

**FOUNDATION INVESTIGATION AND DESIGN REPORT
HODDER AVENUE/COPENHAGEN ROAD UNDERPASS
HIGHWAY 11/17 FOUR-LANING FROM 1.0 KM WEST OF
HODDER AVENUE/COPENHAGEN ROAD EASTERLY FOR 5.8 KM
W.P. 334-94-00**

Geocres Number: 52A-143

Report to

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted for a proposed underpass structure at Hodder Avenue/Copenhagen Road and Highway 11/17 in Thunder Bay, Ontario. The proposed structure will carry Hodder Avenue/Copenhagen Road over the future four-laned Highway 11/17.

MTO carried out a subsurface investigation for the proposed structure in 1991 and the results were presented in a report dated April 10, 1992 (Geocres No. 52A-111). Peto MacCallum Ltd. subsequently prepared a Preliminary Foundation Investigation and Design Report dated May 17, 2007 (Geocres No. 52A-130) based solely on the information obtained during the MTO study.

The purpose of the current investigation was to review the existing subsurface information, conduct additional exploration at the site where deemed necessary for detail design and, based on the data obtained, to provide a borehole location plan, record of borehole sheets, stratigraphic profiles and cross-sections, laboratory test results and a written description of the subsurface conditions. A model of the subsurface conditions was developed from the data obtained in the course of the present and previous investigations.

Thurber carried out the investigation as a sub-consultant to McCormick Rankin Corporation, under the Ministry of Transportation Ontario (MTO) Agreement Number 6008-E-0005.

2 SITE DESCRIPTION

The site lies at the northeast limit of the City of Thunder Bay at the existing intersection of Highway 11/17 and Hodder Avenue/Copenhagen Road. Highway 11/17 is presently an undivided highway with one lane eastbound, one lane westbound entering the intersection, two lanes westbound beyond the intersection, and right and left turning lanes. Hodder Avenue and Copenhagen Road are two-lane undivided roadways.

The existing level crossing is at approximate elevation 262.0 m. Existing grades on Highway 11/17 generally slope down towards the west, falling approximately 16 m to the Current River bridge located about 450 m to the west. Grades on Hodder Avenue slope down to the south, and grades on Copenhagen Road north of Highway 11/17 undulate slightly.

The surrounding lands are forested. Residential properties are present on the west side of Hodder Avenue beginning approximately 200 m to the south of the intersection.

Geologically, the site area is located within the physiographic region known as the Canadian Shield, characterized by Pre-Cambrian bedrock typically occurring as rounded knobs and ridges where exposed. At this site, the bedrock consists of the Gunflint Formation, a sequence of limestone, graphitic shale, tuff, taconite, chert-carbonite and chert. Intrusions/sills of diorite are present locally. The bedrock is overlain by a discontinuous layer of glacial till comprising a heterogeneous mixture of clayey silt, silt, sand and gravel.

Photographs of the site are included in Appendix F.

3 SITE INVESTIGATION AND FIELD TESTING

MTO carried out a foundation investigation at the site in August 1991 and the results were documented in a Foundation Investigation Report dated April 10, 1992 (Geocres No. 52A-111). The investigation consisted of two boreholes at each proposed foundation unit (north abutment, pier and south abutment) advanced to depths of 2.3 to 10.9 m, with 5 m of rock core recovered from one borehole. The Record of Borehole sheets from the earlier investigation are reproduced in Appendix C.

The current site investigation was carried out in several stages between June 24 and December 14, 2009. Initially three supplementary boreholes (numbered 09-92A, 09-92B and 09-93) were advanced at the south abutment where bedrock was previously encountered at shallow depth, and one borehole was drilled approximately 20 m beyond each abutment (boreholes 09-47 and 09-91). Subsequently two additional boreholes (Nos. 09-94 and 09-95) were drilled at the north abutment location.

The four boreholes at the south abutment and approach were advanced to total depths of 4.4 to 6.2 m by coring 3.0 to 3.7 m into bedrock. The boreholes at the north abutment were extended to depths of 10.0 and 7.8 m, including 3.0 m of rock core in borehole 09-94. The borehole at the north approach was terminated upon auger refusal at 2.6 m depth.

The approximate locations of the boreholes drilled during the previous and current investigations are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix G. The borehole elevations, locations and depths are summarized in Table 3.1.

Table 3.1 – Borehole Summary

Foundation Unit	Borehole	Ground Surface Elevation (m)	Location		Total Depth (m)	Length of Core in Bedrock (m)
			Northing	Easting		
North Approach	09-47	260.4	5 372 030.9	365 262.0	2.6*	-
North Abutment	09-94	261.1	5 372 019.4	365 254.2	10.0	3.0
	09-95	260.7	5 372 019.2	365 269.2	7.8	-
	1	261.9	5 372 010.9	365 255.9	10.8	-
	2	260.5	5 372 010.9	365 268.9	7.7	-
Pier	3	261.2	5 371 984.4	365 255.9	9.2	-
	4	261.8	5 371 984.4	365 269.1	10.9	-
South Abutment	09-92A	261.6	5 371 947.2	365 269.1	6.1	3.3
	09-92B	260.2	5 371 947.2	365 256.1	4.4	2.7
	09-93	261.4	5 371 953.4	365 262.6	6.2	3.6
	5	260.5	5 371 957.9	365 256.1	7.6	5.0
	6	261.5	5 371 958.0	365 269.1	2.3*	-
South Approach	09-91	261.4	5 371 937.9	365 262.7	5.8	3.5

* Probable bedrock

Prior to commencing the site investigation, clearance was obtained from utility companies having plant in the area.

Hollow-stem augers were used to advance the boreholes to bedrock or auger refusal. Samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT) in the soils. The boreholes at the abutments where rock was encountered were advanced 3.0 to 3.7 m into bedrock by BQ size diamond coring techniques.

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), Solid Core Recovery (SCR), Rock Quality Designation (RQD) and the Fracture Indices (FI) were determined.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. In boreholes 09-92B and 09-094, standpipe piezometers consisting of 19 mm PVC pipe with a slotted screen were installed and enclosed in filter sand to permit longer term groundwater level monitoring. The completion details of the piezometers are shown in Table 3.2. Following the final water level reading, the piezometers were decommissioned in accordance with MOE Regulation 903.

The boreholes in which no piezometers were installed were backfilled with bentonite and cuttings. The borehole completion details are shown in Table 3.2.

Table 3.2 – Borehole Completion Details

Borehole	Piezometer Tip		Completion Details
	Depth (m)	Elevation (m)	
09-47	-	-	Borehole backfilled with cuttings to surface
09-91	-	-	Borehole backfilled with bentonite to 0.1 m, then gravel cuttings to surface
09-92A	-	-	Borehole backfilled with bentonite to 0.1 m, then cold patch asphalt to surface
09-92B	4.4	255.8	Piezometer with 1.5 m slotted screen installed with sand filter to 2.6 m, bentonite seal from 2.6 m to ground surface.
09-93	-	-	Borehole backfilled with bentonite to 0.1 m, then cold patch asphalt to surface
09-94	6.2	254.9	Piezometer with 1.5 m slotted screen installed with sand filter to 4.4 m, bentonite seal from 2.6 m to 0.6 m, cuttings to ground surface
09-95	-	-	Borehole backfilled with bentonite to 1.5 m, then cuttings to surface

4 LABORATORY TESTING

The recovered soil samples were subjected to Visual Identification (VI) and to natural moisture content determination. The results of this testing are shown on the Record of Borehole sheets in Appendix A. Selected samples were also subjected to gradation analysis and the results of this testing program are shown on the Record of Borehole sheets in Appendix A and on the figures contained in Appendix B.

Point load testing was conducted on rock core samples retrieved from the boreholes. The results of the point load tests are shown on the borehole logs in Appendix A.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets in Appendix A and Appendix C. Details of the encountered soil and rock stratigraphy are presented in these appendices and on the “Borehole Locations and Soil Strata” drawing in Appendix G. 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, the site is underlain by a surficial topsoil, fill and/or silty clay/clayey silt layer overlying compact to very dense/hard glacial till consisting of sand and silt, trace clay to clayey. Sand to sand and gravel was encountered within or below the till. Bedrock was encountered at relatively shallow depth at the south abutment and approach, and below the till in one borehole at the north abutment.

5.1 Topsoil/Organics

A 0.2 to 2.1 m thick layer of topsoil or peaty organics was encountered in the boreholes drilled at the north abutment and approach. The topsoil was loose to compact with SPT 'N' values of 7 to 21 blows/0.3 m. The lower boundary of the topsoil/organic layer was encountered at elevation 259.1 to 260.9 m.

In boreholes 09-47, 09-94 and 09-95, the organic layer was underlain by a further 0.6 to 1.1 m thick layer of dark brown clayey silt/silty clay with roots and rootlets. The silt/clay was firm ('N' values of 4 and 8). Moisture contents of 18 and 19% were determined. The underside of this layer was at elevation 258.9 to 260.3 m.

The topsoil/organic thickness may vary between and beyond the borehole locations, and the data presented in this report should not be used for quantity estimation purposes.

5.2 Fill

Fill was encountered in all boreholes at the pier, south abutment and south approach. The fill thickness ranged from 1.2 to 2.1 m and the underside of the fill layer was recorded at elevations between 258.6 m and 260.6 m. A 125 and 75 mm thick asphalt layer was encountered over the fill in boreholes 09-92A and 09-93 drilled on the existing roadway.

The fill typically comprises sand, trace gravel to gravelly and trace to some silt. Clayey silt fill was encountered in boreholes 09-91 and 09-92B.

Standard Penetration Tests 'N' values ranged from 10 to 60 blows/0.3 m in the sand fill, indicating a compact to very dense condition. 'N' values of 6 to 12 blows/0.3 m were obtained in the clayey silt fill, indicating a firm to stiff consistency.

Moisture contents ranged from approximately 5 to 19 % in the sand fill and 19 to 30% in the clayey silt fill.

The results of laboratory grain size distribution tests carried out on samples of the fill were as follows:

	<u>Sand Fill</u>	<u>Clayey Silt Fill</u>
Gravel (%)	1 to 14	0 to 1
Sand (%)	70 to 91	12 to 17
Silt (%)	8 to 14	66 to 67
Clay (%)	3 to 4	15 to 22

The grain size distribution curves for the samples tested during the current investigation are shown in Figure B1 in Appendix B. The results of Atterberg Limits testing conducted on a sample of clayey silt fill are presented on Figure B4.

5.3 Sand and Silt Till

A glacial till deposit consisting of sand and silt with variable content of clay and gravel was encountered below the organic layer and fill in all boreholes. The till contains cobbles and boulders.

At the south abutment and approach, the till overlies bedrock encountered at depths of 1.4 to 2.6 m (elevation 257.9 to 259.2 m), and varies from 0.2 to 1.2 m in thickness. At the pier, it overlies sand and gravel encountered at 7.0 and 5.4 m depth (elevation 254.2 and 256.4 m) and is 5.8 and 4.2 m thick. In the boreholes at the north abutment, the till variously extends to at least 7.7 m depth (elevation 252.8 m), overlies sand at 6.6 to 6.7 m depth (elevation 254.0 to 254.5 m), or extends to at least 10.8 m depth but is interrupted by a sand and gravel deposit. Refusal was encountered at 2.6 m depth (elevation 257.8 m) in the borehole at the north approach (borehole 09-47).

The results of laboratory grain size distribution tests carried out on samples of the till are illustrated in Figure B2, Appendix B. The results from the current and previous studies were as follows:

Gravel (%)	2 to 50
Sand (%)	23 to 59
Silt (%)	25 to 52
Clay (%)	5 to 20

The till is generally very dense, locally hard, with SPT 'N' values exceeding 50 blows for 0.3 m penetration. 'N' values of 9 to 11 blows/0.3 m were encountered in the upper 0.8 m of this deposit in three boreholes (boreholes 09-92A, 09-94 and 09-95), indicating a compact condition.

The moisture content of samples from this deposit ranged from about 5 to 20%, typically 5 to 12%.

5.4 Sand to Sand and Gravel

Sand, trace gravel, to sand and gravel was encountered below the till in boreholes 3, 4, 09-94 and 09-95, and within the till at 6.9 to 9.9 m depth in borehole 1. In boreholes 3, 4 and 09-95, the sand/gravel layer extended to the exploration depths of 7.8 to 10.9 m (elevation 250.9 to 252.9 m) and was at least 1.1 to 5.5 m thick. The sand was 0.4 m thick and underlain by bedrock at 7.0 m depth (elevation 254.1 m) in borehole 09-94.

The results of laboratory grain size distribution tests carried out on samples of the sand were as follows:

Gravel (%)	3 to 24
Sand (%)	65 to 86
Silt and Clay (%)	3 to 17

The grain size distribution curve for a sample tested during the current investigation is shown in Figure B3 in Appendix B.

Standard Penetration tests in the sand gave 'N' values exceeding 50 blows per 0.3 m penetration, indicating a very dense condition.

The moisture content of samples from this unit varies between 10 and 15%.

5.5 Bedrock

Bedrock and probable bedrock were encountered below the till and sand deposits in all boreholes at the south abutment and approach and in one borehole at the north abutment. Auger refusal was also encountered in borehole 09-47 at the north approach, however this may have occurred on a boulder in the till.

The depths to bedrock proved by coring or inferred by auger refusal are summarized in Table 5.1.

Table 5.1 – Depth to Bedrock at Borehole Locations

Location	BH Number	Depth to Bedrock (m)	Top of Bedrock Elevation (m)	Cored/ Inferred
South Approach	09-91	2.3	259.1	Cored
South Abutment	09-92A	2.6	259.0	Cored
	09-92B	1.4	258.8	Cored
	09-93	2.5	258.9	Cored
	5	2.6	257.9	Cored
	6	2.3	259.2	Inferred
North Abutment	09-94	7.0	254.1	Cored
North Approach	09-47	2.6	257.8	Inferred

The bedrock recovered in the cores was described as chert carbonate at the south abutment and grainstone at the north abutment during the current investigation. It was identified as chert with siderite (iron carbonate) during the MTO investigation. Shale partings and greenalite granules were also noted. The bedrock is considered to be from the Gunflint Formation.

The bedrock is described as thinly banded and fresh to slightly weathered. Its colour is charcoal grey/greyish black to olivine green/light olive grey.

Core recovery in the bedrock was between 83% and 100%. RQD values recorded during the current investigation generally ranged from 72 to 100% indicating good to excellent rock quality. Lower RQD values of 28 and 50% were encountered in two runs, indicating poor quality. RQD values recorded during the MTO study (borehole 5) ranged from 33 to 83%, indicating poor to good rock quality.

The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, ranged from 0 to 6. Rubble zones of 400 and 200 mm in thickness were noted in borehole 09-93. Vertical and sub-vertical fractures were encountered at various depths.

The unconfined compressive strength of the rock, estimated from the results of point load tests conducted on the rock core samples, ranges between 145 and 376 MPa, indicating a very strong to extremely strong intact rock. The point load test results are included on the borehole logs in Appendix A.

5.6 Groundwater

The groundwater depths and elevations observed in the boreholes upon completion of drilling and subsequently measured in the piezometers are shown in Table 5.2.

Table 5.2 – Groundwater Depths and Elevations

Location	Borehole	Borehole Completion Date	Water Levels on Completion (m)		Water Levels in Piezometers		
			Depth (m)	Elevation (m)	Date	Depth (m)	Elevation (m)
North Approach	09-47	14-July-07	Dry	-	-	-	-
North Abutment	09-94	14-Dec-09	*	-	17-Dec-09 01-Mar-10	1.9 2.2	259.2 258.9
	09-95	10-Dec-09	4.2	256.5	-	-	-
	1	15-Aug-91	4.5	257.4	-	-	-
	2	22-Aug-91	2.0	258.5	-	-	-
Pier	3	22-Aug-91	-	-	25-Aug-91	1.5	259.7
	4	20-Aug-91	-	-	25-Aug-91	2.0	259.8
South Abutment	09-92A	24-Jun-09	*	-	-	-	-
	09-92B	26-Jun-09	*	-	23-Nov-09 01-Mar-10	1.9 2.4	258.3 257.8
	09-93	25-Jun-09	*	-	-	-	-
	5	19-Aug-91	*	-	-	-	-
	6	20-Aug-91	Dry	-	-	-	-
South Approach	09-91	25-Jun-09	*	-	-	-	-

* Where rock coring was carried out, water was introduced into the boreholes as part of the coring operation and therefore water levels were not recorded upon completion.

The above water levels reflect the unstabilized conditions in the boreholes upon completion of drilling or the piezometric head at the level of the piezometer tips at the time of the readings. The measurements are short-term observations and seasonal fluctuations of the groundwater level are to be expected.

Based on the water levels measured in the piezometers, the groundwater level is expected to vary from about elevation 259.0 m at the north abutment, to elevation 259.8 m at the centre pier, and elevation 258.0 m at the south abutment. The groundwater level at the time of underpass construction may vary depending upon staging of the highway cut.

6 MISCELLANEOUS

J.D. Barnes Limited determined the co-ordinates and ground elevations at the boreholes prior to or following completion of the site investigation.

TBT Engineering Consulting Group of Thunder Bay, Ontario supplied and operated the drilling and sampling equipment for the current field program. Full time supervision of the field activities, including obtaining utility clearances, was carried out by Mr. Stephane Loranger and Mr. Luke Gilarski of Thurber.

Supervision of the field program, interpretation of the field data, and preparation of the report was performed by Mr. Tony Harte and Mr. Murray Anderson, P.Eng. The report was reviewed by Mr. Alastair Gorman, P.Eng., and by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 GENERAL

This report presents interpretation of the geotechnical data in the factual report and presents geotechnical recommendations to assist the design team to select and design a suitable foundation system and approach embankments for the proposed structure.

It is understood that Hodder Avenue/Copenhagen Road will cross over Highway 11/17 via a two-span structure with a total length of 67 m. Finished grades on Hodder Avenue will be near elevation 264.3 and 263.7 m at the north and south abutments, respectively. Finished grades on Highway 11/17 at the structure will be at elevation 256.7 (eastbound lanes) and 257.0 m (westbound lanes).

Existing grades at the structure range from about elevation 260.2 to 261.9 m. Construction of the grade separation will therefore require a cut of approximately 3 to 5 m for Highway 11/17 and a fill of approximately 2 to 4 m for the approach embankments on Hodder Avenue.

The discussion and recommendations presented in this report are based on our understanding of the project and on the factual data obtained in the course of the current and previous investigations.

8 STRUCTURE FOUNDATIONS

In general, the site is underlain by a surficial topsoil, fill and/or silty clay/clayey silt layer overlying compact to very dense/hard glacial till consisting of sand and silt, trace clay to clayey, with cobbles and boulders. Sand to sand and gravel was encountered within or below the till. Bedrock was encountered at depths of 1.4 to 2.6 m (elevation 257.9 to 259.2 m) at the south abutment, and below the till at 7.0 m depth (elevation 254.1 m) in one borehole at the north abutment.

Groundwater was observed upon completion of drilling or measured in piezometers at depths of 1.5 to 4.5 m (elevation 256.5 to 259.8 m). The preconstruction groundwater level is expected to be about elevation 259.0 m at the north abutment, elevation 259.8 m at the centre pier, and elevation 258.0 m at the south abutment.

Foundation alternatives are presented in the following sections together with the corresponding geotechnical design parameters. A foundation scheme preferred from a foundations perspective is recommended.

A comparison of the technical advantages and disadvantages of alternative foundation schemes is presented in Appendix D. Initial consideration was given to spread footings on native soil or engineered fill, driven steel H-piles, and caissons (drilled shafts).

8.1 Spread Footings on Native Soil and Bedrock

Footings supporting the north abutment and pier of the proposed structure should be founded on native, undisturbed very dense/hard sand and silt till at a minimum frost penetration depth of 2.2 m below finished grade. Footings supporting the south abutment should be constructed on bedrock.

The highest founding levels recommended to provide acceptable values of geotechnical resistance and achieve the frost depth, based on the proposed highway cut geometry shown on the General Arrangement drawing, are provided in Table 8.1 below. The recommended geotechnical resistances at these levels, assuming a typical footing width of 2 to 4 m, are also presented in Table 8.1.

Table 8.1 – Recommended Founding Levels and Resistance Values

Foundation Unit	Founding Stratum	Recommended Founding Level	Resistance Values
North Abutment	Very dense/hard sand/silt till	At or below Elev. 256.4 m	Factored ULS = 750 kPa SLS = 500 kPa
Pier	Very dense sand and gravel	At or below Elev. 253.4 m	Factored ULS = 750 kPa SLS = 500 kPa
South Abutment	Bedrock	Elev. 259.2 m (highest rock elevation)	Factored ULS = 5,000 kPa SLS will not govern

At the south abutment, a founding level at the highest rock elevation encountered in the boreholes, shown in Table 8.1, is recommended to minimize the requirement for rock excavation. At this foundation, all earth materials should be excavated to the bedrock surface, any shattered rock or rock fragments removed from the bearing surface, and grades brought back up to the founding level using concrete of the same class as the footing concrete. Alternatively, rock excavation to a lower founding elevation may be considered in lieu of mass concrete.

The rock mass below the footing must be sound and not subject to sliding, rocking or toppling. The edge of the footing closest to the rock face should be placed a horizontal distance of at least 2.0 m from the rock face.

The resistance values are for vertical, concentric loads. Where eccentric or inclined loads are applied, the resistance used in design must be reduced in accordance with the CHBDC Clause 6.7.3 and Clause 6.7.4.

The sliding resistance of the footings may be computed using unfactored friction coefficients of 0.55 on very dense/hard till and 0.7 on sound bedrock. These are “ultimate” values and require a degree of sliding movement to occur to fully mobilize the resistance.

The bearing surfaces should be prepared by removing all loose/disturbed material and shattered rock. The exposed till surface should be protected from deterioration by placing a minimum 100 mm thick working mat of concrete of the same class as the footing within 4 hours of completing excavation. Areas requiring subexcavation beneath the underside of footing should be backfilled with the same class of concrete as used in the footing.

The recommended founding levels are approximately 2.6 and 6.4 m below the prevailing groundwater levels measured in the piezometers at the north abutment and pier, respectively. The actual water levels encountered at the time of foundation construction will depend upon the construction sequencing. As will be discussed in Section 9 of this report, it is recommended that the mainline cut be completed in advance of bridge construction to allow drawdown of the groundwater to occur, thus reducing the dewatering effort required for footing construction.

8.2 Spread Footings on Engineered Fill

In view of the high resistances available for design of spread footings constructed on the very dense/hard till and bedrock at shallow depth, the use of spread footings on engineered fill is not considered to be a suitable option and has not been developed further.

8.3 Steel Pile Foundations

If integral abutment design is preferred for this structure, consideration may be given to the use of steel H-pile foundations.

To achieve an adequate pile length, pile installation will require coring or trench excavation in rock at the south abutment, and pre-augering into the very dense/hard glacial till as well as rock coring/trenching at the north abutment. Pre-augering will also be required at the pier if piles are employed for this foundation.

The pile length should be no less than 6 m. The recommended pile tip elevations are presented in Table 8.2 for an underside of abutment or pile cap with a minimum frost cover of 2.2 m below the finished grades shown on the General Arrangement drawing. The actual pile tip elevation may vary depending upon the abutment type and design (ie., false abutment).

Table 8.2 – Recommended Pile Tip Elevations

Foundation Unit	Pile Tip Elevation (m)	Founding Stratum
North Abutment	250.4	Very dense sand/silt till and bedrock
Pier	247.4	Very dense sand and gravel, possibly bedrock
South Abutment	252.0	Bedrock

HP 310x110 piles founded in the very dense/hard till or bedrock at the above levels should be designed using the following geotechnical resistances:

	<u>Very Dense/Hard Till</u>	<u>Bedrock</u>
Factored Geotechnical Resistance at ULS =	1,800 kN	2,000 kN
Geotechnical Resistance at SLS =	1,400 kN	Will not govern

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

8.3.1 Downdrag

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

8.3.2 Pile Tips

Driving shoes or rock points are not considered necessary as the piles will be installed by pre-augering in the very dense till and coring or trenching into bedrock without driving. If driving is carried out, the tips of all piles should be fitted with H-section rock points from an approved manufacturer such as Titus Steel (Standard H-point), Pruyn Points or approved equivalent.

8.3.3 Pile Installation

Pile installation should be in accordance with Special Provision No. 903S01.

The Contract Documents should contain a NSSP alerting the Bidders to:

- The very dense/hard condition of the glacial till at this site. Installation of piles within the till soils will require pre-augering. Augering in the till will be laboured. The presence of cobbles and boulders in the glacial till may present obstructions to pile driving or augering equipment, and coring may be required to penetrate larger boulders.
- The presence of bedrock at relatively shallow depth at the south abutment and potentially within the pile depth at the north abutment. Rock coring and/or rock trenching equipment will be required to advance the pile sockets to the required depth in bedrock.

Suggested texts for NSSP's are included in Appendix E.

8.3.4 Pile Lateral Resistance

The geotechnical lateral resistance acting on a pile in sand and silt till may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

$$k_s = n_h \cdot z / D \quad (\text{kN/m}^3)$$

$$p_{ult} = 3 \cdot \gamma \cdot z \cdot K_p \quad (\text{kPa})$$

where z = depth of embedment of pile in metres

D = pile width in metres

n_h = coefficient of horizontal subgrade reaction
= 10,000 kN/m³ in very dense/hard till

γ = unit weight
= 11 kN/m³ (buoyant unit weight below water table)

K_p = passive earth pressure coefficient
= 3.7 for very dense/hard till

For the bedrock at the south abutment, an ultimate lateral resistance, p_{ult} , of 2,500 kPa may be assumed. Lateral deflection in a bedrock socket is expected to be negligible.

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 must not exceed the ultimate lateral resistance. The analysis must also take account of the strain compatibility between the pile and the soil, especially in the case of a pile socketed in bedrock which will form a stiff structural element.

The spring constant, K_s , for analysis may be obtained by the expression, $K_s = k_s \times L \times D$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m³), 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, P_{ult} , may be obtained from the expression, $P_{ult} = p_{ult} \times L \times D$. 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 170 kN at ULS and 70 kN at SLS.

The modulus of subgrade reaction may have to be reduced, based on the pile spacing. The reduction factors to be used for a pile group oriented perpendicular or parallel to the direction of loading are provided in Table 8.3. Intermediate values may be obtained by linear interpolation.

Table 8.3 – Subgrade Reaction Reduction Factors for Pile Spacing

Condition	Pile Spacing, Centre to Centre*	Reduction Factor
Pile group oriented <i>perpendicular</i> to direction of loading	4D	1.0
	1D	0.5
Pile group oriented <i>parallel</i> to direction of loading	8D	1.0
	6D	0.7
	4D	0.4
	3D	0.25

* where D is the breadth of pile

In the case of conventional abutments, i.e. not integral, horizontal loads may be resisted by means of battered piles.

8.3.5 Integral Abutment Considerations

The use of H-piles at the abutments allows for the design of an integral abutment structure. The integral abutment design requires that the piles possess flexibility in the upper 3 m of the pile length. At this site, the near surface materials consist of very dense/hard till or bedrock, and the lateral resistance of a pile may not provide sufficient flexibility. Accordingly, to provide the required flexibility in the piles, the upper 3 m of the piles should be surrounded by a 600 mm diameter CSP as specified by the integral abutment design procedures.

After the pile is installed, the space between the pile and the CSP should be filled with sand. An NSSP should be included in the contract drawings specifying the gradation of the sand according to Table 8.4.

Table 8.4 – 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%

8.4 Caissons

In view of the high resistances available for design of spread footings constructed on the very dense/hard till and bedrock at shallow depth, the use of caissons is not considered to be a cost-effective option and has not been developed further.

8.5 Recommended Foundation

From a geotechnical perspective and based on the subsurface conditions, spread footings founded on bedrock at the south abutment and on native undisturbed very dense/hard glacial till at the north abutment and pier is considered the most cost effective foundation option for supporting the grade separation structure at this site.

8.6 Frost Cover

The depth of frost penetration at this site is 2.2 m. The base of footings and pile caps on earth must be provided with a minimum of 2.2 m of earth cover as protection against frost action. Frost protection is not required for footings on sound rock.

9 EXCAVATION AND DEWATERING

Existing grades at the proposed structure location range from about elevation 260.2 to 261.9 m. Finished grades on Highway 11/17 will be at elevation 256.7 (eastbound lanes) and 257.0 m (westbound lanes), with the deepest part of the cut (in the centre median) extending to elevation 255.6 m. Highway 11/17 in the vicinity of the structure will therefore lie in a cut of approximately 3 to 6 m in depth. The highway profile will slope down at 3% to Current River west of the site.

Staging drawings for the project indicate that the cut for the westbound lanes will be constructed during an earlier stage (Stage 2) than structure construction (Stage 5), and the eastbound lanes will be constructed during the same stage as the structure.

Excavation of the highway cut will be carried out within the surficial organic materials, existing fill, native dense/hard sand and silt till, and into bedrock towards the south abutment. Subsequent excavation for foundation construction, extending about 2.2 m below the highway cut (to elevation 256.4 m at the north abutment and elevation 253.4 m at the pier), will be carried out within very dense/hard sand and silt till, and into very dense sand and gravel at the pier.

The preconstruction groundwater level is expected to be about elevation 259.0 m at the north abutment, elevation 259.8 at the centre pier, and elevation 258.0 m at the south abutment. The highway cut will therefore extend up to 4.2 m below the groundwater level. As the westbound mainline cut will be completed in advance of bridge construction, the water level at the time of foundation construction should be lowered to a level at or below the highway cut base.

It is recommended that the mainline cut be completed at least three months in advance of bridge construction to allow drawdown of the groundwater to occur, thus reducing the dewatering effort required at the structure. The cut should begin at the low end of the highway to permit drainage as excavation progresses.

Provided the work is carried out in accordance with this staging program, foundation excavation at the north abutment will be carried out above the post-construction groundwater level. Excavation for construction of the pier footing will extend into sand and gravel up to 2.2 m below the groundwater level established after cut excavation.

All excavations must be carried out in accordance with the requirements of the Occupational Health and Safety Act (OHSA) and in accordance with Special Provision 902S01. For the purposes of the OHSA, the inorganic soils within the depth of excavation may be classed as Type 1 above the water table and Type 3 below the water table. Flatter slopes may be required at locations where water seepage affects surficial stability.

The selection of the method of excavation is the responsibility of the contractor and must be based on his equipment, experience and interpretation of the site conditions. It is anticipated that a hydraulic excavator will be suitable. Provision must be made for the handling of pavement materials, potential obstructions in the fill, and cobbles, boulders and rock slabs in the till soils. Arduous excavation should be expected in the very dense/hard till materials.

A NSSP should be included in the contract alerting the Contractor to the presence of cobbles and boulders in the till deposits, possible arduous excavation, and potential issues during establishment of the founding surface. Suggested wording is provided in Appendix E, "Foundation Excavation".

The design of the dewatering system that may be required is the responsibility of the Contractor and the Contract Documents must alert him to this responsibility. The dewatering effort required will depend upon the effectiveness of the mainline cut in lowering the groundwater table, particularly for construction of the pier.

While the responsibility for dewatering should remain with the Contractor, suitable systems that might be employed include pumping from filtered sumps to remove any water seeping into foundation excavations within the very dense/hard till. Depending upon the effectiveness of the mainline cut drainage, the use of additional pumps and/or vacuum wellpoints may be required to lower the water level at least 0.5 m below the excavation base where excavation penetrates into the sand and gravel deposits.

The excavation and backfilling for foundations must be carried out in accordance with SP 902S01. All footings must be constructed in the dry.

Excavations should be inspected regularly for evidence of instability if they have been left open for extended periods of time and following periods of heavy rain or thawing. If required, remedial actions must be taken to ensure the stability of the excavation and the safety of workers.

Considering the very dense condition of the sand/silt till deposit at the site, temporary and permanent cut slopes are expected to be stable at inclinations of 1H:1V and 2H:1V, respectively. The highway cut slopes should be examined to identify any areas of continuing seepage and instability, and measures such as a minimum 450 mm thick layer of granular sheeting should be provided in areas of concentrated seepage as needed.

10 APPROACH EMBANKMENTS

The foundation soils governing stability of the approach embankments consist of existing native very dense/hard till and bedrock. It is anticipated that the maximum embankment height will be about 4.0 m.

Embankment construction should be in accordance with OPSS 206, as amended by Special Provision “Amendment to OPSS 206, December 1993”, dated November 2002. It is recommended that earth fill should consist of SSM, granular materials or local inorganic till in compliance with Special Provision 110F13, “Amendment to OPSS 1010 March 1993”.

The embankment foundation soils are considered to provide adequate stability to earth fills inclined at 2H:1V and rockfills inclined at 1.25H:1V.

Considering the embankment height and competency of the foundation soils, post construction settlement induced by embankment loading will be less than 25 mm.

All topsoil, organic soils and existing fill should be stripped from the footprint of the approach fills. Particular attention should be paid to removing all organics and softened material from existing ditches that fall within the footprint of the new embankment.

Earth fill embankment slopes must be provided with erosion protection in accordance with SP572S01.

11 RETAINED SOIL SYSTEMS

Retained soil system (RSS) walls may be used subject to the requirements presented in this section.

RSS walls used in conjunction with the new grade separation structure abutments must be “High Performance” and “High Appearance”. The contract drawings should include information on the longitudinal alignment of the wall in plan, the top and base elevations of the wall in profile, cross-sectional space constraints and an NSSP for the RSS wall.

The performance of a RSS is dependent on, among other factors, the characteristics of its foundation. Failure to provide an adequate foundation may lead to settlement and distortion of the RSS and, in severe cases, to possible failure of the system. It is critical that the RSS walls are not subject to settlement due to compression of the foundation soils and embankment fill. The foundation of the entire RSS mass must be considered, i.e. from the face of the wall to the furthest extent of the reinforcement.

The borehole information indicates that the soil conditions at the wall locations comprise organic deposits and existing fill overlying generally very dense/hard till and/or bedrock. The till and bedrock are generally suitable for the support of RSS walls.

To provide an acceptable foundation performance, the RSS mass must be founded on the native undisturbed very dense/hard till or bedrock. The highest base levels for the underside of the wall are indicated in Table 11.1.

Table 11.1 – Maximum Elevation at Underside of Wall Base or Granular A Fill

RSS Location	Boreholes	Base Elevation (m)
North Abutment		
West Side	1, 09-94	259.8
East Side	2, 09-95	259.0
South Abutment		
West Side	5, 09-92B	258.6 or bedrock
East Side	6, 09-92A	259.5

Walls founded on native very dense/hard till or bedrock at or below the elevations shown in Table 11.1 should be designed for the following geotechnical resistances:

	<u>Very Dense/Hard Till</u>	<u>Bedrock</u>
Factored Geotechnical Resistance at ULS =	450 kPa	5,000 kPa
Geotechnical Resistance at SLS =	300 kPa	Will not govern

Engineered fill placed under the RSS mass to achieve the design founding level should consist of OPSS Granular “A” compacted to 100% of its SPMDD at a moisture content within 2% of optimum. The engineered fill pad must extend at least 500 mm beyond the limits of the RSS mass and levelling strip.

The geotechnical resistances provided above are for concentric, vertical loading. The effects of load inclination and eccentricity need to be taken into account according to the CHBDC 2006 Section 6.7.

The entire block of reinforced earth must be designed against various modes of failure including sliding and overturning. Sliding resistance along the base of the wall on very dense/hard till or engineered granular fill may be estimated using an ultimate friction coefficient of 0.55. For an RSS block founded on bedrock, an ultimate coefficient of sliding friction of 0.7 may be used.

Topsoil, fill, soft/loose soil, and any soft/wet native material should be stripped from the footprint of the RSS. The native soil under the RSS foundation should be proofrolled to detect and replace any soft areas.

The proprietary RSS system must meet the Ministry’s specifications for performance and appearance. The RSS supplier/designer may specify more stringent criteria or other requirements related to the particular design. The internal stability of the RSS wall should be analyzed by the supplier/designer of the proprietary product selected for this site.

The global stability of the RSS wall is dependent on the characteristics of the foundation soils, the geometry of the embankment and location of the RSS within the embankment. Typically, global stability should not be a concern for a RSS wall founded on the native undisturbed till and bedrock at this site.

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. A NSSP is required to limit rock fill used as abutment backfill to fragments no greater than 300 mm and to include adequate spalls to fill voids in the rock fill.

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 should be in accordance with OPSS 902 as amended by Special Provision 902S01. Granular backfill should be placed to the extents shown in OPSD 3101.150, and rock backfill should be placed to the extents shown in OPSD 3101.200.

All granular material should meet the specifications of Special Provision 110F13 “Amendment to OPSS 1010, March 1993”. Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with SP 105S10.

The design of the abutment must include a subdrain as shown in OPSD 3102.100.

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)

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

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 at a depth of 1.7 m for Granular A or Granular B Type II.

Table 13.1 – Earth Pressure Coefficients (K)

Condition	Earth Pressure Coefficient (K)					
	OPSS 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 Backfill (2H:1V)	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)
Active (Unrestrained Wall)	0.27	0.38*	0.31	0.46*	0.20	0.26*
At Rest (Restrained Wall)	0.43	-	0.47	-	0.33	-
Passive	3.7	-	3.3	-	5.0	-

* For wing walls.

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. In the case of integral abutments, material with a lower passive pressure coefficient (e.g. Granular B, Type I) might be preferred as it results in lower forces acting on the ballast wall as the soil moves towards the soil mass.

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.

14 SEISMIC CONSIDERATIONS

14.1 Seismic Design Parameters

The following seismic parameters should be used for design:

- Velocity Related Seismic Zone 0
- Zonal Velocity Ratio 0.0
- Acceleration Related Seismic Zone 0
- Zonal Acceleration Ratio 0.0
- Peak Horizontal Acceleration 0.02

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 foundation soils at the site are assessed as not being prone to liquefaction.

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 Backfill (2H:1V)	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)
Active (K_{AE})*	0.28	0.41	0.32	0.49	0.20	0.28
Passive (K_{PE})	3.7	-	3.2	-	5.0	-
At Rest (K_{OE})**	0.45	-	0.50	-	0.36	-

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

** After Woods

15 CONSTRUCTION CONCERNS

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

- Excavation for footing and pile cap construction is expected to encounter very dense/hard till containing cobbles and boulders. Excavation may be laborious and require removal of numerous and/or large boulders.
- Provision of a level foundation base in the till may be difficult if large boulders are encountered at the excavation base level. Excavation should be carried out in a manner that minimizes disturbance to the subgrade.
- The bedrock surface elevation at the south abutment may vary. Removal of elevated portions of the bedrock may be required in addition to placement of mass concrete to provide a level founding surface.
- Augering for installation of piles, if employed, may be slow and laborious due to the presence of very dense/hard till with cobbles and boulders. Coring may be required to penetrate larger boulders. Coring will be required to penetrate the bedrock.

- Excavation below the groundwater level at the time of construction will require a dewatering system suitable to maintain a sound, dry excavation for footing or pile cap construction. The dewatering effort required will depend upon the effectiveness of the mainline cut in lowering the groundwater table, particularly for construction of the pier foundation.

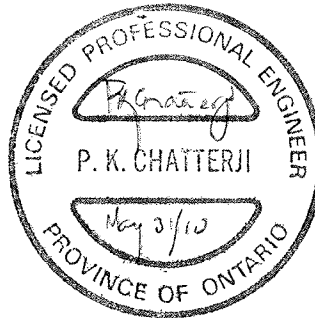
16 CLOSURE

Engineering analysis and preparation of the foundation design report were carried out by Mr. Murray Anderson, P.Eng. The report was reviewed by Mr. Alastair E. Gorman, P.Eng. The report was also reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

Thurber Engineering Ltd.
Murray R. Anderson, P.Eng., M.Eng.
Senior Geotechnical Engineer



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
 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			

RECORD OF BOREHOLE No 09-047

1 OF 1

METRIC

G.W.P. 334-94-00 LOCATION N 5 372 030.9 E 365 262.0 ORIGINATED BY SLL
 HWY 11/17 BOREHOLE TYPE Hollow Stem augers COMPILED BY AN
 DATUM Geodetic DATE 2009-07-14 - 2009-07-14 CHECKED BY TH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								20 40 60 80 100						
								20 40 60 80 100						
260.4														
0.0														
259.5														
0.9			1	SS	8									
258.9														
1.5			2	SS	61									4 34 48 14
257.8			3	SS	50/									
2.6														

RECORD OF BOREHOLE No 09-092A

1 OF 1

METRIC

G.W.P. 334-94-00 LOCATION N 5 371 947.2 E 365 269.1 ORIGINATED BY LG
 HWY 11/17 BOREHOLE TYPE Hollow Stem Augers/BQ Coring COMPILED BY MFA
 DATUM Geodetic DATE 2009.06.24 - 2009.06.24 CHECKED BY TH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
261.6								20 40 60 80 100						
0.0	ASPHALT: (125mm)													
0.1	SAND, some silt, some gravel Compact Black Moist (FILL)		1	SS	21									13 70 14 3
260.2														
1.4	SAND and SILT, trace gravel, trace clay Compact to Very Dense Dark Brown Moist (TILL)		2	SS	11									9 47 36 8
259.0			3	SS	100/ .150									
2.6	GUNFLINT FORMATION, very strong to extremely strong (chert carbonate), thinly banded, fresh to slightly weathered, charcoal grey to olive green Vertical fracture at 3.4 to 3.7m		1	RUN										RUN 1# TCR=100%, SCR=61%, RQD=28%, UCS=257MPa
			2	RUN										RUN 2# TCR=100%, SCR=100%, RQD=100%, UCS=281MPa

RECORD OF BOREHOLE No 09-092B

1 OF 1

METRIC

G.W.P. 334-94-00 LOCATION N 5 371 947.2 E 365 256.1 ORIGINATED BY LG
 HWY 11/17 BOREHOLE TYPE Hollow Stem Augers/BQ Coring COMPILED BY MFA
 DATUM Geodetic DATE 2009.06.26 - 2009.06.26 CHECKED BY TH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60						80	100	20
260.2																		
0.0	Clayey SILT, some sand, trace gravel Firm to Stiff Brown Moist to Wet (FILL)		1	SS	6													
259.0			2	SS	12													
258.8	SAND and SILT, trace gravel, trace clay Compact Brown Moist (TILL)		3	SS	100/													
1.4					.075													
	GUNFLINT FORMATION, very strong to extremely strong (chert carbonate), thinly banded, fresh to slightly weathered, charcoal grey to olivine green Occasional shale partings. Sub-vertical fractures, planar, smooth Vertical fracture at 2.0 to 2.1m		1	RUN														
	Vertical fracture at 3.7 to 4.4m		2	RUN														
255.8																		
4.4	END OF BOREHOLE AT 4.4m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) 2009.11.23 1.9 258.3 2010.03.01 2.4 257.8																	

+³, X³: Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 09-093

1 OF 1

METRIC

G.W.P. 334-94-00 LOCATION N 5 371 953.4 E 365 262.6 ORIGINATED BY LG
HWY 11/17 BOREHOLE TYPE Hollow Stem Augers/BQ Coring COMPILED BY MFA
DATUM Geodetic DATE 2009.06.25 - 2009.06.25 CHECKED BY TH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	*N* VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)
261.4								20	40	60	80	100				
0.0	ASPHALT: (75mm)							○ UNCONFINED	+	FIELD VANE						
0.1	SAND, trace silt, trace gravel Compact Brown to Black Moist (FILL)		1	SS	14		261	● QUICK TRIAXIAL	x	LAB VANE						
			2	SS	12		260									
259.3																
2.1	SAND and SILT, some gravel, trace clay, trace rock pieces Compact to Very Dense Moist to Wet (TILL)		3	SS	100/ 125		259									
258.9			1	RUN			258									
2.5	GUNFLINT FORMATION, very strong to extremely strong (chert carbonate), thinly banded, fresh to slightly weathered, charcoal grey to olive green Occasional shale partings. Sub-vertical fractures, planar, smooth Fracture at 2.7, 2.9, and 3.3m Vertical fractures at 2.8, and 3.5 to 3.7m		2	RUN			257									
			3	RUN			256									
	Rubble zone at 4.0 to 4.4, and 5.6 to 5.8m															
255.2																
6.2	END OF BOREHOLE AT 6.2m. BOREHOLE BACKFILLED WITH BENTONITE TO 0.1m, THEN COLD PATCH ASPHALT TO SURFACE.															

RECORD OF BOREHOLE No 09-094

1 OF 2

METRIC

G.W.P. 334-94-00 LOCATION N 5 372 019.4 E 365 254.2 ORIGINATED BY SLL
 HWY 11/17 BOREHOLE TYPE Hollow Stem Augers/BQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2009.12.11 - 2009.12.14 CHECKED BY TH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE				
261.1							20 40 60 80 100					
0.0	TOPSOIL: (150mm)						20 40 60 80 100					
0.2	Clayey SILT, trace gravel, trace sand Dark Brown						20 40 60 80 100					
260.3							20 40 60 80 100					
0.8	SAND and SILT, some gravel, trace clay Loose to Very Dense Brown Moist (TILL)		1	SS	9		260					
			2	SS	48							
			3	SS	100/ 250		259					
	occasional cobbles and boulders		4	SS	100/ .025		258					
			5	SS	100/ .150		257					
			6	SS	100/ .075		256					
254.5							255					
6.6	SAND, some gravel, some silt, occasional cobbles and boulders Very Dense Grey Moist		1	RUN			254					
254.1			2	RUN			253					
7.0	GUNFLINT FORMATION, very strong to extremely strong (grainstone), occasional quartz veining, carbonate rich layers Haematite precipitation on fracture planes, fractures subhorizontal		3	RUN			252					
	175mm sub-vertical joints at 8.9m											
251.1												

Continued Next Page

+³ ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

ONTMT4S 1156.GPJ 5/28/10

RUN 1#
TCR=100%,
SCR=87%,
RQD=72%
UCS=337MPa
RUN 2#
TCR=100%,
SCR=91%,
RQD=87%
UCS=267MPa
RUN 3#
TCR=100%,
SCR=100%,
RQD=90%
UCS=201MPa

Auger refusal at
2.2m, moved
0.6m west.
11 40 42 7

RECORD OF BOREHOLE No 09-094

2 OF 2

METRIC

G.W.P. 334-94-00 LOCATION N 5 372 019.4 E 365 254.2 ORIGINATED BY SLL
 HWY 11/17 BOREHOLE TYPE Hollow Stem Augers/BQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2009.12.11 - 2009.12.14 CHECKED BY TH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
	Continued From Previous Page																
10.0	END OF BOREHOLE AT 10.0m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) 2009.12.17 1.9 259.2 2010.03.01 2.2 258.9																

ONTMT4S 1156.GPJ 5/28/10

RECORD OF BOREHOLE No 09-095

1 OF 1

METRIC

G.W.P. 334-94-00 LOCATION N 5 372 019.2 E 365 269.2 ORIGINATED BY SLL
 HWY 11/17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2009.12.10 - 2009.12.10 CHECKED BY TH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE						WATER CONTENT (%) w _p w w _L		
260.7							20	40	60	80	100	20	40	60		
0.0	ORGANICS, black peat (275mm)															
260.4																
0.3	Silty CLAY, trace sand, rootlets Firm Dark Brown		1	SS	4							○				
259.3																
1.4	SAND and SILT, some gravel, trace clay, occasional cobbles and boulders Very Dense Brown Moist (TILL)		2	SS	10							○				8 51 32 9
			3	SS	100/ 250							○				
			4	SS	100/ .200							○				
			5	SS	100/ .150							○				
			6	SS	100/ .150							○				
254.0																
6.7	SAND, some silt, trace gravel, occasional cobbles Very Dense Grey Moist															
252.9			7	SS	100/ .150							○				3 86 11
7.8	END OF BOREHOLE AT 7.8m. BOREHOLE OPEN TO 6.3m AND WATER LEVEL AT 4.2m ON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE TO 1.5m, THEN CUTTINGS TO SURFACE.															(SI+CL)

+ 3, X 3: Numbers refer to
Sensitivity

20
15
10

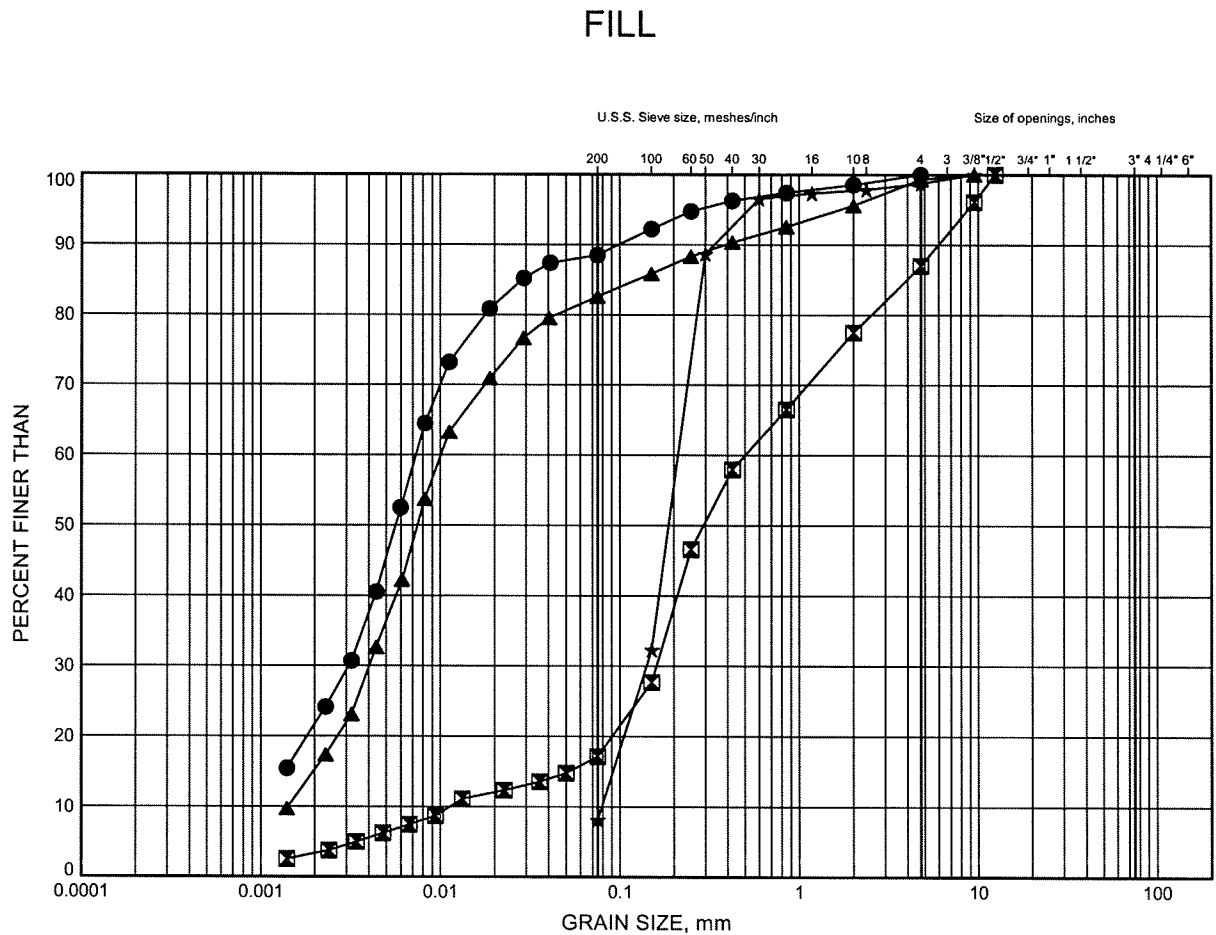
(%) STRAIN AT FAILURE

Appendix B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION

FIGURE B1



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

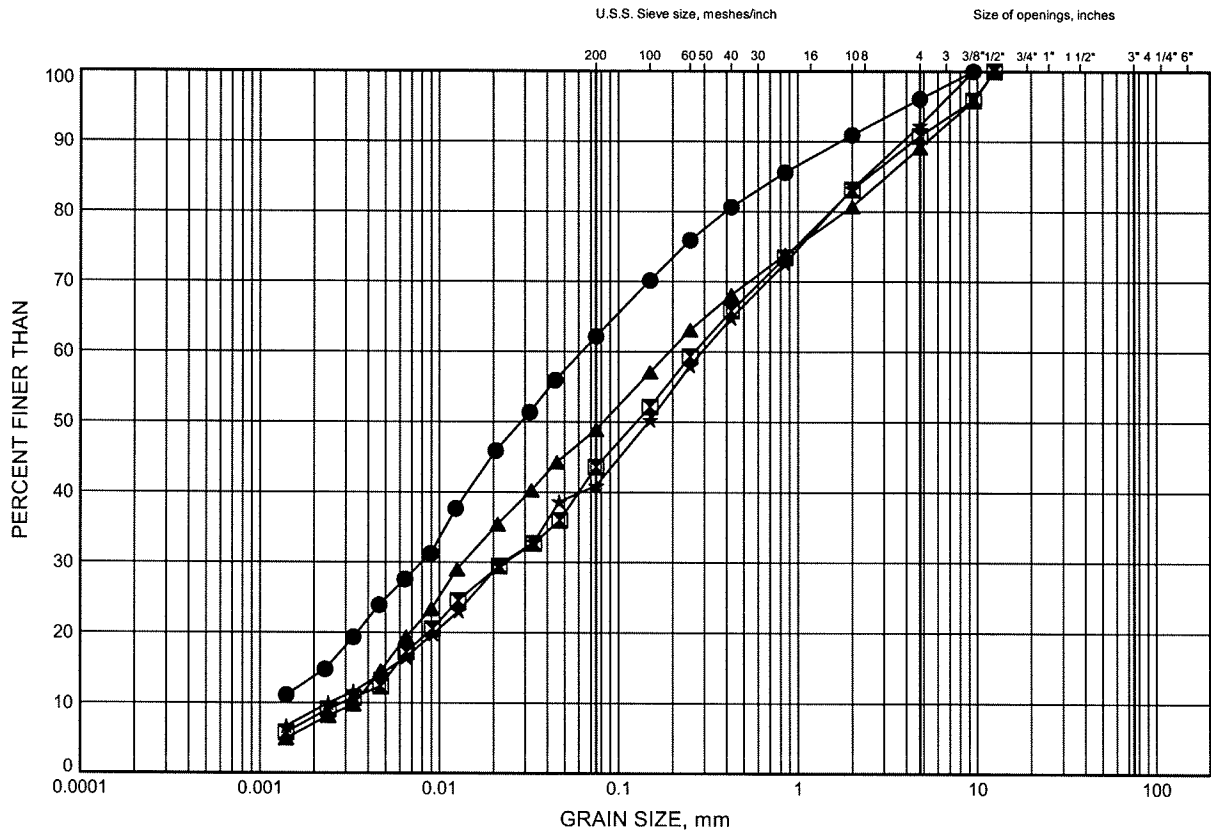
LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	09-091	1.75	259.65
⊠	09-092A	0.99	260.61
▲	09-092B	0.99	259.21
★	09-093	1.80	259.60

GRAIN SIZE DISTRIBUTION

FIGURE B2

SAND AND SILT TILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	09-047	1.83	258.57
⊠	09-092A	1.83	259.77
▲	09-094	2.49	259.01
★	09-095	1.83	259.75

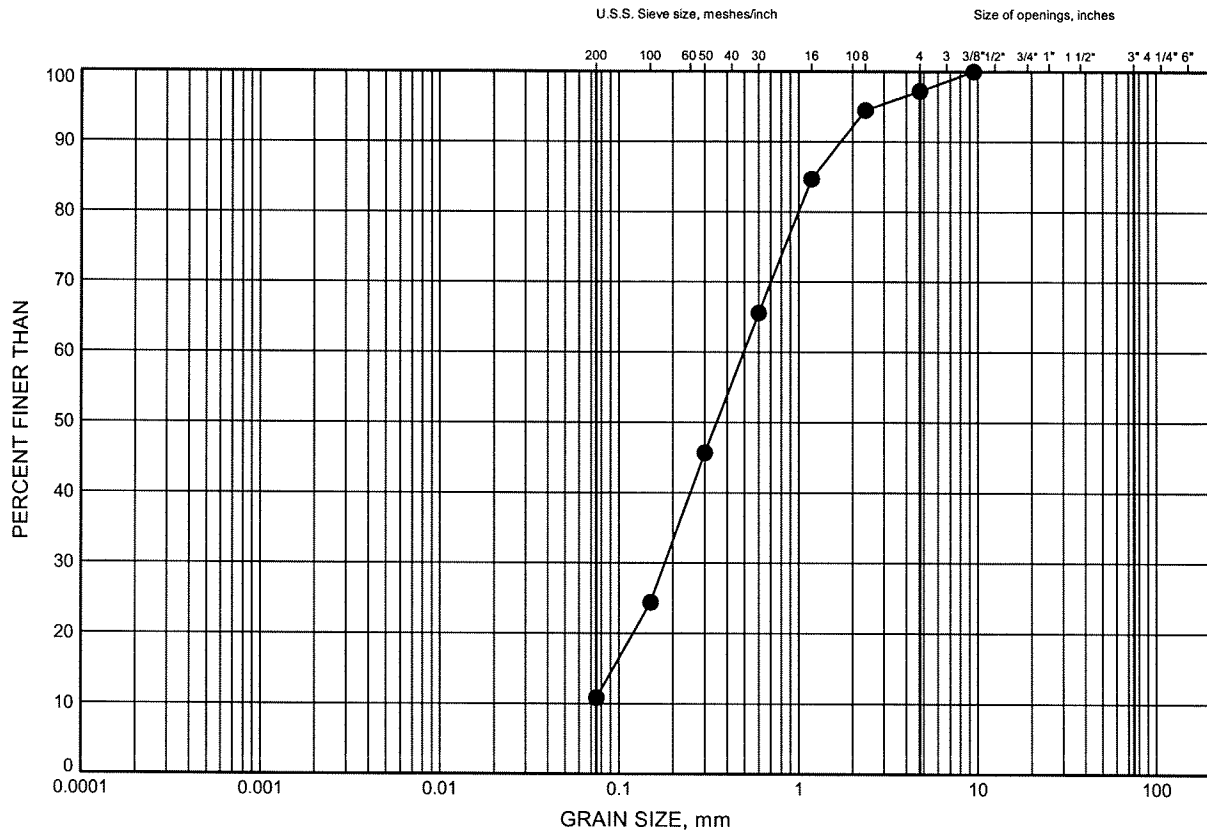


W.P.# 334-94-00
 Prepared By MFA
 Checked By MRA

GRAIN SIZE DISTRIBUTION

FIGURE B3

SAND



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	09-095	7.68	253.89



W.P.# 334-94-00
 Prepared By MFA
 Checked By MRA

FIGURE B4

The chart is a graph of Plasticity Index (Y-axis, 0 to 60) versus Liquid Limit (X-axis, 0 to 80). The 'A' line is a diagonal line starting from (0, 4) and passing through (40, 35). Vertical lines are drawn at Liquid Limit = 35 and Liquid Limit = 50. Horizontal lines are drawn at Plasticity Index = 4, 7, and 10. The regions are labeled as follows:

- CL (Clay of Low Plasticity): Above PI = 7, below the 'A' line, and to the left of LL = 35.
- CH (Clay of High Plasticity): Above PI = 10, to the right of LL = 35.
- CI (Clay of Intermediate Plasticity): Above PI = 10, below the 'A' line, and to the right of LL = 35.
- MI-OI (Silt of Medium to Intermediate Plasticity): Below PI = 7, to the right of LL = 35.
- MH-OH (Silt of Medium to High Plasticity): Below PI = 7, to the right of LL = 50.
- CL-ML (Clay of Low to Medium Plasticity): Below PI = 7, to the left of LL = 35.
- OL (Organic Clay): Below PI = 7, to the left of LL = 35, and below the 'A' line.
- ML (Low Plasticity Silt): Below PI = 4, to the left of LL = 35.

A data point is plotted at Liquid Limit = 30 and Plasticity Index = 9. This point falls within the CL region.

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	09-091	1.75	259.65

Date February 2010
Project 334-94-00



Prep'd MFA
Chkd. MRA

Appendix C

Factual Data from Previous Reports

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 31MM O.D. SPT BARREL SAMPLER TO PENETRATE 0.3M INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5KG, FALLING FREELY A DISTANCE OF 0.76M FOR PENETRATIONS OF LESS THAN 0.3M. N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS \bar{N} .

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (31MM O.D. 60° CONE ANGLE) DRIVEN BY A 100KG IMPACT ENERGY ON A 12.5MM DIAMETER RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF KILOGRAMS PER SQUARE CENTIMETER ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSITY.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	> 200
Consistency	VERY SOFT	SOFT	MEDIUM	STIFF	VERY STIFF	HARD

DENSITY: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSITY AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/30cm)	0 - 3	3 - 10	10 - 30	30 - 50	> 50
Density	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND/OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100MM IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD) OF THE ROCK MODIFIED RECOVERY IS:

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
Recovery	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINTING AND BEDDING:

SPACING	50mm	50 - 100mm	100 - 200mm	200 - 300mm	> 300mm
JOINTING	VERY CLOSE	CLOSE	MEDIUM	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

ABBREVIATIONS AND SYMBOLS

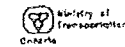
FIELD SAMPLING		MECHANICAL PROPERTIES OF SOIL	
S.S. SPLIT SPOON	T.P. THINWALL PISTON	m_v	COEFFICIENT OF VOLUME CHANGE
W.S. WASH SAMPLE	O.S. OSTERBERG SAMPLE	C_c	COMPRESSION INDEX
S.T. SLOTTED TUBE SAMPLE	R.C. ROCK CORE	C_s	SWELLING INDEX
B.S. BLOCK SAMPLE	P.H. T.W. ADVANCED HYDRAULICALLY	C_d	RATE OF SECONDARY CONSOLIDATION
C.S. CHUNK SAMPLE	P.H. T.W. ADVANCED MANUALLY	C_v	COEFFICIENT OF CONSOLIDATION
T.W. THINWALL OPEN	F.S. FOIL SAMPLE	H	DRAINAGE PATH
		T_v	TIME FACTOR
		U	DEGREE OF CONSOLIDATION
		σ'_{vo}	EFFECTIVE OVERBURDEN PRESSURE
		σ'_p	PRECONSOLIDATION PRESSURE
		τ_f	SHEAR STRENGTH
		c'	EFFECTIVE COHESION INTERCEPT
		ϕ'	EFFECTIVE ANGLE OF INTERNAL FRICTION
		c_u	APPARENT COHESION INTERCEPT
		ϕ_u	APPARENT ANGLE OF INTERNAL FRICTION
		τ_R	RESIDUAL SHEAR STRENGTH
		τ_f	REMOULDED SHEAR STRENGTH
		I_s	SENSITIVITY = $\frac{C_u}{C_c}$

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
u	kPa	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ		COEFFICIENT OF FRICTION

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m ³	DENSITY OF SOLID PARTICLES	e	1.0	VOID RATIO	e_{min}	1.0	VOID RATIO IN DENSEST STATE
γ_s	kN/m ³	UNIT WEIGHT OF SOLID PARTICLES	n	1.0	POROSITY	I_D	1.0	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m ³	DENSITY OF WATER	w	1.0	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m ³	UNIT WEIGHT OF WATER	S_r	1.0	DEGREE OF SATURATION	D_n	mm	N PERCENT - DIAMETER
P	kg/m ³	DENSITY OF SOIL	w_L	1.0	LIQUID LIMIT	C_u	1.0	UNIFORMITY COEFFICIENT
γ	kN/m ³	UNIT WEIGHT OF SOIL	w_p	1.0	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m ³	DENSITY OF DRY SOIL	w_s	1.0	SHRINKAGE LIMIT	q	m ² /s	RATE OF DISCHARGE
γ_d	kN/m ³	UNIT WEIGHT OF DRY SOIL	I_p	1.0	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m ³	DENSITY OF SATURATED SOIL	I_L	1.0	LIQUIDITY INDEX = $\frac{w - w_p}{w_L - w_p}$	i	1.0	HYDRAULIC GRADIENT
γ_{sat}	kN/m ³	UNIT WEIGHT OF SATURATED SOIL	I_C	1.0	CONSISTENCY INDEX = $\frac{w_L - w}{w_L - w_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m ³	DENSITY OF SUBMERGED SOIL	e_{max}	1.0	VOID RATIO IN LOOSEST STATE	j	kg/m ²	SEEPAGE FORCE
γ'	kN/m ³	UNIT WEIGHT OF SUBMERGED SOIL						



Foundation Design

RECORD OF BOREHOLE No 1 1 OF 1 METRIC

W.P. 143-80-01 LOCATION Site 9-973.5 m/s 6.5m H/L from E of Hedder Ave
DIST 19 HWY 11 & 12 BOREHOLE TYPE HS Auger, Core Test
DATUM Oseville DATE 91.08.14-15
ORIGINATED BY: JH
COMPILED BY: JH
CHECKED BY: JH

ELEV. DEPTH	SOIL PROFILE DESCRIPTION	STRAT. PLOT	SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DRAWING CONE PENETRATION RESISTANCE PLOT		WATER CONTENT (%)	UNIT WEIGHT	REMARKS
			NUMBER	TYPE			20 40 60 80 100	20 40 60 80 100			
281.8	Ground Surface										
0.0	Silty Sand, trace organic matter (fossil)		1	SS	15						
259.8			2	SS	21						
2.1	Med. Mixture of Sil. Sand and Gravel, trace of clay occasional boulders Very Dense (Classical TB)		3	SS	100						
258.2			4	SS	100						
3.7			5	SS	100						
	Med. Mixture of Clayey Sil. Sand and Gravel occasional boulders Hard (Classical TB)		6	SS	100						
255.0			7	SS	55						
6.9			8	SS	100						
	Sand with Gravel occasional boulders Very Dense		9	SS	100						
252.0			10	SS	100						
9.8			11	SS	100						
251.1	Med. Mixture of Sil. Sand and Gravel occasional boulders Very Dense (Classical TB)		12	SS	100						
10.8	End of Borehole										

*2, *5, Numbers refer to 100% strain at failure

RECORD OF BOREHOLE No 2										1 OF 1		METRIC					
W.P. 143-99-01		LOCATION Site 9+973.5, n/a 5.5m W from E of Highway Ave				ORIGINATED BY M											
DIST 19 HWY 11 & 12		BOREHOLE TYPE HS Auger				COMPILED BY AS											
DATUM Geodetic		DATE 91 08 22				CHECKED BY JCS											
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS		ELEVATION SCALE		DYNAMIC CONE PENETRATION RESISTANCE (MPa)		SHEAR STRENGTH (kPa)		WATER CONTENT (%)		UNIT WEIGHT (kN/m ³)		REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	UNCONFINED	FIELD VANE	QUICK TRAXIAL	LAB VANE	10 20 30	7	GR SA SI CL		
260.5	Ground Surface																
0.0	Silty Sand, trace of organic (fossil)		1	SS	7												
259.1																	
1.4	Net. Mixture of Sil. Sand and Gravel occasional boulders Very Dense (Clacial Till)		2	SS	58												
257.8																	
257.8	Net. Mixture of Clayey Sil. Sand and Gravel occasional boulders Hard (Clacial Till)		3	SS	100												
257.7																	
255.1																	
5.4	Net. Mixture of Sil. Sand and Gravel occasional boulders Very Dense (Clacial Till)		7	SS	100												
252.8																	
7.7	End of Borehole																

+3, +5, Numbers refer to Sensitivity
20 15-25 (4) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 3										1 OF 1		METRIC					
W.P. 143-99-01		LOCATION Site 10+880.0, n/a 5.5m W from E of Highway Ave				ORIGINATED BY M											
DIST 19 HWY 11 & 12		BOREHOLE TYPE HS Auger, SW Coring, BTL, Core, Borehole				COMPILED BY SD											
DATUM Geodetic		DATE 91 08 21-22				CHECKED BY JCK											
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS		ELEVATION SCALE		DYNAMIC CONE PENETRATION RESISTANCE (MPa)		SHEAR STRENGTH (kPa)		WATER CONTENT (%)		UNIT WEIGHT (kN/m ³)		REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	UNCONFINED	FIELD VANE	QUICK TRAXIAL	LAB VANE	10 20 30	7	GR SA SI CL		
261.2	Ground Surface																
0.0	Sand, some silt and gravel Compact (FR)		1	SS	23												
260.0																	
1.2	Net. Mixture of Sil. Sand and Gravel occasional boulders Very Dense (Clacial Till)		2	SS	100												
258.9																	
258.9	Net. Mixture of Clayey Sil. Sand and Gravel occasional boulders Hard (Clacial Till)		3	SS	100												
257.7																	
257.7	Net. Mixture of Sil. Sand and Gravel occasional boulders Very Dense (Clacial Till)		4	SS	100												
254.2																	
7.0	Sand with Gravel occasional boulders Very Dense		8	SS	100												
252.0																	
252.0	End of Borehole																

91 08 25
* GROUND WATER CONDITIONS
PIEZ. NO. GROUND WATER ELEVATION (Metres)
1 259.7

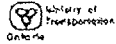
+3, +5, Numbers refer to Sensitivity
20 15-25 (4) STRAIN AT FAILURE
10

W.P. 143-80-91 LOCATION Sig 19-00000; a/s 0.5m 11 from S of Hooper Ave ORIGINATED BY WJ
DIST 19 HWY 11 & 12 BOREHOLE TYPE HS Auger Bx, Core Barrel, Cone Test COMPILED BY AP
DATUM Capeable DATE 21.08.72 CHECKED BY JCK

4. Numbers refer to
Serials

W.P. 143-90-01 LOCATION S12 19+62.5 on S. 50th St. from S. 1st Avenue Ave ORIGINATED BY W
DIST 15 HWY 11 & 12 BOREHOLE TYPE MS Asst. BM Depth BM Date Desc Date Test COMPILED BY AS
DATUM GDA68 DATE 8' 00 12. 5' ON 19 CHECKED BY JCR

23, 24: Numbers refer to
Sensitivity



Foundation Design

RECORD OF BOREHOLE No 6												1 OF 1		METRIC								
W.P. 143-90-01		LOCATION Sig 13+028.2, s/x 6.5m Lt. from E. of Hadder Ave				ORIGINATED BY W																
DIST 15		HWY 11 & 17		BOREHOLE TYPE HS Auger Core Test		COMPILED BY AQ																
DATUM Candelle		DATE 91 08 28		CHECKED BY JOK																		
SOIL PROFILE			SAMPLES			ELEVATION SCALE			SHEAR STRENGTH			WATER CONTENT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION				
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	N° VALUES	GROUND WATER CONDITIONS	ELEVATION	DEPTH	UNCONFINED	FIELD VANE	QUICK TRIAXIAL	LAB VANE	WATER CONTENT (%)	SH/LL	CR	SA	SI	CL				
261.5	Ground Surface																					
0.0																						
	Sands, some Silt and Gravel (Fie)		1	SS	24		381												14	72	10	4
	Compact						260															
259.8			2	SS	60																	
	1.0 Met. Mixture of Sil. Sand and Gravel (Stochel TR)																					
259.2	Very Dense																					
	2.3 End of Borehole at probable bedrock																					

3, 5 Numbers refer to
15% (N) STRAIN AT FAILURE
10

ROCK CORE DESCRIPTION
WP 143-90-01

Page 1 of 1

CORE RECOVERY			CORE DESCRIPTION			
BH#	RC#	DEPTH (m)	% CR*	% ROD*	DEPTH (m)	DESCRIPTION
5	4	2.59-3.20	100	33	2.59-7.64	CHERTY IRON FORMATION: chert with siderite (iron carbonate), granules of greenalite (iron silicate), and shale partings, greyish black to light olive grey; fine to medium grained; medium strong; unweathered to slightly weathered; fractures moderately close to extremely close spaced, flat to near vertical, undulating to planar, smooth.
	5	3.20-4.17	90	76		
	6	4.17-4.32	100	83		
	7	4.32-4.90	83	44		
	8	4.90-5.56	100	73		
	9	5.56-6.17	100	46		
	10	6.17-6.48	100	33		
	11	6.48-7.64	93	73		

(NOTE: Depths are approximated where core recovery is less than 100%)

*CR = CORE RECOVERY
*ROD = ROCK QUALITY DESIGNATION

Logged by: DAW, Soils and Aggregates Section

Appendix D

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Footings on Native Soil or Bedrock	Footings on Engineered Fill	Driven Piles	Caissons
<p>Advantages:</p> <ul style="list-style-type: none"> i. Ease of construction. ii. High geotechnical resistance is available on the till deposits and bedrock. iii. Lower cost than deep foundations. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Dewatering may be required, depending on depth of excavation and groundwater level at time of construction. ii. Potential for subgrade disturbance during excavation of bouldery material. <p>RECOMMENDED</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. Generally less costly construction than deep foundation elements. ii. Allows use of perched abutments. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Cost of engineered fill placement. ii. Not justified in view of high bearing resistance of native soil and bedrock at shallow depth. <p>NOT RECOMMENDED</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. Piles will develop high geotechnical resistance in dense soils and bedrock. ii. Installation of piles could continue in freezing weather. iii. Allows integral abutment design. iv. Foundation construction may require less volume of excavation than footings. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit costs than footings. ii. Possibility that cobbles and boulders may be encountered in till. iii. Need for pre-augering in till and coring or trenching in bedrock to achieve adequate pile length. <p>FEASIBLE</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. High resistance is available for caissons founded in dense till. ii. Construction of caissons could continue in freezing weather. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher cost than spread footings ii. Possibility of cobbles and boulders being encountered during augering. iii. Requires coring of bedrock at the south abutment. iv. More likely to encounter groundwater. Temporary liners may be required to install caissons in cohesionless till under the water table. v. Potential difficulty in cleaning and inspecting bases. <p>NOT RECOMMENDED</p>

Appendix E

Special Provisions

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

- SP 105S10
- SP 110F13
- SP 572S01
- SP 902S01
- SP 903S01
- OPSS 206
- OPSS 902
- OPSS 1010
- OPSD 3000.100
- OPSD 3101.150
- OPSD 3101.200
- OPSD 3102.100

2. Suggested Text for NSSP on “Foundation Excavation”

Cobbles, boulders and slabs of rock should be expected within the glacial till soils on site. Accordingly, equipment suitable for handling and removal of cobbles, boulders and rock slabs should be provided for excavation of the till. Arduous excavation should be expected in the very dense/hard till materials.

Provision of a level foundation base in the till may be difficult if large boulders are encountered at the excavation base level. Excavation should be carried out in a manner that minimizes disturbance to the subgrade.

The bedrock surface elevation may vary between and beyond the borehole locations. Accordingly, establishment of a level founding surface may require removal of elevated portions of the bedrock in addition to placement of concrete to backfill any subexcavation below the design founding level.

3. Suggested Text for NSSP on “Pile Installation”

The glacial till at this site is very dense/hard and pre-augering will be required to install steel piles. Augering in the till might be laboured. The presence of cobbles and boulders in the till may present obstructions to pile driving or augering equipment, and rock coring procedures may be required to penetrate large obstructions.

Bedrock is present at relatively shallow depth at the south abutment. Rock coring and/or rock trenching equipment will be required to advance the pile sockets to the required depth in bedrock. Coring/trenching of bedrock may also be required at the north abutment.

Appendix F

Site Photographs

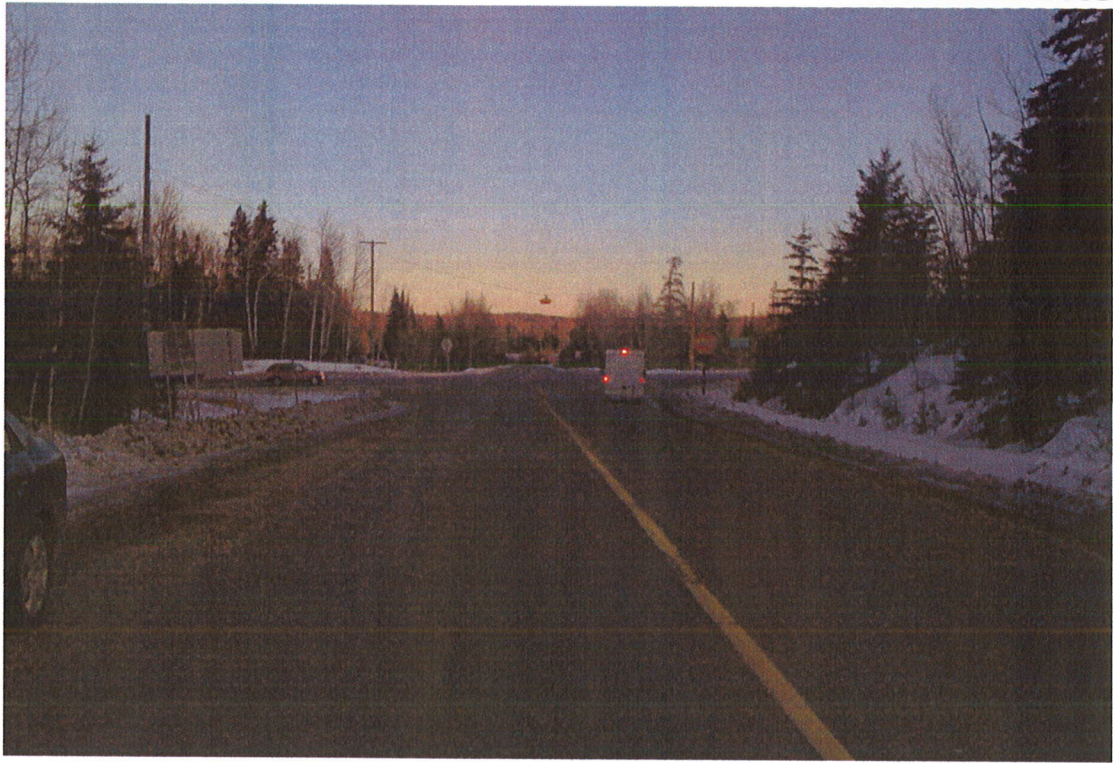


Photo 1 – Looking North on Hodder Avenue towards Hwy 11/17 intersection

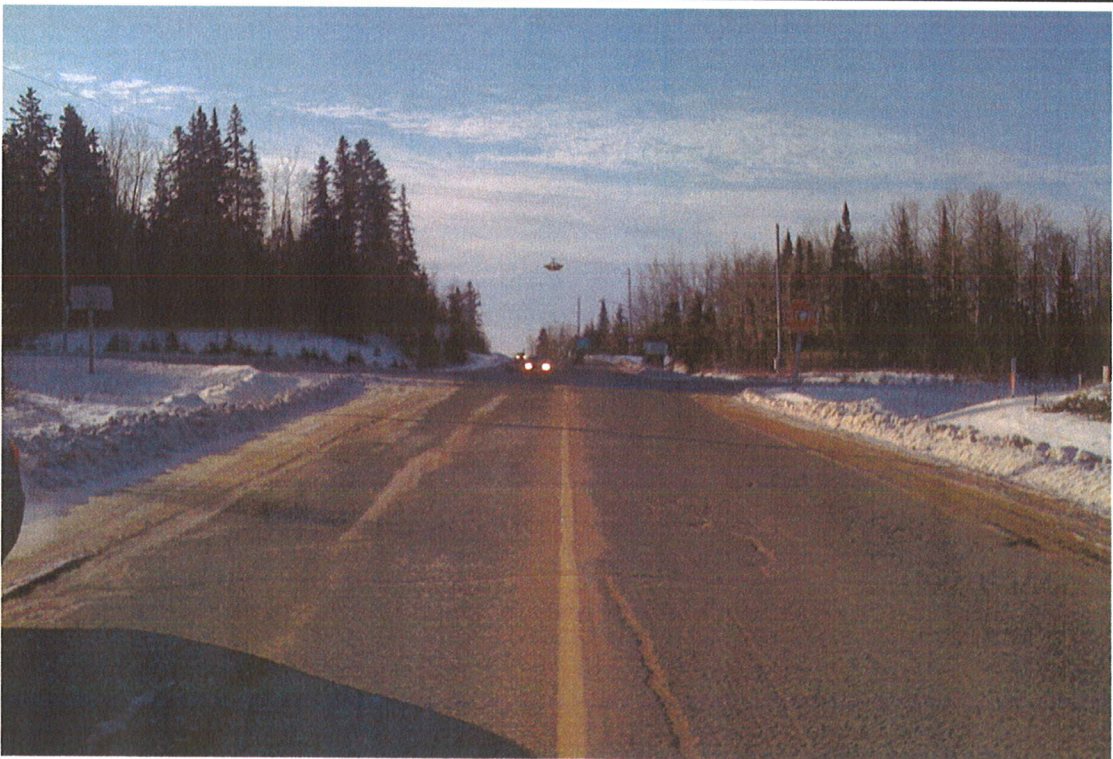


Photo 2 – Looking South on Copenhagen Road towards Hwy 11/17 intersection

Appendix G

Drawings

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

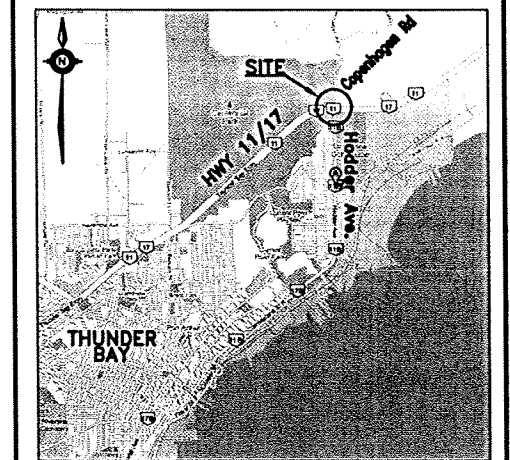
CONT No 2010-6001
GWP No 334-94-00

HIGHWAY 11/17
COPENHAGEN ROAD/
HODDER AVENUE BRIDGE
BOREHOLE LOCATIONS & SOIL STRATA

SHEET
272

MRC McCORMICK RANKIN
CORPORATION

THURBER ENGINEERING LTD.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS



KEYPLAN

LEGEND

- ◆ Borehole (Current Investigation)
- ◆ Borehole (Previous Investigation, 1991)
- N Blows /0.3m (Std Pen Test, 475J/blow)
- CONE Blows /0.3m (60° Cone, 475J/blow)
- PH Pressure, Hydraulic
- W Water Level
- W Head Artesian Water
- P Piezometer
- 90% Rock Quality Designation (RQD)
- A/R Auger Refusal

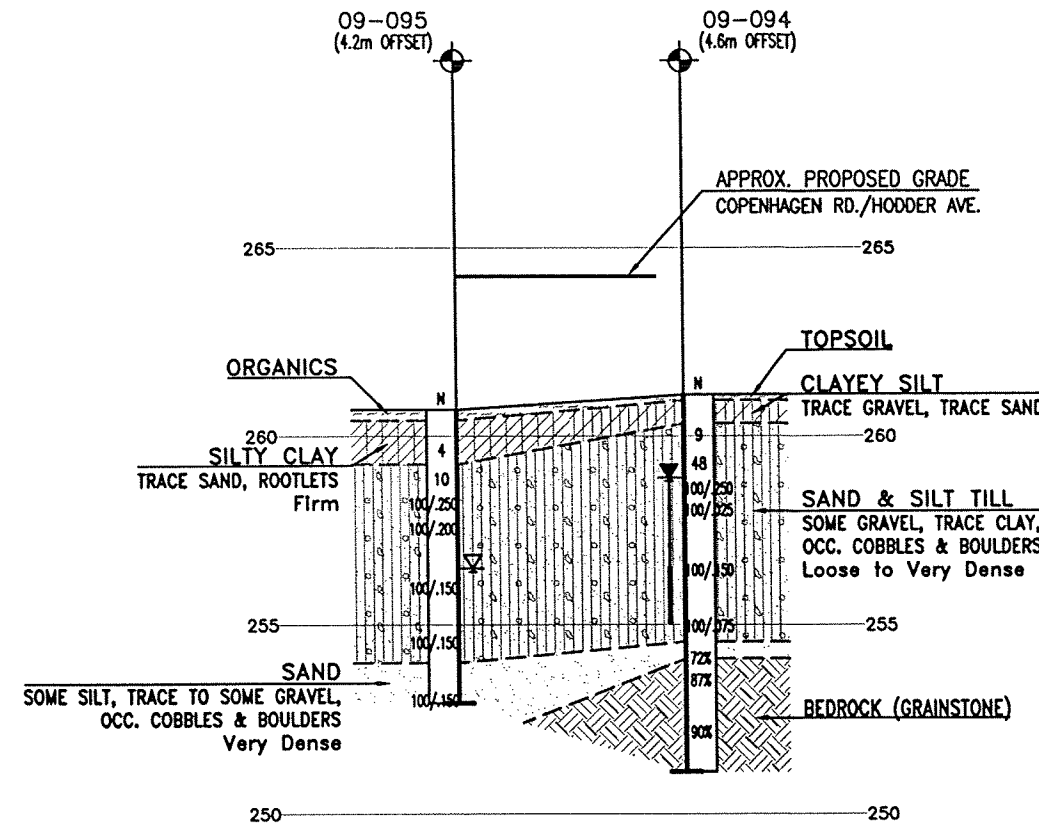
NO	ELEVATION	NORTHING	EASTING
01	261.9	5 372 010.9	365 255.9
02	260.5	5 372 010.9	365 268.9
03	261.2	5 371 984.4	365 255.9
04	261.8	5 371 984.4	365 269.1
05	260.5	5 371 957.9	365 256.1
06	261.5	5 371 958.0	365 269.1
09-047	260.4	5 372 030.9	365 262.0
09-091	261.4	5 371 937.9	365 262.7
09-092A	261.6	5 371 947.2	365 269.1
09-092B	260.2	5 371 947.2	365 256.1
09-093	261.4	5 371 953.4	365 262.6
09-094	261.1	5 372 019.4	365 254.2
09-095	260.7	5 372 019.2	365 269.2

NOTES

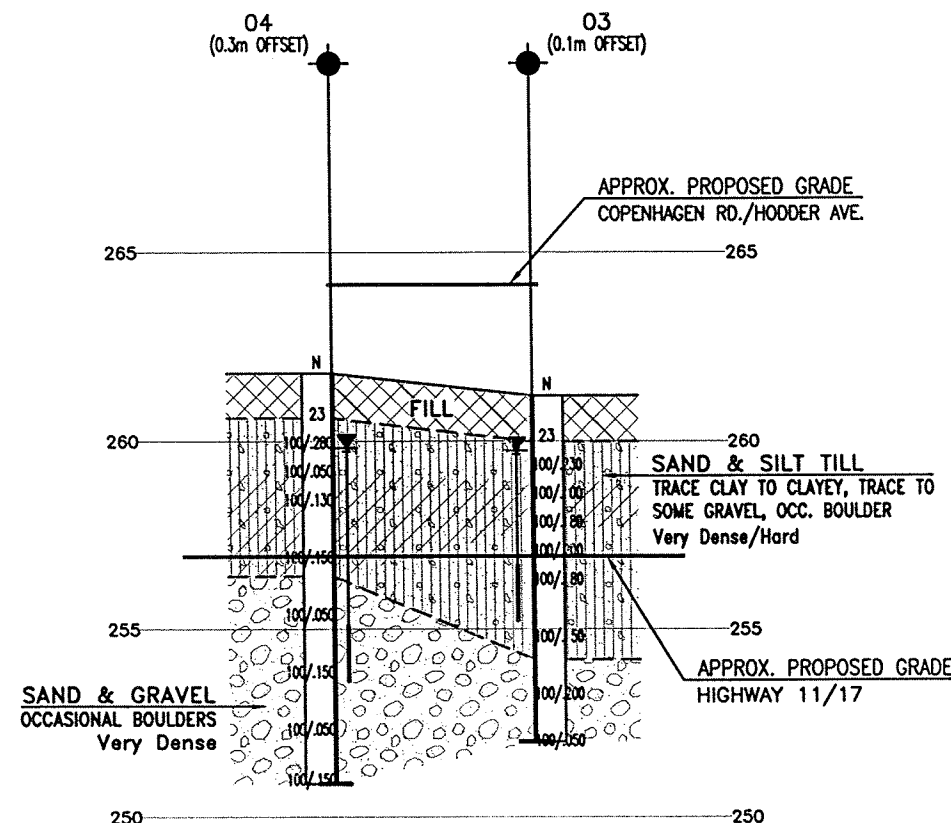
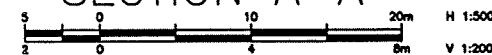
- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

GEORES No. 52A-143

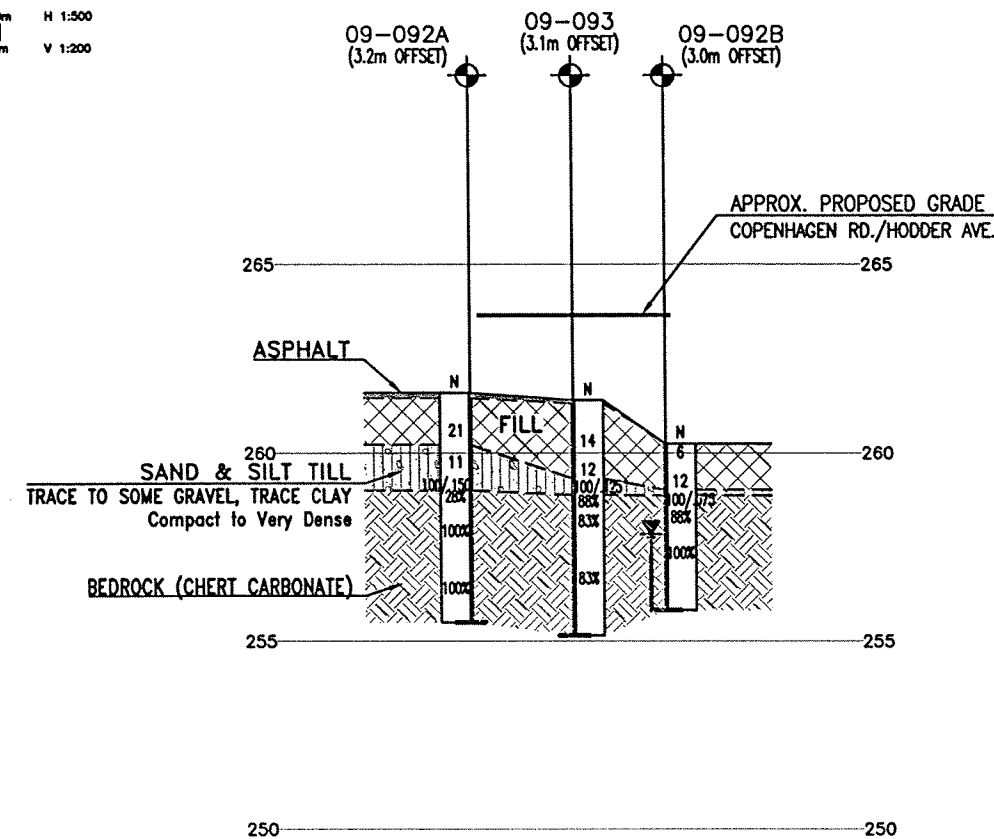
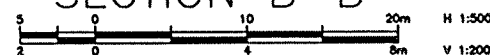
DATE	BY	DESCRIPTION
DESIGN	MRA	CHK AEG
DRAWN	MFA	CHK PKC
		SITE
		STRUCT
		DWG 2



SECTION A-A



SECTION B-B



SECTION C-C

