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Golder Associates

GEOCRES No.
31G-221

DRAFT

FOUNDATION
INVESTIGATION AND DESIGN REPORT
HIGHWAY 417E-7W RAMP
STRUCTURE SITE 3-722
HIGHWAY 7 TWINNING FROM HIGHWAY 417
TO 3 KM WEST OF JINKINSON ROAD
G.W.P. 256-99-00

Submitted to:

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February 2003

021-1155-1

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FOUNDATION INVESTIGATION REPORT
HIGHWAY 417E-7W RAMP
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Golder Associates

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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 in the Regional Municipality of Ottawa-Carleton, and in Beckwith Township in Lanark County. The sections of Highway 7 included in this assignment extend from Highway 417 westerly 7 km to 3 km west of Jinkinson Road (W.P. 256-99-00), and from 3 km west of Jinkinson Road westerly to Carleton Place (W.P. 251-99-00 and 252-99-00). Foundation investigation services are also required as part of this assignment for the widening of Highway 417 from the Highway 417-7 interchange easterly to Carp River (W.P. 458-98-00).

Foundation investigation services are required for the following components:

- **W.P. 256-99-00:** New structures at the Highway 417E-7W ramp and Hazeldean Road, including a high fill embankment along the Highway 417E-7W ramp, and overhead signs.

- **W.P. 251-99-00 and 252-99-00:** Five new structures at Appleton Road, Ashton Station Road, Dwyer Hill Road, the Trans-Canada Trail, and Lavallee Creek.

- **W.P. 458-98-00:** Widening of two existing structures (the Carp River bridge and CN Rail overpass) into the existing Highway 417 median area, a 900 m long section of high fill embankment within the Highway 417 median in the vicinity of the CN Rail overpass, high mast light poles, and overhead signs.

This report addresses the new Highway 417E-7W ramp structure and the associated high fill embankment along this ramp.

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

2.0 SITE DESCRIPTION

The proposed Highway 417E-7W ramp and structure are located immediately northwest of the existing bi-directional Highway 417-7 ramp, in West Carleton Township in the Regional Municipality of Ottawa-Carleton. The proposed ramp structure is designated as MTO's Structure Site 3-722; the existing ramp structure is designated as MTO's Structure Site 3-288.

The terrain at the site is flat to gently undulating, with the natural ground surface varying from about Elevation 128 m to 129 m. The existing Highway 417 grade at the site is at about Elevation 129.5 m to 130 m, slightly above the surrounding natural grade. The site is poorly drained, as evidenced by the presence of standing water and cat-tails in the existing Highway 417 ditches, and the occurrence of surficial organic soil in areas of the site. Further, during a previous subsurface investigation[1] for the existing ramp structure carried out in April 1971, the southwestern portion of the site was flooded by about 150 mm to 200 mm of standing water. To the north and south of the Highway 417 corridor, the site is forested.

The existing Highway 417-7 ramp has been constructed on embankment fill that is up to about 9 m high, with the ramp grade at about Elevation 137 m in the immediate vicinity of the existing structure. According to the *General Layout* and *Foundation Layout* drawings[2] for the existing two-span ramp structure, the abutments are supported on battered steel H-piles driven to bedrock, with the underside of the pile cap at about Elevation 131.5 m. The existing centre pier is supported on a 4.3 m square spread footing, founded on the bedrock at about Elevation 126 m.

[1] Ministry of Transportation, Ontario's GEOCRES No. 31G5-99: *Foundation Investigation Report for Proposed Underpass Structure at the Crossing of Highway 417 and Highway 7 Connection – Township of Huntley, Regional Municipality of Ottawa-Carleton.* Report prepared by Department of Highways, Ontario, dated May 1971.

[2] Ministry of Transportation, Ontario's GEOCRES No. 31G5-99: General Layout Drawing No. D-7072-1 and Foundation Layout Drawing No. D-7072-3, dated March 1972.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out for the Highway 417E-7W ramp structure and associated high fill embankment in November and December 2002, at which time a total of twelve boreholes (Boreholes 02-101 to 02-112) were advanced in the vicinity of the proposed structure foundations and immediate approach embankments. A further five boreholes (Boreholes 02-120 to 02-124) were advanced within the limits of the proposed high fill embankment that will extend from about Station 11+450 to 11+850 along the proposed ramp alignment.

The boreholes were drilled using a bombardier-mounted drill rig supplied and operated by Marathon Drilling Ltd. of Ottawa, Ontario. All of the boreholes were advanced using hollow stem augers, to auger and/or sampler refusal which occurred at depths between 0.9 m and 4.7 m below the existing ground surface. Samples of the overburden were obtained at 0.75 m intervals of depth using 50 mm outside diameter split-spoon samplers driven with an automatic hammer, in accordance with the Standard Penetration Test (SPT) procedure. In eight of the ten boreholes advanced at the proposed foundation locations, the boreholes were advanced 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 six piezometers were installed to monitor the groundwater level(s) at the site: four piezometers were installed within the overburden, and two within the bedrock.

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

The borehole locations and ground surface elevations were established by MMM surveyors or were determined by Golder Associates relative to points staked by MMM. The borehole locations, including MTM NAD83 northing and easting coordinates and ground surface elevations referenced to geodetic datum, are summarized in the following table and are shown on Drawings 1 and 2. *(NOTE: Draft preliminary drawings showing borehole locations only are provided in this report. Drawings 1 and 2 will be provided to this draft report once electronic files of General Arrangement Drawing and alignment are received.)*

Borehole Number	*Borehole Location*	*MTM NAD83 Northing (m)*	*MTM NAD83 Easting (m)*	*Ground Surface Elevation (m)*
02-101	East abutment			128.6 m
02-102	East abutment			128.2 m
02-103	East approach			129.8 m
02-104	West abutment			128.4 m
02-105	West abutment			128.4 m
02-106	West approach			128.3 m
02-107	Centre pier			128.4 m
02-108	Centre pier			128.4 m
02-109	East abutment			128.3 m
02-110	East abutment			128.4 m
02-111	West abutment			128.4 m
02-112	West abutment			128.7 m
02-120	Station 11+845			130.2 m
02-121	Station 11+607			128.6 m
02-122	Station 11+550			129.2 m
02-123	Station 11+499			129.4 m
02-124	Station 11+453			129.2 m

NOTE: Northing and easting coordinates will be provided once electronic drawings are received, to enable plotting of our borehole locations relative to survey points (with known coordinates) staked by MMM.

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*[3], 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.[4] 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.[3]

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.[3] 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.[4]

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[3] 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.

[4] 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, and a further five boreholes were advanced between Stations 11+450 and 11+850, within the limits of the proposed high fill embankment associated with the ramp structure. The borehole locations and ground surface elevations are shown on Drawings 1 and 2. *(These Borehole Locations and Soil Strata Drawings will be provided to the draft report once electronic files of the General Arrangement Drawing and alignment are received.)*

The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of in-situ and laboratory testing are given on the Record of Borehole sheets and Figures 1 to 3. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

In summary, the soils encountered at this site consist of a layer of topsoil or peat between 100 mm and 750 mm thick, overlying relatively thin overburden soils consisting of sands, gravels and silts overlying silty sand till to sand and silt till. These surficial soils are, in turn, underlain by limestone bedrock that was encountered between about 1 m and 5 m depth, but typically between about 2 m and 2.5 m depth (at about Elevation 126 m to 126.5 m). These subsurface conditions are consistent with those encountered during the Department of Highways, Ontario investigation in April 1971 for the existing Highway 417-7 ramp structure near this site.

A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections, and stratigraphic profiles and sections of this site are shown on Drawings 1 and 2. *(These drawings will be provided once electronic files are received.)*

4.2.1 Fill

Fill, associated with the construction of the existing Highway 417 and Highway 417-7 ramp, was encountered in two of the boreholes advanced at this site. In Borehole 02-112, adjacent to the existing drainage ditch along the southwest side of the Highway 417 eastbound lanes, about 0.5 m of **silty sand fill** is present. In Borehole 02-120, northeast of the existing Highway 417 westbound lanes and adjacent to the existing ramp embankment, approximately 1.4 m of **silty sand fill** was encountered. In both locations, the silty sand fill contains trace organics.

4.2.2 Topsoil and Peat

With the exception of those borehole locations noted below, between 100 mm and 750 mm of topsoil was encountered in each of the boreholes advanced at this site. The topsoil is present at ground surface, overlying the native soil deposits at the site.

In Boreholes 02-111 and 02-112, advanced adjacent to the drainage ditch on the southwest side of the Highway 417 eastbound lanes, 300 mm and 450 mm of peat was encountered. The peat is present at ground surface at the location of Borehole 02-111, but underlies about 0.5 m of existing fill in Borehole 02-112.

In Borehole 02-120, advanced near the existing ramp embankment to the northeast of the Highway 417 corridor, no topsoil or peat was encountered above or below the existing fill.

4.2.3 Surficial Sands ~~and~~ Silts and Gravels

Below the existing fill and topsoil or peat lie interlayered cohesionless soils with a total thickness of between 1 m and 2 m, except to the northeast of the existing Highway 417 corridor where thicknesses of about 3 m and 2.5 m were encountered in Boreholes 02-103 and 02-120, respectively. Typically, the upper portion of the deposit ranges in composition from sand and gravel to silty sand, while the lower portion of the deposit consists of silt containing trace sand and clay. At some borehole locations the silt portion overlies the more coarse-grained portion of the deposit. In Boreholes 02-112 and 02-120, 0.3 m and 0.6 m thick layers of clayey silt to silty clay are present within the deposit. Grain size distribution test results carried out on four samples of the sand and gravel to silty sand portions of the deposit are shown on Figure 1, and grain size distribution test results carried out on three samples of the silt portion of the deposit are shown on Figure 2.

Measured Standard Penetration Test (SPT) "N" values in this deposit range from 5 to 40 blows per 0.3 m of penetration, but are typically between 5 and 30 blows, with an average of 18 blows per 0.3 m of penetration. This surficial deposit therefore varies from loose to dense, but is generally compact.

4.2.4 Silty Sand Till to Sand and Silt Till

The surficial sands and silts, where present, are underlain by a till deposit that grades in composition from silty sand to sand and silt, with some gravel and trace clay. Cobbles and boulders were noted or inferred within the till in some of the boreholes. Grain size distribution test results obtained on three samples of this till are shown on Figure 3 following the text of this report. The silty sand to sand and silt till varies from loose to very dense, based on measured SPT

"N" values of 5 to 90 blows per 0.3 m of penetration; however, this till deposit is typically compact to dense, based on typical SPT "N" values of 15 to 40 blows per 0.3 m of penetration.

