°	–
Layered Clayey Silt and Silty Clay	20 kN/m ³	–	100 kPa
Silty Clay to Clay	18 kN/m ³	–	100 kPa
Silty Sand to Sandy Silt Till	21 kN/m ³	N/A (see note)	

NOTE: Due to the significant difference in shear strength and stress-strain response of the (marine) silty clay versus the underlying compact to dense granular till deposit, the critical failure surface for a deep-seated instability of the embankment does not penetrate the till deposit.

5.8.3 Approach Embankment Settlement

Settlement of the approach embankments will occur as a result of compression of the new embankment fill itself, as well as consolidation of the clayey soils on which the approaches will be founded.

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 of settlement since the majority of settlement of granular fills will occur during construction, whereas the majority of the settlement of cohesive fill, if used, would occur after construction.

Some settlement of the embankment subgrade can be expected due to compression of the clayey soils (i.e., the layered clayey silt and silty clay, the weathered clay crust, and in particular of the underlying grey silty clay to clay). The results of the oedometer consolidation testing indicate that the effective stress level in the clayey deposits will remain well below the deposit's preconsolidation pressure. The resulting consolidation settlements therefore correspond solely to recompression of the clayey deposits (i.e. no consolidation into the virgin compression range).

The total estimated magnitude of the primary consolidation settlements resulting from this recompression is about 25 mm, based on the recompression index value indicated by the consolidation testing. The results of that testing also indicate a relatively high coefficient of consolidation and, considering the relatively limited thickness of this deposit, a relatively rapid rate of settlement. It is expected that most of the settlement should be completed within about 6 months.

The results of a sustained load test, at the approximate calculated resulting effective stress level in the clay deposit following embankment construction, indicate a potential for some additional secondary compression settlements. The magnitude of those additional settlements is estimated at about 5 mm after about 5 years and about 10 mm after 50 years.

5.9 Design and Construction Considerations

5.9.1 Driven Steel H-Piles

Due to the hardness of the bedrock and the relatively soft consistency of the overburden, there is potential for damage to the piles and/or deflection of the piles, particularly for battered piles, unless there is appropriate care taken during driving of the piles. The pile driving criteria must be reviewed and accepted by the Quality Verification Engineer as appropriate to the site conditions and proposed pile type and configuration.

5.9.2 Excavation

Given that the abutment pile caps are likely to be perched within the approach embankments, it is anticipated that the only significant excavation required would be for the centre pier pile cap construction. Based on the foundation geometry indicated in the *Structural Planning Report*, that excavation would likely extend to no more than about 1.5 m below the existing grade in the pier area, to about Elevation 128 m. That excavation would therefore extend through the surficial fill material and topsoil, extending into the layered clayey silt and silty clay, and possibly reaching the weathered silty clay crust. The groundwater level in the pier area is relatively shallow, at about Elevation 129.1 m.

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 stiff clayey silt and silty clay below the groundwater level are classified as Type 3 soil, according to the OHSA. Temporary excavations (i.e. those which are only open for a relatively short period) through these overburden soils should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V).

It is not anticipated that temporary roadway protection will be required to permit construction of the new Appleton Sideroad approach embankment and underpass structure.

5.9.3 Groundwater and Surface Water Control

The groundwater level at the site is generally at or less than about 1 m below the natural ground surface. Excavations to the underside of pile cap level for the centre pier will therefore require some groundwater control. However, given the relatively low permeability of the layered clayey silt and silty clay, it is not anticipated that the rate of groundwater inflow will be significant.

As noted in Section 5.4, if drilled shafts are adopted at this site, the use of a temporary liner will be required within the overburden to support the auger holes during pile or concrete placement.


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MIC/LCC/ASP/FJH/mic

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TABLE 1
COMPARISON OF FOUNDATION ALTERNATIVES
APPLETON SIDEROAD (COUNTY ROAD 17) UNDERPASS STRUCTURE

<i>Foundation Option</i>	<i>Feasibility</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Relative Costs</i>	<i>Risks/Consequences</i>
Spread footings supported on native clayey silt to silty clay soils	<ul style="list-style-type: none"> • Not feasible 	N/A	N/A	N/A	N/A
Steel H-pile foundations founded on or socketted into bedrock	<ul style="list-style-type: none"> • Feasible for support of all foundation elements and retaining walls 	<ul style="list-style-type: none"> • High bearing resistance • Negligible settlement • Site conditions appropriate for use of integral abutments 	<ul style="list-style-type: none"> • 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 	<ul style="list-style-type: none"> • May be less expensive than drilled shaft option if rock sockets are drilled, owing to potentially smaller socket diameter 	<ul style="list-style-type: none"> • If required for pile toe fixity, socketting into the medium strong bedrock could be difficult and time-consuming
Drilled shafts founded or socketted into bedrock	<ul style="list-style-type: none"> • Feasible for support of all foundation elements and retaining walls 	<ul style="list-style-type: none"> • High bearing resistance • Negligible settlement 	<ul style="list-style-type: none"> • Temporary liners required to minimize disturbance to surrounding soils • Possibility of encountering cobbles or boulders during drilled shaft installation • If rock socket required, coring or churn drilling will be required to form socket in medium strong bedrock 	<ul style="list-style-type: none"> • May be more expensive than steel H-pile option if rock sockets are necessary, owing to potentially larger socket diameter 	<ul style="list-style-type: none"> • If required for pile toe fixity, socketting into the medium strong bedrock could be difficult and time-consuming
Spread footings "perched" on Granular 'A' pad within the approach embankments	<ul style="list-style-type: none"> • Feasible for abutments if abutment area is preloaded 	-	<ul style="list-style-type: none"> • Advance construction of embankment (preloading / surcharging) required • Still potential for differential settlement between abutments and pier 	<ul style="list-style-type: none"> • Less expensive than abutment piling 	<ul style="list-style-type: none"> • Differential settlement between abutments and pier may be in excess of 25 mm

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

(b) Cohesive Soils

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

Dynamic Cone Penetration Resistance; N_d :

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

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

IV. SOIL TESTS

w water content
w_p plastic limit
w_l liquid limit
C consolidation (oedometer) test
CHEM chemical analysis (refer to text)
CID consolidated isotropically drained triaxial test¹
CIU consolidated isotropically undrained triaxial test with porewater pressure measurement¹
D_R relative density (specific gravity, G_s)
DS direct shear test
M sieve analysis for particle size
MH combined sieve and hydrometer (H) analysis
MPC Modified Proctor compaction test
SPC Standard Proctor compaction test
OC organic content test
SO₄ concentration of water-soluble sulphates
UC unconfined compression test
UU unconsolidated undrained triaxial test
V field vane (LV-laboratory vane test)
γ unit weight

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

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LIST OF SYMBOLS

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

I. General

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta \sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	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

(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 $= \tan \delta$
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 + \sigma_3)/2$ or $(\sigma'_1 + \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 + \sigma_3)$
S_t	sensitivity

- Notes: 1 $\tau = c' + \sigma' \tan \phi'$
 2 shear strength $= (\text{compressive strength})/2$
 * density symbol is ρ . Unit weight symbol is γ where
 $\gamma = \rho g$ (i.e. mass density \times acceleration due to gravity)

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LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of weathering.

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

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

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

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

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

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

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

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

CORE CONDITION

Total Core Recovery

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

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

DISCONTINUITY DATA

Fracture Index

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

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

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

Description and Notes

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

Abbreviations

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



PROJECT 021-1155-8		RECORD OF BOREHOLE No 02-801		1 OF 2		METRIC						
W.P. 252-99-00		LOCATION N 5001259.2 E 338943.7		ORIGINATED BY D.J.S.								
DIST _____ HWY 7		BOREHOLE TYPE CME 55 Bombardier, 108 mm I.D. Hollow Stem Augers		COMPILED BY S.L.								
DATUM Geodetic		DATE May 27, 2003		CHECKED BY M.I.C.								
SOIL PROFILE		SAMPLES		DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID UNIT WEIGHT REMARKS & GRAIN SIZE DISTRIBUTION						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	W _p W W _L	WATER CONTENT (%)	γ	GR SA SI CL
129.5	Ground Surface											
0.0	Sandy topsoil, trace gravel (FILL)											
129.2	Grey brown											
129.0	Topsoil											
0.6	Layered Clayey Silt and Silty Clay, with sand seams		1	SS	6		129					
128.1	Very stiff											
1.4	Grey brown		2	SS	6		128					
	Wet											
			3	SS	9		127					
			4	SS	7		126					
126.0	Silty Clay to Clay											
3.5	Very stiff		5	SS	4		125					
	Grey											
	Wet		6	SS	5		124					
123.9	Sandy Silt, some gravel and clay		7	SS	18/0.23		123					
5.7	(FILL)											
	Loose											
	Grey											
	Wet											
	DOLOMITIC LIMESTONE (BEDROCK)						122					
	Fresh											
	Medium strong											
	Thinly to medium bedded											
	Grey											
122.0	SANDSTONE (BEDROCK)						122					
	Fresh											
	Medium strong											
	Thinly bedded											
	Grey green											
121.8	LIMESTONE AND DOLOMITIC LIMESTONE (BEDROCK) with sandy intervals						121					
7.7	Fresh											
	Medium strong											
	Thinly to medium bedded											
	Grey											
120.5	Bedrock cored between 5.7m and 9.0m depth											
9.0	For Bedrock coring details refer to Record of Drillhole 02-801											

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

Continued Next Page

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



PROJECT 021-1155-8		RECORD OF BOREHOLE No 02-801		2 OF 2		METRIC							
W.P. 252-99-00		LOCATION N 5001259.2 ; E 336943.7		ORIGINATED BY D.J.S.									
DIST HWY 7		BOREHOLE TYPE CME 55 Bombardier, 108 mm I.D. Hollow Stem Augers		COMPILED BY S.L.									
DATUM Geodetic		DATE May 27, 2003		CHECKED BY M.I.C.									
SOIL PROFILE		SAMPLES		DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa		WATER CONTENT (%)		GR SA SI CL	
	--- CONTINUED FROM PREVIOUS PAGE ---							20 40 60 80 100		25 50 75			
	End of Borehole Note: * Split-spoon bouncing after 18 blows Water Level in piezometer at 0.3 m depth (Elev. 129.2 m) on August 1, 2003							20 40 60 80 100		25 50 75			

MISS_MTO 021-1155-8.GPJ ON_MOT.GDT 18/9/04

+³, X³: Numbers refer to Sensitivity

O 3% STRAIN AT FAILURE

PROJECT: 021-1155-5050

RECORD OF DRILLHOLE: 02-801

SHEET 1 OF 1

LOCATION: N 5001259.2 ; E 336943.7

DRILLING DATE: May 27, 2003

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH % RETURN	RECOVERY				FRACT. INDEX PER 0.3	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY				DIP M.I.L. CORE AXIS		TYPE AND SURFACE DESCRIPTION	BC-BROKEN CORE MB-MECH. BREAK B-BEDDING				FL-FLEXURED UE-UNEVEN W-WAVY C-CURVED	SM-SMOOTH R-ROUGH ST-STEPPED PL-PLANAR	J-JOINT P-POLISHED S-SLICKENSIDED	CL-CLEAVAGE SH-SHEAR VN-VEIN	FR-FRACTURE-FAULT	NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
								TOTAL CORE %		SOLID CORE %			R.Q.D. %	DIP #11 CORE AXIS	DIP #12 CORE AXIS	K, cm/sec				DIP #13 CORE AXIS		DIP #14 CORE AXIS	DIP #15 CORE AXIS	DIP #16 CORE AXIS	DIP #17 CORE AXIS							DIP #18 CORE AXIS	DIP #19 CORE AXIS	DIP #20 CORE AXIS																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																														
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DEPTH SCALE

1:50



LOGGED: D.J.S.

