FOUNDATION INVESTIGATION AND DESIGN REPORT
HARRIS LAKE ROAD UNDERPASS
HIGHWAY 69 FOUR-LANING
FROM THE SOUTH JUNCTION OF HIGHWAY 529,
NORTHERLY 15 KM
W.P. 5202-06-01, SITE No. 44-450
G.W.P. 5076-06-00
NORTH SECTION – NAISCOOT LAKE TO NORTH PROJECT LIMIT
DISTRICT 54, SUDBURY

Geocres Number: 41H-98

Report to

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the location of a proposed bridge carrying Harris Lake Road over four-laned Highway 69 in the Township of Wallbridge, Ontario. The proposed bridge is part of the four-laning of Highway 69 from the south junction of Highway 69 and Highway 529 northerly for 15 km.

The purpose of this investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan, records of boreholes, stratigraphic profile and cross-sections, laboratory test results and written descriptions of the subsurface conditions. A model of the subsurface conditions was developed from the data obtained in the course of the investigation.

Thurber carried out the investigation as a sub-consultant to MMM Group Limited (MMM), under the Ministry of Transportation Ontario (MTO) Agreement Number 5006-E-0030.

2 SITE DESCRIPTION

The site of the proposed structure lies approximately 315 m north of the existing Harris Lake Road and Highway 69 intersection in the Township of Wallbridge, Ontario.

The site lies across a tributary of the Harris River which must be realigned prior to construction of the bridge. Lands surrounding the site are generally undeveloped, forested and contain open swamps. Bedrock outcroppings are visible within the site.

At the time of investigation, the locations of the proposed pier, east abutment and east approach, were flooded due to the presence of beaver dams. The standing water was up to 1.2 m deep.

Photographs in Appendix G show the general nature of the site.

The site lies within the physiographic region known as the Georgian Bay Fringe, which covers Parry Sound and Muskoka. The region is characterized by very shallow overburden and bare rock knobs and ridges. Bedrock is exposed in many areas and intermittent depressions were filled in when glacial lake Algonquin inundated the area. The overburden materials consist of sand, silt and clay. Recent organic deposits of peat and muck occur in abundance in the bedrock hollows and valleys.

The area is underlain by strongly foliated and highly to moderately deformed rocks of Precambrian age of the following types:

- Gneisses of metasedimentary origin.
- Migmatitic rocks and gneisses.
- Felsic igneous rocks (tonalite, granodiorite, monzonite, granite, syenite, derived gneisses).
- Tectonite unit (tectonites, various gneisses).

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project was carried out in two phases.

Phase 1 was carried out from August 21 to 24, 2009. Phase 1 consisted of drilling and sampling eight boreholes (numbered HLR09-01 to HLR09-06, HLR09-19 and HLR09-20) near the west abutment and west approach. Boreholes were terminated upon refusal on bedrock at depths ranging from 2.4 m to 9.2 m (elevations 182.3 to 189.6). Boreholes HLR09-01, HLR09-04 and HLR09-05 were further advanced into the bedrock by coring to 11.8 m, 5.6 m and 8.6 m depth (elevations 180.2, 186.3 and 183.2), respectively. The boreholes were supplemented by dynamic cone penetration testing (DCPT) conducted adjacent to selected borehole. Ten additional DCPT's (numbered DHL-01 to DHL-10) were conducted within the west approach and west abutment area, approximately 1.0 m to 3.0 m away from the boreholes.

Due to flooded conditions at the proposed pier, east abutment and east approach areas, Phase 2 was carried out from March 1 to 6, and 10 2010, when the standing water was frozen. Phase 2 consisted of drilling and sampling 14 boreholes (identified as HLR09-07 to HLR09-18, HLR09-21 and HLR09-22) near the proposed pier, east abutment and east approach. Borehole advancement within the overburden generally ranged from 0.9 m to 5.9 m (elevations 185.4 to 190.9), where auger refusal was encountered. Boreholes HLR09-07 to HLR09-09, HLR09-12, HLR09-15, HLR09-17 and HLR09-18 were further advanced into the bedrock by coring to depths ranging from 4.6 m to 9.3 m (elevations 182.1 to 187.3). Bedrock was observed surficially in Boreholes HLR09-13 and HLR09-14.

The approximate borehole locations are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix H.

The borehole locations were marked in the field and utility clearances were obtained prior to drilling.

Phase 1 drilling was carried out using a track mounted CME 55 drill rig. A combination of hollow-stem auger drilling techniques and NQ coring methods were used to advance the boreholes.

For Phase 2 program, a tripod rig (light portable drilling equipment, powered by a Hilti DD-250) using casing and wash boring methods as well as coring equipment were employed at boreholes located at the pier and east abutment/approach, where work was conducted from the ice surface.

Overburden samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). NQ rock coring equipment was used to recover core samples of the underlying bedrock in selected boreholes. A minimum 3.0 m of rock cores were recovered in the selected boreholes.

Visual assessment was used at locations where exposed bedrock was encountered.

The drilling and sampling operations were supervised on a full time basis by a member of Thurber's technical staff. The supervisor logged the boreholes and processed the recovered soil and rock samples for transport to Thurber's laboratory for further examination and testing.

All rock cores were logged, and the Total Core Recovery (TCR), Rock Quality Designation (RQD) and the Fracture Indices (FI) were determined.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. Four standpipe piezometers consisting of 19 mm PVC pipe with slotted screens were installed and enclosed in filter sand to permit longer term groundwater level monitoring. The locations and completion details of the piezometers are shown in Table 3.1.

Table 3.1 – Borehole Completion Details

Foundation Unit	Borehole	Piezometer Tip Depth/ Elevation (m)	Completion Details
West Approach	HLR09-19	None installed	Borehole backfilled with holeplug to 3.0 m then auger cuttings to surface.
	HLR09-20	None installed	Borehole backfilled with holeplug to 1.2 m then auger cuttings to surface.
West Abutment	HLR09-01	11.7/180.2	Sand from 11.7 m to 8.5 m, holeplug from 8.5 m to 6.4 m, sand from 6.4 m to 0.6 m, then auger cuttings to surface.
	HLR09-02	None installed	Borehole backfilled with holeplug to 1.2 m then sand to surface.
	HLR09-03	None installed	Borehole backfilled with holeplug to 3.7 m then auger cuttings to surface.
	HLR09-04	None installed	Borehole backfilled with holeplug to 2.1 m then sand to surface.
	HLR09-05	None installed	Borehole backfilled with holeplug to 0.9 m then auger cuttings to surface.
	HLR09-06	None installed	Borehole backfilled with holeplug to 4.6 m then sand to surface.
Pier	HLR09-07	5.3/186.6	Sand from 5.3 m to 1.0 m, then holeplug to surface.
	HLR09-08 HLR09-09 HLR09-10 HLR09-11 HLR09-12	None installed	Borehole backfilled with holeplug to surface.
East Abutment	HLR09-15	None installed	Borehole backfilled with holeplug to surface.
	HLR09-16	None installed	Borehole backfilled with holeplug to 0.6 m
	HLR09-17	7.8/183.7	Sand from 7.8 m to 6.1 m, holeplug from 6.1 m to ground surface.
	HLR09-18	None installed	Borehole backfilled with holeplug to surface.
East Approach	HLR09-21 HLR09-22	None installed	Borehole backfilled with holeplug to surface.

4 LABORATORY TESTING

All recovered soil samples were subjected to Visual Identification (VI) and rock core samples to geological logging. Moisture content determinations were carried out on all soil samples. Selected samples were also subjected to grain size distribution analyses (sieve and hydrometer) and Atterberg Limits testing where appropriate. The results of this testing program are summarized on the Record of Borehole sheets in Appendix A and on the figures presented in Appendix B.

Point load tests were carried out in the laboratory on selected samples of intact bedrock to assist in evaluation of the compressive strength of the bedrock. Results of point load tests on the selected rock core samples are shown in Point Load Test Sheets included in Appendix B and on the Record of Borehole sheets in Appendix A.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets in Appendix A. Details of the encountered soil and rock stratigraphy are presented in these sheets and on the "Borehole Locations and Soil Strata" drawing in Appendix H. An overall description of the stratigraphy is given in the following paragraphs. However, the factual data presented in the Record of Borehole Sheets governs any interpretation of the site conditions.

In general terms, the overburden encountered at the west abutment and west approach consists of fibrous peat overlying native sand to silty sand. Layers of clayey silt were encountered within the sand/silty sand. Moderately weathered to fresh, grey granitic gneiss bedrock was contacted below the sand/silty sand layers, at depths ranging from 2.4 m to 9.2 m.

At the proposed pier, east abutment and east approach locations, the site was flooded due to the presence of beaver dams. The investigation was carried out while the water surface was frozen, and the boreholes were advanced through 0.3 to 1.5 m depth of ice and water. The stratigraphy encountered below the water generally comprised a relatively thin layer of peat overlying layers of native clayey silt, sand and silt. Granitic gneiss bedrock was encountered below the sand or silt, generally at depths ranging from 0.9 m to 5.9 m. Bedrock was exposed at surface on the north side of the east abutment.

More detailed descriptions of the individual strata are presented below.

5.1 Peat

Dark brown fibrous peat was contacted surficially in boreholes drilled at the west abutment and west approach and below the ice/water in boreholes drilled at the pier, east abutment and east approach. Peat was not encountered in Boreholes HLR09-13 and HLR09-14.

The thickness of the peat varied from 35 mm to 800 mm. The peat thickness may vary between and beyond the borehole.

5.2 Sand to Silty Sand

Native brown to grey sand containing trace silt to silty, trace to some gravel and trace clay was contacted below the peat and also below the native clayey silt at depths and elevations indicated in Table 5.1.

Table 5.1 – Depths and Elevations of Sand/Silty Sand

Foundation Unit	Borehole	Depth below existing ground surface (m)	Elevation (m)	Thickness (m)
West Approach	HLR09-19	0.035 to 9.2	191.5 to 182.3	9.2
	HLR09-20	0.035 to 1.3 2.1 to 8.2	192.0 to 190.7 189.8 to 183.8	1.3 6.1
West Abutment	HLR09-01	0.03 to 1.1 2.4 to 7.2	191.9 to 190.8 189.6 to 184.8	1.1 4.8
	HLR09-02	0.05 to 1.1 2.6 to 5.5	192.0 to 190.9 189.4 to 186.5	1.1 2.9
	HLR09-03	0.1 to 1.3 2.3 to 5.2	191.8 to 190.6 189.6 to 186.7	1.2 2.9
	HLR09-04	0.1 to 2.4	191.9 to 189.6	2.3
	HLR09-05	0.05 to 1.3 2.0 to 5.4	191.8 to 190.5 189.8 to 186.3	1.3 3.4
	HLR09-06	0.035 to 0.9 2.4 to 5.2	191.8 to 190.9 189.5 to 186.6	0.9 2.8
Pier	HLR09-07	0.5 to 0.9 1.3 to 1.7	191.4 to 191.0 190.6 to 190.2	0.4 0.4
	HLR09-08	0.5 to 0.9	191.3 to 190.9	0.4
	HLR09-09	0.6 to 5.9	190.7 to 185.4	5.3
	HLR09-10	0.7 to 1.0 2.4 to 4.3	190.8 to 190.5 189.1 to 187.2	0.3 1.9
	HLR09-11	1.5 to 4.7	190.0 to 186.8	3.2
	HLR09-12	1.7 to 2.5	189.8 to 189.0	0.8
East Abutment	HLR09-15	1.2 to 1.7 2.4 to 4.8	190.2 to 189.7 189.1 to 186.6	0.5 2.4
	HLR09-16	0.7 to 1.8	191.0 to 189.9	1.1
	HLR09-17	0.9 to 2.0 4.0 to 5.0	190.6 to 189.5 187.5 to 186.5	1.1 1.0
East Approach	HLR09-21	0.5 to 3.5	191.2 to 188.3	3.0
	HLR09-22	1.6 to 5.1	190.0 to 186.6	3.5

Occasional cobbles and bedrock fragments were encountered in some boreholes near the base of the sand/silty sand layer.

Boreholes presented in Table 5.1 were terminated in the sand to silty sand layer upon auger refusal on bedrock, except Borehole HLR09-16.

Standard Penetration tests in the sand layer gave SPT N-values generally in the range of 0 to 29 blows per 0.3 m of penetration, indicating a very loose to compact relative density. An SPT N-value of 33 blows per 0.3 m of penetration, indicating a dense relative density, was measured in Borehole HLR09-21, drilled at the east abutment.

The moisture contents of samples from the sand/silty sand layer generally vary between 8% and 31%. Moisture contents ranging from 35% to 42% were measured in Boreholes HLR09-07, HLR09-10, HLR09-15 and HLR09-16 at depths ranging from 0.6 m to 1.2 m.

Grain size distribution curves for the sand and silty sand samples tested are presented in Appendix B, Figures B1 to B5. The results are also summarized on the Record of Borehole sheets in Appendix A. The results of grain size distribution test are summarized as follows:

Soil Particles	Sand (%)	Silty Sand (%)
Gravel	0 to 12	0 to 6
Sand	67 to 99	41 to 80
Silt	-	22 to 51
Clay	-	2 to 9
Silt & Clay	1 to 27	20 to 52

5.3 Clayey Silt

Native brown to grey clayey silt containing some sand to sandy and occasional rootlets was contacted within the sand layer in Boreholes HLR09-01 to HLR09-03, HLR09-05 to HLR09-07, HLR09-10, HLR09-15 to HLR09-17 and HLR0-20, at depths ranging from 0.9 m to 2.0 m (elevations 189.5 to 191.0).

The thickness of the clayey silt layer ranged from 0.4 m to 2.0 m.

The depth to the base of the clayey silt ranged from 1.3 m to 4.0 m (elevations187.5 to 190.6).

SPT N-values in the clayey silt ranged from 1 to 13 blows per 0.3 m of penetration, indicating a very soft to stiff consistency. SPT N-values of 20 and 23 blows per 0.3 m of penetration, indicating a very stiff consistency, were measured in Boreholes HLR09-07 and HLR09-17. Moisture contents ranged from 17% to 30%.

Grain size distribution curves for selected clayey silt samples are presented in Appendix B, Figure B7. The results are also summarized on the Record of Borehole sheets included in Appendix A. Atterberg Limits test results are presented in Figure B8 of Appendix B. The results of the laboratory tests are summarized as follows:

Soil Particles	Clayey Silt (%)
Gravel	0 to 5
Sand	22 to 63
Silt	56 to 63
Clay	12 to 15

Liquid Limit	21 to 22
Plastic Limit	15 to 16

The above results show that the clayey silt is of low plasticity with a group symbol of CL- ML.

5.4 Silt

Native dark brown to grey silt containing some sand to sandy, trace clay and occasional rootlets was contacted at 2.4 m and 0.8 m depth (elevations 189.3 and 190.7) in Boreholes HLR09-16 and HLR09-18, respectively.

The thickness of the silt layer was 1.6 m and 3.1 m.

Boreholes HLR09-16 and HLR09-18 were terminated in the silt layer at 4.0 m and 3.9 m (elevations 187.7 and 187.6), upon auger refusal on bedrock.

SPT N-values in the silt ranged from 0 to 20 blows per 0.3 m of penetration, indicating a very loose to compact relative density. Moisture contents ranged from 10% to 41%.

Grain size distribution curves for selected silt samples are presented in Appendix B, Figure B6. The results are also summarized on the Record of Borehole sheets included in Appendix A. The results of the laboratory tests are summarized as follows:

Soil Particles	Silt (%)
Gravel	0
Sand	26 to 27
Silt	69 to 71
Clay	3 to 4

5.5 Bedrock

The overburden soils described above are underlain by granitic gneiss bedrock. The bedrock is moderately weathered to fresh. The bedrock was generally grey with occasional pink and white bands visible in most cores. Occasional mechanical breaks and sub-vertical fractures were observed in the rock cores.

Bedrock was encountered at various depths and it was proved by coring near the locations of the proposed abutments and pier. Table 5.2 summarizes depths and elevations to the top of bedrock in the boreholes and DCPTs. Where coring was not carried out, bedrock was inferred from auger refusal.

Core recovery in the bedrock generally ranged from 63% to 100%, with the exception of 34% recovery in Borehole HLR09-01 Run 1. The RQD values generally ranged from 60% to 100% indicating fair to excellent rock quality. RQD values of 0%, indicating a very poor rock quality, were observed in Boreholes HLR09-01 Run 1, HLR09-08 Run 1 and HLR09-09 Run 1. An RQD value of 41% was noted in Borehole HLR09-09 Run 2, indicating a poor rock quality.

The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, generally ranged from 0 to 5 in most of the cores. In some cores the FI was greater than 10.

The estimated uniaxial compressive strength of the rock cores generally ranges from 74 MPa to 248 MPa, indicating a strong to very strong rock. Lower uniaxial compressive strength values of 5 MPa, 21 MPa and 37 MPa were measured on cores from Boreholes HLR09-01 and HLR-09-04, indicating the rock to be weak to medium strong. Uniaxial compressive strengths of 259 MPa to 266 MPa, indicating an extremely strong rock, were measured in Boreholes HLR09-04 and HLR-09-12.

These estimated rock strength values are interpreted from point load tests that were conducted on rock cores recovered from the boreholes. A summary of the Point Load Test Results is presented in Table 1 immediately following the text of this report.

Table 5.2 – Depths and Elevations of Top of Bedrock

Foundation Unit	Borehole/ DCPT	Top of Bedrock	
		Depth (m)	Elevation (m)
West Approach	HLR09-19	9.2	182.3
	HLR09-20	8.2	183.8
West Abutment	HLR09-01	7.2*	184.8
	HLR09-02	5.5	186.5
	HLR09-03	5.2	186.7
	HLR09-04	2.4*	189.6
	HLR09-05	5.4*	186.3
	HLR09-06	5.3	186.6
	DHL-01	3.4	188.9
	DHL-02	2.0	190.2
	DHL-03	5.3	186.8
	DHL-04	8.2	184.1
	DHL-05	5.0	187.2
	DHL-06	4.1	187.9
	DHL-07	5.7	186.1
	DHL-08	6.1	186.0
	DHL-09	6.8	185.1
	DHL-10	5.7	186.0
Pier	HLR09-07	1.7*	190.2
	HLR09-08	0.9*	190.9
	HLR09-09	5.9*	185.4
	HLR09-10	4.3	187.2
	HLR09-11	4.7	186.8
	HLR09-12	2.5*	189.0
East Abutment	HLR09-13	0.0	194.3
	HLR09-14	0.0	192.8
	HLR09-15	4.8*	186.6
	HLR09-16	4.0	187.7
	HLR09-17	5.0*	186.5
	HLR09-18	3.9*	187.6
East Approach	HLR09-21	3.5	188.3
	HLR09-22	5.1	186.6

* Bedrock proved by coring

5.6 Water Levels

Water levels were observed in the boreholes during and upon completion of drilling. Three standpipe piezometers were installed to monitor water levels after completion of drilling. The water levels measured in the piezometers and open boreholes are summarized in Table 5.3.

Table 5.3 – Water Level Measurements

Location	Borehole	Date	Water Level (m)		Comment
			Depth	Elevation	
West Approach	HLR09-19	August 20, 2009	Ground surface	191.5	Open borehole
	HLR09-20	August 2, 2009	0.3	191.7	Open borehole
West Abutment	HLR09-01	August 25, 2009 September 14, 2009 September 24, 2009 October 7, 2009 October 27, 2009 November 20, 2009	0.6 0.3 0.5 0.5 0.4 0.8	191.3 191.6 191.4 191.4 191.5 191.1	In piezometer
	HLR09-06	August 22, 2009	0.4	191.4	Open borehole
Pier	HLR09-07	March 5, 2010	0.0		Water at surface
	HLR09-08	March 4 and 5, 2010	0.0	-	Water at surface
	HLR09-09				
	HLR09-10	March 6, 2010	0.5	191.0	Open borehole
	HLR09-11	March 4 and 10, 2010	0.0	-	Water at surface
	HLR09-12				
East Abutment	HLR09-15	March 2 and 3, 2010	0.0	-	Water at surface
	HLR09-16				
	HLR09-17				
	HLR09-18				
East Approach	HLR09-21	March 1, 2010	0.0	-	Water at surface
	HLR09-22				

The piezometric readings at the west abutment indicate that the groundwater levels range from Elevations 191.1 to 191.6. At the east abutment and pier, ponded water was present.

The above values are short-term readings and seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall.

6 MISCELLANEOUS

Borehole locations were selected by Thurber Engineering Ltd. Surveyors from MMM Group Limited staked these locations in the field, confirmed the co-ordinates and obtained the ground surface elevations.

Thurber obtained utility clearances for the borehole locations prior to drilling.

Eastern Ontario Diamond Drilling Ltd. from Hawkesbury, Ontario supplied a track mounted CME 55 drill rig and conducted the drilling, sampling and in-situ testing operations.

OGS Drilling Inc. of Almonte, Ontario supplied the portable drilling/coring equipment to drill and core boreholes that were not accessible using a track mounted rig.

The field program was supervised by Ms. Eckie Siu, Mr. Jason Mei and Mr. Stephane Loranger of Thurber.

Routine laboratory testing was carried out by Thurber Engineering Ltd.

Overall supervision of the field program, interpretation of the data and preparation of the report were carried out by Mr. Alastair E. Gorman, P.Eng and Ms. R. Palomeque Reyna, P.Eng.

The report was reviewed by Dr. P.K. Chatterji, P.Eng. a Designated Principal Contact for MTO Foundations Projects.

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NORTH SECTION – NAISCOOT LAKE TO NORTH PROJECT LIMIT

DISTRICT 54, SUDBURY

Geocres Number: 41H-98

PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 GENERAL

This report presents interpretation of the geotechnical data in the factual report and presents geotechnical design recommendations to assist the design team to select and design a suitable foundation system and approach embankments for the proposed Harris Lake Road underpass structure over four-laned Highway 69 in the Township of Wallbridge, Ontario.

Currently, Highway 69 is a two lane undivided roadway. At this site, the project involves widening of Highway 69 from a two-lane undivided roadway to a four-lane divided highway. The planned four-lane alignment will run roughly parallel to the existing alignment. The existing highway will become the SBL and the new NBL will be constructed between the pier and the east abutment of the proposed underpass.

Based on the preliminary General Arrangement (GA) drawing provided by MMM Group Limited, a double-span structure supported on two abutments and one pier is proposed. The total length of the structure will be 85.8 m, with 38.4 m between the west abutment and pier, and 47.4 m between the pier and the east abutment. The proposed structure will be approximately 12.0 m wide.

The proposed finished grade of Harris Lake Road at the structure will be at about elevation 203.7 m at the west abutment. The original ground surface is near elevation 191.9 m, resulting in an approach embankment, 11.8 m high. At the east abutment, the finished grade will be at elevation 203.6 m and the original ground surface elevation (exposed bedrock and native sand below water and peat) ranged from 190.5 to 194.3. The east approach embankment will be up to 13.1 m high.

At the pier location, 4.5 m to 6.0 m of fill will be placed over the existing ground surface to achieve the design highway grade.

The proposed grade of Highway 69 NBL and SBL will be near elevation 196.7.

The discussion and recommendations presented in this report are based on the information provided by MMM Group Ltd. and on the factual data obtained in the course of the investigations.

8 STRUCTURE FOUNDATIONS

In general terms, the overburden encountered at this site consists of a relatively thin layer of fibrous peat, overlying native layers of very loose to compact sand and silt with very soft to stiff clayey silt layer. A 0.3-m to 1.5-m depth of standing water and ice was encountered surficially at the locations of the pier and east abutment. Moderately weathered to fresh, grey granitic gneiss bedrock was contacted below the native sand and silt layers at depths ranging from 2.4 m to 7.2 m at the west abutment, from 0.9 m to 5.9 m at the pier and from 0.0 m to 5.0 m at the east abutment. The top of bedrock slopes sharply across the proposed foundation elements.

The piezometric readings at the west abutment indicate that the groundwater levels range from Elevations 191.1 to 191.6. At the east abutment and pier, water was ponded at the surface.

Initial consideration was given to the following foundation types:

- Spread footings on native soils
- Spread footings on bedrock
- Augered Caissons (drilled shafts)
- Driven steel H-piles

A comparison of the foundation alternatives based on advantages and disadvantages of each one is included in Appendix C.

8.1 Spread Footings on Native Soils

Spread footings founded on native soils are not a feasible foundation option at this site due to the following reasons:

- Low geotechnical capacities are present at this site in the loose native sands and silts. In fact, much of the overburden soils above the bedrock have low geotechnical capacity.

- Relatively large settlements under footing loads will occur if footings are placed on the native soils.

- Groundwater levels at the site are high at the proposed west abutment and the proposed pier and east abutment locations are currently flooded. Unwatering/groundwater control will be difficult for construction of footings.

- Sloping bedrock exists at the each foundation element and the bedrock is exposed on the north side of the east abutment. This precludes consideration of spread footings on native soils.

Due to the above factors, spread footings on native soils are not recommended at this site.

8.2 Spread Footings on Bedrock

At the foundation elements, relatively deep excavation will be required to found the footings on bedrock. The variable depths of excavation to the top of bedrock at each foundation are:

West abutment - 2.4 m to 7.2 m
Pier - 0.9 m to 5.9 m
East abutment - 0.0 m to 5.0 m, bedrock was contacted at surface at two borehole locations

Footing excavation will extend in cohesionless soils below the groundwater level which will require temporary cofferdam enclosure and significant dewatering measures.

In addition, the bedrock surface is sloping and generally dips steeply to the west at the west abutment, and to the south at the pier and east abutment. Preparation of a level founding surface by stepping the bedrock and placing mass concrete will be necessary after excavation of the overburden soils.

In light of the need for relatively deep excavation to reach bedrock and the difficulty anticipated in dewatering in the cohesionless overburden, spread footings on bedrock at the abutments and pier are not recommended at this site.

8.3 Augered Caissons (drilled shafts)

Supporting the pier on drilled shafts socketed into bedrock is considered to be technically feasible at this site.

Caissons must be socketed into the bedrock to develop the required geotechnical resistance. Table 5.2 provides depths and elevations to the top of bedrock at the pier and the abutments.

Since it may be difficult to clean and inspect the base of the caisson, the caisson must be designed on the basis of the shaft friction between the concrete in the socket and the surrounding bedrock.

At this site, the recommended value of shaft friction for SLS design is 5% of the compressive strength of the concrete used in the socket. Thus, if 30 MPa concrete is used, the value for shaft friction is 1,500 kPa. Higher values of shaft friction can be used if higher strength concrete is specified for the foundation. For the ULS condition, an adhesion 20% higher than the SLS value can be used, i.e. 1,800 kPa in this example.

Using the 1,500 kPa adhesion and a 1.2 diameter socket, the geotechnical resistance developed per metre length of socket is 5,650 kN.

The top 1.0 m of penetration into the bedrock must be ignored in resistance calculations to allow for the variations in the bedrock surface and possible, localized, surface fracturing.

The minimum spacing between the caissons should be as per CHBDC. Where sloping bedrock is encountered, the socket depth shall be measured from the lowest point where bedrock is exposed in the caisson wall. Accordingly the design caisson depth should be increased by 0.5 m to account for variation of the bedrock surface across the caisson diameter.

8.3.1 Caisson Installation

Caisson installation must be in accordance with OPSS 903, November 2009.

The Contractor must determine the details of the method of construction, having regard to his equipment and experience. However, for the purposes of design and evaluation, the following methodology is suggested:

1. Advance a steel liner with an internal diameter equal to or greater than the required diameter of the caisson shaft.
2. Drill the liner into the top of the bedrock to exclude, as much as feasible, infiltration of soil.
3. Advance a socket of the required diameter into the bedrock to the required depth.
4. Remove as much drill debris from the socket as is feasible.
5. If required, place the concrete for the caisson by tremie methods.

Unless the Contractor brings forward an alternate proposal that is acceptable to the Ministry, the methodology described above inherently requires that:

- The steel liner be left in place permanently and be cut off at the top of the caisson shaft
- That the liner be fitted with a drill shoe that will be left in place with the liner and thus will be a consumable.

8.4 Driven Piles

The abutments and pier may be supported on steel H-piles driven to bedrock.

The stratigraphy encountered at the pier and east and west abutments consists of native deposits of sand, silt and clayey silt overlying bedrock. The top of bedrock is sloping at the pier and at both abutment locations.

For driven steel H-piles, the following factors must be considered at this site at the east abutment and pier:

- The relatively shallow depth of bedrock at the pier, generally ranging from 0.9 m to 5.9 m. Furthermore, bedrock is exposed at the north side of the east abutment.
- The depth of overburden will be insufficient at some locations to provide adequate lateral stability for the piles
- Piles will be driven to sloping bedrock conditions.

For locations where the bedrock is at shallow depth, the H-piles must be socketed into the bedrock to provide lateral pile stability.

Table 8.1 below gives details on the bedrock elevations and the estimated pile lengths.

Table 8.1 – Estimated Pile Lengths (Pier and Abutment Piles)

Location	Underside of Abutment Stem /pile cap Elevation (m)	Borehole	Depth to Bedrock (m)	Top of Bedrock Elevation (m)	Estimated Length of Pile (m)
West Abutment	196.0*	HLR09-01	7.2	184.8	11.2
		HLR09-02	5.5	186.5	9.5
		HLR09-03	5.2	186.7	9.3
		HLR09-04	2.4	189.6	6.4
		HLR09-05	5.4	186.3	9.7
		HLR09-06	5.3	186.6	9.4
Pier	194*	HLR09-07	1.7	190.2	3.8**
		HLR09-08	0.9	190.9	3.1**
		HLR09-09	5.9	185.4	8.6
		HLR09-10	4.3	187.2	6.8
		HLR09-11	4.7	186.8	7.2
		HLR09-12	2.5	189.0	5.0**
East Abutment	196.8*	HLR09-13	Exposed at surface	194.3	2.5**
		HLR09-14		192.8	4.0**
		HLR09-15	4.8	186.6	10.2
		HLR09-16	4.0	187.7	9.1
		HLR09-17	5.0	186.5	10.3
		HLR09-18	3.9	187.6	9.2

* Estimated from General Arrangement Drawings.
** Overburden depth less than 6.0 m

8.4.1 Driven piles at the pier

Within the footprint of the pier where the overburden thickness is 6.0 m or more, piles may be driven to bedrock.

At locations where the overburden thickness is less than 6.0 m, the piles must be socketed a minimum of 1.0 m into bedrock to provide lateral pile stability.

Refer to Section 8.4.6 of this report for a possible construction sequence.

8.4.2 Driven piles at the abutments

For an integral abutment design, the recommended minimum length of pile below the underside of the abutment is 5.0 m

It is anticipated that on the north side of the east abutment, where bedrock is exposed, the underside of the pile cap would be at an elevation that would result in very short piles. It is recommended that bedrock be excavated to 5.0 m below abutment, holes predrilled in the bedrock and then proceed with pile installation.

For these cases, refer to Section 8.4.7 of this report.

8.4.3 Axial Resistance

The vertical, concentric, factored geotechnical resistances at Ultimate Limit States (ULS_f) for three H-pile sections bearing on bedrock are presented in Table 8.2.

Table 8.2 – Axial Resistance of Three Pile Sections Founded on Bedrock

Pile Section		
HP 310 x 110 ULS (Factored) (kN)	HP 310 x 132 ULS (Factored) (kN)	HP 310 x 152 ULS (Factored) (kN)
2,000	2,400	2,750

The SLS condition will not govern for piles founded on the bedrock.

The structural resistance of the pile should be checked by the structural designer.

Pipe piles driven to bedrock were also considered as a foundation alternative, however due to the presence of slopping bedrock and relatively short pile lengths at some locations, pipe piles are not recommended at this site.

8.4.4 Pile Tips

If driven piles are selected, the tips of all piles should be fitted with cast steel, H-section rock points from an approved manufacturer such as Titus Steel (Standard H-point) or approved equivalent.

In areas of steeply sloping bedrock, the driving energy should be reduced as required to seat the pile in bedrock and avoid sliding of the pile tip.

If the piles are placed into sockets in the bedrock, pile tips are not required.

8.4.5 Pile Installation

Driven piles must be installed in accordance with OPSS 903, November 2009.

More specific commentary on the use of piles at the pier and east abutments are provided in Sections 8.4.6 and 8.4.7.

8.4.6 Pier

For pile locations where depth of overburden is less than 6.0 m, the following possible construction sequence may be followed:

i. For each pile, drill a cased hole of suitable size to bedrock, to accept the pile and allow grouting. Drill a 600-mm diameter socket for each pile extending a minimum 1.0 m into bedrock.

ii. Clean the base of the socket by removing soil, cuttings or broken rock.

iii. Place the pile into the bottom of the cleaned socket and hold it in position by use of temporary support.

iv. Grout the pile in place in the socket using 30 MPa concrete.

v. From a geotechnical perspective, it is only necessary to ensure that the piles are grouted into the bedrock. However, since installation will take place through the water column and through very loose to compact sand and soft to stiff clayey silt, consideration should be given to grouting the piles in the casing, leaving the outer casing in place permanently.

8.4.7 Abutment

For integral abutment design, a minimum pile length of 5.0 m below the underside of the abutment is recommended.

This requirement cannot be met by the existing conditions at the east abutment, as the bedrock is exposed in Boreholes HLR09-13 and HLR09-14. For achieving pile lengths of 5.0 m, the pile tip must extend to at least elevation 191.8. Accordingly, the following possible construction sequence should be considered:

i. Expose bedrock where or if necessary near the location of Boreholes HLR09-13 and HLR09-14.

ii. For each pile, drill a socket into the bedrock to elevation 191.8 at a diameter sufficient to accept the pile and allow grouting.

iii. The base of the socket must be cleaned of soil, cuttings or broken rock.

iv. Place the pile in the socket and hold it in position by use of temporary supports.

v. Grout the pile in place in the socket using 30 MPa concrete.

vi. Backfill to the limits shown in Figure 1 in Appendix E using granular backfill and concurrently installing CSPs, if specified.

vii. Continue with the balance of construction.