The till stratum ranges generally ranges in thickness from 0.2 m to 0.8 m, although it is between 1 m and 1.5 m thick in three of the boreholes (Boreholes 02-121 to 02-123) advanced along the proposed ramp alignment to the south of the existing Highway 417 corridor. The base of this till deposit was encountered between about Elevations 126.5 m and 126.0 m in Boreholes 02-101 to 02-112, which were advanced in the vicinity of the proposed structure.

4.2.5 Limestone Bedrock

Limestone bedrock underlies the till deposit at this site. In the boreholes put down at the proposed bridge foundations, the surface of the bedrock was encountered between Elevation 126 m and 126.5 m. Typically, the depth to bedrock below existing ground surface is about 2 m to 3 m in the area of the proposed bridge foundations, increasing to about 4.7 m depth in Borehole 02-120 to the east of the bridge. The following table summarizes the bedrock surface depth and elevation as encountered at the borehole locations. It should be noted that bedrock was cored in eight of the boreholes; the surface of the limestone bedrock was inferred in the remaining nine boreholes by refusal to split-spoon sampler and/or auger advance.

Borehole Number	*Borehole Location*	*Ground Surface Elevation*	*Depth to Bedrock*	*Bedrock Surface Elevation*
02-101	East abutment	128.6 m	2.2 m	126.4 m (Cored)
02-102	East abutment	128.2 m	2.0 m	126.2 m (Cored)
02-103	East approach	129.8 m	3.7 m	126.1 m
02-104	West abutment	128.4 m	2.1 m	126.2 m (Cored)
02-105	West abutment	128.4 m	2.0 m	126.5 m (Cored)
02-106	West approach	128.3 m	1.9 m	126.3 m
02-107	Centre pier	128.4 m	2.4 m	126.0 m (Cored)
02-108	Centre pier	128.4 m	2.0 m	126.4 m (Cored)
02-109	East abutment	128.3 m	2.2 m	126.1 m (Cored)
02-110	East abutment	128.4 m	2.3 m	126.1 m (Cored)
02-111	West abutment	128.4 m	2.2 m	126.2 m
02-112	West abutment	128.7 m	2.7 m	126.0 m
02-120	Station 11+845	130.2 m	4.7 m	125.5 m
02-121	Station 11+607	128.6 m	1.7 m	126.9 m
02-122	Station 11+550	129.2 m	2.1 m	127.1 m
02-123	Station 11+499	129.4 m	1.7 m	127.7 m
02-124	Station 11+453	129.2 m	0.9 m	128.3 m

(1)

The limestone bedrock at the site is a member of the Ottawa Formation; it is moderately strong, thinly- to medium-bedded, and contains characteristic shale partings and interbeds. Rock Quality Designation (RQD) values measured on recovered bedrock core samples ranged from 0 to 65 per cent in the upper 1 m of the bedrock, and from 16 to 98 per cent (but typically from about 65 to 98 per cent) in the lower 2 m of the recovered bedrock core. The discontinuities observed in the rock core are typically horizontal to sub-horizontal, associated with the bedding planes.

· jointing
· weathering
· quality
· fracturing

4.3 Groundwater Conditions

Water was encountered in all of the boreholes during drilling; in the boreholes where water was not introduced for coring operations, the water level was at depths ranging from about 0.3 m to 2.3 m below the existing ground surface. Four piezometers were installed within the overburden soil deposits, and two piezometers were sealed within the limestone bedrock to monitor the groundwater level(s) at the site.

The water level measured in the piezometers on January 8, 2003 varied from Elevation 127.5 m to 128.3 m, typically about 1.5 m to 2 m above the surface of the limestone bedrock. The measured groundwater levels are summarized in the following table:

Borehole No.	*Piezometer Screen Interval*	*Water Level on Jan 8, 2003*	
		Elevation	*Depth*
02-101	Bedrock below Elevation 125.3 m	127.9 m	0.7 m
02-103	Overburden below Elevation 127.4 m	127.5 m	1.8 m
02-104	Bedrock below Elevation 125.9 m	128.0 m	0.4 m
02-106	Overburden below Elevation 127.2 m	128.3 m	0.0 m
02-120	Overburden below Elevation 126.3 m	128.1 m	2.1 m
02-122	Overburden below Elevation 127.7 m	128.6 m	0.6 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
HIGHWAY 417E-7W RAMP
STRUCTURE SITE 3-722
HIGHWAY 7 TWINNING FROM HIGHWAY 417
TO 3 KM WEST OF JINKINSON ROAD
G.W.P. 256-99-00

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides foundation design recommendations for the proposed Highway 417E – 7W ramp structure. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation at this site. The interpretation and recommendations provided are intended only to provide the designers with sufficient information to assess the feasible foundation alternatives and to design the proposed structure foundations. As such, where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

It is understood that the proposed Highway 417E-7W ramp structure will be two spans, with a central pier to be located within the existing Highway 417 median. In order to eliminate the requirement for expansion joints, three alternative integral or semi-integral abutment configurations 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; and
- 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 the use of RSS walls in a false abutment configuration 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. However, it is further understood that, in the absence of detailed subsurface data, MTO Eastern Region had expressed concern regarding the potential for settlement of the engineered fill and/or the supporting soil stratum, leading to separation of the joints in the precast facing panels. The results of the current subsurface investigation indicate that the subsoils at the site are essentially competent granular materials and, as such, are suitable for the use of RSS walls in a false abutment configuration.

advantages/disadvantages/costs/Risks/[illegible]

Option 1
Shallow

bedrock.
Perched Abutments

5.2 Bridge and Retaining Wall Foundation Options

At the proposed ramp structure site, the natural ground surface varies from about Elevation 128 m to 129 m and the existing Highway 417 grade is slightly higher, at about Elevation 129.5 m to 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 ramp grade at the structure site is about Elevation 138 m to 138.5 m, about 9 m to 10 m above the existing natural grade.

The native soils at the site consist of topsoil and peat overlying generally compact surficial sands and silts, in turn underlain by a generally compact to dense silty sand till to sand and silt till stratum. These overburden soils are underlain by moderately strong limestone bedrock, the surface of which was encountered at about Elevation 126 m to 126.5 m, about 2 m to 2.5 m below the existing ground surface, in the boreholes in the area of the bridge foundations. The limestone bedrock is suitable for support of the proposed pier, abutments and associated retaining walls, such as concrete cantilever retaining walls, on shallow foundations. The overburden soils at the site are suitable for the support of RSS walls, either as wingwalls or in front of the abutments.

Since integral abutments are under consideration, steel H-piles can also be considered for support of the abutments. Given the proposed ramp grade of about Elevation 138 m to 138.5, an assumed underside of pile cap at about Elevation 135 m, and the limestone bedrock surface at about Elevation 126.5 m to 126 m, it is estimated that the pile length will be approximately 8.5 m to 9 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 spread footings or steel H-pile foundations, drilled shaft foundations resting on or socketted into the limestone bedrock could be used for support of the abutments and centre pier. This option has the advantage of minimizing the groundwater control that would be required to advance spread footing excavations to bedrock.

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

5.3 Spread Footings

The bridge pier, abutments and any associated concrete cantilever wing walls / retaining walls may be supported on spread footings placed on the properly prepared limestone bedrock. The surface of the bedrock was encountered in the boreholes between Elevations 126 m and 126.5 m, as summarized in the following table.

Foundation Element	*Borehole Numbers*	*Depth to Bedrock*	*Bedrock Surface Elevation*
North abutment	02-101, 02-102, 02-109, 02-110	2.0 m to 2.3 m	126.1 m to 126.4 m
Centre pier	02-107, 02-108	2.0 m to 2.4 m	126.0 m to 126.4 m
South abutment	02-104, 02-105, 02-111, 02-112	2.0 m to 2.7 m	126.0 m to 126.5 m

Based on the borehole results, there is some variability in the bedrock surface within the limits of each foundation element. In addition, the upper portion of the bedrock is, in local areas, highly fractured (RQD values of less than 40 per cent, as encountered in Boreholes 02-104, 02-108 and 02-109), and it may be necessary to subexcavate loose or fractured rock from within the foundation footprints. For design, the following options for founding levels may be considered:

- A founding level of Elevation 126.5 m may be assumed. In this case, the bedrock surface would have to be exposed and cleaned, and then mass concrete would be placed to raise the grade to the founding level. Provision should be made in the Contract Documents for mass concrete placement to accommodate variations in the bedrock surface. The benefit of this approach is that excavation into the weak to medium strong bedrock is avoided.

- Alternatively, a design founding level of Elevation 125.7 m may be assumed. In this case, excavation of the higher portions of the bedrock will be required within the foundation footprints. Based on the borehole results, subexcavation of up to about 0.8 m of bedrock will be required in some foundation areas. It is noted that the bedrock is weak to moderately strong (corresponding to unconfined compressive strengths in the range of 5 MPa to 50 MPa), making excavation relatively difficult particularly where only small depths are needed. Bedrock excavation could be carried out using hoe ramming techniques; however, line drilling and pre-shearing techniques, if properly executed and inspected, provide better control over the configuration of the founding surface. comment on degree of difficulty of rock excavat

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

It is noted that footing excavations to expose the bedrock surface will extend through water-bearing sands, gravels, silts and cohesionless tills. A suitable dewatering scheme will be required in order to maintain a stable excavation.

5.3.1 ~~Axial~~ Geotechnical Bearing Resistance

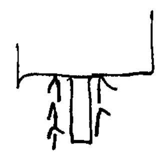
Spread footings placed on the surface of the properly prepared limestone bedrock may be designed based on a factored geotechnical bearing resistance at Ultimate Limit States (ULS) of 2,000 kPa. conservative The geotechnical bearing resistance at Serviceability Limit States (SLS) for 25 mm of settlement will be greater than the factored axial resistance at ULS, since the limestone bedrock is considered to be an unyielding material; as such, ULS conditions will govern for this foundation type.

The geotechnical resistances provided herein are given under the assumption that the loads will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with the *Canadian Highway Bridge Design Code (CHBDC)*. — clause #. (3)

5.3.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footings and the limestone bedrock should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, tan N', may be taken as 0.70 for cast-in-place concrete footings constructed on the bedrock. This represents an unfactored value; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

If necessary, the sliding resistance can be supplemented by dowelling into the bedrock. A factored value of 500 kPa may be assumed for the grout-to-rock bond stress for ULS design. The dowels should have a minimum embedded length within the bedrock of 1 m, and the structural strength of the dowel and compressive strength of the grout should not be exceeded. Provision should also be made in the contract for longer dowels or for tensioned bolts in the event that there are adversely oriented joints in the rock under the footing that could potentially result in a sliding failure toward the bedrock surface.



(4) horizontal resistance of rock.

5.3.3 Frost Protection

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

5.4 Steel H-Pile Foundations

Steel H-piles driven to found on the 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 ramp grade at Elevation 138 m to 138.5 m and the assumed pile cap base at Elevation 135 m, the pile or drilled shaft length will be approximately 8.5 m to 9 m without socketting into bedrock. If necessary, for additional pile length and/or to resist seismic forces, the piles could be placed within the bedrock. The limestone bedrock is weak to moderately strong, however, and this would require socket formation using coring or churn drilling to advance the hole. Alternatively, consideration could be given to open excavation to the bedrock surface and trenching / excavating into the bedrock to

(5) Is this practical?

provide a preformed slot into which the piles could be subsequently driven. It should be noted that groundwater control measures would be required in order to complete such excavation.

For determination of the point of fixity for driven steel H-piles, further details of the actual pile length and layout with respect to the adjacent ground surface will be required. It is expected that the point of fixity will be near or at the toe of the piles for the 8.5 m to 9 m lengths assumed; however, this will be confirmed once these details and loadings are known.

5.4.1 Axial Geotechnical Resistance

For HP 310 x 110 piles driven to found on or socketted at least 2 m into the limestone bedrock, a factored axial resistance at ULS of 2,000 kN may be assumed for design. In the case of the driven H-piles, this value represents a structural limitation for the pile rather than a geotechnical limitation. In the case of the socketted piles, this value assumes a socket length of at least 2 m within the bedrock and a socket diameter of 0.9 m. In both cases, the geotechnical resistance at SLS for 25 mm of settlement will be greater than the factored axial resistance at ULS, since the limestone bedrock is considered to be an unyielding material; as such, ULS conditions will govern for this foundation type.

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. For these driven piles, consideration must be given to the presence of cobbles and boulders within the glacially-derived soils at the site. The pile tips should be equipped with suitable flange reinforcement to minimize damage to the pile during driving. If fixity is of concern from a seismic design perspective, consideration should be given to provision of rock points on the piles to ensure seating on the bedrock. For this site, the piles will essentially be driven to practical refusal. The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, and selected pile. The criteria also need to be set to avoid overdriving and possible damage to the piles. Provision should be made to re-strike selected piles to confirm the set after adjacent piles have been driven, in accordance with MTO's current Special Provision.

Piles driven to bedrock! No restrike, no retap

5.4.2 Resistance to Lateral Loads

Lateral loading could be resisted fully or partially by the use of battered steel H-piles. If vertical piles are used, the resistance to lateral loading will have to be derived from the soil in front of the piles. Where integral abutments are under consideration, there will also be a requirement for the piles to move sufficiently to accommodate the bridge deck deflections.

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

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

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

The following ranges for the value of n_h may be assumed in the structural analysis:

Soil Unit	n_h
Embankment fill (assumed to be compacted granular fill) and existing surficial soil above Elevation 128 m	5 to 15 MPa/m 5-6
Surficial soils below Elevation 128 m (i.e. below the highest groundwater level measured in the vicinity of the proposed structure)	2 to 8 MPa/m 2-3

too high don't give rge

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

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

5.4.3 Frost Protection

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

5.5 Drilled Shaft Foundations

Drilled shafts founded on or socketted into the limestone bedrock may be used for support of the abutments. It is assumed that the abutment pile caps will be "perched" within the approach embankment fill in order to minimize the abutment wall height. Based on the proposed ramp grade at Elevation 138 m to 138.5 m and the assumed pile cap base at Elevation 135 m, the length of drilled shafts used for abutment support will be approximately 8.5 m to 9 m. The use of drilled shaft foundations is not considered appropriate for support of the centre pier or of any concrete wing walls / retaining walls associated with the structure, owing to the shallow depth to bedrock and the frost protection depth required for the pile caps.

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

As discussed in Section 5.4, the limestone bedrock at the site is moderately strong. If socketting of the drilled shafts into the bedrock is required, the sockets will have to be advanced by rock coring or churn drilling. Comment on degree of difficulty

5.5.1 Axial Geotechnical Resistance

Drilled shafts founded on the surface of the limestone bedrock, or socketted nominally (less than 1 m) into the bedrock, should be designed based on end-bearing resistance and a factored geotechnical resistance at ULS of 2 MPa should be used. Serviceability Limit States resistances do not apply to drilled shafts founded on the limestone bedrock, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

5.5.2 Resistance to Lateral Loads

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

5.5.3 Frost Protection

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

5.6 Retained Soil System (RSS) Walls

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

Use of an RSS wall is considered appropriate for the proposed wing walls / retaining walls at the structure, which will be up to about 8 m or 9 m high in the immediate vicinity of the proposed structure. The use of an RSS wall is also considered appropriate along the ramp alignment north of Highway 417, in the vicinity of Borehole 02-120, where property constraints adjacent to an existing communications tower preclude the use of standard embankment side slopes. Once the limits of this wall are finalized, it may be necessary to carry out additional field investigation work to confirm the composition, relative density and thickness of the overburden soils.

A typical RSS wall is founded at least 0.3 m below the existing ground surface in front of the wall, below any topsoil and/or peat. Assuming that the RSS wall acts as a unit and utilizes the full width of the reinforced soil mass, which is taken as two-thirds of the height of the ~~wall, the~~ following factored geotechnical resistances at ULS may be used for design:

- 200 kPa for a 5 m high wall; and
- 325 kPa for an 9 m high wall.

$q = \frac{1}{2}\gamma B \; c_u N_c + \gamma D N_q + \frac{1}{2}$

The geotechnical resistance at SLS, for 25 mm of settlement, may be taken as 200 kPa. The majority of the settlement of the RSS walls will occur during construction since the founding soils are essentially granular (i.e. sands and silts), overlying bedrock at a shallow depth. This is particularly the case in the areas of the abutments, where RSS walls may be used in a false abutment configuration.

Suddenly this appears

The resistance to lateral forces / sliding resistance between the compacted Granular "A" and the subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, tan N', between the compacted Granular "A" of the RSS wall and the loose to compact sands and silts may be taken as 0.45. This represents an unfactored value; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

$\phi = 35°$? Check

The internal stability of the mechanically-reinforced soil walls should be checked by the RSS supplier / designer. In this regard, the internal stability must also be checked for seismic loading. The Factor of Safety related to global stability under static loading for properly designed and constructed RSS walls at this site is greater than 1.3.

The liquefaction potential of the soils below the RSS wall under seismic loading has been considered using the empirical method outlined in Section C.4.6.2 of the *CHBDC Commentary*, which correlates the cyclic resistance ratio of the soils with their normalized penetration resistance and fines content. Based on this assessment, a factor of safety of greater than 1.1 against liquefaction for an earthquake of magnitude 7.5 is obtained for the surficial sand soils below the water table. The factor of safety against liquefaction is higher for the surficial silts and till soils below the water table. Pseudo-static stability analysis indicates that the ground surface acceleration due to the design earthquake event does not result in global instability of the RSS wall.

5.7 Lateral Earth Pressures for Design

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

The following recommendations are made concerning the design of the walls. It should be noted that these design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular 'A' or Granular 'B' but with less than 5 per cent passing the 200 sieve should be used as backfill behind the walls. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 per cent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3501.00 and 3504.00.

- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with *CHBDC* Section 6.9.3 and Figure 6.9.3. Compaction equipment should be used in accordance with OPSS 501.06. Other surcharge loadings should be accounted for in the design, as required.

- The granular fill may be placed either in a zone with width equal to at least 1.8 m behind the back of the wall stem (Case I in Figure C6.9.1(1) 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(1) 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_0	0.50

$\phi = 30$

$1 - \sin\phi = 0.5$

provide ϕ

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

	Granular 'A'	Granular 'B' Type II
Soil unit weight:	22 kN/m³	21 kN/m³
Coefficients of static lateral earth pressure:		
Active, K_a	0.27	0.31
At rest, K_o	0.43	0.47

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

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

- In accordance with Sections 4.6.4 and C.4.6.4 of the *CHBDC* and its *Commentary*, for structures which allow lateral yielding, the horizontal seismic coefficient, k_h, used in the calculation of the seismic active pressure coefficient, is taken as 0.5 times the zonal acceleration ratio (i.e. k_h = 0.11). For structures that do not allow lateral yielding, k_h is taken as 1.5 times the zonal acceleration ratio (i.e. k_h = 0.33). The following seismic active pressure coefficients (k_{AE}) for the two cases (Case I and Case II) may be used in design. 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.

To be discussed with DD

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Case I	Case II	
		Granular A	Granular B Type II
Yielding wall	0.40	0.31	0.36
Non-yielding wall	0.66	0.53	0.60

- The above K_{AE} values for yielding walls are applicable provided that the wall can move up to 250A (mm), where A is the design zonal acceleration ratio of 0.22. This corresponds to outward displacements of up to 55 mm at this site.

- The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e. an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

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

where K_a is the static active earth pressure coefficient;
K_{AE} is the seismic active earth pressure coefficient;
γ' is the effective unit weight of the soil (kN/m^3) as given on page 19;
d is the depth below the top of the wall (m); and
H is the height of the wall above the toe (m).

5.8 Embankment Design

– are approach embankments covered?

The construction of the Highway 417E-7W ramp will require placement of between 5 m to 9 m of fill above the existing ground surface, between Station 11+450 (about 230 m southwest of the proposed structure) and Station 11+850 (about 70 m northeast of the proposed structure). Based on the borehole results, the embankment subgrade soils will consist of loose to compact, surficial sands and silts or, in places, compact to dense silty sand till to sand and silt till. Northeast of Highway 417, where the proposed Highway 417E-7W ramp is in close proximity to the existing Highway 417-7 ramp, existing embankment fill was encountered in the borehole; this fill consists of compact silty sand containing some gravel, trace clay and organics. A 0.7 m thick layer of stiff silty clay was encountered in Borehole 02-120 in this area, at a depth of 2.4 m.

Any topsoil, organic matter and softened / loosened soils should be stripped from below the approach embankment areas and within the limits of the high fill embankment (between Stations 11+450 and 11+850) and all subgrade soils proof-rolled prior to fill placement. Embankment fill should be placed in regular lifts with loose thickness not exceeding 300 mm, and be compacted to at least 95 per cent of the material's Standard Proctor maximum dry density. The final lift prior to placement of the granular subbase and base courses should be compacted to 100 per cent of the Standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified personnel during placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

With appropriate subgrade preparation and proper placement and compaction of embankment fill materials, the 5 m to 9 m high embankments with side slopes maintained at 2 horizontal to 1 vertical (2H:1V) will have a factor of safety of greater than 1.3 against deep-seated slope instability. Static slope stability analyses for this embankment configuration were carried out using the following parameters:

Settlement of Embankment
– Due to recompression of native soils
– within embankment proper

Soil Deposit	*Bulk Unit Weight*	*Effective Friction Angle*	*Undrained Shear Strength*
Embankment Fill	20 – 22 kN/m^3	32°	–
Surficial Sands and Silts	19 – 20 kN/m^3	30°	–
Silty Clay (where present)	19 kN/m^3	28°	100 kPa
Silty Sand Till to Sand and Silt Till	21 kN/m^3	32°	–

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

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

5.9 Design and Construction Considerations

5.9.1 Excavation

Excavations for construction of spread footings or to allow excavation into the bedrock, if required, will typically extend through between 1 m and 2 m of loose to compact sands, silts and gravels, overlying less than 1 m of compact to dense silty sand till to sand and silt till. The surface of the limestone bedrock is present between about Elevations 126.5 m and 126 m (typically at 2 m to 2.5 m depth) in the vicinity of the proposed ramp structure. The groundwater level at the site is typically 1.5 m to 2 m above the bedrock surface.

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

depth of excavat

It is not anticipated that temporary roadway protection will be required along the existing Highway 417-7 ramp to permit construction of the new Highway 417E-7W ramp and structure.

5.9.2 Groundwater and Surface Water Control

The groundwater level at the site is typically 1.5 m to 2 m above the bedrock surface (i.e. generally less than about 1 m below ground surface). Excavations to expose the bedrock surface, either for founding of spread footings or to enable formation of a trench within the bedrock to provide toe support to piles, will require groundwater control. The shallow depth to bedrock at the site will dictate the type of groundwater control system that may be used. Given the available space, it is likely that open-cut excavations with sufficient sumping will adequately control the groundwater; in this ~~case~~, however, the excavation side slopes will probably have to be maintained at about 3H:1V. Alternatively, a shallow eductor system could be used to lower the groundwater level ~~within~~ the overburden, supplemented by pumping from sumps formed within the bedrock at the base of the excavations.

It is noted that during a previous subsurface investigation near the site, carried out for the existing bi-directional Highway 417-7 ramp structure by the Department of Highways, Ontario in April 1971, the southwestern portion of the site was flooded by between 150 mm and 200 mm of standing water. Consideration should be given to scheduling the construction work to avoid foundation excavation in the spring.

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

5.9.3 Obstructions

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

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

Inconsistent with 2nd paragraph on page 23

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LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

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

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

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

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils

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

IV. SOIL TESTS

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

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

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

LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

γ shear strain
Δ change in, e.g. in stress: $\Delta\sigma$
ε linear strain
ε_v volumetric strain
η coefficient of viscosity
ν Poisson's ratio
σ total stress
σ' effective stress ($\sigma' = \sigma - u$)
σ'_{vo} initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$ principal stresses (major, intermediate, minor)
σ_{oct} mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ shear stress
u porewater pressure
E modulus of deformation
G shear modulus of deformation
K bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (con't.)

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

(c) Hydraulic Properties

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

(d) Consolidation (one-dimensional)

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

(e) Shear Strength

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

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

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of weathering.

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

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

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

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

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

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

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

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

CORE CONDITION

Total Core Recovery

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

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

DISCONTINUITY DATA

Fracture Index

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

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

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

Description and Notes

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

Abbreviations

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

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

Golder Associates

RECORD OF BOREHOLE No 02-101

METRIC

PROJECT 021-1155-1
W.P. 256-99-00
DIST ____ HWY 7
DATUM Geodetic

LOCATION ____
BOREHOLE TYPE CME 55 Bombardier, 108mm I.D. Hollow Stem Auger
DATE Nov. 12 - 13, 2002

ORIGINATED BY P.A.H.
COMPILED BY M.I.C.
CHECKED BY L.C.C.

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
128.6	Ground Surface					
0.0	Topsoil					
0.2	Sand and Gravel, trace silt Compact Brown Wet	1	SS	22	o	36 55 (9)
127.5 1.1	Silt, trace sand and clay Compact to dense Grey- brown to grey Wet	2	SS	40	o	0 8 80 12
126.6	Sand and Silt, some gravel, trace clay (Till) Dense Grey Wet					
126.4 2.2	LIMESTONE (BEDROCK) with shale interbeds. Fresh Weak to medium strong Thinly to medium - bedded Grey Bedrock cored between 2.2 m and 6.0m depth. For bedrock coring details refer to Record of Drillhole 02-101					
122.7 6.0	End of Borehole Note: Water level in piezometer at 0.7 m depth (Elev. 127.9m) on Jan. 8, 2003					

Elevation scale: 128, 127, 126, 125, 124, 123

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED; 20 40 60 80 100
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L
WATER CONTENT (%): 10 20 30
UNIT WEIGHT γ kN/m³

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

PROJECT: 021-1155-1

RECORD OF DRILLHOLE: 02-101

SHEET 2 OF 2

LOCATION: See Site Plan

DRILLING DATE: Nov. 12, 13, 2002

DATUM: Geodetic

DRILL RIG: CME 55 Bombardier

INCLINATION: -90° AZIMUTH: —

DRILLING CONTRACTOR: Marathon Drilling

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		Refer to previous page		126.41												
3 – 5	Rotary Drill, NQ Core	LIMESTONE (BEDROCK) with shale interbeds. Fresh Weak to medium strong Thinly to medium - bedded Grey		2.19	1		100									
					2		100									
				122.65 5.95	3		100									
6		End of Borehole														

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: P.A.H.

CHECKED: M.I.C.

DRILLHOLE 021-1155-5000-ROCK.GPJ GLDR_CAN.GDT 20/2/03

RECORD OF BOREHOLE No 02-102

METRIC

PROJECT 021-1155-1
W.P. 256-99-00
DIST ___ HWY 7
DATUM Geodetic
LOCATION ___
BOREHOLE TYPE CME 55 Bombardier, 108mm I.D. Hollow Stem Auger
DATE Nov. 13, 2002
ORIGINATED BY P.A.H.
COMPILED BY M.I.C.
CHECKED BY L.C.C.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
128.2	Ground Surface					
0.0	Topsoil					
0.2	Sand, trace to some gravel, trace silt Compact Brown Moist to wet				▼	
127.2 / 1.0	Silt, trace sand and clay Dense Grey brown to grey Wet	1	SS	36		0 9 79 12
126.8 / 1.4	Sand and Silt, some gravel, trace clay (Till) Compact Grey Wet	2	SS	14		
126.2 / 2.0	LIMESTONE (BEDROCK) with shale interbeds. Fresh Weak to medium strong Thinly to medium - bedded Grey Bedrock cored between 2.0 m and 6.0 m depth. For bedrock coring details refer to Record of Drillhole 02-102					
122.3 / 6.0	End of Borehole					

Sample 1 natural moisture content plotted at approximately 15%.

Chart column headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (20, 40, 60, 80, 100): ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10, 20, 30; UNIT WEIGHT γ kN/m³; ELEVATION SCALE 128, 127, 126, 125, 124, 123.

Note:
Water level in open borehole at 0.3 m depth (Elev. 127.9m) on completion of drilling on Nov. 13, 2002

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

RECORD OF DRILLHOLE: 02-102

PROJECT: 021-1155-1

LOCATION: See Site Plan

DRILLING DATE: Nov. 13, 2002

DATUM: Geodetic

DRILL RIG: CME 55 Bombardier

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR: Marathon Drilling

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		Refer to previous page		126.20												
2		LIMESTONE (BEDROCK) with shale interbeds. Fresh Weak to medium strong Thinly to medium - bedded Grey		2.00	1		100									
3					2		100									
4																
5					3		100									
				122.25 5.95												
6		End of Borehole														
7																
8																
9																
10																
11																
12																

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: P.A.H.

CHECKED: M.I.C.

Golder Associates

RECORD OF BOREHOLE No 02-103

METRIC

PROJECT 021-1155-1

W.P. 256-99-00

DIST ______ HWY 7

DATUM Geodetic

LOCATION

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

DATE Nov. 12, 2002

ORIGINATED BY D.B.

COMPILED BY M.I.C.

CHECKED BY L.C.C.

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
129.8	Ground Surface						
0.0	Topsoil						
0.2	Sand, trace gravel and silt Loose to dense Brown Moist				129		
		1	SS	9			0 89 (11)
		2	SS	32	128		
127.5 2.3	Silt, trace sand and clay Compact Brown to grey Wet	3	SS	21	127		
126.6 3.2	Sand and Silt, some gravel, trace clay (Till) Compact Grey Wet	4	SS	28			
126.1 3.7	End of Borehole Refusal to Auger Penetration						

Notes:
1. Water level in open borehole at 2.3m depth (Elev. 127.5m) during drilling operations.
2. Water level in piezometer at 1.8 m depth (Elev. 128.0m) on Jan. 8, 2003

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT W_L, WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³

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

RECORD OF BOREHOLE No 02-104

METRIC

PROJECT 021-1155-1

W.P. 256-99-00 LOCATION

DIST HWY 7 BOREHOLE TYPE CME 55 Bombardier, 108mm I.D. Hollow Stem Auger

DATUM Geodetic DATE Nov. 14, 2002

ORIGINATED BY D.B.

COMPILED BY M.I.C.

CHECKED BY L.C.C.

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS
128.4	Ground Surface						
0.0	Topsoil				128		
127.6 / 0.8	Silt, some gravel, trace sand and clay; Loose; Grey brown; Wet	1	SS	5	127		
126.7 / 1.7	Silty Sand, some gravel, trace clay (Till); Loose; Grey; Wet	2	SS	5		~10	
126.3 / 2.1	LIMESTONE (BEDROCK) with shale interbeds. Fresh; Weak to medium strong; Thinly to medium - bedded; Grey. Bedrock cored between 2.1m and 6.0m depth. For bedrock coring details refer to Record of Drillhole 02-104				126, 125, 124, 123		
122.4 / 6.0	End of Borehole. NOTE: Water level in piezometer at 0.4 m depth (Elev. 128.0m) on Jan. 8, 2003						

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

+ 3, × 3: Numbers refer to Sensitivity

O 3% STRAIN AT FAILURE

RECORD OF DRILLHOLE: 02-104

PROJECT: 021-1155-1
LOCATION: See Site Plan
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: Nov. 14, 2002
DRILL RIG: CME 55 Bombardier
DRILLING CONTRACTOR: Marathon Drilling

DATUM: Geodetic

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		Refer to previous page		126.27												
	Rotary Drill NQ Core	LIMESTONE (BEDROCK) with shale interbeds. Fresh Weak to medium strong Thinly to medium - bedded Grey		2.13	1		100									
3																
					2		100									
4																
5					3		100									
6		End of Borehole		122.43 5.97												
7																
8																
9																
10																
11																
12																

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: D.B.
CHECKED: M.I.C.

Golder Associates

RECORD OF BOREHOLE No 02-105

METRIC

PROJECT 021-1155-1			
W.P. 256-99-00	LOCATION		ORIGINATED BY D.B.
DIST	HWY 7	BOREHOLE TYPE CME 55 Bombardier, 108mm I.D. Hollow Stem Auger	COMPILED BY M.I.C.
DATUM Geodetic		DATE Nov. 14, 2002	CHECKED BY L.C.C.

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
128.4	Ground Surface												
0.0	Topsoil		1	AS			128						
127.7 0.7	Silty Sand, some gravel Loose Grey Wet		2	SS	7	▼							
127.2	Silt, trace sand and clay Grey brown Wet						127						
127.0 1.4	Silty Sand, some gravel, trace clay (Till) Loose Grey Wet		3	SS	8			o		18	45	29	8
126.4 2.0	LIMESTONE (BEDROCK) with shale interbeds. Fresh Weak to medium strong Thinly to medium - bedded Grey Bedrock cored between 2.0 m and 6.0 m depth. For bedrock coring details refer to Record of Drillhole 02-105						126 125 124 123						
122.5 6.0	End of Borehole NOTE: Water level in open hole at 0.9 m depth (Elev. 128.5m) on Nov. 14, 2002												

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

ON_MOT 021-1155-5000-MTO.GPJ ON_MOT.GDT 27/2/03

PROJECT: 021-1155-1

RECORD OF DRILLHOLE: 02-105

SHEET 2 OF 2

LOCATION: See Site Plan

DRILLING DATE: Nov. 14, 2002

DATUM: Geodetic

DRILL RIG: CME 55 Bombardier

INCLINATION: -90° AZIMUTH: —

DRILLING CONTRACTOR: Marathon Drilling

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	RECOVERY: TOTAL CORE %	RECOVERY: SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA: DIP w.r.t. CORE AXIS	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		Refer to previous page		126.42												
2	Rotary Drill, NQ Core	LIMESTONE (BEDROCK) with shale interbeds. Fresh Weak to medium strong Thinly to medium - bedded Grey		1.98	1		100									
3					2		100									
4																
5					3		100									
				122.45 5.95												
6		End of Borehole														
7																
8																
9																
10																
11																

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: D.B.
CHECKED: M.I.C.

DRILLHOLE 021-1155-5000-ROCK.GPJ GLDR_CAN.GDT 20/2/03

RECORD OF BOREHOLE No 02-106

METRIC

PROJECT	021-1155-1		
W.P.	256-99-00	LOCATION	
DIST	HWY 7	BOREHOLE TYPE	CME 55 Bombardier, 108mm I.D. Hollow Stem Auger
DATUM	Geodetic	DATE	Nov. 14, 2002

ORIGINATED BY D.B.
COMPILED BY M.I.C.
CHECKED BY L.C.C.

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
128.3	Ground Surface				
0.0	Topsoil				128
128.1 / 0.2	Silt, some gravel, trace sand and clay, Loose, Grey brown, Wet	1	SS	6	127
126.8 / 1.5	Sand and Silt, some gravel, trace clay (Till), Compact, Grey, Wet	2	SS	* 18/0.23	
126.4 / 1.9	End of Borehole. Refusal to Split-Spoon Sampler and Auger Penetration				

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

NOTES:
1. Piezometer frozen at ground surface (Elev. 128.3m) on Jan. 8, 2003.
2. * Split-spoon bouncing after 18 blows.

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

Golder Associates

RECORD OF BOREHOLE No 02-107

METRIC

PROJECT 021-1155-1

W.P. 256-99-00

DIST ___ HWY 7

DATUM Geodetic

LOCATION

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

DATE Nov. 13, 2002

ORIGINATED BY D.B.

COMPILED BY M.I.C.

CHECKED BY L.C.C.

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	NATURAL MOISTURE CONTENT w (%)	GR	SA	SI	CL
128.4	Ground Surface								
0.0	Topsoil								
128.1 / 0.3	Sand, trace silt and gravel, Compact, Grey, Wet	1	SS	16	~20	3	83	11	3
127.0 / 1.4	Silt, trace sand and clay, Compact, Grey, Wet	2	SS	14					
126.3	Sand and Silt, some gravel, trace clay (Till), Compact, Grey, Wet	3	SS	7/0.10*					
126.0 / 2.4	LIMESTONE (BEDROCK) with shale interbeds. Fresh, Weak to medium strong, Thinly to medium - bedded, Grey. Bedrock cored between 2.4 m and 5.9m depth. For bedrock coring details refer to Record of Drillhole 02-107								
122.5 / 5.9	End of Borehole								

Note:
* Split-spoon bouncing after 7 blows.

Elevation scale: 128, 127, 126, 125, 124, 123

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

GROUND WATER CONDITIONS

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

RECORD OF DRILLHOLE: 02-107

PROJECT: 021-1155-1
LOCATION: See Site Plan
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: Nov. 13, 2002
DRILL RIG: CME 55 Bombardier
DRILLING CONTRACTOR: Marathon Drilling

DATUM: Geodetic

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		Refer to previous page														
	Rotary Drill, NQ Core	LIMESTONE (BEDROCK) with shale interbeds. Fresh Weak to medium strong Thinly to medium - bedded Grey		2.38	1		100									
3																
					2		100									
4																
5					3		100									
6		End of Borehole		5.88												
7																
8																
9																
10																
11																
12																

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: D.B.
CHECKED: M.I.C.

RECORD OF BOREHOLE No 02-108

METRIC

PROJECT 021-1155-1			
W.P. 256-99-00	LOCATION		ORIGINATED BY D.B.
DIST	HWY 7	BOREHOLE TYPE CME 55 Bombardier, 108mm I.D. Hollow Stem Auger	COMPILED BY M.I.C.
DATUM Geodetic		DATE Nov. 13, 2002	CHECKED BY L.C.C.

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
128.4	Ground Surface								
128.2 / 0.0	Topsoil								
0.2	Sand, some gravel Compact Grey Wet					128			
		1	SS	28	▼				
						127			
126.7 / 1.7	Sand and Silt, some gravel, trace clay (Till) Compact Wet	2	SS	18			o (~10)		
126.4 / 2.0	LIMESTONE (BEDROCK) with shale interbeds and some near - vertical jointing Fresh Weak to medium strong Thinly to medium - bedded Grey					126 125			
	Bedrock cored between 2.0 m and 5.9 m depth. For bedrock coring details refer to Record of Drillhole 02-108					124 123			
122.5 / 5.9	End of Borehole Note: Water level in open hole at 1.2 m depth (Elev. 127.2m) on Nov. 13, 2002								

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

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

RECORD OF DRILLHOLE: 02-108

PROJECT: 021-1155-1

LOCATION: See Site Plan

INCLINATION: -90°   AZIMUTH: —

DRILLING DATE: Nov. 13, 2002

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Marathon Drilling

DATUM: Geodetic

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		Refer to previous page														
	Rotary Drill, NQ Core	LIMESTONE (BEDROCK) with shale interbeds and some near - vertical jointing Fresh Weak to medium strong Thinly to medium - bedded Grey		2.01	1		100									
3																
					2		100									
4																
5					3		100									
6		End of Borehole		5.88												
7																
8																
9																
10																
11																
12																

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: D.B.

CHECKED: M.I.C.

Golder Associates

RECORD OF BOREHOLE No 02-109

METRIC

PROJECT 021-1155-1

W.P. 256-99-00

LOCATION

ORIGINATED BY P.A.H.

DIST

HWY 7

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

COMPILED BY M.I.C.

DATUM Geodetic

DATE Dec. 9, 2002

CHECKED BY L.C.C.

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	NATURAL MOISTURE CONTENT
128.3	Ground Surface						
0.0	Topsoil				▼		
0.1	Sand, trace gravel, trace silt Compact Grey brown Wet	1	SS	22		128	
127.1 1.2	Silt, trace sand and clay Brown Wet					127	o
126.8 1.5	Sand and Silt, some gravel, trace clay (Till) Dense Grey Wet	2	SS	37			
126.1 2.2	LIMESTONE (BEDROCK) with shale interbeds and some near - vertical jointing Fresh Weak to medium strong Thinly to medium - bedded Grey Bedrock cored between 2.2 m and 5.4 m depth. For bedrock coring details refer to Record of Drillhole 02-109					126 125 124 123	
122.9 5.4	End of Borehole NOTE: Water level in open borehole at 0.2 m depth (Elev. 128.1m) on Dec. 9, 2002						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m³

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

+ ^{3}, × ^{3}: Numbers refer to Sensitivity

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

PROJECT: 021-1155-1

RECORD OF DRILLHOLE: 02-109

LOCATION: See Site Plan

DRILLING DATE: Dec. 9, 2002

DATUM: Geodetic

DRILL RIG: CME 55 Bombardier

INCLINATION: -90° AZIMUTH: —

DRILLING CONTRACTOR: Marathon Drilling

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		Refer to previous page		126.10												
3–5	Rotary Drill, NQ Core	LIMESTONE (BEDROCK) with shale interbeds and some near - vertical jointing Fresh Weak to medium strong Thinly to medium - bedded Grey		2.20	1		100									
					2		100									
					3		100									
				122.86 / 5.44												
		End of Borehole														

DEPTH SCALE

1 : 50

LOGGED: P.A.H.

CHECKED: M.I.C.

Golder Associates

RECORD OF BOREHOLE No 02-110

METRIC

PROJECT 021-1155-1
W.P. 256-99-00
LOCATION
ORIGINATED BY P.A.H.
DIST
HWY 7
BOREHOLE TYPE CME 55 Bombardier, 108mm I.D. Hollow Stem Auger
COMPILED BY M.I.C.
DATUM Geodetic
DATE Dec. 9, 2002
CHECKED BY L.C.C.

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)
128.4	Ground Surface						
0.0	Topsoil						
0.1	Sand and Gravel, trace silt Compact Grey brown Wet	1	SS	16	▼	128	
		2	SS	30			
127.3 1.1	Silt, trace sand and clay Dense Brown to grey Wet						o (approx. 19)
		3	SS	30		127	
126.6 1.8	Sand and Silt, with cobbles and some gravel (Till) ** Grey Wet						
126.1 2.3	LIMESTONE (BEDROCK) with shale interbeds and some near - vertical jointing Fresh Weak to medium strong Thinly to medium - bedded Grey Bedrock cored between 2.3 m and 5.2 m depth. For bedrock coring details refer to Record of Drillhole 02-110					126 125 124	
123.2 5.2	End of Borehole						

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

NOTES:
1. Water level in open borehole at 0.3 m depth (Elev. 128.1m) on Dec. 9, 2002.
2. ** As retrieved from core sampling.

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

PROJECT: 021-1155-1

RECORD OF DRILLHOLE: 02-110

LOCATION: See Site Plan

INCLINATION: -90° AZIMUTH: —

DRILLING DATE: Dec. 9, 2002

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Marathon Drilling

DATUM: Geodetic

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	COLOUR % RETURN / FLUSH	RECOVERY: TOTAL CORE %	RECOVERY: SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA: DIP w.r.t. CORE AXIS	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		Refer to previous page		126.42												
2	Rotary Drill, NQ Core	Sand and Silt, with cobbles and some gravel (TILL)		126.42 / 1.98												
		LIMESTONE (BEDROCK) with shale interbeds and some near - vertical jointing Fresh Weak to medium strong Thinly to medium - bedded Grey		126.11 / 2.29	1		100									
3					2		100									
4																
5					3		100									
		End of Borehole		123.17 / 5.23												

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: P.A.H.

CHECKED: M.I.C.

DRILLHOLE 021-1155-5000-ROCK.GPJ GLDR_CAN.GDT 20/2/03

RECORD OF BOREHOLE No 02-111

METRIC

PROJECT 021-1155-1			
W.P. 256-99-00	LOCATION	ORIGINATED BY P.A.H.	
DIST	HWY 7	BOREHOLE TYPE CME 55 Bombardier, 108mm I.D. Hollow Stem Auger	COMPILED BY M.I.C.
DATUM Geodetic		DATE Dec. 12, 2002	CHECKED BY L.C.C.

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
128.4	Ground Surface									
0.0 / 128.1	Peat	1	AS	-				68.6		
0.3	Sand, some silt Loose Grey brown Wet				▼	128				
		2	SS	8						
127.0 / 1.4	Sand and Silt, some gravel, trace clay (Till) Dense Grey-brown to grey Wet					127				
		3	SS	31						
126.2 / 2.2	End of Borehole Refusal to Auger Penetration									

NOTE:
Water level in open borehole at 0.6 m depth (Elev. 127.8m) on Dec. 12, 2002

Legend: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; DCPT scale 20 40 60 80 100; shear strength scale 20 40 60 80 100; water content scale 10 20 30

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

RECORD OF BOREHOLE No 02-112

METRIC

PROJECT 021-1155-1

W.P. 256-99-00

DIST ___ HWY 7

DATUM Geodetic

LOCATION ___

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

DATE Dec. 12, 2002

ORIGINATED BY P.A.H.

COMPILED BY M.I.C.

CHECKED BY L.C.C.

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)
128.7	Ground Surface						
0.0	Silty sand, trace gravel and organics (Fill)	1	AS	-			
128.2 0.5	Peat					128	
127.8 0.9	Sand, trace silt and gravel Compact Grey-brown to grey Wet	2	SS	13	▼		
127.2	Clayey Silt, trace sand Grey-brown					127	
127.0 1.7	Sand and Silt to Silty Sand, some gravel, trace clay (Till) Loose to very dense Grey Wet	3	SS	6			~10
		4	SS	90/0.23			
126.1 2.7	End of Borehole Refusal to Auger Penetration						

NOTE:
Water level in open borehole at 0.9 m depth (Elev. 127.8m) on Dec. 12, 2002

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

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

Golder Associates

RECORD OF BOREHOLE No 02-120

METRIC

PROJECT 021-1155-1
W.P. 256-99-00
LOCATION
ORIGINATED BY D.B.
DIST
HWY 7
BOREHOLE TYPE CME 55 Bombardier, 108mm I.D. Hollow Stem Auger
COMPILED BY M.I.C.
DATUM Geodetic
DATE Dec. 18, 2002
CHECKED BY L.C.C.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
130.2	Ground Surface					
0.0	Silty sand, some gravel, trace clay and organics (Fill) Compact Dark brown Moist	1	SS	11	130	
128.8 / 1.4	Sand, some silt, trace gravel Compact Brown Moist	2	SS	13	129	4 82 11 3
127.8 / 2.4	Silty Clay, trace sand Stiff Grey-brown Wet	3	SS	7	128	
127.2 / 3.1	Silt, trace to some sand, trace clay Dense Grey Wet	4	SS	32	127	0 11 81 8
126.2 / 4.0	Sand and Silt, some gravel, trace clay, with cobbles and boulders (Till) Compact Grey Wet	5	SS	29	126	
		6	SS	* 7/0.08		
125.5 / 4.7	End of Borehole Refusal to Auger Penetration					

NOTES:
1. Water Level in Piezometer at 2.1 m depth (Elev. 128.1m) on Jan. 8, 2003.
2. * Split-spoon bouncing after 7 blows

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

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

Golder Associates

RECORD OF BOREHOLE No 02-121 1 OF 1 METRIC

PROJECT 021-1155-1

W.P. 256-99-00 LOCATION ORIGINATED BY D.B.

DIST HWY 7 BOREHOLE TYPE CME 55 Bombardier, 108mm I.D. Hollow Stem Auger COMPILED BY M.I.C.

DATUM Geodetic DATE Dec. 19, 2002 CHECKED BY L.C.C.

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
128.6	Ground Surface										
0.0	Topsoil										
128.4 / 0.2	Sand and Silt to Silty Sand, trace gravel and clay, with cobbles (Till) Compact Grey-brown to grey Wet						128				
			1	SS	12				○ (approx. 9)		9 49 34 8
126.9 / 1.7			2	SS	3/0.05 *		127				
	End of Borehole Refusal to Split-Spoon Sampler and Auger Penetration Note: * Split-spoon bouncing after 3 blows.										

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

RECORD OF BOREHOLE No 02-122

METRIC

PROJECT 021-1155-1

W.P. 256-99-00

DIST ____ HWY 7

DATUM Geodetic

LOCATION

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

DATE Dec. 19, 2002

ORIGINATED BY D.B.

COMPILED BY M.I.C.

CHECKED BY L.C.C.

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)
129.2	Ground Surface					
0.0	Topsoil				129	
128.9 / 0.3	Silty Sand, some gravel Compact Grey brown Wet	1	SS	14		
128.0 / 1.2	Sand and Silt, some gravel, trace clay with cobbles and boulders (Till) Compact to dense Grey Wet	2	SS	30/0.23*	128	~10
127.1 / 2.1	End of Borehole Refusal to Auger Penetration					

Notes:
1. Water Level in piezometer at 0.6 m depth (Elev. 128.6m) on Jan. 8, 2003.
2. * Split-spoon bouncing after 30 blows.

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

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

Golder Associates

RECORD OF BOREHOLE No 02-123

METRIC

PROJECT 021-1155-1
W.P. 256-99-00
LOCATION
DIST
HWY 7
BOREHOLE TYPE CME 55 Bombardier, 108mm I.D. Hollow Stem Auger
DATUM Geodetic
DATE Dec. 19, 2002
ORIGINATED BY D.B.
COMPILED BY M.I.C.
CHECKED BY L.C.C.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
129.4	Ground Surface											
0.0	Topsoil											
129.1 / 0.3	Silty Sand, some gravel, trace clay with cobbles and boulders (Till) Compact Grey-brown to grey Wet					129						
		1	SS	19	▼		~10		17	48	28	7
						128						
		2	SS	* 8/0.05								
127.7 / 1.7	End of Borehole Refusal to Auger Penetration											

NOTES:
1. Water level in open borehole at 1.1 m depth (Elev 128.3m) on Dec. 19, 2002.
2. * Split-spoon bouncing after 8 blows.

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (10, 20, 30); REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

Golder Associates

RECORD OF BOREHOLE No 02-124

METRIC

PROJECT 021-1155-1

W.P. 256-99-00

DIST ____ HWY 7

DATUM Geodetic

LOCATION ____

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

DATE Dec. 19, 2002

ORIGINATED BY D.B.

COMPILED BY M.I.C.

CHECKED BY L.C.C.

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L / WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
129.2	Ground Surface										
0.0	Topsoil										
129.0 0.2	Silty Sand, some gravel Grey Wet						129				
128.6 0.6	Sand and Silt, some gravel, trace clay with cobbles (Till) Compact Grey Wet		1	SS	15/0.13 *						
128.3 0.9	End of Borehole Refusal to Split-Spoon Sampler and Auger Penetration										

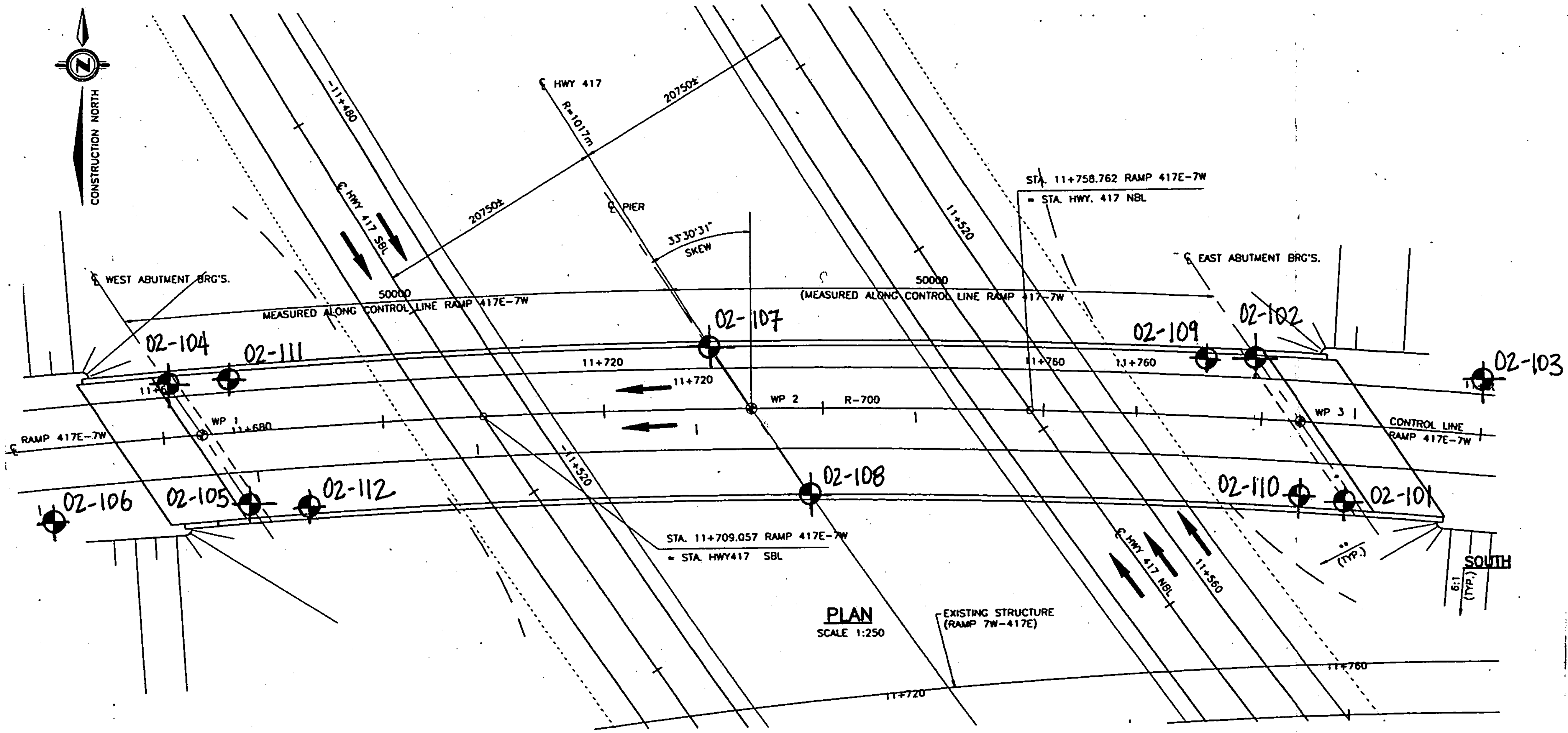
NOTE:
* Split-spoon bouncing after 15 blows.

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

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

HIGHWAY 417E-7W
RAMP STRUCTURE
BOREHOLE LOCATIONS

DRAWING 1



LEGEND:

Borehole Location

SCALE:

1 : 500 Horizontal

NOTES:

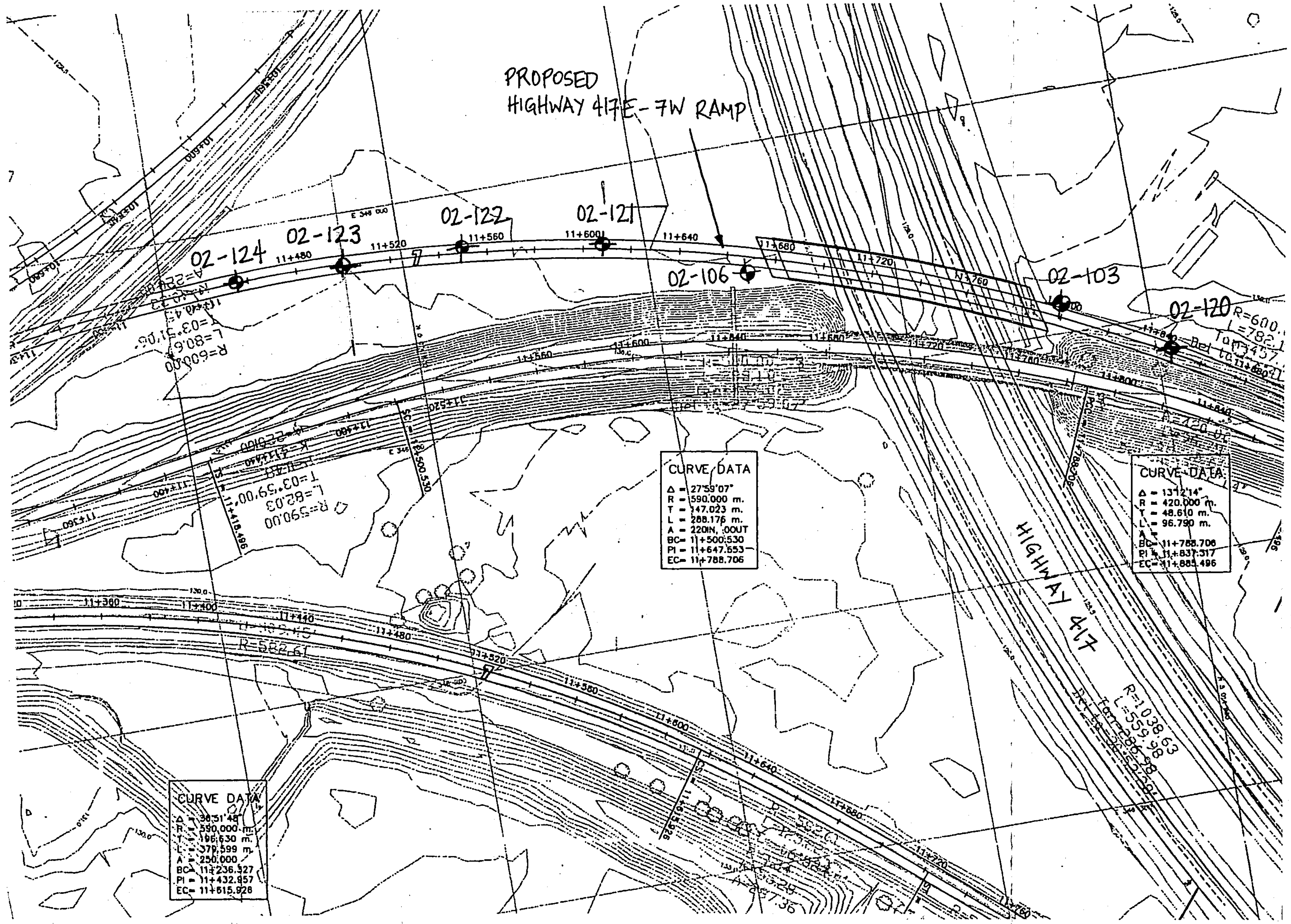
1 This drawing was created using Exhibit 13 ("Hwy 7 Ramp 417E-7W Structure, Conceptual General Arrangement, Structural Steel Alternative"), from the Structural Planning Report for Highway 7 from Carleton Place to Highway 417, G.W.P. 142-78-00, prepared by Totten Sims Hubicki and dated August 2002.

2 This drawing will be updated in accordance with MTO's terms of reference, once electronic files are received for use in drawing preparation.

DRAFT PRELIMINARY

Date	Feb 2003		Drawn	LCC
Project	021-1155-1	Golder Associates	Chkd	LCC

HIGHWAY 417E-7W RAMP HIGH FILL EMBANKMENT BOREHOLE LOCATIONS	DRAWING 2



LEGEND:

Borehole Location

SCALE:

20M 0 40M
Horizontal

NOTES:

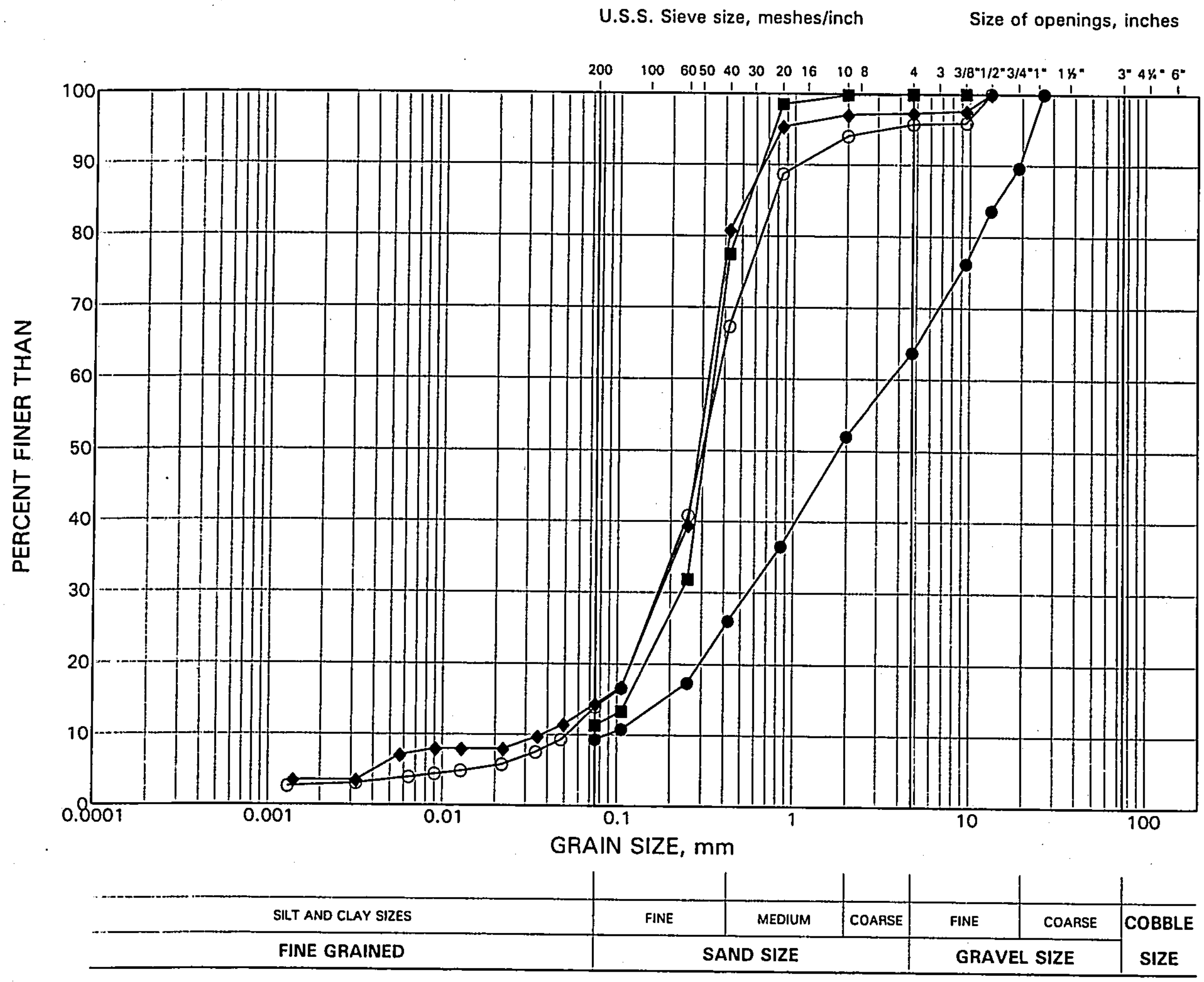
1. This drawing was created using Plates A-40 and A-41 from the Update to the Highway 7 Preliminary Design Study (Highway Engineering) Report, dated June 2002, prepared by Totten Sims Hubicki.
2. This drawing will be updated in accordance with MTO's terms of reference, once electronic files are received for use in drawing preparation.

DRAFT PRELIMINARY

Date	Feb 2003	Golder Associates	Drawn	LCC
Project	021-1155-1		Chkd	LCC

GRAIN SIZE DISTRIBUTION TEST RESULTS
Surficial Sands and Gravels

FIGURE 1

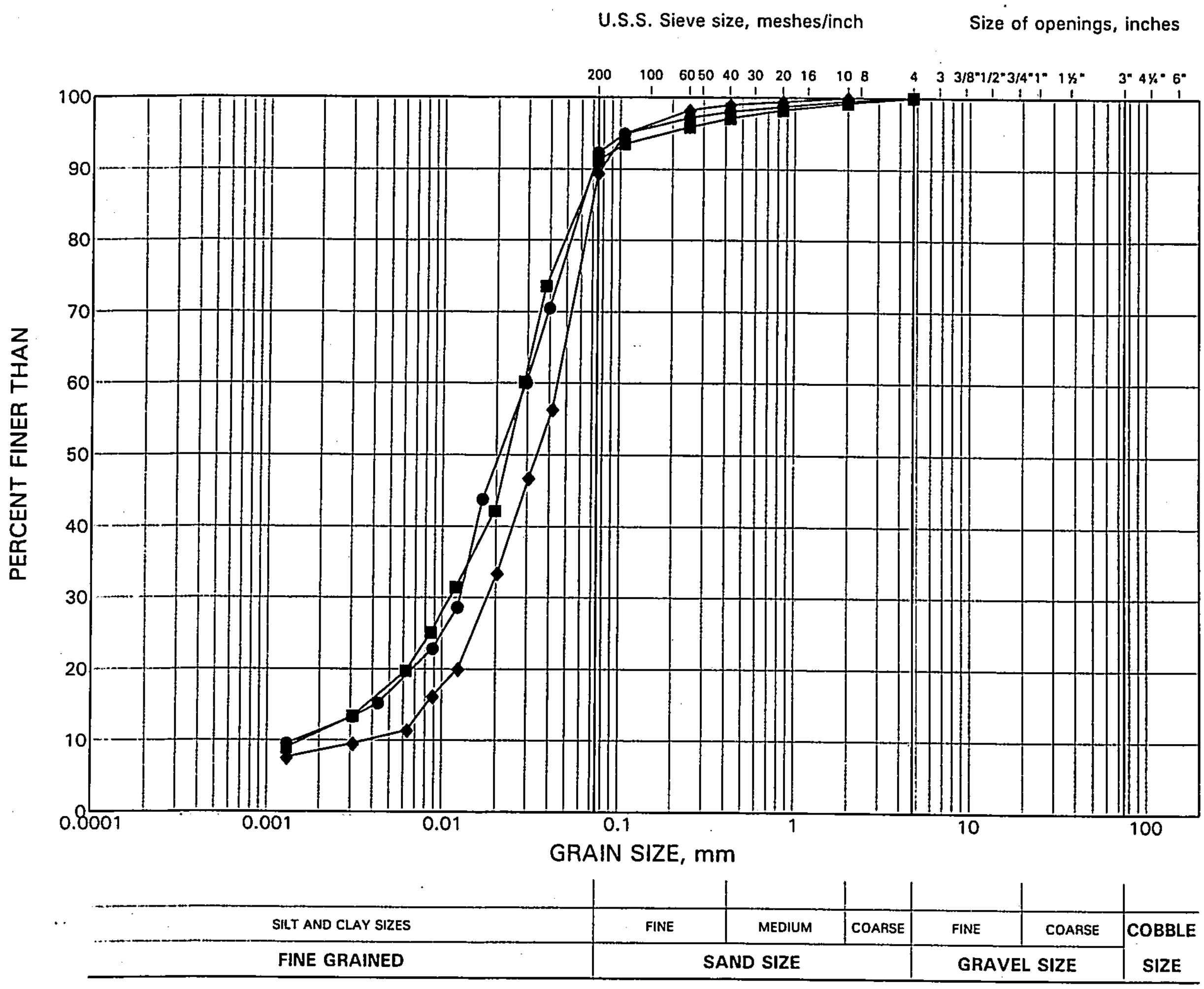


LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	02-101	1A	127.5
■	02-103	1	128.4
♦	02-107	1	127.0
○	02-120	2	128.4

GRAIN SIZE DISTRIBUTION TEST RESULTS
Surficial Silts

FIGURE 2

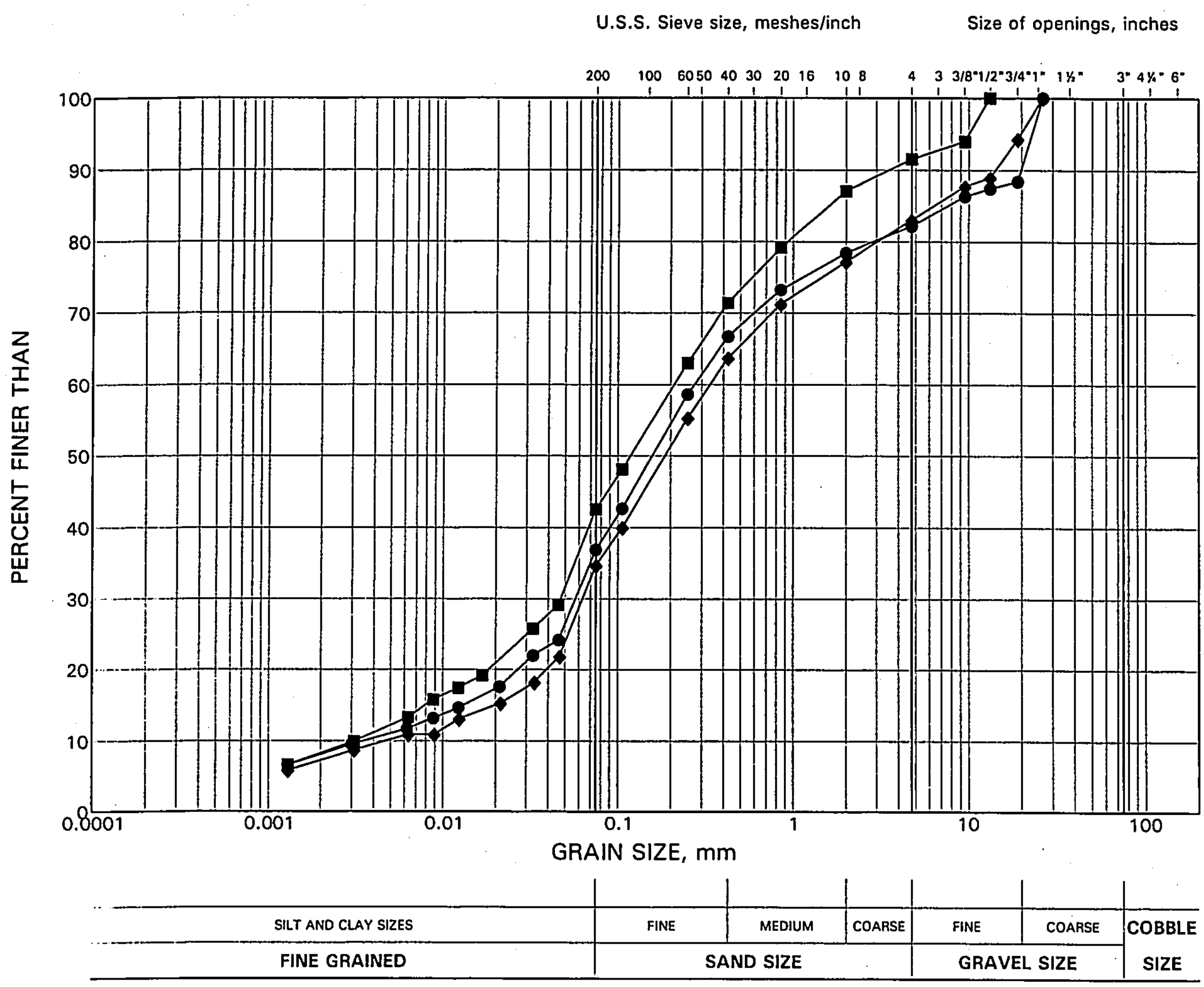


LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	02-101	2A	126.8
■	02-102	1B	127.0
◆	02-120	4	126.8

GRAIN SIZE DISTRIBUTION TEST RESULTS
Silty Sand Till to Sand and Silt Till

FIGURE 3



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	02-105	3	126.4
■	02-121	1B	127.0
◆	02-123	1	128.2