CHECKED: *llk*

MISS. ROCK 021-1155-8 ROCK.GPJ GLDR. CAN.GDT 18/8/04



PROJECT 021-1155-8		RECORD OF BOREHOLE No 02-802		1 OF 1		METRIC											
W.P. 252-99-00		LOCATION N 5001252.6 ; E 336946.0		ORIGINATED BY D.J.S.													
DIST _____ HWY 7		BOREHOLE TYPE CME 55 Bombardier, 108 mm I.D. Hollow Stem Augers		COMPILED BY S.L.													
DATUM Geodetic		DATE May 28, 2003		CHECKED BY M.I.C.													
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	20 40 60 80 100	W _p W W _L	25 50 75	γ	GR SA SI CL				
129.3	Ground Surface																
0.0	Sandy Topsoil, trace gravel (Fill)																
129.0	Topsoil																
128.9																	
0.6	Layered Clayey Silt to Silty Clay, with sand seams Very stiff Grey brown Wet		1	SS	4												
128.1																	
1.2	Clay (Weathered Crust) Very stiff Grey brown Wet		2	SS	6												
			3	SS	9												
			4	SS	7												
129.2			5	SS	5												
4.1	Silty Clay to Clay Very stiff Grey Wet																
			6	SS	5												
124.0																	
5.3	Sandy Silt, some gravel and clay, occasional sand seam (Till) Compact Grey Wet		7	SS	25												
123.2																	
6.1	End of Borehole Auger Refusal																

MISS MTO 021-1155-8.GPJ ON MOT.GDT 18/8/04

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



PROJECT		RECORD OF BOREHOLE		No 02-803		1 OF 2		METRIC			
W.P. 252-99-00		LOCATION		N 5001225.8 E 336978.3		ORIGINATED BY		D.J.S.			
DIST HWY 7		BOREHOLE TYPE		CME 55 Bombardier, 108 mm I.D. Hollow Stem Augers		COMPILED BY		S.L.			
DATUM Geodetic		DATE		May 22, 2003		CHECKED BY		M.I.C.			
SOIL PROFILE		SAMPLES		DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	W _p W W _L	WATER CONTENT (%)	GR SA SI CL
129.5	Ground Surface										
0.0	Sand and gravel (FILL) Brown										
129.0											
0.6	Topsoil Dark brown Layered Clayey Silt and Silty Clay with silty sand seams Very stiff Grey brown Wet		1	SS	7						
128.0											
1.5	Clay (Weathered Crust) Very stiff Grey brown Wet		2	SS	10						
			3	SS	9						
			4	SS	7						
126.0											
3.5	Silty Clay to Clay Very stiff Grey Wet		5	SS	7						
			6	SS	8						
124.2											
5.3	Sandy Silt, some gravel and clay (Till) Compact to very dense Grey Wet		7	SS	17						
123.4			8	SS	50/0.15						
6.1	SANDSTONE (BEDROCK) Fresh Medium strong Thinly to medium bedded Grey green										
122.9											
6.6	LIMESTONE and DOLOMITIC LIMESTONE (BEDROCK) with sandy intervals Fresh Medium strong Thinly to medium bedded Grey										
	Bedrock cored between 6.1m and 9.9m depth										
	For Bedrock coring details refer to Record of Drillhole 02-803										
119.6											
9.9											

MISS MTO 021-1155-8.GPJ ON MOT.GDT 18/8/04

Continued Next Page

+ 3 . X 3. Numbers refer to Sensitivity 0 3% STRAIN AT FAILURE



PROJECT 021-1155-8		RECORD OF BOREHOLE No 02-803				2 OF 2		METRIC			
W.P. 252-99-00		LOCATION N 5001225.8 ; E 336976.3				ORIGINATED BY D.J.S.					
DIST _____ HWY 7		BOREHOLE TYPE CME 55 Bombardier, 108 mm I.D. Hollow Stem Augers				COMPILED BY S.L.					
DATUM Geodetic		DATE May 22, 2003				CHECKED BY M.I.C.					
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT		UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER			TYPE	"N" VALUES	20 40 60 80 100	W _p W W _L		
--- CONTINUED FROM PREVIOUS PAGE ---											
	End of Borehole										
	Note: Water Level in piezometer at 0.6 m depth (Elev. 128.9 m) on August 1, 2003										
	* Split-spoon bouncing after 50 blows										

MISS_MTO 021-1155-8.GPJ ON_MOT.GDT 18/0/04

PROJECT: 021-1155-5050

RECORD OF DRILLHOLE: 02-803

SHEET 1 OF 1

LOCATION: N 5001225.8 ; E 336976.3

DRILLING DATE: May 22, 2003

DATUM: Geodetic

INCLINATION: -90°

AZIMUTH: --

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH % RETURN	FFVFX-FRACTURE-FAULT		J-JOINT		R-ROUGH		FL-FLEXURED		BC-BROKEN CORL		DIA METRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
								CL-CLEAVAGE		P-POLISHED		R-ROUGH		UE-UNEVEN		MB-MECH. BREAK			
								SH-SHEAR		S-SLICKENSIDED		ST-STEPPED		W-WAVY		B-BEDDING			
								VN-VEIN		PL-PLANAR		C-CURVED							
RECOVERY		R.Q.D. %		FRACT. INDEX PER 0.3		DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY											
TOTAL CORE %		SOLID CORE %		O.P. WALL CORE AXIS		TYPE AND SURFACE DESCRIPTION		10 ⁻³ K, cm/sec		10 ⁻³ K, cm/sec		10 ⁻³ K, cm/sec							
88888		88888		88888		88888		88888		88888		88888							
123.38 6.10		122.84 6.84		9		100													
7		SANDSTONE (BEDROCK) Fresh Medium strong Thinly to medium bedded Grey green LIMESTONE and DOLOMITIC LIMESTONE (BEDROCK) with sandy intervals Fresh Medium strong Thinly to medium bedded Grey		10		100													
8		Rotary Drilling NQ Core		11		100													
9																			
10		End of Borehole		119.60 9.88															
11																			
12																			
13																			
14																			
15																			
16																			

DEPTH SCALE

1:50



LOGGED: D.J.S.

CHECKED: *lcl*

MISS. ROCK 021-1155-8 ROCK GPJ GLDR. CAN.GDT. 18/8/04



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

PROJECT: 021-1155-5050

RECORD OF DRILLHOLE: 02-804

SHEET 1 OF 1

LOCATION: N 5001200.5 ; E 337000.7

DRILLING DATE: May 26, 2003

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN NO.	PENETRATION RATE (mm/min)	FLUSH % RETURN	FR-FX-FRACTURE-F-FAULT										SM-SMOOTH		FL-FLEXURED		BC-BROKEN CORE		DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										
								CL-CLEAVAGE		J-JOINT		R-ROUGH		UE-UNEVEN		MB-MECH. BREAK		B-BEDDING																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
								SH-SHEAR		P-POLISHED		ST-STEPPED		W-WAVY																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					
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								RECOVERY		R.Q.D. %		FRACT. INDEX PER 0.3		DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K, cm/sec		TYPE AND SURFACE DESCRIPTION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
								TOTAL CORE %	SOLID CORE %																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										

DEPTH SCALE

1 : 50



LOGGED: D.J.S.

CHECKED: *ll*

MISS. ROCK 021-1155-8 ROCK.GPJ GLDR. CAN.GDT 18/8/04



PROJECT		RECORD OF BOREHOLE		No 02-805		1 OF 1		METRIC														
W.P. 021-1155-8		LOCATION		N 5001196.7; E 337003.4		ORIGINATED BY		D.J.S.														
DIST. HWY 7		BOREHOLE TYPE		CME 55 Bombardier, 108 mm I.D. Hollow Stem Augers		COMPILED BY		S.L.														
DATUM Geodetic		DATE		May 26, 2003		CHECKED BY		M.J.C.														
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION										
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20	40	60	80	100	PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	WATER CONTENT (%)	UNIT WEIGHT	GR	SA	SI	CL
130.4	0.0	Ground Surface																				
0.1	0.1	Asphalt																				
129.9	0.1	Crushed stone (FILL) Grey																				
129.9	0.5	Crushed stone and silty clay, trace cobbles, plastic and organic matter (FILL) Compact Brown and grey Moist		1	SS	14																
128.9		Topsoil																				
128.7	1.7	Layered Clayey Silt and Silty Clay with sand seams Very stiff Grey Very Moist		2	SS	8																
128.0	2.4	Clay (Weathered Crust) Very stiff Grey brown Wet		3	SS	8																
				4	SS	10																
				5	SS	7																
125.5	4.9	Silty Clay to Clay Very stiff Grey Wet		6	SS	6																
				7	SS	7																
124.4	6.1	Sandy Silt (Till) Grey End of Borehole Auger Refusal																				
<p>Note: Water Level in open hole at 1.5m depth (Elev. 128.9 m) on May 26, 2003</p>																						

MISS MTO 021-1155-8.GPJ ON MOT.GDT 19/8/04

+³, x³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE



PROJECT 021-1155-8		RECORD OF BOREHOLE No 02-806		1 OF 1		METRIC												
W.P. 252-99-00		LOCATION N 5001242.9; E 338927.4		ORIGINATED BY D.J.S.														
DIST _____ HWY 7		BOREHOLE TYPE CME 55 Bombardier, 108 mm I.D. Hollow Stem Augers		COMPILED BY S.L.														
DATUM Geodetic		DATE May 27, 2003		CHECKED BY M.I.C.														
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	20 40 60 80 100	W _p	W	W _L	γ	GR	SA	SI	CL
129.4	0.0	Ground Surface																
0.2		Sand, some gravel and topsoil (FILL)																
129.0	0.2	Dark brown Silty Clay Topsoil																
129.0	0.4	Layered Silty Clay and Clayey Silt with sand seams Very stiff Grey brown Wet		1	SS	8		129										
128.9	1.2	Clay (Weathered Crust) Very stiff Grey brown Wet		2	SS	9		129										
				3	SS	10		127										
125.9	3.5	Silty Clay to Clay Very stiff Grey Wet		4	SS	6		120										
				5	SS	5		125										
124.1	5.3	Sandy Silt, some gravel and clay (Till)		6	SS	8		125										
123.8	5.6	Compact Grey Wet		7	SS	14/0.3*		124										
		End of Borehole Auger Refusal																
Note: * Split-spoon bouncing after 14 blows																		