8.4.8 Pile Driving

If driven piles are selected, the piles must be driven to bedrock. The appropriate pile driving note is "Piles to be driven to bedrock".

An NSSP should require the QVE to terminate driving before the pile is damaged by overdriving. Suggested texts for NSSP's are included in Appendix D.

To facilitate pile installation, embankment fill through which piles will be driven must not contain oversize material, i.e. no particles exceeding 75 mm in size.

The driving procedure for piles driven against sloping bedrock must ensure an adequate pile seating on the bedrock without damaging the pile. The pile driving procedure on sloping bedrock is described in OPSS 903.07.02.07.03.03.

8.4.9 Downdrag

Downdrag on the piles is not considered to be an issue at this site.

8.5 Abutment Design Considerations

The ground conditions at this site are considered suitable for conventional or semi-integral abutment design.

To develop an integral abutment design, a minimum pile length below the abutment stem of 5.0 m is required. At the east abutment, an integral abutment design can be used provided the pile installation procedures described in Section 8.4.7 are followed.

The integral abutment design requires that the piles possess flexibility in the upper 3 m of the pile length. However, the upper 3 m of the pile will lie within the compacted fill of the approach embankment. Accordingly, to provide the required flexibility in the piles, the upper 3 m of the piles must be surrounded by a 600 mm diameter CSP as specified by the integral abutment design procedures.

After the pile is driven, the space between the pile and the CSP must be filled with sand. An NSSP should be included in the contract drawings specifying the gradation of the sand according to Table 8.3.

Table 8.3 – Integral Abutment Sand Backfill Grading

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

8.6 Lateral Resistance

The lateral resistance of a pile in the predominantly cohesionless soils encountered at this site may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

$$k_s = n_h \, . \, z / D \qquad (\text{kN/m}^3)$$

$$p_{ult} = 3 \, . \, \gamma \, . \, z \, . \, K_p \qquad (\text{kPa})$$

where z = depth of embedment of pile in metres
D = pile width in metres
n_h = value from Table 8.4
γ = unit weight (Table 8.4)
K_p = passive earth pressure coefficient (Table 8.4)

The above equations and recommended parameters may be used to analyze the interaction between a pile and the surrounding soil. The lateral pressures obtained from the analysis should not exceed the ultimate lateral resistance.

The spring constant, K, for analysis may be obtained by the expression, $K = k_s*L*D$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m^3), D is the pile width (m) and L is the length (m) of the pile segment or element used in the analysis. The ultimate lateral resistance on any one segment of pile, P_{ult}, may be obtained from the expression, $P_{ult} = p_{ult}*L*D$. This represents the ultimate load at which the pile fails and will not support any additional load at greater displacements. It is recommended, however, that the total lateral resistance assumed in one pile be limited to no more than 150 kN at ULS and 50 kN at SLS.

Parameters for lateral pile resistance are shown in Table 8.4.

Table 8.4 – Parameters for Lateral Pile Resistance

Location	Elevation	n_h (kN/m^3)	K_p	Unit Weight (kN/m^3)	Soil Conditions
West abutment	As specified on GA	10,000	3.3	22	Compacted granular fill
	OGL to 186.1 (average)	1,300	3.0	11*	Sand, silt, very loose to compact
Pier	OGL to 185.4	1,300	3.0	11*	Sand, silt, very loose to compact
East abutment	As specified on drawings	10,000	3.3	22	Compacted granular fill
	OGL to 186.5 (average)	1,300	3.0	11*	Sand, silt, very loose to compact

*Buoyant unit weight below the water table.

Pile interaction should be considered with reference to CHBDC Clause 6.8.9.2.

For lateral soil/pile group interaction analysis, the modulus of subgrade reaction (k_s) may have to be reduced based on pile spacing.

Where a pile group is oriented ***perpendicular*** to the direction of loading, group action may be considered by reducing values for k_s by a reduction factor R as follows:

Pile Spacing Perpendicular to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
4 D*	1.00
1 D*	0.50

* D is the width of the pile, and spacing is measured centre to centre

Where a pile group is oriented ***parallel*** to the direction of loading, group action may be considered by reducing values for k_s by a reduction factor R as follows:

Pile Spacing Parallel to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
8 D	1.00
6 D	0.70
4 D	0.40
3 D	0.25

Intermediate values may be obtained by interpolation.

Where piles are grouted into rock sockets, consideration must be given to the effect of the rock sockets. The ultimate passive force that can be mobilized by the embedded portion of the pile is given by:

$$P_p = 6.c.D.L \text{ (kN)}$$

Where:

c = 5,000 kPa for this site (from Mohr-Coulomb cohesion based on Hoek and Brown rock mass classification)
L = depth of socket, m
D = pile width, m

For conventional abutments, the lateral resistance may be provided by battered piles.

8.7 Recommended Foundation

From a geotechnical perspective, the recommended foundation for the abutments and pier at this site are steel H-piles driven and/or socketted into bedrock.

At the pier, caisson foundations are a technically feasible alternative, if it is advantageous from a structural view point.

8.8 Frost Cover

The design depth of frost penetration at this site is 1.8 m.

However, frost penetration is not an issue for footings bearing on bedrock or mass concrete fill placed on bedrock.

9 EXCAVATION

Peat removal will be required at this site. Peat is generally 35 mm to 800 mm thick. Rock excavation may be necessary at the east abutment for pile installation.

9.1 Earth Excavation

If earth excavation is required, it must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the native soils within the probable depth of excavation at this site may be classed as Type 3 soils above water level and Type 4 below water level.

The excavation must be carried out in accordance with OPSS 902, November 2010.

9.2 Rock Excavation

Rock excavation may be required at the east abutment in order to achieve the necessary minimum pile length.

Due to the relatively shallow depth of rock excavation, it may be possible to avoid blasting. It is recommended that excavation be carried out using pneumatic breakers, or other methods that will avoid shattering and disturbing the bedrock.

10 UNWATERING

The piezometric readings at the west abutment indicate that the groundwater levels range from Elevations 191.1 to 191.6.

Visual observation indicates that standing water may be encountered across the site.

Based on the preliminary GA for the bridge structure and the use of pile foundations, it is not expected that work at the abutments will require excavation below the groundwater level.

Also, the preliminary GA indicates that the final grade of the Highway 69 pavement is around elevation 196.7, which is approximately 4.5 m to 6.0 m above the present ground surface at the pier.

Provided the fill to raise the Highway 69 grade to elevation 196.7 is placed prior to foundation construction at the pier, the construction of the pier pile cap is not likely to require an excavation below the ground water level.

Unwatering requirements will consist of provision to drain/remove ponded surface water during approach fill construction. This could be achieved by draining the beaver pond and implementing groundwater control measures such as ditching and pumping from filtered sumps.

In general, the design of the dewatering system should be the responsibility of the Contractor and the Contract Documents should alert him to this responsibility and the need to engage a dewatering specialist.

11 APPROACH EMBANKMENTS

Approach embankment construction using either earth fill or rock fill is feasible on the foundation soils encountered at this site. The fills will be constructed on native sands or bedrock.

11.1 Settlement

It is estimated that at the west abutment, settlements in the order of 90 mm will occur in the foundation soils under the loading imposed by approximately 11.8 m of the approach fill. At the east abutment, the settlement in the foundation soils is estimated to be in the order of 65 mm under loading imposed by 13.4 m of approach fill.

Due to the non-cohesive nature of the foundation soils, this settlement will be immediate and essentially complete when construction of the fill is completed.

Earth fill embankments typically compress by as much as 0.5% of the height of the fill. However, provided non-cohesive fill is used, this compression occurs as the embankment is constructed and can be assumed to be complete at the end of earthworks construction.

In the case of rock fill, experience has shown that post construction settlement at the top of the embankment can amount to 1% of the fill height, approximately 118 mm and 134 mm at the west and east approaches, respectively.

Based on the above settlement estimates, it is considered prudent to overbuild the approach embankments to account for a total settlement in the order of 208 mm and 200 mm at the west and east approaches, respectively.

Rock fill embankments should be overbuilt in accordance with current Northeastern Region policies and guidelines.

11.2 Slope stability

The global, internal and surficial stability of the approach embankment fills will depend on the slope geometry and also to a large degree on the material used to construct the embankment. If the embankment is constructed of blast rock fill, it may be assumed that the side slopes will be stable at inclinations up to 1.25H:1V. Embankments constructed using granular material, select subgrade material or non-cohesive earth fill will have stable side slopes at inclinations of up to 2H:1V.

For the purpose of embankment stability analyses, the commercially available slope stability program GSLOPE developed by Mitre Software Inc. was used. The Bishop's simplified method for stability analysis was employed.

Global stability analyses were conducted for 2H:1V SSM or earth fill embankments and for 1.25H:1V rock fill embankments. The stability of the embankments was also checked under seismic loading assuming an acceleration of 0.08g. The computed factors of safety are as shown in Table 11.1. Slope stability computation outputs are included in Appendix F.

Table 11.1 Computed Factors of Safety

Location / Material	Condition	Factor of Safety	Figure (Appendix F)
West Approach			
Earth Fill	Normal	1.4	1
Earth Fill	Seismic = 0.08g	1.1	2
Rock Fill	Normal	1.4	3
Rock Fill	Seismic = 0.08g	1.2	4
East Approach			
Earth Fill	Normal	1.3	5
Earth Fill	Seismic = 0.08g	1.0	6
Rock Fill	Normal	1.3	7
Rock Fill	Seismic = 0.08g	1.1	8

In each case of normal loading, the factor of safety against global failure was equal to or greater than 1.3. Under the assumed seismic loading, the minimum factor of safety calculated was 1.0. These factors of safety are considered to be acceptable for the proposed embankments bearing on non-cohesive soil.

It is recommended that all ponded water be drained from the embankment areas and all peat, organics, deleterious materials and loose sand be stripped prior to constructing the approach fills. Embankment construction must be in accordance with SP206S03, dated July 2007.

The approach fills should be constructed, to the underside of the CSP, in advance of pile driving operations.

Where earth fill embankments are higher than 8 m, mid-height berms must be incorporated in the design. The berms must:

- extend for the length through which the embankment height exceeds 8 m
- be at least 2 m wide
- have 2% positive grade to shed run-off water.

Where rock fill embankments are higher than 10 m, mid-height berms must be incorporated in the design. The berms must:

- extend for the length through which the embankment height exceeds 10 m
- be at least 2 m wide

Earth fill embankment slopes must be provided with erosion protection in accordance with OPSS 804, November 2010.

12 BACKFILL TO ABUTMENTS

In the case of integral or semi-integral abutments, backfill to the abutment must be granular material.

In the case of a conventional abutment, granular backfill is recommended but rock backfill can be permitted. An NSSP is required to specify grading limits for the rock fill. The rock fill used as backfill for the abutment should be limited to fragments no greater than 150 mm.

In all cases where the approach embankment consists of rock fill and granular backfill to the abutment wall is used, the granular backfill must consist of OPSS Granular "B" Type II.

The backfill to the abutment walls must be in accordance with OPSS 902 dated November 2010. Granular backfill must be placed to the extents shown in OPSD 3101.150, and rock backfill must be placed to the extents shown in OPSD 3101.200. All granular material should meet the requirements of SP 110S13 "Amendment to OPSS 1010, April 2004".

Compaction equipment to be used adjacent to the abutment walls must be restricted in accordance with OPSS 501 dated November 2010.

The design of the abutment must incorporate a subdrain as shown in OPSD 3101.150 or OPSD 3101.200, as applicable.

13 EARTH PRESSURE

Earth pressures acting on the structure may be assumed to be triangular and to be governed by the characteristics of the abutment backfill. For a fully drained condition, the pressures should be computed in accordance with the CHBDC but generally are given by the expression:

$$P_h = K^*(\gamma h + q)$$

Where:

P_h = horizontal pressure on the wall at depth h (kPa)

K = earth pressure coefficient (see Table 13.1)

γ = unit weight of retained soil (see Table 13.1)

h = depth below top of fill where pressure is computed (m)

q = value of any surcharge (kPa)

In accordance with Clause 6.9.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 2.0 m for Granular B Type I or 1.7 m for Granular A or Granular B Type II.

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical values are shown in Table 13.1.

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) might be preferred as it results in lower earth pressures acting on the wall.

The factors in Table 13.1 are "ultimate" values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.9.1 (a) in the Commentary to the Canadian Highway Bridge Design Code.

Table 13.1 – Earth Pressure Coefficient (K)

Condition	Earth Pressure Coefficient (K)					
	OPSS Granular A or OPSS Granular B Type II $\phi = 35°, \gamma = 22.8$ kN/m^3		OPSS Granular B Type I $\phi = 32°, \gamma = 21.2$ kN/m^3		Rock Fill (Limited to 300 mm size) $\phi = 42°, \gamma = 19$ kN/m^3	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall(2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall(2H: 1V)
Active (Unrestrained Wall)	0.27	0.40*	0.31	0.48*	0.2	0.28*
At rest (Restrained Wall)	0.43	-	0.47	-	0.33	-
Passive (Movement Towards Soil Mass)	3.7	-	3.3	-	5.0	-

* For wing walls.

14 SEISMIC CONSIDERATIONS

14.1 Seismic Design Parameters

The site is treated as lying in Seismic Zone 1. The following seismic parameters should be used for design:

Velocity Related Seismic Zone	1
Zonal Velocity Ratio	0.05
Acceleration Related Seismic Zone	1
Zonal Acceleration Ratio	0.05
Peak Horizontal Acceleration	0.08

The soil profile type at this site has been classified as Type I. Therefore, according to Table 4.4.6.1 of the CHBDC, a Site Coefficient "S" (ground motion amplification factor) of 1.0 should be used in seismic design.

14.2 Liquefaction Potential

The potential for liquefaction of the foundations soils was assessed using the Seed and Idriss (1971) method[1].

Using this method, it is estimated that under the existing conditions the foundation soils at the abutments is not prone to liquefaction. At the abutments, the approach embankments will increase the effective stress on the soil under the embankment and around the piles and as a result, liquefaction at the foundation is not considered to be likely.

If the structure is supported on steel piles and drilled shafts, the foundation loads will be transferred by the steel piles to bedrock. In this case, it is not considered likely that the vertical geotechnical resistance of the piles will be compromised.

The embankments themselves will be constructed above the groundwater level and are not considered to be in danger of undergoing liquefaction. Some toe failure may occur but it is expected to be of limited nature and readily repairable.

14.3 Retaining Wall Dynamic Earth Pressures

In accordance with Clause 4.6.4 of the CHBDC, retaining structures should be designed using active (K_{AE}) and passive (K_{PE}) earth pressure coefficients that incorporate the effects of earthquake loading.

For the design of retaining walls, the coefficients of horizontal earth pressure in Table 14.1 may be used:

Table 14.1 – Earth Pressure Coefficient for Earthquake Loading

Earth Pressure Coefficient (K) for Earthquake Loading						
Wall Condition	**Granular A or Granular B Type II $\phi = 35^\circ$; $\gamma = 22.8\ \text{kN/m}^3$**		**OPSS Granular B Type I $\phi = 32^\circ$; $\gamma = 21.2\ \text{kN/m}^3$**		**Rock Fill $\phi = 42^\circ$; $\gamma = 19.0\ \text{kN/m}^3$**	
	Horizontal Surface Behind Wall	**Sloping Surface Behind Wall (2H:1V)**	**Horizontal Surface Behind Wall**	**Sloping Surface Behind Wall (2H:1V)**	**Horizontal Surface Behind Wall**	**Sloping Surface Behind Wall (2H:1V)**
Active (K_{AE})*	0.3	0.47	0.34	0.58	0.22	0.31
Passive (K_{PE})	3.6	-	3.2	-	4.9	-
At Rest (K_{OE})**	0.53	-	0.57	-	0.43	-

* After Mononobe and Okabe, passive case assumes a horizontal surface in front of the wall.
** After Woods

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[1] Seed, H.B. and Idriss, I.M. 1971, "Simplified Procedure for Evaluating Soil Liquefaction Potential" *Journal of Soil Mechanics and Foundations Division,* ASCE, Vol. 101, No. SM9, September, pp. 1249-1273.

15 CONSTRUCTION CONCERNS

Potential construction concerns include, but are not necessarily limited to:

1. Variations in the elevation of the bedrock surface.

The surface of the bedrock has been shown in the investigation to be variable. Since the elevation of the bedrock surface was only established at discrete points, it is possible that higher or lower elevations will be encountered during construction. Also, the slope of the bedrock is expected to be locally steeper than that depicted by joining the elevations established at the boreholes.

The bedrock elevation variability may lead to one or more of the following issues:

- Deeper rock excavations than expected at the east abutment
- H-piles sliding on steep bedrock surfaces

The Construction Administrator and the Contractor should work cooperatively to solve these issues on site. However, if it appears there is an impact on the design, the matter must be referred to the design team for comment.

2. Caisson installation

Adequate cleaning and unwatering of caisson bases will be difficult. Placement of concrete by tremie methods may be required.

16 CLOSURE

Engineering analysis and preparation of the report were carried out by Mr. Alastair E. Gorman, P.Eng and Ms. R. Palomeque Reyna, P.Eng.

The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

Thurber Engineering Ltd.

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Report reviewed by:
P.K. Chatterji, P.Eng., Ph.D.
Review Principal

Appendix A

Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

CLASSIFICATION	PARTICLE SIZE	VISUAL IDENTIFICATION
Boulders	Greater than 200mm	same
Cobbles	75 to 200mm	same
Gravel	4.75 to 75mm	5 to 75mm
Sand	0.075 to 4.75mm	Not visible particles to 5mm
Silt	0.002 to 0.075mm	Non-plastic particles, not visible to the naked eye
Clay	Less than 0.002mm	Plastic particles, not visible to the naked eye

2. COARSE GRAIN SOIL DESCRIPTION (50% greater than 0.075mm)

TERMINOLOGY	PROPORTION
Trace or Occasional	Less than 10%
Some	10 to 20%
Adjective (e.g. silty or sandy)	20 to 35%
And (e.g. sand and gravel)	35 to 50%

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROXIMATE SPT(1) 'N' VALUE
Very Soft	12 or less	Less than 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	Greater than 200	Greater than 30

NOTE: Hierarchy of Soil Strength Prediction

1) Laboratory Triaxial Testing
2) Field Insitu Vane Testing
3) Laboratory Vane Testing
4) SPT value
5) Pocket Penetrometer

4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

DESCRIPTIVE TERM	SPT "N" VALUE
Very Loose	Less than 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	Greater than 50

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE

SS	Split Spoon Sample
WS	Wash Sample
AS	Auger (Grab) Sample
TW	Thin Wall Shelby Tube Sample
TP	Thin Wall Piston Sample
PH	Sampler Advanced by Hydraulic Pressure
PM	Sampler Advanced by Manual Pressure
WH	Sampler Advanced by Self Static Weight
RC	Rock Core
SC	Soil Core

$$\text{Sensitivity} = \frac{\text{Undisturbed Shear Strength}}{\text{Remoulded Shear Strength}}$$

▼ Water Level

C_{pen} Shear Strength Determination by Pocket Penetrometer

(1) SPT 'N' Value Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.

(2) DCPT Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

UNIFIED SOILS CLASSIFICATION

MAJOR DIVISIONS		GROUP SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS	SW	Well-graded sands or gravelly sands, little or no fines.
		SP	Poorly-graded sands or gravelly sands, little or no fines.
		SM	Silty sands, sand-silt mixtures.
		SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILTS AND CLAYS $W_L < 50\%$	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays. ($W_L < 30\%$).
		CI	Inorganic clays of medium plasticity, silty clays. ($30\% < W_L < 50\%$).
		OL	Organic silts and organic silty-clays of low plasticity.
	SILTS AND CLAYS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.
CLAY SHALE			
SANDSTONE			
SILTSTONE			
CLAYSTONE			
COAL			

EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION

Fresh (FR)	No visible signs of weathering.
Fresh Jointed (FJ)	Weathering limited to the surface of major discontinuities.
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.

SYMBOLS

▨	CLAYSTONE
▤	SILTSTONE
░	SANDSTONE
■	COAL
▩	Bedrock (general)

DISCONTINUITY SPACING

Bedding	**Bedding Plane Spacing**
Very thickly bedded	Greater than 2m
Thickly bedded	0.6 to 2m
Medium bedded	0.2 to 0.6m
Thinly bedded	60mm to 0.2m
Very thinly bedded	20 to 60mm
Laminated	6 to 20mm
Thinly Laminated	Less than 6mm

TERMS

Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a percentage of total core run length.
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.

STRENGTH CLASSIFICATION

Rock Strength	**Approximate Uniaxial Compressive Strength**		**Field Estimation of Hardness***
	(MPa)	**(psi)**	
Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail

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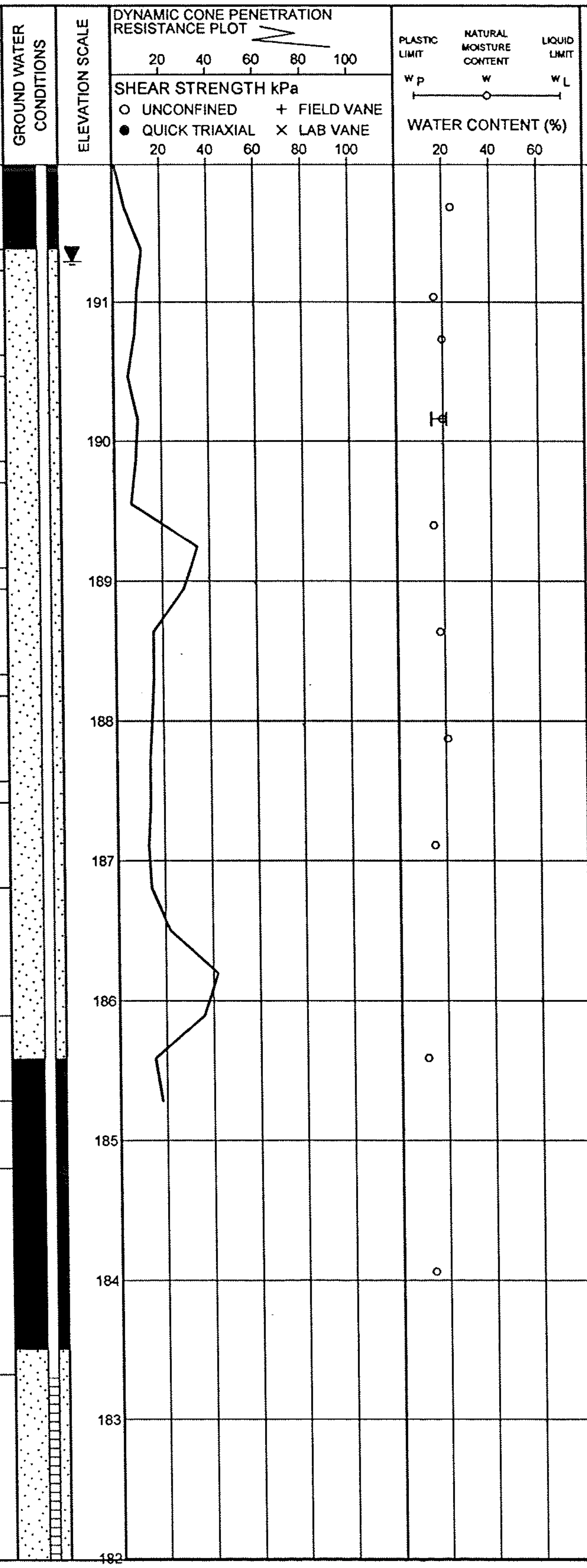
RECORD OF BOREHOLE No HLR09-01

1 OF 2

METRIC

W.P. 5076-06-00
LOCATION N 5 061 697.3 E 230 765.2 Harris Lake Road Underpass
ORIGINATED BY ES
HWY 69
BOREHOLE TYPE Hollow Stem Augers / Casing
COMPILED BY AN
DATUM Geodetic
DATE 2009.08.24 - 2009.08.24
CHECKED BY RPR

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	"N" VALUES	UNIT WEIGHT kN/m3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
191.9	Ground surface					
0.0	**PEAT**, mixed with roots and rootlets (35mm)					
	SAND, fine grained Very Loose to Compact Brown Moist	1	SS	3		
		2	SS	11		
190.8 1.1	Clayey **SILT**, some sand to sandy, occasional oxide staining Firm to Stiff Grey	3	SS	5		0 28 57 15
189.6 2.4	**SAND**, fine grained, trace to some gravel, trace silt, trace clay Very Loose to Loose Brown Wet	4	SS	1		
		5	SS	8		
		6	SS	3		
		7	SS	3		11 88 1 (SI+CL)
	Coarse grained, occasional bedrock fragments and cobbles Compact Loose	8	SS	16		
		9	SS	5		4 94 2 (SI+CL)
184.8 7.2	**BEDROCK**, granitic, moderately weathered to fresh, grey, occasional dark grey, brown and white bands, occasional mechanical and sub-vertical breaks Coring started at 7.2m Sub-vertical fractures: 100mm at 9.1m 50mm at 9.8m 150mm at 10.8m 100mm at 11.2m 25mm at 11.4m 50mm at 11.6m	1	RUN		FI >10 >10	TCR=34% SCR=0% RQD=0% UCS=157MPa (Average)
		2	RUN		0 1 0 3	TCR=100% SCR=89% RQD=89% UCS=116MPa (Average)



Continued Next Page

+3, ×3: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No HLR09-01

2 OF 2

METRIC

W.P.	5076-06-00	LOCATION	N 5 061 697.3 E 230 765.2 Harris Lake Road Underpass	ORIGINATED BY	ES
HWY	69	BOREHOLE TYPE	Hollow Stem Augers / Casing	COMPILED BY	AN
DATUM	Geodetic	DATE	2009.08.24 - 2009.08.24	CHECKED BY	RPR

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 × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION Continued From Previous Page	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	BEDROCK, granitic gneiss, slightly weathered to fresh, grey, occasional mechanical breaks Highly broken zone: 175mm at 10.8m 125mm at 11.4m		3	RUN			181			2 1 3 >10 >10 >10	TCR=100% SCR=60% RQD=60% UCS=85MPa (Average)
180.2 11.8	END OF BOREHOLE AT 11.8m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS:										

DATE	DEPTH (m)	ELEV. (m)
Aug 25/09	0.6	191.3
Sep 14/09	0.3	191.6
Sep 24/09	0.5	191.4
Oct 07/09	0.5	191.4
Oct 27/09	0.4	191.5
Nov 20/09	0.8	191.1
Apr 27/11	0.7	191.2

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

$15 \overset{20}{\underset{10}{\oplus}} 5$ (%) STRAIN AT FAILURE

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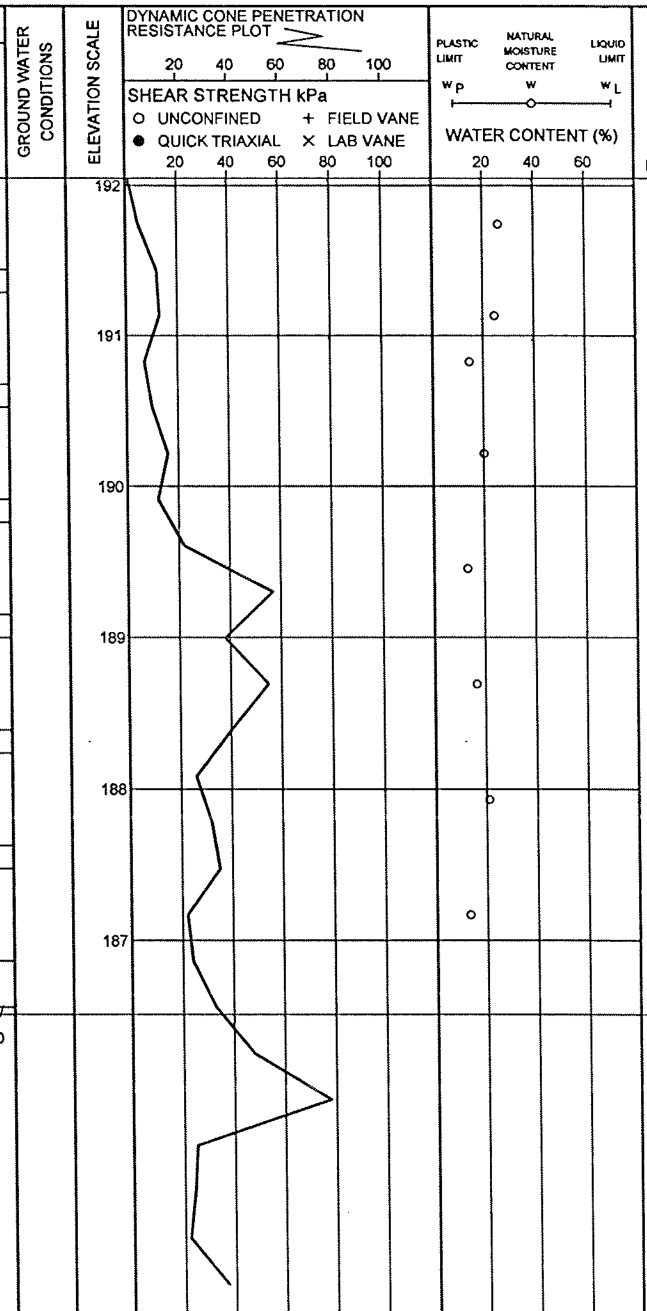
RECORD OF BOREHOLE No HLR09-02

1 OF 1 **METRIC**

W.P. 5076-06-00
LOCATION N 5 061 699.2 E 230 769.9 Harris Lake Road Underpass
ORIGINATED BY ES
HWY 69
BOREHOLE TYPE Hollow Stem Augers / Casing
COMPILED BY AN
DATUM Geodetic
DATE 2009.08.23 - 2009.08.23
CHECKED BY RPR

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
192.0	Ground surface							
0.0	**PEAT:** (50mm)							
	SAND, fine grained, occasional roots Loose to Compact Brown Moist	1	SS	4				
190.9 1.1	Clayey **SILT**, some sand to sandy, occasional oxide staining Firm to Stiff Grey	2	SS	10				
		3	SS	7	0	26	60	14
189.4 2.6	**SAND**, fine grained, trace gravel Loose to Compact Brown Wet	4	SS	14				
		5	SS	6				
	Fine to coarse grained	6	SS	4				
		7	SS	26				
186.5	**BEDROCK**, granitic gneiss, grey	8	SS	100/ .050				
5.5	END OF BOREHOLE AT 5.5m UPON AUGER REFUSAL ON PROBABLE BEDROCK. BOREHOLE BACKFILLED WITH HOLEPLUG TO 1.2m, THEN SAND TO SURFACE.							



+³, ×³: Numbers refer to Sensitivity

20
15-◇-5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No HLR09-03

1 OF 1

METRIC

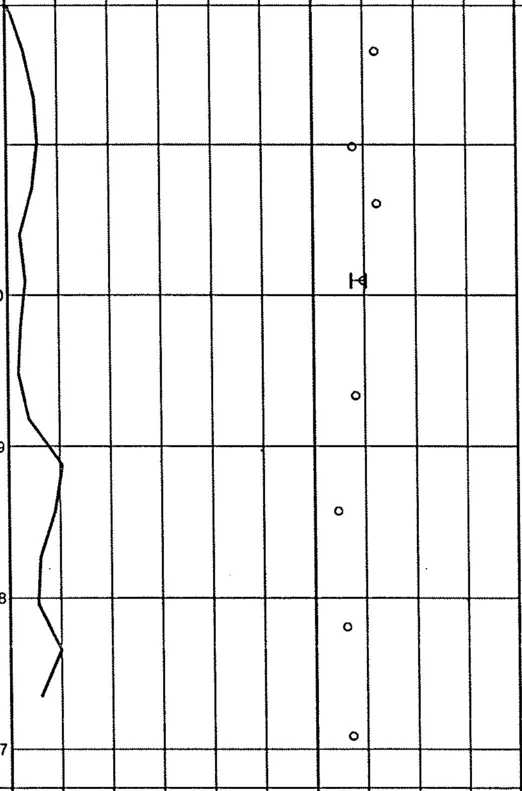
W.P. 5076-06-00
LOCATION N 5 061 692.2 E 230 767.3 Harris Lake Road Underpass
ORIGINATED BY ES
HWY 69
BOREHOLE TYPE Hollow Stem Augers / Casing
COMPILED BY AN
DATUM Geodetic
DATE 2009.08.21 - 2009.08.21
CHECKED BY RPR

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
191.9	Ground surface											
0.0	**PEAT:** (125mm)											
0.1	**SAND**, fine grained Loose to Compact Brown Moist		1	SS	4							
			2	SS	10		191					
190.6 1.3	Clayey **SILT**, some sand to sandy, occasional sand pockets, occasional oxide staining Soft to Stiff Brown to Grey		3	SS	3		190		0	28	58	14
189.6 2.3	**SAND**, fine grained, some silt to silty, trace gravel, trace clay Very Loose to Compact Brown to Grey Wet		4	SS	0		189					
			5	SS	19				5	64	29	2
	trace silt		6	SS	9		188					
			7	SS	12		187		9	86	5 (SI+CL)	
186.7 186.7 5.2	**BEDROCK**, granitic gneiss, grey END OF BOREHOLE AT 5.2m UPON AUGER REFUSAL ON PROBABLE BEDROCK. BOREHOLE BACKFILLED WITH HOLEPLUG TO 3.7m, THEN AUGER CUTTINGS TO SURFACE.											

