Golder Associates Ltd.

2390 Argentia Road
Mississauga, Ontario, Canada L5N 5Z7
Telephone: (905) 567-4444
Fax: (905) 567-6561

REPORT ON

FOUNDATION INVESTIGATION AND DESIGN
WILDLIFE CROSSING UNDER HIGHWAY 69 AT STATION 18+400
HIGHWAY 69 FOUR-LANING FROM 1.0 KM NORTH OF THE FUTURE INTERCHANGE AT HIGHWAY 637, NORTHERLY 1.8 KM
G.W.P 5379-02-00
MINISTRY OF TRANSPORTATION, ONTARIO

Submitted to:

URS Canada Inc.
75 Commerce Valley Drive East
Markham, Ontario
L3T 7N9

GEOCRES No. 41I-201

DISTRIBUTION

3	Copies	-	Ministry of Transportation, Ontario, North Bay, Ontario (Northeastern Region)
1	Copy	-	Ministry of Transportation, Ontario, Downsview, Ontario (Foundations Section)
2	Copies	-	URS Canada Inc. Markham, Ontario
2	Copies	-	Golder Associates Ltd., Mississauga, Ontario

July 2006 06-1111-001-1

TABLE OF CONTENTS

SECTION **PAGE**

PART A - FOUNDATION INVESTIGATION REPORT

PART B - FOUNDATION DESIGN REPORT

LIST OF TABLES

LIST OF FIGURES

LIST OF DRAWINGS

LIST OF APPENDICES

PART A

FOUNDATION INVESTIGATION REPORT
WILDLIFE CROSSING UNDER HIGHWAY 69 AT STATION 18+400
HIGHWAY 69 FOUR-LANING FROM 1.0 KM NORTH OF
THE FUTURE INTERCHANGE AT HIGHWAY 637, NORTHERLY 1.8 KM
G.W.P 5379-02-00
MINISTRY OF TRANSPORTATION, ONTARIO

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by URS Canada Inc. (URS) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services as part of the design of the Wildlife Crossings under and over the future Highway 69 Four-Laning between the interchange at Highway 637 and Sheppard Lake, in the vicinity of Burwash, Ontario. The general location of the Wildlife Crossings and the future Highway 69 alignment in this area is shown on the Site Location Map on Figure 1.

This report addresses the Wildlife Crossing Under Highway 69 structure and associated approach embankments only. A report detailing the foundation investigation for the Wildlife Crossing Over Highway 69 structure is provided under separate cover.

The terms of reference for the scope of work are outlined in Golder's proposal P51-1786, dated November 2005, that forms part of the Consultant's Agreement (Number P.O. 5005-E-0032) for this project. The work was carried out in accordance with the Quality Control Plan for this project dated February 20, 2006. The latest preliminary General Arrangement drawing for the preferred Wildlife Crossing Under Highway 69 structure was provided to Golder by URS on June 6, 2006. The preferred structure consists of twin double-span bridges that will allow the wildlife to cross under the highway. Each bridge is comprised of a deck supported on CPCI girders with individual span lengths of 38.5 m for a total bridge span length of 77 m at both the northbound lane (NBL) and southbound lane (SBL).

It should be noted that the initial preliminary General Arrangement drawing provided by URS to Golder in May 2006, at the time of the field investigation for this crossing, consisted of twin 53.4 m single-span structures. As such, the boreholes were located to provide subsurface information at the foundation units for the single-span bridges only. Although the locations of the north abutment and north approach for the NBL and SBL structures are coincident for both the double-span and the original single-span arrangements, the south abutments and south approaches are offset by about 24 m between the two options. In addition, the double-span arrangement has an extra foundation unit (i.e. the pier) at each bridge. Since the single-span structure was the preferred option at the time of the field investigation, no boreholes have been advanced within the footprints of the proposed centre piers, south abutments or south approach embankments for the currently preferred double-span arrangement.

It is our understanding that at present the twin double-span bridges are the preferred arrangement for the Wildlife Crossing Under Highway 69 at this location and, as such, this report addresses the foundation design for this arrangement. However, considering the inherent limitations as a result of the absence of subsurface information within some of the foundation units for this arrangement, this report should not be used for detail design and it is recommended that additional boreholes be put down prior to the final design of this crossing to address the unknowns and assumptions made in this report. For detail design, boreholes should be advanced to obtain subsurface information within the footprints of the proposed centre piers, south abutments and south approach embankments.

2.0 SITE DESCRIPTION

The proposed Wildlife Crossing Under Highway 69 structures is located in the Township of Servos, approximately 2.4 km north of the existing Highway 69 and Highway 637 intersection and approximately 0.7 km east of the existing Highway 69. The proposed Wildlife Crossing is part of the proposed new four laning of Highway 69 from 1 km north of the future interchange at Highway 637, northerly 1.8 km (at about Station 18+400).

In general, the topography in the area of the project consists of rolling terrain, including densely treed areas and numerous bedrock outcrops separated by low-lying swamp areas. The proposed Wildlife Crossing Under Highway 69 structure is to be situated in a swampy, topographic low with bedrock outcrops exposed at ground surface to the north-west and south-east areas of the site. Bedrock exists at the site at depths ranging from less than 2 m to more than 15 m below ground surface. The ground surface within the limits of the proposed structure and approach embankment areas generally lies between about Elevations 213.2 m (to the east) and 217.1 m (to the west), referenced to Geodetic Datum.

3.0 INVESTIGATION PROCEDURES

3.1 Foundation Investigation

The field work at the Wildlife Crossing Under Highway 69 site was carried out between April 27 and May 3, 2006 and on May 15, 2006 at which time nineteen (19) boreholes, numbered 06-1 to 06-15, 06-25 to 06-27 and 06-12A and one (1) Dynamic Cones Penetration Test (DCPT) were advanced. The location of these boreholes are shown on Drawing 1.

The field investigation was carried out using a track-mounted CME-55 drill rig supplied and operated by Marathon Drilling Ltd. of Ottawa, Ontario. The boreholes were advanced using 108 mm inside diameter (I.D.) continuous flight hollow stem augers. Soil samples were obtained at intervals ranging from 0.75 m to 1.5 m in depth, using a 50 mm outer diameter (O.D.) split-spoon sampler, in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586-99), or using 76 mm O.D. thin-walled ‘Shelby’ tubes (ASTM D1587-00) for relatively undisturbed samples in cohesive soils. Samples of the bedrock were obtained using an ‘NQ’ size rock core barrel. Field vane shear tests were conducted (or attempted) in cohesive soils for assessing undrained shear strengths (ASTM D 2573-01). It should be noted that boreholes 06-25 to 06-27 were put down specifically to carry out field vane shear tests and to obtain ‘Shelby’ tube samples from the cohesive portions of the subsurface strata and no SPT samples were obtained in these boreholes.

The boreholes and the DCPT were advanced to depths ranging from about 2.2 m to 15.0 m below the existing ground surface (not including rock coring). All of the boreholes (except 06-12, 06-12A and 06-25 to 06-27) were advanced to refusal on probable bedrock. At boreholes 06-01, 06-05, 06-06 and 06-07 (located within the footprints of the proposed north abutment foundation units) and at boreholes 06-10 and 06-13 (located about halfway between the pier and south abutment foundation units) the drilling was further advanced into the bedrock by coring between about 3 m and 3.5 m.

The groundwater conditions in the open boreholes were observed during the drilling operations and one piezometer was installed in borehole 06-11 to permit monitoring of the groundwater level at this location. The piezometer consisted of 25 mm outside diameter rigid PVC tubing with a 1.0 m long slotted tip sealed at a selected depth within the borehole. The installation details and water level readings are described on the Record of Borehole and Drillhole sheets that follow the text of this report. All boreholes and the piezometer were abandoned in accordance with O. Reg. 128 (amendment to O. Reg. 903).

The field work was supervised throughout by members of our technical staff, who located the boreholes, arranged for the clearance of underground service locations, supervised the drilling, sampling and in situ testing operations, logged the boreholes, and examined and cared for the soil and rock samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Mississauga geotechnical laboratory where the samples underwent further detailed visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards as appropriate. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected samples. In addition, a one-dimensional consolidation (oedometer) test was carried out on a select sample from the cohesive deposit. The results of the laboratory testing are included in Appendix A.

The boreholes were laid out in the field by Callon Dietz Inc. (a sub-contractor to URS) using the UTM NAD 83 Zone 10 co-ordinate system and the geodetic datum for elevations. Where boreholes were shifted away from the originally proposed location at the time of drilling, the station, offset and elevation of the as-drilled boreholes were measured in the field relative to the existing staked locations by members of our technical staff.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

From published geologic information, the site is located in the physiographic region known as the Abitibi Uplands that form the central section of the Canadian Shield (Geology of Ontario; OGS Special Volume 4). The Abitibi Uplands form a rocky landscape, scattered with lakes and large areas which are mantled by deposits from Pleistocene glaciation consisting of the lacustrine clays and former shorelines of pre-glacial lakes. Landforms include outwash channels, tills and moraines. The local physiography is generally characterized by variable overburden materials including clayey silt, silt, sand, gravel, cobbles and boulders and an irregular, variable bedrock surface with rock outcrops.

4.2 Subsoil Conditions

The detailed subsurface soil, bedrock and groundwater conditions as encountered in the boreholes advanced during this investigation, together with the results of the laboratory tests carried out on selected soil samples, are given on the attached Record of Borehole and Drillhole sheets following the text of this report. The results from the laboratory testing are provided in Appendix A. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling and observations of drilling progress and the results of Standard Penetration Tests (SPT). These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole locations. The inferred soil stratigraphy based on results of the boreholes at the bridge location is shown on Drawings 2 and 3.

In general, the subsoils at the site consist of surficial deposits of topsoil/organics and in some places fill and/or a thin layer of silty sand, overlying a varved to layered clayey silt stratum. Below the clayey silt stratum, deposits of silt grading to sandy silt to sand and silt till are present, and are in turn underlain by bedrock. A layer of cobbles and boulders was encountered below the sand and silt till (immediately overlying the bedrock) in one borehole and the presence of cobbles in the sand and silt till stratum was inferred from resistance encountered during auger advance in a number of other boreholes. The overburden thickness at the site is variable, ranging from about 2 m to 15 m at the borehole locations. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Topsoil/Fill

A deposit of silty organics/topsoil was encountered at the ground surface in all boreholes except 06-14 and 06-15 where it was encountered below a layer of fill. The surface of the topsoil (i.e. ground surface except in 06-14 and 06-15) ranged between Elevations 212.9 m and 216.9 m and the thickness ranged from about 0.1 m to 0.5 m at the borehole locations.

A layer of fill was encountered at the ground surface in boreholes 06-14 and 06-15. The fill is described as light brown to reddish brown sand containing trace silt and gravel. The surface of the fill (i.e. ground surface) was encountered at Elevation 214.4 m and 214.2 m in boreholes 06-14 and 06-15, respectively, and the fill ranged between 0.6 m to 0.8 m in thickness.

The Standard Penetration Testing (SPT) ‘N’ values measured within the topsoil range from about 1 blow to 5 blows per 0.3 m of penetration, indicating a very loose to loose relative density.

The natural water content measured on one sample of the topsoil was about 32 percent.

4.2.2 Silty Sand to Sand

A thin layer of silty sand to sand was encountered below the topsoil/organics in boreholes 06-08, 06-09, and 06-14. The top of the sand layer ranged between Elevation 213.5 m and 216.9 m and the thickness ranged from about 0.1 m to 0.6 m. Trace gravel, cobbles and organics and some signs of oxidation were noted within this layer.

A single Standard Penetration Test (SPT) ‘N’ value measured within the sand deposit was about 5 blows per 0.3 m of penetration, indicating a loose relative density.

4.2.3 Clayey Silt

A deposit of clayey silt was encountered below the organics/topsoil and below the silty sand to sand layer in all of the boreholes. The top of this deposit ranged between Elevation 212.9 m and 216.3 m and the thickness ranged from about 0.3 m to 3.8 m with an average of about 2.6 m. The upper 0.1 m to 0.2 m of the clayey silt was found to contain some black organics at some of the borehole locations. In general, the deposit is described as varved to layered, light brown to reddish brown and grey, clayey silt containing trace sand, gravel, cobbles and occasionial clay and sand seams.

At the borehole locations, Standard Penetration Test (SPT) ‘N’ values ranged between 0 (i.e. weight of hammer) to 19 blows per 0.3 m of penetration (on average 6 blows per 0.3 m of penetration). In situ field vane testing was carried out within this deposit in boreholes 06-10 and 06-25 to 06-27 using a standard MTO 'N' vane. The measured undrained shear strengths ranged from 52 kPa to greater than 96 kPa. Sensitivity was found to range from 3.7 to 11.2. In general, the field vane test results together with the SPT 'N' values suggest the clayey silt stratum has a very stiff to firm consistency. The results of the vane testing is summarized below:

Borehole	***Location***	***Sample Depth/Elevation (m)***	***Undisturbed Shear Strength (kPa)***	***Remoulded Shear Strength (kPa)***	***Sensitivity***
06-10	NBL bridge/ offset 26 m North of South Abutment	2.4 / 211.0	82	22	3.7
		2.6 / 210.8	>96	-	-
06-25	NBL bridge/ North abutment	1.1 / 212.3	>96	-	-
		1.8 / 211.6	>96	-	-
		2.1 / 211/3	>96	-	-
		2.6 / 210.8	54	4.8	11.2
		2.9 / 212.0	52	11.5	4.5
		3.4 / 210.0	54	10.5	5.1
06-26	SBL bridge/ North abutment	1.1 / 213.8	>96	-	-
		1.4 / 213.5	>96	-	-
		1.8 / 213.1	>96	-	-
		2.1 / 212.8	>96	-	-
		2.6 / 212.3	77	10.5	7.3
		2.9 / 212.0	78	19.2	4.1
		3.4 / 211/6	77	14.4	5.3
06-27	SBL bridge/ offset 21 m North of South abutment	1.1 / 212.4	>96	-	-
		1.8 / 211.7	>96	-	-
		2.1 / 211.4	>96	-	-

Atterberg limits testing was carried out on seven (7) samples from the clayey silt deposit. The liquid limit ranged from about 25 to 34 percent and the plastic limit ranged from about 20 to 22 percent, yielding a plasticity index ranging from about 5 to 12 percent. The test results, summarized below and shown on the plasticity chart on Figure A-1 Appendix A, indicate that the material is typically clayey silt of low plasticity.

Borehole	***Sample***	***Elevation (m)***	***Liquid Limit (%)***	***Plastic Limit (%)***	***Plasticity Index (%)***
06-5	3	211.0	25	20	5
06-6	2	211.9	31	21	10
06-7	3	211.6	25	20	5
06-8	2	213.1	32	21	11
06-10	2	211.6	34	22	12
06-10	3	210.0	27	21	6
06-14	3	211.8	28	22	6
Average	-	-	32	21	8

The natural water content measured on samples from this deposit ranged between about 22 and 48 percent, with an average of about 30 percent.

A grain size distribution curve for one (1) sample from this deposit (borehole 06-05, Sa#3) is shown on Figure A-2 in Appendix A.

A laboratory consolidation (i.e. oedometer) test was carried out on a specimen of the clayey silt obtained from borehole 06-26. A preconsolidation pressure ranging from about 250 to 280 kPa was estimated from the void ratio versus logarithmic pressure plot and from the total work versus pressure plot. Details of the test results are shown on Figure A-3 in Appendix A. The following relevant consolidation test results are summarized as follows:

Borehole and Sample No.	***Elevation (m)***	σ_{vo}' ***(kPa)***	σ_p' ***(kPa)***	***OCR***	e_o	C_r	C_c	c_v^* (cm^2/s)
06-26 Sa#1	212.3	24	250	10	0.78	0.03	0.19	2.9×10^{-2}

Note: *For stress range of $20 \leq \sigma_v' \leq 310$ kPa

where:
σ_{vo}' effective overburden pressure in kPa
σ_p' preconsolidation pressure in kPa
OCR overconsolidation ratio
e_o initial void ratio
C_c compression index (based on void ratio)
C_r recompression index (based on void ratio)
c_v coefficient of consolidation in cm^2/s in the normally consolidated range

4.2.4 Silt

A deposit of silt was encountered below the clayey silt stratum in boreholes 06-01, 06-02, 06-06, 06-08, 06-10 to 06-14, and 06-12A. In general, the silt is described as grey with some sand, trace to some clay and containing occasional sand or clay seams and layers. The top of this deposit ranged between Elevation 209.0 m and 211.6 m and the thickness ranged from about 0.8 m to 3.9 m with an average of about 2.0 m. Boreholes 06-02, 06-08, 06-12 and 06-14 terminated within this deposit at depths ranging from 3.1 m to 5.8 m below ground surface.

The measured SPT ‘N’ values within the silt deposit ranged from 3 to 10 blows per 0.3 m of penetration, indicating a very loose to compact relative density. Typically, the deposit is loose.

The natural water content measured on samples from the silt deposit range from about 21 to 34 percent, with an average of about 26 percent.

Grain size distribution curves for two (2) selected samples from this deposit are shown on Figure A-4 in Appendix A.

4.2.5 Sandy Silt to Sand and Silt (Till)

A deposit of sandy silt to sand and silt till was encountered below the clayey silt or silt strata in boreholes 06-01, 06-03 to 06-07, 06-9 to 06-11, 06-12A and 06-13.

In general, the sandy silt to sand and silt (till) is described as light brown to grey with trace clay, trace to some gravel and cobbles and containing silt to silty sand seams. The top of this deposit ranged between Elevation 205.8 m and 214.8 m and the thickness ranged from about 0.2 m to 7.4 m with an average of about 2.1 m. The bottom of this deposit (where encountered) was generally defined by refusal to further auger or DCPT advancement and was confirmed by rock coring in select boreholes. The sand and silt till deposit was generally encountered in the boreholes drilled at the north to northeast corner of the site (i.e. boreholes 06-01 and 06-04 to 06-07).

The measured SPT ‘N’ values within the sandy silt to sand and silt till deposit ranged between 0 (i.e. weight of hammer) and 25 blows per 0.3 m of penetration, indicating a very loose to compact relative density.

The natural water content measured on samples from the sandy silt to sand and silt (till) deposit range from about 23 to 24 percent.

Grain size distribution curves for three (3) selected samples from this deposit are shown on Figure A-5 in Appendix A..

4.2.6 Cobbles and Boulders

A layer of cobbles and boulders was encountered below the sand and silt stratum in borehole 06-07 at Elevation 209.5 m. This layer was about 0.8 m thick and was encountered directly over the bedrock.

4.2.7 Bedrock

Bedrock was encountered and cored in boreholes 06-01, 06-05 to 06-07, 06-10 and 06-13. The presence of bedrock was inferred from auger refusal in all of the other boreholes except 06-12 (where it is inferred from DCPT refusal) and 06-25 to 06-27 (where it was not encountered). At the borehole locations, the bedrock surface (as confirmed by coring and/or inferred from auger refusal) ranges from as high as Elevation 214.6 m to as low as Elevation 198.4 m.

The details of the bedrock surface and inferred bedrock surface elevations at the borehole locations are summarized below:

Structure	***Location***	***Borehole***	***Coring/Auger or DCPT Refusal***	***Ground Surface Elevation (m)***	***Depth to Bedrock (m)***	***Bedrock Surface Elevation (m)***
North Bound Lane Bridge	Offset 6 m north of South Abutment	06-12A	DCPT refusal	213.4	12.6	200.8
	Offset 13 m south of Central Pier	06-10	Cored	213.4	13.1	200.3
		06-11	Auger Refusal	213.4	15.0	198.4
	North abutment	06-01	Cored	213.7	3.4	210.3
		06-02	Auger refusal	213.7	3.1	210.6
		06-04	Auger refusal	213.6	4.4	209.2
		06-05	Cored	213.6	3.8	209.8
		06-06	Cored	213.7	3.7	210.0
	North approach	06-03	Auger refusal	214.4	2.2	212.2
South Bound Lane Bridge	Offset 6 m north of South Abutment	06-15	Auger refusal	214.2	2.9	211.3
	Offset 13 m south of Central Pier	06-13	Cored	213.5	6.2	207.3
		06-14	Auger refusal	214.4	4.5	209.9
	North abutment	06-07	Cored	214.2	5.5	208.7
		06-08	Auger refusal	214.9	5.8	209.1
	North approach	06-09	Auger refusal	217.1	2.5	214.6

The bedrock surface elevation is highly variable across the site as well as across each proposed foundation location, where investigated. The bedrock generally slopes downward towards the south across the site. At the north abutments the bedrock surface varies up to about 1.6 m. In the boreholes drilled in the vicinity of the south abutment the bedrock surface varies up to about 2.6 m.

The bedrock samples are described as pink, black and grey, slightly weathered, medium grained granitic gneiss with healed and partially healed joints. Some of the joints were healed with hematite and quartz carbonate. The total core recovery was 100 percent. The Rock Quality Designation (RQD) measured on the core samples from the boreholes typically ranged from about 61 to 100 percent, typically about 82 percent. This indicates a rock mass of fair to excellent quality, typically good. In borehole 06-01 a RQD value as low as about 38 percent was measured, indicating a rock mass of poor quality at this location.

Point load strength tests were performed on selected samples of the rock core from borehole 06-7 and 06-10 and the results are summarized in Table 1. Axial and diametral point load strength index values are shown on Table 1 and diametral point load strength index values are shown on the Record of Drillhole Sheets following the text of this report. Diametral point load index values on samples of the granitic gneiss range from 5.4 MPa to 9.0 MPa, which corresponds to an estimated unconfined compressive strength (UCS) ranging from about 109 MPa to 181 MPa. The axial point load index values on the samples of the granitic gneiss range from 6.7 MPa to 9.3 MPa corresponding to approximate UCS values ranging from about 135 MPa to 186 MPa. Using the Intact Rock Strength Classification table, these values indicate that the granitic rock is classified as very strong.

4.2.8 Groundwater Conditions

In general, the samples taken in the overburden from the boreholes were noted to be moist to wet. The water levels in the open boreholes ranged between about Elevation 211.3 m and 215.5 m upon completion of drilling, corresponding to depths of about 3.1 m below ground surface and 2.1 m above ground surface (i.e. artesian conditions), respectively.

The groundwater level in the piezometer installed at the interface between the silt and sandy silt till strata in borehole 06-11 was measured at Elevation 215.3 m (1.9 m above ground surface) on May 15, 2006. Details of the piezometer installation, groundwater conditions and water levels observed in the open boreholes at the time of drilling are summarized on the Record of Borehole sheets following the text of this report. The water levels measured in the piezometers and open boreholes upon completion of drilling and in the piezometer about two weeks later are summarized below. It should be noted that groundwater levels in the area are subject to seasonal fluctuations.

Structure	*Location*	*Borehole*	*Measured in*	*Ground Surface Elevation (m)*	*Water Level Depth (m)*	*Water Level Elevation (m)*	*Date*
North Bound Lane Bridge	Offset 6 m north of South Abutment	06-12A	Open hole	213.4	2.1	211.3	May 1, 2006
	Offset 13 m south of Central Pier	06-10	Open hole	213.4	2.1^{+}	215.5	April 30, 2006
		06-11	Piezometer	213.4	1.9^{+}	215.3	May 15, 2006
	North abutment	06-01	Open hole	213.7	0.8^{+}	214.5	April 27, 2006
		06-02	Open hole	213.7	1.2	212.5	April 27, 2006
		06-04	Open hole	213.6	2.3	211.3	April 28, 2006
		06-05	Open hole	213.6	0.9^{+}	214.5	April 28, 2006
		06-06	Open hole	213.7	$>0^{+}$	>213.7	April 29, 2006
		06-25	Open hole	213.6	1.8	211.8	May 15, 2006

Structure	*Location*	*Borehole*	*Measured in*	*Ground Surface Elevation (m)*	*Water Level Depth (m)*	*Water Level Elevation (m)*	*Date*
	North approach	06-03	Open hole	214.4	0.8	213.6	April 27, 2006
South Bound Lane Bridge	Offset 6 m north of South Abutment	06-15	Open hole	214.2	2.8	211.4	May 2, 2006
	Offset 13 m south of Central Pier	06-13	Open hole	213.5	>0$^+$	>213.5	May 2, 2006
		06-27	Open hole	213.5	2.4	211.1	May 15, 2006
		06-14	Open hole	214.4	3.1	211.3	May 2, 2006
	North abutment	06-07	Open hole	214.2	0.4	213.8	April 29, 2006
		06-08	Open hole	214.9	1.4	213.5	April 29, 2006
	North approach	06-09	Open hole	217.1	2.2	214.9	April 30, 2006

Note: $^+$indicates artesian groundwater condition present

4.3 Closure

This Foundation Investigation Report was prepared by Ms. Karyn Gallant and reviewed by Mr. J. Paul Dittrich, Ph.D., P. Eng., an Associate with Golder. Mr. Jorge M. Costa, P. Eng., Golder's Designated MTO Contact for this project, conducted an independent quality review of the report.

GOLDER ASSOCIATES LTD.

for /
Karyn Gallant
Geotechnical Group

Paul Dittrich, Ph.D., P.Eng.,
Associate, Senior Geotechnical Engineer

LICENSED PROFESSIONAL ENGINEER
J. P. DITTRICH
PROVINCE OF ONTARIO

Jorge M. A. Costa, P.Eng.,
Principal, Designated MTO Contact

LICENSED PROFESSIONAL ENGINEER
J. M. A. COSTA
PROVINCE OF ONTARIO

KG/JPD/JMAC/kg/jpd

n:\active\2006\1111\06-1111-001 urs-hwy69 wildlife crossings\reporting\final\06-1111-001 wildlife crossing under hwy 69\06-1111-001-1 rpt 06july final wildlife xing under hwy 69 report.doc

PART B

FOUNDATION DESIGN REPORT
WILDLIFE CROSSING UNDER HIGHWAY 69 AT STATION 18+400
HIGHWAY 69 FOUR-LANING FROM 1.0 KM NORTH OF THE FUTURE INTERCHANGE AT HIGHWAY 637, NORTHERLY 1.8 KM
G.W.P 5379-02-00
MINISTRY OF TRANSPORTATION, ONTARIO

5.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides recommendations on the foundation aspects of the proposed bridge structures carrying the new Highway 69 NBL and SBL alignments over the Wildlife Crossing Under Highway 69 (at about Station 18+400). The recommendations are based on interpretation of the factual geotechnical data obtained from the boreholes advanced during the subsurface investigation at the 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.

5.1 General

As per the general arrangement drawing provided to Golder by URS on June 6, 2006, the Wildlife Crossing Under Highway 69 is proposed to be comprised of twin double-span structures to carry the NBL and SBL alignments of the new Highway 69 over the low lying area at about Station 18+400. Each two-span bridge is comprised of a deck supported on CPCI girders with individual span lengths of 38.5 m for a total bridge length of 77 m. The bridge decks are to be at approximate Elevation 223.0 m. The existing ground surface varies from about Elevation 213.4 m near the south abutments to between about Elevation 214.3 m and 217.1 m at the north abutments, requiring approach embankments up to about 10 m high.

It should be noted that the initial preliminary General Arrangement drawing provided by URS to Golder in May 2006, at the time of the field investigation for this crossing, consisted of twin 53.4 m single-span structures. As such, the boreholes were located in the field to provide subsurface information at the foundation units for the single-span bridges only. Although the locations of the north abutment and north approach for the NBL and SBL bridges are coincident for both the double-span and the original single-span arrangements, the south abutments and south approaches are offset by about 24 m between the two options. In addition, the double-span arrangement has an extra foundation unit (i.e. the pier) at each bridge. Since the single-span structure was the preferred option at the time of the field investigation, no boreholes have been advanced within the footprints of the proposed centre piers, south abutments or south approach embankments for the currently preferred double-span arrangement. As such, all ground surface elevations and bedrock surface elevations at the south approaches, south abutments, and the centre piers for the NBL and SBL structures presented in this report have been interpreted from the information obtained during the field investigation carried out for the originally proposed single-span arrangement. This information is considered an estimate only and must be confirmed by additional field investigation prior to the final design. For detail design, boreholes should be advanced to obtain subsurface information within the footprints of the proposed centre piers, south abutments and south approach embankments.