MISS MTO 021-1155-8.GPJ ON MOT.GDT 18/6/04

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



PROJECT		RECORD OF BOREHOLE		No 02-807		1 OF 1		METRIC																							
W.P. 252-99-00		LOCATION		N 5001238.9; E 3389930.4		ORIGINATED BY		D.J.S.																							
DIST		HWY 7		BOREHOLE TYPE		CME 55 Bombardier, 108 mm I.D. Hollow Stem Augers		COMPILED BY		S.L.																					
DATUM		Geodetic		DATE		May 27, 2003		CHECKED BY		M.I.C.																					
SOIL PROFILE		SAMPLES		GROUND WATER		ELEVATION SCALE		DYNAMIC CONE PENETRATION		RESISTANCE PLOT		PLASTIC		NATURAL		LIQUID		UNIT		REMARKS											
ELEV		DESCRIPTION		STRAT PLOT		NUMBER		TYPE		"N" VALUES		ELEVATION SCALE		SHEAR STRENGTH kPa		WATER CONTENT (%)		Wp		W		Wl		GR		SA		SI		CL	
129.3		Ground Surface																													
0.0		Topsoil																													
129.1		Layered Clayey Silt and Silty Clay, occ. sand seams																													
0.2		Very stiff																													
		Grey brown																													
		Wet																													
128.2		Clay (Weathered Crust)				1		SS		8																					
1.1		Very stiff																													
		Grey brown																													
		Wet																													
128.0		Silty Clay to Clay				2		SS		10																					
3.4		Stiff																													
		Grey																													
		Wet																													
124.1		Silty Silt, some gravel and clay				3		SS		10																					
123.9		(Till)																													
5.4		Compact				4		SS		8																					
		Grey																													
		Wet																													
		DOLOMITIC LIMESTONE (BEDROCK)				5		SS		6																					
		Fresh																													
		Medium strong																													
		Thinly to medium bedded																													
		Dark grey																													
122.4		SANDSTONE (BEDROCK)				6		SS		5																					
7.0		Fresh																													
		Medium strong																													
		Medium bedded																													
		Grey green																													
		LIMESTONE and DOLOMITIC																													
		LIMESTONE (BEDROCK) with																													
		sandy intervals																													
		Fresh																													
		Medium strong																													
		Thinly to medium bedded																													
		Grey																													
120.4		Bedrock cored between 5.4m and 8.9m depth				7		SS		50.08																					
8.9		For Bedrock coring details refer to Record of Borehole 02-807																													
		End of Borehole																													
		Note: * Split-spoon bouncing after 15 blows																													

MISS MTD 021-1155-8.GPJ ON MOT.GDT 18/04

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

PROJECT: 021-1155-5050

RECORD OF DRILLHOLE: 02-807

SHEET 1 OF 1

LOCATION: N 5001238.9 ; E 336930.4

DRILLING DATE: May 27, 2003

DATUM: Geodetic

INCLINATION: -90°

AZIMUTH: —

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN NO.	PENETRATION RATE (mm/min)	FLUSH % RETURN	RECOVERY				FRACT. INDEX PER 0.3	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)		NOTES WATER LEVELS INSTRUMENTATION
								TOTAL CORE %	SOLID CORE %	R.O.D. %	DIP w.r.t. CORE AXIS		TYPE AND SURFACE DESCRIPTION	10 ⁻⁴		10 ⁻³	10 ⁻²	
				123.91 5.43			100											
6		DOLOMITIC LIMESTONE (BEDROCK) Fresh Medium strong Thinly to medium bedded Dark grey			8		100											
7	Rotary Drill NQ Core	SANDSTONE (BEDROCK) Fresh Medium strong Medium bedded Grey green		122.42 8.32 7.04	9		100											
8		LIMESTONE and DOLOMITIC LIMESTONE (BEDROCK) with sandy intervals Fresh Medium strong Thinly to medium bedded Grey			10		100											
9		End of Borehole		120.44 8.90														
10																		
11																		
12																		
13																		
14																		
15																		

DEPTH SCALE

1:50



LOGGED: D.J.S.

CHECKED: *ll*

MISS. ROCK 021-1155-8 ROCK.GPJ GLDR. CAN.GDT 18/8/04



PROJECT 021-1155-8		RECORD OF BOREHOLE No 02-808		1 OF 1		METRIC		
W.P. 252-99-00		LOCATION N 5001211.6 ; E 336959.1		ORIGINATED BY D.J.S.				
DIST HWY 7		BOREHOLE TYPE CME 55 Bombardier, 108 mm I.D. Hollow Stem Augers		COMPILED BY S.L.				
DATUM Geodetic		DATE May 22, 2003		CHECKED BY M.J.C.				
SOIL PROFILE		SAMPLES		DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID UNIT WEIGHT REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER TYPE "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x REMOULDED	WATER CONTENT (%) W _p W W _L	UNIT WEIGHT γ kN/m ³	GR SA SI CL
129.5	Ground Surface							
0.0	Topsoil (FILL)							
0.2	Sand and gravel, occ. cobble (FILL)							
129.0	Topsoil							
0.6	Layered Clayey Silt and Silty Clay, with sand seams Very stiff Grey brown Moist to wet		1 SS 5					0 28 59 13
127.9	Clay (Weathered Crust) Very stiff Grey brown Wet		2 SS 12					
1.6			3 SS 7					
			4 SS 7					
125.5	Silty Clay to Clay Very stiff Grey Wet		5 SS 6					
4.0			6 SS 6					
124.2	Silty Sand, some gravel and clay (Till) Compact Grey Wet		7 SS 19					18 38 32 12
5.3								
123.6	SANDSTONE (BEDROCK) Fresh Medium strong Thinly to medium bedded Grey green							
6.0								
122.9	LIMESTONE and DOLOMITIC LIMESTONE (BEDROCK) with sandy intervals Fresh Medium strong Thinly to medium bedded Grey							
6.6								
	Bedrock cored between 6.0 m and 9.3 m depth For Bedrock coring details refer to Record of Drillhole 02-808							
120.2	End of Borehole							
9.3								

MISS MTO 021-1155-8.GPJ ON MOT.GDT 18/8/04

PROJECT: 021-1155-5050

LOCATION: N 5001211.6 ; E 336959.1

INCLINATION: -90° AZIMUTH: ---

RECORD OF DRILLHOLE: 02-808


SHEET 1 OF 1

DRILLING DATE: May 22, 2003

DATUM: Geodetic

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	CORRELATION CHART																NOTES WATER LEVELS INSTRUMENTATION
				ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH % RETURN	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-SLICKEY		PL-PLANAR		C-CURVED														
RECOVERY		R.Q.D. %		FRACT. INDEX PER 0.3		DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY				DIAMETRAL POINT LOAD INDEX (MPa)						
TOTAL CORE %		SOLID CORE %				DIP w.r.t. CORE AXIS				TYPE AND SURFACE DESCRIPTION				K, cm/sec						
88888		88888		88888		88888				88888				10 ⁻⁴ 10 ⁻³ 10 ⁻² 10 ⁻¹				2 4 6		
6		SANDSTONE (BEDROCK) Fresh Medium strong Thinly to medium bedded Gray LIMESTONE and DOLOMITIC LIMESTONE (BEDROCK) with sandy intervals Fresh Medium strong Thinly to medium bedded Gray		123.47 6.00																
7				122.89 6.58	8	100														
8	Rotary Drill NO Core				9	100														
9					10	100														
		End of Borehole		120.20 9.27																
10																				
11																				
12																				
13																				
14																				
15																				
16																				

DEPTH SCALE

1:50



LOGGED: D.J.S.

CHECKED: *lll*

MISS. ROCK 021-1155-8 ROCK.GPJ GLDR. CAN.GDT 18/8/04



PROJECT 021-1155-8		RECORD OF BOREHOLE No 02-809		1 OF 1		METRIC					
W.P. 252-99-00		LOCATION N 5001185.9 : E 336984.5		ORIGINATED BY D.J.S.							
DIST _____ HWY 7		BOREHOLE TYPE CME 55 Bombardier, 108 mm I.D., Hollow Stem Augers		COMPILED BY S.L.							
DATUM Geodetic		DATE May 23, 2003		CHECKED BY M.L.C.							
SOIL PROFILE		SAMPLES		DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT		UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	W _p W W _L	γ	GR SA SI CL
129.8	Ground Surface										
0.0	Topsail (FILL)										
0.2	Sand and Gravel, occ. cobble, trace silt (FILL)										
129.2	Brown										
0.8	Topsail										
128.1	Layered Clayey Silt and Silty Clay with sand seams		1	SS	7		12S				0 20 64 16
1.7	Very stiff										
	Gray brown										
	Moist										
128.1	Clay (Weathered Crust)		2	SS	12		12C				
	Very stiff										
	Gray brown										
	Wet										
			3	SS	12		127				
			4	SS	7		12C				
			5	SS	7		12C				
125.5	Silty Clay to Clay										
4.3	Very stiff										
	Gray										
	Wet										
			6	SS	7		125				0 2 45 53
124.4	Sandy Silt, some gravel and clay										
124.2	(Till)		7	SS	17/0.28						
5.6	Loose										
	Gray										
	Wet										
	End of Borehole										
	Auger Refusal										
<p>Note: Water Level in piezometer at 1.02 m depth (Elev. 128.8 m) on August 1, 2003</p> <p>* Split-spoon bouncing after 17 blows</p>											

MISS MTO 021-1155-8.GPJ ON MOT.GDT 18/04



PROJECT 021-1155-8			RECORD OF BOREHOLE No 02-810			1 OF 1			METRIC								
W.P. 252-99-00			LOCATION N 50D1182.5:E 336987.4			ORIGINATED BY D.J.S.											
DIST HWY 7			BOREHOLE TYPE CME 55 Bombardier, 108 mm I.D. Hollow Stem Augers			COMPILED BY S.L.											
DATUM Geodetic			DATE May 23, 2003			CHECKED BY M.I.C.											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	20 40 60 80 100	25 50 75	W _p W W _L	γ	GR SA SI CL				
129.8	Ground Surface																
0.0	Topsoil (Fill)																
0.2	Sand and Gravel, some silt, occ. cobble																
129.2	Brown (Fill)																
129.0	Topsoil																
0.9	Layered Clayey Silt and Silty Clay, with fine sand seams		1	SS	9		129										
128.1	Very stiff																
1.7	Grey brown		2	SS	15		128										
	Moist																
			3	SS	12		127										
			4	SS	9		126										
125.8																	
4.0	Silty Clay to Clay		5	SS	7		125										
	Very stiff																
	Grey		6	SS	6												
	Wet																
124.4																	
124.2	Sandy Silt, some gravel and clay (Till)		7	SS	26/0.23		124										
5.6	Compact																
	Grey																
	Wet																
	LIMESTONE and DOLOMITIC LIMESTONE (BEDROCK) with sandy intervals																
	Fresh																
	Medium strong																
	Thinly to medium bedded																
	Grey to dark grey																
	Bedrock cored between 5.6 m and 9.0 m depth																
	For Bedrock coring details refer to Record of Drillhole 02-810																
120.8							123										
9.0	End of Borehole						122										
	Note: * Split-spoon bouncing after 26 blows						121										

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

MISS-MTO 021-1155-8.GPJ ON MOT.GDT 18/6/04

PROJECT: 021-1155-5050

RECORD OF DRILLHOLE: 02-810

SHEET 1 OF 1

LOCATION: N 5001182.5 ; E 336987.4

DRILLING DATE: May 23, 2003

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: --

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (cm/min)	FLUSH % RETURN	RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K, cm/sec			DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
								TOTAL CORE %	SOLID CORE %			DIP W.I.L. CORE AXIS	TYPE AND SURFACE DESCRIPTION	10 ⁻³	10 ⁻⁴	10 ⁻⁵		
								8888	8888			8888	8888	8888	8888	8888		
6	Relay Drill NQ Core	LIMESTONE and DOLOMITIC LIMESTONE (BEDROCK) with sandy intervals Fresh Medium strong Thinly to medium bedded Grey to dark grey		124.19 5.58	8	100	100											
7		9		100														
8		10		100														
9		End of Borehole		120.81 8.96														
10																		
11																		
12																		
13																		
14																		
15																		

DEPTH SCALE

1:50



LOGGED: D.J.S.