DYNAMIC CONE PENETRATION RESISTANCE PLOT
20 40 60 80 100
SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE
20 40 60 80 100

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L
WATER CONTENT (%)
20 40 60

REMARKS & GRAIN SIZE DISTRIBUTION (%)



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

$15 \overset{20}{\underset{10}{\oplus}} 5$ (%) STRAIN AT FAILURE

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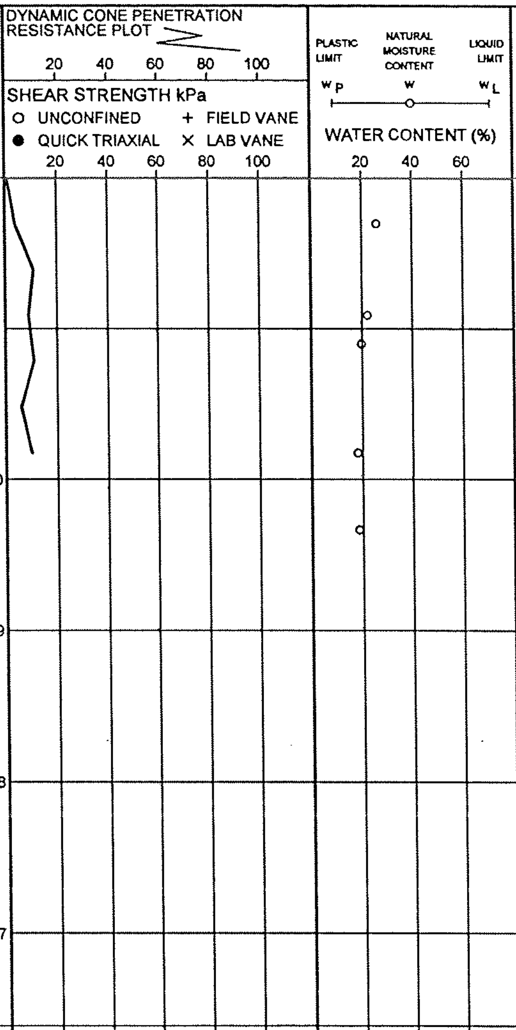
RECORD OF BOREHOLE No HLR09-04

1 OF 1

METRIC

W.P.	5076-06-00	LOCATION	N 5 061 694.1 E 230 771.9 Harris Lake Road Underpass	ORIGINATED BY	ES
HWY	69	BOREHOLE TYPE	Hollow Stem Augers / Casing	COMPILED BY	AN
DATUM	Geodetic	DATE	2009.08.23 - 2009.08.23	CHECKED BY	RPR

SOIL PROFILE			SAMPLES						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m 3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
192.0	Ground surface								
0.0 0.1	**PEAT:** (75mm)						192		
	SAND, fine to coarse grained, trace silt, trace clay, occasional roots and organics Very Loose to Loose Dark Brown Damp to Moist Layer of grey clayey silt (400mm)		1	SS	3				
			2	SS	9		191		0 90 10 (SI+CL)
	Layer of grey silty clay below 1.5m		3	SS	4		190	FI	
189.6 2.4	silty							0	0 41 50 9
	BEDROCK, granitic gneiss, slightly weathered to fresh, grey, occasional dark grey, brown and white bands, occasional mechanical and sub-vertical breaks Coring started at 2.4m		1	RUN			189	0 0 0	TCR=100% SCR=100% RQD=100% UCS=168MPa (Average)
	Sub-vertical fractures: 25mm at 4.2m 100mm at 4.9m		2	RUN			188	0 0 0 2	TCR=100% SCR=100% RQD=100% UCS=138MPa (Average)
186.3			3	RUN			187	0 3 3 0	TCR=100% SCR=91% RQD=80% UCS=156MPa (Average)
5.6	END OF BOREHOLE AT 5.6m. BOREHOLE BACKFILLED WITH HOLEPLUG TO 2.1m, THEN SAND TO SURFACE.								



+ 3, X 3 : Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No HLR09-05

1 OF 1

METRIC

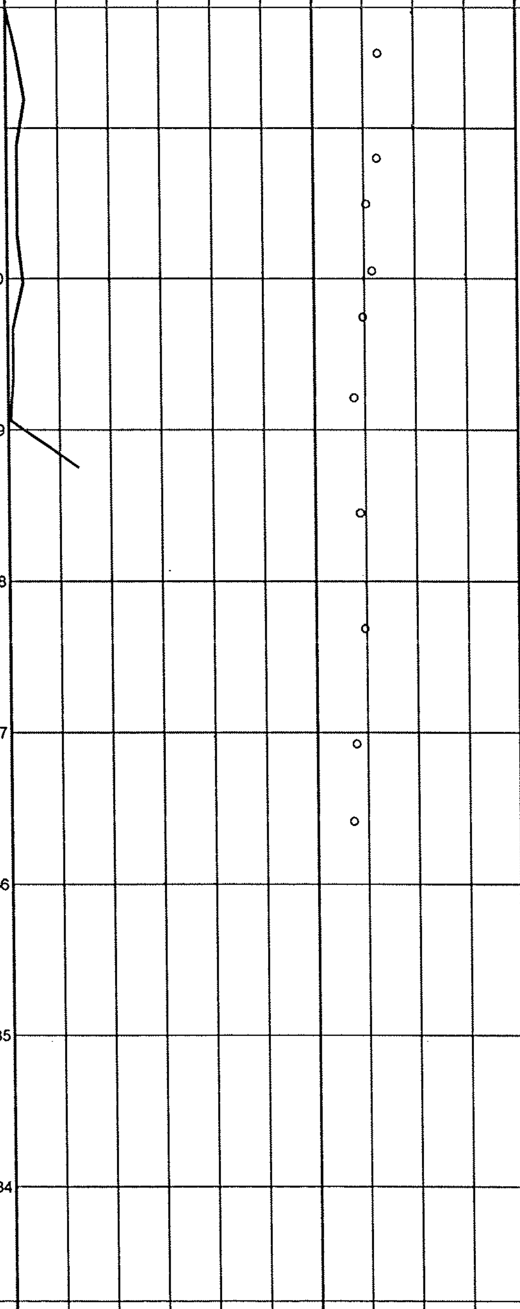
W.P.	5076-06-00	LOCATION	N 5 061 687.1 E 230 769.3 Harris Lake Road Underpass	ORIGINATED BY	ES
HWY	69	BOREHOLE TYPE	Hollow Stem Augers / Casing	COMPILED BY	AN
DATUM	Geodetic	DATE	2009.08.22 - 2009.08.22	CHECKED BY	RPR

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
191.8	Ground surface								
0.0	**PEAT**: (50mm)								
	SAND, fine grained Very Loose Brown Wet		1	SS	2				
			2	SS	3		191		
190.5 1.3	Clayey **SILT**, sandy, occasional oxide staining Very Soft to Soft Grey		3	SS	1		190		0 32 56 12
189.8 2.0	**SAND**, fine to coarse grained, trace clay, occasional oxide staining Very Loose to Compact Brown to Grey Wet		4	SS	1		189		
	silty		5	SS	12				0 51 44 5
			6	SS	16		188		
	trace silt		7	SS	7		187		1 93 6 (SI+CL)
			8	SS	100/			FI	
186.3 5.4	**BEDROCK**, granitic gneiss, slightly weathered to fresh, grey, occasional dark grey, brown and white bands, occasional mechanical joints and sub-vertical breaks Coring started at 5.4m		1	RUN	.100		186	2 0	TCR=100% SCR=93% RQD=93% UCS=203MPa (Average)
	100mm vertical fractures at 5.5m Sub-vertical fractures: 75mm at 6.7m 75mm at 6.9m		2	RUN			185	1 0 >10 1	TCR=100% SCR=83% RQD=83% UCS=171MPa (Average)
	150mm at 8.2m		3	RUN			184	0 2 3 0	TCR=100% SCR=88% RQD=88% UCS=130MPa (Average)
183.2 8.6	END OF BOREHOLE AT 8.6m. BOREHOLE BACKFILLED WITH HOLEPLUG TO 0.9m, THEN CUTTINGS TO SURFACE.								

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L
WATER CONTENT (%)
20 40 60



ONTMT4S 6121(HLR).GPJ 12/7/11

+³, ×³: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

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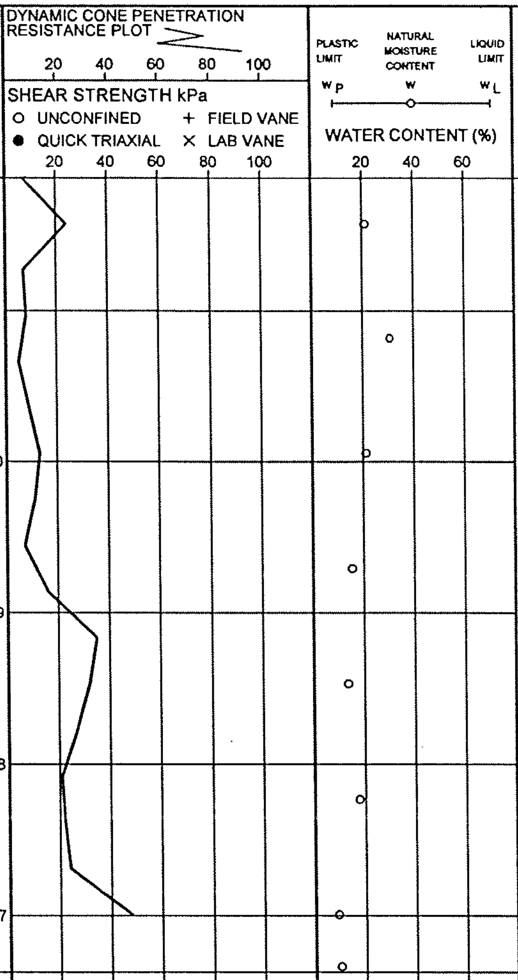
RECORD OF BOREHOLE No HLR09-06

1 OF 1 **METRIC**

W.P. 5076-06-00
LOCATION N 5 061 689.0 E 230 774.0 Harris Lake Road Underpass
ORIGINATED BY ES
HWY 69
BOREHOLE TYPE Hollow Stem Augers / Casing
COMPILED BY AN
DATUM Geodetic
DATE 2009.08.22 - 2009.08.22
CHECKED BY RPR

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
191.8	Ground surface						
0.0	**PEAT:** (35mm)						
	SAND, mixed with organics, occasional roots, trace gravel Loose Dark Brown Damp	1	SS	5	∇		
190.9 0.9	Clayey **SILT**, some sand Soft Grey	2	SS	3		191	0 22 63 15
	Occasional oxide staining	3	SS	3		190	
189.5 2.4	**SAND**, trace silt to silty, trace clay Very Loose to Compact Grey Wet	4	SS	3		189	
		5	SS	21			
	Coarse grained	6	SS	7		188	0 72 26 2
		7	SS	10		187	
186.6 5.2		8	SS	100/ .075			
186.5 5.3	**BEDROCK**, granitic gneiss, grey END OF BOREHOLE AT 5.3m UPON AUGER REFUSAL ON PROBABLE BEDROCK. WATER LEVEL AT 0.4m UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH HOLEPLUG TO 4.6m, THEN SAND TO SURFACE.						



UNIT WEIGHT γ kN/m³

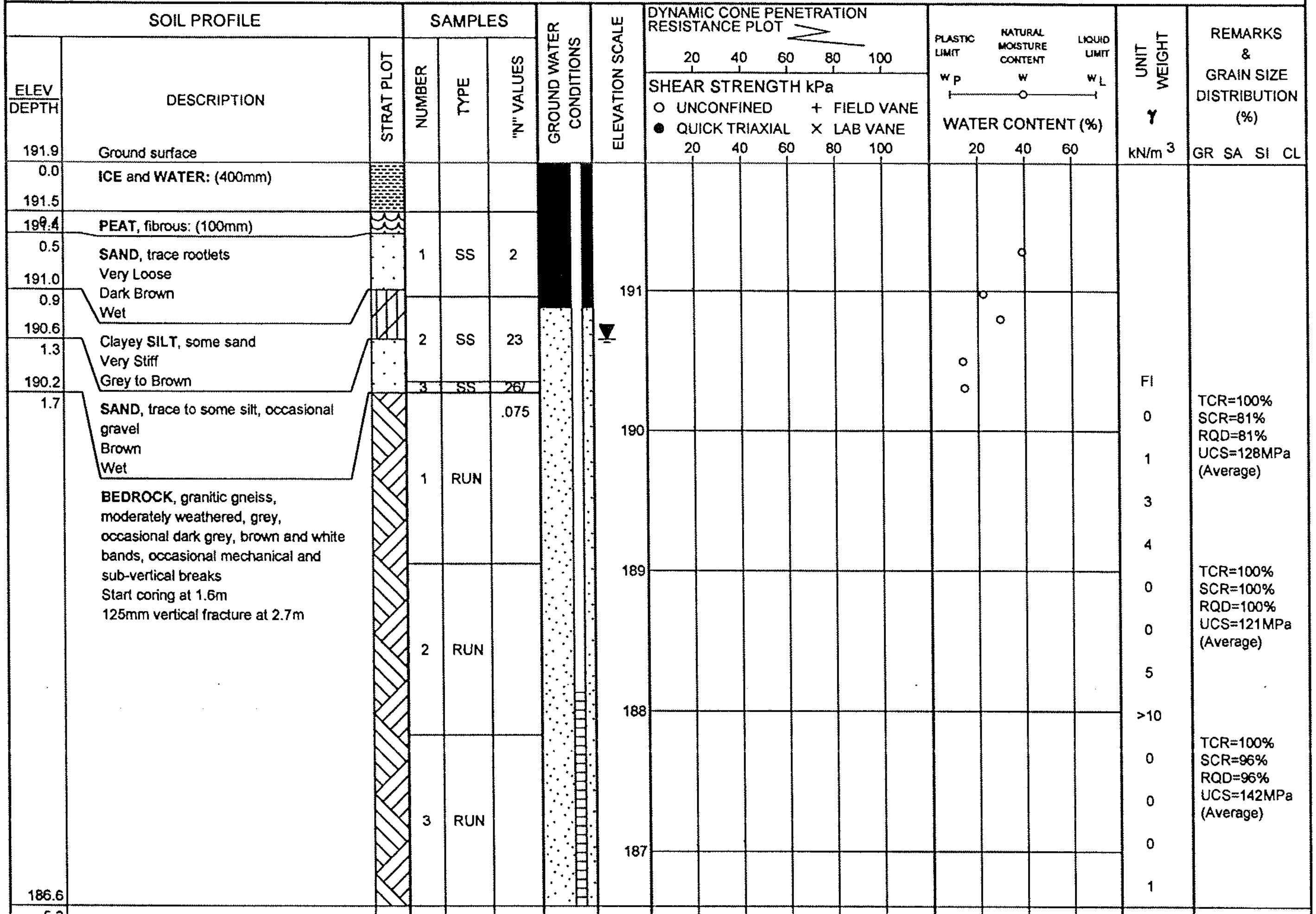
+ ³, × ³: Numbers refer to Sensitivity

20 15 ⊕ 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No HLR09-07

1 OF 1 **METRIC**

W.P.	5076-06-00	LOCATION	N 5 061 711.7 E 230 800.8 Harris Lake Road Underpass	ORIGINATED BY	JM
HWY	69	BOREHOLE TYPE	Wash Boring	COMPILED BY	AN
DATUM	Geodetic	DATE	2010.03.05 - 2010.03.05	CHECKED BY	RPR



ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
191.9	Ground surface					
0.0	**ICE** and **WATER:** (400mm)					
191.5 0.4						
191.4 0.5	**PEAT**, fibrous: (100mm)					
	SAND, trace rootlets Very Loose Dark Brown Wet	1	SS	2		
191.0 0.9						
190.6 1.3	Clayey **SILT**, some sand Very Stiff Grey to Brown	2	SS	23		
190.2 1.7		3	SS	26/ .075	FI	
	SAND, trace to some silt, occasional gravel Brown Wet					
	BEDROCK, granitic gneiss, moderately weathered, grey, occasional dark grey, brown and white bands, occasional mechanical and sub-vertical breaks Start coring at 1.6m 125mm vertical fracture at 2.7m	1	RUN		0 1 3 4	TCR=100% SCR=81% RQD=81% UCS=128MPa (Average)
		2	RUN		0 0 5 >10	TCR=100% SCR=100% RQD=100% UCS=121MPa (Average)
186.6		3	RUN		0 0 0 1	TCR=100% SCR=96% RQD=96% UCS=142MPa (Average)
5.3	END OF BOREHOLE AT 5.3m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.					

WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
Apr 27/11	1.3	190.6

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

$15 \overset{20}{\underset{10}{\oplus}} 5$ (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

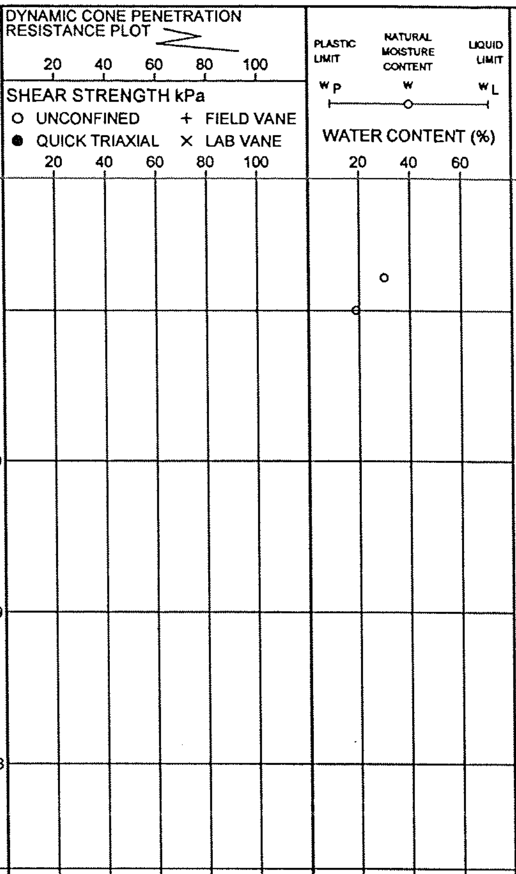
RECORD OF BOREHOLE No HLR09-08

1 OF 1

METRIC

W.P.	5076-06-00	LOCATION	N 5 061 713.6 E 230 805.5 Harris Lake Road Underpass	ORIGINATED BY	SLL
HWY	69	BOREHOLE TYPE	Wash Boring	COMPILED BY	AN
DATUM	Geodetic	DATE	2010.03.05 - 2010.03.05	CHECKED BY	RPR

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES				
191.8	Ground surface							
0.0 191.5	**ICE and WATER:** (300mm)							
191.3 0.3	**PEAT**, fibrous: (100mm)							
0.5 190.9	**SAND**, trace silt, rootlets, occasional cobbles Loose Dark Brown Wet	1	SS	4		191	FI	TCR=100% SCR=100% RQD=0% UCS=191MPa (Average)
0.9	**BEDROCK**, granitic gneiss, slightly weathered to fresh, dark grey, occasional pink and white bands, mechanical and sub-vertical breaks Start coring at 0.9m	1	RUN				2 1	
		2	RUN			190	2 3 4	TCR=100% SCR=100% RQD=72% UCS=140MPa (Average)
	75mm vertical joints at 2.0m	3	RUN			189	1 2 2	TCR=100% SCR=100% RQD=100% UCS=156MPa (Average)
187.3		4	RUN			188	1 4 0 0	TCR=100% SCR=100% RQD=100% UCS=181MPa (Average)
4.6	END OF BOREHOLE AT 4.6m. BOREHOLE BACKFILLED WITH HOLEPLUG TO SURFACE.							



+³, ×³: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

ONTMT4S 6121(HLR).GPJ 12/7/11

Ministry of Transportation
Ontario

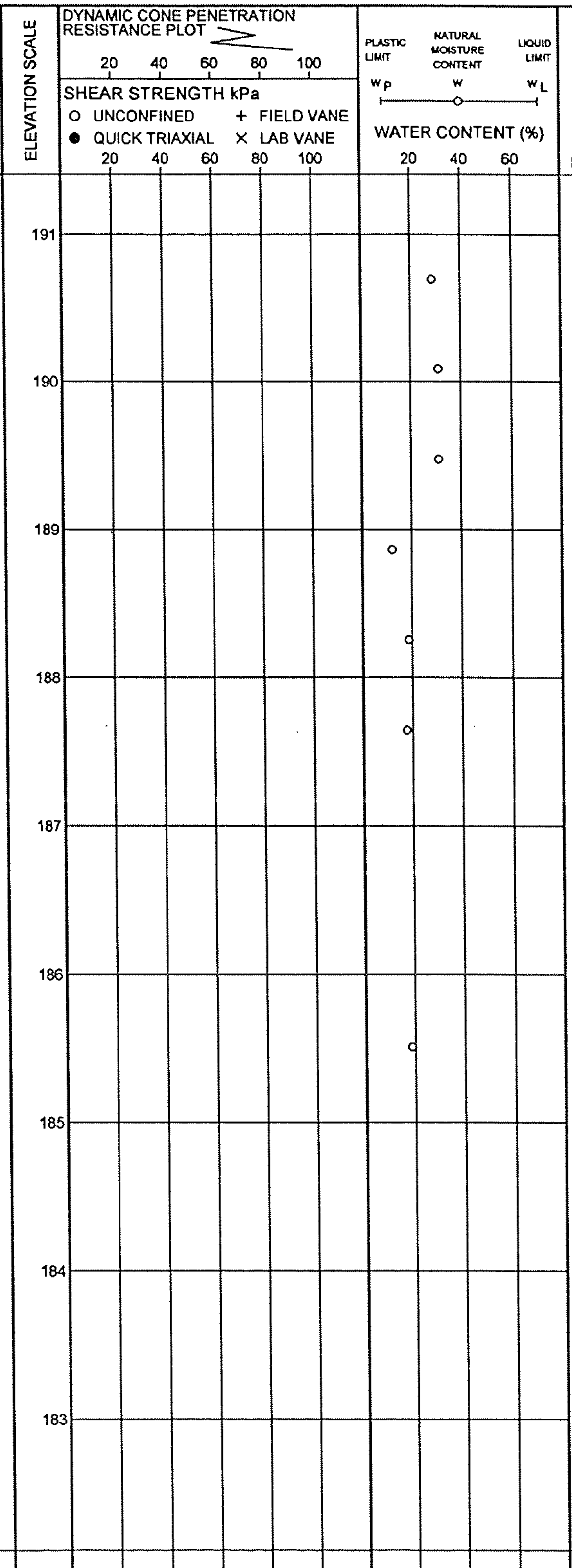
THURBER

RECORD OF BOREHOLE No HLR09-09

1 OF 1 **METRIC**

W.P.	5076-06-00	LOCATION	N 5 061 705.9 E 230 803.8 Harris Lake Road Underpass	ORIGINATED BY	SLL
HWY	69	BOREHOLE TYPE	Wash Boring	COMPILED BY	AN
DATUM	Geodetic	DATE	2010.03.04 - 2010.03.04	CHECKED BY	RPR

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
191.4	Ground surface					
0.0	**ICE and WATER:** (400mm)					
191.0 0.4	**PEAT**, fibrous Dark Brown Wet					
190.7 0.6	**SAND**, trace silt, trace clay, rootlets Very Loose to Compact Dark Brown Wet	1	SS	1		
		2	SS	3		0 92 8 (SI+CL)
	Wood fragments	3	SS	3		
		4	SS	12		
		5	SS	8		
	Occasional cobbles	6	SS	6		
185.4		7	SS	18/ .050	FI	
5.9	**BEDROCK**, granitic gneiss, moderately weathered, grey, continuous pink and white bands, mechanical and sub-vertical breaks Start coring at 5.9m 350mm vertical joints at 5.9m	1	RUN		10	TCR=100% SCR=14% RQD=0%
		2	RUN		5 4	TCR=100% SCR=59% RQD=41% UCS=156MPa (Average)
	225mm sub-vertical joints at 6.3m 350mm quartz seams at 6.7m and 7.1m	3	RUN		1 2 2	TCR=100% SCR=100% RQD=96% UCS=330MPa (Average)
182.1	fresh	4	RUN		0 0 0 0 0	TCR=96% SCR=96% RQD=96% UCS=186MPa (Average)
9.3	END OF BOREHOLE AT 9.3m. BOREHOLE BACKFILLED WITH HOLEPLUG TO SURFACE.					



+³, ×³: Numbers refer to Sensitivity

20 15-ϕ-5 10 (%) STRAIN AT FAILURE

ONTMT4S 6121(HLR).GPJ 12/7/11

Ministry of Transportation
Ontario

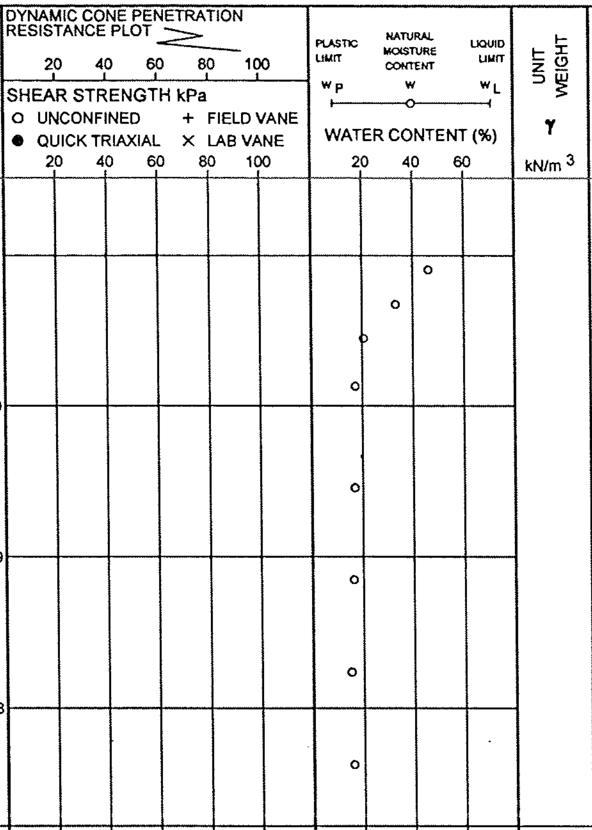
THURBER

RECORD OF BOREHOLE No HLR09-10

1 OF 1 **METRIC**

W.P. 5076-06-00	LOCATION N 5 061 708.2 E 230 808.3 Harris Lake Road Underpass		ORIGINATED BY JM
HWY 69	BOREHOLE TYPE Wash Boring		COMPILED BY AN
DATUM Geodetic	DATE 2010.03.06 - 2010.03.06		CHECKED BY RPR

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
191.5	Ground surface						
0.0	**ICE:** (500mm)						
190.9 0.5					∇	191	
190.8 0.7	**PEAT**, fibrous, rootlets: (200mm) Dark Brown Wet	1	SS	2			
190.5 1.0	Silty **SAND**, occasional gravel Very Loose Brown Wet	2	SS	6		190	0 24 61 15
	Clayey **SILT**, trace to some sand Soft to Firm Brown	3	SS	3			
189.1 2.4	**SAND**, some silt to silty, trace to some clay, trace gravel Loose to Compact Grey Wet	4	SS	13		189	
		5	SS	8		188	
		6	SS	6			1 47 52 (SI+CL)
187.2 4.3	END OF BOREHOLE AT 4.3m UPON REFUSAL ON PROBABLE BEDROCK. BOREHOLE OPEN TO 4.3m. WATER LEVEL AT 0.5m UPON COMPLETION OF BOREHOLE. BOREHOLE BACKFILLED WITH HOLEPUG TO SURFACE.						



+3, ×3: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

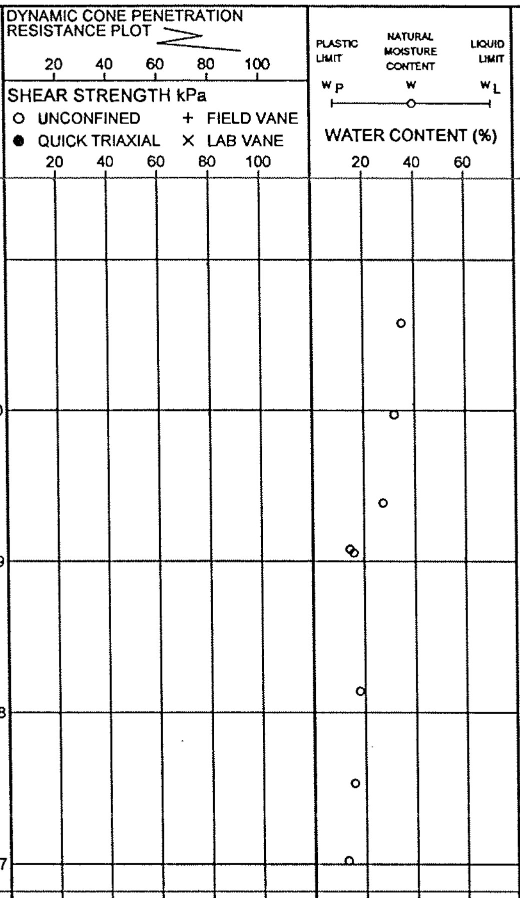
RECORD OF BOREHOLE No HLR09-11

1 OF 1

METRIC

W.P.	5076-06-00	LOCATION	N 5 061 701.0 E 230 804.9 Harris Lake Road Underpass	ORIGINATED BY	SLL
HWY	69	BOREHOLE TYPE	Wash Boring	COMPILED BY	AN
DATUM	Geodetic	DATE	2010.03.04 - 2010.03.04	CHECKED BY	RPR

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES				GR	SA	SI	CL
191.5	Ground surface										
0.0	**ICE and WATER:** (700mm)					191					
190.8 0.7	**PEAT**, fibrous, trace rootlets Soft Dark Brown Wet	1	WH								
190.0 1.5	**SAND**, some silt to silty, trace clay, trace gravel Very Loose to Loose Dark Brown Wet	1	SS	1		190					
		2	SS	1							
	grey					189					
		3	SS	10				1	46	51	2
		4	SS	10		188					
		5	SS	9				6	67	27 (SI+CL)	
186.8		6	SS	25/ 250		187					
4.7	END OF BOREHOLE AT 4.7m UPON AUGER REFUSAL ON PROBABLE BEDROCK. BOREHOLE BACKFILLED WITH HOLEPLUG TO SURFACE.										



+³, ×³: Numbers refer to Sensitivity

15 20 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

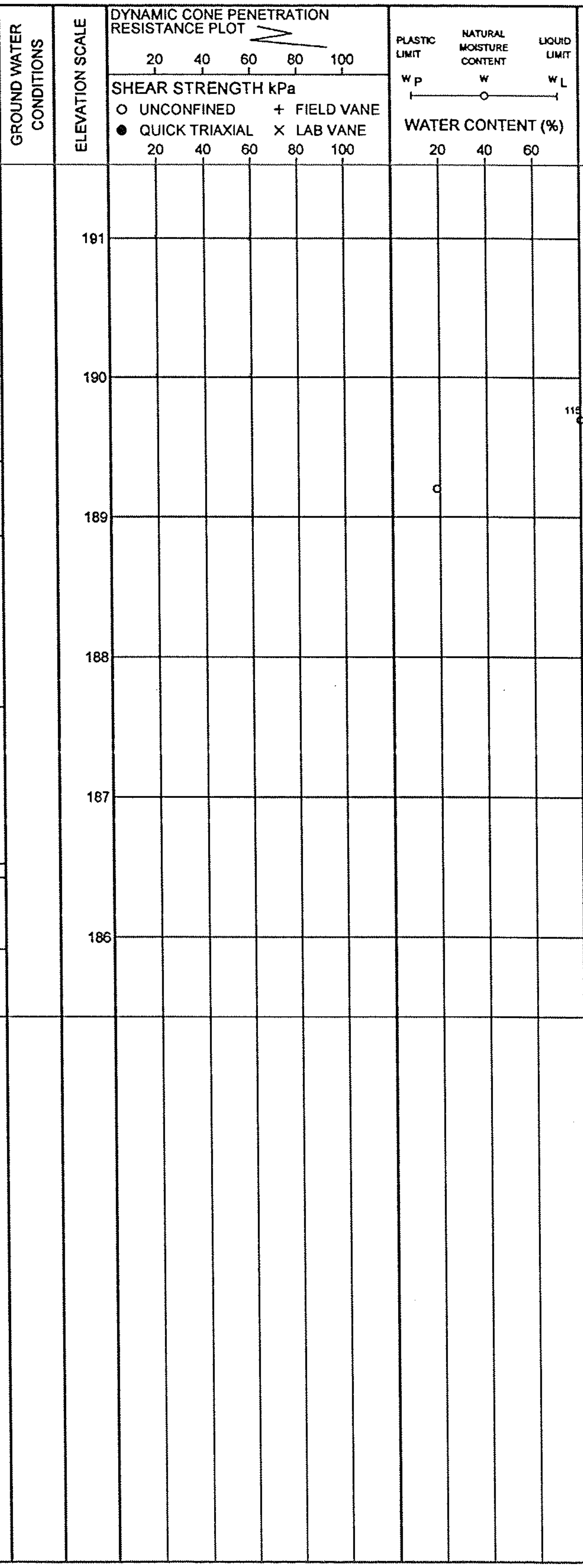
THURBER

RECORD OF BOREHOLE No HLR09-12

1 OF 1 METRIC

W.P. 5076-06-00
LOCATION N 5 061 702.9 E 230 809.7 Harris Lake Road Underpass
ORIGINATED BY SLL
HWY 69
BOREHOLE TYPE Wash Boring
COMPILED BY AN
DATUM Geodetic
DATE 2010.03.10 - 2010.03.10
CHECKED BY RPR

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
191.5	Ground surface					
0.0	**ICE and WATER**					
190.0 1.5	**PEAT**, fibrous: (200mm) Dark Brown Wet	1	SS	22/ .225		
189.8 1.7	**SAND**, trace silt, occasional rootlets Loose Dark Brown Wet				FI	
189.0 2.5	**BEDROCK**, granitic gneiss, slightly weathered to fresh, dark grey with pink bands, mechanical and sub-vertical breaks Coring started at 2.6m	1	RUN		0 0 1 2	TCR=100% SCR=100% RQD=96% UCS=182MPa (Average)
	50mm sub-horizontal joints at 3.8m	2	RUN		1 2 2 1	TCR=92% SCR=90% RQD=100% UCS=206MPa (Average)
		1.5	RUN		0 2 2 0	TCR=100% SCR=100% RQD=100% UCS=194MPa (Average)
185.4 6.1	END OF BOREHOLE AT 6.1m. BOREHOLE BACKFILLED WITH HOLEPLUG TO SURFACE.					