5.2 Bridge Foundation Options

Based on the subsurface conditions encountered in the boreholes, the native soils at this location consist of topsoil/organics and in places silty sand, underlain be successive deposits of layered to varved clayey silt, silt and sandy silt to silt and sand (till) over bedrock. Cobbles and boulders were sometimes encountered within or below the sandy silt to silt and sand till overlying the bedrock. Fill was encountered at the ground surface in the boreholes located near the south abutment of the southbound lane structure. The bedrock underlying the overburden soils is a described as a very strong, medium grained, granitic gneiss of generally good quality. The groundwater level as encountered in the boreholes is variable at this site ranging from about 0.5 m to 3 m below ground surface to up to about 2 m above ground surface (i.e. artesian condition).

The depth to the bedrock below the original ground surface is highly variable across the site and ranges from as shallow as about 2 m to as deep as about 15 m at the investigated locations. The following summarizes the measured or inferred/estimated depth to bedrock at the foundation units for the NBL and SBL structures:

Structure Location	***Foundation Unit***	***Approximate Depth to Bedrock (m)***
North Bound Lane	South Abutment	12 to 13*
	Centre Pier	11 to 15*
	North Abutment	3.1 to 4.4
South Bound Lane	South Abutment	2 to 3*
	Centre Pier	5 to 7*
	North Abutment	5.5 to 5.8

Note: *implies inferred depth to bedrock based on information from closest boreholes and requires confirmation at detail design stage.

Due to the nature of the subsurface soil and groundwater conditions and highly variable depth to bedrock at this site, shallow footings either perched within the embankment fill (i.e. at the abutments on well compacted granular material) or founded on the overburden soils or on the bedrock are not recommended for supporting the bridge abutments or piers. Given the very stiff to firm consistency of the near surface clayey silt stratum and the very loose to compact relative density of the underlying silts and sands, supporting the bridge abutments or piers on shallow spread footings is not recommended due to the low axial geotechnical resistance and expected settlement of these strata. In addition, shallow spread footings on bedrock (i.e. at the north abutment of the NBL structure or at the south abutment of the SBL structure) is also not recommended due to the potential for difficult construction and excavation and need for dewatering given the artesian groundwater conditions encountered at or near these locations. Therefore, supporting the abutments and piers on piles driven to bedrock is considered to be the most technically feasible option from a foundations perspective based on the currently available information.

Based on the preliminary General Arrangement drawing for the twin double-span structures for this Wildlife Crossing (as provided by URS), it is understood that a pile foundation option with integral abutments is being considered for these structures. The details of the recommendations for this option are presented in the following sections. A summary of the advantages/disadvantages, relative costs and risk/consequences for the various foundation alternatives considered for this site is presented in Table 2 following the text of this report.

5.3 Steel H-Pile Foundations

As noted above, steel H-piles driven to refusal on the granitic gneiss bedrock may be used for support of the integral abutments and piers of the proposed NBL and SBL structures. If corrugated steel pipes (CSPs) are installed as part of the integral abutment design (through which the piles will be driven), the CSPs should be loosely backfilled with an uncompacted, fine to medium sand. A NSSP detailing the gradation of this sand should be included in the Contract Documents (see example in Appendix B).

For preliminary design, the following pile tip elevations may be assumed for piles terminating on the bedrock surface. The elevations for the north abutments have been assessed based on a review of the depth to bedrock as encountered in boreholes put down within the footprints of these foundation elements. The elevations for the centre piers and south abutments have been estimated/inferred based on a review of the depth to bedrock as encountered in boreholes located in the vicinity of the footprints of these foundation elements. It is recommended that additional boreholes be advanced within the footprints of the centre piers and south abutments to confirm the elevation of the bedrock surface prior to final design. There should also be a provision made in the Contract for dealing with varying pile lengths.

Bridge Sructure	*Foundation Unit*	*Approximate Pile Tip Elevation (m)*
North Bound Lane	South Abutment	200 – 201*
	Centre Pier	199 – 202*
	North Abutment	210 – 211
South Bound Lane	South Abutment	211 – 212*
	Centre Pier	207 – 209*
	North Abutment	208 – 209

Note: *implies inferred bedrock elevation based on information from closest boreholes and requires confirmation at detail design stage.

5.3.1 Axial Geotechnical Resistance

For HP 310 x 110 piles driven to practical refusal on the granitic gneiss 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. Where piles are less than 3 m in length, it is recommended that a lower factored axial resistance value of 1,600 kN at ULS be used to provide some leeway for accommodating a small percentage of piles sliding along the bedrock surface given the hardness of the rock, the loose nature of the overburden and the variability in the bedrock surface.

The geotechnical resistance at SLS for 25 mm of settlement (for the length of piles required at this site) will be greater than the factored axial resistance at ULS, since the granitic gneiss bedrock is considered to be an unyielding material; as such, ULS conditions will govern for this foundation type.

5.3.2 Downdrag Load (Negative Skin Friction)

The loading from the up to 10 m high approach embankments adjacent to the abutment foundations will cause consolidation and settlement of the underlying very stiff to firm clayey silt strata. If the piles are installed prior to completion of this settlement, because the piles are end-bearing on bedrock, a small amount of settlement of the clayey silt relative to the stiff pile will result in the development of negative skin friction on the piles. In this case, downdrag loads will need to be taken into account for design of the piles supporting the abutments.

If integral abutment design utilizes a CSP surrounding the entire portion of the piles embedded in the clayey silt strata, downdrag loads may be neglected.

Where the clayey silt foundation soils remain in place and are not preloaded, the structural design of all abutment piles should be based on the full downdrag load acting on the piles. However, given the distance separating the front slope of the approach embankment and the pier foundation units (i.e. about 25 m) downdrag loads do not have to be considered on the pier piles. It should be noted that the unfactored downdrag loads for the south abutments for both the NBL and SBL structures are estimated/interpreted from the subsurface conditions encountered in the boreholes located in the vicinity of these foundations units. It is recommended that boreholes be advanced at the south abutments prior to final design to confirm the subsurface conditions at these locations. The estimated unfactored downdrag loads acting on the HP 310x110 piles at each abutment may be taken as follows:

Foundation Unit	***Unfactored Downdrag Load (kN)***
NBL – north abutment	130
NBL – south abutment	110*
SBL – north abutment	150
SBL – south abutment	115*

Note: *implies estimate only based on inferred soil conditions from closest boreholes and requires confirmation at detail design stage

The downdrag loads indicated above are unfactored loads. The structural capacity of the piles must be checked for the factored dead loads and downdrag loads in accordance with Section C6.8.4 of the *CHBDC Commentary* for ULS conditions. The piles at this location are designed as end-bearing on the bedrock. For this condition (basically classified as non-yielding foundations), the settlement of the piles is largely governed by compression of the pile and will not be greater than 25 mm under the combined SLS and downdrag loading.

Downdrag loads can be reduced or eliminated by either removing and replacing the clayey silt subsoils, utilizing a CSP surrounding the entire portion of the piles embedded with the clayey silt stratum or by constructing preload embankments in the abutment area (as discussed in Section 5.6.4) and allowing the settlement to occur prior to installing the piles.

5.3.3 Lateral Loads (due to Horizontal Soil Deformations)

In addition to downdrag loads, the effect of lateral loading on the piles caused by horizontal soil deformations (i.e. due to consolidation of clayey silt strata and lateral spreading under new embankment loading) should also be considered in the pile design.

Where the clayey silt foundation soils remain in place and are not preloaded prior to pile installation, there will be some additional lateral loads acting on the abutment piles. However, given the very stiff to firm nature of the clayey silt strata, the magnitude of the lateral loads will be moderate and are difficult to quantify given the complex nature of the soil-structure interaction problem. The horizontal component of the soil deformations (i.e. lateral spreading due to the approach embankment loading on the compressible clayey silt soils) are anticipated to be on the order of about 20 mm to 30 mm after Ladd (1991) and the magnitude of this deformation and the effect it will have on the integral abutments should also be considered in the design.

Lateral loads on the piles (and horizontal soil deformations) can be reduced or eliminated by either removing and replacing the clayey silt subsoils or by constructing preload embankments in the abutment areas (as discussed in Section 5.6.4) and allowing the settlement and lateral movement to occur prior to pile installation.

5.3.4 Set Criteria, Rock Points and Driving Shoes

Set criteria are highly dependent on pile driving hammer type and the selected pile. The set criteria can be established through a variety of methods, including empirical correlations and wave equation analyses, at the time of construction once the hammer and pile types are known. The choice of set criteria is dependent on the experience of the engineer and traditional use where a substantial database has been developed over the years. The criteria needs to be set to also avoid overdriving and possibly damaging the piles.

All pile installation/driving should be in accordance with SP 903S01. A refusal rate of 20 blows per 25 mm should not be exceeded in order to prevent/minimize damage to the hammer and the pile.

The piles should be fitted with rock points for adequate seating on the sloping bedrock surface. In addition, given the potential for the presence of cobbles and boulders immediately above the bedrock and/or within the silt and sand till strata overlying the bedrock, the piles should also be provided with driving shoes to reinforce the end of the piles and protect the piles during driving through the cobbles and boulders. To satisfy these two requirements, a Modified Oslo Point (or equivalent) should be utilized that combines both the typical Steel H-Pile Driving Shoe (as per OPSD 3000.100) and the typical Steel HP 310 Oslo Point (as per OPSD 3000.201). A NSSP or Modified OPSD should be included in the Contract Documents to address the requirements for the modified rock points.

5.3.5 Pile Driving Note

The pile driving note to be added to the drawings is Note 4 in Clause 2.5.11 of the Structural Manual – “Piles to be driven to bedrock”.

5.3.6 Resistance to Lateral Loads

The design of piles subjected to lateral loads should take into account such factors as the batter of the piles (if any), the relative rigidity of the pile to the surrounding soil, the fixity condition at the head of the pile (pile cap level), the structural capacity of the pile to withstand bending moments, the soil resistance that can be mobilized, the tolerable lateral deflections at the head of the pile and pile group effects. For a longer, more flexible pile, the maximum yield moment of the pile may be reached prior to mobilization of the lateral geotechnical resistance. For design purposes, both the structural and geotechnical resistances should be evaluated to establish the governing case.

It is understood that integral abutment foundations are being considered for support of the bridge structures. Where very stiff or dense soils are present near the level of the pile cap, the integral abutment design typically consists of surrounding the upper portion of each H-pile with either a double corrugated steel pipe (CSP) liner (with the annulus between the two CSPs unfilled) or a single CSP liner with the space between the pile and the liner filled with uniform grained, uncompacted sand. In either case, this design allows the upper portion of the H-pile to flex more freely. With this design, the passive lateral resistance over the length of the CSP liner may be neglected. However, at sites where the soil at and below the pile cap level is softer or in a looser state, the CSP liner system may not be required because the low lateral resistance of the soil may provide adequate freedom of movement in the system. Given the stiff to very stiff consistency of the clayey silt strata just below the existing ground surface at the sites, it is recommended that the installation of CSP liners be considered to provide the required flexibility in the integral abutment system.

Based on the proposed elevation of the bottom of the CSP liners for the north and south abutments as shown on the GA drawing provided by URS (on June 6, 2006) and considering the depth to bedrock encountered (or inferred) at each abutment foundation unit, the length of H-piles below the CSPs in direct contact with the foundation soils will range from about 1 m to 12 m. The short pile lengths at the north abutment of the NBL structure and the potential for short pile lengths at the south abutment of the SBL structure should be reviewed by the structural designer.

For the relatively long HP 310 x 110 piles driven to bedrock through the very stiff to firm clayey silt and very loose to compact silts and sands at the abutments, the horizontal resistance at Ultimate Limit States (ULS) will be controlled by structural limitations such as the yield moment (M_{YIELD}) of the pile. In this case, the lateral loading will create bending moments in the pile and generate excessive bending stresses in the pile material (CFEM, 1992). However, for the short H-piles driven to bedrock through the thin overburden, the horizontal resistance at Ultimate Limit States (ULS) will be controlled by the lateral capacity of the soil adjacent to the pile. In this case, the lateral loading may exceed the capacity of the soil, resulting in large horizontal movements of the piles (CFEM, 1992).

The following provides an estimate of the factored lateral resistance at ULS for the varying pile lengths at the abutment foundation units.

Foundation Unit	***Factored Lateral Resistance at ULS (kN)***
NBL – north abutment	100
NBL – south abutment	140*
SBL – north abutment	120
SBL – south abutment	120*

Note: *implies estimate only based on inferred soil conditions from closest boreholes and requires confirmation at detail design stage

At Serviceability Limit States (SLS), the horizontal resistance of the piles will be controlled by deflections of the pile heads which may be too large to be compatible with the superstructure. In this case, the horizontal resistance of the pile is calculated based on the coefficient of horizontal subgrade reaction, k_h (kPa/m) of the soil.

The horizontal soil reaction to a vertical pile can be estimated using the following formulae depending on the soil type supporting the pile:

<u>For cohesive soils:</u>

$$k_h = \frac{67 s_u}{B}$$

where s_u is the undrained shear strength of the soil (kPa), as given below; and B is the pile diameter (m).

<u>For cohesionless soils:</u>

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

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

Presented below are the ranges for the values of s_u and n_h at each abutment location that may be assumed in the structural analysis. It should be noted that no boreholes were advanced within the footprint of the south abutments and centre piers for the NBL and SBL structures and the stratigraphy and parameters are estimated/interpreted from the subsoil conditions of the boreholes advanced in the vicinity of these foundation units.

Foundation Unit	***Soil Unit***	***Depth Below Existing Ground Surface (m)***	***s_u (kPa)***	***n_h (kPa/m)***
NBL North Abutment	Very Stiff to firm clayey silt	0 – 2.5	60	-
	Loose silt	2.5 – 3.5	-	1300
	Compact sand and silt (till)	3.5 – 4.1	-	4400
NBL Centre Pier*	Very stiff to stiff clayey silt	0 – 2.5	75	-
	Firm clayey silt	2.5 – 3.5	40	-
	Loose to very loose silt	3.5 – 7	-	1300
	Very loose to loose sand	7 – 14.5	-	2750
NBL South Abutment*	Very stiff to stiff clayey silt	0 – 2	75	-
	Firm clayey silt	2 – 3	40	-
	Loose to very loose silt	3 – 7.5	-	1300
	Very loose to loose sandy silt	7.5 – 12.5	-	1300
SBL North Abutment	Very stiff to stiff clayey silt	0 – 2.5	75	-
	Firm clayey silt	2.5 – 3.5	40	
	Very loose to loose silt to sand and silt till	3.5 – 5.0	-	1300
	Very dense cobbles and boulders	5.0 – 5.5	-	11000
SBL Centre Pier*	Very stiff to stiff clayey silt	0 – 1.5	75	-
	Firm clayey silt	1.5 – 2.5	40	-
	Very loose to loose silt	2.5 – 3.8	-	1300
	Compact sand	3.8 – 4.5	-	4400
SBL South Abutment*	Very stiff to stiff clayey silt	0 – 2.8	75	-

Note: *implies estimate only based on inferred soil conditions from closest boreholes and requires confirmation at detail design stage.

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, R, as follows:

Pile Spacing in Direction of Loading ***d = Pile Diameter***	***Subgrade Reaction Reduction Factor (R)***
8d	1.00
6d	0.70
4d	0.40
3d	0.25

Reference: Foundations and Earth Structures – Design Manual 7.2, NAVFAC DM-7.2. Department of the Navy, Naval Facilities Engineering Command (1982).

The proposed pile spacing at the centre piers and abutments has not been provided by URS. The subgrade reaction reduction factor should be interpolated for pile spacings in between those listed above.

5.3.7 Frost Protection

The pile caps should be provided with a minimum of 2.0 m of soil cover for frost protection.

5.4 Earthquake Consideration

For seismic design purposes, the Site Coefficient, S, for this site in accordance with Section 4.4.6 of the CHBDC may be taken as 1.2, consistent with Soil Profile Type II.

5.5 Lateral Earth Pressure 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 in accordance with *CHBDC* Section 6.9.1(e) and 6.9.2.2 and the associated *Commentary to the CHBDC*.

- Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular ‘A’ or Granular ‘B’ Type II 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 percent 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 Special Provision SP105S10. Other surcharge loadings should be accounted for in the design, as required.

- The granular fill may be placed either in a zone with width equal to at least 2.0 m behind the back of the wall stem (see Case I in Figure C6.9.1(l)(i) 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 (see Case II in Figure C6.9.1(l)(ii) 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 (SSM) or rock fill:

	SSM (sand fill)	**Rock Fill**
Soil / rock unit weight:	20 kN/m^3	19 kN/m^3
Coefficients of static lateral earth pressure:		
Active, K_a	0.33	0.22
At rest, K_o	0.50	0.36

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

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

If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure, may be taken as:

- Rotation of approximately 0.002 about the base of a vertical wall;
- Horizontal translation of 0.001 times the height of the wall; or
- A combination of both.

Seismic (earthquake) loading must also be taken into account in the design in accordance with Section 4.6 of the CHBDC. In this regard, the following should be included in the assessment of lateral earth pressures:

- 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 Table A3.1.7 of the CHBDC, this site is located in Seismic Zone 1. The site-specific zonal acceleration ratio (A) for Sudbury is 0.05. Based on experience, for the overburden soils at the site and embankment heights of up to about 10 m, a 20 per cent amplification of the ground motion may occur (i.e. Site Coefficient, S=1.2), resulting in an increase in the ground surface acceleration from 0.05g to 0.06g. The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of A = 0.06.

- 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. $k_h = 0.03$). For structures that do not allow lateral yielding, k_h is taken as 1.5 times the zonal acceleration ratio (i.e. $k_h = 0.09$). The seismic active and passive earth pressure coefficients are also dependent on the vertical component of the earthquake acceleration, k_v. Three discrete values of vertical acceleration are typically selected for analysis, corresponding to $k_v = +2.3k_h$, $k_v = 0$, and $k_v = -2/3\ k_h$.

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

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Case I (Rock Fill)	**Case II**	
		Granular A	**Granular B Type II**
Yielding wall	0.21	0.26	0.26
Non-yielding wall	0.25	0.30	0.30

Note : These CHBDC seismic K_{AE} values include the effect of wall friction ($\delta = \phi'/2$) and are less than the static values of K_a and K_o reported above for the very low zonal acceleration ratio for this site.

- 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 0.06. This corresponds to displacements of up to 15 mm at this site.

- The earthquake-induced dynamic active lateral 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:

$$p = K \gamma' d + (K_{AE} - K) \gamma' H$$

Where: p is the total (static plus seismic) pressure distribution (kPa)
K is either the static active earth pressure coefficient (K_a) or the static at rest earth pressure coefficient (K_o);
K_{AE} is the seismic active earth pressure coefficient;
γ' is the effective unit weight of the soil (kN/m^3)
  - taken as soil unit weights given above for fill materials
  - taken as 9.2 kN/m^3 for the native materials
d is the depth below the top of the wall (m); and
H is the height of the wall above the toe (m).

5.6 Approach Embankment Design

The construction of the NBL and SBL bridge structures for the Wildlife Crossing will require placement of fill within the limits of the north approach embankments ranging from about 6 m to 9 m, respectively. It is estimated/inferred that the NBL and SBL bridge structures will require placement of fill within the limits of the south approach embankments ranging from approximately 10 m to 9 m, respectively. It is recommended that boreholes be advanced within the footprints of the proposed south approach embankments for the NBL and SBL structures as part of the final design of these structures.

Based on the subsurface information obtained from the boreholes drilled within the limits of the north approach embankments, following removal of the topsoil and organic layers, the NBL and SBL north approach embankments will be founded on a very stiff to firm layered to varved clayey silt, underlain by successive deposits of silt and sandy silt to silt and sand till over bedrock. Based on the boreholes advanced in the vicinity of the south approach embankments (closest boreholes are offset 26 m north), it is inferred that following the removal of surficial fills and topsoil/organic layers, the SBL south approach embankment will be founded on very stiff to firm clayey silt, overlying silt, in turn underlain by bedrock. Similarly, it is inferred that the NBL south approach embankment will be founded on very stiff to firm clayey silt, underlain by successive deposits of silt and sandy silt to silt and sand till underlain by bedrock. At the south approaches, the total thickness of the overburden is inferred to range from about 2 m to 12 m (with the clayey silt stratum ranging from about 2 m to 3 m in thickness), while at the north approaches, the total thickness of the overburden ranges from about 1.9 m to 2.3 m (with the clayey silt stratum ranging from about 0.3 m to 1.5 m in thickness).

The results of stability and settlement analysis for the approach embankments are presented in the following sections.

5.6.1 Stability

Analyses were performed on the critical (i.e. highest) sections of the proposed new approach embankments to assess the stability and liquefaction potential for the proposed heights and geometries.

Limit equilibrium slope stability analyses were performed using the commercially available program SLOPE/W (Version 5.20), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. For all analyses, the factor of safety of numerous potential failure surfaces were computed in order to establish the minimum factor of safety. The factor of safety is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum factor of safety of 1.3 is normally used in the design of embankment slopes under static conditions. This factor of safety is considered adequate for the embankments at this site considering the design requirements and the field data available. The stability analyses were performed to check that the target minimum factor of safety was achieved for the proposed embankment heights and geometries.

As noted above, the subsoils encountered in the area of the north approaches and in the vicinity of the south approaches are composed of a combination of cohesive soils (near the surface) and cohesionless soils (at depth). For the cohesive soils, total stress parameters (i.e. average mobilized undrained shear strength – s_u) were used in the analysis based on the results of the field vane testing as well as from correlations with the SPT results and other laboratory test data. For the cohesionless soils, effective stress parameters were employed in the analysis assuming drained conditions and the shear strength parameters were estimated from empirical correlations using the results of the field SPTs. The correlations proposed by Peck et al. (1974), Schmertmann (1975) and US Navy (1982) were employed and the results were tempered by engineering judgment based on precedent experience in similar soils.

The analysis assumes that all organic soils (encountered at or below the ground surface) will be removed prior to the construction of the new embankments. The piezometric conditions required in the analysis (as they apply to the cohesionless layers at depth) were based on the groundwater level measured in the standpipe piezometer on May 15, 2006 (2 weeks after installation) and were assigned to be 2 m above the existing ground surface (i.e. artesian conditions).

The simplified stratigraphy (i.e. average conditions at the north approaches and conservative conditions at the south approaches) and the associated shear strengths and unit weights employed for the different soil types in the analysis are summarized below. For the purposes of analysis, both earth fill and rock fill were considered for the construction of the approach embankments (as indicated below). Rock fill is assumed to have side slopes at 1.25H:1V and the earth fill is assumed to have side slopes at 2H:1V. A discussion on the different fill types, with respect to stability, is provided in Section 5.6.1.1.

North Approach Embankments

Soil Type	***Depth Below Ground Surface (m)***	***Unit Weight (kN/m^3)***	***Strength Parameters***
Rock Fill	(9 m high)	19	c'=0 kPa, ϕ'=40°
Earth Fill (Sand and Gravel)	(9 m high)	21	c'=0 kPa, ϕ'=35°
Very stiff to stiff clayey silt	0 – 1.5	20	s_u = 75 kPa
Firm clayey silt	1.5 – 2.5	19	s_u = 40 kPa
Very loose to loose silt	2.5 – 3.0	18	c'=0 kPa, ϕ'=27°
Loose to compact sand and silt (till)	3.0 – 3.5	18.5	c'=0 kPa, ϕ'=32°

South Approach Embankments (inferred conditions)

Soil Type	***Depth Below Ground Surface (m)***	***Unit Weight (kN/m^3)***	***Strength Parameters***
Rock Fill	(10 m high)	19	c'=0 kPa, ϕ'=40°
Earth Fill (Sand and Gravel)	(10 m high)	21	c'=0 kPa, ϕ'=35°
Very stiff to stiff clayey silt	0 – 2.5	20	s_u = 75 kPa
Firm clayey silt	2.5 – 3.5	19	s_u = 40 kPa
Very loose to loose silt	3.5 – 7.5	18	c'=0 kPa, ϕ'=27°
Very loose to loose sandy silt	7.5 – 12.5	18.5	c'=0 kPa, ϕ'=30°

The results of the stability analyses indicate a Factor of Safety (FoS) > 1.3 for both the side slopes and front slopes of the proposed approach embankments based on the conditions and soil parameters described above. As such, no stabilizing berms are required for the construction of the approaches. Comments on the requirements for side benches or mid-height berms as per the current MTO Northeastern Region guidelines are provided in the following section.

It should be noted that the ground surface elevations and subsurface conditions in the area of the south approaches were estimated based on the information from the boreholes located nearest to this area. At this time, no boreholes have been advanced within the footprints of the south approach embankments.

5.6.1.1 Embankment Fill Types and Berm Requirements

Based on the anticipated embankment fill heights and existing soil conditions, either earth fill or rock fill embankment options may be considered. The different fill alternatives (i.e. earth fill and rock fill) provide relative advantages and disadvantages (as described below) in terms of weight (i.e. driving force and applied load to founding subsoils), post-construction settlement characteristics, construction cost and time, and ease of construction/availability. The recommended fill type to be placed for the approach embankments is summarized in Table 3.

5.6.1.1.1 Earth Fill

The main advantage of using earth fill (i.e. sand and gravel) is the ease of construction and the lack of post-construction settlements within the fill embankment itself. However, this option will require a larger volume of fill and wider right-of-way because the side slopes will be flatter than rock fill slopes. For this project, acceptable earth fill is considered to be suitable locally available and/or imported, granular material.

For earth fill option, the incorporation of a 2 m wide mid-height bench (or berm) into the uniform side slope profile is required wherever the embankment will exceed a height of 8 m.

5.6.1.1.2 Rock Fill

The main advantage of using rock fill is the ability to achieve steeper embankment side slopes. This is useful in areas with limited right-of-ways. In addition, rock fill will likely be available from the rock cuts excavated for the adjacent highway alignment, thus providing an advantage in cost. The disadvantage in using rock fill for the construction of high embankments is that some post-construction settlement of the embankment fill itself will occur within about the first and second year of construction.

For rock fill option, the incorporation of 2 m wide berms (or successive benches) into the uniform side slope profiles is required wherever the embankment will exceed a height of 10 m such that uninterrupted rock fill slope never exceeds a height of 10 m (as per the most recent MTO Northern Region guidelines).

5.6.2 Liquefaction Potential

The liquefaction potential of the soils below the approach embankments 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, and assuming a ground surface acceleration of 0.06g, a factor of safety of greater than 1.0 against liquefaction is obtained for magnitude 7.0 earthquake events under the approach embankment. Pseudo-static methods of embankment stability analysis indicate that a yield acceleration of ranging from approximately 0.16g to 0.22g (at the south and north approaches respectively) results in a factor of safety against side slope instability of 1.0. Based on this yield acceleration and the correlation proposed by Makdisi and Seed (1978), it is estimated that very little additional deformations (i.e. less than about 5 mm) of the embankment could result under the design earthquake event. Localized failures at the embankment toe, resulting in steepening of the embankment side slopes, could occur. Since deep-seated global instability is not anticipated under the design earthquake event, localized toe failures would be mainly a maintenance issue. This should be considered in the life-cycle costing when assessing the relative costs of the works. Alternatively, consideration could be given to sub-excavation and removal of these silty subsoils prior to construction of the approach embankments in order to eliminate the potential for seismically induced instability at the embankment toes.

5.6.3 Settlement

Settlement of the approach embankments can be expected as a result of the loading from the new fills on the compressible foundation soils at this site. In addition, depending on the type of fill materials employed in the construction, settlements may also occur due to compression of the embankment fill itself.

To estimate the magnitude of the expected settlements, analyses were carried out on the critical sections of the proposed approach embankments using spreadsheet calculations. The rate of settlement of the cohesive foundation soils was also assessed by spreadsheet calculations using Terzaghi's one-dimensional consolidation theory.

For these analyses, the critical sections are assumed to correspond to the greatest new embankment heights, approximately 9 m and 10 m for the north and south approaches, respectively. The unit weights and slope profiles for the embankment fill described in Section 5.6.1 were employed in the analyses. The analyses performed assume that the organic soils/topsoil and fills have been removed prior to construction and that rock fill has been used for the embankment construction.

As noted previously, the foundation soils at this site are composed of a combination of cohesive (i.e. clayey silt) and cohesionless (i.e. silt and sand) strata of varying thickness. The immediate compression of the very loose to compact sandy silt to sand and silt was assessed by estimating an elastic modulus of deformation based on the SPT 'N'-values and empirical correlations found in literature by Bowles (1984) and Kulhawy and Mayne (1990).

The consolidation settlement of the very stiff to firm clayey silt layers was assessed using the results of the in situ field vane and SPT tests and the laboratory consolidation tests to estimate the deformation parameters for these soils. In addition, the results of the laboratory index testing were also employed to estimate deformation parameters using empirical correlations proposed in literature by Terzaghi and Peck (1967), Kulhawy and Mayne (1990), Azzouz et al. (1976) and Britto and Gunn (1987).

The over-consolidation ratio (OCR) required in settlement analyses was established using the results of the consolidation test, as well as correlations with the results of the in situ vane shear tests. The following correlation relating preconsolidation pressure to in situ undrained shear strength was employed:

$$\sigma_p' = \frac{s_{u(mob)}}{0.22} \qquad \text{(Mesri, 1975)}$$

where :

$s_{u(mob)} = \mu s_{u(FV)}$ (after Bjerrum, 1973)

σ_p' = preconsolidation pressure

$s_{u(mob)}$ = average mobilized undrained shear strength (kPa)

$s_{u(FV)}$ = undrained shear strength from field vane test (kPa)

μ = Bjerrum's correction factor based on Plasticity Index

The coefficient of consolidation, c_v, required in the analysis of time rate of settlement was established using the results of the consolidation tests and/or from the US Navy (1982) correlation with liquid limits for normally-consolidated soils. Where both consolidation test and liquid limit data were available, c_v was taken as the average of the results from the two tests for a particular area.

In addition to primary consolidation within clays, secondary compression may also occur. Secondary compression is referred to as creep settlement and occurs over a long period of time under a constant stress. The following relationships have been employed for estimating the magnitude of creep settlement over a period of 50 years following the completion of construction at each location.

$$S_c = HC_{\alpha\varepsilon}\log(t_{50}/t_{EOP})$$

$$C_{\alpha\varepsilon} \sim w_n/100 \qquad \text{(after Mesri, 1973)}$$

where :

S_c = Secondary (creep) settlement (mm)
$C_{\alpha\varepsilon}$ = Secondary compression index
H = Initial thickness of compressible clay deposit (mm)
t_{50} = 50 years post construction period
t_{EOP} = Time to reach end of primary consolidation

The settlement analyses for the approach embankments assume that all surficial or near surface organic soils have been removed prior to construction of the new embankments. The piezometric conditions required in the analyses were based on the groundwater levels noted during drilling and measured in the piezometer installation. In general, the groundwater level was assumed to be located at about 2 m above the ground surface (i.e. artesian conditions).

The following sections summarize the simplified stratigraphy, unit weights and deformation parameters employed for the different soils types in the approach areas. In these sections, the maximum estimated settlement of the foundation soils in these areas (due to the loading imposed by the new approach embankment fills) is presented and a discussion on the rate of settlement is included.

5.6.3.1 Settlement of Foundation Soils (North Approaches)

The following simplified stratigraphy and deformation parameters have been developed for and employed in the settlement analysis of the proposed up to 9 m high rock fill embankment at the north approaches.

Soil	*Thickness (m)*	*Unit Weight (kN/m^3)*	*Estimated Deformation Properties*
Rock fill (9 m embankment + removal of 0.3 m organics)	9.3 (high)	19	0.01H
Very stiff to stiff clayey silt	1.5	20	$m_v = 7.5\text{x}10^{-5}$ kPa^{-1}
Firm clayey silt	1.0	19	$m_v = 3.1\text{x}10^{-4}$ kPa^{-1}

Very loose to loose silt	0.5	18	E’ = 2.5 MPa
Loose to compact sand and silt (till)	0.5	18.5	E’ = 10 MPa

Based on the results of the settlement analysis, the maximum total settlement of the foundation soils in the area of the north approaches is estimated to be about 120 mm. This total settlement is estimated to be comprised of about 45 mm of immediate settlement due to compression of the cohesionless soil layers and about 75 mm of time dependent settlement of the cohesive soil layers. It should be noted that if earth fill is used for the construction of the approaches, the settlement of the foundation soils will be approximately 10% greater than those indicated above.

Assuming a coefficient of consolidation (c_v) of about 2.9×10^{-2} cm^2/s (based on the results from the laboratory consolidation test and empirical correlations with liquid limit using US Navy (1982) for an over-consolidated soil) and assuming two-way drainage of the approximately 2.5 m thick clayey silt layer, it is estimated that the about 95 percent of the consolidation settlement will be completed in less than about 1 month.

The magnitude of creep settlement for the clayey silt stratum is expected to be negligible (i.e. less than 5 mm per log-cycle of time) at this location.

5.6.3.2 Settlement of Foundation Soils (South Approaches)

The following simplified stratigraphy and deformation parameters have been inferred (based on the data obtained from the nearest boreholes to these locations) and employed in the settlement analysis of the proposed up to 10 m high rock fill embankment at the south approaches.

Soil	*Thickness (m)*	*Unit Weight (kN/m^3)*	*Estimated Deformation Properties*
Rock fill (10 m embankment + removal of 0.4 m organics)	10.4 (high)	19	0.01H
Very stiff to stiff clayey silt	2.5	20	$m_v = 7.5 \times 10^{-5}$ kPa^{-1}
Firm clayey silt	1.0	19	$m_v = 3.1 \times 10^{-4}$ kPa^{-1}
Very loose to loose silt	4.0	18	E’ = 2.5 MPa
Very loose to loose sandy silt	5.0	18.5	E’ = 5 MPa

Based on the results of the settlement analysis, the maximum total settlement of the foundation soils in the area of the south approaches is estimated to be up to about 600 mm. This total settlement is estimated to be comprised of about 510 mm of immediate settlement due to compression of the cohesionless soil layers and about 90 mm of time dependent settlement of the cohesive soil layers. It should be noted that this maximum settlement is considered to be most applicable to the south approach of the NBL structure were the thickness of the overburden is inferred to be the greatest. At the south approach of the SBL structure, it is anticipated that the overburden could be much thinner and as such, the estimated total settlement would be less. However, the magnitude of the time dependent settlement (i.e. about 90 mm) could be similar at both the NBL and SBL south approaches. It should be noted that if earth fill is used for the construction of the approaches, the settlement of the foundation soils will be approximately 10% higher than those indicated above.

Assuming a coefficient of consolidation (c_v) of about 2.9×10^{-2} cm^2/s (based on the results from the laboratory consolidation test and empirical correlations with liquid limit using US Navy (1982) for an over-consolidated soil) and assuming two-way drainage of the approximately 3.5 m thick clayey silt layer, it is estimated that the about 95 percent of the consolidation settlement will be completed in less than about 1 month.

The magnitude of creep settlement for the silty clay to clay strata is expected to be negligible (i.e. less than 5 mm per log-cycle of time) at this location.

5.6.3.3 Settlement of Rock Fill

If rock fill is used for the construction of the embankments, in addition to the settlement due to compression/consolidation of the foundation soils described above, there will also be settlement due to compression of the rock fill itself. Settlement of the rock fill depends on the type of rock and on the method and sequence of placement and compaction of the fill. Assuming that the rock fill is not end dumped in its final position but rather is placed in accordance with the requirements as outlined in Special Provision SP 206S03, the settlement of the rock fill will likely be about 1% of the effective height of the new fill. As such, for the 9.3 m to 10.4 m high approaches, the embankment rock fill could be expected to settle up to about 95 mm to 105 mm at the north and south approaches, respectively. It is anticipated that the majority (approximately 60%) of this settlement will occur in the first year following construction.

5.6.3.4 Settlement of Earth Fill

Where earth fill (granular fill) is used for the construction of the embankments, the settlement of the approved new embankment fill itself is expected to be less than 25 mm. The majority of settlement will occur during construction.

It is noted that these modest amounts of settlement are conditional on the topsoil, surficial fills and organic soils being stripped and removed from the area of the embankment footprint prior to fill placement.

5.6.4 Settlement Mitigation Measures

As described above, although stability of the approach embankments is not anticipated to be an issue, the presence of the approximately 2.5 m to 3.5 m thick clayey silt strata in the area of the approaches will result in up to about 90 mm of time dependent consolidation settlement following completion of the up to 10 m high embankments.

The following sections outline the options and recommendations for minimizing post-construction settlements that could effect roadway performance and impact the design of the abutment pile foundations. The advantages, disadvantages, relative costs and risks/consequences for the mitigation options at the approaches are also summarized and ranked in Table 4.

5.6.4.1 Preloading

For the up to approximately 3.5 m thick clayey silt strata at this site, it is estimated that 95 percent of the post-construction foundation soil settlements will be completed in less than about one (1) month. Given this relatively short time frame, preloading the foundation soils by building the approach embankments as early as possible should be considered. For this alternative, sub-excavation into the artesian groundwater conditions would be avoided. In addition, if installation of the abutment piles is also delayed until completion of the foundation soil settlements, the abutment piles would not have to be designed to accommodate the downdrag loads and lateral displacements described in Section 5.3. This mitigation option is considered to be the best technical solution to the long-term performance of the roadway from a foundations perspective.

5.6.4.2 Full Sub-excavation

The bottom of the clayey silt layer within the area of the approaches is generally less than about 4.0 m below the existing ground surface. Sub-excavation of the clayey silt strata to this depth (and replacement with competent fill) would minimize post-construction settlements of the approach fills and eliminate the downdrag and lateral loads on the abutment piles. However, the artesian groundwater conditions encountered below the clayey silt strata at the site are expected to make sub-excavation and replacement difficult and as such, this alternative is not recommended for mitigating the post-construction settlements.

5.6.4.3 Surcharging

Although surcharging to accelerate the rate of time-dependent settlements is an option, it is not considered to be the most practical solution for this site. This is due to the fact that the approach embankments are relatively high (about 10 m) so the addition of a 1 m or 2 m high surcharge fill does not represent a very large percentage of the overall embankment height and so will not have a large effect on inducing additional settlements in a shorter time period. In addition, considering that the rate of settlement is already relatively fast for this site (i.e. 95% consolidation anticipated to be completed in less than about 1 months time), the extra effort required to place and then remove surcharge fills would not warrant the relatively small savings that would be gained in the time to complete the consolidation settlements. For these reasons, surcharging is not recommended for mitigating the post-construction settlements at this site.

5.6.4.4 Wick Drains

Given the relatively thin nature of the clayey silt strata and considering that a large portion of this strata is currently and is expected to remain over-consolidated even after the construction of the approach embankments, the use of wick drains will not provide much benefit for accelerating the consolidation settlements at this site. As such, the additional costs associated with the installation and monitoring of a wick drain foundation can not be justified for this site.

5.6.4.5 Light Weight (EPS) Fill

Although the use of light weight fill could be considered to reduce the magnitude of the post-construction settlements, the high costs associated with this type of fill material and the large volume of EPS that would be required to construct the 10 m high embankments do not justify its use at this site, especially considering the relatively modest magnitudes of settlement expected and the relatively short period of time required to complete the settlements.

5.6.5 Subgrade Preparation and Embankment Construction

The existing native subsoils are considered to be appropriate subbase for the proposed approach embankments; however, prior to the placement of any fill, all surfaces and near surface layers of topsoil, surficial fills and organics layers and any softened or loosened soils should be stripped from the plan limits of the proposed works and the remaining subgrade soils should be proof-rolled, where possible.

Table 3 summarizes the recommended fill type to be placed for the approach embankment construction, the location and depth of organics, the recommended side slope profiles, the requirements for side berms, the anticipated differential settlements, platform widenings (in accordance with NRE 98-200) and the recommended method of removal of organics. The following sections provide details on the recommendations for subgrade preparation and embankment construction.

5.6.6 Removal of Topsoil, Surficial Fills and Organic Layers

Based on the information from the borings conducted during the field investigation, topsoil, surficial fills and organic layers of up to about 0.9 m deep (below existing ground surface) but typically less than about 0.4 m deep, can be expected in the areas of the approach embankments. Additional boreholes should be advanced within the footprints of the south approaches to confirm the depth of organics in these areas.

5.6.7 Embankment Fill Placement

If earth fill (granular) is to be used for construction of the approach embankments, placement of all granular fill material should be carried out in accordance with Special Provision SP 206S03, in regular lifts with loose thickness not exceeding 300 mm, and be compacted to at least 95 percent of the Standard Proctor maximum dry density. The final lift prior to placement of the granular subbase course should be placed and compacted to current MTO requirements for pavements. Inspection and field density testing should be carried out by qualified geotechnical personnel during all earth fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved. Side slopes for earth fill embankments should be no steeper than 2H:1V.

If rock fill is used for the construction of the new embankments, placement of all rock fill material should be carried out in accordance with the requirements as outlined in the Special Provision SP 206S03. The rock should not be dumped in final position, but should be deposited on and pushed forward over the end of the layer being constructed. Voids and bridging shall be minimized by blading, dozing and ‘chinking’ the rock to form a dense compact mass. Side slopes for rock fill embankments should be no steeper than 1.25H:1V.

It should be noted that if rock fill is to be used for construction of the new embankments and if the abutment piles are to be driven after fill placement, it will be necessary to construct the portion of the approach embankments in the abutment area with a granular fill with less than 75 mm particle sizes (not rock fill) so that pile driving can be carried out without encountering obstructions.

Vegetation cover should be established on all soil slopes to protect the embankment fill against surficial erosion.

5.7 Design and Construction Considerations

5.7.1 Excavations

Considering that pile foundations are being proposed for support of the structures and preloading is recommended as the preferred settlement mitigation option, only limited excavation should be required at the site as part of the stripping operations and removal of existing fills. At this site, excavations up to about 1 m deep could be carried out by open cut with side slopes of about 1H:1V. Conventional excavation equipment should be suitable for the excavation of the on-site soils. Groundwater and surface water inflows should be controlled as discussed in Section 5.7.2.

All excavations must be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety Act and Regulations for Construction Projects.

5.7.2 Groundwater and Surface Water Control

As noted in Section 4.2.8, the groundwater level at the site varies from as deep as about 3 m below ground surface to as high as about 2 m above ground surface depending on the location and the soil unit being considered. The artesian conditions are generally confined to the more permeable cohesionless strata (i.e. silts and sands) present below the clayey still strata. For the depth of excavation required to remove the surficial fills and topsoil/organics layers present on site (i.e. less than 1 m), groundwater inflows (and the potential for base heave) should be negligible. However, if deeper excavations are required, the potential for base heave and large groundwater inflows exists and should be carefully reviewed by a geotechnical engineer and/or a specialist dewatering contractor. In all cases, surface water should be directed away from the excavations at all times.

Given the presence of artesian groundwater conditions at some locations, a filter blanket should be installed around the CSPs/piles near the ground surface to prevent loss of fines due to potential migration of water/seepage along the piles. In addition, a subdrain should be installed adjacent to the CSPs/piles at each abutment to promote drainage of artesian waters away from the abutment areas. A schematic drawing showing the details of this recommendation is presented on Figure 2.

5.8 Closure

This Foundation Design Report was prepared by Ms. Karyn Gallant and reviewed by Mr. J. Paul Dittrich, Ph.D., P. Eng., an Associate with Golder. Mr. Jorge M. Costa, P. Eng., Golder's Designated MTO Contact for this project, conducted an independent quality review of the report.

GOLDER ASSOCIATES LTD.

Paul Dittrich

for Karyn Gallant,
Geotechnical Group

Paul Dittrich

J. Paul Dittrich, Ph.D., P.Eng.,
Associate, Senior Geotechnical Engineer

Licensed Professional Engineer
J. P. DITTRICH
Province of Ontario

Jorge M. A. Costa, P.Eng.,
Principal, Designated MTO Contact

Licensed Professional Engineer
J. M. A. COSTA
Province of Ontario

KG/JPD/JMAC/kg/jpd

n:\active\2006\1111\06-1111-001 urs-hwy69 wildlife crossings\reporting\final\06-1111-001 wildlife crossing under hwy 69\06-1111-001-1 rpt 06july final wildlife xing under hwy 69 report.doc

REFERENCES

Azzouz, A.S., R.J. Krizek, and R.B. Corotis. 1976. Regression Analysis of Soil Compressibility. Soils and Foundations, Tokyo, Vol. 16, no.2, pp.19-29.

Bjerrum, L. 1973. Problems of Soil Mechanics and Construction of Soft Clays and Structurally Unstable Soils. State-of-the-art Report, Session 4. Proceedings, 8th International Conference on Soil Mechanics and Foundation Engineering, Moscow, Vol. 3, pp. 111-159.

Bowles, J.E. 1984. Physical and Geotechnical Properties of Soils, 2nd Edition. McGraw-Hill Book Company, New York.

Britto, A.M. and Gunn, M.J. 1987. Critical State Soil Mechanics via Finite Elements. Ellis Horwood Ltd., Chichester, England.

Canadian Foundation Engineering Manual. 1992. Third Edition. Canadian Geotechnical Society, Technical Committee on Foundations.

Canadian Highway Bridge Design Code (CHBDC) and Commentary on CAN/CSA-S6-00. 2001. CSA Special Publication, S6.1-00. Canadian Standards Association.

Koppula, S.D. 1986. Consolidation Parameters Derived from Index Tests. Geotechnique, V.36, No. 2, pp 291-292.

Kulhawy, F.H. and Mayne, P.W. 1990. Manual on Estimating Soil Properties for Foundation Design. EL-6800, Research Project 1493-6. Prepared for Electric Power Research Institute, Palo Alto, California.

Ladd, C.C. 1991. Stability Evaluation During Staged Construction. ASCE Journal of Geotechnical Engineering, Vol. 117, No. 4, pp. 540-615.

Lo, K.Y. 1982. Rockfill in the Foundation Design of Highway Structures. Research and Development Branch - Ministry of Transportation Ontario.

Geology of Ontario. 1991. Ontario Geological Society, Special Volume 4, Part 1. Eds. P.C. Thurston, H.R. Williams, R.H. Sutcliffe and G.M. Stott. Ministry of Northern Development and Mines, Ontario.

Makdisis, F.I., and Seed, H.B. 1978. Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations. ASCE Journal of the Geotechnical Engineering Division, V. 104, GT7, pp 849-867.

Mesri, G. 1973. One-dimensional Consolidation of a Clay Layer with Impeded Drainage Boundaries. American Geophysical Union, Water Resources Research, , 9, No. 4, pp. 1090-1093.

Mesri, G. 1975. Discussion on New Design Procedure for Stability of Soft Clays. ASCE Journal of the Geotechnical Engineering Division, V. 101, GT4, pp. 409-412.

Peck, R.B., Hanson, W.E., and Thornburn, T.H. 1974. Foundation Engineering, 2nd Edition, John Wiley and Sons, New York.

Schmertmann, J.H. 1975. Measurement of In-Situ Shear Strength. In Proceedings, ASCE Specialty Conference on In-Situ Measurement of Soil Properties, Vol. 2, Raleigh, pp. 57-138.

Terzaghi, K. and Peck, R.B. 1967. Soil Mechanics in Engineering Practice, 2nd Edition, John Wiley and Sons, New York, pp.72.

U.S. Navy. 1982. Soil Mechanics, Foundations and Earth Structures. NAVFAC Design Manual DM-7, Washington, D.C.

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

IV. SOIL TESTS

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

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

LIST OF SYMBOLS

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

I. General

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\ \sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
v	poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma$-u)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_3	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1+\sigma_2+\sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

Notes:

1 $\tau = c' + \sigma' \tan \phi'$

2 shear strength = (compressive strength)/2

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

S:\FINALDAT\SYMBOLS\2000\SYMB-D00.DOC

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of weathering.

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

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

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

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

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

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

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

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

CORE CONDITION

Total Core Recovery

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

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

DISCONTINUITY DATA

Fracture Index

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

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

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

Description and Notes

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

Abbreviations

B - Bedding
FO - Foliation/Schistosity
CL - Cleavage
SH - Shear Plane/Zone
VN - Vein
F - Fault
CO - Contact
J - Joint
FR - Fracture
MF - Mechanical Fracture
‖ - Parallel To
⊥ - Perpendicular To

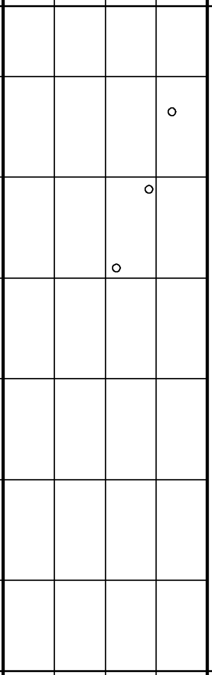
P - Polished
S - Slickensided
SM - Smooth
R - Ridged/Rough
ST - Stepped
PL - Planar
FL - Flexured
UE - Uneven
W - Wavy
C - Curved

RECORD OF BOREHOLE No 06-01

1 OF 1 **METRIC**

PROJECT 06-1111-001
W.P. 5379-02-00
DIST HWY 69
DATUM Geodetic
LOCATION Sta. 18+414 o/s 13.5 m Rt CL Med.
BOREHOLE TYPE CME-55 Bombardier
DATE April 27, 2006
ORIGINATED BY EHS
COMPILED BY MM
CHECKED BY KG

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L / WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	▽					
213.7	GROUND SURFACE									
0.0	Silty organics (TOPSOIL) Dark brown									
213.4 0.3	CLAYEY SILT, trace sand and organics containing clay seams Very stiff to firm Varved to layered light brown and grey with oxidized layers Wet	1	SS	6		213				
		2	SS	5		212				
211.6 2.1	SILT, some sand, trace to some clay, containing sand seams and layers Loose Grey Wet	3	SS	8		211				
210.6	SAND and SILT, some gravel and cobbles (TILL) Compact Grey Wet	4	SS	15/0.15						
210.3 3.4	BEDROCK Refer to Record of Drillhole log 06-01 for coring details.					210 209 208				
207.1 6.6	END OF BOREHOLE Note: 1. Artesian ground water conditions. Hydrostatic water level at 0.8 m above ground surface upon completion of drilling.									



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

PROJECT: 06-1111-001

RECORD OF DRILLHOLE: 06-01

SHEET 2 OF 2

LOCATION: Sta. 18+414 o/s 13.5 m Rt CL Med.

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: April 27, 2006

DRILL RIG: CME-55 Bombardier

DRILLING CONTRACTOR: Marathon Drilling Ltd.

DATUM: Geodetic

JN - Joint
FLT - Fault
SHR- Shear
VN - Vein
CJ - Conjugate

BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Cleavage

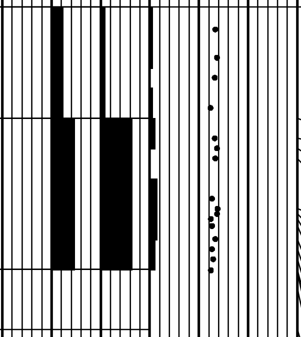
PL - Planar
CU- Curved
UN- Undulating
ST - Stepped
IR - Irregular

PO- Polished
K - Slickensided
SM- Smooth
Ro - Rough
MB- Mechanical Break

BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE % (80 60 40 20)	RECOVERY SOLID CORE % (80 60 40 20)	R.Q.D. % (80 60 40 20)	FRACT. INDEX PER 0.3 m (5 10 15 20)	DISCONTINUITY DATA B Angle (0 90 180 270)	DISCONTINUITY DATA DIP w.r.t. CORE AXIS (0 30 60 90)	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3})	Diametral Point Load Index (MPa) (2 4 6)	RMC -Q' AVG.	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Borehole 06-01		210.30														
4 5 6	CME 55 NQ Apr. 27/06	Slightly weathered (W2), medium strong (R3), medium grained, pink, black and grey GRANITE GNEISS with healed and partially healed joints (crystalline open flow joint at 5.5 m). Some joints healed with Hematite and Quartz Carbonate. Healed vertical brecciated joint from 6.0 to 6.6 m.		3.40	1		reddish/grey 100							JN,FO,PL,Ro ,JN,PL,SM JN,FO,,SM ,JN,UN,Ro				
					2		reddish/grey 100							MB,,, JN,FO,,SM ,JN,PL,SM ,JN,ST,Ro BD,JN,PL,Ro ,JN,PL,Ro ,JN,UN,Ro ,JN,PL,Ro ,JN,PL,SM				
				207.10 6.60	3		reddish/grey 100							JN,FO,,SM ,JN,UN,SM ,JN,UN,SM ,JN,ST,Ro MB,,, MB,,, MB,,,				
7 8 9 10 11 12 13 14 15 16 17 18		END OF DRILLHOLE																



DEPTH SCALE

1 : 75

Golder Associates

LOGGED: EHS

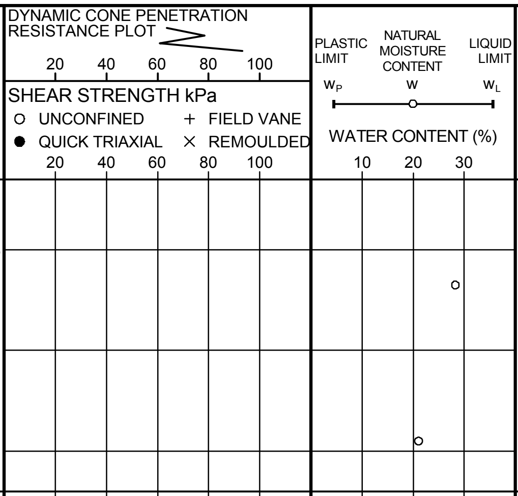
CHECKED: KG

RECORD OF BOREHOLE No 06-02

1 OF 1 **METRIC**

PROJECT 06-1111-001
W.P. 5379-02-00
DIST
HWY 69
DATUM Geodetic
LOCATION Sta. 18+419 o/s 13.0 m Rt CL Med.
BOREHOLE TYPE CME-55 Bombardier
DATE April 27, 2006
ORIGINATED BY EHS
COMPILED BY MM
CHECKED BY KG

SOIL PROFILE			SAMPLES				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE
213.7	GROUND SURFACE						
0.0	Silty organics (TOPSOIL)						
0.2	Dark brown CLAYEY SILT, trace sand, organics and cobbles containing clay seams to 1.4 m depth Very stiff to firm Varved to layered grey and light brown Wet						213
			1	SS	7	▽	
			2	SS	6		212
211.6 2.1	SILT, some sand, containing sand seams and layers Compact Grey Wet						
			3	SS	10		211
210.6 3.1	END OF BOREHOLE Note: 1. Auger refusal at 3.1 m depth. 2. Water level in open borehole measured at 1.2 m depth (Elev. 212.5 m) upon completion of drilling operations.						



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 06-1111-001	**RECORD OF BOREHOLE No 06-03**	1 OF 1	**METRIC**
W.P. 5379-02-00	LOCATION Sta. 18+436 o/s 18.9 m Rt CL Med.		ORIGINATED BY EHS
DIST HWY 69	BOREHOLE TYPE CME-55 Bombardier		COMPILED BY MM
DATUM Geodetic	DATE April 27, 2006		CHECKED BY KG

SOIL PROFILE			SAMPLES					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 (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				GR	SA	SI	CL
214.4	GROUND SURFACE													
0.0 214.1	Silty organics (TOPSOIL) Dark brown		1	SS	1		214		○					
213.8 0.6	CLAYEY SILT, some sand, trace gravel and organics Very soft Varved to layered grey and light brown Wet		2	SS	15	∇								
213.0 1.4	SANDY SILT, trace clay Compact Light brown with oxidized layers Wet		3	SS	8		213				0	61	37	2
212.2 2.2	SAND and SILT, trace clay, containing silt seams Loose Light brown with oxidized layers Wet													
	END OF BOREHOLE													

Notes:

1. Auger refusal at 2.2 m depth.

2. Water level in open borehole measured at 0.8 m depth (Elev. 213.6 m) upon completion of drilling operations.

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

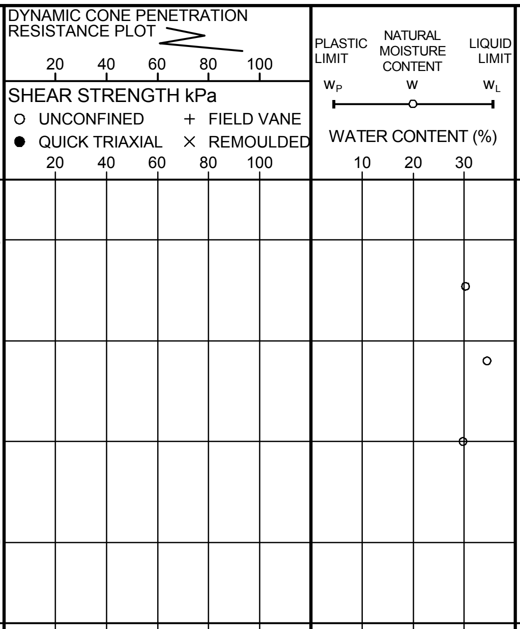
Golder Associates

RECORD OF BOREHOLE No 06-04

1 OF 1 **METRIC**

PROJECT 06-1111-001
W.P. 5379-02-00
LOCATION Sta. 18+414 o/s 25.5 m Rt CL Med.
ORIGINATED BY EHS
DIST HWY 69
BOREHOLE TYPE CME-55 Bombardier
COMPILED BY MM
DATUM Geodetic
DATE April 28, 2006
CHECKED BY KG

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
213.6	GROUND SURFACE											
0.0	Silty organics (TOPSOIL) Dark brown											
213.2 0.4	CLAYEY SILT, trace sand, containing clay seams Very stiff to firm Varved to layered grey and reddish brown with oxidized layers to 2.1 m depth Moist to wet		1 2 3 4	SS SS SS SS	6 4 4 5	∇	213 212 211 210					
209.9 3.7	SAND and SILT, trace gravel and cobbles (TILL) Compact Grey Wet		5	SS	23							
209.2 4.4	END OF BOREHOLE											



REMARKS & GRAIN SIZE DISTRIBUTION (%)

Notes:

1. Auger refusal at 4.4 m depth.

2. Water level in open borehole measured at 2.3 m depth (Elev. 211.3 m) upon completion of drilling operations.

3. Refer to borehole 06-25 for vane shear strength.

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

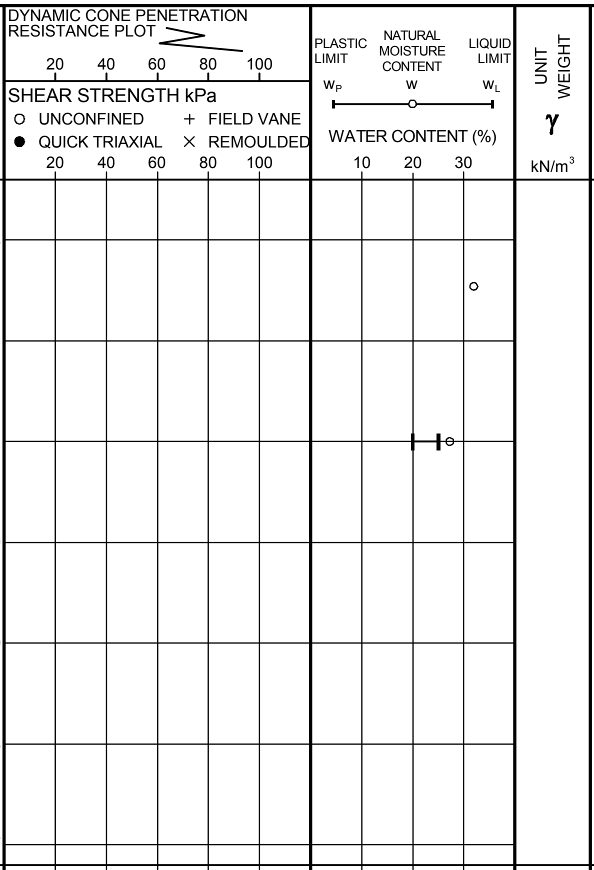
Golder Associates

RECORD OF BOREHOLE No 06-05

1 OF 1 **METRIC**

PROJECT 06-1111-001
W.P. 5379-02-00
DIST HWY 69
DATUM Geodetic
LOCATION Sta. 18+418 o/s 25.7 m Rt CL Med.
BOREHOLE TYPE CME-55 Bombardier
DATE April 28, 2006
ORIGINATED BY EHS
COMPILED BY MM
CHECKED BY KG

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES			GR	SA	SI	CL
213.6	GROUND SURFACE									
0.0	Silty Organics (TOPSOIL)									
0.2	Dark brown CLAYEY SILT, trace sand and organics to 1.4 m depth, containing clay seams Very stiff to firm Varved to layered grey and reddish brown with oxidized layers to 1.4 m depth Moist to wet					213				
		1	SS	8						
		2	SS	5		212				
		3	SS	7		211	0	0	85	15
210.3 3.3	SAND and SILT, some gravel and cobbles (TILL) Compact Grey Wet	4	SS	25		210				
209.8 3.8	BEDROCK Refer to Record of Drillhole log 06-05 for coring details.					209 208 207				
206.8 6.8	END OF BOREHOLE Note: 1. Artesian ground water conditions. Hydrostatic water level at 0.9 m above ground surface (Elev. 214.5m) upon completion of drilling.									



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

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

PROJECT: 06-1111-001

RECORD OF DRILLHOLE: 06-05

SHEET 2 OF 2

LOCATION: Sta. 18+418 o/s 25.7 m Rt CL Med.

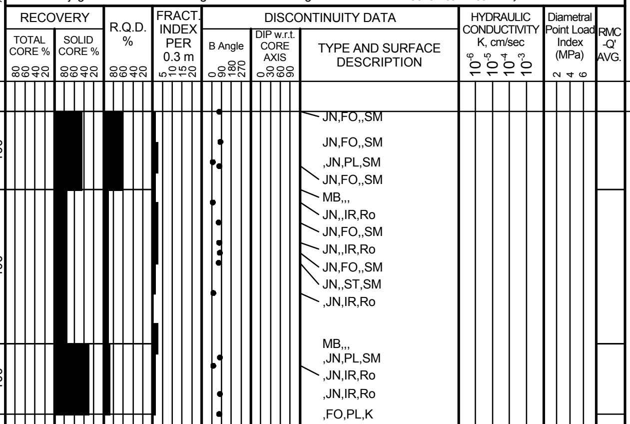
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: April 28, 2006
DRILL RIG: CME-55 Bombardier
DRILLING CONTRACTOR: Marathon Drilling Ltd.

DATUM: Geodetic

JN - Joint	BD - Bedding	PL - Planar	PO - Polished	BR - Broken Rock
FLT - Fault	FO - Foliation	CU - Curved	K - Slickensided	**NOTE:** For additional abbreviations refer to list of abbreviations & symbols.
SHR - Shear	CO - Contact	UN - Undulating	SM - Smooth	
VN - Vein	OR - Orthogonal	ST - Stepped	Ro - Rough	
CJ - Conjugate	CL - Cleavage	IR - Irregular	MB - Mechanical Break	

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Borehole 06-05		209.80					
4	CME-55 NQ Apr. 28/06	Slightly weathered (W2), medium strong (R3), medium grained, pink, black and grey GRANITE GNEISS with healed and partially healed joints, clay and sand infill at 4.27 m. Hematite and Quartz Carbonate in healed and partially healed joints.		3.80	1		reddish/grey 100	JN,FO,,SM JN,FO,,SM ,JN,PL,SM JN,FO,,SM	
5					2		reddish/grey 100	MB,,, JN,,IR,Ro JN,FO,,SM JN,,IR,Ro JN,FO,,SM JN,,ST,SM ,JN,IR,Ro	
6				206.80 6.80	3		reddish/grey 100	MB,,, ,JN,PL,SM ,JN,IR,Ro ,JN,IR,Ro ,FO,PL,K	
7		END OF DRILLHOLE							
8									
9									
10									
11									
12									
13									
14									
15									
16									
17									
18									



MIS-RCK 004 06-1111-001.GPJ GAL-MISS.GDT 7/7/06 MSM

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: EHS

CHECKED: KG

PROJECT 06-1111-001

RECORD OF BOREHOLE No 06-06

1 OF 1 **METRIC**

W.P. 5379-02-00

LOCATION Sta. 18+415 o/s 18.5 m Rt CL Med.

ORIGINATED BY EHS

DIST HWY 69

BOREHOLE TYPE CME-55 Bombardier

COMPILED BY MM

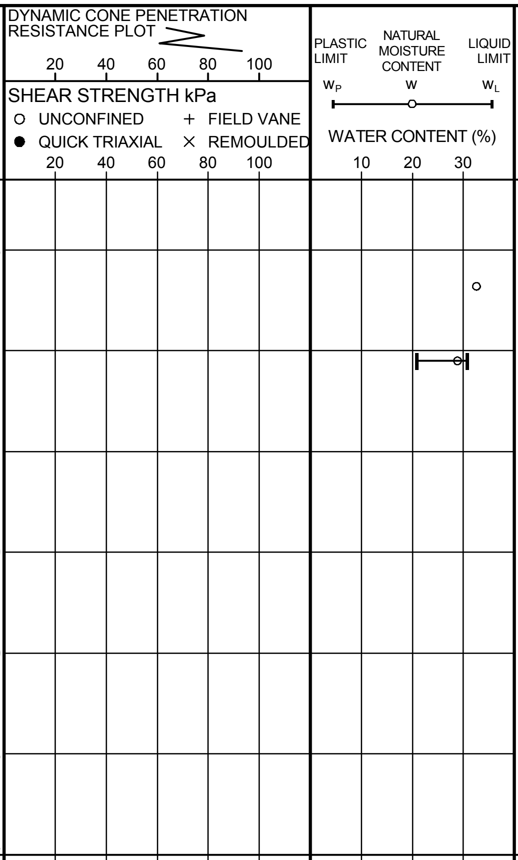
DATUM Geodetic

DATE April 29, 2006

CHECKED BY KG

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
213.7	GROUND SURFACE				
0.0	Silty organics (TOPSOIL)				
0.2	Dark brown CLAYEY SILT, trace sand and organics to 1.4 m depth, containing clay seams Very stiff to firm Varved to layered grey and brown with oxidized layers to 2.1 m depth Moist to wet				213
		1	SS	6	
		2	SS	4	212
211.6 2.1	SILT, trace sand and clay Loose Grey Wet	3	SS	7	211
210.7 3.0	SAND and SILT, some gravel, (TILL) Compact Grey	4	SS	19	
210.0 3.7	Wet BEDROCK Refer to Record of Drillhole log 06-01 for coring details.				210 209 208
207.0 6.7	END OF BOREHOLE Notes: 1. Water flowing above ground surface upon completion of drilling operations (artesian).				207

STRAT PLOT · GROUND WATER CONDITIONS ∇



UNIT WEIGHT γ kN/m^3

REMARKS & GRAIN SIZE DISTRIBUTION (%)

GR SA SI CL

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

PROJECT: 06-1111-001

RECORD OF DRILLHOLE: 06-06

SHEET 2 OF 2

LOCATION: Sta. 18+415 o/s 18.5 m Rt CL Med.

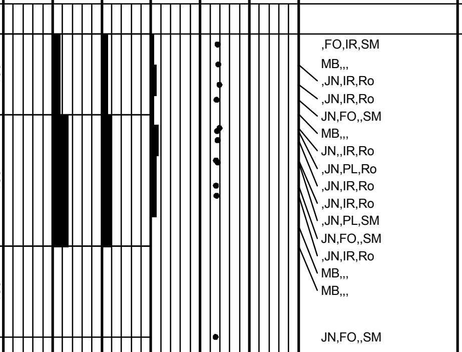
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: April 29, 2006
DRILL RIG: CME-55 Bombardier
DRILLING CONTRACTOR: Marathon Drilling Ltd.

DATUM: Geodetic

JN - Joint
FLT - Fault
SHR- Shear
VN - Vein
CJ - Conjugate
BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Cleavage
PL - Planar
CU- Curved
UN - Undulating
ST - Stepped
IR - Irregular
PO- Polished
K - Slickensided
SM- Smooth
Ro - Rough
MB- Mechanical Break
BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.



DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	COLOUR / % RETURN / FLUSH	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3})	Diametral Point Load Index (MPa) (2 4 6)	RMC -Q' AVG.	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Borehole 06-06		210.00								
4	CME-55 NQ Apr. 29/06	Slightly weathered (W2), medium strong (R3), medium grained, pink, black and grey GRANITE GNEISS with healed and partially healed joints. Joints healed with Hematite and Quartz Carbonate, some infilled with clay and silt, less than 0.5 mm thick		3.70	1		reddish/grey 100	,FO,IR,SM MB,,, ,JN,IR,Ro ,JN,IR,Ro JN,FO,,SM				
5					2		reddish/grey 100	MB,,, JN,,IR,Ro ,JN,PL,Ro ,JN,IR,Ro ,JN,IR,Ro ,JN,PL,SM JN,FO,,SM				
6				207.00	3		reddish/grey 100	,JN,IR,Ro MB,,, MB,,,				
7		END OF DRILLHOLE		6.70				JN,FO,,SM				
8												
9												
10												
11												
12												
13												
14												
15												
16												
17												
18												

DEPTH SCALE
1 : 75

Golder Associates

LOGGED: EHS
CHECKED: KG

RECORD OF BOREHOLE No 06-07

1 OF 1 **METRIC**

PROJECT 06-1111-001

W.P. 5379-02-00

DIST ___ HWY 69

DATUM Geodetic

LOCATION Sta. 18+418 o/s 13.2 m Lt CL Med.

BOREHOLE TYPE CME-55 Bombardier

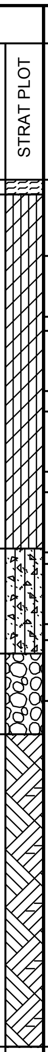
DATE April 29, 2006

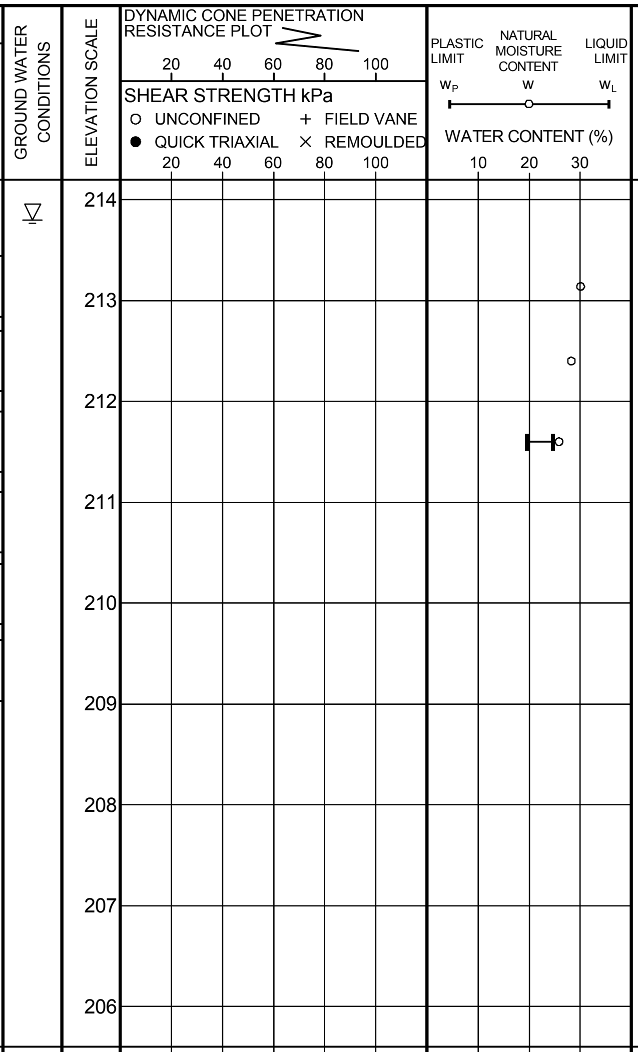
ORIGINATED BY EHS

COMPILED BY MM

CHECKED BY KG

SOIL PROFILE		SAMPLES			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
214.2	GROUND SURFACE							
0.0	Silty organics (TOPSOIL) Dark brown							
0.2	CLAYEY SILT, trace sand and gravel, containing clay seams and sand seams Very stiff to firm Layered light brown to reddish brown with oxidized layers to 2.9 m depth Moist to wet	1	SS	9				
		2	SS	5				
	Becoming grey below 2.3 m depth	3	SS	9				
		4	SS	6				
210.5 3.7	SAND and SILT, trace to some gravel, trace clay (TILL) Loose Grey	5	SS	7	11	31	55	3
209.5 4.7	COBBLES and BOULDERS	6	SS	8/0.13				
208.7 5.5	BEDROCK Refer to Record of Drillhole log 06-07 for coring details.							
205.6 8.6	END OF BOREHOLE							





Note:

1. Water level in open borehole measured at 0.4 m depth (Elev. 213.8 m) upon completion of drilling operations.

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

PROJECT: 06-1111-001

RECORD OF DRILLHOLE: 06-07

SHEET 2 OF 2

LOCATION: Sta. 18+418 o/s 13.2 m Lt CL Med.

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: April 29, 2006

DRILL RIG: CME-55 Bombardier

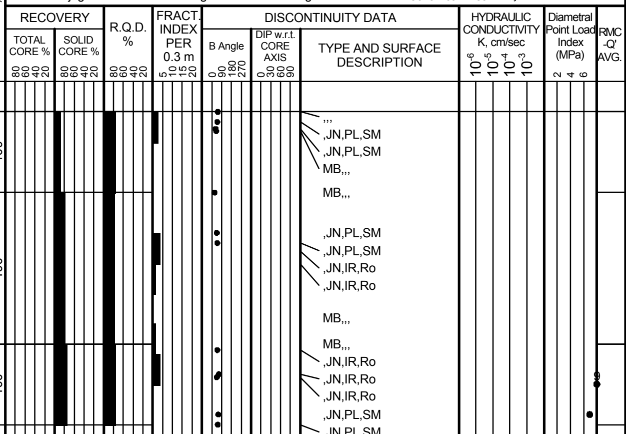
DRILLING CONTRACTOR: Marathon Drilling Ltd.

DATUM: Geodetic

JN - Joint
FLT - Fault
SHR - Shear
VN - Vein
CJ - Conjugate
BD - Bedding
FO - Foliation
CO - Contact
OR - Orthogonal
CL - Cleavage
PL - Planar
CU - Curved
UN - Undulating
ST - Stepped
IR - Irregular
PO - Polished
K - Slickensided
SM - Smooth
Ro - Rough
MB - Mechanical Break
BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	COLOUR % RETURN FLUSH	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Borehole 06-07		208.70					
6	CME-55 NQ Apr. 29/06	Slightly weathered (W2), medium strong (R3), medium grained, pink, black and grey GRANITE GNEISS with healed and partially healed joints. Joints healed with Hematite and Quartz Carbonate, some infilled with clay/silt.		5.50	1		reddish/grey 100	,,, ,JN,PL,SM ,JN,PL,SM MB,,, MB,,,	
7					2		reddish/grey 100	,JN,PL,SM ,JN,PL,SM ,JN,IR,Ro ,JN,IR,Ro MB,,,	
8				205.60	3		reddish/grey 100	MB,,, ,JN,IR,Ro ,JN,IR,Ro ,JN,IR,Ro ,JN,PL,SM ,JN,PL,SM	
9		END OF DRILLHOLE		8.60					
10									
11									
12									
13									
14									
15									
16									
17									
18									
19									
20									



DEPTH SCALE

1 : 75

Golder Associates

LOGGED: EHS

CHECKED: KG

RECORD OF BOREHOLE No 06-08

1 OF 1 **METRIC**

PROJECT 06-1111-001
W.P. 5379-02-00
LOCATION Sta. 18+414 o/s 26.1 m Lt CL Med.
ORIGINATED BY EHS
DIST ___ HWY 69
BOREHOLE TYPE CME-55 Bombardier
COMPILED BY MM
DATUM Geodetic
DATE April 29, 2006
CHECKED BY KG

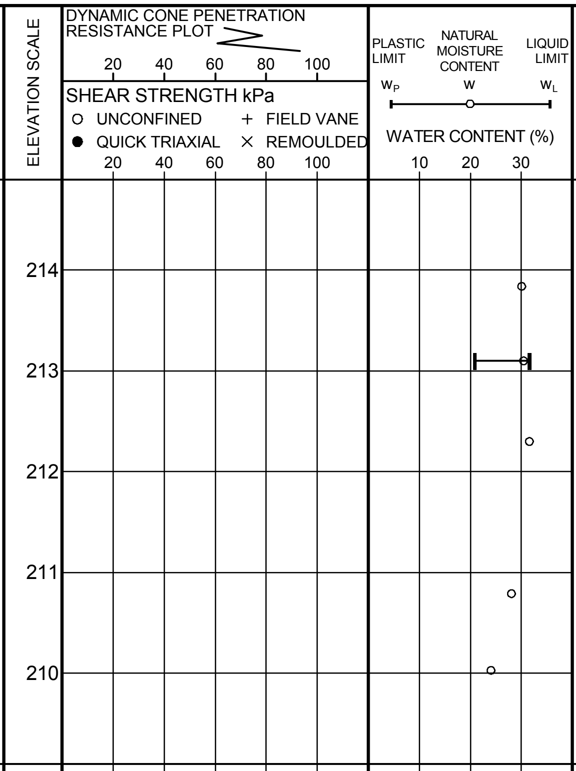
SOIL PROFILE

ELEV DEPTH	DESCRIPTION
214.9	GROUND SURFACE
0.0	Silty organics (TOPSOIL) Dark brown
214.6 0.3	SILTY SAND Light brown Moist CLAYEY SILT, trace sand, gravel and organics Very stiff to firm Layered light brown and reddish brown, with oxidized layers to 2.3 m depth Moist to wet
211.2 3.7	SILT, trace sand and clay, containing clay seams and sand seams with depth Very loose to compact Grey Wet
209.1 5.8	END OF BOREHOLE

SAMPLES

NUMBER	TYPE	"N" VALUES
1	SS	9
2	SS	6
3	SS	5
4	SS	10
5	SS	3
6	SS	10

GROUND WATER CONDITIONS: ∇ (at approx. Elev. 213.5)



UNIT WEIGHT γ kN/m³

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

Note:

1. Auger refusal at 5.8 m depth (augers sliding towards the east).

2. Water level in open borehole measured at 1.4 m depth (Elev. 213.5 m) upon completion of drilling operations.

3. Refer to borehole 06-26 for vane shear strength.

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

RECORD OF BOREHOLE No 06-09

1 OF 1 **METRIC**

PROJECT 06-1111-001

W.P. 5379-02-00

LOCATION Sta. 18+436 o/s 18.6 m Lt CL Med.

ORIGINATED BY EHS

DIST

HWY 69

BOREHOLE TYPE CME-55 Bombardier

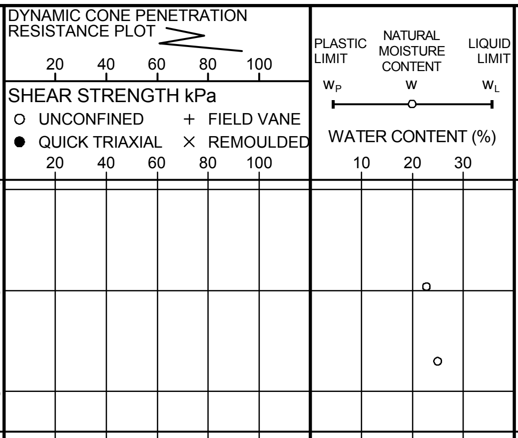
COMPILED BY MM

DATUM Geodetic

DATE April 30, 2006

CHECKED BY KG

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
217.1	GROUND SURFACE					217		
0.0	Silty organics (TOPSOIL) Dark brown							
216.3 0.2	SILTY SAND, trace gravel, cobbles and organics Moist							
0.8	CLAYEY SILT, trace sand, gravel and organics, containing clay seams Very stiff to stiff Layered light brown and grey with oxidized layers Moist to wet	1	SS	19		216		
		2	SS	8	⊽	215		
214.8 2.5	SAND and SILT, some gravel, containing sand layers Brown Wet	3	SS	3/0.05				
	END OF BOREHOLE							



Note:

1. Auger refusal at 2.5 m depth.

2. Water level in open borehole measured at 2.2 m depth (Elev. 214.9 m) upon completion of drilling operations.

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

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

RECORD OF BOREHOLE No 06-10

1 OF 2 **METRIC**

PROJECT 06-1111-001
W.P. 5379-02-00
LOCATION Sta. 18+365 o/s 26.5 m Rt CL Med.
ORIGINATED BY EHS
DIST HWY 69
BOREHOLE TYPE CME-55 Bombardier
COMPILED BY MM
DATUM Geodetic
DATE April 30, 2006
CHECKED BY KG

ELEV DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	TYPE	"N" VALUES
213.4	GROUND SURFACE			
0.0	Silty organics Dark brown (TOPSOIL)			
213.0 0.5	CLAYEY SILT, trace sand and organics to 2.1 m depth, containing clay seams Very soft to stiff Grey with oxidized layers to 1.4 m depth Wet			
		1	SS	2
		2	SS	4
		3	SS	0
209.7 3.7	SILT, trace sand and clay, containing occasional sand seams and clay seams Loose Grey Wet			
		4	SS	5
		5	SS	4
205.8 7.6	SANDY SILT, trace gravel and clay, containing silty sand seams Very loose to loose Grey Wet			
		6	SS	WR
		7	SS	5
		8	SS	3
		9	SS	7
200.3 13.1	BEDROCK Refer to Record of Drillhole log 06-10 for coring details.			

STRAT PLOT | GROUND WATER CONDITIONS | ELEVATION SCALE

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

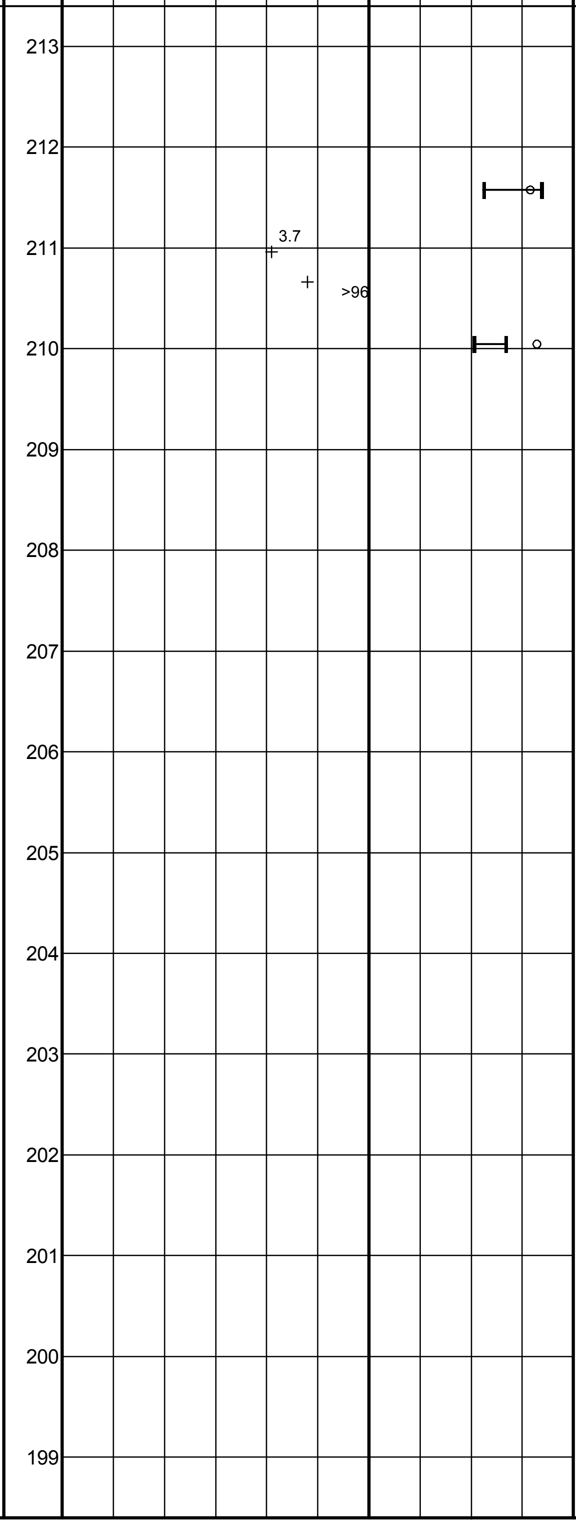
SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × REMOULDED
20 40 60 80 100

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



Continued Next Page

+³, ×³: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No 06-10

2 OF 2 **METRIC**

PROJECT 06-1111-001
W.P. 5379-02-00
LOCATION Sta. 18+365 o/s 26.5 m Rt CL Med.
ORIGINATED BY EHS
DIST HWY 69
BOREHOLE TYPE CME-55 Bombardier
COMPILED BY MM
DATUM Geodetic
DATE April 30, 2006
CHECKED BY KG

SOIL PROFILE			SAMPLES								
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	BEDROCK Refer to Record of Drillhole log 06-10 for coring details.	▨					198 197				
196.8 16.6	END OF BOREHOLE Note: 1. Artesian ground water conditions. Hydrostatic water level at 2.1 m above ground surface upon completion of drilling.										

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

PROJECT: 06-1111-001

RECORD OF DRILLHOLE: 06-10

SHEET 3 OF 3

LOCATION: Sta. 18+365 o/s 26.5 m Rt CL Med.

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: April 30, 2006
DRILL RIG: CME-55 Bombardier
DRILLING CONTRACTOR: Marathon Drilling Ltd.

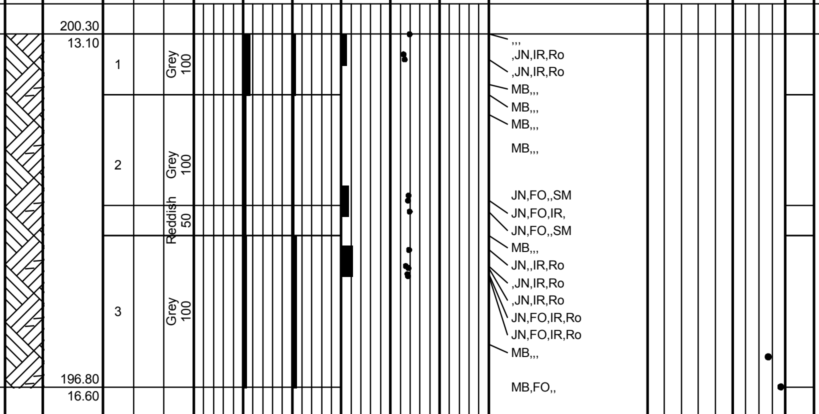
DATUM: Geodetic

JN - Joint
FLT - Fault
SHR - Shear
VN - Vein
CJ - Conjugate
BD - Bedding
FO - Foliation
CO - Contact
OR - Orthogonal
CL - Cleavage
PL - Planar
CU - Curved
UN - Undulating
ST - Stepped
IR - Irregular
PO - Polished
K - Slickensided
SM - Smooth
Ro - Rough
MB - Mechanical Break
BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN (FLUSH)	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Borehole 06-10				
	CME-55 NQ Apr. 30/06	Slightly weathered (W2), strong (R4), medium grained, pink, black and grey GRANITE GNEISS with healed joints and opened joints. Joint at 14.6 m filled with silt and joint at 15.4 m filled with silty clay.	200.30 13.10	1	Grey 100	
14				2	Grey 100	
15					Reddish 50	
16			196.80 16.60	3	Grey 100	
		END OF DRILLHOLE				

Depth scale: 17, 18, 19, 20, 21, 22, 23, 24, 25, 26, 27, 28



TYPE AND SURFACE DESCRIPTION

,,,
,JN,IR,Ro
,JN,IR,Ro
MB,,,
MB,,,
MB,,,
MB,,,
JN,FO,,SM
JN,FO,IR,
JN,FO,,SM
MB,,,
JN,,IR,Ro
,JN,IR,Ro
,JN,IR,Ro
JN,FO,IR,Ro
JN,FO,IR,Ro
MB,,,
MB,FO,,

DEPTH SCALE
1 : 75

Golder Associates

LOGGED: EHS
CHECKED: KG

RECORD OF BOREHOLE No 06-11

1 OF 2 **METRIC**

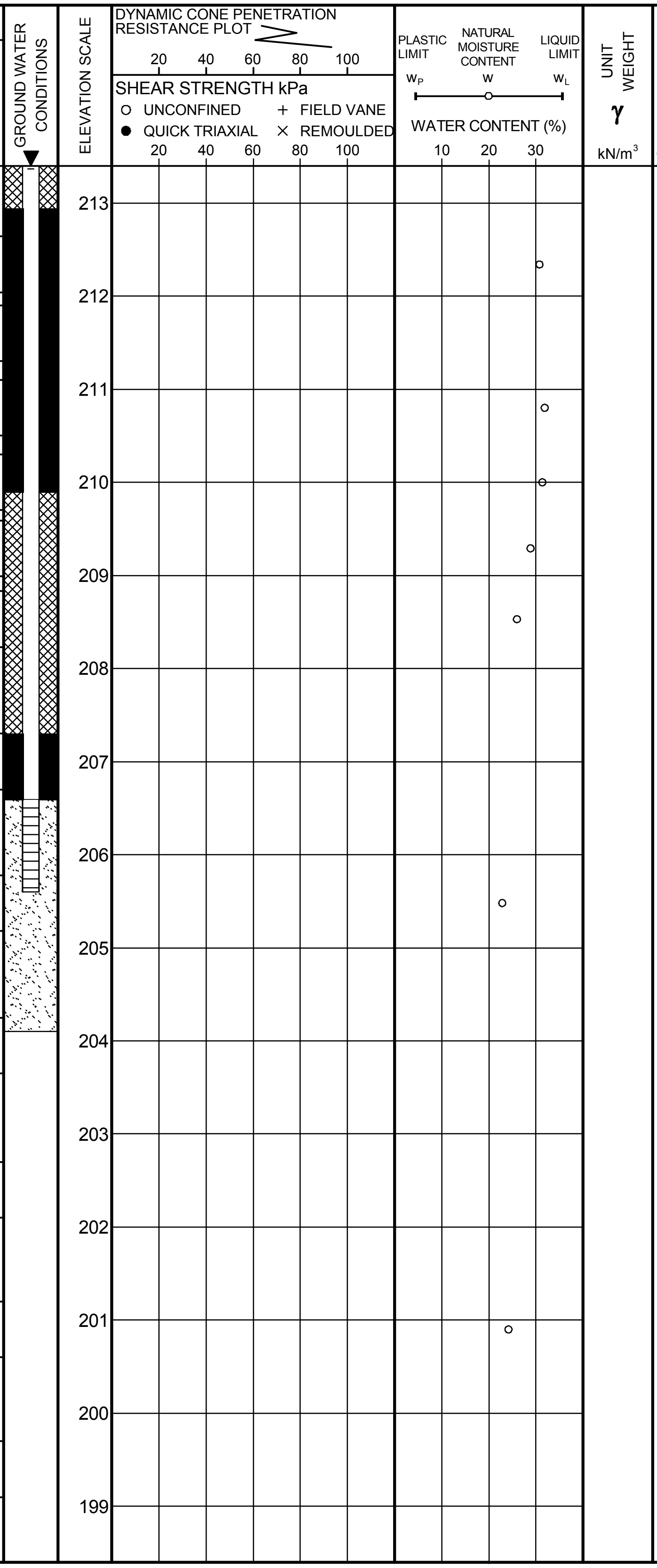
PROJECT 06-1111-001
W.P. 5379-02-00
LOCATION Sta. 18+365 o/s 13.0 m Rt CL Med.
ORIGINATED BY EHS
DIST HWY 69
BOREHOLE TYPE CME-55 Bombardier
COMPILED BY MM
DATUM Geodetic
DATE May 1, 2006
CHECKED BY KG

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
213.4	GROUND SURFACE
0.0	Silty organics (TOPSOIL) Dark brown
212.9 0.6	CLAYEY SILT, with organics Black
	CLAYEY SILT, trace sand and trace organics to 2.1 m depth, containing clay seams Very stiff to firm Grey and light brown, becoming layered grey and brown below 1.4 m depth Wet
209.0 4.4	SILT, trace to some sand, trace clay, containing sand seams and occasional clay and silty sand seams Loose Grey Wet
205.8 7.6	SANDY SILT, trace gravel and clay Very loose to compact Grey Wet
198.4	

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	4	
2	SS	6	
3	SS	6	
4	SS	2	
5	SS	3	
6	SS	7	
7	SS	5	0 10 87 3
8	SS	1	
9	SS	13	
10	SS	1	
11	SS	WR	
12	SS	WR	



Continued Next Page

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

RECORD OF BOREHOLE No 06-11

2 OF 2 **METRIC**

PROJECT 06-1111-001

W.P. 5379-02-00

LOCATION Sta. 18+365 o/s 13.0 m Rt CL Med.

ORIGINATED BY EHS

DIST HWY 69

BOREHOLE TYPE CME-55 Bombardier

COMPILED BY MM

DATUM Geodetic

DATE May 1, 2006

CHECKED BY KG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION --- CONTINUED FROM PREVIOUS PAGE ---	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
15.0	END OF BOREHOLE Note: 1. Auger refusal at 15.0 m depth. 2. Artesian ground water conditions. Hydrostatic water level at 1.4 m above ground surface upon completion of drilling. 3. Artesian ground water conditions. Hydrostatic water level in piezometer at 1.9 m above ground surface (Elev. 215.3m) on May 15, 2006. 4. Piezometer abandoned in accordance with O. Reg 128 on May 15, 2006.										

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

RECORD OF BOREHOLE No 06-12

1 OF 1 **METRIC**

PROJECT 06-1111-001

W.P. 5379-02-00

LOCATION Sta. 18+345 o/s 19.0 m Rt CL Med.

ORIGINATED BY EHS

DIST HWY 69

BOREHOLE TYPE CME-55 Bombardier

COMPILED BY MM

DATUM Geodetic

DATE May 1, 2006

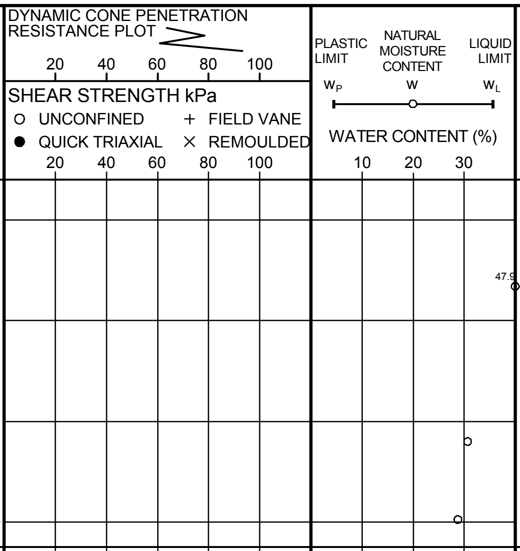
CHECKED BY KG

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
213.4	GROUND SURFACE							
0.0 213.0	Silty organics (TOPSOIL) Dark brown Wet					213		
0.6	CLAYEY SILT, some organics Black Wet	1	SS	5				
	CLAYEY SILT, trace sand, containing clay seams Very stiff to firm Grey and light brown with oxidized layers Wet					212		
		2	SS	10	∇			
		3	SS	6		211		
210.5 2.9	SILT, trace clay and sand Loose Grey Wet							
209.8		4	SS	4		210		
3.7	END OF BOREHOLE							

Note:

1. Water level in open borehole measured at 2.1 m depth (Elev. 211.3 m) upon completion of drilling operations.

2. Stratigraphy below 3.7 m depth, continued on Record of Borehole log 06-12A, drilled to a depth of 11.3 m, offset 0.6 m to the east of Borehole 06-12.



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

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

Golder Associates

RECORD OF BOREHOLE No 06-12A

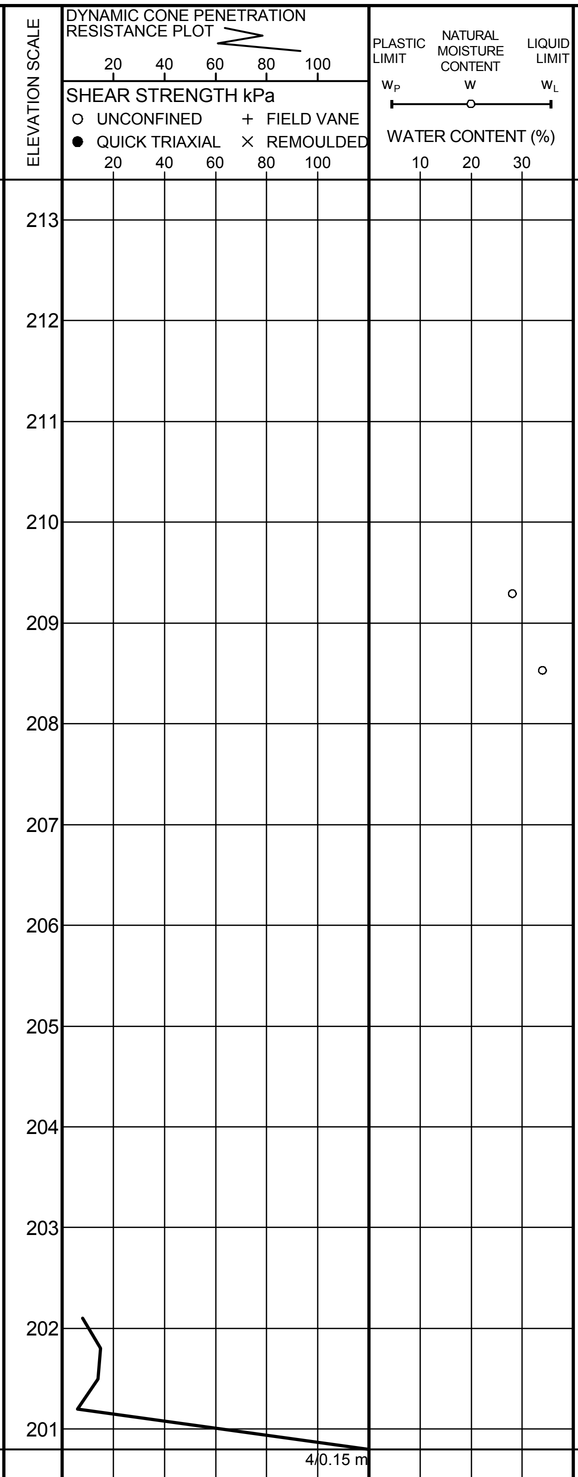
1 OF 1 **METRIC**

PROJECT 06-1111-001			
W.P. 5379-02-00	LOCATION Sta. 18+345 o/s 19.5 m Rt CL Med.		ORIGINATED BY EHS
DIST	HWY 69	BOREHOLE TYPE CME-55 Bombardier	COMPILED BY MM
DATUM Geodetic		DATE May 2-3, 2006	CHECKED BY KG

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
213.4	GROUND SURFACE					
0.0	Silty organics (TOPSOIL) Dark brown Saturated					
213.0 0.6	CLAYEY SILT, some organics Black Wet					
	CLAYEY SILT, trace sand, containing clay seams Very stiff to firm Grey and light brown with oxidized layers Wet				∇	
209.8 3.7	SILT, trace to some sand, containing silty sand and sand seams Loose Grey Wet	1	SS	4		
		2	SS	4		
		3	SS	4		
205.8 7.6	SANDY SILT, trace gravel and clay, containing silt seams and gravel Loose to very loose Layered Grey Wet	4	SS	5		
		5	SS	3		1 26 69 4
		6	SS	2		
202.1 11.3	END OF BOREHOLE Started Dynamic Cone Penetration Test					
200.8 12.6	END OF DYNAMIC CONE PENETRATION TEST					

Note:

1. Water level in open borehole measured at 2.1 m depth (Elev. 211.3 m) upon completion of drilling operations.



UNIT WEIGHT γ kN/m³

+³, ×³: Numbers refer to Sensitivity

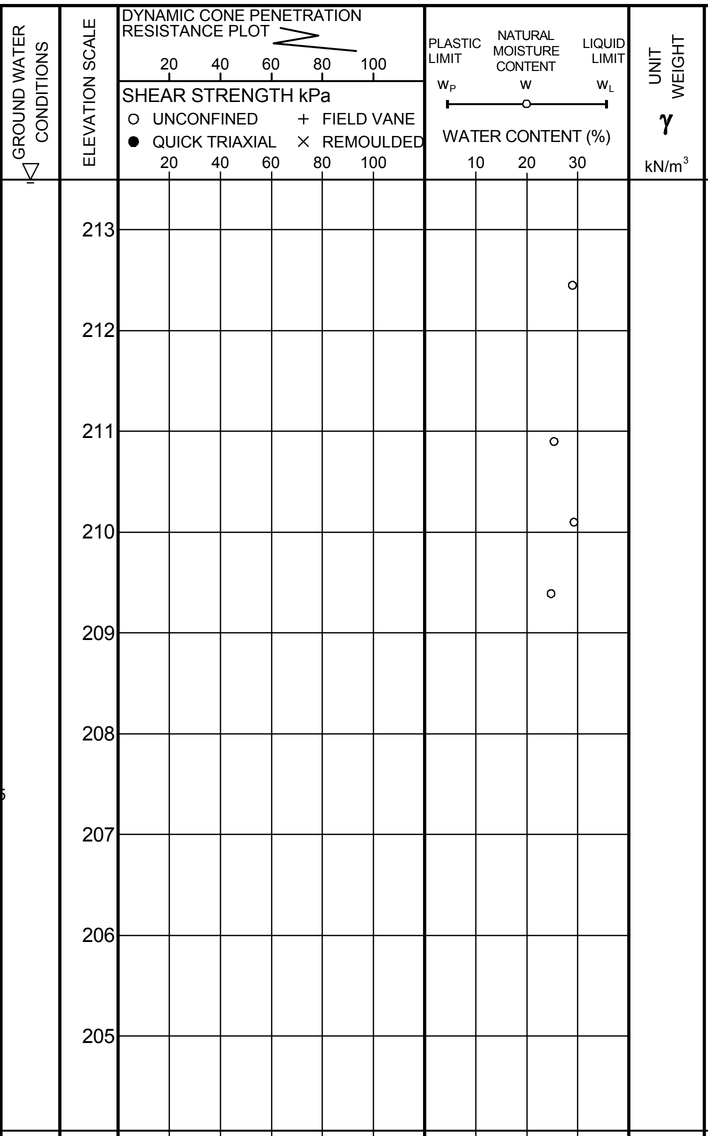
O 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 06-13

1 OF 1 **METRIC**

PROJECT 06-1111-001
W.P. 5379-02-00
LOCATION Sta. 18+363 o/s 13.3 m Lt CL Med.
ORIGINATED BY EHS
DIST HWY 69
BOREHOLE TYPE CME-55 Bombardier
COMPILED BY MM
DATUM Geodetic
DATE May 2, 2006
CHECKED BY KG

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
213.5	GROUND SURFACE							
0.0	Silty organics (TOPSOIL) Dark brown							
0.3	CLAYEY SILT Black Wet							
	CLAYEY SILT, trace sand and gravel, containing clay and sand seams Very stiff to firm Grey Moist becoming wet below 3.0 m depth	1	SS	9				
		2	SS	6				
		3	SS	6				
		4	SS	4				
209.8 3.7	SILT, trace to some sand and clay, containing sand seams Loose Grey Wet	5	SS	5	0	8	83	9
		6	SS	5				
207.7 5.8	SANDY SILT, trace gravel Very dense	7	SS	100/0.15				
207.3 6.2	BEDROCK Refer to Record of Drillhole log 06-13 for coring details.							
204.1 9.4	END OF BOREHOLE							



Note:

1. Water flowing above ground surface upon completion of drilling operations (artesian).

2. Refer to borehole 06-27 for vane shear strength.

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

PROJECT: 06-1111-001

RECORD OF DRILLHOLE: 06-13

SHEET 2 OF 2

LOCATION: Sta. 18+363 o/s 13.3 m Lt CL Med.

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: May 2, 2006
DRILL RIG: CME-55 Bombardier
DRILLING CONTRACTOR: Marathon Drilling Ltd.

DATUM: Geodetic

JN - Joint	BD - Bedding	PL - Planar	PO - Polished	BR - Broken Rock
FLT - Fault	FO - Foliation	CU - Curved	K - Slickensided	**NOTE:** For additional abbreviations refer to list of abbreviations & symbols.
SHR - Shear	CO - Contact	UN - Undulating	SM - Smooth	
VN - Vein	OR - Orthogonal	ST - Stepped	Ro - Rough	
CJ - Conjugate	CL - Cleavage	IR - Irregular	MB - Mechanical Break	

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	COLOUR % RETURN FLUSH	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Borehole 06-13		207.30					
7	CME 55 NQMay 2/06	Slightly weathered (W2), medium to strong (R3 to R4), medium grained, black, grey and pink GRANITE GNEISS with healed and partially healed joints (Hematite and Quartz Carbonate Infill).		6.20	1		reddish/grey 100	JN,FO,,SM JN,FO,,SM JN,FO,,SM JN,FO,,SM ,JN,PL,Ro ,JN,IR,SM SM,JN,,K MB,,, JN,FO,,SM ,JN,PL,SM JN,FO,,SM	
8					2		reddish/grey 100	JN,FO,,SM MB,,,	
9				204.06 9.44	3		reddish/grey 100	MB,,, ,JN,PL,SM JN,FO,,Ro MB,,,	
		END OF DRILLHOLE							
10									
11									
12									
13									
14									
15									
16									
17									
18									
19									
20									
21									

HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3} | Diametral Point Load Index (MPa) 2 4 6 | RMC -Q' AVG.

DEPTH SCALE
1 : 75

LOGGED: EHS
CHECKED: KG

RECORD OF BOREHOLE No 06-14

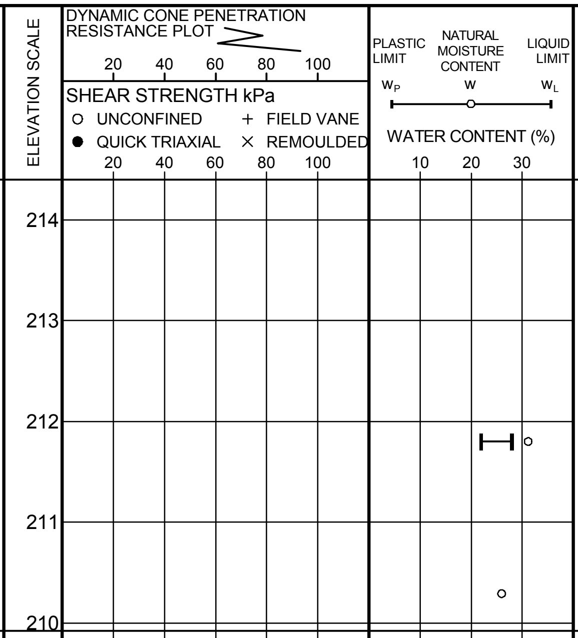
1 OF 1 **METRIC**

PROJECT 06-1111-001
W.P. 5379-02-00
DIST
HWY 69
DATUM Geodetic
LOCATION Sta. 18+365 o/s 25.9 m Lt CL Med.
BOREHOLE TYPE CME-55 Bombardier
DATE May 2, 2006
ORIGINATED BY EHS
COMPILED BY MM
CHECKED BY KG

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES			
214.4	GROUND SURFACE						
0.0	SAND, trace gravel and silt (FILL) Reddish Brown Moist						
213.6	ORGANICS Dark brown Wet	1	SS	5			
1.1	SAND, oxidized Very loose to loose Moist						
	CLAYEY SILT, trace sand, containing clay seams Very stiff to firm Reddish and light brown, grey with oxidized layers Wet	2	SS	10			
		3	SS	5	▽		
211.2 3.2	SILT, trace to some sand Loose to very loose Grey	4	SS	9			
		5	SS	3			
209.9 4.5	END OF BOREHOLE						

Note:

1. Auger refusal at 4.5 m depth.
2. Water level in open borehole measured at 3.1 m depth (Elev. 211.3 m) upon completion of drilling operations.



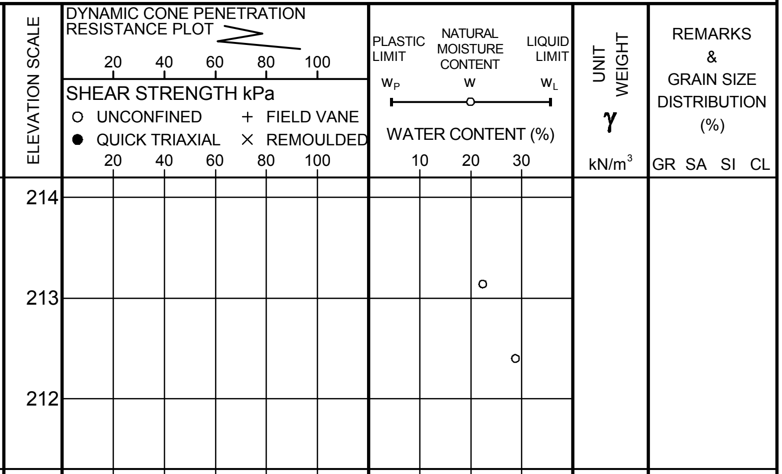
+3, ×3: Numbers refer to Sensitivity ○3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 06-15

1 OF 1 **METRIC**

PROJECT 06-1111-001
W.P. 5379-02-00
DIST ____ HWY 69
DATUM Geodetic
LOCATION Sta. 18+344 o/s 18.5 m Lt CL Med.
BOREHOLE TYPE CME-55 Bombardier
DATE May 2, 2006
ORIGINATED BY EHS
COMPILED BY MM
CHECKED BY KG

SOIL PROFILE			SAMPLES			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS
214.2	GROUND SURFACE					
0.0	SAND, trace silt and gravel (FILL) Light brown Wet					
213.6 0.7	ORGANIC Dark brown Wet		1	SS	11	
	CLAYEY SILT, trace sand, containing clay seams and occasional sand seams Very stiff to stiff Grey Moist to wet		2	SS	10	
			3	SS	14	⊻
211.3 2.9	END OF BOREHOLE Note: 1. Auger refusal at 2.9 m depth. 2. Water level in open borehole measured at 2.8 m depth (Elev. 211.4 m) upon completion of drilling operations.					



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

RECORD OF BOREHOLE No 06-25

1 OF 1 **METRIC**

PROJECT 06-1111-001

W.P. 5379-02-00

LOCATION Sta. 18+414 o/s 27.5 m East CL Med.

ORIGINATED BY EHS

DIST ______ HWY 69

BOREHOLE TYPE CME-55 Bombardier

COMPILED BY MM

DATUM Geodetic

DATE May 15, 2006

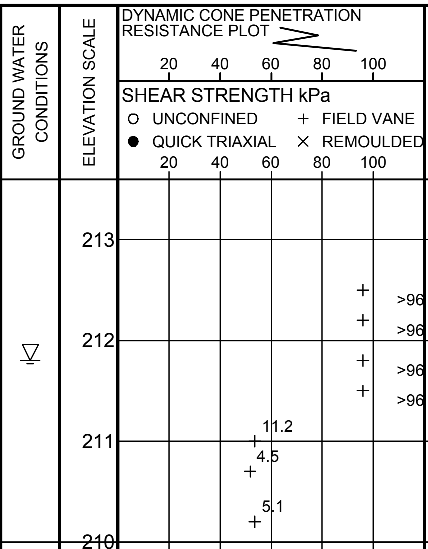
CHECKED BY KG

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES
213.6	GROUND SURFACE			
0.0 213.2				
0.4				
211.3 2.3	CLAYEY SILT with thin silt seams Stiff Varved/layered grey and light brown Moist	1	TO	PH
210.7 2.9				
210.1 3.6	END OF BOREHOLE			

Note:

1. Water level in open borehole measured at 1.8 m depth (Elev. 211.8 m) upon completion of drilling operations.

2. Where no samples were taken refer to adjacent borehole BH06-04, located 2.0 m west and 0.2 m south of borehole BH06-25 for stratigraphy.



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

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m^3

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

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

RECORD OF BOREHOLE No 06-26

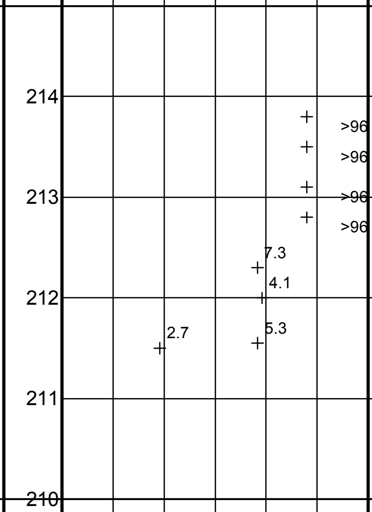
1 OF 1 **METRIC**

PROJECT 06-1111-001
W.P. 5379-02-00
LOCATION Sta. 18+415.5 o/s 26.0 m West CL Med.
ORIGINATED BY EHS
DIST HWY 69
BOREHOLE TYPE CME-55 Bombardier
COMPILED BY MM
DATUM Geodetic
DATE May 15, 2006
CHECKED BY KG

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
214.9	GROUND SURFACE						
0.0 214.6 0.3							
212.6 2.3	CLAYEY SILT with thin silt seams Stiff Grey, becoming layered grey and brown with depth Moist	1	TO	PH		19.2	
212.0 2.9							
211.2 3.7							
210.0 4.9	END OF BOREHOLE 1. Where no samples were taken refer to adjacent borehole BH06-08, located 1.6 m south of borehole BH06-26 for stratigraphy.						

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



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

RECORD OF BOREHOLE No 06-27

1 OF 1 **METRIC**

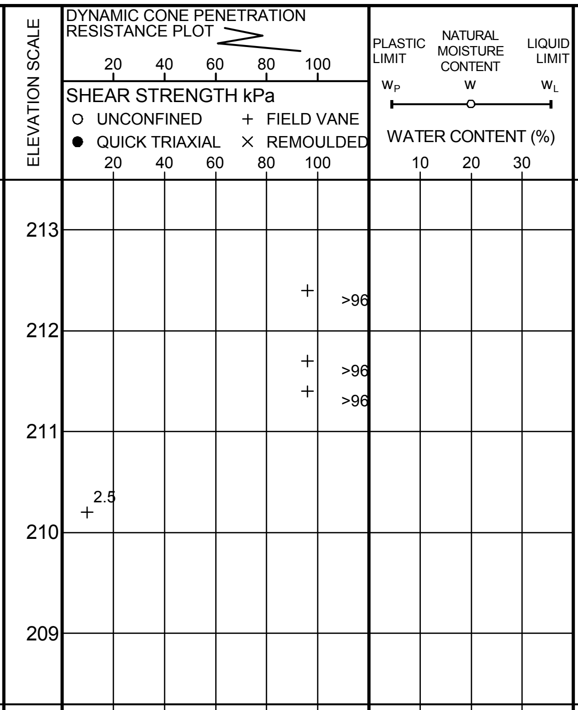
PROJECT 06-1111-001
W.P. 5379-02-00
LOCATION Sta. 18+360.5 o/s 14.0 m West CL Med.
ORIGINATED BY EHS
DIST HWY 69
BOREHOLE TYPE CME-55 Bombardier
COMPILED BY MM
DATUM Geodetic
DATE May 15, 2006
CHECKED BY KG

ELEV DEPTH	DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
213.5 0.0	GROUND SURFACE						
0.2							
212.0 1.6	SANDY SILT, some organics Loose to Compact Grey Wet	1	TO	PH			
211.4 2.1	CLAYEY SILT Very stiff Grey Moist				▽		
209.8 3.7							
208.3 5.2	END OF BOREHOLE						

Note:

1. Water level in open borehole measured at 2.4 m depth (Elev. 211.1 m) upon completion of drilling operations.

2. Where no samples were taken refer to adjacent borehole BH06-13, located 2.5 m north and 0.8 m east of borehole BH06-27 for stratigraphy.



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

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

TABLE 1 - SUMMARY OF POINT LOAD TESTS ON ROCK CORE SAMPLES

PROJECT NO.: 06-1111-001-1
LOCATION: Wildlife Crossing Under Highway 69
DATE: July, 2006

Borehole Number	Sample Number	Rock Type	Sample Depth (ft)	Sample Depth (m)	Test Type	**Is (50mm)** (MPa)	**Approx. UCS[1]** (Is_{50}x20)(MPa)
06-07	1	Granitic Gneiss	26.75	8.2	D	9.04	181
06-07	2	Granitic Gneiss	27.00	8.2	A	9.30	186
06-07	3	Granitic Gneiss	27.50	8.4	D	6.91	138
06-07	4	Granitic Gneiss	27.25	8.3	A	6.70	134
06-10	1	Granitic Gneiss	53.00	16.2	A	7.69	177
06-10	2	Granitic Gneiss	53.30	16.2	D	5.45	125
06-10	3	Granitic Gneiss	53.50	16.3	A	-	0
06-10	4	Granitic Gneiss	53.90	16.4	A	7.95	183
06-10	5	Granitic Gneiss	54.30	16.6	D	7.33	169

SUMMARY[2]

Average Axial	**7.9**	**158.2**
Average Diametral	**7.2**	**143.6**
St. Dev. Axial	**1.1**	**24.2**
St. Dev. Diametral	**1.5**	**25.9**
Number of Axial Tests	**5**	
Number of Diametral Tests	**4**	

[1] UCS = Is x 20 is based on previous experience and would require UCS testing to further validate this relationship.

[2] Statistical summary based on the removal of the 2 highest and 2 lowest values.

Note: Specimens tend to be anisotropic in nature (ie. stronger axial than diametral).

TABLE 2
EVALUATION OF FOUNDATION ALTERNATIVES
Wildlife Crossing Under Highway 69 at Station 18+400
G.W.P. 5379-02-00

Footing Option	*NF*	*Advantages*	*Disadvantages*	*Relative Costs*	*Risks/Consequences*
Steel H Piles		Relatively straight forward construction. Allows for integral abutment design.	Downdrag loads and lateral displacements have to be considered unless embankment preloading is carried out prior to pile installation as part of approach embankment settlement mitigation alternative.	Lower relative costs than spread footings on bedrock.	Relatively thin overburden at North Abutment of NBL and anticipated thin overburden at South Abutment of SBL. Pile lengths below integral abutment CSPs at these locations should be reviewed by the structural engineer for lateral resistance. Given the presence of artesian groundwater conditions at some locations, a filter blanket should be installed around the CSPs/piles near the ground surface (to prevent loss of fines) along with a subdrain (to promote drainage away from the abutment area).
Spread Footings perched within embankment fill	X		Potential for differential settlement between north and south abutments due to compression of embankment fill and underlying subsoils.	Lower relative costs than piled foundations.	Not recommended due to potential for differential settlements anticipated between north and south abutments.
Spread Footings on bedrock or mass concrete pad	X	Mitigates differential settlement between abutments.	Deep excavations through artesian groundwater conditions would be required at some locations. Temporary sheeting and groundwater control required to expose bedrock surface. Variable bedrock surface will require bedrock and soil excavation with mass concrete placement to achieve level footing.	Increased cost for groundwater control and temporary sheeting as compared with shallower footings.	Not recommended due to significant depth of excavations and artesian groundwater conditions at some locations.
Shallow Spread Footings on very stiff clayey silt	X		Low geotechnical resistance. Differential settlements between north and south abutments due to consolidation of underlying firm clayey silt and compression of silts and sand and silt strata.	Lower relative costs than piled foundations.	Not recommended due to potential for differential settlements anticipated between north and south abutments.

n:\active\2006\1111\06-1111-001 urs-hwy69 wildlife crossings\reporting\draft\06-1111-001 wildlife xing under hwy 69\table2_evaluation foundation alternatives.doc

NF: Indicates that the founding option is considered not feasible.

TABLE 3
Summary of Recommendations at Structure Approach Embankments (incl. Platform Widening)
Wildlife Crossing Under Highway 69 at Station 18+400
G.W.P. 5379-02-00

Structure Approach	Approx. Station	Proposed Works	Surface Conditions	Recommended Embankment Fill Type	Organics Encountered Along Alignment	Recommended Side Slope	Preferred Mitigation Alternative (Side Berm Requirement)	Estimated Post-Construction Settlement (δ)* and Platform Widening (w)** (mm)	Swamp Excavation / Organic Removal OPSD
NBL and SBL – South Approaches	18+319 to 18+339	South Approach Embankments (fill up to 10m high).	Generally flat, low lying area with tree, shrub cover and surface organics. Bedrock outcropping to the northwest and southeast.	Rock fill	Yes. Up to 0.7 m below ground surface.+	1.25H : 1V	Preloading (no side berms).	δ = 105 + 90 = 195 w = 1000	Remove all organics within footprint of embankment.
NBL and SBL – North Approaches	18+416 to 18+436	North Approach Embankments (fill up to 9 m high).	Generally flat, low lying area with tree, shrub cover and surface organics. Bedrock outcropping to the northwest and southeast.	Rock fill	Yes. Up to 0.3 m below ground surface.	1.25H: 1V	Preloading (no side berms).	δ = 95 + 75 = 170 w = 1000	Remove all organics within footprint of embankment.

n:\active\2006\1111\06-1111-001 urs-hwy69 wildlife crossings\reporting\draft\06-1111-001 wildlife xing under hwy 69\table3_summaryapproachembankmentrecommendations (incl nre 98-200).doc

Note : *** Settlements include compression of rock fill plus compression of cohesive soil layers below embankment (where encountered). Assumes that preloading will be adopted as mitigation measure.**

**** Recommended embankment platform widening (per embankment side) based on guidelines in NRE 98-200.**

+ Inferred based on closest boreholes to the area.

TABLE 4

EVALUATION OF APPROACH EMBANKMENT SETTLEMENT MITIGATION ALTERNATIVES

Wildlife Crossing Under Highway 69 at Station 18+400
G.W.P. 5379-02-00

Stability/ Settlement Mitigation Option	*Rank*	*Advantages*	*Disadvantages*	*Relative Costs*	*Risks/Consequences*
Pre-Loading (about 1 month)	1	Relatively simple operation; no deep sub-excavation or temporary shoring.	May lengthen construction time required. Settlement of foundation soil takes less than 1 month to reach 95% consolidation.	Low cost. However some additional costs required for placement and then removal of preload fills in the abutment areas to allow piling and abutment construction.	Settlement of embankment/ foundation soils will occur.
Full Subexcavation and Replacement (up to about 4 m deep)	2	Post-construction settlement issues reduced since all or nearly all compressible materials are removed.	High groundwater levels and artesian conditions may result in base heave failures and difficult ‘in-the-wet’ excavation and then placement of new fill under water.	Additional costs for sub-excavation and replacement fills.	Low risk with respect to long term settlement of foundation soils. Additional fill settlement due to increased effective embankment height.
Surcharging	3	Marginal decrease in length of time required to reach 95% consolidation.	Stability berms may be required depending on height of surcharge adopted.	Increased cost of construction and material for surcharge.	Still expect some settlement of embankment fill.
Wick Drains	X	Marginal decrease in length of time required to reach 95% consolidation.	Increased time for installation of wicks. Monitoring of settlements and pore pressures required. Wick drains less effective in over-consolidated and partially over-consolidated soils.	Higher costs due to requirement for drain installation and monitoring costs.	Still expect some settlement of embankment fill.
Light Weight Fill (EPS)	X	Reduces load on compressible soils thereby reducing settlement of foundation soils. Settlement of embankment fill reduced.	Very high material costs.	Relative cost of fill is up to an order of magnitude higher than for the other options.	Settlements of foundation soils and embankment fill minimized.

n:\active\2006\1111\06-1111-001 urs-hwy69 wildlife crossings\reporting\draft\06-1111-001 wildlife xing under hwy 69\table4_evaluation settlement mitigation alternatives.doc

Golder Associates
Mississauga, Ontario, Canada

SCALE	NOT TO SCALE	TITLE	
DATE	JUNE 2006		
DESIGN			
CAD	MSM		
CHECK	KG		
REVIEW	JPD		

SITE LOCATION MAP

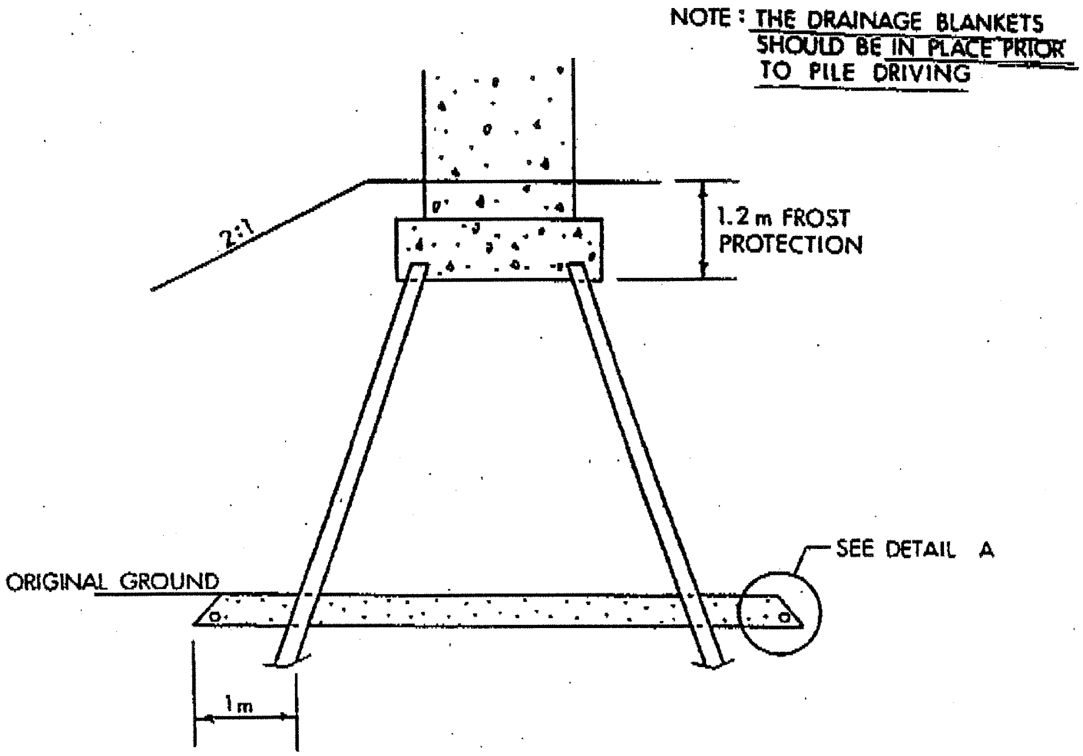
WILDLIFE CROSSING UNDER HIGHWAY 69

FIGURE 1

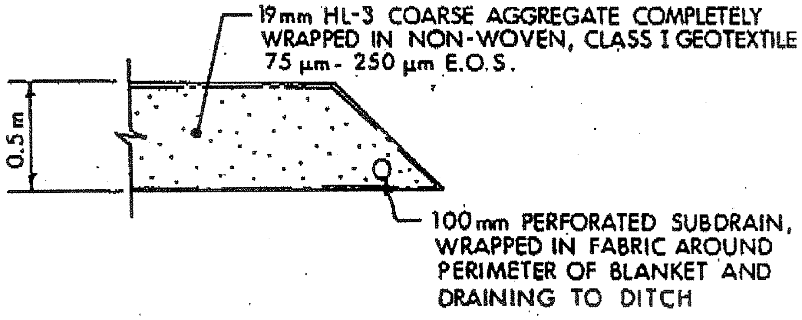
FILE No. 061111001CA001.dwg

PROJECT No. 06-1111-001 REV.

PLOT DATE: June 16, 2006
FILENAME: T:\Projects\2006\06-1111-001 (MTO URS, Burwash)\-CA- (Figures)\061111001CA001.dwg



ABUTMENT SECTION (TYP)



DETAIL A.

PLOT DATE: July 24, 2006
FILENAME: T:\Projects\2006\06-1111-001 (MTO URS, Burwash)\-CA- (Figures)\061111001CA02.dwg

Golder Associates
Mississauga, Ontario, Canada

FILE No. 061111001CA02.dwg
PROJECT No. 06-1111-001
REV.

SCALE	AS SHOWN
DATE	JULY 2006
DESIGN	
CAD	MSM
CHECK	JPD
REVIEW	JMAC

TITLE

DRAINAGE BLANKET DETAILS FOR ABUTMENT AND PIER PILES

HIGHWAY 69 WILDLIFE CROSSING

FIGURE 2

MINISTRY OF TRANSPORTATION, ONTARIO

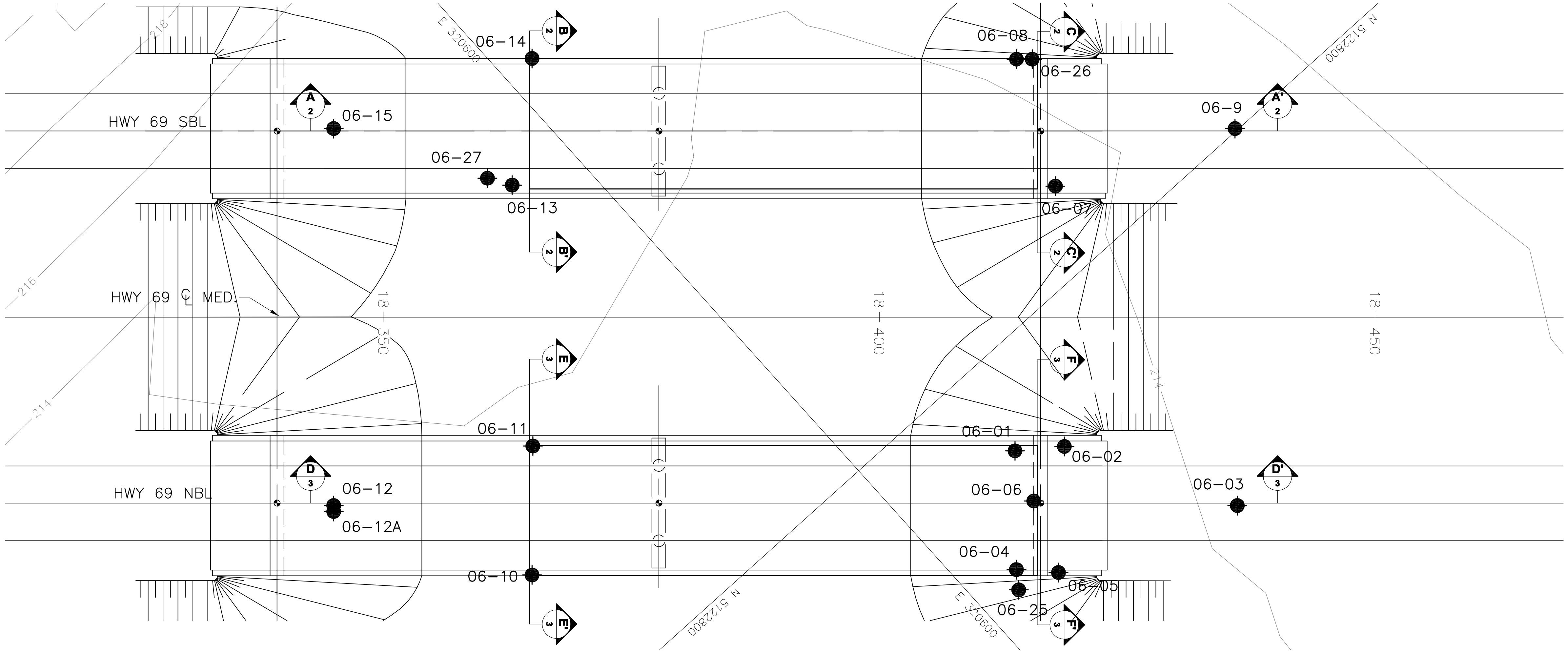
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 5379–02–00

SHEET

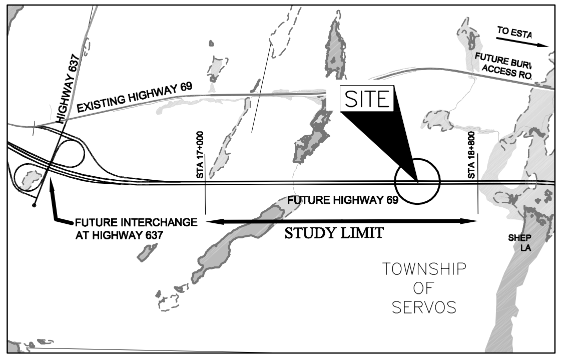
HIGHWAY 69 FOUR–LANING
WILDLIFE CROSSING
UNDER HIGHWAY 69
BOREHOLE LOCATION

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



PLAN

SCALE
5 0 5 10 m
1:250



KEY PLAN

SCALE
0.5 0 0.5 1 km
1:25000

LEGEND

● Borehole – Current Investigation

No.	ELEVATION	CO–ORDINATES	
		NORTHING	EASTING
06–01	213.70	5122809	320587
06–02	213.70	5122812	320583
06–03	214.40	5122828	320574
06–04	213.60	5122818	320595
06–05	213.60	5122821	320592
06–06	213.70	5122814	320589
06–07	214.20	5122792	320566
06–08	214.90	5122780	320560
06–09	217.10	5122800	320549
06–10	213.40	5122786	320632
06–11	213.40	5122776	320623
06–12	213.40	5122767	320642
06–12A	213.40	5122768	320642
06–13	213.50	5122755	320607
06–14	214.40	5122747	320597
06–15	214.20	5122739	320616
06–25	213.60	5122820	320596
06–26	214.90	5122781	320559
06–27	213.50	5122753	320608

NOTES

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

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

J. M. A. COSTA

J. P. DITTRICH
Paul Dittrich

REFERENCE

Base plans provided in digital format by URS, drawing file nos. Future Hwy 69 – PLAN.dwg, dtm_contours_labeled.dwg, received April 12, 2006 and 03–GA–Hwy69WildlifeUNDER–CPCI–NOSKEW2D.dwg, recieved June 7, 2006.

NO.	DATE	BY	REVISION

Geocres No. 41I–201

HWY. 69		PROJECT NO. 06–1111–001	DIST.
SUBM'D. KG	CHKD. KG	DATE: JULY 2006	SITE:
DRAWN: MSM	CHKD. JPD	APPD. JMAC	DWG. 1

PLOT DATE: July 10, 2006
FILENAME: T:\Projects\2006\06–1111–001 (MTO URS, Burwash)\–AB–\061111001AB001.dwg

MINISTRY OF TRANSPORTATION, ONTARIO

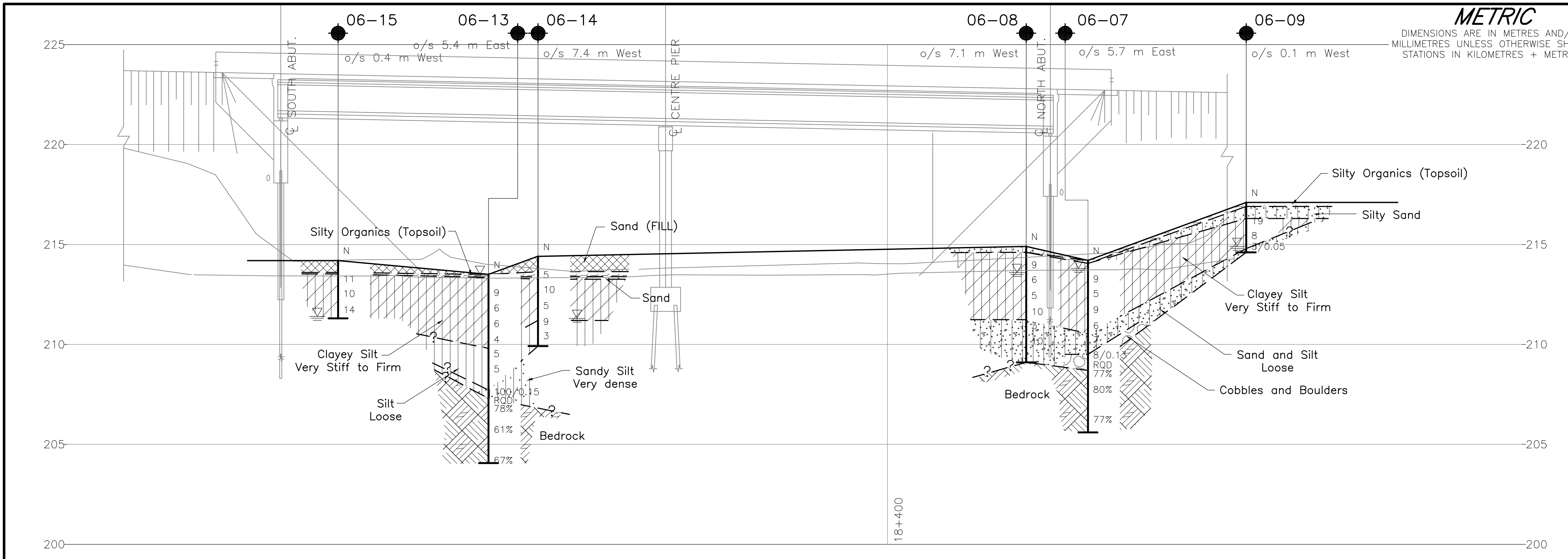
CONT No.
WP No. 5379-02-00

HIGHWAY 69 FOUR-LANING
WILDLIFE CROSSING
UNDER HIGHWAY 69
BOREHOLE SOIL STRATA

SHEET

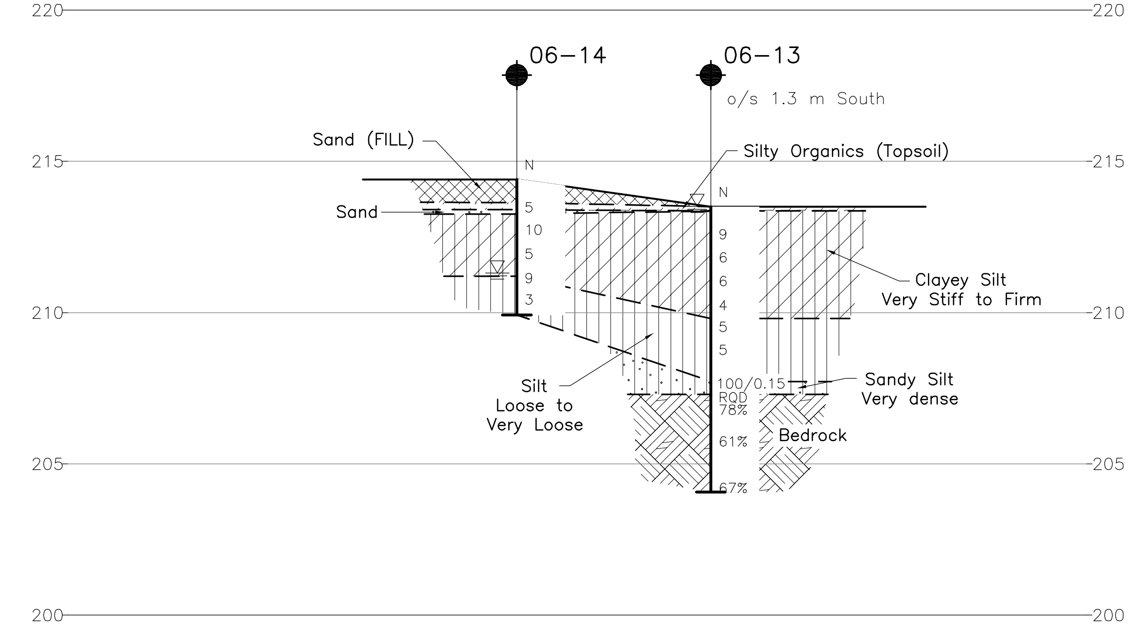
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

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



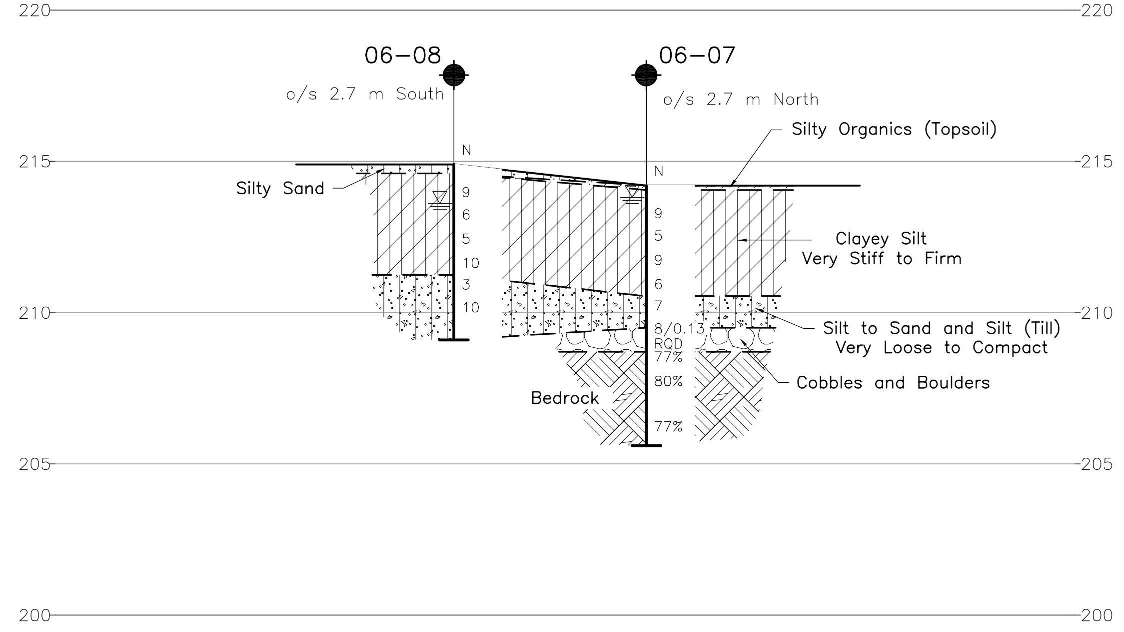
WILDLIFE CROSSING UNDER HWY 69 SBL BRIDGE

A-A'
1



WILDLIFE CROSSING UNDER HWY 69 SBL BRIDGE
OFFSET 13 m SOUTH OF CENTRE PIER
OFFSET 26 m NORTH OF SOUTH ABUTMENT

B-B'
1



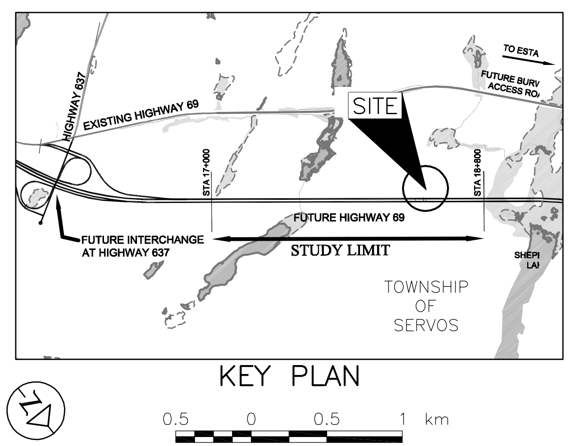
WILDLIFE CROSSING UNDER HWY 69 SBL BRIDGE
NORTH ABUTMENT

C-C'
1

HORIZONTAL SCALE
5 0 5 10 m
1:250

VERTICAL SCALE
2.5 0 2.5 5 m
1:125

J. M. A. COSTA
J. P. DITTRICH



KEY PLAN

LEGEND

●	Borehole – Current Investigation
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock Quality Designation (RQD)
▽	WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
06-01	213.70	5122809	320587
06-02	213.70	5122812	320583
06-03	214.40	5122828	320574
06-04	213.60	5122818	320595
06-05	213.60	5122821	320592
06-06	213.70	5122814	320589
06-07	214.20	5122792	320566
06-08	214.90	5122780	320560
06-09	217.10	5122800	320549
06-10	213.40	5122786	320632
06-11	213.40	5122776	320623
06-12	213.40	5122767	320642
06-12A	213.40	5122768	320642
06-13	213.50	5122755	320607
06-14	214.40	5122747	320597
06-15	214.20	5122739	320616

NOTES

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

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

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

REFERENCE

Base plans provided in digital format by URS, drawing file nos. Future Hwy 69 – PLAN.dwg, dtm_contours_labeled.dwg, received April 12, 2006 and 03-GA-Hwy69WildlifeUNDER-CPCI-NOSKEW2D.dwg, recieved June 7, 2006.

NO.	DATE	BY	REVISION

Geocres No. 41I-201

HWY. 69	PROJECT NO. 06-1111-001		DIST.
SUBM'D. KG	CHKD. KG	DATE: JULY 2006	SITE:
DRAWN: MSM	CHKD. JPD	APPD. JMAC	DWG. 2

PLOT DATE: July 10, 2006
FILENAME: T:\Projects\2006\06-1111-001 (MTO URS, Burwash)\-AB-\06111001AB002.dwg

MINISTRY OF TRANSPORTATION, ONTARIO

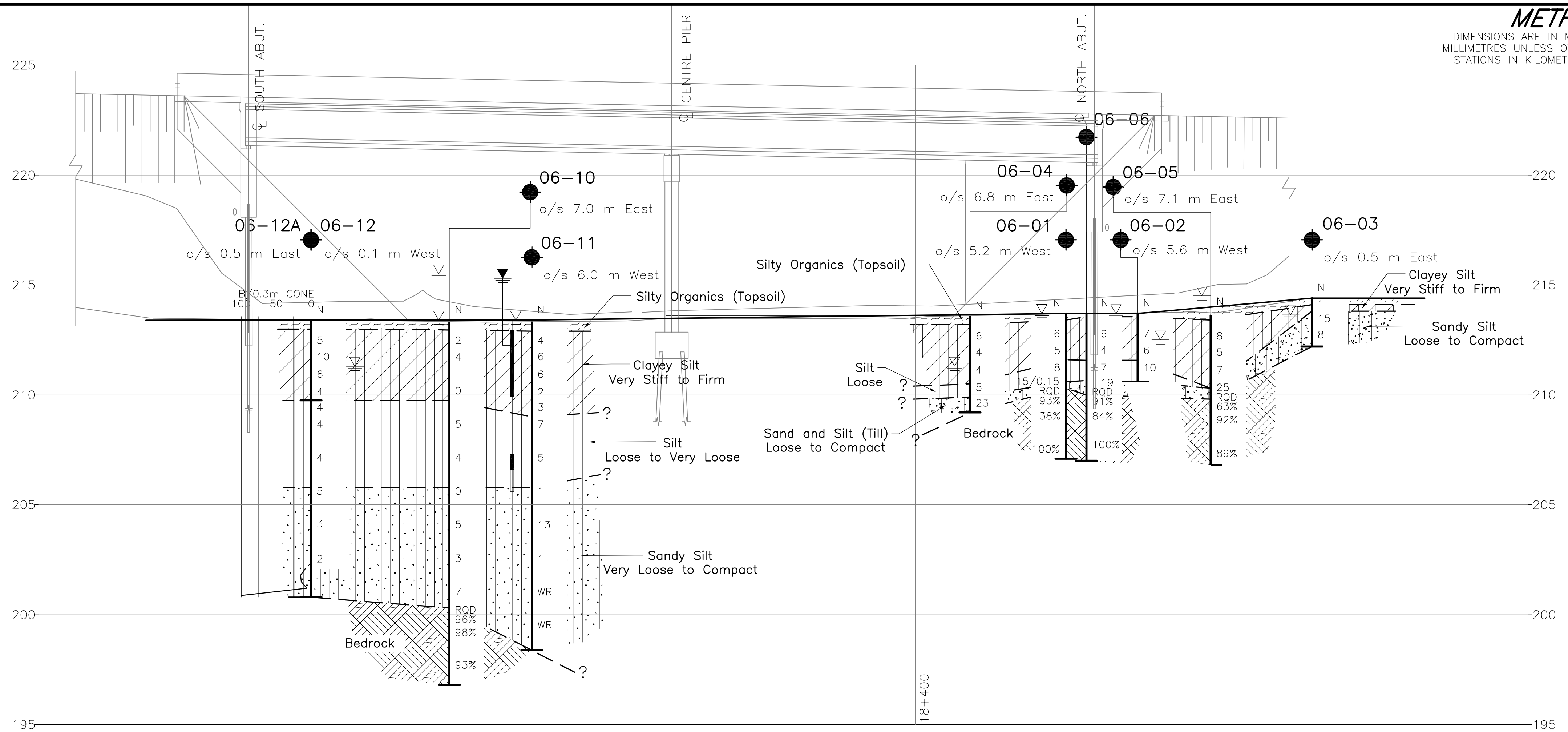
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 5379-02-00

SHEET

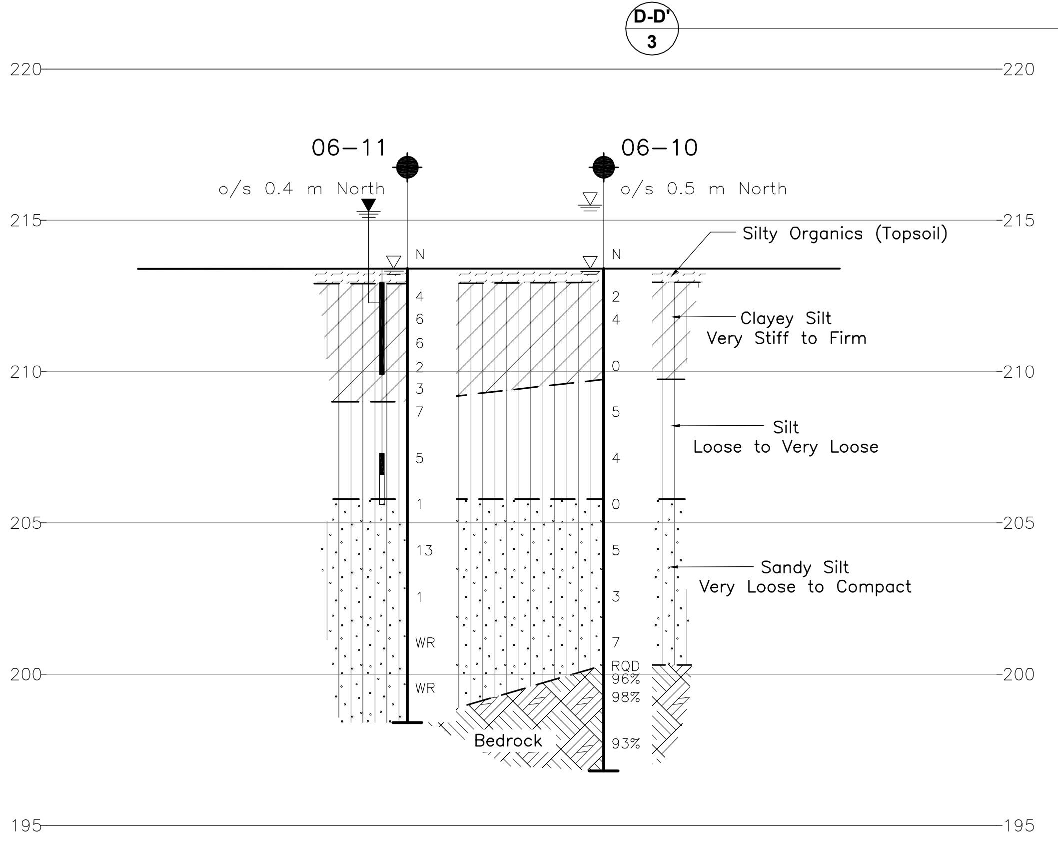
HIGHWAY 69 FOUR-LANING
WILDLIFE CROSSING
UNDER HIGHWAY 69
BOREHOLE SOIL STRATA

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



WILDLIFE CROSSING UNDER HWY 69 NBL BRIDGE

D-D'
3

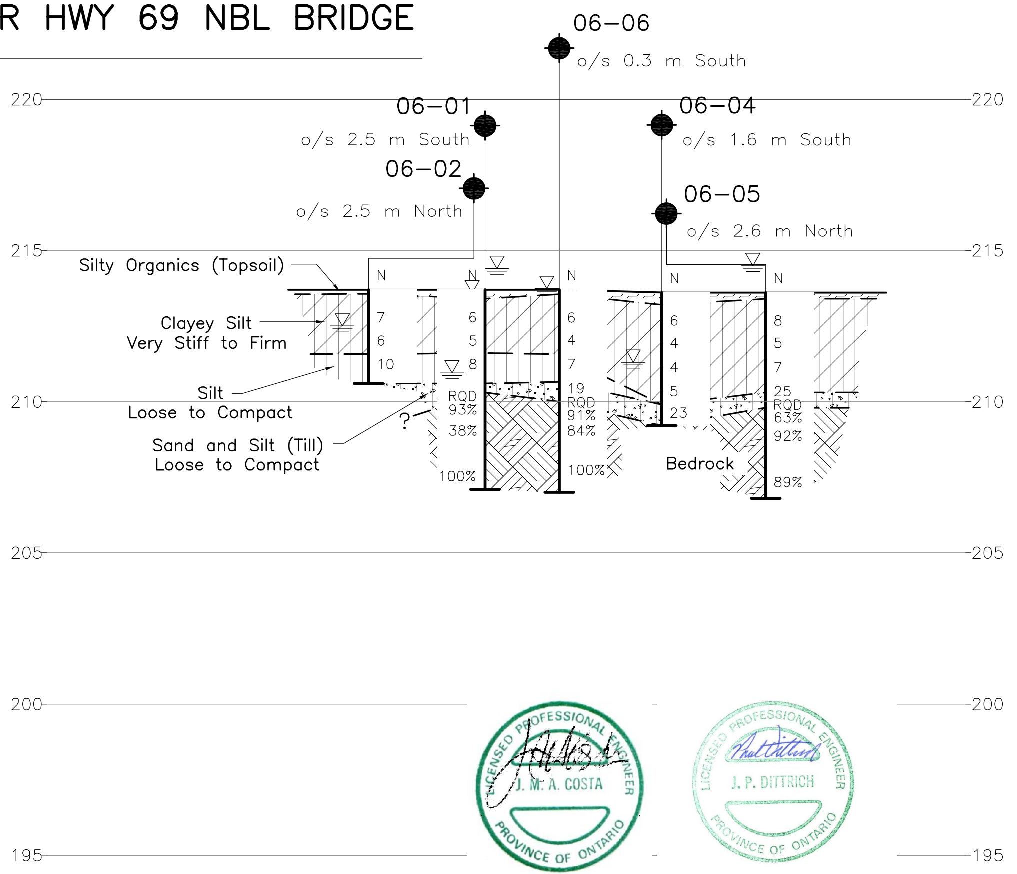


WILDLIFE CROSSING UNDER HWY 69 NBL BRIDGE
OFFSET 13 m SOUTH OF CENTRE PIER
OFFSET 26 m NORTH OF SOUTH ABUTMENT

E-E'
3

HORIZONTAL SCALE
5 0 5 10 m
1:250

VERTICAL SCALE
2.5 0 2.5 5 m
1:125

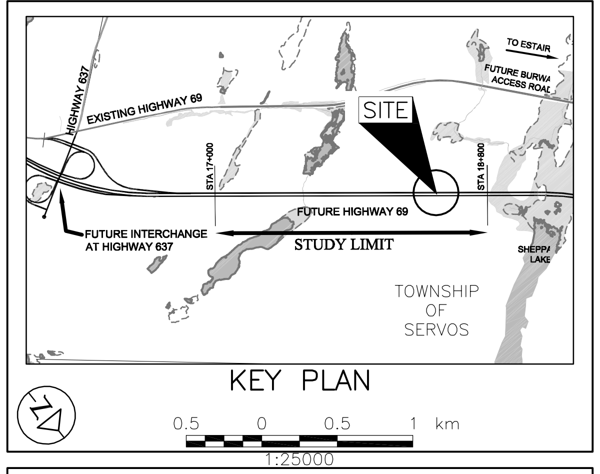


J. M. A. COSTA

J. P. DITTRICH

WILDLIFE CROSSING UNDER HWY 69
NBL BRIDGE NORTH ABUTMENT

F-F'
3



KEY PLAN

LEGEND

●	Borehole – Current Investigation
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock Quality Designation (RQD)
▽	WL upon completion of drilling
▼	WL in piezometer, measured on May 15, 2006

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
06-01	213.70	5122809	320587
06-02	213.70	5122812	320583
06-03	214.40	5122828	320574
06-04	213.60	5122818	320595
06-05	213.60	5122821	320592
06-06	213.70	5122814	320589
06-07	214.20	5122792	320566
06-08	214.90	5122780	320560
06-09	217.10	5122800	320549
06-10	213.40	5122786	320632
06-11	213.40	5122776	320623
06-12	213.40	5122767	320642
06-12A	213.40	5122768	320642
06-13	213.50	5122755	320607
06-14	214.40	5122747	320597
06-15	214.20	5122739	320616

NOTES

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

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

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

REFERENCE

Base plans provided in digital format by URS, drawing file nos. Future Hwy 69 – PLAN.dwg, dtm_contours_labeled.dwg, received April 12, 2006 and 03-GA-Hwy69WildlifeUNDER-CPCI-NOSKEW2D.dwg, recieved June 7, 2006.

NO.	DATE	BY	REVISION

Geocres No. 41I-201

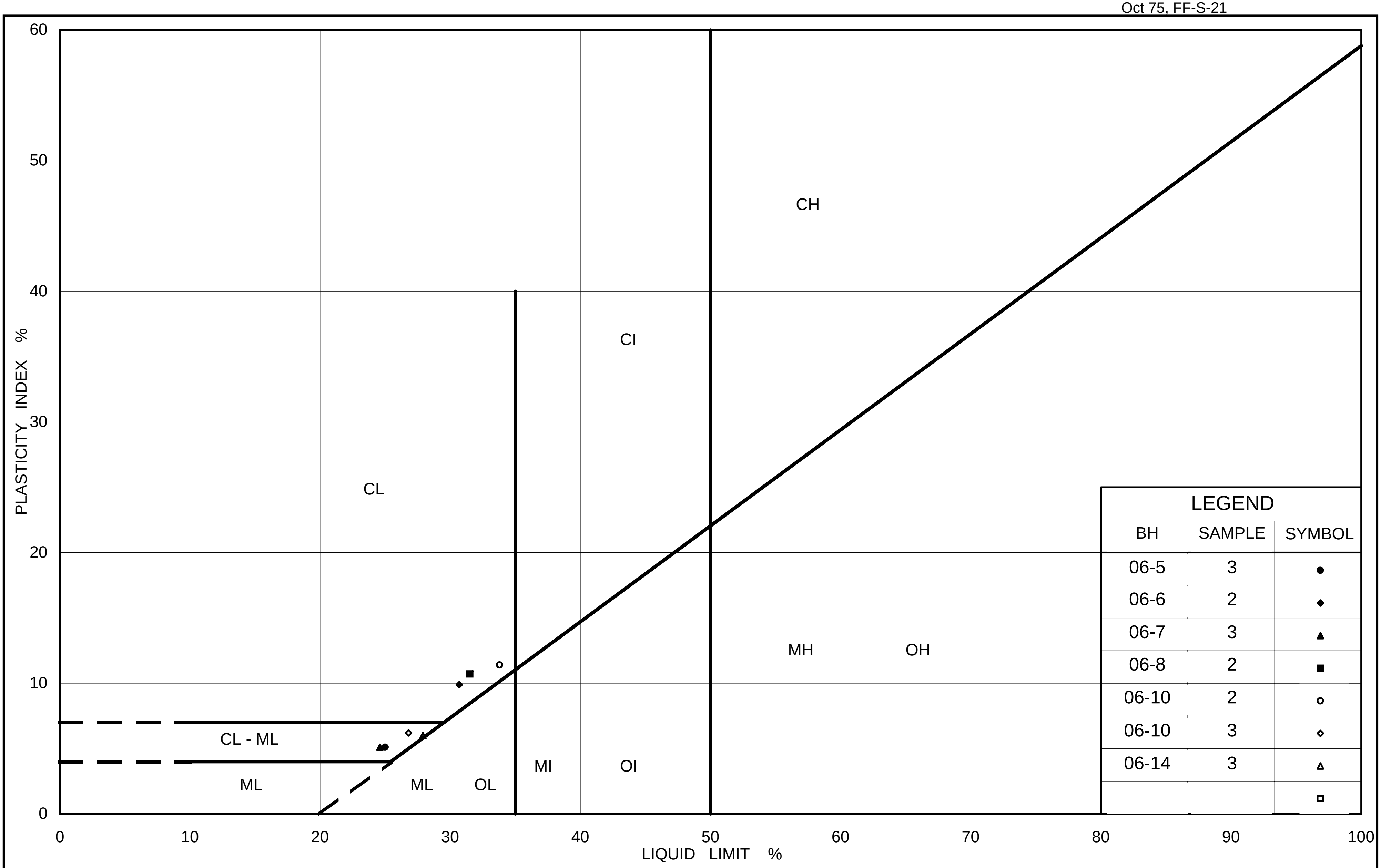
HWY. 69		PROJECT NO. 06-1111-001	DIST.
SUBM'D. KG	CHKD. KG	DATE: JULY 2006	SITE:
DRAWN: MSM	CHKD. JPD	APPD. JMAC	DWG. 3

PLOT DATE: July 10, 2006
FILENAME: T:\Projects\2006\06-1111-001 (MTO URS, Burwash)\-AB-\0611111001AB003.dwg

APPENDIX A

LABORATORY TEST DATA

Oct 75, FF-S-21



LEGEND

BH	SAMPLE	SYMBOL
06-5	3	●
06-6	2	◆
06-7	3	▲
06-8	2	■
06-10	2	○
06-10	3	◇
06-14	3	△
		□

Ministry of Transportation

Ontario

PLASTICITY CHART
Clayey Silt

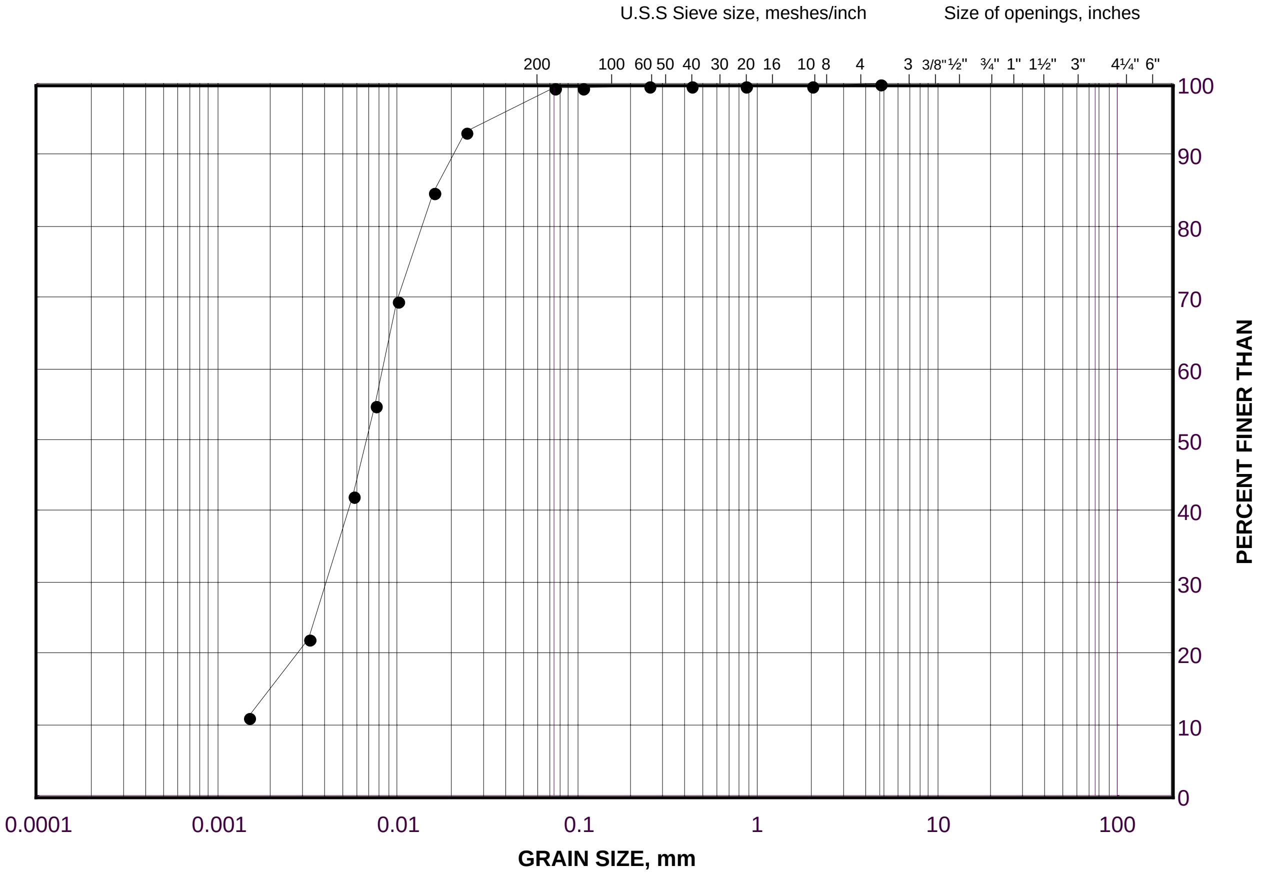
FIG No. A-1

Project No. 06-1111-001

GRAIN SIZE DISTRIBUTION

Clayey Silt

FIGURE A-2



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	06-5	3	2.3-2.9

Project Number: 06-1111-001

Checked By: ______________________

Golder Associates

Date: 13-Jun-06

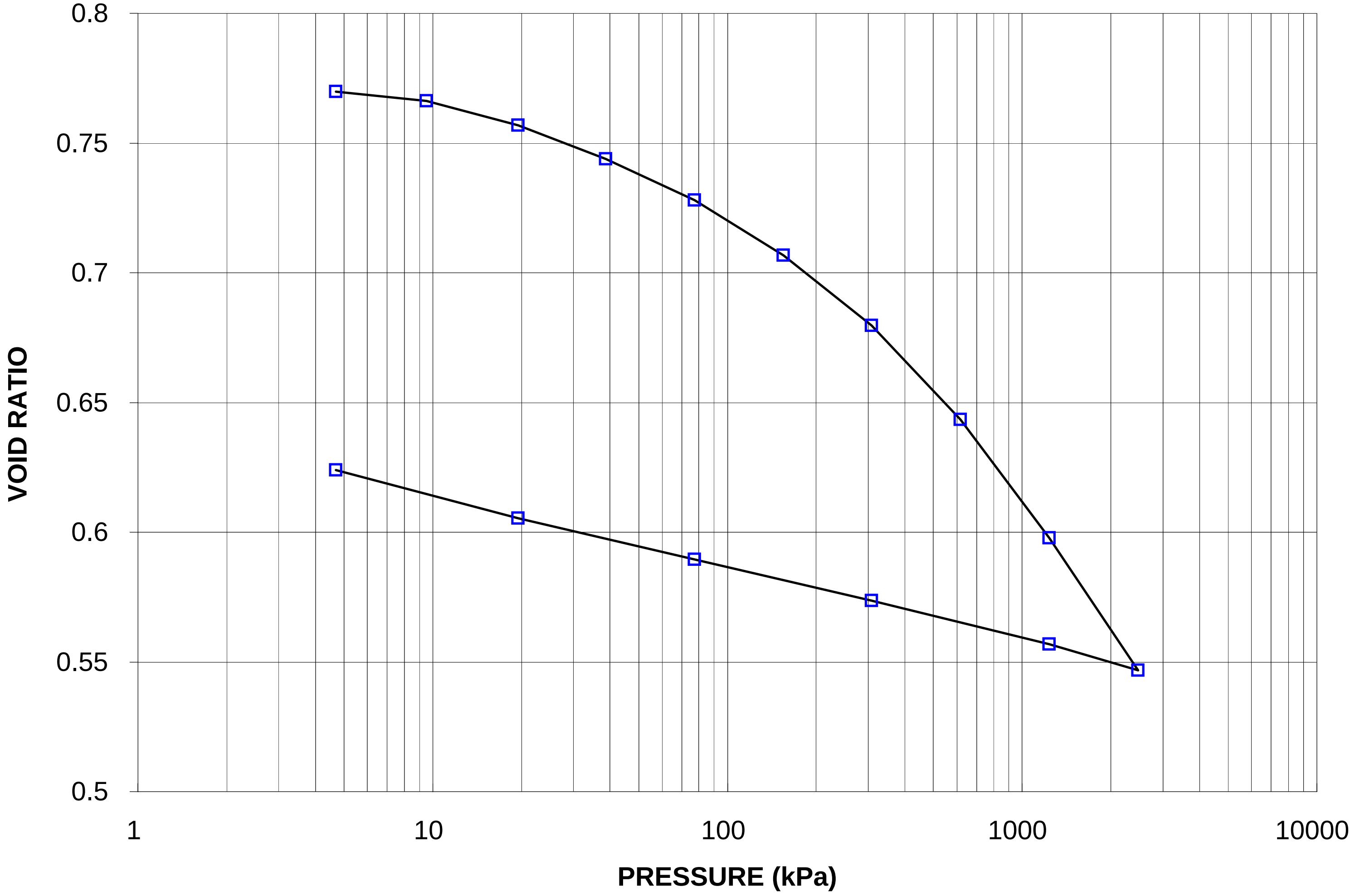
Project No. 06-1111-001

Prepared By: LFG

Golder Associates

Checked By: MM

CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BH 06-26 SA 1



CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE A-3a

Project No. 06-1111-001
Prepared By: LFG

Golder Associates

Checked By: MM

CONSOLIDATION TEST
TOTAL WORK, kJ/m^3 vs PRESSURE
BH 06-26 SA 1

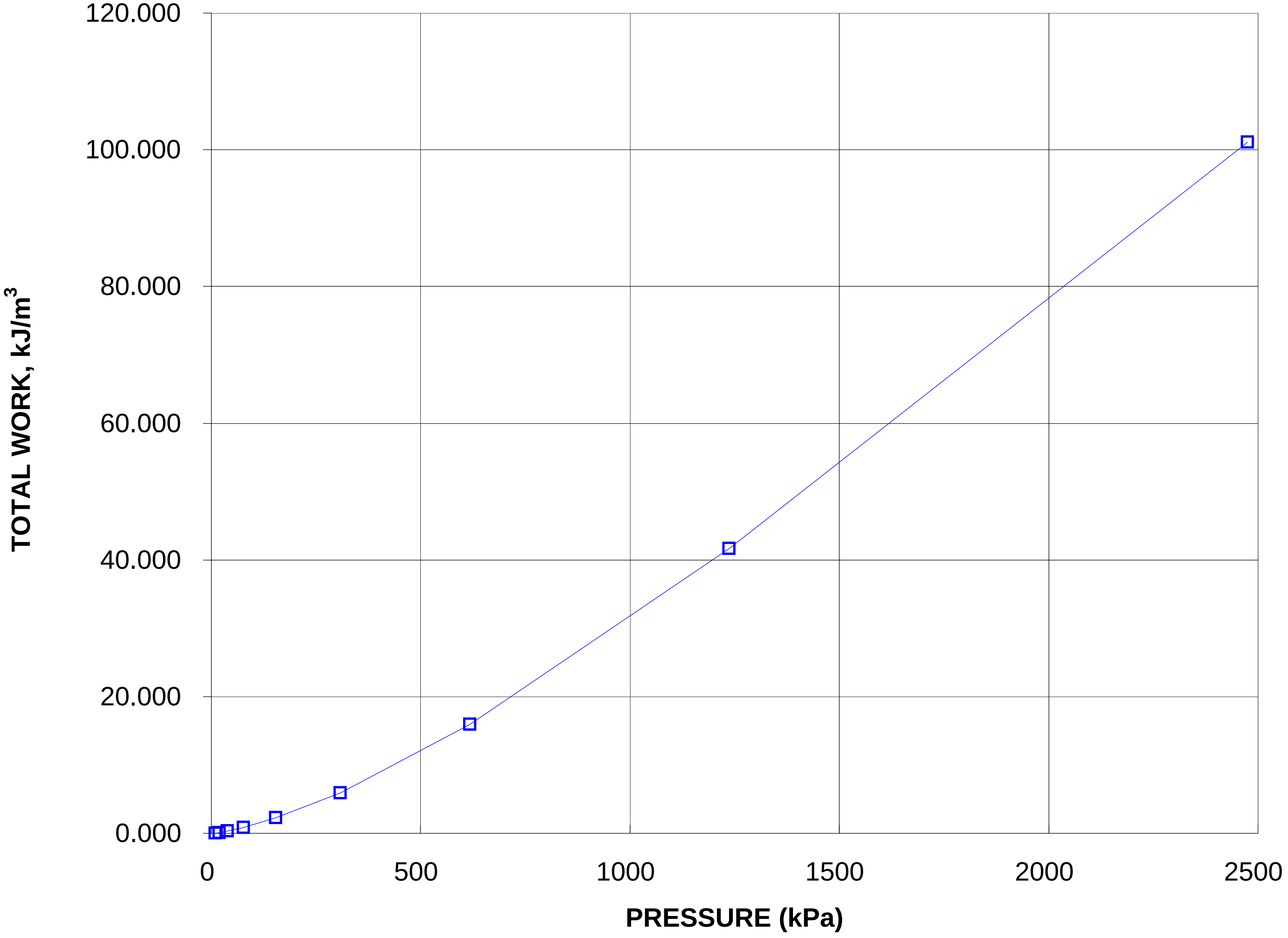
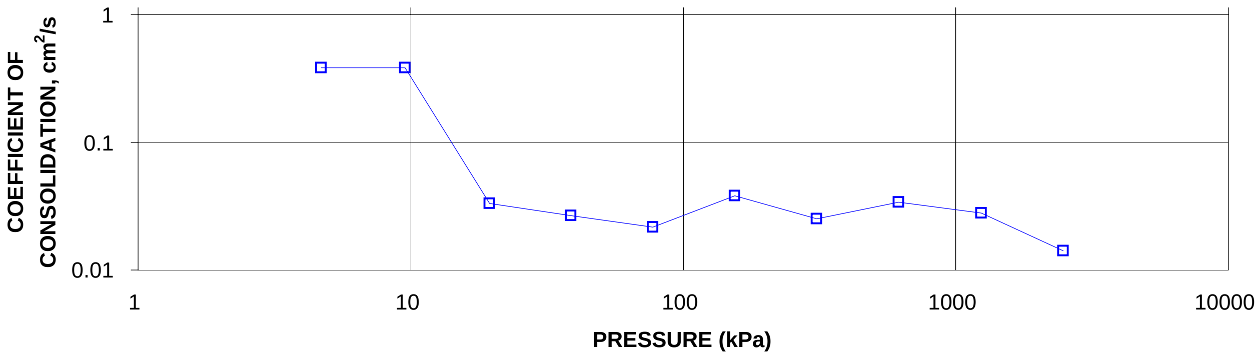
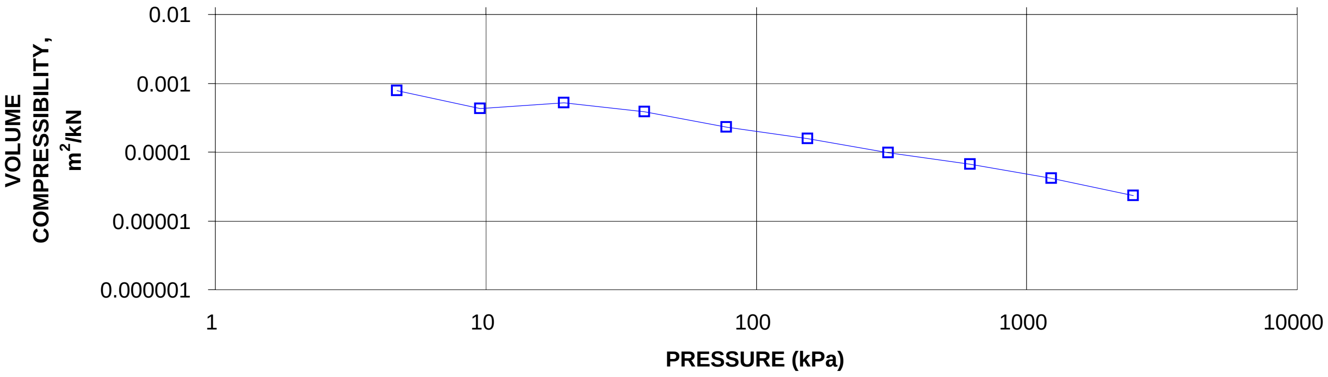


FIGURE A-3b

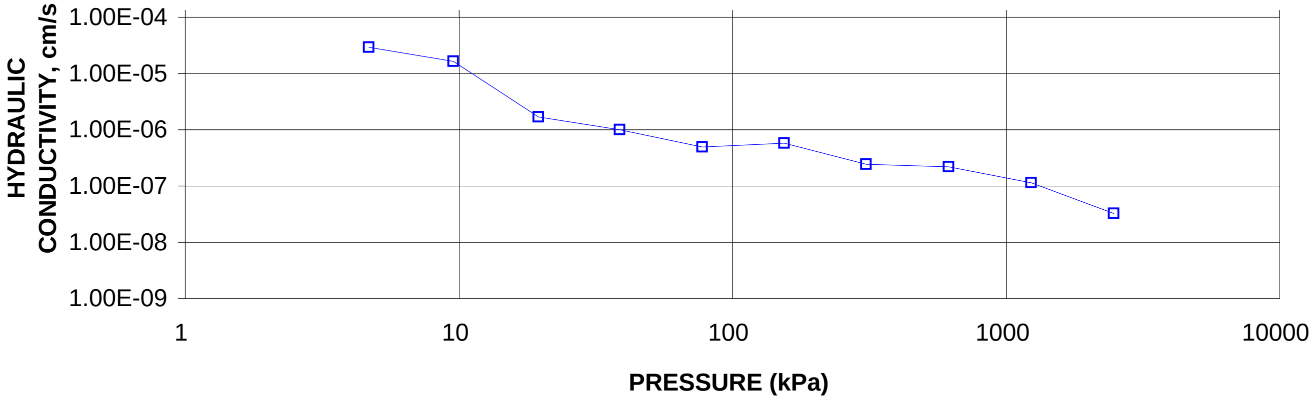
OEDOMETER CONSOLIDATION SUMMARY



CONSOLIDATION TEST
CV cm^2/s VS PRESSURE (kPa)
BH 06-26 SA 1



CONSOLIDATION TEST
MV m^2/kN vs PRESSURE (kPa)
BH 06-26 SA 1



CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH 06-26 SA 1

Project No. 06-1111-001
Prepared By: LFG

Golder Associates

Checkey By: MM

OEDOMETER CONSOLIDATION SUMMARY

SAMPLE IDENTIFICATION

Project Number	06-1111-001	Sample Number	1
Borehole Number	06-26	Sample Depth, m	2.3-2.9

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	5		
Date Started	05/24/2006		
Date Completed	06/03/2006		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.91	Unit Weight, kN/m^3	19.21
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	14.91
Area, cm^2	31.65	Specific Gravity, assumed	2.70
Volume, cm^3	60.45	Solids Height, cm	1.075
Water Content, %	28.90	Volume of Solids, cm^3	34.03
Wet Mass, g	118.43	Volume of Voids, cm^3	26.42
Dry Mass, g	91.88	Degree of Saturation, %	100.5

TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s
0.00	1.910	0.776	1.910				
4.70	1.903	0.770	1.907	2	3.85E-01	7.80E-04	2.94E-05
9.54	1.899	0.766	1.901	2	3.83E-01	4.33E-04	1.62E-05
19.51	1.889	0.757	1.894	23	3.31E-02	5.25E-04	1.70E-06
38.71	1.875	0.744	1.882	28	2.68E-02	3.82E-04	1.00E-06
77.44	1.858	0.728	1.867	34	2.17E-02	2.30E-04	4.89E-07
154.78	1.835	0.707	1.847	19	3.80E-02	1.56E-04	5.80E-07
308.76	1.806	0.680	1.821	28	2.51E-02	9.86E-05	2.42E-07
618.08	1.767	0.643	1.787	20	3.38E-02	6.60E-05	2.19E-07
1237.03	1.718	0.598	1.743	23	2.80E-02	4.14E-05	1.14E-07
2475.45	1.663	0.547	1.691	43	1.41E-02	2.33E-05	3.21E-08
1237.03	1.674	0.557	1.669				
308.76	1.692	0.574	1.683				
77.44	1.709	0.589	1.701				
19.51	1.726	0.605	1.718				
4.70	1.746	0.624	1.736				

Note:
k calculated using cv based on t_{90} values.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.75	Unit Weight, kN/m^3	20.30
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	16.31
Area, cm^2	31.65	Specific Gravity, assumed	2.70
Volume, cm^3	55.26	Solids Height, cm	1.075
Water Content, %	24.50	Volume of Solids, cm^3	34.03
Wet Mass, g	114.39	Volume of Voids, cm^3	21.23
Dry Mass, g	91.88		

Prepared By: LFG

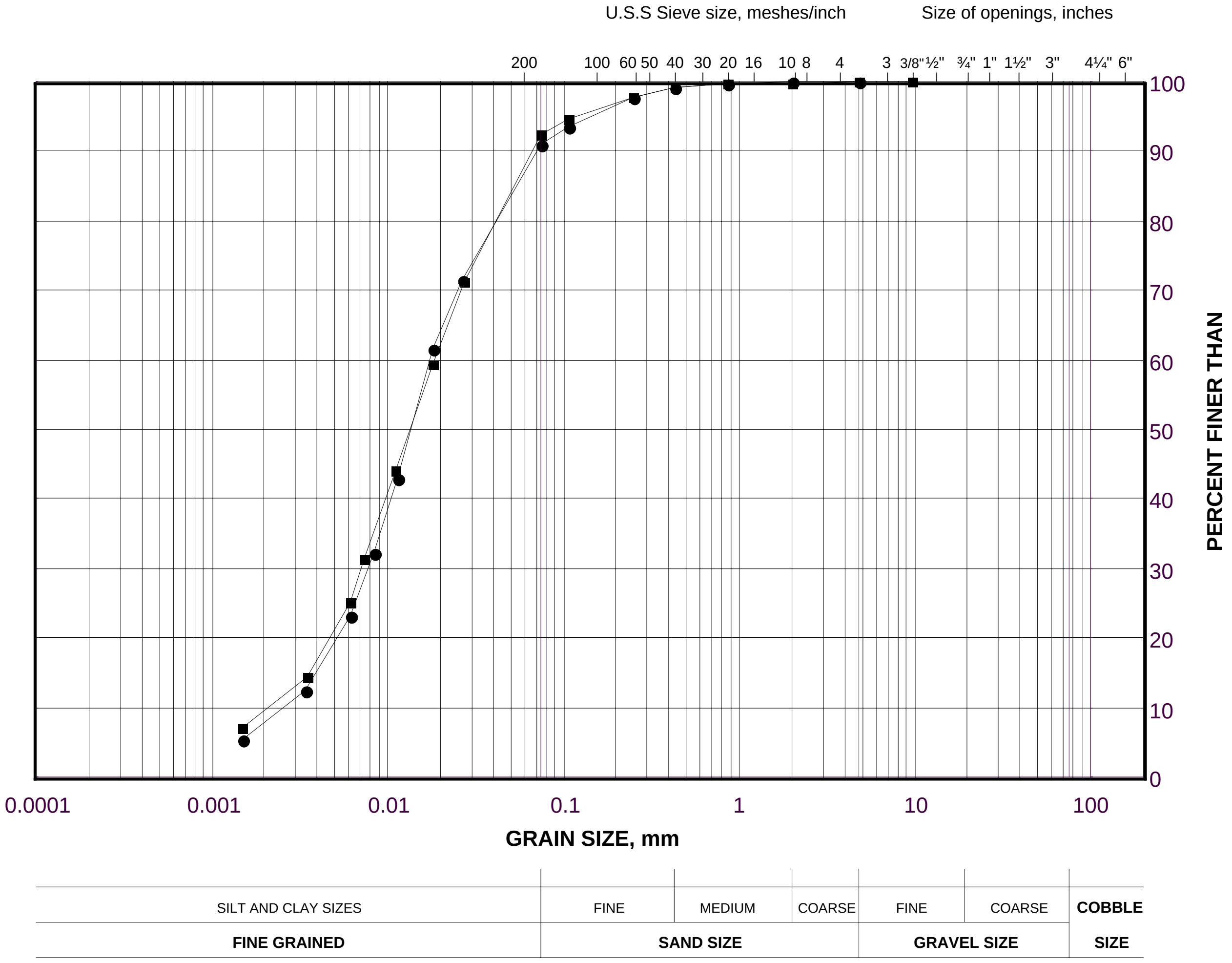
Golder Associates

Checked By: MM

GRAIN SIZE DISTRIBUTION

Silt

FIGURE A-4



LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	06-11	7	6.1-6.7
■	06-13	5	3.8-4.4

Project Number: 06-1111-001

Checked By: ____________________

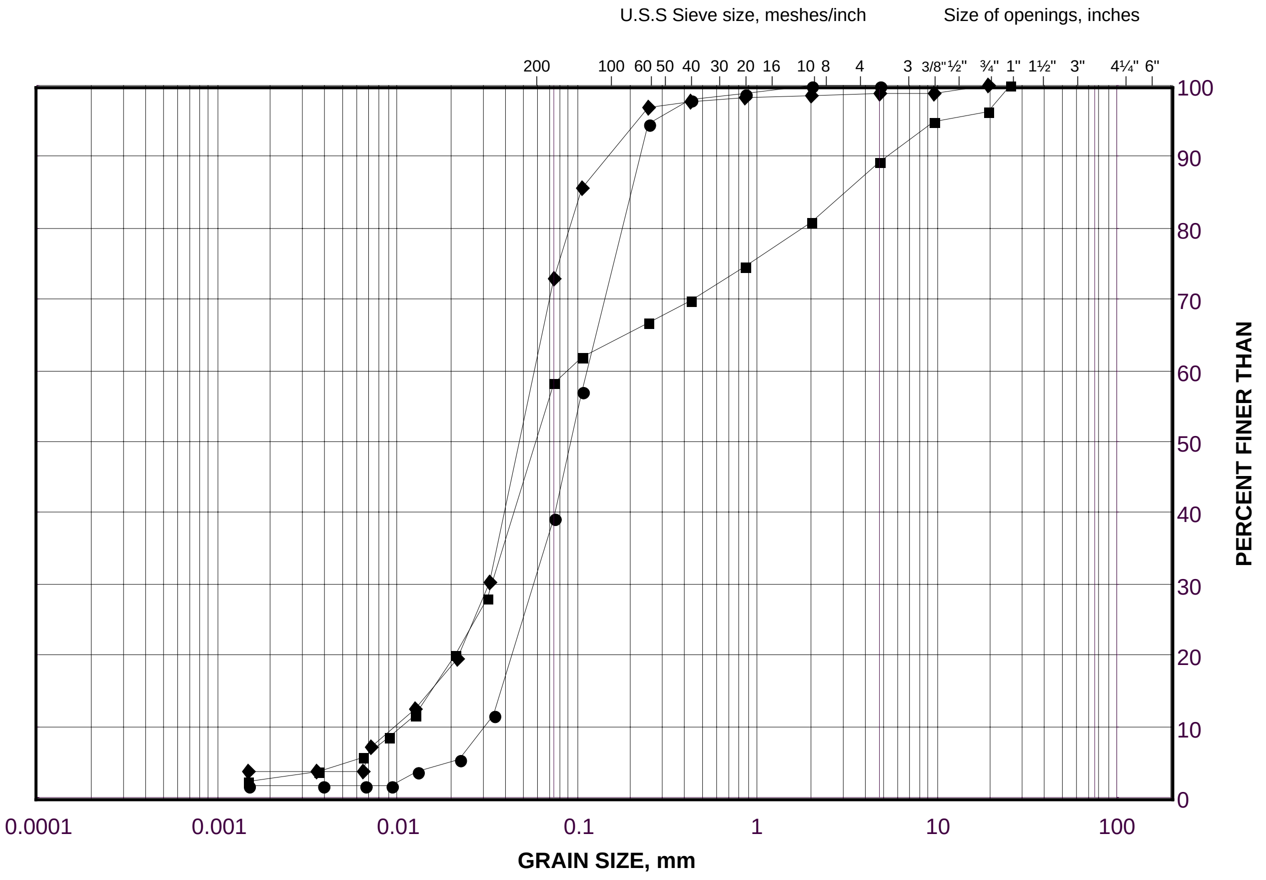
Golder Associates

Date: 13-Jun-06

GRAIN SIZE DISTRIBUTION

Sand and Silt to Sandy Silt

FIGURE A-5



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	06-3	3	1.5-2.1
■	06-7	5	3.8-4.4
◆	06-12A	5	9.1-9.8

Project Number: 06-1111-001

Checked By: ____________________

Golder Associates

Date: 13-Jun-06

APPENDIX B

NON-STANDARD SPECIAL PROVISIONS

CSP FOR INTEGRAL ABUTMENTS – Item No.

Special Provision

SCOPE

This specification covers the requirements for the installation of the corrugated steel pipes (CSPs) at the integral abutments.

SUBMISSION AND DESIGN REQUIREMENTS

All submissions shall bear the seal and signature of an Engineer.

At least two weeks prior to commencement of installation of the abutment piles, the Contractor shall submit to the Contract Administer, for information purposes only, three (3) sets of the working drawings.

The Contractor shall have a copy of the submitted working drawings on site at all times. Working drawings shall include at least the following:

1. Layout and elevations of the CSPs;
2. Location of reference points, and location of the centroid of each pile with respect to the reference points;
3. Construction sequence and details;
4. Source of the sand fill, and description of placing methods and equipment;
5. Location and details of all temporary bracing and spacers for the piles and CSPs;
6. Method for preventing water and debris from entering the CSP prior to placing sand; and
7. Method for preventing concrete from abutment pours from entering the CSPs during placement.

The Contractor shall be responsible for the complete detailed design of all temporary bracing, including spacers required to maintain the piles, CSP spacing and abutment stems in their specified positions through all stages of construction until the CSPs have been backfilled. All temporary bracing shall be removed.

MATERIAL

Corrugated Steel Pipe

CSP shall be in accordance with OPSS 1801, and shall be from a supplier listed under DSM#4.60.80. The CSP shall be of the diameter and wall thickness specified on the Contract drawings, and shall be galvanized in accordance with CSA G164-M.

CSP FOR INTEGRAL ABUTMENTS – Item No.

Special Provision

CSPs shall be supplied in the lengths and with the end treatments, either square or skew, as specified on the Contract drawings; field cutting and splicing of CSPs will not be permitted. Cut ends shall be neat and free of burrs. The planes defined by the end treatments of each CSP shall be parallel to each other.

Handling and storage of CSPs shall be in accordance with the manufacturer’s recommendations. Damaged CSPs shall be rejected. Localized areas of damaged galvanizing on otherwise acceptable CSPs shall be repaired with two coats of zinc-rich paint.

Sand Fill

The sand fill for backfilling the CSP shall meet the gradation requirements of Table 1 below:

Table 1 – Sand Fill Gradation Requirements

MTO Sieve Designation		Percentage Passing by Mass
2 mm	#10	100%
600 µm	#30	80% to 100%
425 µm	#40	40% to 80%
250 µm	#60	5% to 25%
150 µm	#100	0% to 6%

CONSTRUCTION

The sequence of construction shall be in accordance with the working drawings and as follows, unless otherwise approved:

1. Form concrete levelling pad and place CSPs and spacers.
2. Construct concrete levelling pads.
3. Place loose sand into 600 diameter CSP.
4. Install piles by driving to bedrock.
5. Remove temporary spacers.

The CSP shall be positioned such that the piles are centrally positioned within the CSP. Temporary blocking and bracing shall be used to hold the CSP in position.

The Contractor shall ensure the full perimeter of the tops of all CSPs at each abutment are at the elevation and orientation shown on the working drawings.

CSP FOR INTEGRAL ABUTMENTS – Item No.

Special Provision

The CSP at each pile shall be constructed to the following tolerances:

Criteria	**Tolerance**
Maximum deviation of CSP from pile centroid	**+/- 50 mm**
Maximum deviation of any point on the top perimeter of the CSP from the specified elevation	**+/- 10 mm**

The sand fill shall be placed dry of optimum and free-flowing, completely filling the volume between the CSP and pile. No additional compaction effort other than the action of placing the sand itself shall be applied to the sand fill.

The placing of the sand fill shall be carried out in a manner such as to not damage and displace the CSP.

BASIS OF PAYMENT

Payment at the contract price for the above tender item shall include all labour, equipment and material required to do the work.