CHECKED: *ll*

MISS. ROCK 021-1155-8 ROCK.GPJ GLDR. CAN.GDT 18/04



PROJECT		RECORD OF BOREHOLE		No 02-820		1 OF 1		METRIC													
W.P. 252-99-00		LOCATION		N 5001260.2; E 336915.7		ORIGINATED BY		D.J.S.													
DIST		HWY 7		BOREHOLE TYPE		CME 55 Bombardier, 108 mm I.D. Hollow Stem Augers		COMPILED BY		S.L.											
DATUM		Geodetic		DATE		May 28, 2003		CHECKED BY		M.I.C.											
SOIL PROFILE		SAMPLES		GROUND WATER		ELEVATION SCALE		DYNAMIC CONE PENETRATION		RESISTANCE PLOT		PLASTIC LIMIT		NATURAL MOISTURE		LIQUID LIMIT		UNIT WEIGHT		REMARKS	
ELEV. DEPTH		DESCRIPTION		STRAT. PLOT		NUMBER		TYPE		"N" VALUES		ELEVATION SCALE		SHEAR STRENGTH kPa		WATER CONTENT (%)		UNIT WEIGHT		REMARKS	
129.7		Ground Surface																			
0.0		Asphalt																			
0.2		Crushed Stone (FILL)																			
		Gray																			
		Sand and Gravel (FILL)																			
		Brown																			
128.8		Layered Clayey Silt and Silty Clay				1		SS		6		120									
0.9		with sand seams																			
		Very stiff																			
		Gray brown																			
		Moist to wet																			
128.0		Clay (Weathered Crust)				2		SS		10		120									
1.7		Very stiff																			
		Gray brown																			
		Wet																			
						3		SS		11		127									
						4		SS		6		126									
125.9		Silty Clay to Clay				5		SS		4		125									
3.8		Very stiff																			
		Gray																			
		Wet																			
						6		SS		5											
						7		SS		5		120									
123.5		Sandy Silt, some gravel and clay				8		SS		58		120									
8.3		(Till)																			
		Very dense																			
123.0		Gray																			
6.7		Wet																			
		End of Borehole																			

MISS MTO 021-1155-8.GPJ ON MOT.GDT 18/8/04

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



PROJECT		RECORD OF BOREHOLE		No 02-820A		1 OF 1		METRIC							
W.P.		LOCATION		N 5001259.5 ; E 336915.0		ORIGINATED BY		P.A.H.							
DIST		BOREHOLE TYPE		CME 55 Bombardier, 108 mm I.D. Hollow Stem Augers		COMPILED BY		P.M.							
DATUM		DATE		July 21, 2003		CHECKED BY		M.I.C.							
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID UNIT REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	N° VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	20 40 60 80 100	Wp	W	Wl	Y	GR SA SI CL
129.7	0.0	Ground Surface													
129.7	0.1	Asphalt													
129.7	0.2	Crushed Stone (FILL)													
129.7	0.2	Gray Sand and Gravel (FILL)													
129.7	0.2	Brown													
128.8	0.9	Layered Clayey Silt and Silty Clay with sand seams													
128.8	0.9	Very stiff													
128.8	0.9	Gray brown													
128.8	0.9	Moist to wet													
128.0	1.7	Clay (Weathered Crust)													
128.0	1.7	Very stiff													
128.0	1.7	Gray brown													
128.0	1.7	Wet													
125.9	3.8	Silty Clay to Clay													
125.9	3.8	Very stiff													
125.9	3.8	Gray													
125.9	3.8	Wet													
123.6	6.1	End of Borehole													
Notes:															
1. >> Indicates shear strength greater than 130 KPa															
2. Water level in open hole at 2.0 m depth during drilling															

MISS_MTO 021-1155-8.GPJ ON MOT.GDT 19/8/04



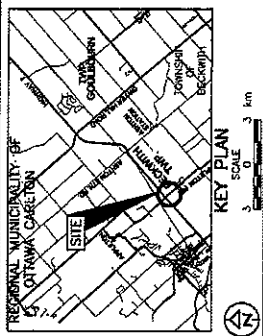
PROJECT		RECORD OF BOREHOLE		No 02-820B		1 OF 1		METRIC													
W.P. 252-99-00		LOCATION		N 5001258.8; E 336914.3		ORIGINATED BY		P.A.H.													
DIST HWY 7		BOREHOLE TYPE		CME 55 Bombardier, 108 mm I.D. Hollow Stem Augers		COMPILED BY		P.M.													
DATUM Geodetic		DATE		July 21, 2003		CHECKED BY		M.I.C.													
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			REMARKS & GRAIN SIZE DISTRIBUTION (%)												
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20	40	60	80	100	PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	GR	SA	SI	CL
129.7	0.0	Ground Surface																			
0.2		Asphalt																			
		Crushed Stone (FILL)																			
		Gray Sand and Gravel (FILL)																			
		Brown																			
128.8	0.9	Layered Clayey Silt and Silty Clay with sand seams																			
		Very stiff																			
		Gray brown																			
		Moist to wet																			
128.0	1.7	Clay (Weathered Crust)																			
		Very stiff																			
		Gray brown																			
		Wet																			
125.9	3.8	Silty Clay to Clay																			
		Very stiff																			
		Gray																			
		Wet																			
124.7	5.0	End of Borehole		1	TP	PH											19.4				

MISS_MTO_021-1155-9.GPJ ON MOT.GDT 19/9/04



PROJECT 021-1155-8		RECORD OF BOREHOLE No 02-821		1 OF 1		METRIC						
W.P. 252-99-00		LOCATION N 5001173.5 : E 337008.2		ORIGINATED BY D.J.S.								
DIST HWY 7		BOREHOLE TYPE CME 55 Bombardier, 108 mm I.D. Hollow Stem Augers		COMPILED BY S.L.								
DATUM Geodetic		DATE May 26, 2003		CHECKED BY M.I.C.								
SOIL PROFILE			SAMPLES		DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID		UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	W _p W W _L	γ	GR SA SI CL	
130.1	Ground Surface											
0.0	Topsoil (Fill)											
0.1	Sand and gravel, occ. cobble (Fill)											
	Loose											
	Brown											
	Moist											
129.2	Topsoil											
0.9	Dark brown		1	SS	6							
129.9	Moist											
1.2	Layered Clayey Silt and Silty Clay, with sand seams											
	Very stiff											
	Grey brown		2	SS	8							
128.3	Moist to wet											
1.8	Clay (Weathered Crust)											
	Very stiff											
	Grey brown											
	Wet											
			3	SS	10							
			4	SS	8							
126.0	Silty Clay to Clay		5	SS	7							
4.1	Very stiff											
	Grey											
	Wet		6	SS	6							
124.6	Sandy Silt, some clay and gravel (Till)		7	SS	20/0.23							
124.4	Compact											
5.7	Grey											
	Wet											
	End of Borehole											
	Auger Refusal											
<p>Note: Water Level in piezometer at 1.45 m depth (Elev. 128.6 m) on August 1, 2003</p> <p>* Split-spoon bouncing after 20 blows</p>												

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

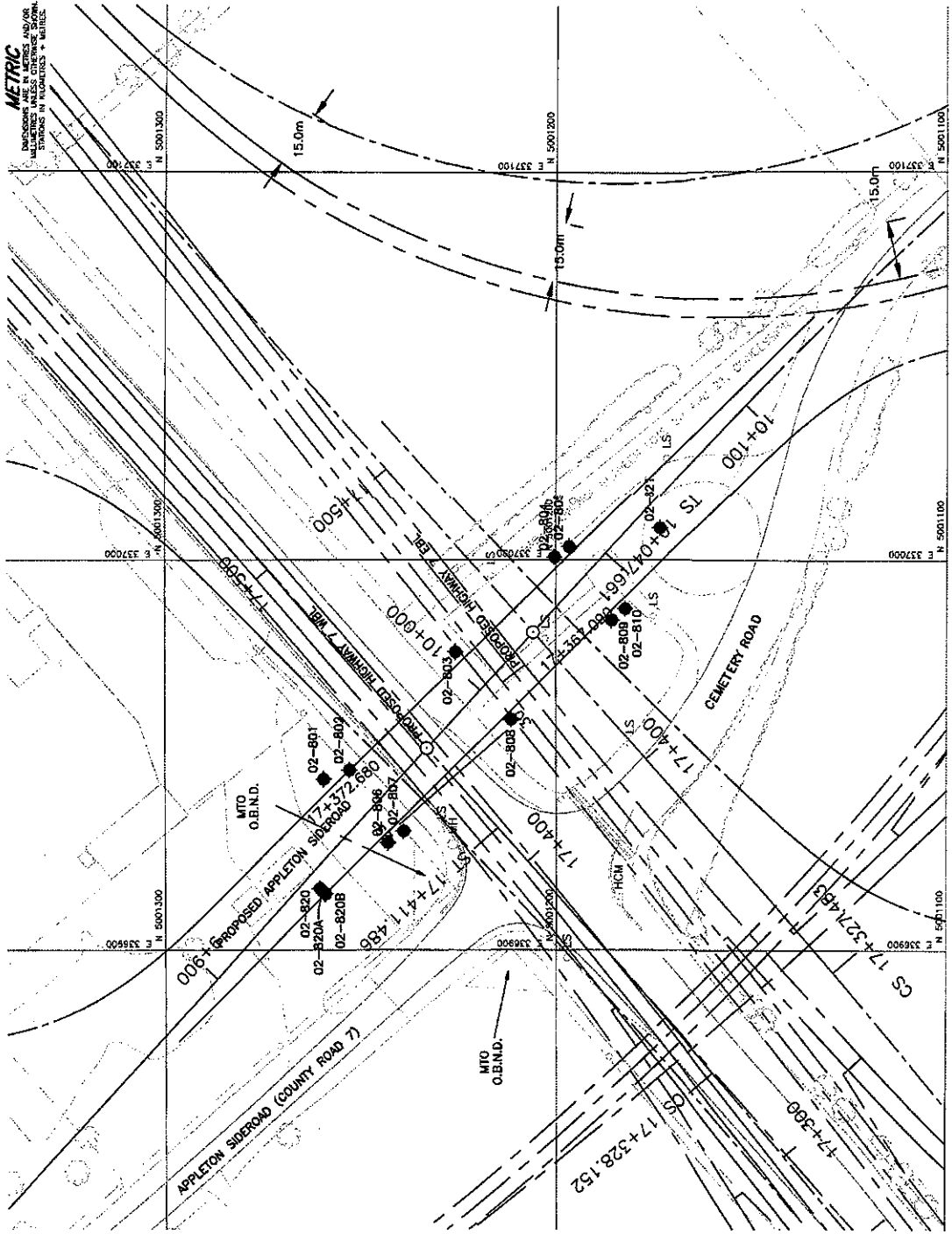


LEGEND

● Borehole - Current Installation

No.	ELEVATION	CO-ORDINATES	INSTRUMENT	LASTING
32-01	125.5	5001259.2	5001259.2	336844.7
32-02	125.1	5001258.6	5001258.6	336846.0
32-03	125.1	5001258.6	5001258.6	336846.0
32-04	130.3	5001260.5	5001260.5	337003.7
32-05	130.4	5001186.7	5001186.7	337003.4
32-06	125.3	5001238.9	5001238.9	336830.4
32-07	125.3	5001211.6	5001211.6	336859.1
32-08	125.5	5001182.5	5001182.5	336867.4
32-09	125.7	5001260.2	5001260.2	336815.5
32-10	125.8	5001248.8	5001248.8	336814.6
32-11	125.7	5001248.8	5001248.8	336814.6
32-12	130.1	5001173.5	5001173.5	337008.2