+3, ×3: Numbers refer to Sensitivity

15 20 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No HLR09-13

1 OF 1

METRIC

W.P.	5076-06-00	LOCATION	N 5 061 729.5 E 230 844.8 Harris Lake Road Underpass	ORIGINATED BY	SLL
HWY	69	BOREHOLE TYPE	Visual Assessment	COMPILED BY	AN
DATUM	Geodetic	DATE	2010.03.01 - 2010.03.01	CHECKED BY	RPR

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 × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
194.3	Ground surface										
0.0	BEDROCK AT SURFACE.										

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

$15\overset{20}{\underset{10}{\oplus}}5$ (%) STRAIN AT FAILURE

ONTMT4S 6121(HLR).GPJ 12/7/11

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No HLR09-14

1 OF 1 **METRIC**

W.P. 5076-06-00
LOCATION N 5 061 731.3 E 230 849.4 Harris Lake Road Underpass
ORIGINATED BY SLL
HWY 69
BOREHOLE TYPE Visual Assessment
COMPILED BY AN
DATUM Geodetic
DATE 2010.03.01 - 2010.03.01
CHECKED BY RPR

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 × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
192.8	Ground surface										
0.0	BEDROCK AT SURFACE.										

ONTMT4S 6121(HLR).GPJ 12/7/11

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

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No HLR09-15

1 OF 1

METRIC

W.P.	5076-06-00	LOCATION	N 5 061 723.9 E 230 847.0 Harris Lake Road Underpass	ORIGINATED BY	SLL
HWY	69	BOREHOLE TYPE	Wash Boring	COMPILED BY	AN
DATUM	Geodetic	DATE	2010.03.03 - 2010.03.03	CHECKED BY	RPR

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
191.4	Ground surface
0.0	**ICE** and **WATER**
190.5 1.0	**PEAT**, fibrous: (200mm) Dark Brown Wet
190.2 1.2	**SAND**, trace silt, rootlets Very Loose Dark Brown Wet
189.7 1.7	Clayey **SILT**, trace rootlets Very Soft to Stiff Brown
189.1 2.4	**SAND**, some silt to silty, trace gravel, trace clay Dense Brown Moist
186.6 4.8	**BEDROCK**, granitic gneiss, fresh, grey with pink and white bands, thinly bedded Coring started at 4.8m
183.6 7.8	END OF BOREHOLE AT 7.8m. BOREHOLE BACKFILLED WITH HOLEPLUG TO SURFACE.

SAMPLES

NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	PH			
1	SS	1		
2	SS	13		4 58 34 4
3	SS	19		
4	SS	18/	FI	
1	RUN	.075	0	TCR=83% SCR=83% RQD=83%
2	RUN		0 1 0	TCR=100% SCR=100% RQD=100% UCS=174MPa (Average)
3	RUN		0 0 0 0	TCR=100% SCR=100% RQD=100% UCS=172MPa (Average)
4	RUN		0	TCR=68% SCR=68% RQD=68% UCS=197MPa (Average)

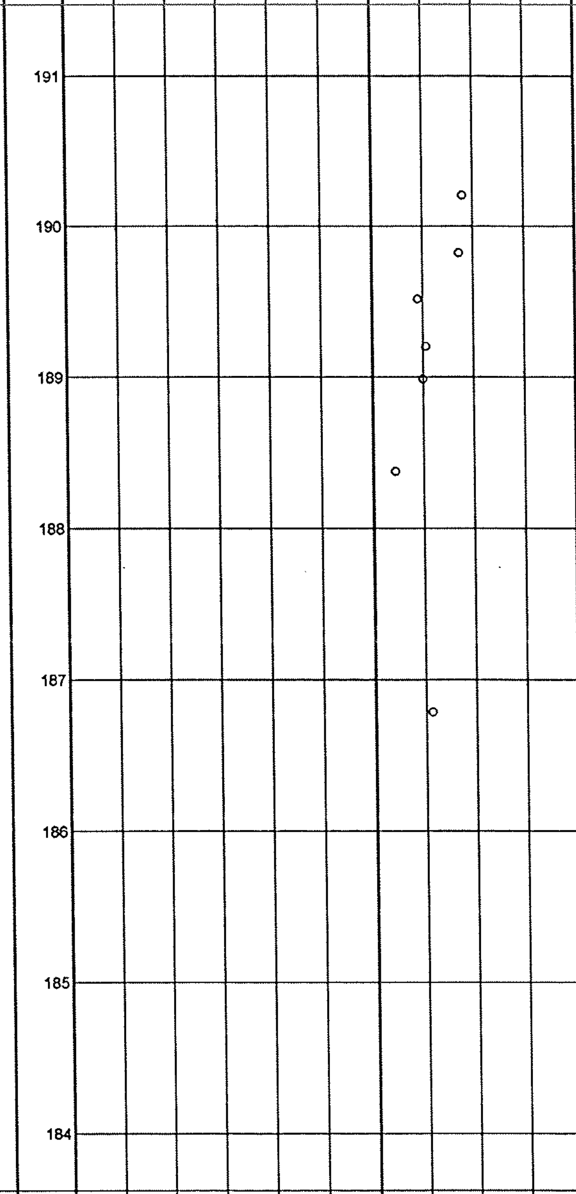
STRAT PLOT — GROUND WATER CONDITIONS — ELEVATION SCALE

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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

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

WATER CONTENT (%) 20 40 60



+³, ×³: Numbers refer to Sensitivity

15-⊕-5 (20 above, 10 below) (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

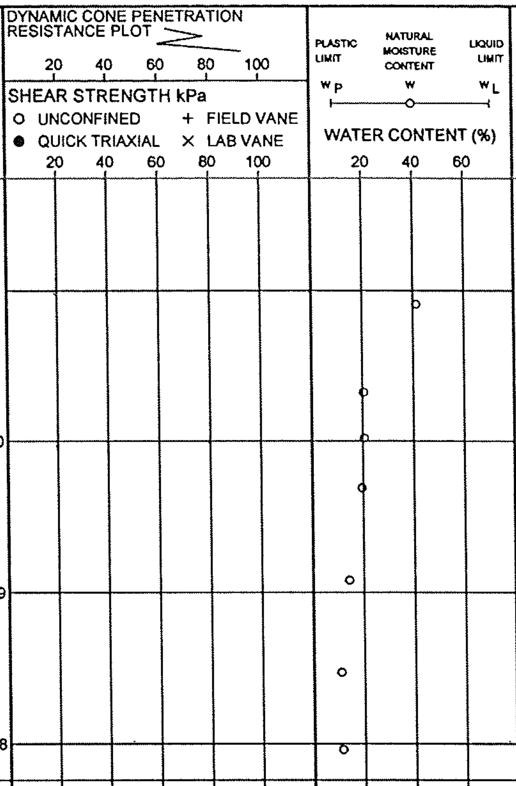
THURBER

RECORD OF BOREHOLE No HLR09-16

1 OF 1 **METRIC**

W.P.	5076-06-00	LOCATION	N 5 061 724.7 E 230 852.5 Harris Lake Road Underpass	ORIGINATED BY	SLL
HWY	69	BOREHOLE TYPE	Wash Boring	COMPILED BY	AN
DATUM	Geodetic	DATE	2010.02.03 - 2010.02.03	CHECKED BY	RPR

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
191.7	Ground surface											
0.0 191.3	**ICE and WATER:** (400mm)											
0.4 191.0	**PEAT**, fibrous: (200mm) Dark Brown Wet						191					
0.7	**SAND**, trace silt, rootlets Very Loose Dark Brown Wet		1	SS	0							
189.9			2	SS	0		190					
1.8 189.3	Clayey **SILT**, trace rootlets Very Soft Grey		3	SS	1							
2.4	**SILT**, some sand to sandy, trace clay Compact Grey Moist		4	SS	20		189		0	26	71	3
			5	SS	16							
187.7			6	SS	35/250		188					
4.0	END OF BOREHOLE AT 4.0m UPON AUGER REFUSAL ON PROBABLE BEDROCK. BOREHOLE OPEN TO 3.1m. BOREHOLE BACKFILLED WITH HOLEPLUG TO 0.6m.											



+³, ×³: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

ONTMT4S 6121(HLR).GPJ 12/7/11

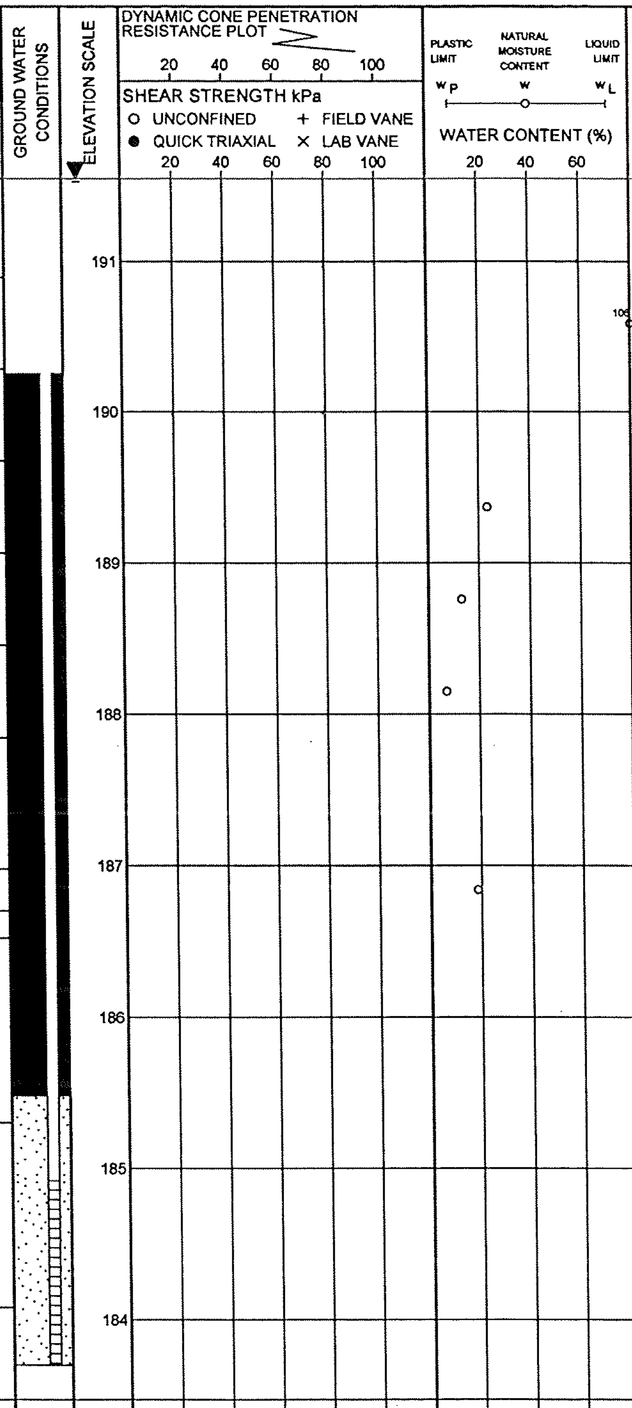
Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No HLR09-17 1 OF 1 METRIC

W.P. 5076-06-00
LOCATION N 5 061 718.6 E 230 849.1 Harris Lake Road Underpass
ORIGINATED BY SLL
HWY 69
BOREHOLE TYPE Wash Boring
COMPILED BY AN
DATUM Geodetic
DATE 2010.03.02 - 2010.03.02
CHECKED BY RPR

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
191.5	Ground surface					
0.0	**ICE and WATER**					
190.8 0.7	**PEAT**, fibrous: (200mm) Dark Brown Wet	1	WH			
190.6 0.9	**SAND**, trace to some silt, trace clay, occasional rootlets Very Loose Dark Brown Wet	1	SS	0		0 80 20 (SI+CL)
189.5 2.0	Clayey **SILT**, sandy, occasional rootlets Very Soft to Very Stiff Grey	2	SS	1		
		3	SS	9		
		4	SS	20		5 63 32 (SI+CL)
187.5 4.0	**SAND**, trace silt, occasional cobbles Grey Wet	5	SS	21/ .125	FI	
186.5 5.0	**BEDROCK**, granitic gneiss, slightly weathered to fresh, dark grey with pink and white bands, mechanical and sub-vertical breaks Coring started at 5.0m	1	RUN		1 1 3 1	TCR=100% SCR=100% RQD=98% UCS=172MPa (Average)
		2	RUN		4 2 3 3	TCR=100% SCR=100% RQD=75% UCS=166MPa (Average)
		3	RUN		3	TCR=63% SCR=63% RQD=63% UCS=195MPa (Average)
183.4 8.1	END OF BOREHOLE AT 8.0m. BOREHOLE BACKFILLED WITH HOLEPLUG TO SURFACE. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Apr 27/11 Surface					



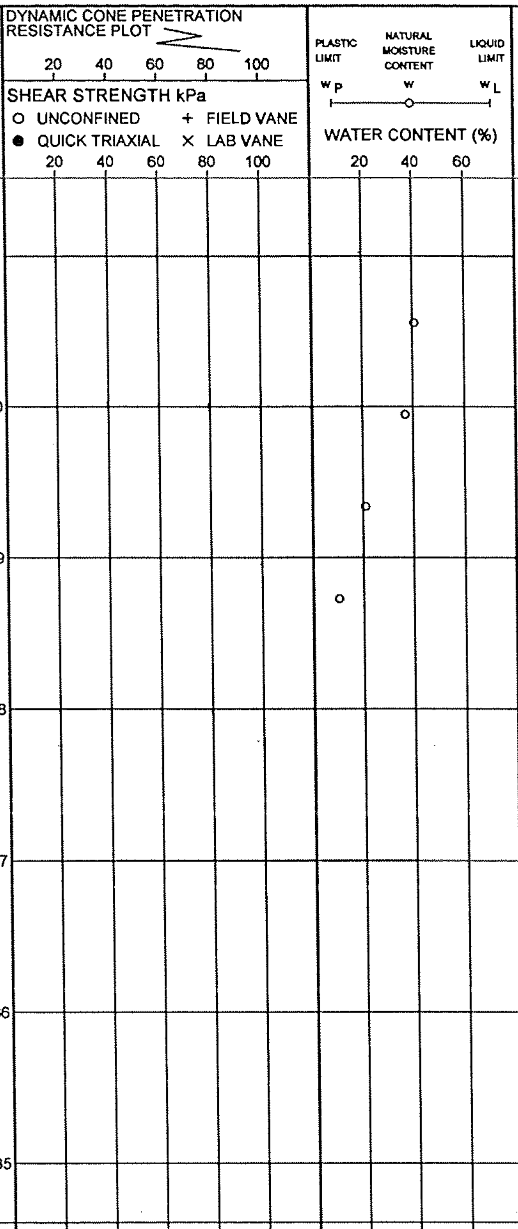
+³, ×³: Numbers refer to Sensitivity

$15 \overset{20}{\underset{10}{\oplus}} 5$ (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No HLR09-18

1 OF 1 **METRIC**

W.P. 5076-06-00	LOCATION N 5 061 721.0 E 230 853.8 Harris Lake Road Underpass		ORIGINATED BY SLL
HWY 69	BOREHOLE TYPE Wash Boring		COMPILED BY AN
DATUM Geodetic	DATE 2010.03.02 - 2010.03.02		CHECKED BY RPR



ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	ELEVATION SCALE
191.5	Ground surface	
0.0	**ICE and WATER:** (700mm)	191
190.8 190.7 0.7 0.8	**PEAT**, fibrous: (100mm) Brown Wet	
	SILT, some sand to sandy, trace clay, occasional rootlets Very Loose Dark Brown Wet	190
	Compact	189 188
187.6 3.9	**BEDROCK**, granitic gneiss, slightly weathered to fresh, dark grey with pink and white bands, mechanical and sub-vertical breaks Coring astarted at 3.9m Horizontal joints (greater than 25mm) at: 175mm at 4.0m 100mm at 4.5m	187 186
184.6	Vertical joints (greater than 25mm): 50mm at 6.5m	185
6.9	END OF BOREHOLE AT 6.9m. BOREHOLE BACKFILLED WITH HOLEPLUG TO SURFACE.	

SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	0			
2	SS	0			
3	SS	1			0 27 69 4
4	SS	14			
1	RUN			FI 1 2 2 1	TCR=100% SCR=100% RQD=88% UCS=135MPa (Average)
2	RUN			2 1 1 2	TCR=100% SCR=100% RQD=98% UCS=138MPa (Average)
3	RUN			1	TCR=100% SCR=100% RQD=88% UCS=208MPa (Average)

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

20 15-⌖-5 10 (%) STRAIN AT FAILURE

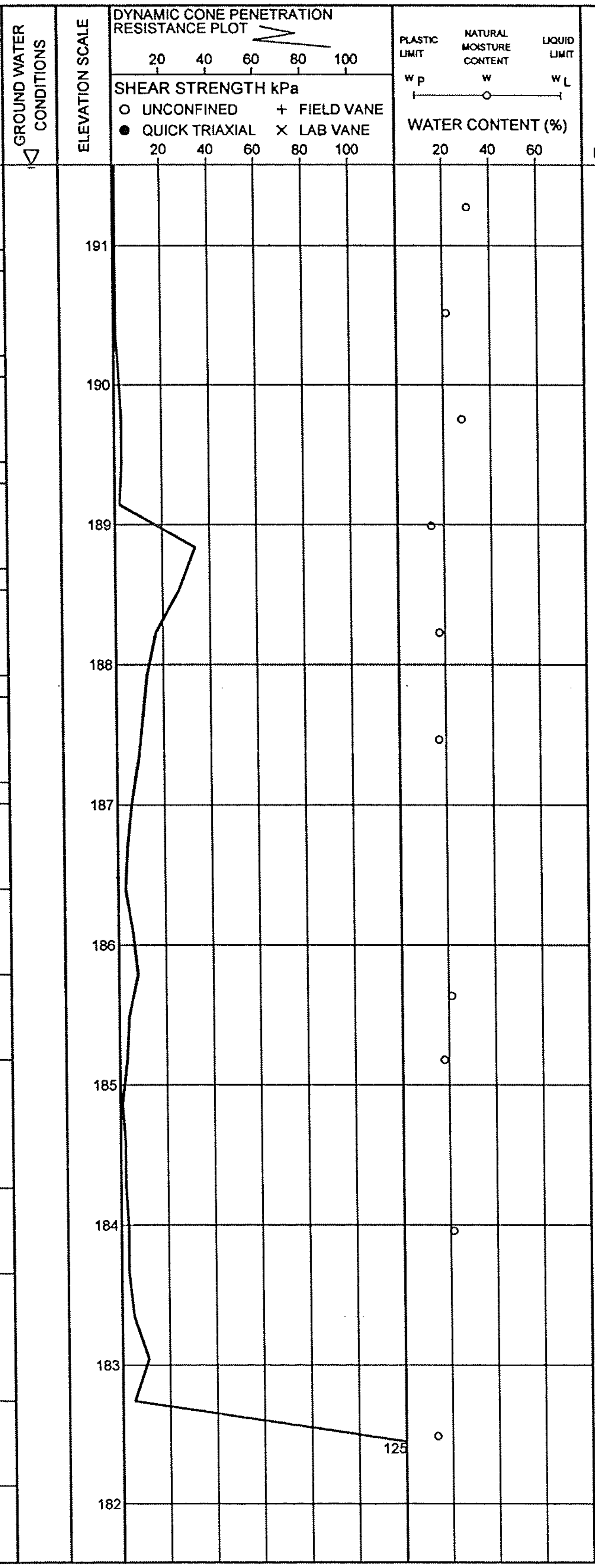
RECORD OF BOREHOLE No HLR09-19

1 OF 2

METRIC

W.P. 5076-06-00
LOCATION N 5 061 686.0 E 230 754.7 Harris Lake Road Underpass
ORIGINATED BY ES
HWY 69
BOREHOLE TYPE Hollow Stem Augers / Casing
COMPILED BY AN
DATUM Geodetic
DATE 2009.08.21 - 2009.08.21
CHECKED BY RPR

SOIL PROFILE		SAMPLES			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
191.5	Ground surface							
0.0	**PEAT:** (35mm)							
	SAND, some silt to silty, trace clay Very Loose to Loose Greyish Brown Moist	1	SS	1				
		2	SS	1	0	86	14 (SI+CL)	
		3	SS	1				
		4	SS	1	0	57	37	6
	Occasional oxide staining	5	SS	9				
	Coarse grained, trace silt, trace gravel	6	SS	3	4	93	3 (SI+CL)	
		7	SS	9				
		8	SS	3				
		9	SS	2	0	99	1 (SI+CL)	
		10	SS	34				
182.3 9.2	**BEDROCK**, granitic gneiss, slightly weathered to fresh, grey							



Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

15-⊕-5 (20 above, 10 below) (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No HLR09-19

2 OF 2

METRIC

W.P.	5076-06-00	LOCATION	N 5 061 686.0 E 230 754.7 Harris Lake Road Underpass	ORIGINATED BY	ES
HWY	69	BOREHOLE TYPE	Hollow Stem Augers / Casing	COMPILED BY	AN
DATUM	Geodetic	DATE	2009.08.21 - 2009.08.21	CHECKED BY	RPR

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 × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page										
181.2	**BEDROCK**, granitic gneiss, grey		11	SS	100/						
10.3	END OF BOREHOLE AT 10.3m UPON AUGER REFUSAL ON PROBABLE BEDROCK. WATER LEVEL AT SURFACE UPON COMPLETION. BOREHOLE BACKFILLED WITH HOLEPLUG TO 3.0m, THEN AUGER CUTTINGS TO SURFACE.				.100		181				

ONTMT4S 6121(HLR).GPJ 12/7/11

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

$15\text{-}\oplus\text{-}5$ (20 above, 10 below) (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No HLR09-20

1 OF 1

METRIC

W.P.	5076-06-00	LOCATION	N 5 061 690.2 E 230 762.2 Harris Lake Road Underpass	ORIGINATED BY	ES
HWY	69	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	AN
DATUM	Geodetic	DATE	2009.08.21 - 2009.08.21	CHECKED BY	RPR

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES				GR	SA	SI	CL
192.0	Ground surface					192					
0.0 0.0	**PEAT:** (35mm)										
	SAND, fine granined Very Loose to Loose Brown Moist	1	SS	2	∇						
		2	SS	5		191					
190.7 1.3	Clayey **SILT**, sandy Firm to Soft Grey	3	SS	3				0	28	57	15
189.8 2.1	**SAND**, fine grained, trace to some gravel, some silt, trace clay Compact to Loose Brown to Grey Wet					190					
		4	SS	20		189					
		5	SS	11		188		1	79	20 (SI+CL)	
		6	SS	8		187					
						186					
	Coarse grained, trace silt Very Loose to Loose	7	SS	3				12	86	2 (SI+CL)	
						185					
		8	SS	9							
		9	SS	100/ .100		184					
183.8 8.2	END OF BOREHOLE AT 8.2m UPON AUGER REFUSAL ON PROBABLE BEDROCK. WATER LEVEL AT 0.3m UPON COMPLETION. BOREHOLE BACKFILLED WITH HOLEPLUG TO 1.2m, THEN AUGER CUTTINGS TO SURFACE.										

+³, ×³: Numbers refer to Sensitivity

15-ф-5 20 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

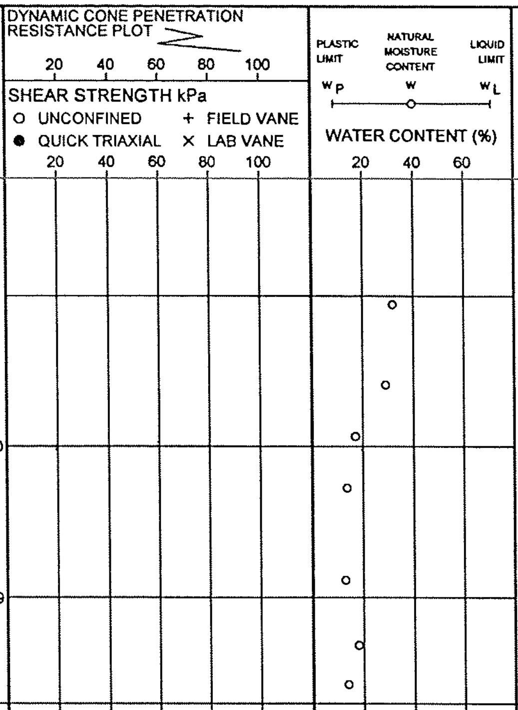
RECORD OF BOREHOLE No HLR09-21

1 OF 1

METRIC

W.P. 5076-06-00
LOCATION N 5 061 726.4 E 230 857.7 Harris Lake Road Underpass
ORIGINATED BY SLL
HWY 69
BOREHOLE TYPE Wash Boring
COMPILED BY AN
DATUM Geodetic
DATE 2010.03.01 - 2010.03.01
CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES				GR	SA	SI	CL
191.7	Ground surface											
0.0 191.2	**ICE and WATER:** (500mm)	▨										
0.5	**SAND**, some silt to silty, trace clay, occasional rootlets Very Loose to Compact Dark Brown Wet Occasional cobbles		1	SS	1		191		0	73	22	5
			2	SS	29							
	Dense Brown Moist		3	SS	33		190					
	Loose		4	SS	5		189					
			5	SS	36/ .100							
188.3			6	SS	36/ .100							
3.5	END OF BOREHOLE AT 3.5m UPON AUGER REFUSAL ON PROBABLE BEDROCK. BOREHOLE BACKFILLED WITH HOLELPUG TO SURFACE.											



+ ³, × ³: Numbers refer to Sensitivity

$15 \overset{20}{\underset{10}{\oplus}} 5$ (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

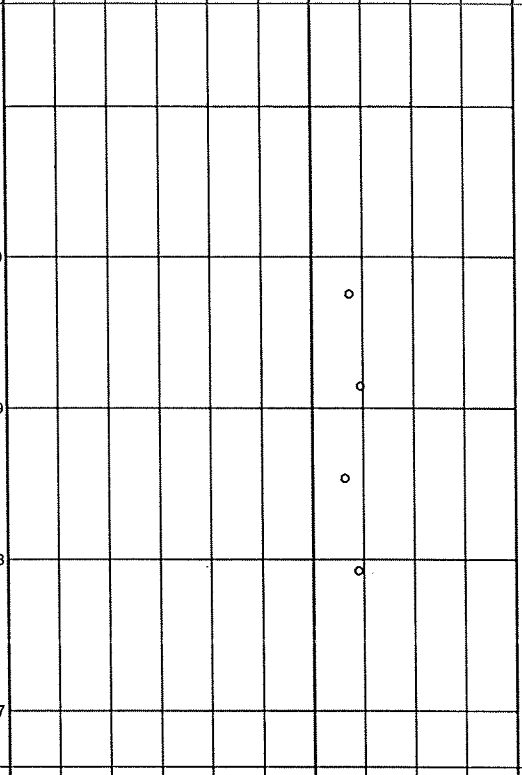
RECORD OF BOREHOLE No HLR09-22 1 OF 1 METRIC

W.P. 5076-06-00
LOCATION N 5 061 728.5 E 230 862.0 Harris Lake Road Underpass
ORIGINATED BY SLL
HWY 69
BOREHOLE TYPE Wash Boring
COMPILED BY AN
DATUM Geodetic
DATE 2010.03.01 - 2010.03.01
CHECKED BY RPR

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
191.6	Ground surface											
0.0	**ICE and WATER**						191					
190.0 1.6	**SAND**, some silt, some gravel, trace clay, occasional cobbles and wood fragments Loose to Compact Brown Wet		1	SS	8		190					
			2	SS	7		189		11	67	22 (SI+CL)	
			3	SS	6							
			4	SS	11		188					
							187					
186.6 5.1	END OF BOREHOLE AT 5.1m UPON AUGER REFUSAL ON PROBABLE BEDROCK. BOREHOLE BACKFILLED WITH HOLEPLUG TO SURFACE.		5	SS	36/ .075							

DYNAMIC CONE PENETRATION RESISTANCE PLOT
20 40 60 80 100
SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE
20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L
WATER CONTENT (%)
20 40 60



$+^3, \times^3$: Numbers refer to Sensitivity
$15\overset{20}{\underset{10}{\oplus}}5$ (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

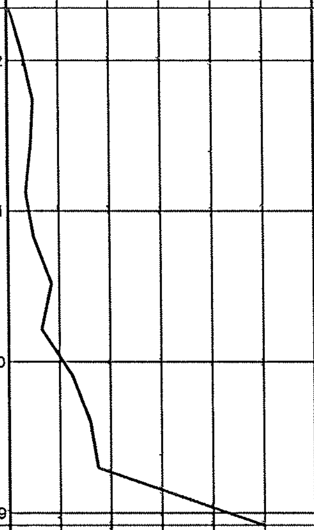
THURBER

RECORD OF BOREHOLE No DHL-01

1 OF 1

METRIC

W.P.	5076-06-00	LOCATION	N 5 061 699.9 E 230 771.7 Harris Lake Road Underpass	ORIGINATED BY	ES
HWY	69	BOREHOLE TYPE	Dynamic Cone Penetration Test (DCPT)	COMPILED BY	AN
DATUM	Geodetic	DATE	2009.08.24 - 2009.08.24	CHECKED BY	RPR

SOIL PROFILE			SAMPLES						DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE					
192.3	Ground surface											
0.0	DCPT from surface.						192 191 190 189					
188.9 3.4	END OF DCPT AT 3.4m UPON REFUSAL ON PROBABLE BEDROCK.											

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

$15 \overset{20}{\underset{10}{\oplus}} 5$ (%) STRAIN AT FAILURE

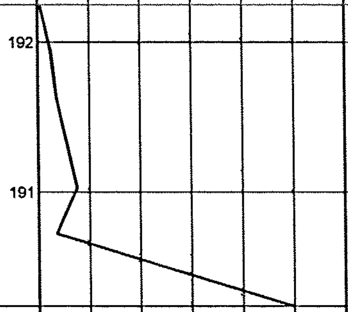
RECORD OF BOREHOLE No DHL-02

1 OF 1

METRIC

W.P.	5076-06-00	LOCATION	N 5 061 694.9 E 230 773.7 Harris Lake Road Underpass	ORIGINATED BY	ES
HWY	69	BOREHOLE TYPE	Dynamic Cone Penetration Test (DCPT)	COMPILED BY	AN
DATUM	Geodetic	DATE	2009.08.24 - 2009.08.24	CHECKED BY	RPR

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 × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
192.2	Ground surface										
0.0	DCPT from surface.						192 191				
190.2 2.0	END OF DCPT AT 2.0m UPON REFUSAL ON PROBABLE BEDROCK.										



+3, ×3: Numbers refer to Sensitivity

20
15-⊕-5
10
(%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

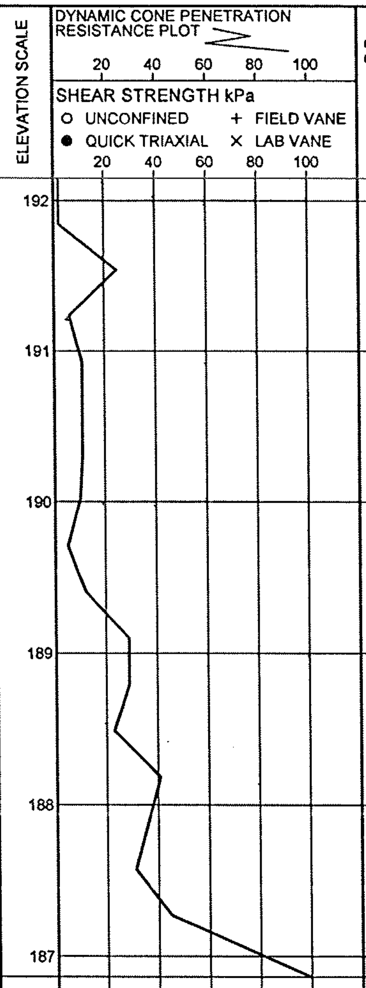
THURBER

RECORD OF BOREHOLE No DHL-03

1 OF 1 **METRIC**

W.P.	5076-06-00	LOCATION	N 5 061 690.1 E 230 776.7 Harris Lake Road Underpass	ORIGINATED BY	ES
HWY	69	BOREHOLE TYPE	Dynamic Cone Penetration Test (DCPT)	COMPILED BY	AN
DATUM	Geodetic	DATE	2009.08.24 - 2009.08.24	CHECKED BY	RPR

SOIL PROFILE			SAMPLES								
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L / WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
192.1	Ground surface										
0.0	DCPT from surface.										
186.8 5.3	END OF DCPT AT 5.3m UPON REFUSAL ON PROBABLE BEDROCK.										



WATER CONTENT (%) 20 40 60

ONTMT4S 6121(HLR).GPJ 12/7/11

+³, ×³: Numbers refer to Sensitivity

$15 \overset{20}{\underset{10}{\oplus}} 5$ (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

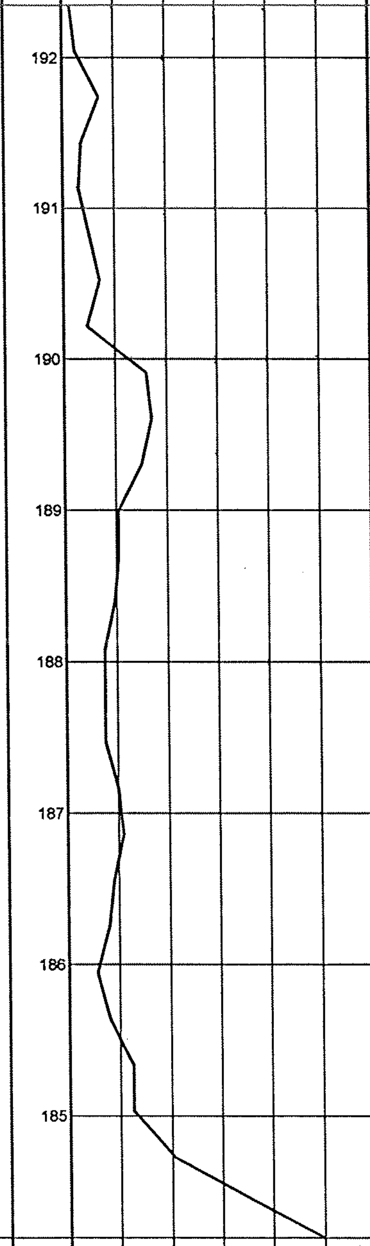
THURBER

RECORD OF BOREHOLE No DHL-04

1 OF 1 **METRIC**

W.P. 5076-06-00
LOCATION N 5 061 699.2 E 230 767.2 Harris Lake Road Underpass
ORIGINATED BY ES
HWY 69
BOREHOLE TYPE Dynamic Cone Penetration Test (DCPT)
COMPILED BY AN
DATUM Geodetic
DATE 2009.08.24 - 2009.08.24
CHECKED BY RPR

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
192.3	Ground surface										
0.0	DCPT from surface.										
184.1 8.2	END OF DCPT AT 8.2m UPON REFUSAL ON PROBABLE BEDROCK.										