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. Descriptive information contained in this report and related documents is specifically excluded in accordance with section GC 2.01 of NPS General Conditions.

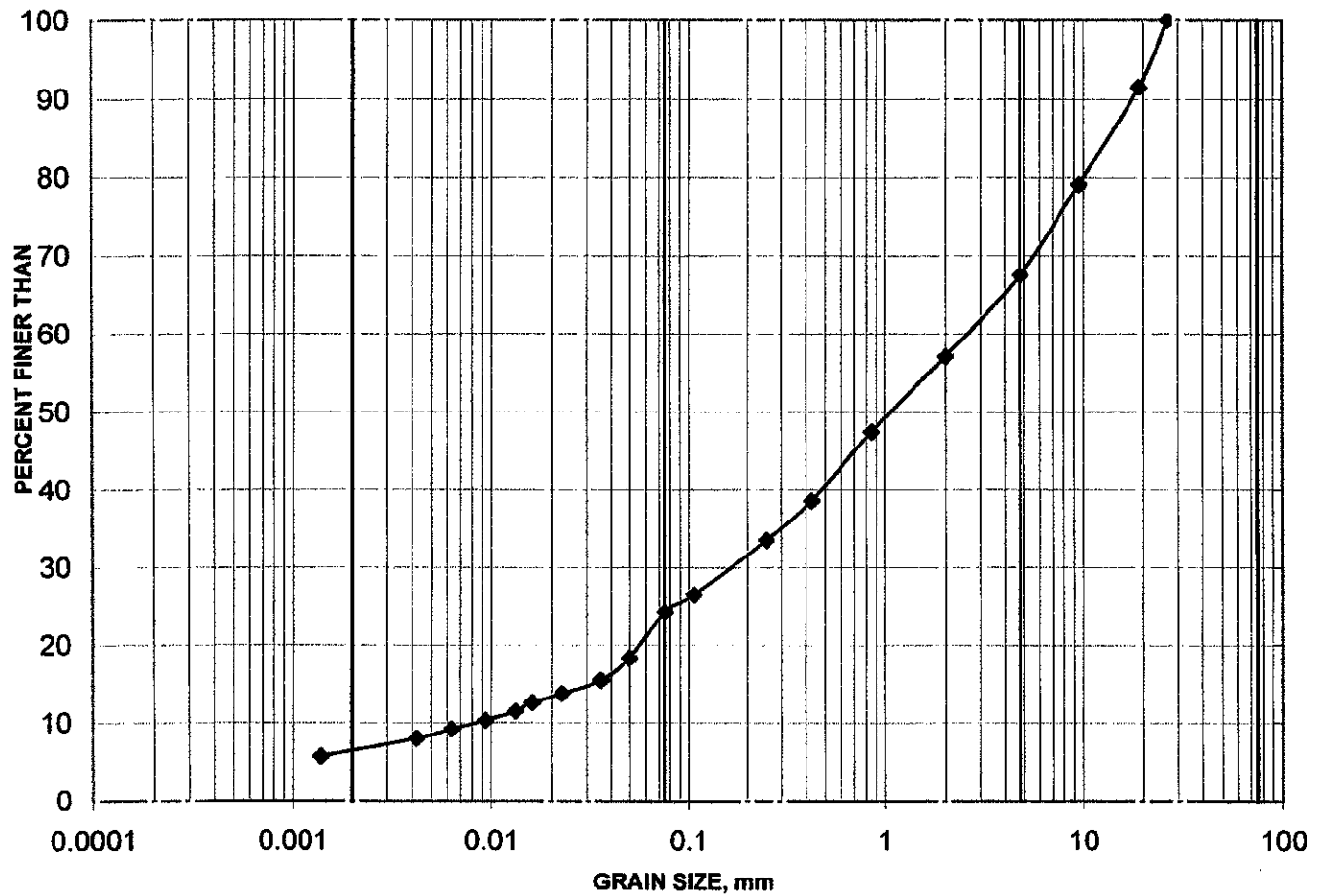


NAME	DATE	BY	REASON
Securities No.			
JANUARY 7			PROJECT NO. 021-1155
GROUPING	CHRG. LOC	CHRG. LOC	DATE, APTS., 2004
GROUPING	CHRG. LOC	CHRG. LOC	APPO. LOC
			SHEET 15-225
			IND. 1

GRAIN SIZE DISTRIBUTION TEST RESULT

Fill

FIGURE 1



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

Borehole	Sample	Depth (m)
02-804	1	0.8-1.4

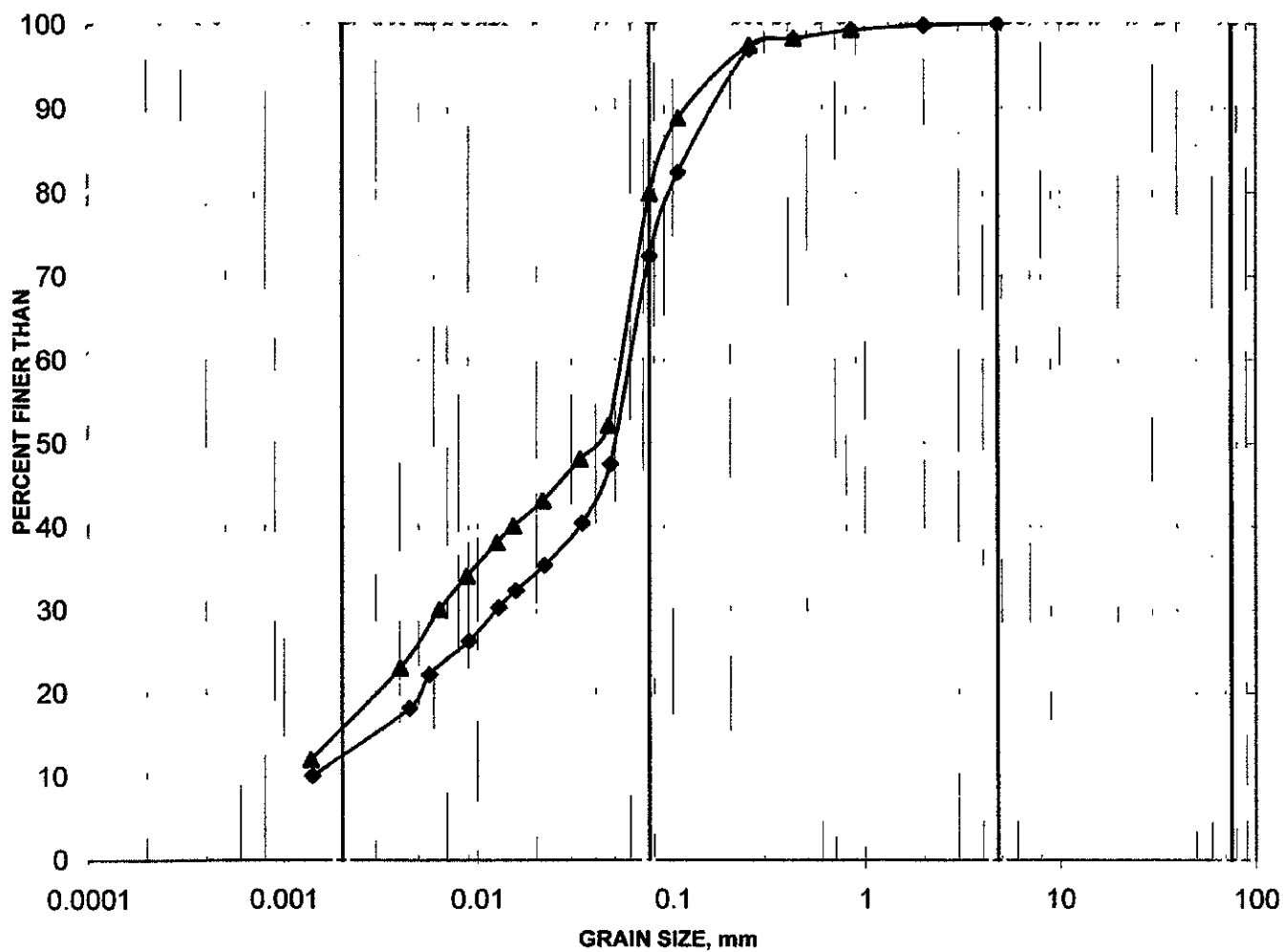
Project: 021-1155

Golder Associates

GRAIN SIZE DISTRIBUTION TEST RESULTS

Layered Clayey Silt and Silty Clay

FIGURE 2



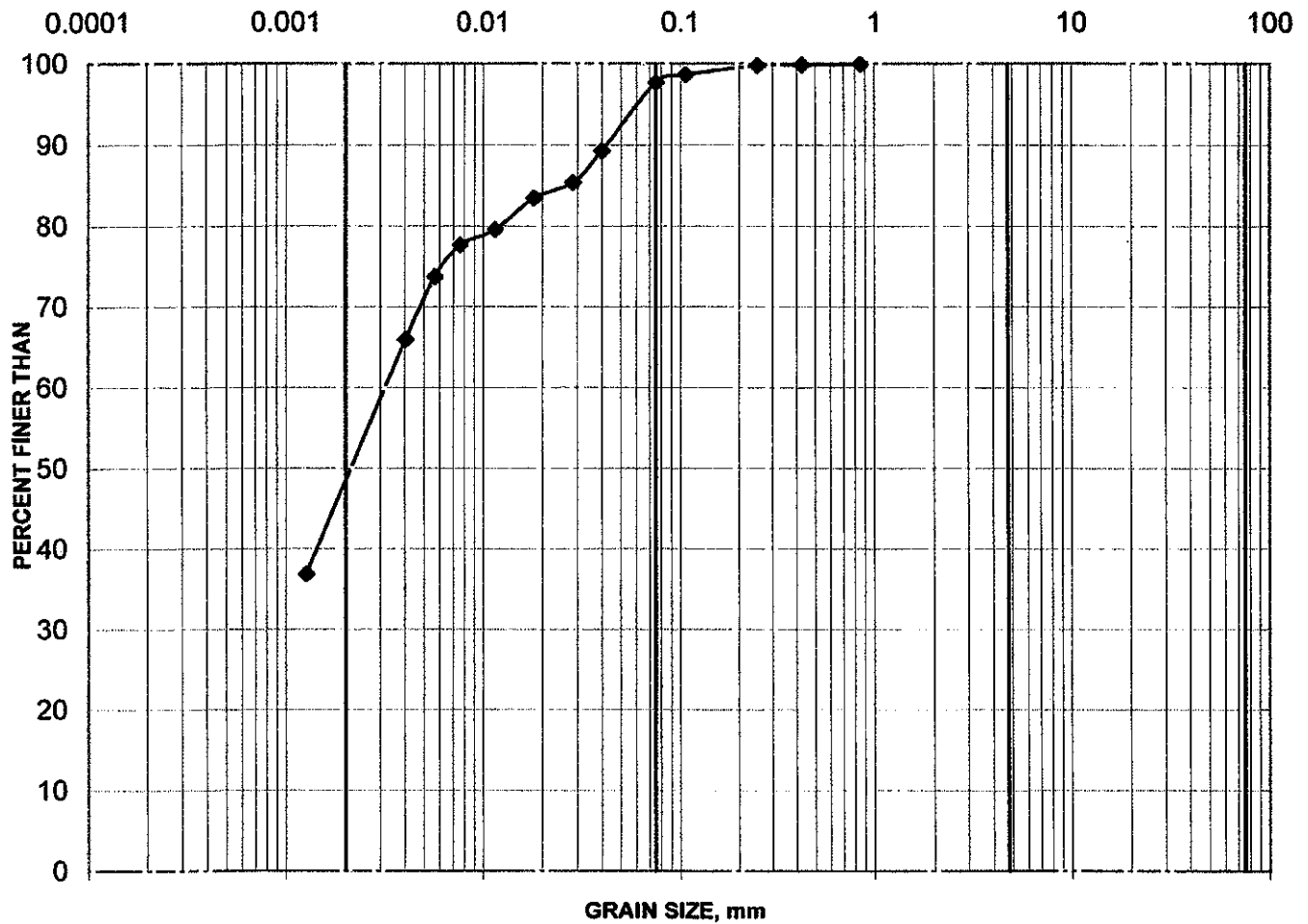
SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND SIZE			GRAVEL SIZE		

Borehole	Sample	Depth (m)
—◆— 02-808	1	0.8-1.4
—▲— 02-809	1	0.8-1.4

GRAIN SIZE DISTRIBUTION TEST RESULT

Weathered Clay Crust

FIGURE 3



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

Borehole	Sample	Depth (m)
—◆— 02-802	3	2.3-2.9

Oct 75, FF-S-21

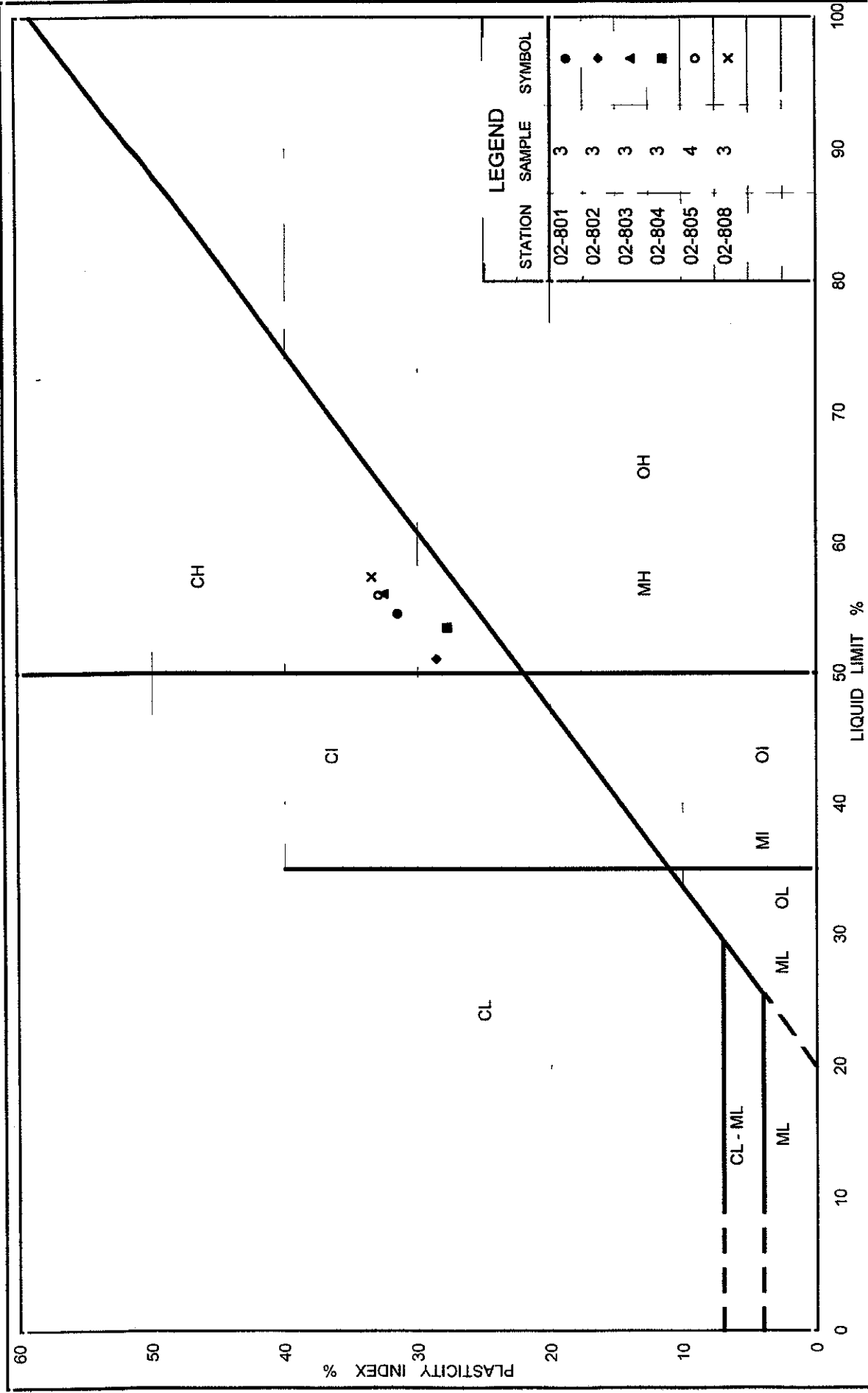
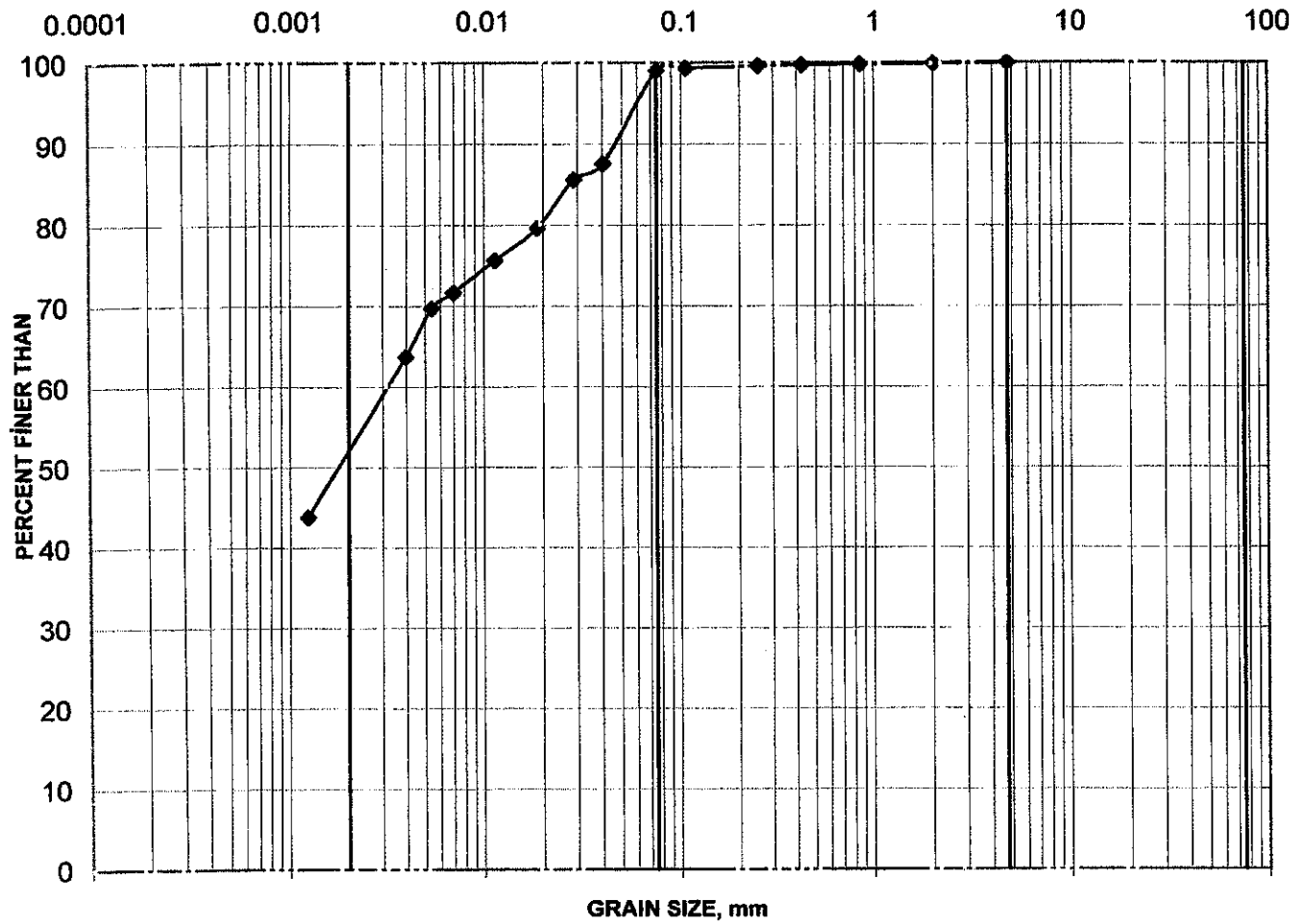