+³, ×³: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

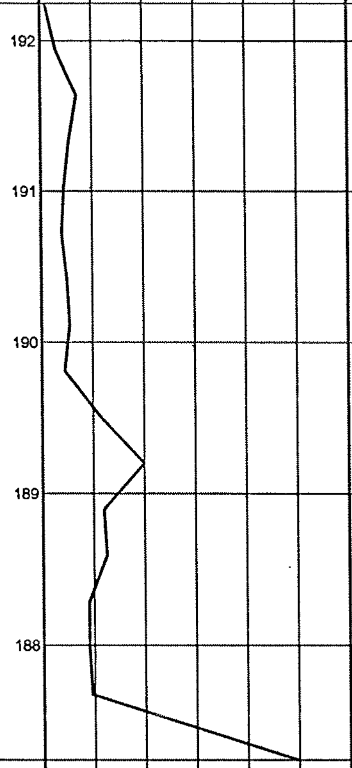
RECORD OF BOREHOLE No DHL-05

1 OF 1

METRIC

W.P.	5076-06-00	LOCATION	N 5 061 695.2 E 230 768.8 Harris Lake Road Underpass	ORIGINATED BY	ES
HWY	69	BOREHOLE TYPE	Dynamic Cone Penetration Test (DCPT)	COMPILED BY	AN
DATUM	Geodetic	DATE	2009.08.24 - 2009.08.24	CHECKED BY	RPR

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 × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
192.2	Ground surface										
0.0	DCPT from surface.										
187.2 5.0	END OF DCPT AT 5.0m UPON REFUSAL ON PROBABLE BEDROCK.										



ONTMT4S 6121(HLR).GPJ 12/7/11

+³, ×³: Numbers refer to Sensitivity

20 / 15 ⌖ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

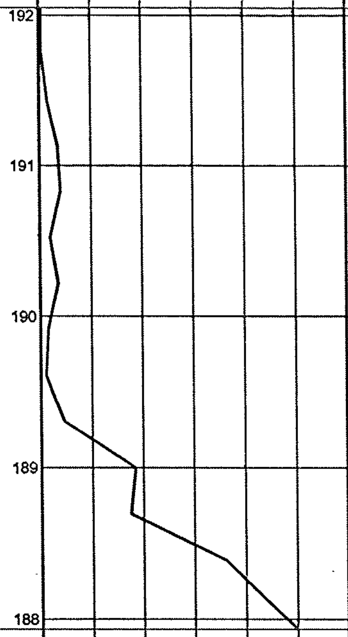
RECORD OF BOREHOLE No DHL-06

1 OF 1

METRIC

W.P. 5076-06-00
LOCATION N 5 061 691.2 E 230 770.4 Harris Lake Road Underpass
ORIGINATED BY ES
HWY 69
BOREHOLE TYPE Dynamic Cone Penetration Test (DCPT)
COMPILED BY AN
DATUM Geodetic
DATE 2009.08.24 - 2009.08.24
CHECKED BY RPR

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
192.0	Ground surface										
0.0	DCPT from surface.						192 191 190 189 188				
187.9 4.1	END OF DCPT AT 4.1m UPON REFUSAL ON PROBABLE BEDROCK.										



+³, ×³: Numbers refer to Sensitivity

15-20-5-10 (%) STRAIN AT FAILURE

ONTMT4S 6121(HLR).GPJ 12/7/11

Ministry of Transportation
Ontario

THURBER

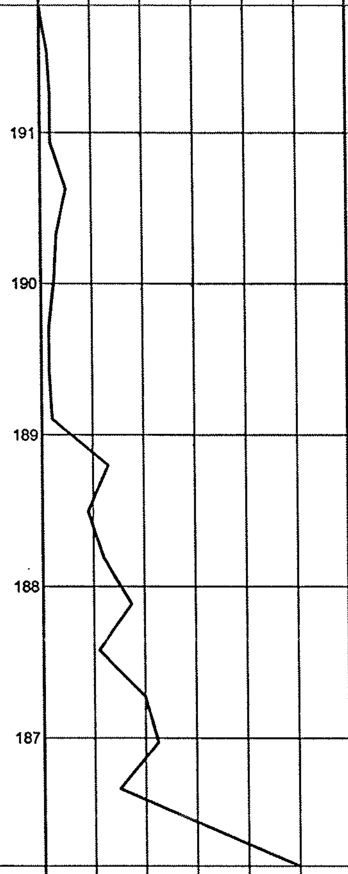
RECORD OF BOREHOLE No DHL-07

1 OF 1

METRIC

W.P.	5076-06-00	LOCATION	N 5 061 687.1 E 230 772.0 Harris Lake Road Underpass	ORIGINATED BY	ES
HWY	69	BOREHOLE TYPE	Dynamic Cone Penetration Test (DCPT)	COMPILED BY	AN
DATUM	Geodetic	DATE	2009.08.24 - 2009.08.24	CHECKED BY	RPR

SOIL PROFILE			SAMPLES									
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
191.8	Ground surface										
0.0	DCPT from surface.						191 190 189 188 187				
186.1 5.7	END OF DCPT AT 5.7m UPON REFUSAL ON PROBABLE BEDROCK.										



+³, ×³: Numbers refer to Sensitivity

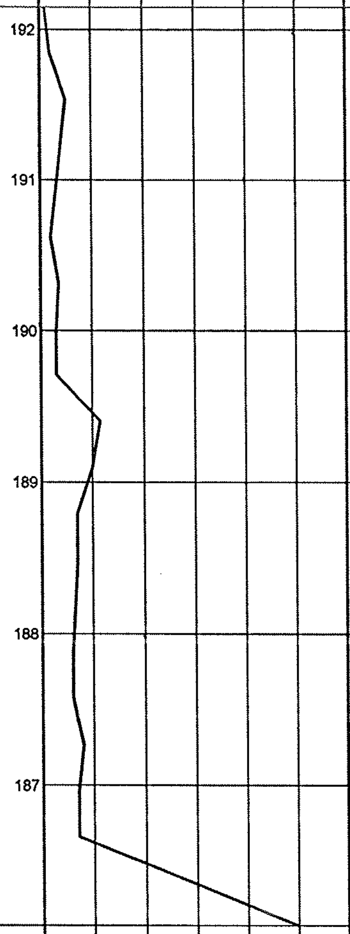
20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No DHL-08

1 OF 1 **METRIC**

W.P. 5076-06-00
LOCATION N 5 061 696.6 E 230 763.4 Harris Lake Road Underpass
ORIGINATED BY ES
HWY 69
BOREHOLE TYPE Dynamic Cone Penetration Test (DCPT)
COMPILED BY AN
DATUM Geodetic
DATE 2009.08.24 - 2009.08.24
CHECKED BY RPR

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
192.1	Ground surface										
0.0	DCPT from surface.						192 191 190 189 188 187				
186.0 6.1	END OF DCPT AT 6.1m UPON REFUSAL ON PROBABLE BEDROCK.										



+³, ×³: Numbers refer to Sensitivity

15-⌖-5 (20 above, 10 below) (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

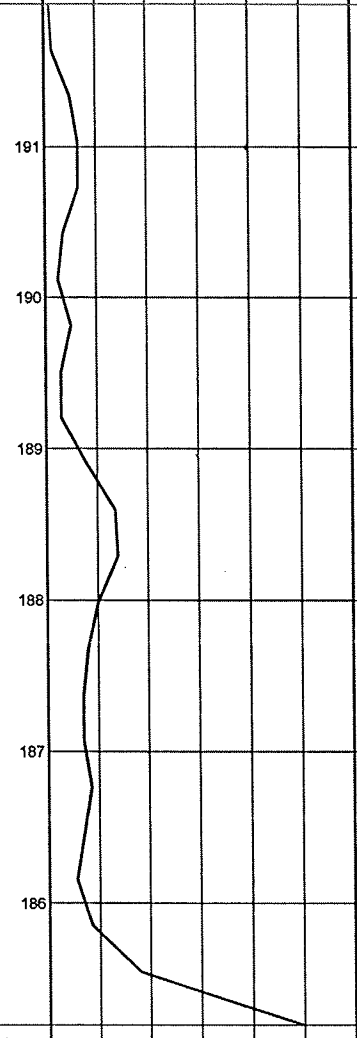
THURBER

RECORD OF BOREHOLE No DHL-09

1 OF 1 METRIC

W.P.	5076-06-00	LOCATION	N 5 061 691.1 E 230 764.5 Harris Lake Road Underpass	ORIGINATED BY	ES
HWY	69	BOREHOLE TYPE	Dynamic Cone Penetration Test (DCPT)	COMPILED BY	AN
DATUM	Geodetic	DATE	2009.08.24 - 2009.08.24	CHECKED BY	RPR

SOIL PROFILE			SAMPLES										
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
191.9	Ground surface												
0.0	DCPT from surface.												
185.1 6.8	END OF DCPT AT 6.7m UPON REFUSAL ON PROBABLE BEDROCK.												



ONTMT4S 6121(HLR).GPJ 12/7/11

+³, ×³: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

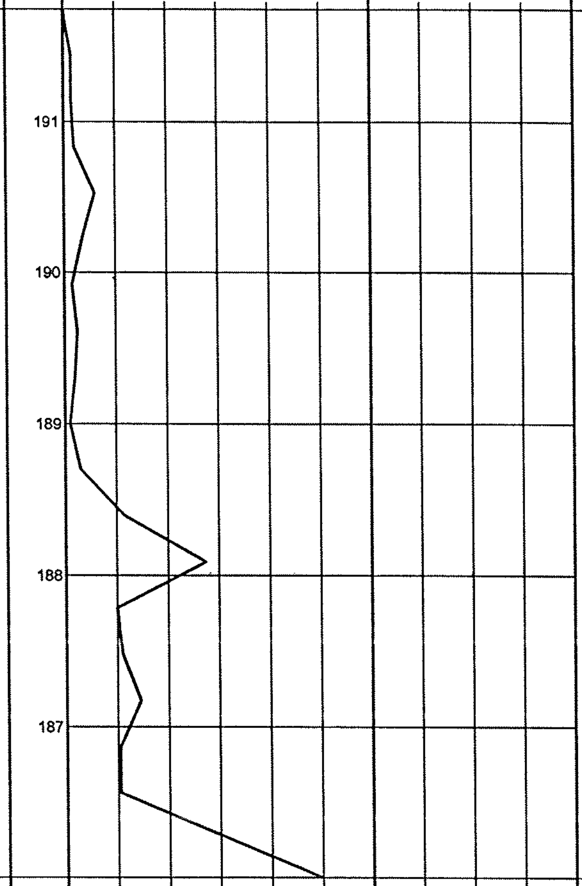
THURBER

RECORD OF BOREHOLE No DHL-10

1 OF 1 **METRIC**

W.P.	5076-06-00	LOCATION	N 5 061 686.0 E 230 766.6 Harris Lake Road Underpass	ORIGINATED BY	ES
HWY	69	BOREHOLE TYPE	Dynamic Cone Penetration Test (DCPT)	COMPILED BY	AN
DATUM	Geodetic	DATE	2009.08.24 - 2009.08.24	CHECKED BY	RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
191.7	Ground surface										
0.0	DCPT from surface.						191 190 189 188 187				
186.0 5.7	END OF DCPT AT 5.7m UPON REFUSAL ON PROBABLE BEDROCK.										



+³, ×³: Numbers refer to Sensitivity

$15 \overset{20}{\underset{10}{\oplus}} 5$ (%) STRAIN AT FAILURE

Appendix B

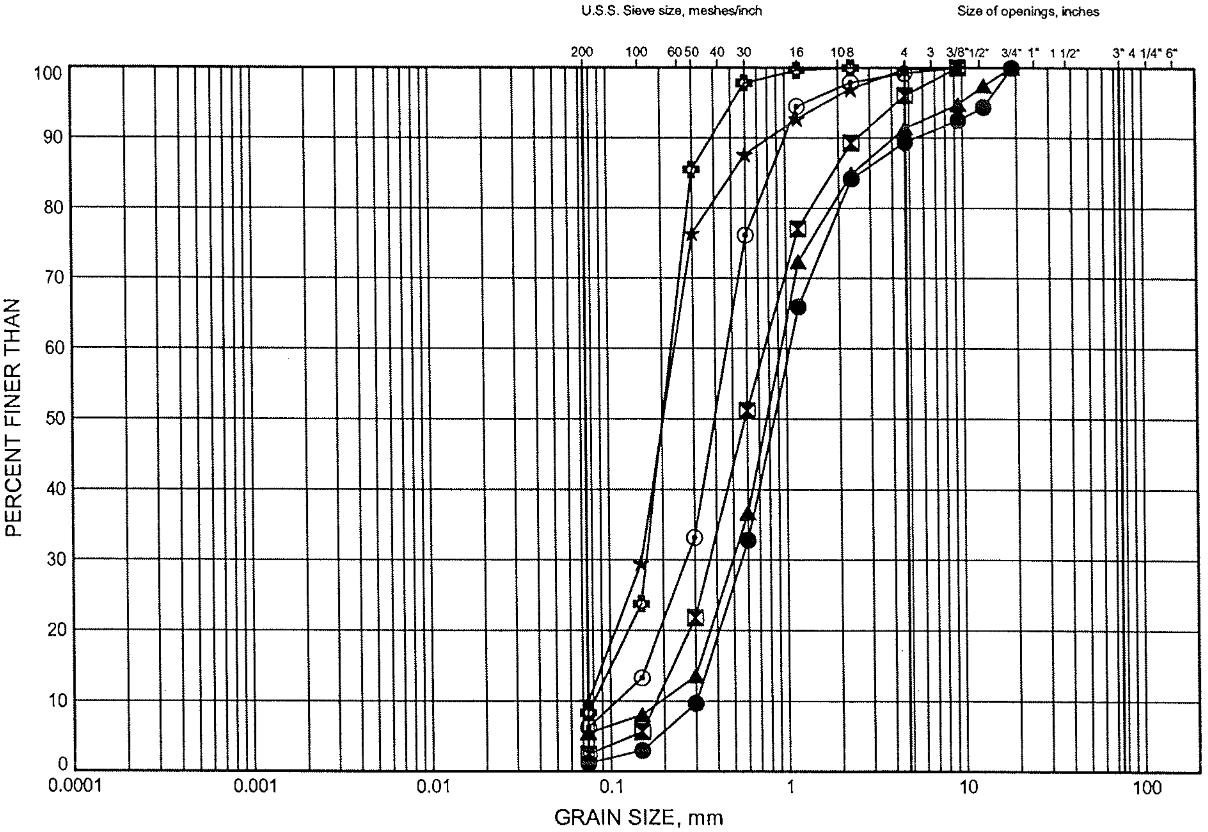
Laboratory Test Results

Hwy 69 Four-Laning North of Hwy 529

GRAIN SIZE DISTRIBUTION

FIGURE B1

SAND



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	HLR09-01	4.88	187.06
☒	HLR09-01	6.95	184.99
▲	HLR09-03	4.84	187.04
★	HLR09-04	0.91	191.04
⊙	HLR09-05	4.88	186.88
✤	HLR09-09	1.32	190.04

W.P.# 5076-06-00

Prepared By MFA

Checked By RPR

THURBER

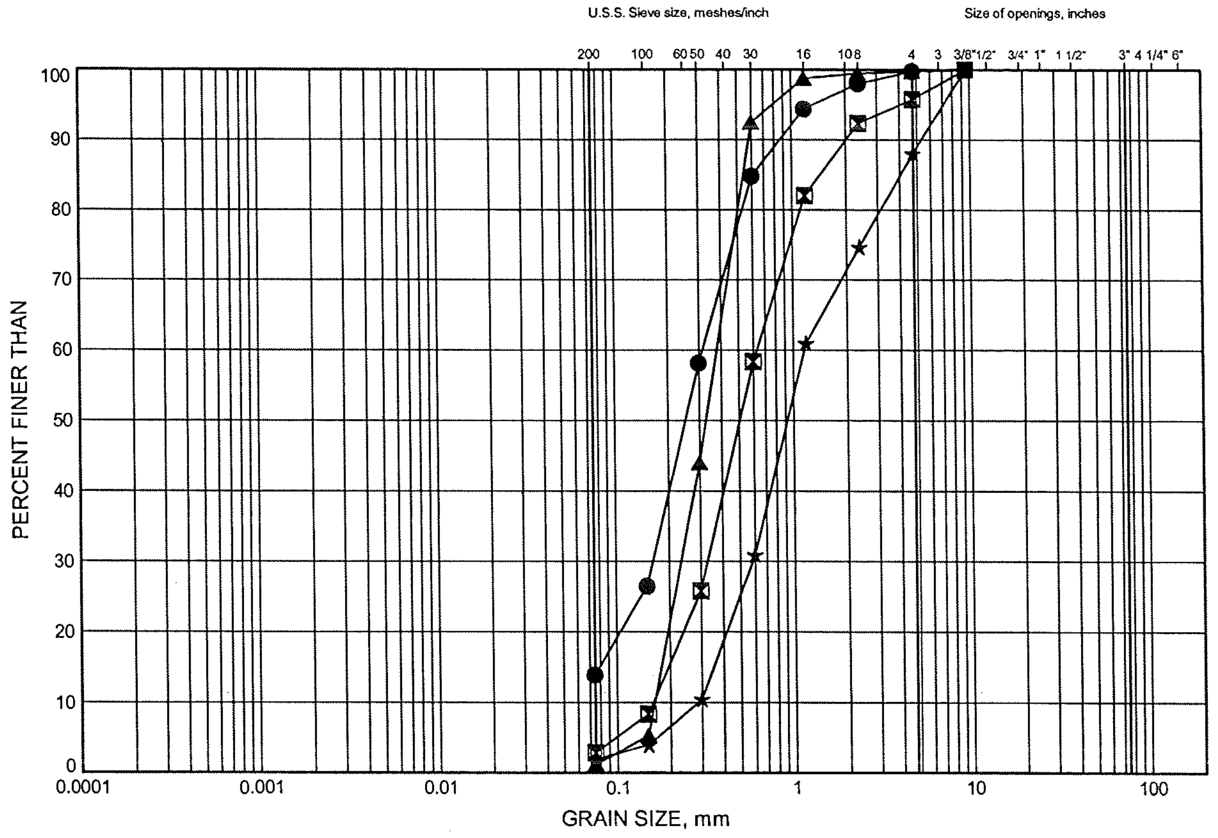
GRAIN SIZE DISTRIBUTION - THURBER 6121(HLR).GPJ 4/15/11

Hwy 69 Four-Laning North of Hwy 529

GRAIN SIZE DISTRIBUTION

FIGURE B2

SAND



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	HLR09-19	1.07	190.47
☒	HLR09-19	4.11	187.42
▲	HLR09-19	7.62	183.91
★	HLR09-20	6.40	185.57

W.P.# 5076-06-00

Prepared By MFA

Checked By RPR

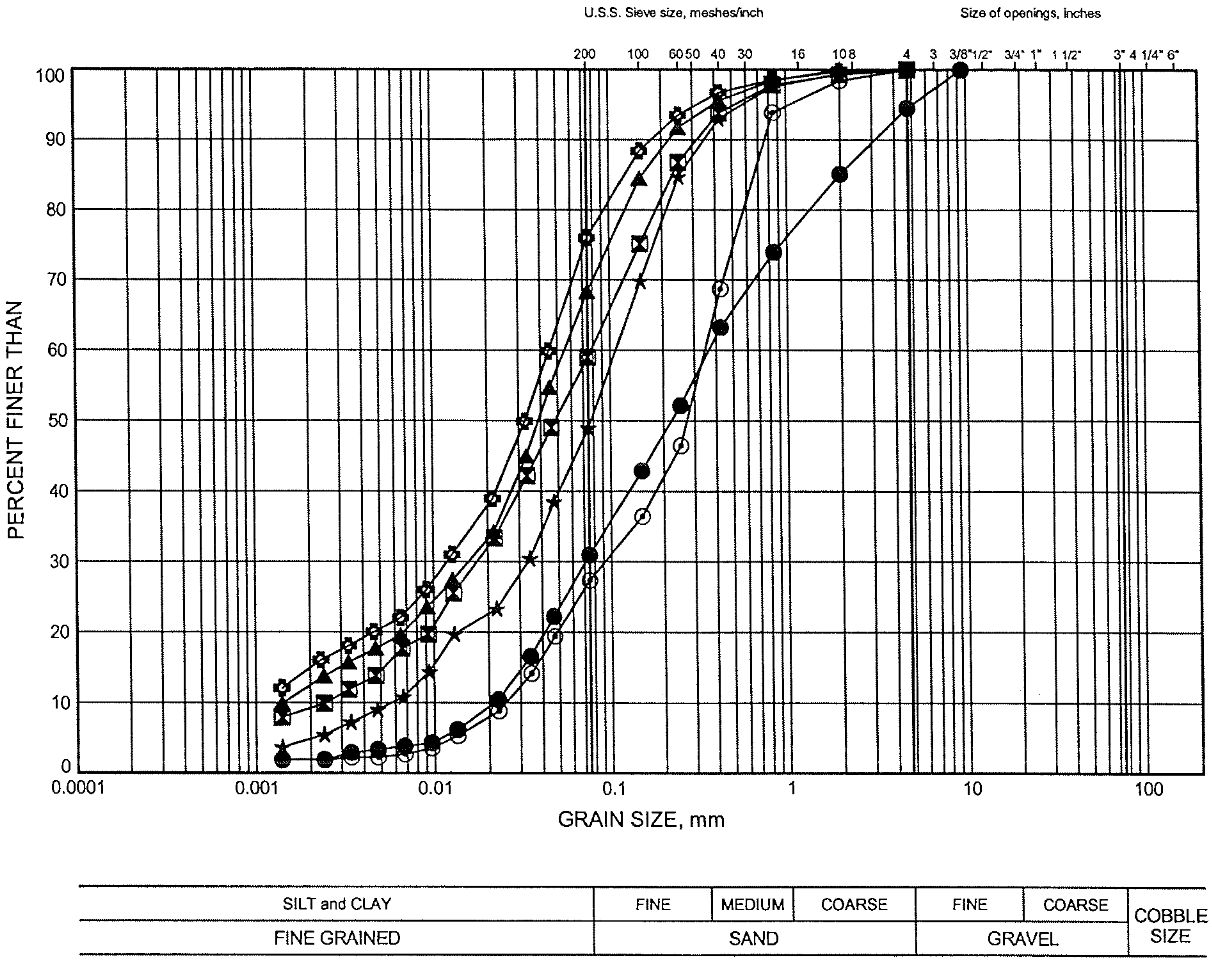
THURBER

Hwy 69 Four-Laning North of Hwy 529

GRAIN SIZE DISTRIBUTION

FIGURE B3

SAND (some silt to silty)



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	HLR09-03	3.35	188.52
☒	HLR09-04	2.34	189.62
▲	HLR09-05	1.75	190.00
★	HLR09-05	3.35	188.40
⊙	HLR09-06	4.11	187.72
✿	HLR09-10	1.45	190.02

W.P.# 5076-06-00
Prepared By MFA
Checked By RPR

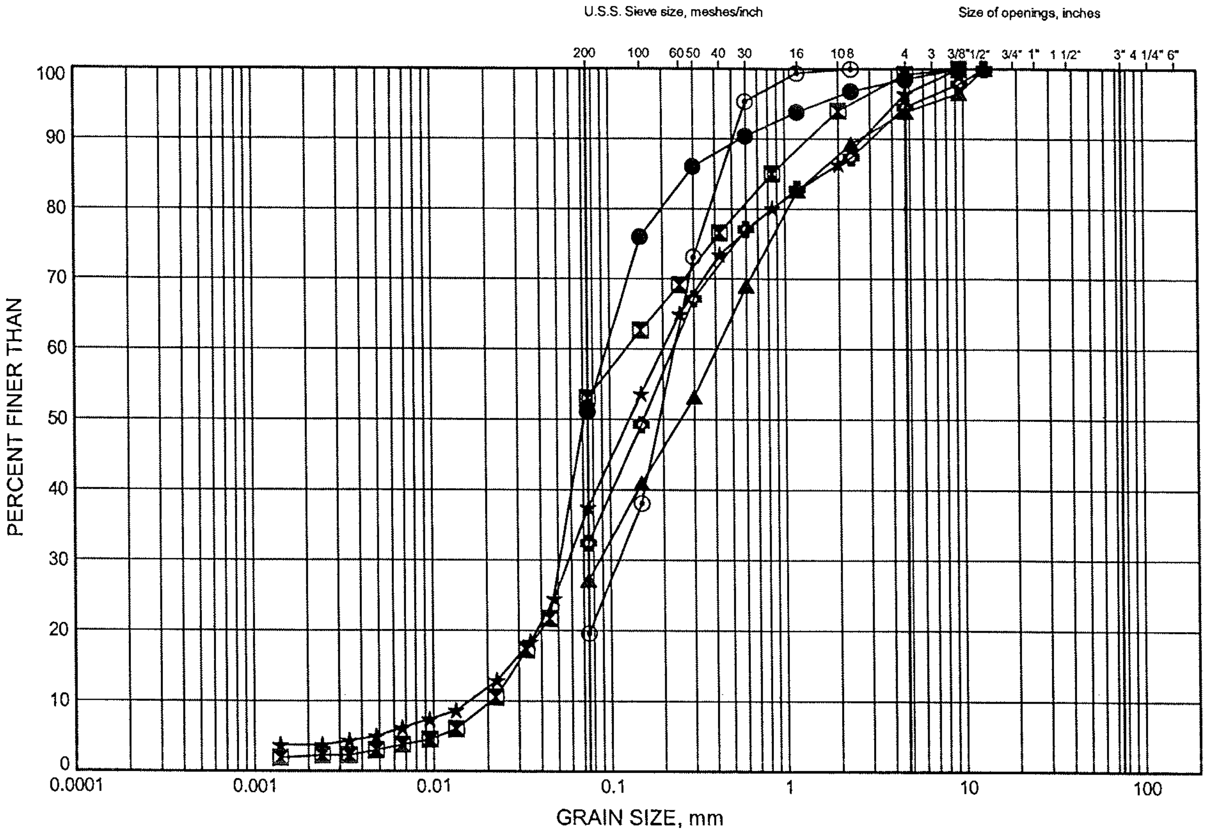
THURBER

Hwy 69 Four-Laning North of Hwy 529

GRAIN SIZE DISTRIBUTION

FIGURE B4

SAND (some silt to silty)



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	HLR09-10	3.89	187.58
⧗	HLR09-11	2.79	188.70
▲	HLR09-11	4.01	187.48
★	HLR09-15	2.49	188.94
⊙	HLR09-17	1.57	189.93
✿	HLR09-17	3.40	188.10

W.P.# 5076-06-00

Prepared By MFA

Checked By RPR

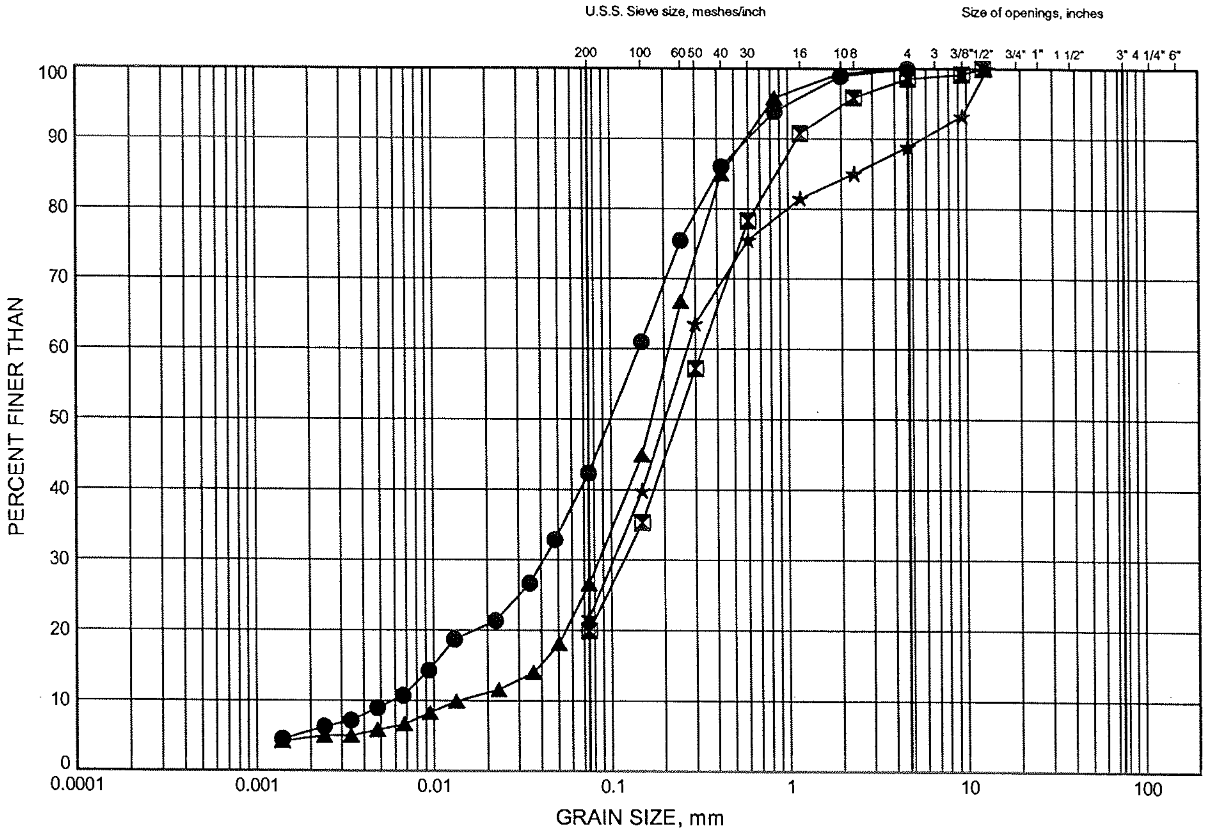
THURBER

Hwy 69 Four-Laning North of Hwy 529

GRAIN SIZE DISTRIBUTION

FIGURE B5

SAND (some silt to silty)



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	HLR09-19	2.59	188.94
⊠	HLR09-20	4.11	187.85
▲	HLR09-21	0.84	190.89
★	HLR09-22	2.54	189.10

W.P.# 5076-06-00

Prepared By MFA

Checked By RPR

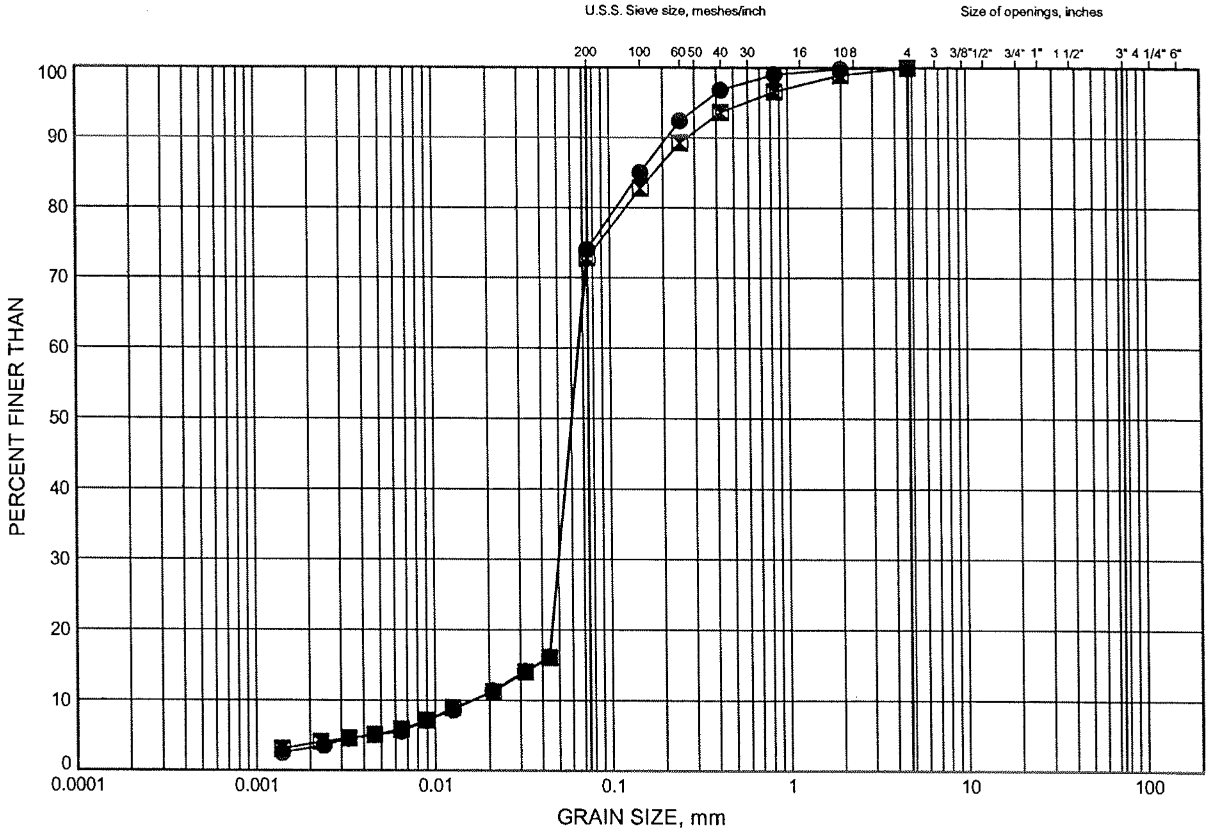
THURBER

Hwy 69 Four-Laning North of Hwy 529

GRAIN SIZE DISTRIBUTION

FIGURE B6

SILT (some sand to sandy)