FIGURE 4

PLASTICITY CHART
Weathered Clay Crust

Project No. 021-1155-8



FIGURE 5

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

Borehole	Sample	Depth (m)
◆ 02-809	6	4.6-5.2

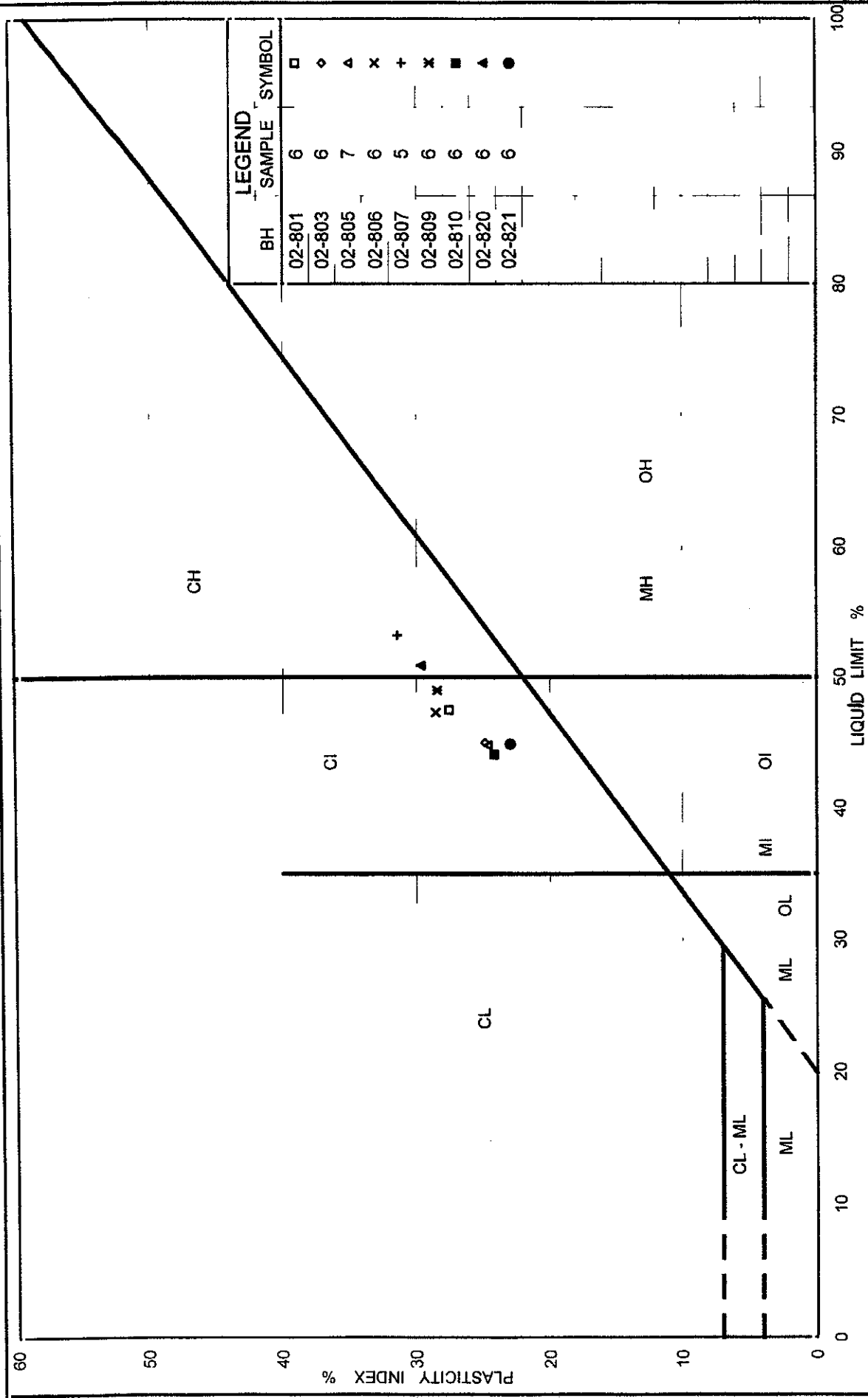
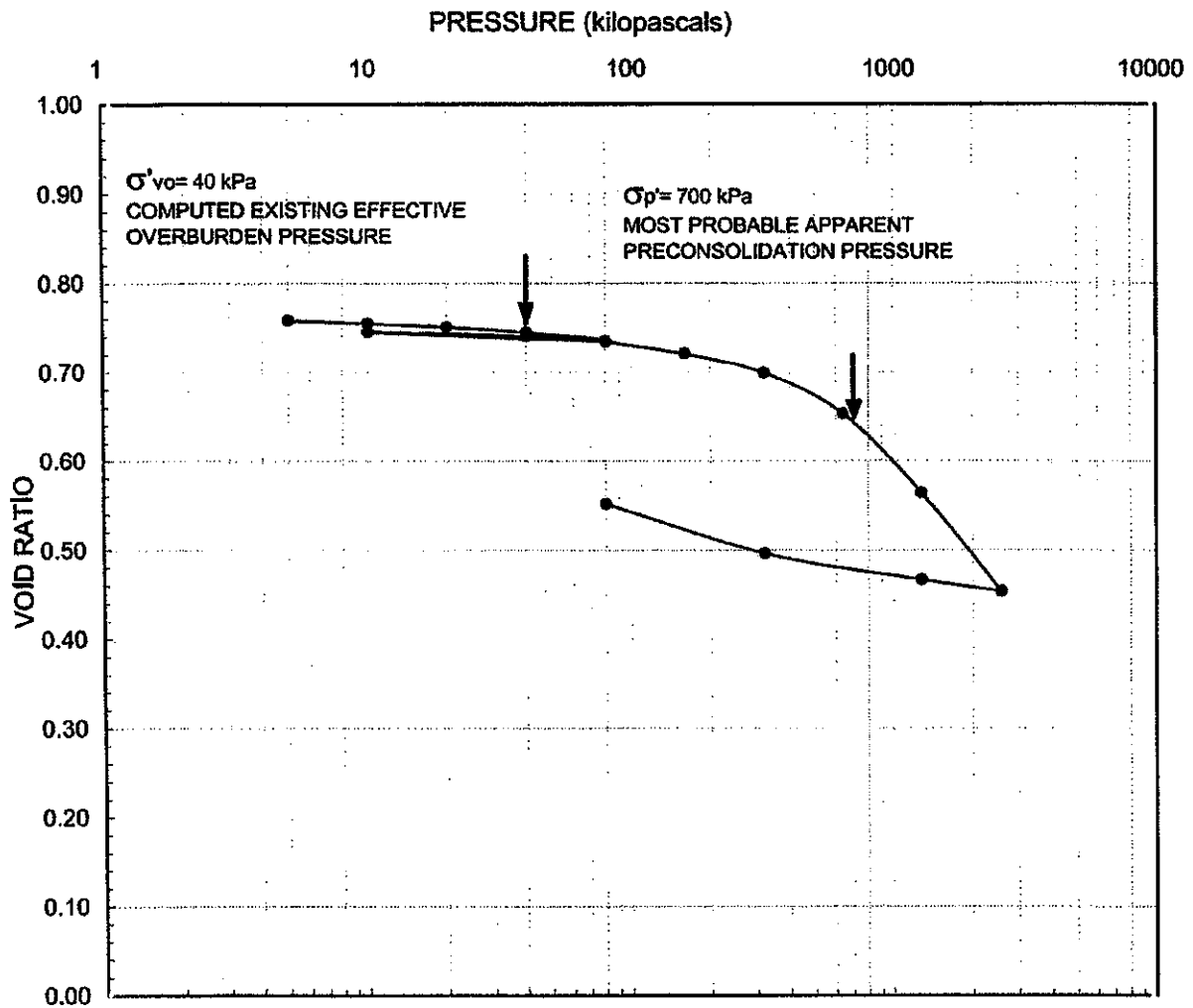


FIGURE 6

PLASTICITY CHART
Grey Silty Clay to Clay

Project No. 021-1155-8



LEGEND

Borehole: 02-820f	$w_l = 28\%$	$S_o = 98\%$
Sample: 1	$w_f = 25\%$	$C_c = 0.37$
Depth (m): 5.0	$w_l = 46\%$	$C_r = 0.012$
	$w_p = 29\%$	



SCALE	AS SHOWN
DATE	08/19/04
DESIGN	
CADD	EWK

CONSOLIDATION TEST RESULTS

FILE No.	Consolidation summary
PROJECT No.	021-1155

CHECK	<i>mw</i>
REVIEW	

FIGURE

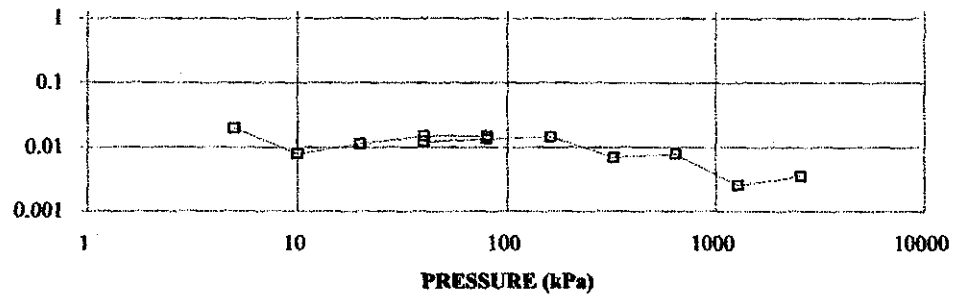
7a

OEDOMETER CONSOLIDATION SUMMARY

FIGURE 7b

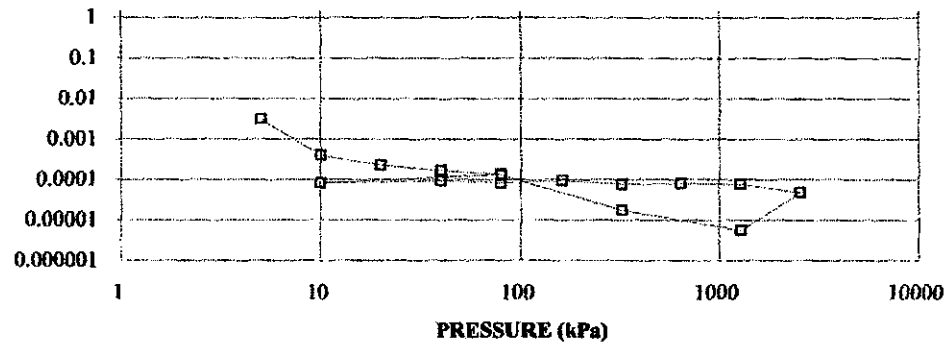
COEFFICIENT OF CONSOLIDATION, cm^2/s

CONSOLIDATION TEST
cv cm^2/s vs PRESSURE (kPa)

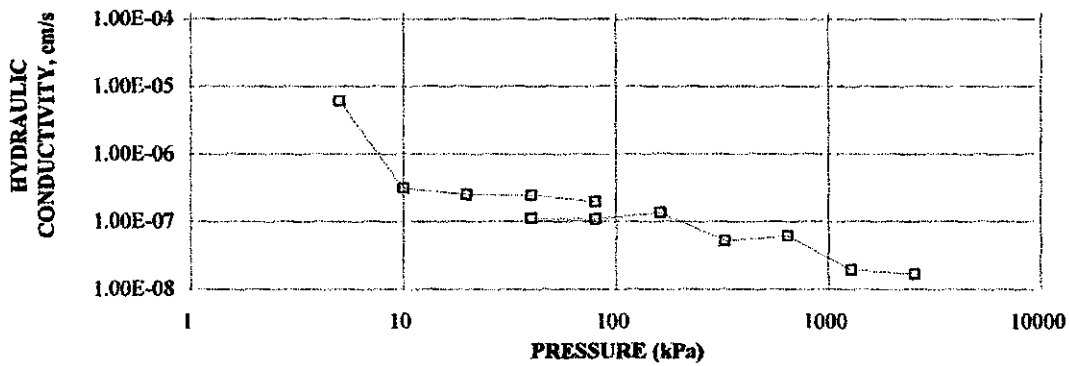


VOLUME
COMPRESSIBILITY,
 m^2/kN

CONSOLIDATION TEST
mv, m^2/kN vs PRESSURE (kPa)



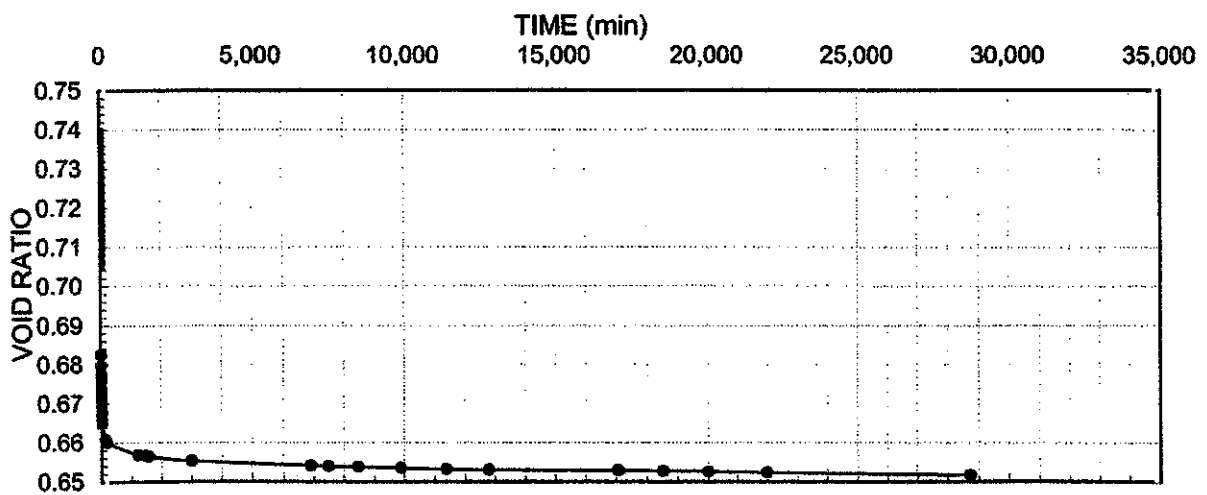
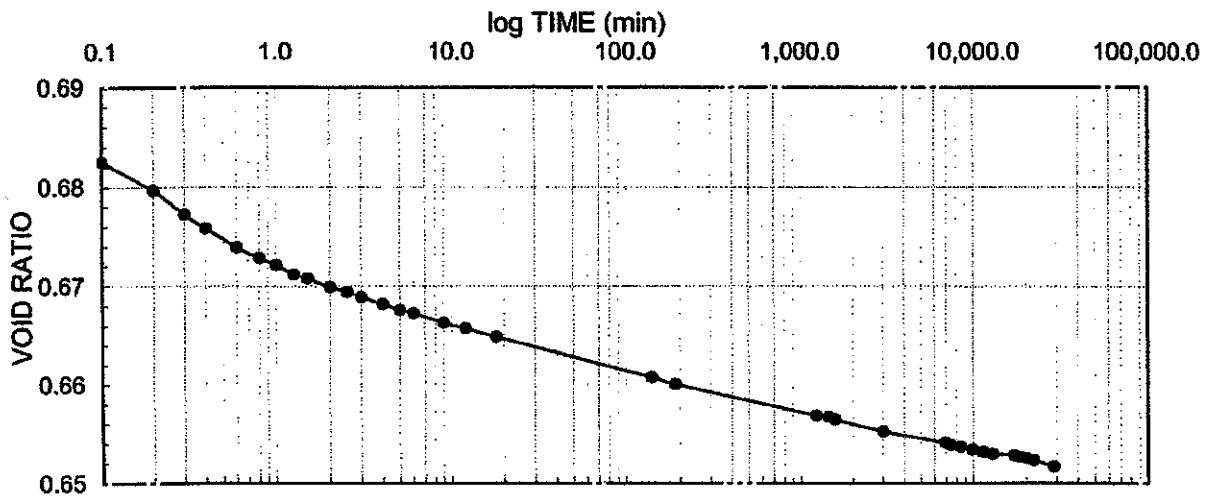
CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE



Project No. 021-1155

Golder Associates

Load = 280 kPa



LEGEND

Borehole: 02-820B

$C_{\alpha} = 0.0038$

Sample: 1

Depth (m): 5.00



FILE No. Consolidation summary
PROJECT No. 021-1155 REV. 0

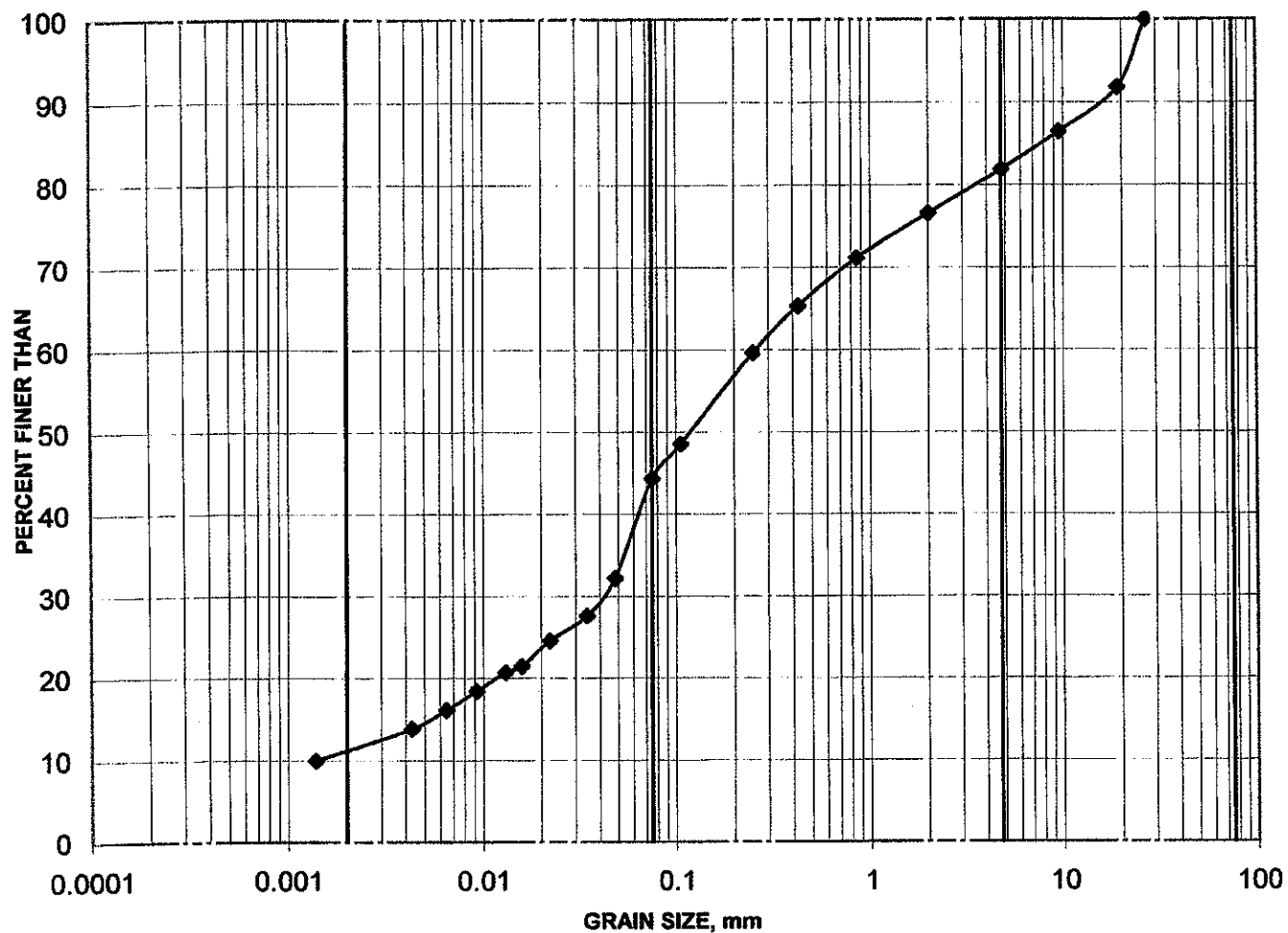
SCALE	AS SHOWN
DATE	08/19/04
DESIGN	
CADD	EWK
CHECK	
REVIEW	

TITLE	
SUMMARY OF SECONDARY COMPRESSION TEST	
FIGURE	7c

GRAIN SIZE DISTRIBUTION TEST RESULT

Silty Sand to Sandy Silt Till

FIGURE 8



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

Borehole	Sample	Depth (m)
02-808	7	5.4-6.0

August 2004

021-1155-8

APPENDIX A

**CERTIFICATE OF ANALYSIS
SLS LAKEFIELD RESEARCH LIMITED
REPORT NO. CA9559-JUL03**

SGS Lakefield Research Limited
P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - K0L 2H0
Phone: 705-852-2038 FAX: 705-852-6441

Golder Associates
Attn : Emily Kwok

1796 Courtwood Cresent
Ottawa, Ontario, K2C 2B5
Canada

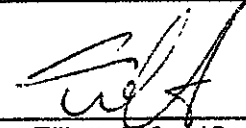
Phone: (613) 224-5864
Fax: (613) 224-9928
e-mail: ekwok@golder.com

Thursday, August 07, 2003

Date Rec. : 30 July 2003
LR Report : CA9559-JUL03
Project : 2302189
Client Ref : vProject #'s
021-1155(5050)

CERTIFICATE OF ANALYSIS

Sample ID	SiO2 %	Al2O3 %	Fe2O3 %	MgO %	CaO %	Na2O %	K2O %	TiO2 %	P2O5 %	MnO %	Cr2O3 %	V2O5 %	LOI %	Sum %
1: 02-801 A	61.6	13.7	3.21	2.75	4.58	0.84	5.67	0.55	0.30	0.03	0.02	< 0.01	7.03	100.3
2: 02-801 B	75.9	5.47	1.20	1.09	6.26	0.42	2.56	0.27	0.65	0.03	0.01	< 0.01	6.10	100.0
3: 02-801 C	3.12	0.35	2.34	15.6	34.0	< 0.05	0.16	0.02	< 0.01	0.31	< 0.01	< 0.01	44.1	100.0


Tim Elliott, B.Sc. (Geol)
Senior Project Coordinator