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	HLR09-16	2.67	189.03
☒	HLR09-18	2.18	189.29

GRAIN SIZE DISTRIBUTION - THURBER 6121(HLR).GPJ 4/15/11

W.P.# 5076-06-00

Prepared By MFA

Checked By RPR

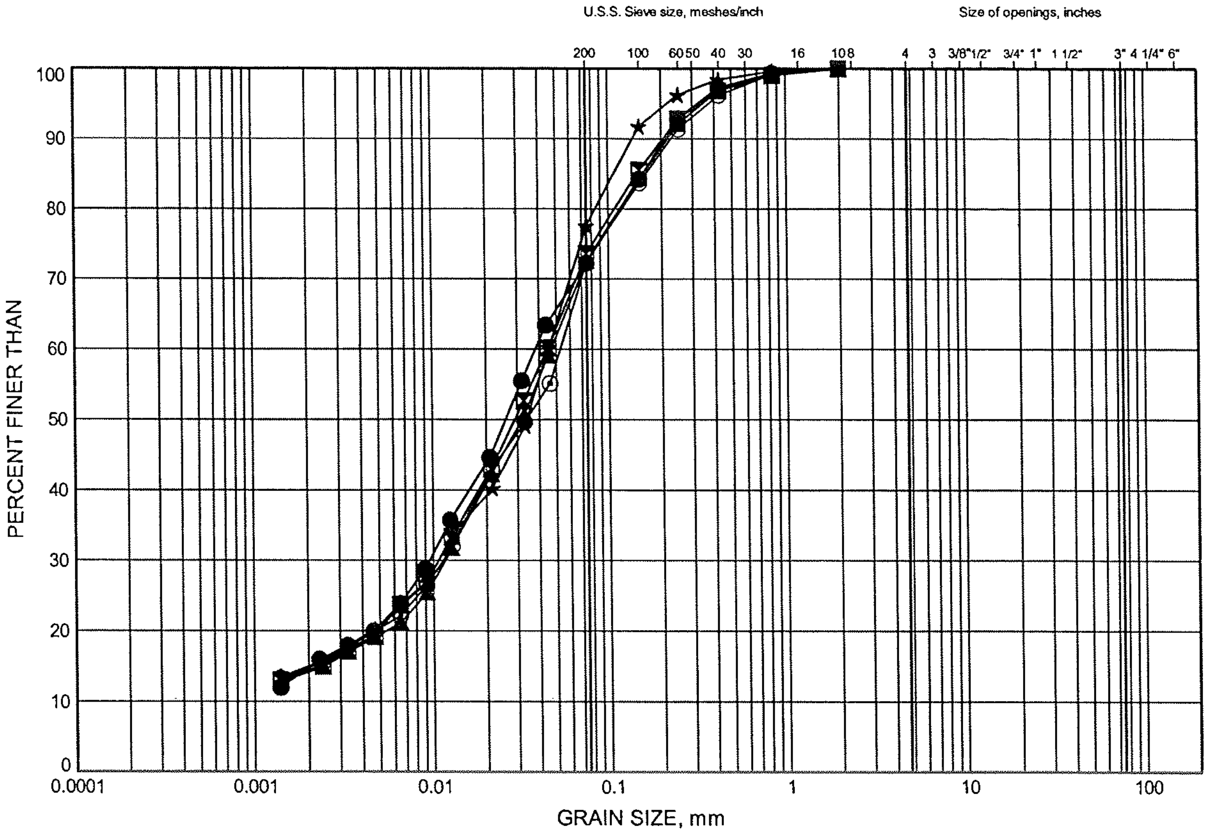
THURBER

Hwy 69 Four-Laning North of Hwy 529

GRAIN SIZE DISTRIBUTION

FIGURE B7

Clayey SILT



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	HLR09-01	1.83	190.11
⊠	HLR09-02	1.83	190.17
▲	HLR09-03	1.83	190.05
★	HLR09-06	0.99	190.85
⊙	HLR09-20	1.83	190.14

W.P.# 5076-06-00

Prepared By MFA

Checked By RPR

THURBER

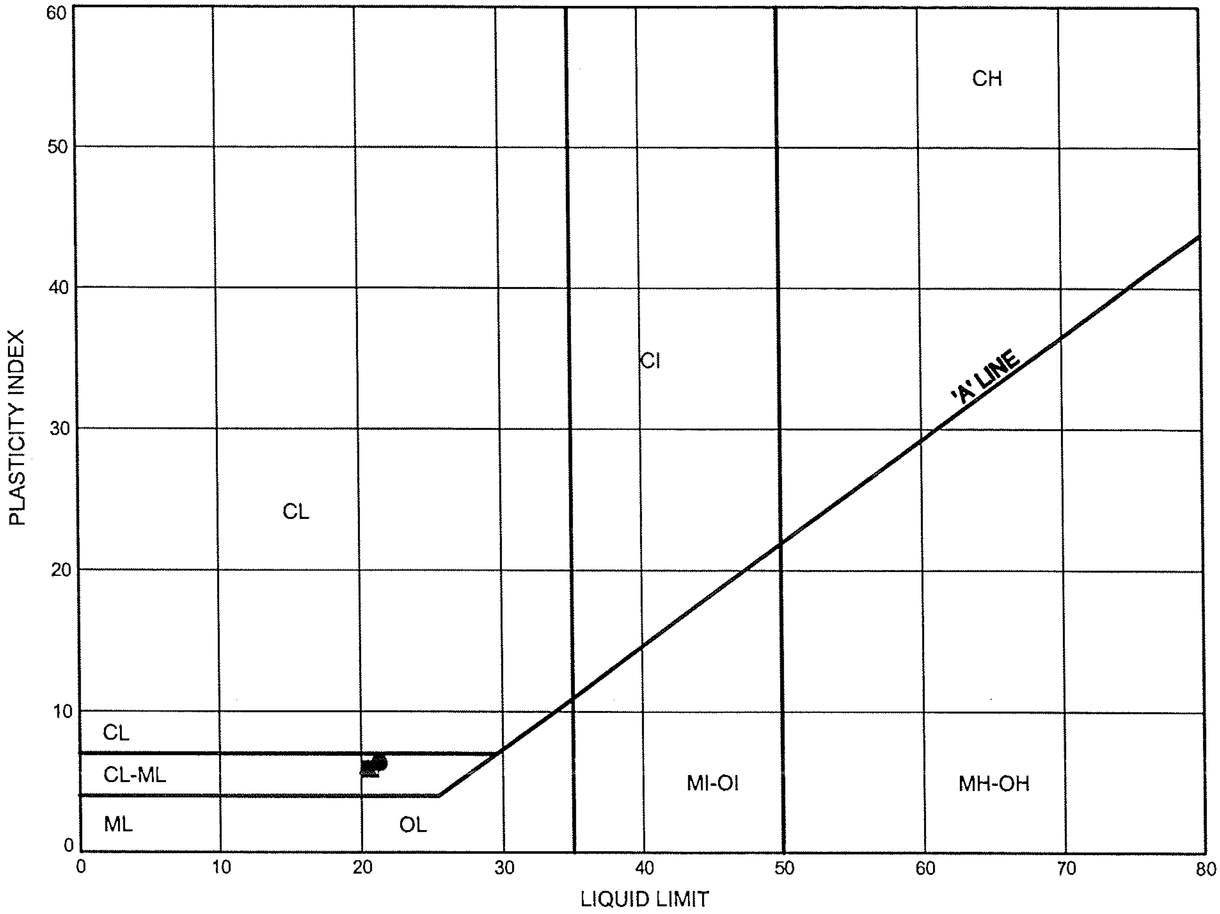
GRAIN SIZE DISTRIBUTION - THURBER 6121(HLR).GPJ 4/15/11

Hwy 69 Four-Laning North of Hwy 529

ATTERBERG LIMITS TEST RESULTS

FIGURE B8

Clayey SILT



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	HLR09-01	1.83	190.11
☒	HLR09-03	1.83	190.05
▲	HLR09-20	1.83	190.14

THURBALT 6121(HLR).GPJ 4/15/11

Date April 2011

Project 5076-06-00

THURBER

Prep'd MFA

Chkd. RPR

TABLE 1 -Point Load Test Results

HARRIS LAKE ROAD UNDERSPASS

HWY 69 FOUR-LANING

FROM THE SOUTH JUNCTION OF HIGHWAY 69 AND HIGHWAY 529 TO NAISCOOT LAKE

19-5161-21

HLR09-01	DEPTH			FORCE	AXIAL /	DIAMETER	LENGTH	UCS	CONCLUSIONS			
	FT.	IN.	METERS	(kN)	DIAMETRIC	(mm)	(mm)	(Mpa)				
RUN #1	23	12	7.30	15.0	D	47.37	104.02	156.58				
RUN #2	28	7	8.71	8.0	D	47.13	69.81	84.17				
	28	9	8.76	8.3	A	47.12	45.54	73.93				
	30	3	9.22	13.5	D	47.31	62.73	141.20				
	31	7.5	9.64	18.0	D	47.38	112.09	187.83				
	31	6	9.60	21.5	A	47.40	50.67	176.56				
	32	9.5	9.99	16.5	D	47.33	122.69	172.46				
RUN #3	33	9	10.29	8.0	D	47.33	122.69	83.62		AVERAGE	MAX	MIN
	34	12	10.66	29.8	A	47.40	49.58	248.47	RUN #1:	157	157	157
	36	9	11.20	3.5	D	47.38	95.09	36.52	RUN #2:	139	188	74
	37	6	11.43	0.5	D	47.36	108.50	5.22	RUN #3:	93	248	5

HLR09-04	DEPTH			FORCE	AXIAL /	DIAMETER	LENGTH	UCS	CONCLUSIONS			
	FT.	IN.	METERS	(kN)	DIAMETRIC	(mm)	(mm)	(Mpa)				
RUN #1	7	10	2.39	14.5	D	47.10	87.75	152.71				
	7	9	2.36	20.5	A	47.08	53.99	161.11				
	9	5	2.87	14.8	D	47.26	83.43	154.53				
	9	4	2.84	21.0	A	47.26	40.77	204.57				
RUN #2	9	7	2.92	9.0	D	47.20	90.41	94.47				
	9	8	2.95	15.5	A	47.30	43.54	143.40				
	11	6	3.51	12.0	D	47.36	84.18	125.30				
	11	7	3.53	23.0	A	47.31	48.63	195.28				
	11	10	3.61	12.0	D	47.34	96.66	125.39				
	11	11	3.63	29.0	A	47.45	44.93	261.20				
	13	7.5	4.15	2.0	D	47.46	95.72	20.82				
RUN #3	14	7	4.45	15.0	D	47.46	107.28	156.12		AVERAGE	MAX	MIN
	14	6	4.42	24.0	A	47.48	50.20	198.26	RUN #1:	168	205	153
	16	7	5.05	13.5	D	47.45	75.46	140.55	RUN #2:	138	261	94
	18	2	5.54	28.0	A	47.38	41.94	266.32	RUN #3:	190	266	141

TABLE 1 -Point Load Test Results

HARRIS LAKE ROAD UNDERSPASS

HWY 69 FOUR-LANING

FROM THE SOUTH JUNCTION OF HIGHWAY 69 AND HIGHWAY 529 TO NAISCOOT LAKE

19-5161-21

HLR09-05	DEPTH			FORCE	AXIAL /	DIAMETER	LENGTH	UCS				
	FT.	IN.	METERS	(kN)	DIAMETRIC	(mm)	(mm)	(Mpa)	CONCLUSIONS			
RUN #1	19	0	5.79	19.3	D	47.41	107.45	200.68				
	19	1	5.82	27.5	A	47.40	57.36	205.14				
RUN #2	19	3.5	5.88	18.0	D	47.48	122.36	187.22				
	20	6.5	6.26	26.0	A	47.42	61.91	182.75				
	20	9	6.32	17.5	D	47.37	101.88	182.68				
	20	8.5	6.31	20.0	A	47.43	47.82	171.70				
	22	0	6.71	17.0	D	47.42	83.99	177.17				
	21	12	6.69	16.3	A	47.47	45.80	144.15				
	23	4	7.11	12.5	D	47.38	92.32	130.44				
	23	5	7.14	21.5	A	47.47	44.60	194.69				
RUN #3	24	3	7.39	13.5	D	47.45	126.67	140.55				
	24	2	7.37	24.5	A	47.44	43.66	225.66				
	25	9	7.85	19.0	D	47.42	109.27	198.01		AVERAGE	MAX	MIN
	25	10	7.87	23.0	A	47.47	44.05	210.29	RUN #1:	203	205	201
	27	11	8.50	19.0	D	46.88	102.29	201.56	RUN #2:	171	195	130
	28	0	8.53	21.0	A	47.14	42.46	198.62	RUN #3:	196	226	141

HLR09-07	DEPTH			FORCE	AXIAL /	DIAMETER	LENGTH	UCS				
	FT.	IN.	METERS	(kN)	DIAMETRIC	(mm)	(mm)	(Mpa)	CONCLUSIONS			
RUN #1	5	10	1.78	8.9	D	50.07	252.00	85.25				
	6	9	2.06	14.0	D	50.32	305.00	133.08				
	7	9	2.36	16.9	D	50.28	279.00	160.84				
	8	8	2.64	13.8	D	50.31	198.00	131.22				
RUN #2	9	10	3.00	17.2	D	50.30	255.00	163.60				
	10	9	3.28	15.2	D	50.34	332.00	144.40				
	11	8	3.56	10.2	D	50.36	197.00	96.84				
	12	5	3.78	8.5	D	50.33	243.00	80.77				
RUN #3	13	10	4.22	13.2	D	50.33	243.00	125.43		AVERAGE	MAX	MIN
	14	10	4.52	19.2	D	50.35	356.00	182.34	RUN #1:	128	161	85
	15	10	4.83	8.4	D	50.31	244.00	79.87	RUN #2:	121	164	81
	16	8	5.08	18.9	D	50.33	297.00	179.60	RUN #3:	142	182	80

TABLE 1 -Point Load Test Results

HARRIS LAKE ROAD UNDERSPASS
HWY 69 FOUR-LANING
FROM THE SOUTH JUNCTION OF HIGHWAY 69 AND HIGHWAY 529 TO NAISCOOT LAKE

19-5161-21

HLR09-08	DEPTH			FORCE	AXIAL /	DIAMETER	LENGTH	UCS	CONCLUSIONS			
	FT.	IN.	METERS	(kN)	DIAMETRIC	(mm)	(mm)	(Mpa)				
RUN #1	3	4	1.02	18.4	D	47.60	76.32	190.63				
RUN #2	4	5	1.35	16.8	D	50.35	242.00	159.55				
	5	6	1.68	16.2	D	50.50	162.00	153.14				
	6	0	1.83	13.2	D	50.55	156.00	124.59				
RUN #3	8	0	2.44	13.8	D	50.45	291.00	130.65				
	9	0	2.74	14.0	D	47.85	246.00	143.88				
	10	1	3.07	18.9	D	47.88	147.55	194.04				
RUN #4	11	11	3.63	19.3	D	50.53	291.00	182.28		AVERAGE	MAX	MIN
	12	10	3.91	17.1	D	50.47	261.00	161.80	RUN #1:	191	191	191
	13	10	4.22	22.3	D	50.46	320.00	211.06	RUN #2:	146	160	125
	14	11	4.55	17.8	D	50.36	358.00	168.99	RUN #3:	156	194	131
									RUN #4:	181	211	162

HLR09-09	DEPTH			FORCE	AXIAL /	DIAMETER	LENGTH	UCS	CONCLUSIONS			
	FT.	IN.	METERS	(kN)	DIAMETRIC	(mm)	(mm)	(Mpa)				
RUN #2	22	0	6.71	16.3	D	50.08	98.11	156.09				
RUN #3	23	0	7.01	7.8	D	50.24	256.00	74.33				
	24	6	7.47	22.0	D	50.61	193.00	207.27				
	25	2	7.67	17.9	D	50.58	184.00	168.80				
	26	0	7.92	21.2	D	50.62	372.00	199.67				
RUN #4	27	0	8.23	22.0	D	50.67	301.00	206.89		AVERAGE	MAX	MIN
	27	11	8.51	25.9	D	50.64	211.00	243.79				
	28	7	8.71	17.0	D	50.60	197.00	160.21	RUN #2:	156	156	156
	29	5	8.97	17.2	D	50.61	266.00	162.05	RUN #3:	163	207	74
	30	3	9.22	16.8	D	50.59	254.00	158.37	RUN #4:	186	244	158

TABLE 1 -Point Load Test Results
HARRIS LAKE ROAD UNDERSPASS
HWY 69 FOUR-LANING
FROM THE SOUTH JUNCTION OF HIGHWAY 69 AND HIGHWAY 529 TO NAISCOOT LAKE

19-5161-21

HLR09-12	DEPTH			FORCE	AXIAL /	DIAMETER	LENGTH	UCS	CONCLUSIONS			
	FT.	IN.	METERS	(kN)	DIAMETRIC	(mm)	(mm)	(Mpa)				
RUN #1	9	7	2.92	16.3	D	50.78	397.00	152.77				
	10	4	3.15	17.5	D	50.63	328.00	164.77				
	11	4	3.45	21.1	D	50.69	197.00	198.30				
	12	1	3.68	22.6	D	50.63	214.00	212.79				
RUN #2	13	2	4.01	22.5	D	50.54	218.00	212.43				
	13	11	4.24	24.0	D	50.62	97.82	226.04				
	15	0	4.57	18.2	D	50.65	281.00	171.26				
	16	1	4.90	22.8	D	50.61	297.00	214.80				
RUN #3	16	8	5.08	27.6	D	50.71	147.56	259.23		**AVERAGE**	**MAX**	**MIN**
	17	9	5.41	23.2	D	50.62	286.00	218.51				
	18	7	5.66	20.5	D	50.59	215.00	193.25	**RUN #1:**	182	213	153
	19	9	6.02	11.0	D	50.64	139.37	103.54	**RUN #2:**	206	226	171
									RUN #3:	194	259	104

HLR09-15	DEPTH			FORCE	AXIAL /	DIAMETER	LENGTH	UCS	CONCLUSIONS			
	FT.	IN.	METERS	(kN)	DIAMETRIC	(mm)	(mm)	(Mpa)				
RUN #2	16	8	5.08	18.3	D	50.30	244.00	174.06				
	17	6	5.33	16.0	D	50.17	284.00	152.79				
	18	6	5.64	15.8	D	50.43	291.00	149.68				
	19	8	5.99	23.0	D	50.28	411.00	218.90				
RUN #3	21	6	6.55	20.2	D	50.65	236.00	190.08				
	22	5	6.83	10.5	D	50.45	337.00	99.41				
	23	7	7.19	23.9	D	50.50	362.00	225.93				
RUN #4	24	8	7.52	20.8	D	50.52	287.00	196.50	**RUN #2:**	174	219	150
									RUN #3:	172	226	99
									RUN #4:	197	197	197

TABLE 1 -Point Load Test Results

HARRIS LAKE ROAD UNDERSPASS

HWY 69 FOUR-LANING

FROM THE SOUTH JUNCTION OF HIGHWAY 69 AND HIGHWAY 529 TO NAISCOOT LAKE

19-5161-21

HLR09-15	DEPTH			FORCE (kN)	AXIAL / DIAMETRIC	DIAMETER (mm)	LENGTH (mm)	UCS (Mpa)	CONCLUSIONS			
	FT.	IN.	METERS									
RUN #1	17	0	5.18	25.0	D	50.69	225.00	234.96				
	18	0	5.49	14.8	D	50.69	280.00	139.09				
	19	1	5.82	21.6	D	50.73	114.41	202.75				
	19	9	6.02	11.9	D	50.68	258.00	111.87				
RUN #2	20	7	6.27	12.0	D	36.96	94.47	184.02				
	21	9	6.63	14.0	D	37.38	119.73	210.97				
	22	9	6.93	10.5	D	37.41	106.00	158.03				
	23	8	7.21	7.5	D	37.46	105.03	112.64		AVERAGE	MAX	MIN
RUN #3	24	4	7.42	11.0	D	37.36	119.94	165.90	RUN #1:	172	235	112
	25	1	7.65	8.2	D	37.44	96.34	123.26	RUN #2:	166	211	113
									RUN #3:	145	166	123

HLR09-18	DEPTH			FORCE (kN)	AXIAL / DIAMETRIC	DIAMETER (mm)	LENGTH (mm)	UCS (Mpa)	CONCLUSIONS			
	FT.	IN.	METERS									
RUN #1	13	0	3.96	12.6	D	50.66	166.00	118.53				
	14	0	4.27	14.5	D	50.68	260.00	136.32				
	14	9	4.50	13.5	D	50.55	77.67	127.42				
	15	9	4.80	16.6	D	50.55	267.00	156.68				
RUN #2	17	1	5.21	13.0	D	50.67	212.00	122.25				
	17	7	5.36	18.9	D	50.79	10.11	177.08				
	19	1	5.82	15.2	D	50.56	217.00	143.42				
	20	4	6.20	11.9	D	50.91	171.00	111.09		AVERAGE	MAX	MIN
RUN #3	21	0	6.40	22.0	D	50.53	276.00	207.78	RUN #1:	135	157	119
									RUN #2:	138	177	111
									RUN #3:	208	208	208

Appendix C

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Footing on Native Soil	Footings on Bedrock	Driven Piles	Augered Caissons (drilled shafts)
Advantages: i. Economical to install.	***Advantages:*** i. High values of geotechnical resistance are available on the bedrock. ii. Lower cost than deep foundations	***Advantages:*** i. High geotechnical resistance available by driving piles to achieve resistance in the bedrock. ii. Permits integral abutment design. iii. Readily installed. iv. Installation less influenced by weather and groundwater than spread footings.	***Advantages:*** i. High geotechnical resistance available for units founded on bedrock. ii. Installation less influenced by weather than spread footings. iii. May be a method of controlling excavation and unwatering problems at this site.
Disadvantages: i. Overburden soil conditions encountered at this site are considered unsuitable (low geotechnical capacities). ii. High groundwater levels. iii. Deep excavation extending below the groundwater level is required.	***Disadvantages:*** i. Stepped footing may be required ii. High cost of excavation to bedrock. iii. Mass concrete fill required to create a level founding surface. iv. Control of groundwater will be required. v. Not suitable for integral abutment design.	***Disadvantages:*** i. Rock excavation may be required in order to install the piles to minimum length and to accommodate an integral abutment at the east abutment. ii. At the pier, some piles may have to be socketed into bedrock. iii. Higher unit cost compared to footings	***Disadvantages:*** i. Higher unit cost compared to other foundation options such as footings or driven piles. ii. Difficulty in unwatering, cleaning and inspecting bases. May require placement of concrete by tremie methods. iii. More costly than conventional footing construction in normal situations.
NOT RECOMMENDED	**NOT RECOMMENDED**	**RECOMMENDED**	**FEASIBLE AT THE PIER**

Appendix D

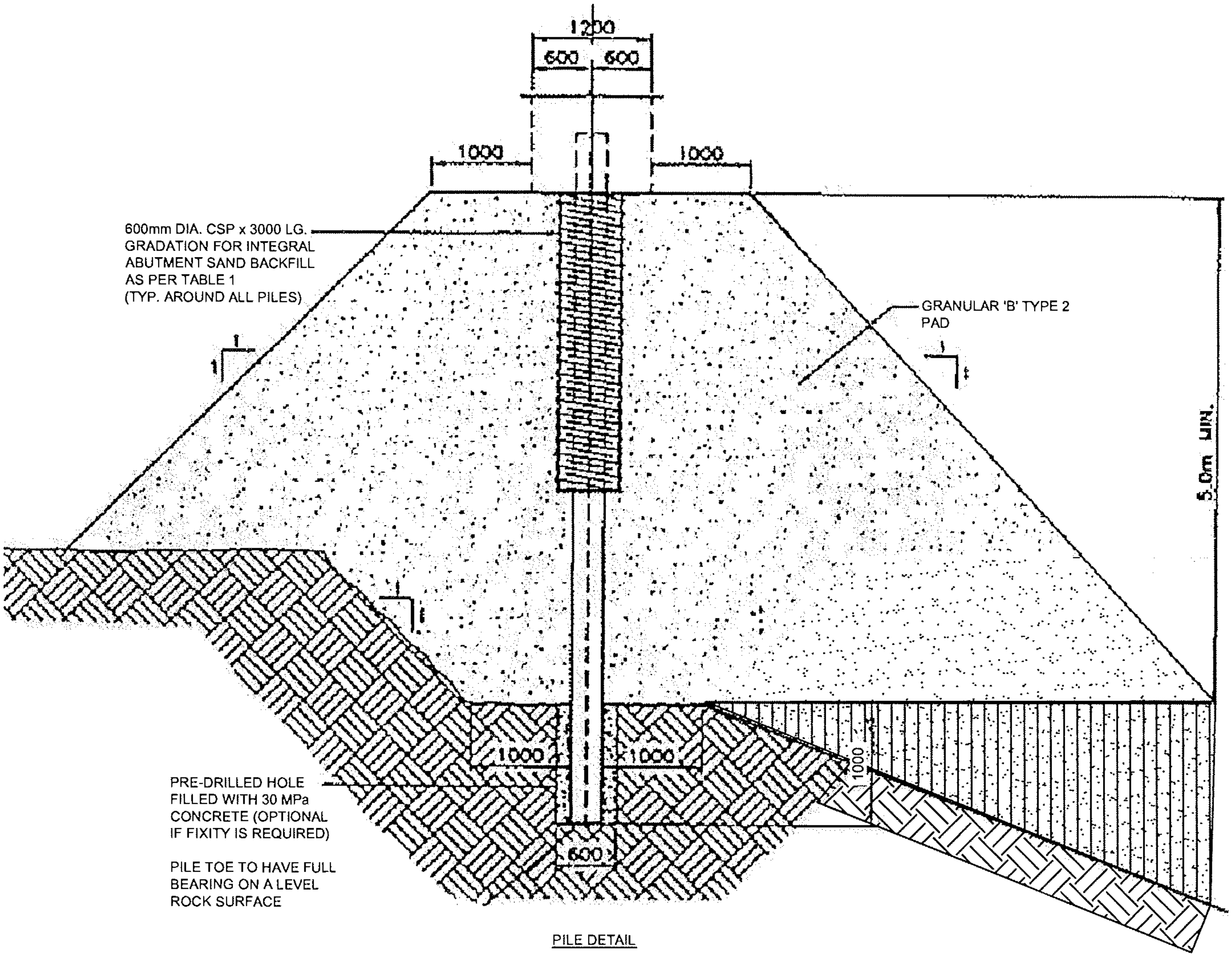
List of SPs and OPSS, and Suggested Text for Selected NSSP

1. List of Special Provisions and OPSS Documents Referenced in this Report

- OPSS 903, November 2009
- OPSS 903.07.02.07.03.03
- SP206S03, dated July 2007
- OPSS 804, November 2010
- OPSS 902, November 2010
- OPSD 3101.200.
- SP 110S13 “Amendment to OPSS 1010, April 2004”.
- OPSS 501 dated November 2010.
- OPSD 3101.150
- OPSD 3101.200

Appendix E

Figure



PILE DETAIL

TABLE 1	
MTO SEIVE DESIGNATION	PERCENTAGE PASSING
2mm #10	100%
600μm #30	80%-100%
425μm #40	40%-80%
250μm #60	5%-25%
150μm #100	0%-6%

FOUNDATION DETAILS FOR INTEGRAL ABUTMENT CONSTRUCTION IN SHALLOW BEDROCK

THURBER ENGINEERING LTD.

GEOTECHNICAL ▪ ENVIRONMENTAL ▪ MATERIALS

ENGINEER :	DRAWN :	APPROVED :
AEG	MFA	PKC
DATE :	SCALE :	DRAWING No.
NOVEMBER 2011	NTS	FIGURE 1

Appendix F

Slope Stability Output

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Earth Fill	21	0	30	0	1
Sand	19	0	28	0	1
Clayey Silt	18	0	27	0	1
Sand	19	0	28	0	1
Bedrock	(Infinitely Strong)				

Thurber Engineering Ltd. - Toronto
19-5161-21 Hwy 69 Four-Laning
Harris Lake Road Underpass
April 26, 2011
West Approach Earth fill
12 m high

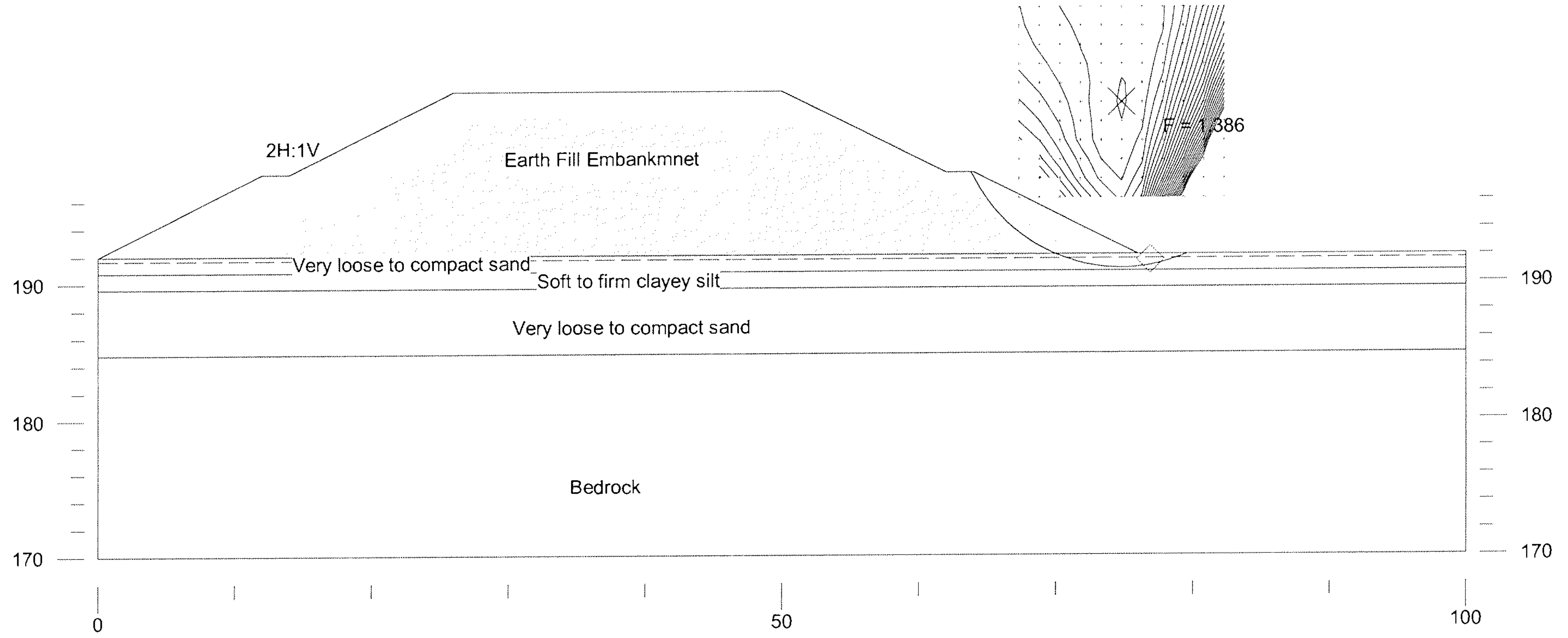


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FIGURE 1

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Earth Fill	21	0	30	0	1
Sand	19	0	28	0	1
Clayey Silt	18	0	27	0	1
Sand	19	0	28	0	1
Bedrock	(Infinitely Strong)				

Seismic coefficient = 0.08

Thurber Engineering Ltd. - Toronto
19-5161-21 Hwy 69 Four-Laning
Harris Lake Road Underpass
April 26, 2011
West Approach Earth fill
12 m high

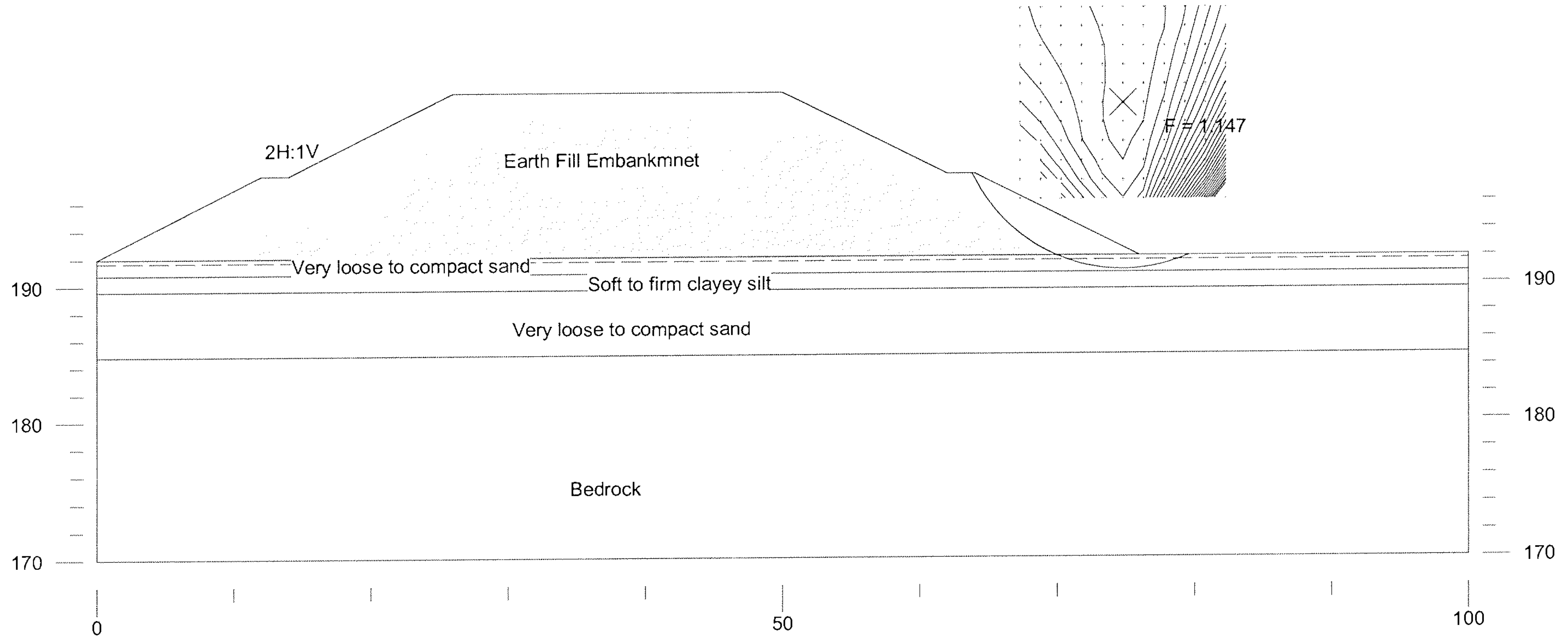

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FIGURE 2

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Rock Fill	20	0	42	0	1
Sand	19	0	28	0	1
Clayey Silt	18	0	27	0	1
Sand	19	0	28	0	1
Bedrock	(Infinitely Strong)				

Thurber Engineering Ltd. - Toronto
19-5161-21 Hwy 69 Four-Laning
Harris Lake Road Underpass
April 26, 2011
West Approach Rock fill
12 m high

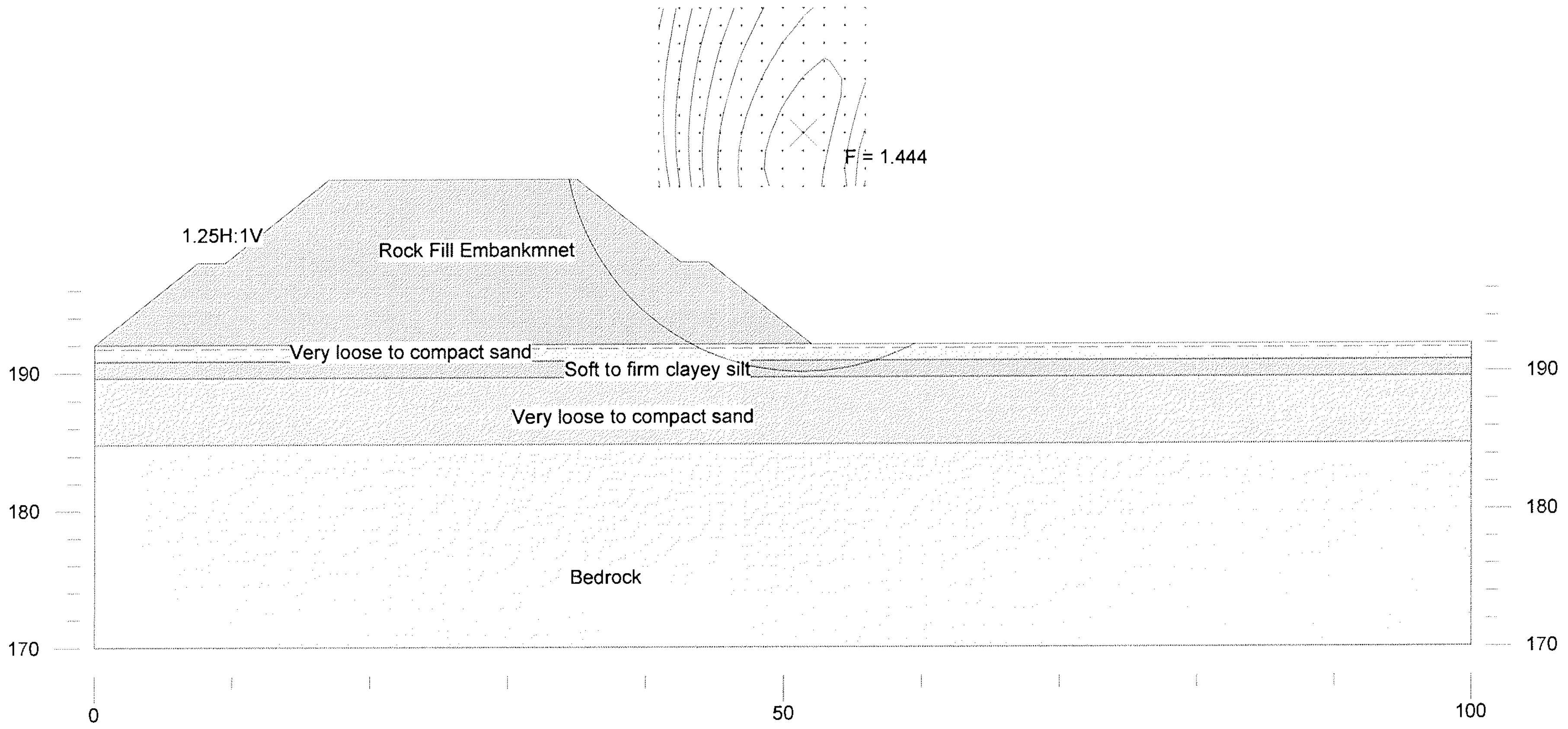

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FIGURE 3

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Rock Fill	20	0	42	0	1
Sand	19	0	28	0	1
Clayey Silt	18	0	27	0	1
Sand	19	0	28	0	1
Bedrock	(Infinitely Strong)				

Seismic coefficient = 0.08

Thurber Engineering Ltd. - Toronto
19-5161-21 Hwy 69 Four-Laning
Harris Lake Road Underpass
April 26, 2011
West Approach Rock fill
12 m high

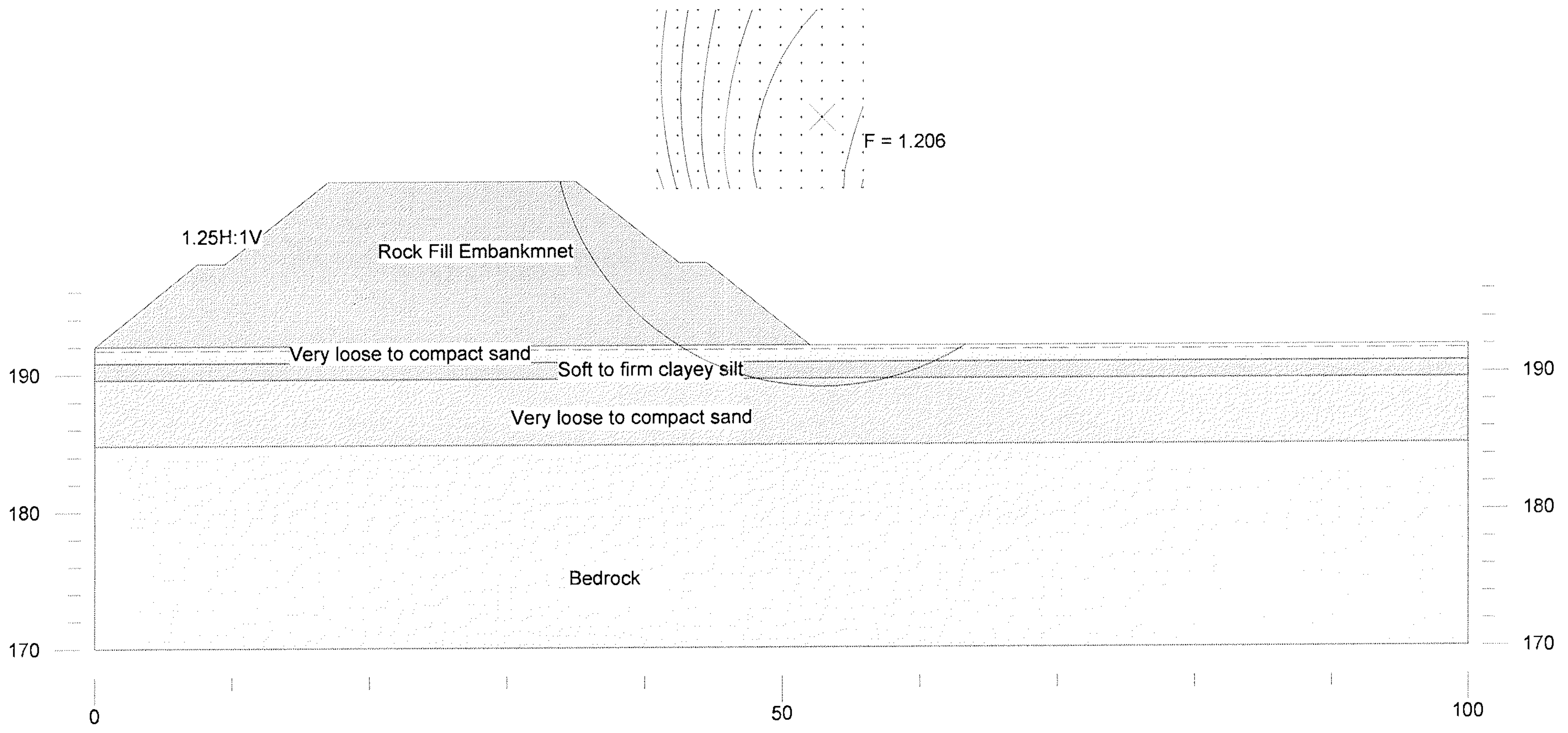

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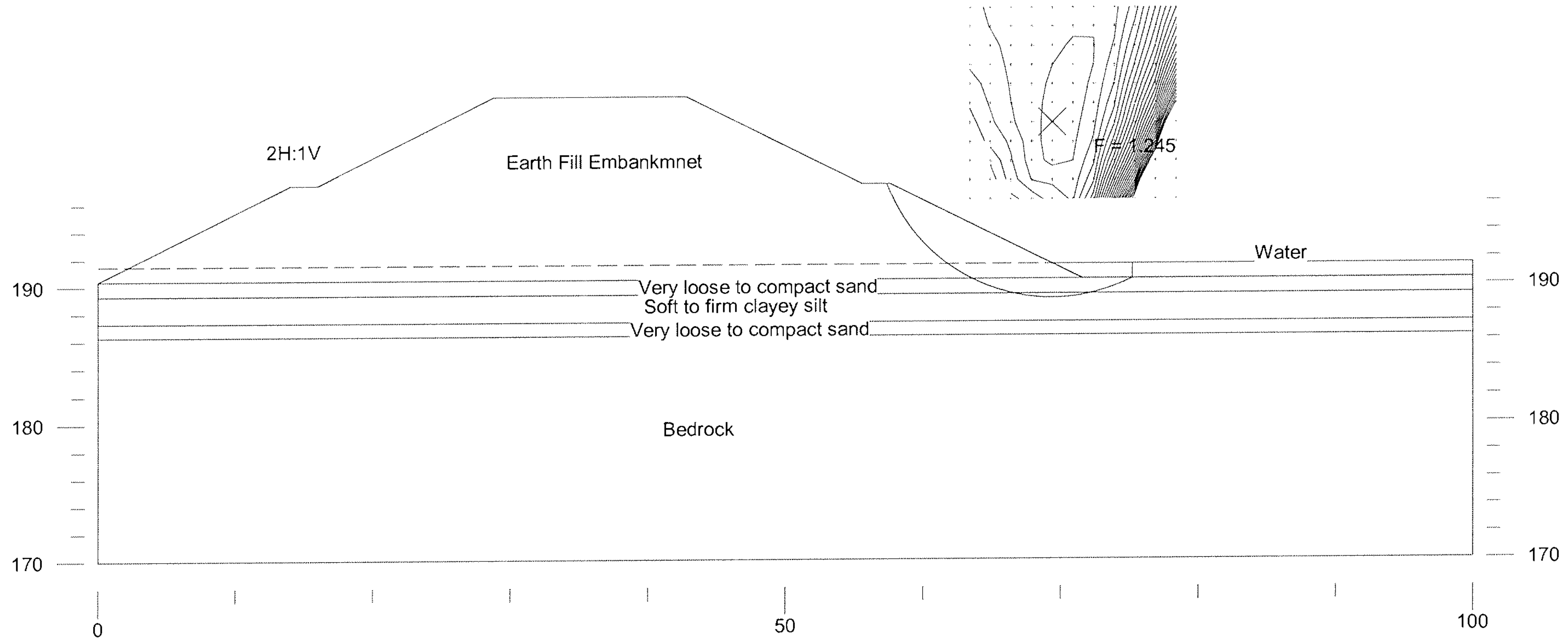
FIGURE 4

Thurber Engineering Ltd. - Toronto
19-5161-21 Hwy 69 Four-Laning
Harris Lake Road Underpass
April 26, 2011
East Approach Earth fill
13.4 m high

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Water	9.81	0	0	0	1
Earth Fill	21	0	30	0	1
Sand	19	0	28	0	1
Clayey Silt	18	0	27	0	1
Sand	19	0	28	0	1
Bedrock	(Infinitely Strong)				



4/26/2011 9:30:35 AM C:\DOCUME~1\RREYNA\MYDOCU~1\HLR-EAF.GSL Thurber Engineering Ltd. - Toronto F = 1.245

FIGURE 5

Thurber Engineering Ltd. - Toronto
19-5161-21 Hwy 69 Four-Laning
Harris Lake Road Underpass
April 26, 2011
East Approach Earth fill
13.4 m high

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Water	9.81	0	0	0	1
Earth Fill	21	0	30	0	1
Sand	19	0	28	0	1
Clayey Silt	18	0	27	0	1
Sand	19	0	28	0	1
Bedrock	(Infinitely Strong)				

Seismic coefficient = 0.08

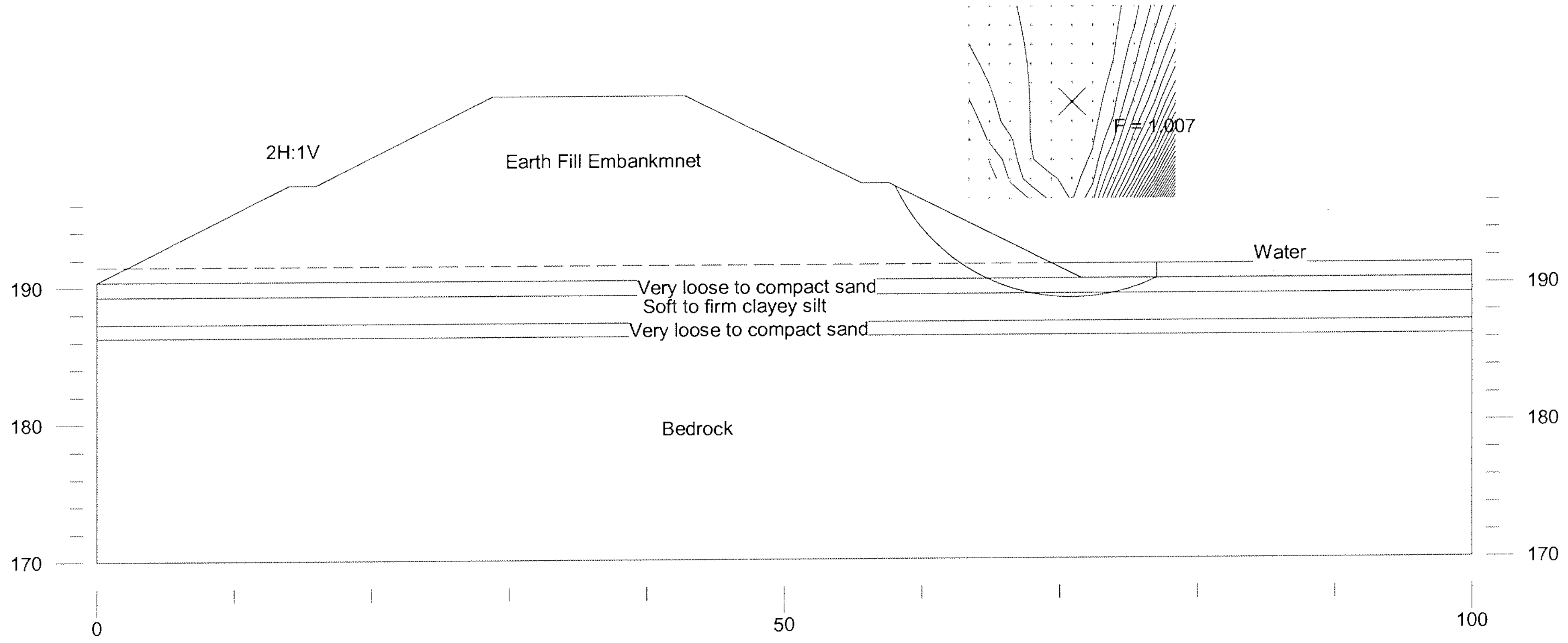


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FIGURE 6

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Water	9.81	0	0	0	1
Rock Fill	20	0	42	0	1
Sand	19	0	28	0	1
Clayey Silt	18	0	27	0	1
Sand	19	0	28	0	1
Bedrock	(Infinitely Strong)				

Thurber Engineering Ltd. - Toronto
19-5161-21 Hwy 69 Four-Laning
Harris Lake Road Underpass
April 26, 2011
East Approach Rock fill
13.4 m high

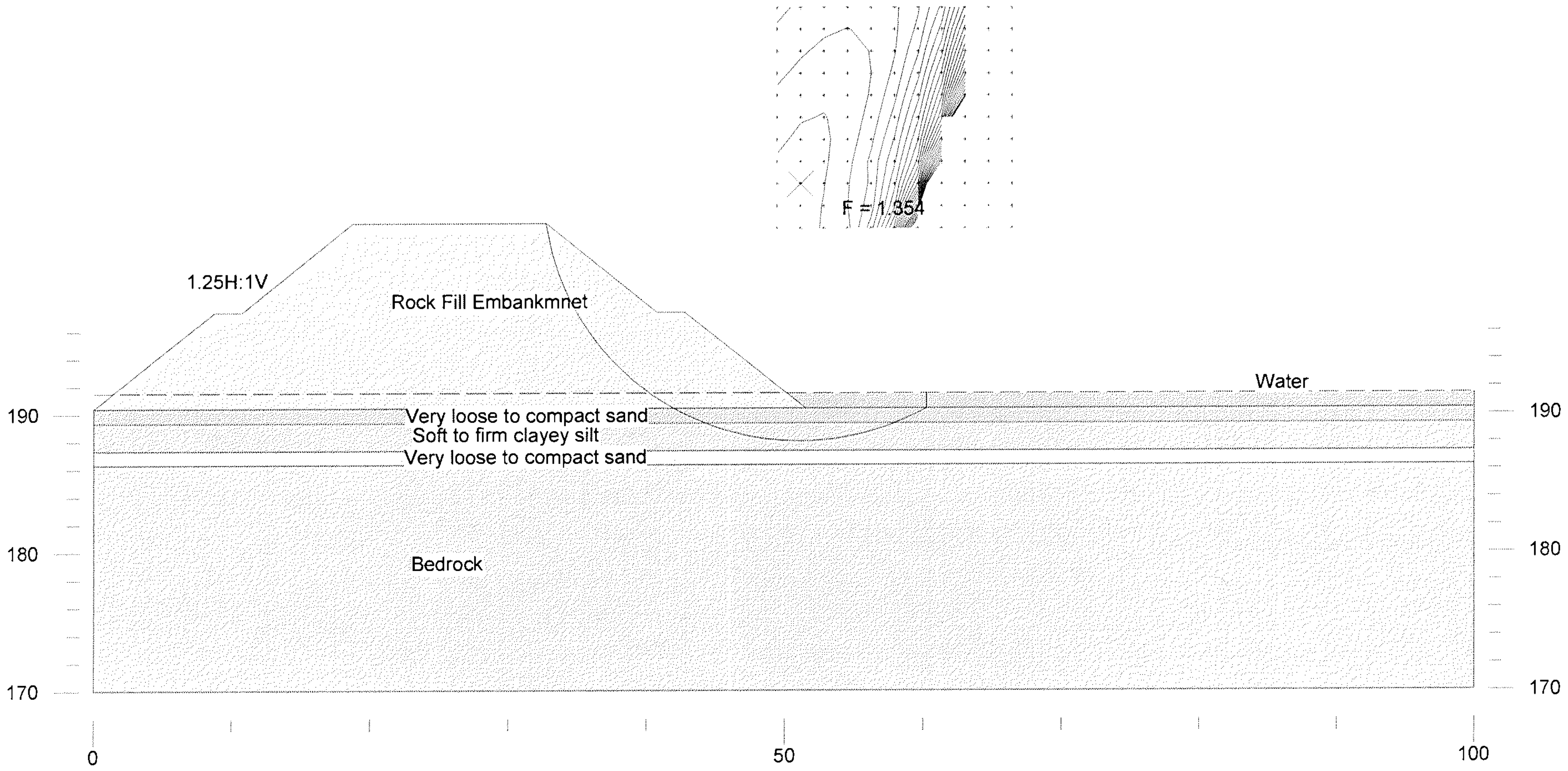

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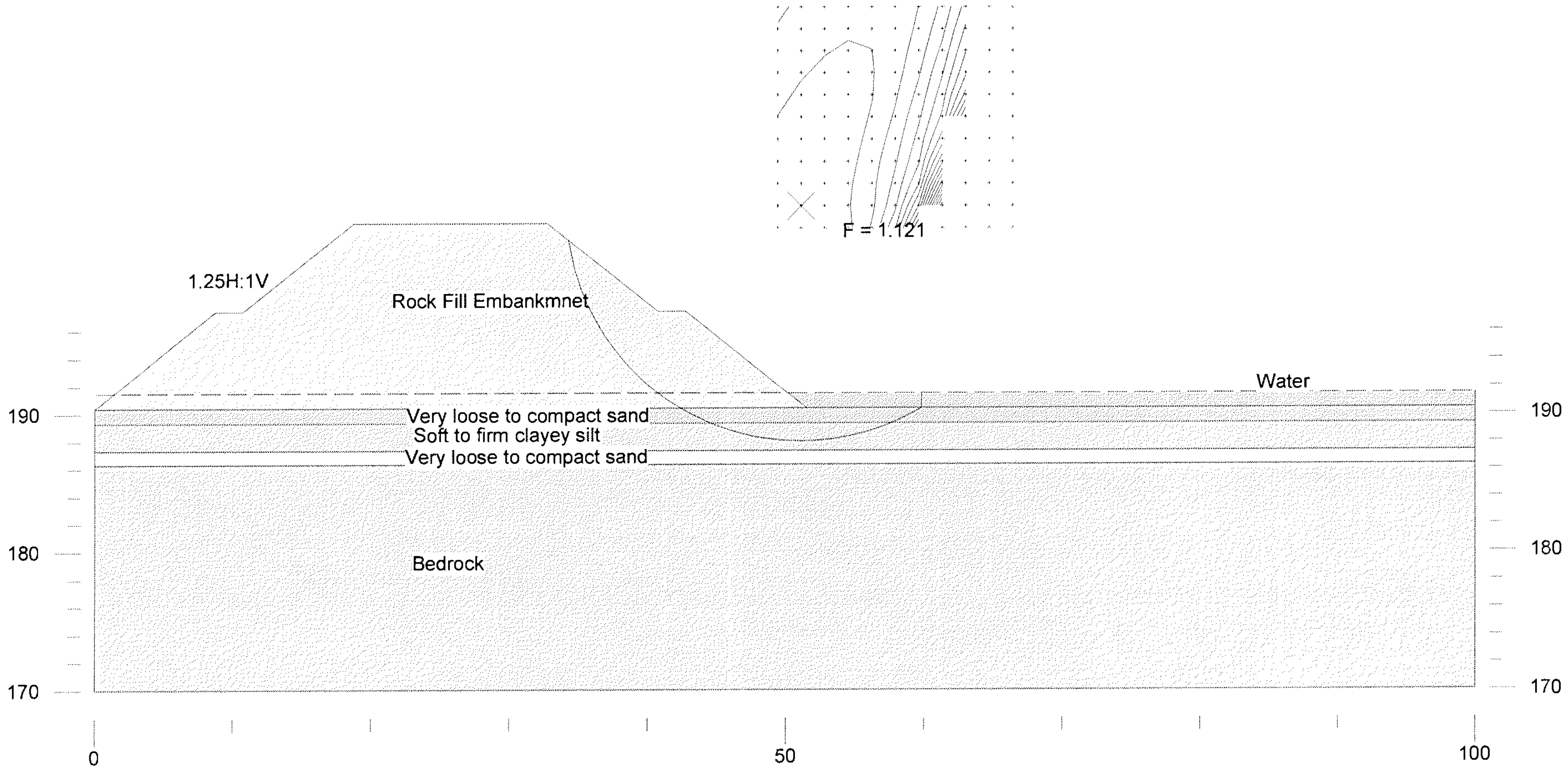
FIGURE 7

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Water	9.81	0	0	0	1
Rock Fill	20	0	42	0	1
Sand	19	0	28	0	1
Clayey Silt	18	0	27	0	1
Sand	19	0	28	0	1
Bedrock	(Infinitely Strong)				

Seismic coefficient = 0.08

Thurber Engineering Ltd. - Toronto
19-5161-21 Hwy 69 Four-Laning
Harris Lake Road Underpass
April 26, 2011
East Approach Rock fill
13.4 m high



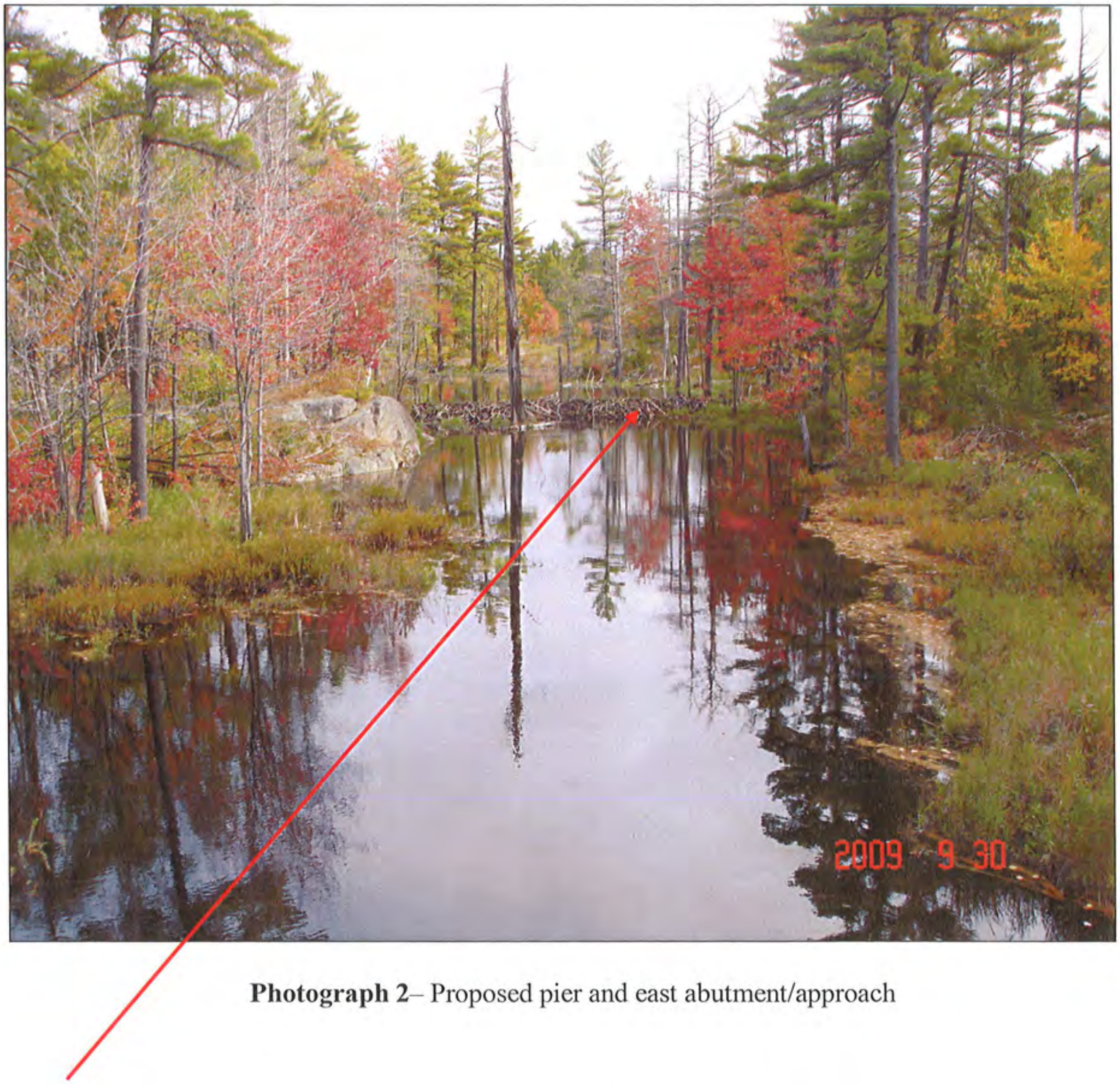
5/12/2011 11:23:24 AM C:\DOCUME~1\RREYNA\MYDOCU~1\HLR-RAFS.GSL Thurber Engineering Ltd. - Toronto F = 1.121

FIGURE 8

Appendix G

Site Photographs

Photograph 1 – East side of existing Hwy 69 at the proposed location of the new Harris Lake Road Underpass



Photograph 2– Proposed pier and east abutment/approach

Beaver dam


2009 9 30

Photograph 3– Proposed pier and east abutment/approach


2009 9 30

Photograph 4– Proposed pier and east abutment/approach

Photograph 5 – West side of existing Hwy 69 at the proposed location of the new Harris Lake Road Underpass

Photograph 6 – Proposed for the west abutment/approach location

Photograph 7 – Proposed for the west abutment/approach location
Boreholes HLR09-01 to HLR09-06, HLR09-19 and HLR09-20

Appendix H

Drawing
Borehole Locations and Soil Strata

GWP 5076-06-00
CONT No
WP No 5202-06-01

SHEET

HIGHWAY 69 FOUR-LANING
HARRIS LAKE ROAD UNDERPASS
NBL & SBL
BOREHOLE LOCATIONS AND SOIL STRATA

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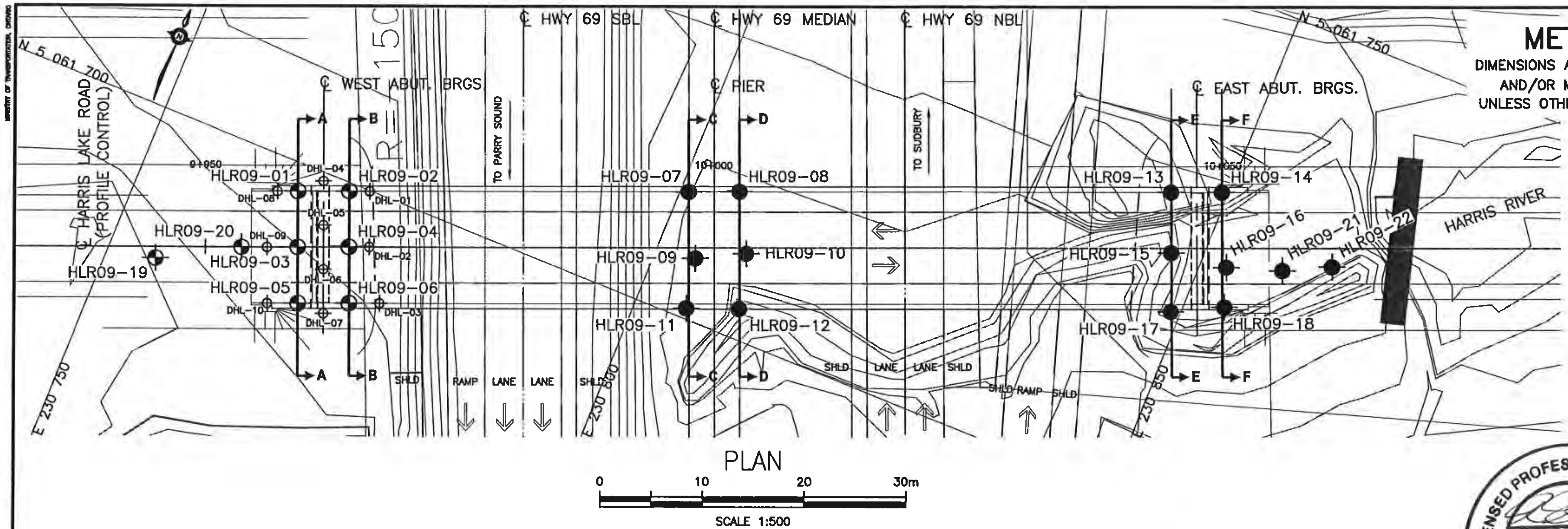
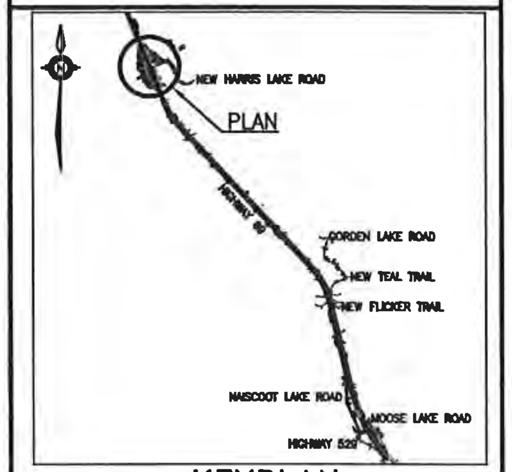


PLAN

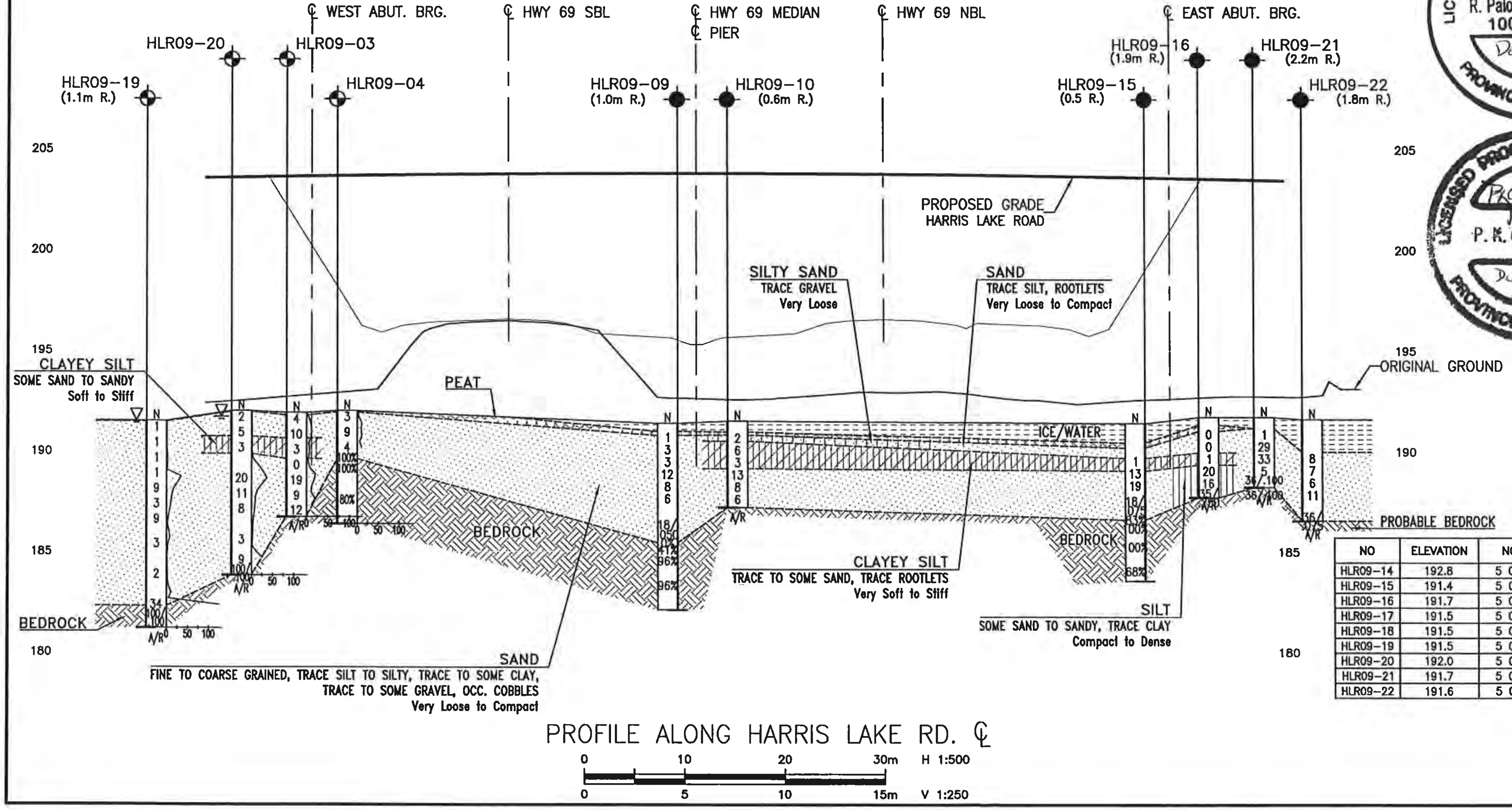
SCALE 1:500

R. Palomeque Reyna
100083209
Dec/7/2011

P. K. CHATTERJI
Dec 7/11



KEYPLAN



PROFILE ALONG HARRIS LAKE RD. Ȼ

H 1:500

V 1:250

NO	ELEVATION	NORTHING	EASTING
HLR09-14	192.8	5 061 731.3	230 849.4
HLR09-15	191.4	5 061 723.9	230 847.0
HLR09-16	191.7	5 061 724.7	230 852.5
HLR09-17	191.5	5 061 718.6	230 849.1
HLR09-18	191.5	5 061 721.0	230 853.8
HLR09-19	191.5	5 061 686.0	230 754.7
HLR09-20	192.0	5 061 690.2	230 762.1
HLR09-21	191.7	5 061 726.4	230 857.7
HLR09-22	191.6	5 061 728.5	230 862.0

LEGEND

●	Borehole
⊕	Borehole and Cone
⊕	Cone Penetrations
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
▽	Water Level
▼	Head Artesian Water
┃	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
HLR09-01	191.9	5 061 697.3	230 765.2
HLR09-02	192.0	5 061 699.2	230 769.9
HLR09-03	191.9	5 061 692.2	230 767.3
HLR09-04	192.0	5 061 694.1	230 771.9
HLR09-05	191.8	5 061 687.1	230 769.3
HLR09-06	191.8	5 061 689.0	230 774.0
HLR09-07	191.9	5 061 711.7	230 800.8
HLR09-08	191.8	5 061 713.6	230 805.4
HLR09-09	191.4	5 061 705.9	230 803.8
HLR09-10	191.5	5 061 708.2	230 808.3
HLR09-11	191.5	5 061 701.0	230 804.9
HLR09-12	191.5	5 061 702.9	230 809.7
HLR09-13	194.3	5 061 729.5	230 844.8

-NOTES-

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GEOCRES No. 41H-98

REVISIONS	DATE	BY	DESCRIPTION

DESIGN RPR	CHK AEG	CODE	LOAD	DATE NOV. 2011
DRAWN MFA	CHK PKC	SITE 44-450	STRUCT	DWG 2-1

FILENAME:
PLOTDATE:

MINISTRY OF TRANSPORTATION, ONTARIO

METRIC
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GWP 5076-06-00
CONT No
WP No 5202-06-01

HIGHWAY 69 FOUR-LANING
HARRIS LAKE ROAD UNDERPASS
NBL & SBL
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

MMM GROUP

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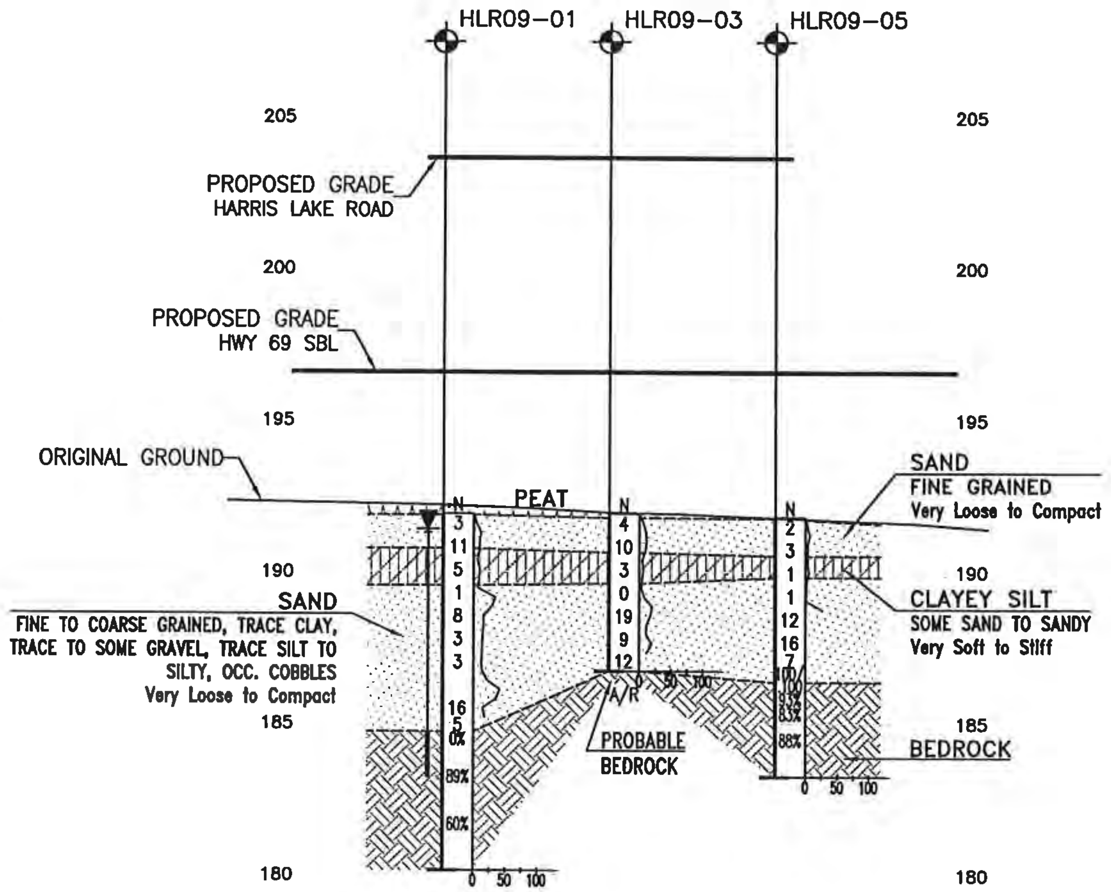

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SECTION A-A

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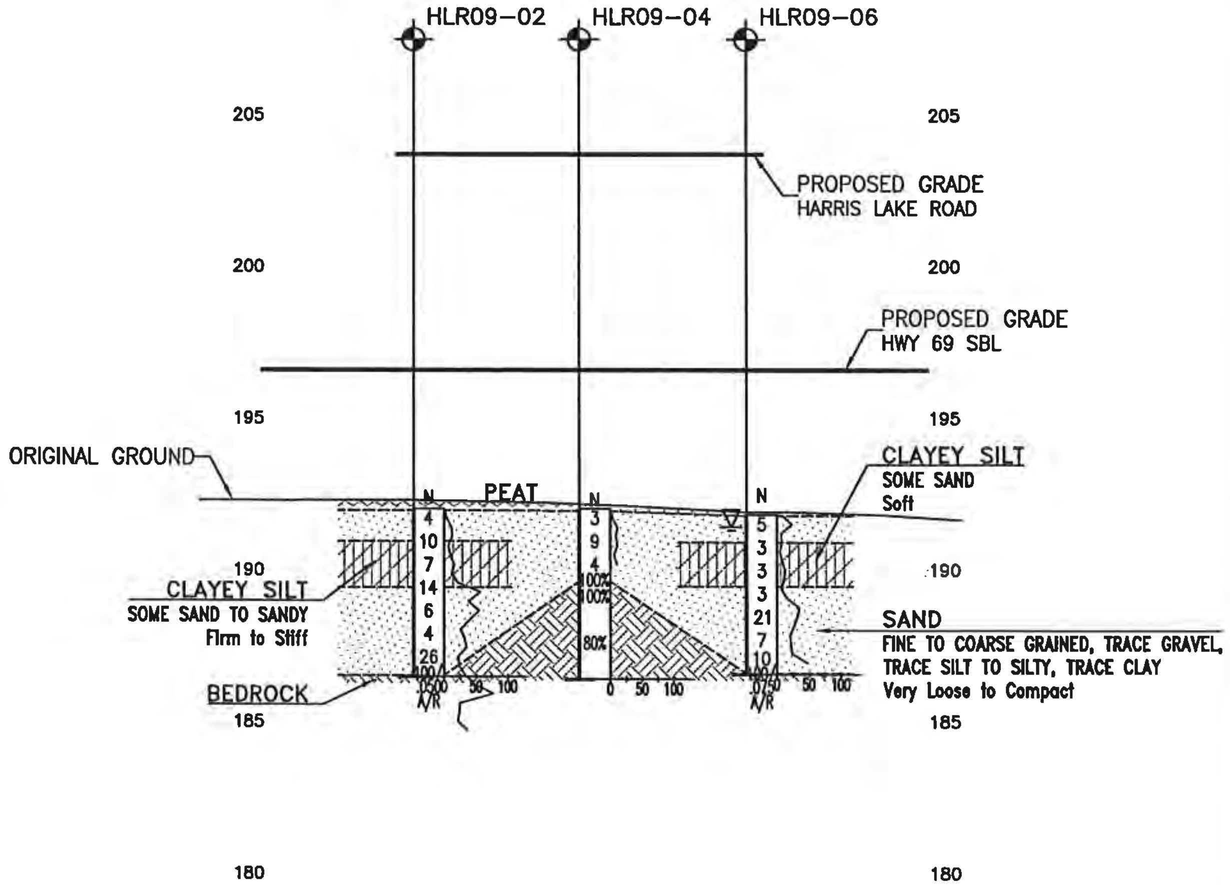
SCALE 1:250



WEST ABUTMENT
SECTION B-B

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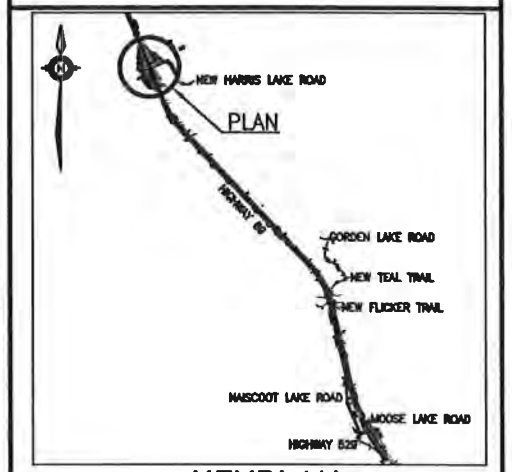

KEYPLAN

L E G E N D

⊕	Borehole
⊕	Borehole and Cone
⊕	Cone Penetrations
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
▽	Water Level
▼	Head Artesian Water
	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
HLR09-01	191.9	5 061 697.3	230 765.2
HLR09-02	192.0	5 061 699.2	230 769.9
HLR09-03	191.9	5 061 692.2	230 767.3
HLR09-04	192.0	5 061 694.1	230 771.9
HLR09-05	191.8	5 061 687.1	230 769.3
HLR09-06	191.8	5 061 689.0	230 774.0
HLR09-07	191.9	5 061 711.7	230 800.8
HLR09-08	191.8	5 061 713.6	230 805.4
HLR09-09	191.4	5 061 705.9	230 803.8
HLR09-10	191.5	5 061 708.2	230 808.3
HLR09-11	191.5	5 061 701.0	230 804.9
HLR09-12	191.5	5 061 702.9	230 809.7
HLR09-13	194.3	5 061 729.5	230 844.8

NO	ELEVATION	NORTHING	EASTING
HLR09-14	192.8	5 061 731.3	230 849.4
HLR09-15	191.4	5 061 723.9	230 847.0
HLR09-16	191.7	5 061 724.7	230 852.5
HLR09-17	191.5	5 061 718.6	230 849.1
HLR09-18	191.5	5 061 721.0	230 853.8
HLR09-19	191.5	5 061 686.0	230 754.7
HLR09-20	192.0	5 061 690.2	230 762.1
HLR09-21	191.7	5 061 726.4	230 857.7
HLR09-22	191.6	5 061 728.5	230 862.0

-NOTES-

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GEOCRES No. 41H-98

R. Palomeque Reyna
100083209
Dec/7/2011

P. K. CHATTERJI
Dec 7/11

REVISIONS	DATE	BY	DESCRIPTION

DESIGN RPR	CHK AEG	CODE	LOAD	DATE NOV. 2011
DRAWN MFA	CHK PKC	SITE 44-450	STRUCT	DWG 2-2

FILENAME:
PLOTDATE:

MINISTRY OF TRANSPORTATION, ONTARIO

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DIMENSIONS ARE IN METRES
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GWP 5076-06-00
CONT No
WP No 5202-06-01

HIGHWAY 69 FOUR-LANING
HARRIS LAKE ROAD UNDERPASS
NBL & SBL
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

MMM GROUP

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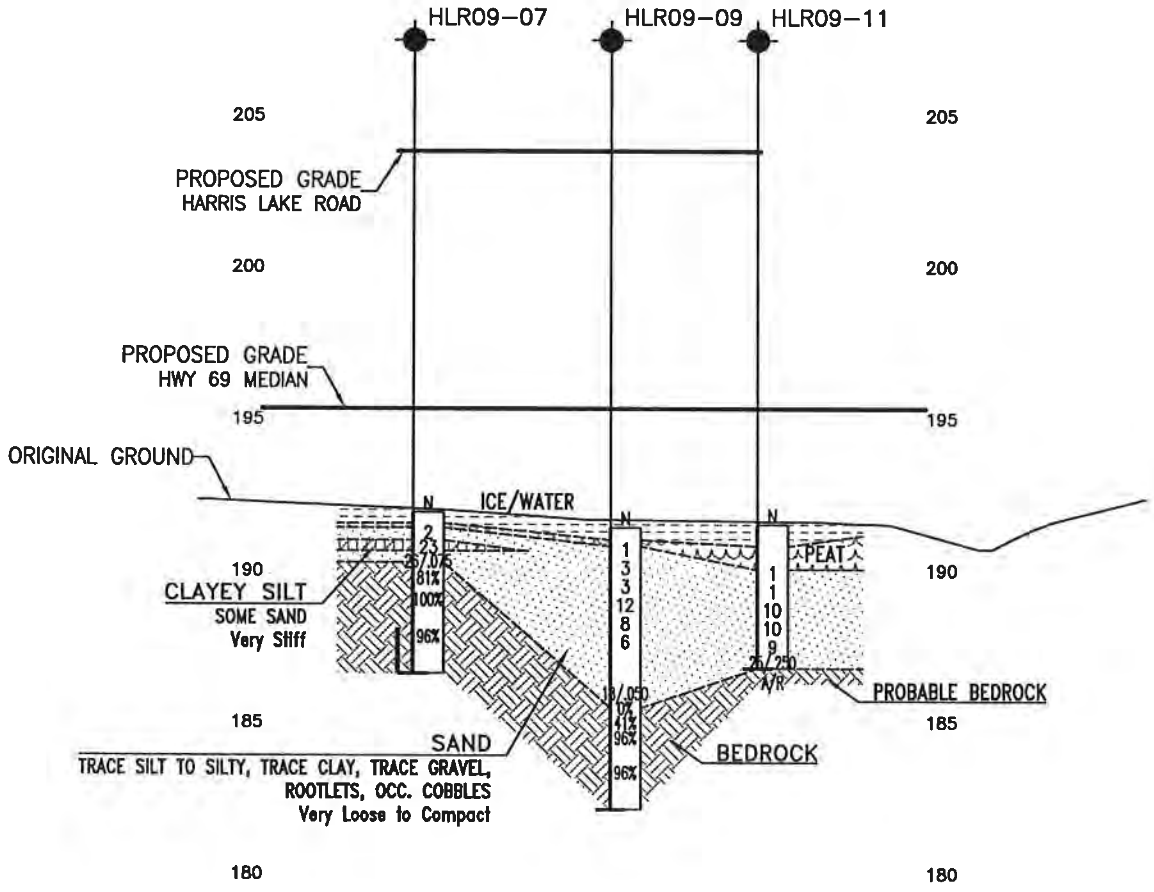

PIER
SECTION C-C

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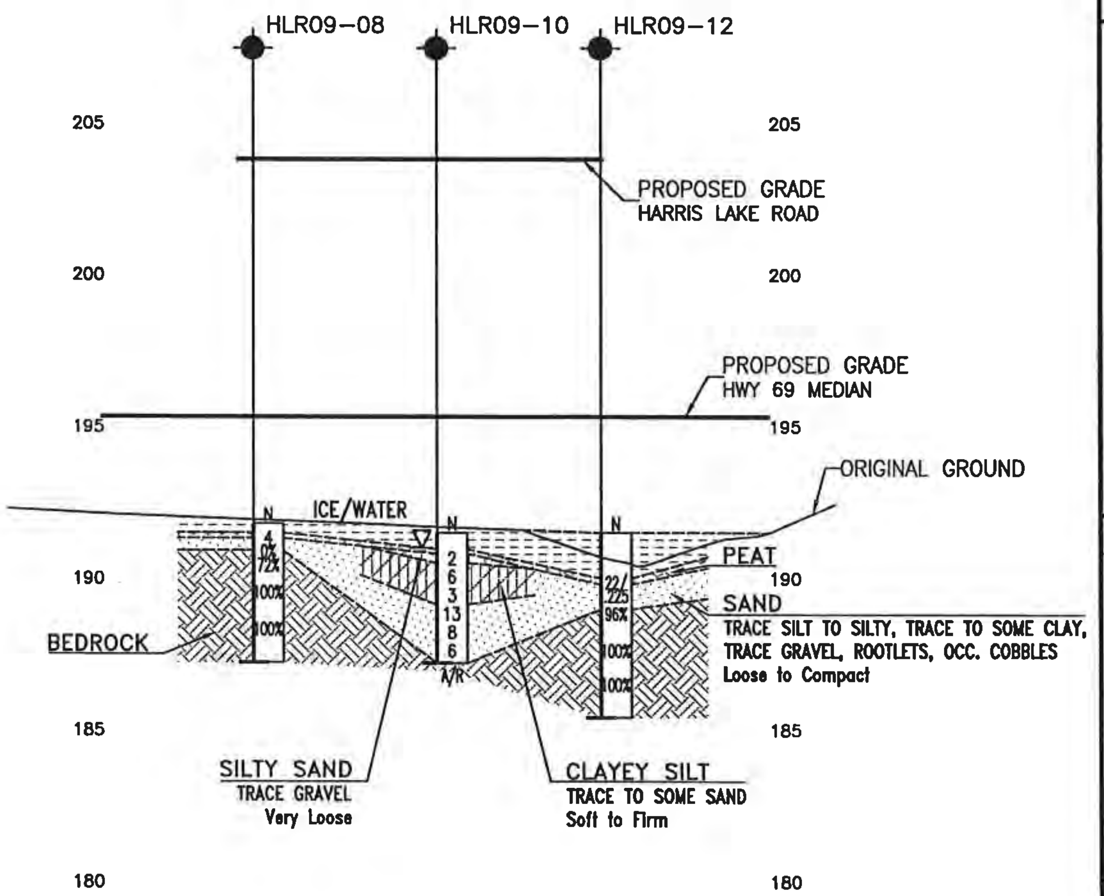

PIER
SECTION D-D

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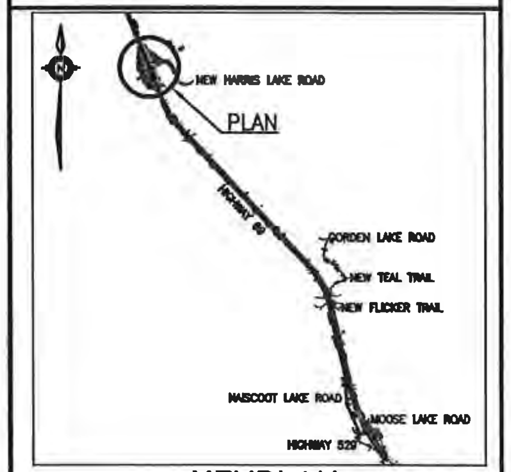
SCALE 1:250



KEYPLAN

LEGEND

⯁	Borehole
⊕	Borehole and Cone
⊕	Cone Penetrations
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
▽	Water Level
▼	Head Artesian Water
	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
HLR09-01	191.9	5 061 697.3	230 765.2
HLR09-02	192.0	5 061 699.2	230 769.9
HLR09-03	191.9	5 061 692.2	230 767.3
HLR09-04	192.0	5 061 694.1	230 771.9
HLR09-05	191.8	5 061 687.1	230 769.3
HLR09-06	191.8	5 061 689.0	230 774.0
HLR09-07	191.9	5 061 711.7	230 800.8
HLR09-08	191.8	5 061 713.6	230 805.4
HLR09-09	191.4	5 061 705.9	230 803.8
HLR09-10	191.5	5 061 708.2	230 808.3
HLR09-11	191.5	5 061 701.0	230 804.9
HLR09-12	191.5	5 061 702.9	230 809.7
HLR09-13	194.3	5 061 729.5	230 844.8

NO	ELEVATION	NORTHING	EASTING
HLR09-14	192.8	5 061 731.3	230 849.4
HLR09-15	191.4	5 061 723.9	230 847.0
HLR09-16	191.7	5 061 724.7	230 852.5
HLR09-17	191.5	5 061 718.6	230 849.1
HLR09-18	191.5	5 061 721.0	230 853.8
HLR09-19	191.5	5 061 686.0	230 754.7
HLR09-20	192.0	5 061 690.2	230 762.1
HLR09-21	191.7	5 061 726.4	230 857.7
HLR09-22	191.6	5 061 728.5	230 862.0

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GEOCRES No. 41H-98

LICENSED PROFESSIONAL ENGINEER
R. Palomeque Reyna
100083209
Dec/7/2011
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
Dec 7/11
PROVINCE OF ONTARIO

REVISIONS	DATE	BY	DESCRIPTION

DESIGN RPR	CHK AEG	CODE	LOAD	DATE NOV. 2011
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FILENAME:
PLOTDATE:

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CONT No
WP No 5202–06–01

HIGHWAY 69 FOUR–LANING
HARRIS LAKE ROAD UNDERPASS
NBL & SBL
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

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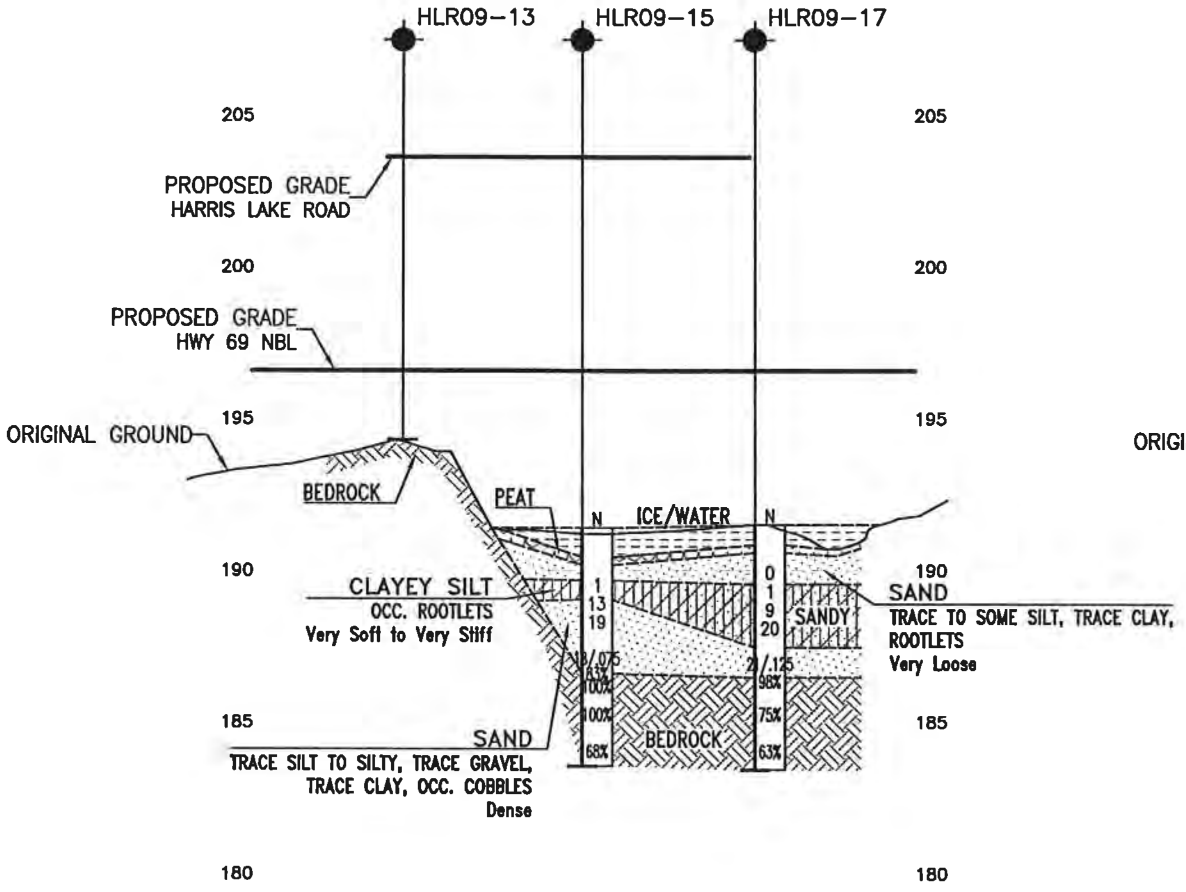

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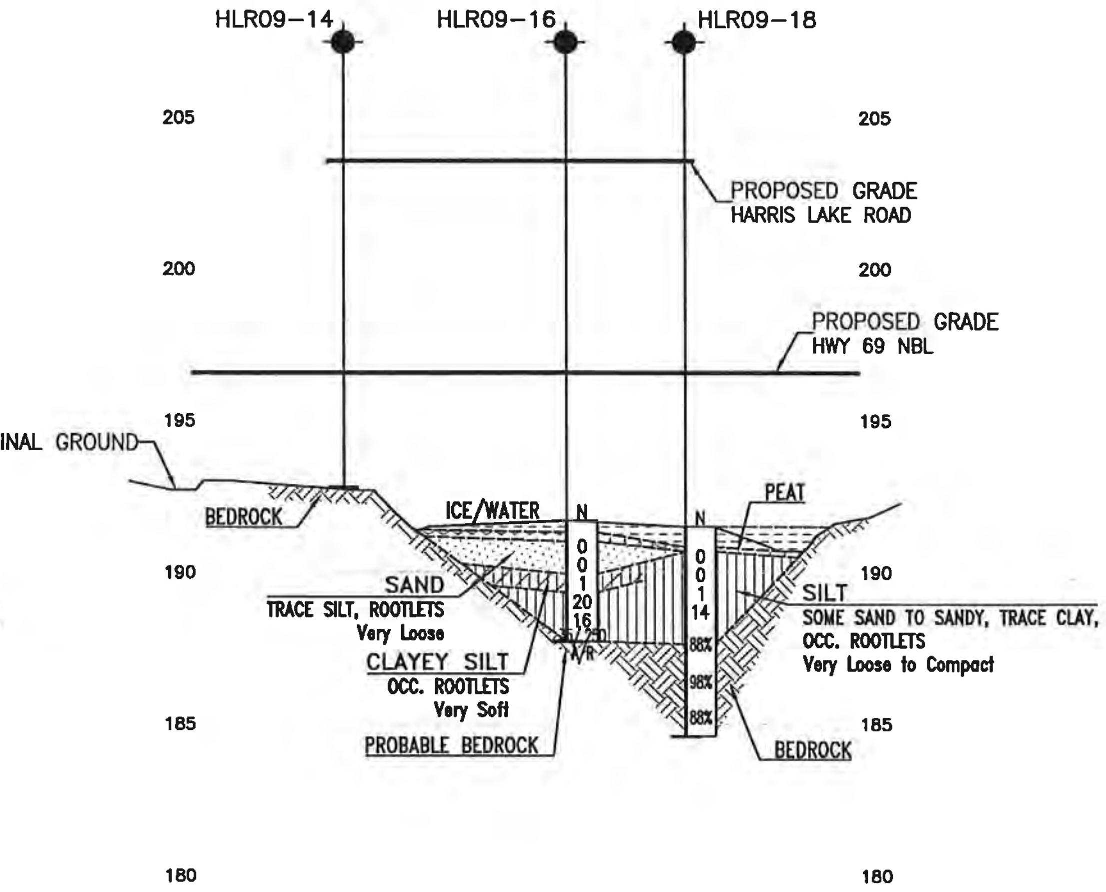

EAST ABUTMENT
SECTION F–F

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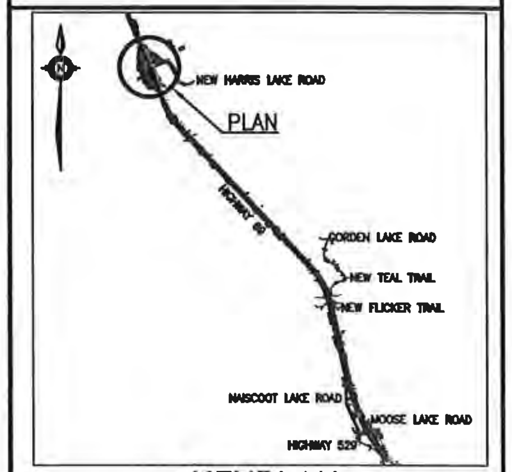
SCALE 1:250



KEYPLAN

L E G E N D

●	Borehole
⊕	Borehole and Cone
⊕	Cone Penetrations
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
⊻	Water Level
⊻	Head Artesian Water
⊥	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
HLR09–01	191.9	5 061 697.3	230 765.2
HLR09–02	192.0	5 061 699.2	230 769.9
HLR09–03	191.9	5 061 692.2	230 767.3
HLR09–04	192.0	5 061 694.1	230 771.9
HLR09–05	191.8	5 061 687.1	230 769.3
HLR09–06	191.8	5 061 689.0	230 774.0
HLR09–07	191.9	5 061 711.7	230 800.8
HLR09–08	191.8	5 061 713.6	230 805.4
HLR09–09	191.4	5 061 705.9	230 803.8
HLR09–10	191.5	5 061 708.2	230 808.3
HLR09–11	191.5	5 061 701.0	230 804.9
HLR09–12	191.5	5 061 702.9	230 809.7
HLR09–13	194.3	5 061 729.5	230 844.8

NO	ELEVATION	NORTHING	EASTING
HLR09–14	192.8	5 061 731.3	230 849.4
HLR09–15	191.4	5 061 723.9	230 847.0
HLR09–16	191.7	5 061 724.7	230 852.5
HLR09–17	191.5	5 061 718.6	230 849.1
HLR09–18	191.5	5 061 721.0	230 853.8
HLR09–19	191.5	5 061 686.0	230 754.7
HLR09–20	192.0	5 061 690.2	230 762.1
HLR09–21	191.7	5 061 726.4	230 857.7
HLR09–22	191.6	5 061 728.5	230 862.0

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GEOCRES No. 41H-98

R. Palomeque Reyna
100083209
Dec/7/2011

P. K. CHATTERJI
Dec 7/11

REVISIONS	DATE	BY	DESCRIPTION		
DESIGN RPR	CHK AEG	CODE	LOAD	DATE NOV. 2011	
DRAWN MFA	CHK PKC	SITE 44–450	STRUCT	DWG 2–4	

FILENAME:
PLOTDATE: