

G.I.-30 SEPT. 1976

GEOCRES No. 31D-359DIST. 52 REGION _____W.P. No. 429-94-02

CONT. No. _____

W. O. No. _____

STR. SITE No. 42-142HWY. No. 11LOCATION Doe Lake Road IC
BridgeNo of PAGES -

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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. _____REMARKS: _____

ENGINEERING MATERIALS OFFICE
FOUNDATION DESIGN SECTION

WP 429-94-02 DIST 52
HWY 11 STR SITE 42-142

Doe Lake Road Interchange Bridge

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GEOGRES 31D-359

DATE AUG 14 1996

FOUNDATION INVESTIGATION REPORT

FOR:

Doe Lake Road Interchange Bridge

WP 429-94-02, Site No. 42-142

Highway 11, District 52, Huntsville

Introduction

This report summarizes the results of a foundation investigation conducted at the aforementioned site. It is proposed to construct an interchange for Muskoka Roads 41 and 6 (Doe Lake Road) at Highway 11. It is understood that a two span structure will be constructed with the central pier located in the median of the existing highway. This report contains factual information obtained from this investigation pertaining to structural foundation and related earthworks.

Site Description

The site is located on Hwy. 11, just south of the north entrance to Gravenhurst, in the Township of Gravenhurst, District of Muskoka.

The topography of the area is typical for the muskoka region, consisting of bush/tree covered and swamp regions with frequent outcrops of bedrock. At this location the private properties on both sides of the highway are fenced off and are heavily vegetated with small trees. The existing highway embankment at this location is about 1 - 2 m high. The NBL and SBL are separated by a ditch area in the median.

Physiographically, the site lies in the area known as the Algonquin Highlands. This region is characterized by its underlying granite and other precambrian rock which can be seen by frequent outcrops. The soils are generally shallow sandy-glacial tills, but thicknesses over the bedrock varies greatly over short distances. The surface of the till is smoothed and moulded with occasional drumlins appearing. Frequent swamps and bogs in hollows detract from this areas usefulness for farming.

Investigation Procedures

Soil data and inherent properties were obtained by in situ and laboratory testing. The procedures employed are discussed below.

Field Investigation

The field work for this investigation was carried out between 95 08 02 and 95 08 04 and consisted of nine samples boreholes which were advanced to a maximum depth of 5.3 m below the ground surface. Two boreholes were located at each pier and abutment

locations with an additional borehole placed at each approach. Dynamic Cone Penetration Tests were advanced at each borehole location to a maximum depth of 5.3 m. Bedrock cores were advanced at four borehole locations with a maximum thickness of 3.2 metres.

In general, the subsoil samples in the overburden were retrieved at 0.75 m intervals using a split spoon sampler in accordance with standard practice (ASTM D 1587). Rock core testing was conducted to confirm bedrock depth and quality with samples of 3 m lengths.

All subsoil samples were identified in the field and returned to the laboratory for further examination and applicable testing. Water levels were monitored throughout the duration of the investigation in open boreholes. All boreholes were backfilled upon completion at the field work. Survey information relate to the location and elevation of boreholes was provided by the Northern Region, Surveys and Plans Section.

Laboratory Analysis

The following laboratory tests were carried out on select soil samples.

1. Grain Size Distribution
2. Unit Weights
3. Natural Moisture Contents

Laboratory test results are given in the following section of this report and are illustrated on Figures and Borehole Log Sheets included in the Appendix.

Subsurface Conditions

The subsoil stratigraphy consists of a surficial layer of Silty Sand with Organics (Top Soil) having a depth of 0.3 m to 1.7 m underlain by a Sand and Silt containing occasional layers of boulders, trace Clay and trace Gravel. This layer had a thickness of 0.5 to 4.2 metres and was not encountered in the boreholes which had a very shallow overburden. Underlying the above was the bedrock which was found at depths ranging from 0.3 m to 5.3 m. The overburden became very thin primarily at the pier location within the median of Hwy. 11 and than became deeper further east and west at each abutment and approach borehole location.

The plan and location of borings and the stratigraphical profile are shown on Drg. No. 4299402-A in the attached appendix. The field and laboratory test results are plotted on the Record of Borehole sheets also included in the appendix of this report. A brief description of the different soil types is given below.

Silty Sand with Organics (Topsoil)

The surficial material consisted of 0.3 to 1.7 m of a Silty Sand with organics and some

occasional root hairs.

The results of a grain sized distribution test carried out on a sample are shown in Figure 1 in the appendix. The material contained 0 % Gravel, 53 % Sand, 45 % Silt and 2 % Clay.

The Standard Penetration resistance 'N' values ranged from 3 - 14 Blows/0.3 m indicating a Compact state of denseness.

Sand and Silt

Underlying the above topsoil was a second non-cohesive layer of Sand and Silt. This layer contained pockets of trace clay, and trace gravel, with boulders encountered upon approaching the bedrock surface.

Results of grain size distribution tests carried out on select samples are shown on Figure 2 in the appendix in an envelope form. The results indicate a large percentage of silt and sand. The deposit is comprised of 0 % gravel, 30 - 59 % sand, 32 - 66 % silt and 2 - 9 % clay.

The standard penetration resistance 'N' values ranged from 4 - 27 Blows/0.3 m indicating a loose to compact state of denseness. Cone tests penetrated this layer fully down to either the bedrock surface or to the overlying boulders.

Bedrock

The bedrock was proven by obtaining BXL cores in four boreholes. The bedrock is fairly flat ranging in elevation from 252.1 m to 257.0 at the bridge site location.

Detailed descriptions of the rock are attached in the appendix. Classification by an MTO Petrographer described then as bedrock inter-layered with Biotite-Hornblende Gneiss, Amphibolite, Marble and Pegmatite of the Grenville province. The rock is strong to medium strong, unweathered to slightly weathered with fractures very closely spaced to wide. An unconfined compression test was carried out on a rock core sample in the Marble layer and the unconfined compressive strength obtained was 36.1 MPa. The Marble is therefore classified as medium strong while the rest of the rock is considered strong. Rock core recovery and Rock Quality Designation values ranged from 60 - 100 % and 13 - 100 % respectively, with the average of samples recovered having a relatively high RQD and RC.

Groundwater level was measured in the open boreholes during the investigation. It is generally between El. 256.5 \pm m to El. 257 \pm m. The unstabilized water level measured at BH 9 was El. 255.8. Seasonal fluctuations in ground water level are expected.

Discussions and Recommendations

It is proposed to construct a 20 m wide two span underpass to carry Doe Lake Road over the existing Hwy. 11. The proposed construction would involve approach embankments of approximately 7.3 metres above the existing grade.

Recommendations from a foundation standpoint are given in the following sections regarding the design and construction of foundations and associated earth works.

FOUNDATION

According to the investigation results, competent subsoil or bedrock exists at relatively shallow depths. The foundation for the structure may therefore be founded on conventional spread footings to achieve a cost effective design.

West Abutment

At this location, the sand and silt layer is overlain by a 1.5 ±m thick topsoil stratum. It is recommended to remove the topsoil to El. 257 ±m and backfill the excavation with granular material to form a pad for placement of footings, as illustrated in Figure 3. Footings should be perched as high as possible within the fill on a minimum 3 m thick granular pad. The granular pad should be constructed to a minimum 1 m edge distance from the top of the footing to the crest of the pad and with 1H:1V slopes. The granular 'A' material must be placed and compacted to achieve 100% of the Proctor maximum density as outlined in OPSS 501-08-02 (Method A). For the purpose of the O.H.B.D.C., the following bearing capacities can be used in the design:

Factored Capacity at U.L.S. = 900 kPa

Bearing Capacity at S.L.S. = 350 kPa

A 1.8 m earth cover should be provided for frost protection purpose.

Pier

At this location, bedrock is encountered at shallow depth. It is therefore recommended to place the footings directly on bedrock. For the purpose of O.H.B.D.C., the following bearing capacities can be used.

Factored Capacity at U.L.S. = 3500 kPa

Bearing Capacity at S.L.S. does not govern for 'unyielding soils'

A slightly lower bearing capacity is assigned to the bedrock due to the weaker marble layer present in the rock. The highest founding elevations for the footing, as estimated from the

investigation results are as follows:

South End	El. 256.5 m
Centre	El. 257.0 m
North End	El. 256.8 m

The above values are for preliminary estimating purposes. Actual founding elevations have to be verified during construction. The rock surface at the footing base should be inspected and all loosened or highly fractured rock should be removed prior to placement of concrete. For footings founded on bedrock, frost cover is not required.

East Abutment

At this location, bedrock dips from El. 255 m to El. 252 m in a northerly direction. Placing the footings on bedrock is impractical. It is therefore recommended to adopt the same design as the West Abutment and place the footing on a compacted Granular 'A' pad. The plan area of the granular pad should be cleared of topsoil. This would involve subexcavation to El. 256.5 \pm m and backfilling with granular material. As recommended above for the West Abutment, the following bearing capacities can be used:

Factored Capacity at U.L.S. = 900 kPa
Bearing Capacity at S.L.S. = 350 kPa

A 1.8 m earth cover should be provided for frost protection purpose.

Sliding Resistance -

The computation of the sliding resistance of the foundation shall be carried out in accordance with Section 6-8.4.2 of the O.H.B.D.C. $\phi = 35^\circ$ can be used for sliding within the granular fill and $\phi = 30^\circ$ can be used for sliding at footing/granular fill interface. The same values may be used for footings on bedrock provided that the rock surface is relatively rough.

BACKFILL

Backfill to abutments or retaining walls should consist of granular materials such as Granular 'A' and 'B', or rock fill. Computation of earth pressure shall be in accordance with Section 6.7.4 of the O.H.B.D.C. Unfactored properties for backfill materials are provided in the following table:

<u>Material</u>	ϕ	γ
Granular 'A'	35°	22.8 kN/m ³
Granular 'B'	30°	21.2 kN/m ³
Rock Fill	35°	19.0 kN/m ³

ABUTMENT SLOPES

The maximum fill height is about 8 m. It is recommended that the fill slopes be formed at a gradient of 2H:1V or flatter, up to a maximum height of 10 m. Topsoil and any soft or unsuitable material should be removed prior to filling. Normal slope protection such as vegetation cover shall be established as soon as possible after completion of the slope formation in order to control surficial erosion.

CONSTRUCTION CONSIDERATIONS

Temporary excavations of up to 2 ±m is required for footing construction. Cut slopes can be formed at a gradient of 1H:1V. Some dewatering may be required. Unwatering can be carried out by conventional sump pumping techniques.

Any loose or organic materials should be removed and replaced with suitable granular fill.

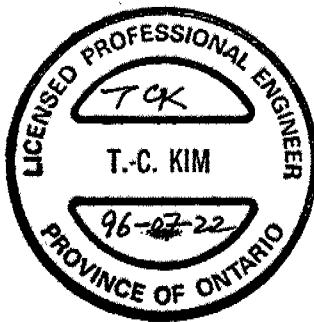
MISCELLANEOUS

The field work for this project was supervised by D. Kwok, Project Foundation Engineer. The equipment used was owned and operated by Landtech Drilling Services. This report was prepared by D. Kwok and M. Michalek, Jr. Foundation Engineer and reviewed by T. C. Kim, Sr. Foundation Engineer.



A handwritten signature in black ink, appearing to read "M. Michalek, Jr.", written in a cursive style.

M. Michalek, P. Eng.
Jr. Foundation Engineer

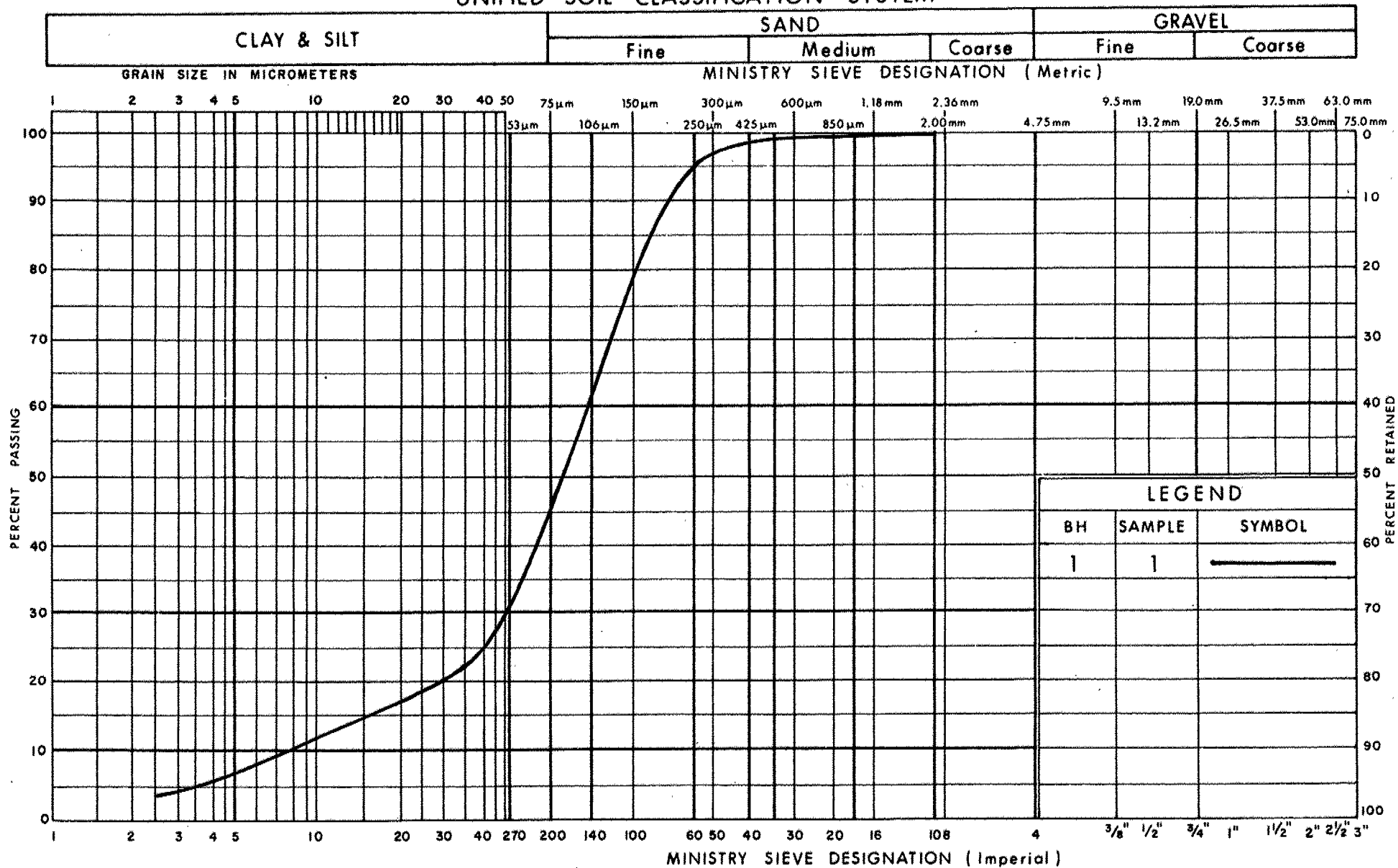


A handwritten signature in black ink, appearing to read "Taechul Kim", written in a cursive style.

T. C. Kim, P. Eng.
Sr. Foundation Engineer

APPENDIX

UNIFIED SOIL CLASSIFICATION SYSTEM



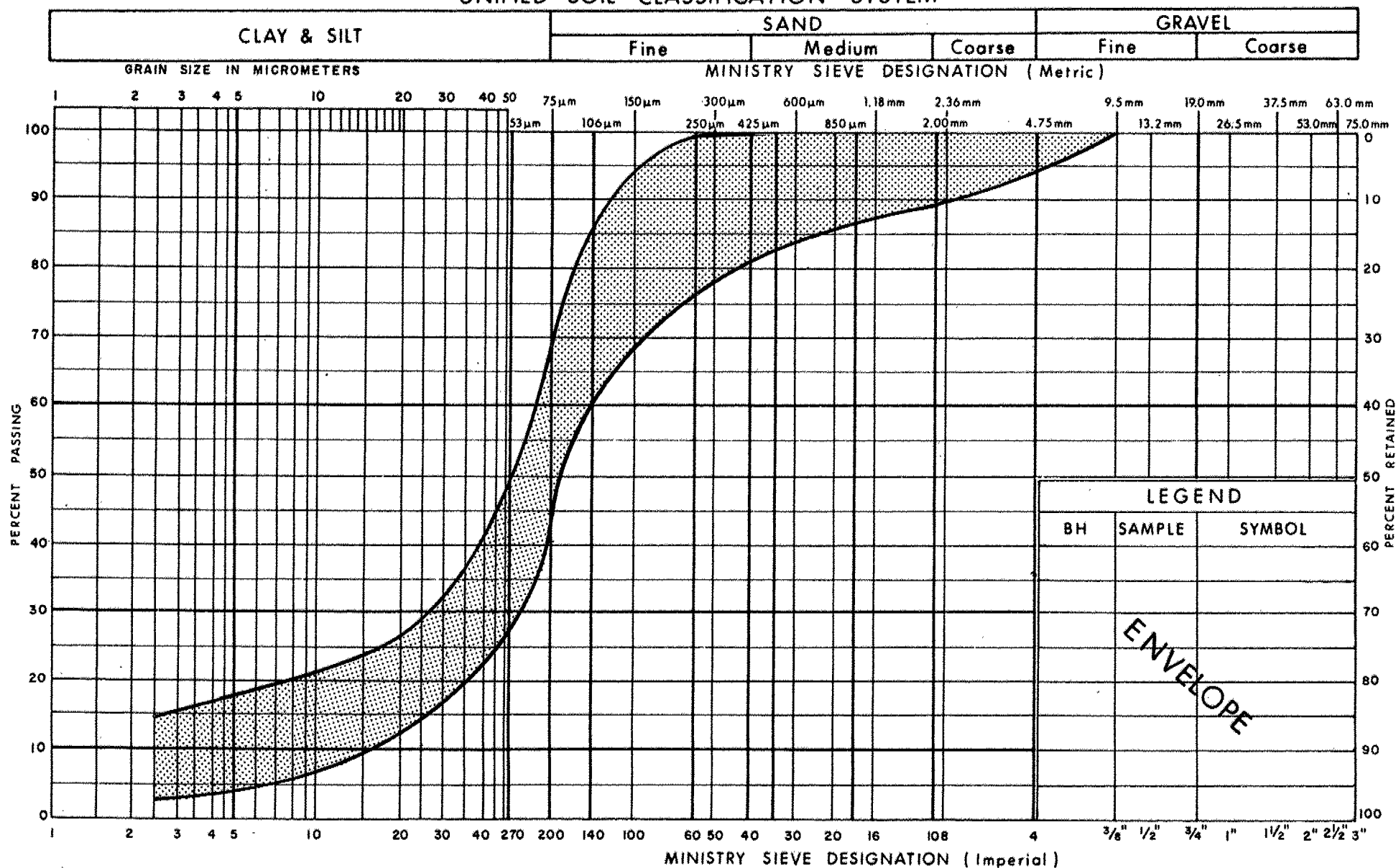
Ministry of
Transportation

GRAIN SIZE DISTRIBUTION
SILTY SAND WITH ORGANICS

FIG No 1

W P 429-94-02

UNIFIED SOIL CLASSIFICATION SYSTEM

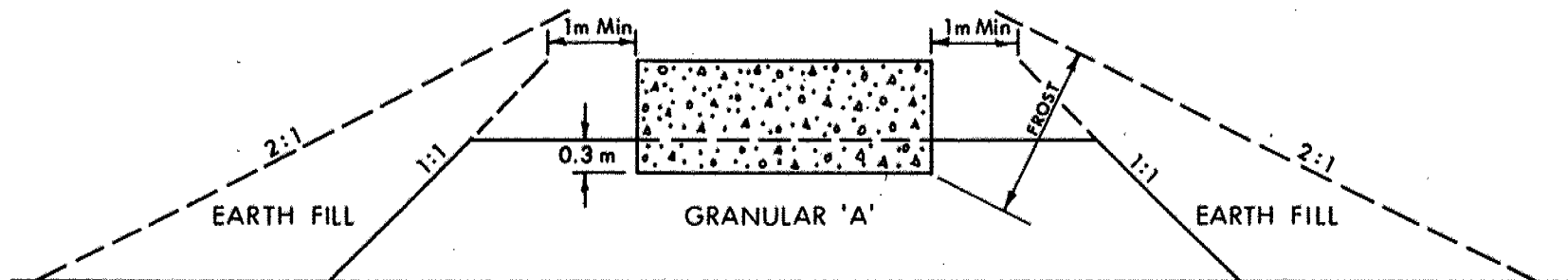


Ministry of
Transportation

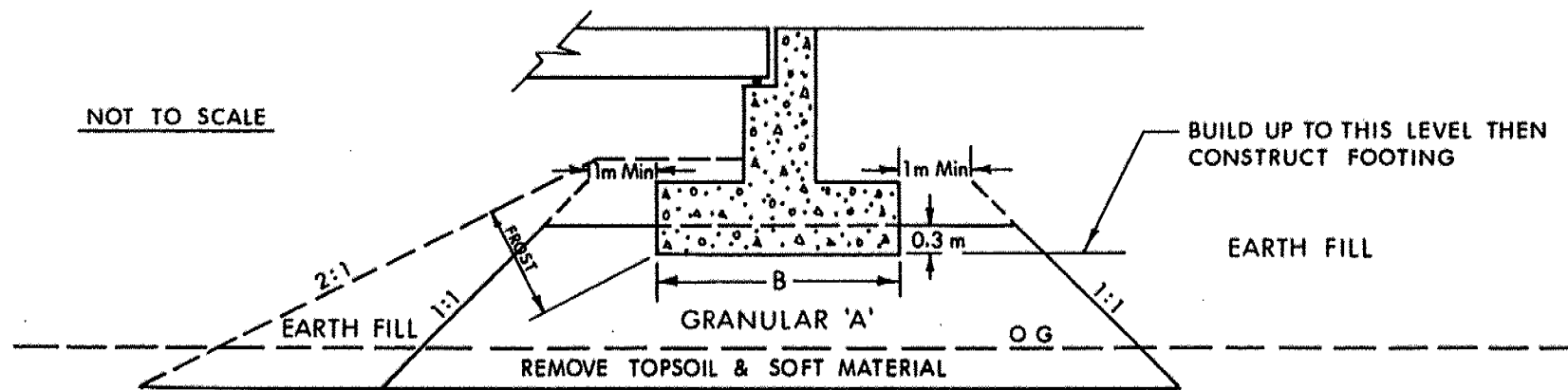
GRAIN SIZE DISTRIBUTION SAND & SILT

FIG No 2

W P 429-94-02



X SECTION



LONGITUDINAL SECTION

NOTES:

- 1 - REMOVE TOPSOIL & /OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' & EARTH FILL.
- 2 - PLACE GRANULAR 'A' & EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT M T O STANDARDS.
- 3 - CONSTRUCT CONCRETE FOOTING.
- 4 - PLACE REMAINDER OF GRANULAR 'A' & EARTH FILL AS REQUIRED.



Ontario

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Transportation

ABUTMENT ON COMPACTED FILL
SHOWING GRANULAR 'A' CORE

FIG No 3

W P 429 - 94 - 02

RECORD OF BOREHOLE No 1

1 OF 1

METRIC

W.P. 429-94-02 LOCATION Co-ords: N 4 977 429.3; E 317 284.0 ORIGINATED BY DK
DIST 52 HWY 11 BOREHOLE TYPE H.S. Auger, Cone Test COMPILED BY DK
DATUM Geodetic DATE 95 08 02 CHECKED BY TK

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100	W _p	W	W _L		
258.4	Ground Surface																
0.0	Silty Sand with Organics Some Root hairs Brown, Compact (Topsoil)		1	SS	14											0 53 45 2	
256.7			2	SS	23												
1.7	Sand and Silt Brown and Grey, Compact		3	SS	21											0 42 54 4	
255.6																	
2.8	End of Borehole * 95 08 03 ** Auger Refusal on Probable Bedrock																

RECORD OF BOREHOLE No 2

1 OF 1

METRIC

W.P. 429-94-02 LOCATION Co-ords: N 4 977 438.0; E 317 290.5 ORIGINATED BY DK
DIST 52 HWY 11 BOREHOLE TYPE Cone Test COMPILED BY DK
DATUM Geodetic DATE 95 08 02 CHECKED BY TK

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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	SHEAR STRENGTH kPa								
							20	40	60	80	100					
258.5	Ground Surface															
0.0	Probable Silty Sand With Organics (Topsoil)															
257.0																
1.5	Probable Sand and Silt															
255.6																
2.9	End of Borehole Probable Bedrock ** Auger Refusal on Probable Bedrock											120/	20cm			

RECORD OF BOREHOLE No 3

1 OF 1

METRIC

W.P. 429-94-02 LOCATION Co-ords: N 4 977 445.4; E 317 297.3 ORIGINATED BY DK
 DIST 52 HWY 11 BOREHOLE TYPE H.S. Auger, NQ Core Barrel, Cone Test COMPILED BY DK
 DATUM Geodetic DATE 95 08 02 CHECKED BY TK

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			20 40 60 80 100	PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L		
258.9	Ground Surface							SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE	WATER CONTENT (%) 10 20 30				
0.0	Silty Sand with Organics Trace of Roothair Brown to Dark Brown, Loose (Topsoil)		1	SS	9		258						
257.4	Trace Clay Occasional Roothair		2	SS	10		257						0 30 60 10
1.5	Sand and Silt Grey, Compact		3	SS	17		256						
			4	SS	11		255						0 30 66 4
254.9	Boulders		5	RC	REC 100%		254						RQD 100%
4.0	Bedrock Biotite-Hornblende Gneiss with interlayered Pegmatite		6	RC	REC 100%		253						RQD 82%
251.9			7	RC	REC 100%		252						RQD 85%
7.0	End of Borehole • 95 08 03												

RECORD OF BOREHOLE No 4

1 OF 1

METRIC

W.P. 429-94-02 LOCATION Co-ords: N 4 977 421.0; E 317 327.0 ORIGINATED BY DK
 DIST 52 HWY 11 BOREHOLE TYPE H.S. Auger, NQ Core Barrel, Cone Test COMPILED BY DK
 DATUM Geodetic DATE 95 08 03 CHECKED BY TK

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 x LAB VANE 10 20 30 40 50	PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES								
257.2	Ground Surface												
0.0	Silty Sand with Organics Brown, Very Loose (Topsoil)		1	SS	3		257						
256.5													
0.7	Bedrock		2	RC	REC 90%		256						RQD 19%
	Amphibolite		3	RC	REC 100%		255						RQD 79%
			4	RC	REC 100%								RQD 100%
	Marble		5	RC	REC 100%		254						RQD 97%
253.4													
3.8	End of Borehole												
	* 95 08 04												

1 OF 1

METRIC

DATUM Geodetic DATE 95 08 03 CHECKED BY TK

+3, x5: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 6

1 OF 1

METRIC

W.P. 429-94-02 LOCATION Co-ords: N 4 977 437.6; E 317 337.7 ORIGINATED BY DK
 DIST 52 HWY 11 BOREHOLE TYPE H.S. Auger, NO Core Barrel, Cone Test COMPILED BY DK
 DATUM Geodetic DATE 95 08 03 CHECKED BY TK

SOIL PROFILE		STRAT PLOT	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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION		NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
257.6	Ground Surface																
0.0	Silty Sand with Organics (Topsoil) Loose		1	SS	8		257										
0.3	Sand and Silt Brown, Loose																
256.8																	
0.8	Bedrock		2	RC	REC 71%												RQD 25%
			3	RC	REC 62%		256										RQD 13%
	Biotite-Hornblende Gneiss with interlayered Pegmatite		4	RC	REC 100%		255										RQD 100%
			5	RC	REC 100%		254										RQD 93%
253.6	Marble with interlayered Amphibolite																
4.0	End of Borehole																
	* 95 08 04																

RECORD OF BOREHOLE No 7

1 OF 1

METRIC

W.P. 429-94-02 LOCATION Co-ords: N 4 977 413.6; E 317 367.6 ORIGINATED BY DK
 DIST 52 HWY 11 BOREHOLE TYPE H.S. Auger, NQ Core Barrel, Cone Test COMPILED BY DK
 DATUM Geodetic DATE 95 08 03 - 95 08 04 CHECKED BY TK

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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
257.2	Ground Surface													
0.0	Silty Sand with Organics Dark Brown, Compact (Topsoil)													
256.2			1	SS	19									
1.0	Sand and Silt Brown and Grey Compact		2	SS	19									
255.1														
2.1	Boulders													
	Bedrock Biotite-Hornblende with interlayered Amphibolite		3	RC	REC 100%									
			4	RC	REC 100%									
			5	RC	REC 100%									
251.9														
5.3	End of Borehole • 95 08 04													

RECORD OF BOREHOLE No 8

1 OF 1

METRIC

W.P. 429-94-02 LOCATION Co-ords: N 4 977 421.4; E 317 372.4 ORIGINATED BY DK
 DIST 52 HWY 11 BOREHOLE TYPE Cone Test COMPILED BY DK
 DATUM Geodetic DATE 95 08 04 CHECKED BY TK

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES								
257.4	Ground Surface												
0.0	Probable Silty Sand with Organics (Topsoil)						257						
256.8							256						
0.6	Probable Sand and Silt						255						
							254						
253.2													
4.2	End of Borehole Probable Bedrock							120	28cm				

RECORD OF BOREHOLE No 9

1 OF 1 METRIC

W.P. 429-94-02 LOCATION Co-ords: N 4 977 428.4; E 317 383.0 ORIGINATED BY DK
 DIST 52 HWY 11 BOREHOLE TYPE H.S. Auger, Cone Test COMPILED BY DK
 DATUM Geodetic DATE 95 08 04 CHECKED BY TK

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								
							20	40	60	80	100					
257.4	Ground Surface															
0.0	Silty Sand with Organics Scattered Rootlets Brown, Compact (Topsoil)															
256.5			1	SS	27											
0.9			2	SS	14											
			3	SS	14											
			4	SS	4											
			5	SS	9											
			6	SS	50											
252.1																
5.3	End of Borehole															
	* Unstabilized water level measured upon completion ** Auger refusal on probable bedrock															

ROCK CORE DESCRIPTION

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Page 1 of 2

CORE RECOVERY					CORE DESCRIPTION	
BH#	RC#	DEPTH (m)	% CR*	% RQD*	DEPTH (m)	DESCRIPTION
3	5	3.99-4.29	100	100	3.99-7.03	BIOTITE-HORNBLENDE GNEISS with interlayered very light grey to medium grey PEGMATITE (23%), greyish black to medium grey; medium to coarse grained; strong; unweathered to slightly weathered; fractures moderate to very close spaced, flat to near vertical, undulating to planar, rough to smooth.
	6	4.29-5.82	100	82		
	7	5.82-7.03	100	85		
4	2	0.74-1.27	90	19	0.74-1.27	AMPHIBOLITE (garnet- and calcite-bearing), greyish black; medium grained; strong; unweathered to slightly weathered; fractures close to very close spaced, flat to dipping, undulating, rough to smooth.
	3	1.27-2.34	100	79		
	4	2.34-2.87	100	100	1.27-3.76	MARBLE (amphibole-, serpentine-, epidote-, and garnet-bearing), greyish pink (dark greenish grey to greenish grey to greyish pink, 1.27-2.51m); medium to coarse grained; medium strong; unweathered to slightly weathered; fractures wide to very close spaced, flat to near vertical, undulating, rough to smooth.
	5	2.87-3.76	100	97		

*CR = CORE RECOVERY

*RQD = ROCK QUALITY DESIGNATION

Note: Depths are approximated where core recovery is less than 100%
 Logged by: DAW, Soils and Aggregates Section

ROCK CORE DESCRIPTION

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Page 2 of 2

CORE RECOVERY					CORE DESCRIPTION	
BH#	RC#	DEPTH (m)	% CR*	% RQD*	DEPTH (m)	DESCRIPTION
6	2	0.79-1.40	71	25	0.79-2.62	BIOTITE-HORNBLLENDE GNEISS with interlayered moderate orange pink PEGMATITE (15%), greyish black to medium light grey; medium to coarse grained; strong; unweathered to slightly weathered; fractures moderate to extremely close spaced, flat to near vertical, undulating to planar, smooth to rough.
	3	1.40-2.19	60	13		
	4	2.19-2.87	100	100		
	5	2.87-3.99	100	93		
					2.62-3.99	MARBLE (amphibole-, serpentine-, epidote-, garnet-, and pyrite-bearing) with interlayered greyish black AMPHIBOLITE (39%), very light grey to greyish pink to greenish grey to dark greenish grey; medium to coarse grained; medium strong to strong; unweathered to slightly weathered; fractures moderate to very close spaced, flat to dipping, undulating, rough to smooth.
7	3	2.13-2.90	100	97	2.13-5.26	BIOTITE-HORNBLLENDE GNEISS with interlayered greyish black AMPHIBOLITE (13%), greyish black to light grey to moderate orange pink; medium to coarse grained; strong; unweathered to slightly weathered; fractures wide to close spaced, dipping to flat, undulating, rough to smooth.
	4	2.90-4.42	100	100		
	5	4.42-5.26	100	94		

*CR = CORE RECOVERY

*RQD = ROCK QUALITY DESIGNATION

Note: Depths are approximated where core recovery is less than 100%
Logged by: DAW, Soils and Aggregates Section

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 51mm O.D. SPLIT 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 (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

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

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	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), FOR MODIFIED RECOVERY, IS:

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

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	> 3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

SS	SPLIT SPOON	TP	THINWALL PISTON
WS	WASH SAMPLE	OS	OSTERBERG SAMPLE
ST	SLOTTED TUBE SAMPLE	RC	ROCK CORE
BS	BLOCK SAMPLE	PH	T W ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	T W ADVANCED MANUALLY
TW	THINWALL OPEN	FS	FOIL SAMPLE

MECHANICAL PROPERTIES OF SOIL

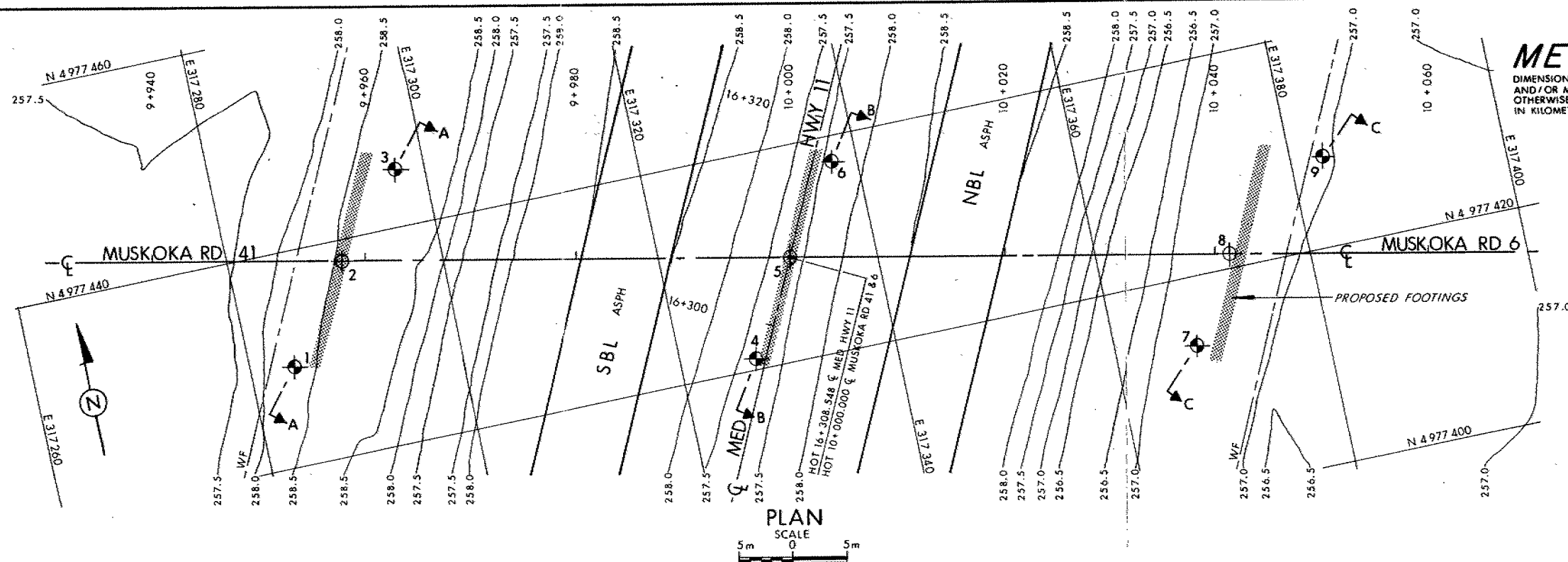
m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	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	1	COEFFICIENT OF FRICTION

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{\min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{\max} - e}{e_{\max} - e_{\min}}$
ρ_w	kg/m^3	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m^3	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	mm	n PERCENT - DIAMETER
ρ	kg/m^3	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kN/m^3	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m^3	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL	e_{\max}	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m^2	SEEPAGE FORCE
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						



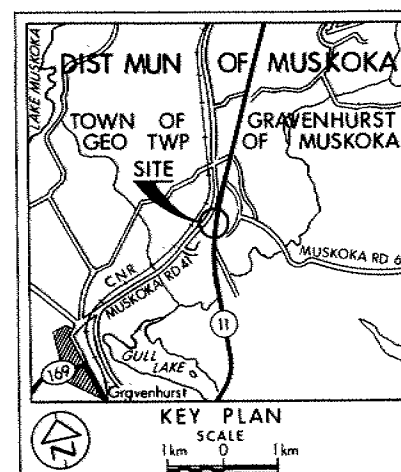
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
IN KILOMETRES + METRES.

CONT No
WP No 429-94-02

MUSKOKA RD 41 & 6
(DOE LAKE RD INTERCHANGE)
BORE HOLE LOCATIONS & SOIL STRATA



SHEET



LEGEND

- Bore Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ⊕ Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- W L at time of investigation 1995 08

No	ELEVATION	CO-ORDINATES NORTH	EAST
1	258.4	4 977 429.3	317 284.0
2	258.5	4 977 438.0	317 290.5
3	258.9	4 977 445.4	317 297.3
4	257.2	4 977 421.0	317 327.0
5	257.3	4 977 429.6	317 332.0
6	257.6	4 977 437.6	317 337.7
7	257.2	4 977 413.6	317 367.6
8	257.4	4 977 421.4	317 372.4
9	257.4	4 977 428.4	317 383.0

NOTE

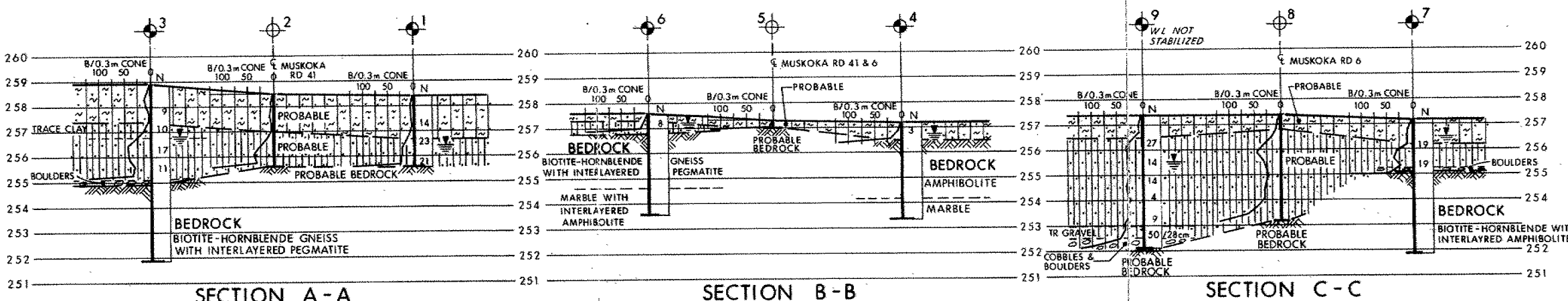
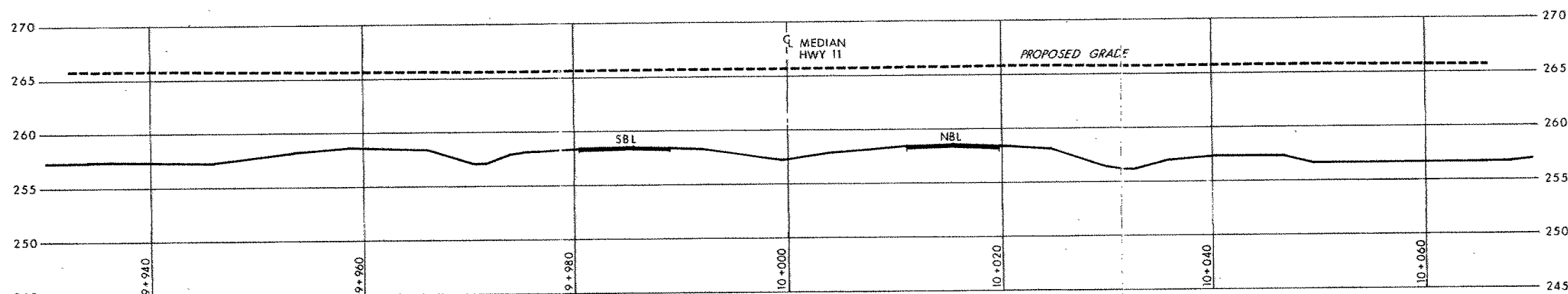
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

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

REV	DATE	BY	DESCRIPTION

Geocres No 31D-359

HWY No 11	CHECKED	DATE 1996 06 14	DIST 52
SUBMD MAM	CHECKED		SITE 42-142
DRAWN RS	CHECKED		DWG 4299402-A



SOIL STRATIGRAPHY LEGEND

SILTY SAND, WITH ORGANICS,
(Topsoil)
Very Loose to Compact

SAND & SILT
Loose to Compact

From: Tae C. Kim
To: MTOHS2.STKITS3.Husain, MTONH.HUNTSVILLE.PepperTo, ...
Subject: Pile seating problem Contract 98-13 Doe Lake Rd/Hwy 11 -Reply

Dear Dave

Further to your request, I have reviewed project files and refreshed my memory. Based on my review, the following comments can be made:

- 1) Our section had clearly indicated our reluctance to use integral abutment design for this bridge and suggested socketing the piles into the bedrock and /or using some kind of rock points, such as Oslo point as a result of possible sliding of pile tips (See our recommended letters dated February 24, 1997 and April 16, 1998).
- 2) As you mentioned in your previous e-mail, I also recommended to continue to try to seat those piles that have not been driven using pile pions.
- 3) If necessary, drive additional piles to achieve the required capacities.
- 4) Finally, if the above options do not work, It is recommended that the piles should be socketed into the bedrock as our section originally suggested..

As you suggested, I agree that NR Structural Section should arrange a meeting with consultants and MTO counterparts as soon as possible. If you need further assistance, please let me know. Also please let me know the meeting time.

Thanks

Tae

>>> Dave Dundas 01/18/99 01:46pm >>>
Peter and Iqbal and Tom

Foundations has provided the CA with modified pile driving recs to keep the Contractor moving. Foundations also recommended to the CA that the pile cap should not be constructed until this situation is resolved in order to keep our options open.

We recommend that NR Structural Section should arrange a meeting with Totten Simms Hubicki and MTO's Construction, Structural and Foundations, to evaluate the problem and the risks of accepting piles driven to a relaxed seating standard and to develop our course of action. I have requested Tae Kim to provide his input since he produced the Foundation design.

Dave

CC: Kazmiero, Sangiuli, Ahmad

From: Peter Stuart
To: MTOHS2.STKITS3.Husain
Date: 1/18/99 1:07pm
Subject: Piling, Doe Lake Road

There has been a problem with piles sliding sideways on the rock surface at this site. In some cases the piles move out of line before they can be seated on the rock. In other cases they have just kept sliding. Dave Dundas has changed the type of rock point and the setting procedure. Hopefully this will improve the situation.

The abutment is designed with a single line of piles. A key question for us is how much out of line can the piles be and still be acceptable. Dave has suggested that if we can't seat all the piles we may be able to accept them at a reduced loading. This would require that we add extra piles. If we did this do these extra piles have to be in line with the others?

peter

CC: MTOHO1.TORHO2.Dundas, MTOHS2.STKITS3.Tharmaba, Fur...

From: Iqbal Husain
To: MTONR.NORTHBAY(StuartPe)
Date: 1/18/99 2:58pm
Subject: Piling, Doe Lake Road -Reply

Peter:

It is my understanding that extra piles are needed because some or most of the piles may not be sufficiently seated in the rock due to the hardness of the rock or reduced energy levels used in trying to make them bite into the rock. Ideally the piles should be in a single row of piles, but some stagger in the piles may not effect the rigidity of the abutment too much, provided the piles are not battered. The effect of eccentricity on the structure and the piles should be evaluated, My fear is that these extra piles may also be not seated and may not provide much comfort.

There could be other structural and geotechnical alternatives to this situation which would have to satisfy the designer and should be discussed among all concerned. We should also review if there could be similar situation on other structures.

Iqbal

CC: MTOHO1.TORHO2(Dundas), MTOHS2.STKITS3(Tharmaba), M...

From: Dave Dundas
To: MTONR.NORTHBAY.StuartPe, MTOHS2.STKITS3.Husain, MT...
Date: 1/18/99 1:46pm
Subject: Pile seating problem Contract 98-13 Doe Lake Rd/Hwy 11

Peter and Iqbal and Tom

Foundations has provided the CA with modified pile driving recs to keep the Contractor moving. Foundations also recommended to the CA that the pile cap should not be constructed until this situation is resolved in order to keep our options open.

We recommend that NR Structural Section should arrange a meeting with Totten Simms Hubicki and MTO's Construction, Structural and Foundations, to evaluate the problem and the risks of accepting piles driven to a relaxed seating standard and to develop our course of action. I have requested Tae Kim to provide his input since he produced the Foundation design.

Dave

CC: Kazmiero, Kim, Sangiuli

From: Bala Tharmabala
To: MTONR.NORTHBAY(FurstPer, StuartPe), Husain
Date: 1/17/99 7:45pm
Subject: Doe Lake Road U' Pass- Foundation Problems -Reply

Iqbal,

Can we discuss this issue.

Bala

>>> Iqbal Husain 01/16/99 11:00am >>>
Peter:

I guess you called me sometimes last week> I was completely snowed in and could not come to St. Catharines on Wednesday, Thursday and Friday. I am in on Saturday to catchup with some of the work.

I guess the problem in not being able to secure the piles in the rock for this bridge is a clear reminder of the limitations of the Integral Abutment Design. Foundation section had clearly indicated its reluctance to use integral abutment design and suggested socketing the piles into rock and or using some kind of rock points.

The problem in not being able to firmly locate the tip of pile in the rock is that it is very difficult to tell how much movement would take place ? Any estimate would be just a guess. Once the determination of the translation is made the effect the structural design is not difficult and that can be easily determined by the designer. The effect could be instantaneous and long term.

In my opinion it is important for the stability, integrity and durability of the structure that the pile tips are firmly secured in to the rock.

Iqbal

CC: MTOHO1.TORHO2(Kim),

From: Dave Dundas
To: MTOHS3.MTOHSGI."hcio_9810@yahoo.com"
Date: 1/12/99 1:40pm
Subject: Contract 98-13, Pile Problems -Reply

Matt

As discussed:

I have consulted the pile driving sub-constructor - Basil Gallant of Bermingham (905- 528-7924) and based on our discussions recommend that Oslo rock points be used. I understand that Bermingham has a stock of 8 Oslo points while there are 12 piles that remain to be seated. Consequently, I recommend that you instruct the Contractor

- to continue to try to seat those piles that have not been driven yet, using the Titus points, until you have reduced the number of unseated piles to 8.

- make arrangements for supply of the available Oslo points

- remove those piles that have been driven but have not been successfully seated using a vibratory hammer and redrive those piles and seat with Oslo points

- drive and seat the remaining piles with Oslo points

I suggest that you should make these arrangements through the prime contractor and that the critical issue is to minimize delays. You may want to consult NR for their approval if that is required.

If there are any other questions, please contact me.

Dave Dundas

CC: Sangiuli, Kazmiero

HCIO Highway Construction Inspection Ontario inc.

268 Pine Drive, Barrie, Ontario L4N 4H8, phone(705)734-2973 fax(705)734-2296

Ⓢ Ontario

MEMORANDUM

January 15, 1999

To: Dave Dundas, MTO Foundation Engineer

From: Matt Gleben, HCIO

Dear Dave;

Re: Contract 98-13, Doe Lake Road East Abutment Piles

Please provide comments on the following information regarding the difficulty experienced by the Contractor in driving the piles at the east abutment at the Doe Lake Road Interchange.

As per previous recommendations the contractor has removed three piles that did not seat properly at the east abutment. Upon examination of the piles that were removed, it was evident that the flanges of pile had contacted the rock prior to the point. To prevent this from happening on future piles the ends of the flanges were trimmed to enable the point to contact the rock first. Piles were then refitted with an Oslo Rock Point and pile driving started again.

At Pile location No. 13 the Contractor has failed to achieve a set into the rock.

The Contractor moved to Pile No. 1. On this pile a set was achieved, however the Contractor wary of applying full force and having the pile break the set, stopped the pile driving at approximately 25,000 Joules/Blow.

On Pile No. 2 the pile appeared to set and penetrate into the rock 25mm. As the steps were increased the pile kicked at the 30,000 Joules/Blow.

We would appreciate your comments on:

1. Can the number of blows per 150mm step be reduced from 5 X 20?
2. Can the maximum driving force be reduced once rock is reached?
3. If the Contractor is unable to achieve a set into the rock what is the next step?

Gravenhurst Site Office

From: Dave Dundas
To: MTOHS3.MTOHSGI."hcio_9810@yahoo.com"
Subject: Contract 98-13, Doe Lake Road, East Abutment Pile Driving -Reply

Matt

I have reviewed the difficulties that have been encountered with seating rock points at this site and recommend that

1) The rock point installation procedure should be modified so that the # of hammer blows at each hammer energy increment is reduced to a minimum of 20 blows provided that no further penetration is being obtained at that increment and that the maximum hammer energy increment should be 25000 joules provided that the rock point has penetrated a minimum of 25 mm. Please keep us advised of the progress.

2) If this modified criteria can not be obtained we will have to review our options. In preparation, I have requested Peter Stuart to investigate the structural engineering position on supplementing the existing pile group at locations where rock points could not be seated by installing additional piles driven to bedrock but not seated in bedrock. Concurrently, Foundations will review the design requirements for seating the piles in bedrock.

Dave

CC: Kazmiero, Sangiuli, MTONH.HUNTSVILLE.PepperTo, MTO...

From: Peter Stuart
To: MTOHO1.TORHO2 (Ahmad)
Date: 4/16/98 5:06pm
Subject: W.P. 429-94-02, Doe Lake Road Underpass -Reply

The contract package presently requires the use of Ottawa sand around the piles and Oslo points to fix the tips.

Peter

>>> Ken Ahmad 04/16/98 02:57pm >>>
Peter:

Please find the attached memo on the above project.

Ken

CC: MTOHO1.TORHO2 (Kazmiero, Kim), MTOHS2.STKITS3 (Husai...



memorandum

To: Peter Stuart, P. Eng.
Senior Structural Engineer
Structural Section
Northern Region

From: Pavements and Foundation Section
Room 223, Central Building
Downsview, Ontario

Re: Final General Arrangement Drawing
Doe Lake Road Underpass
W.P. 429-94-02, Cont. 98-13, Site 42-142
Highway # 11, District 52, Huntsville

1998 04 16

We reviewed the final GA drawing for the above project. Following are our comments:

- In our memo dated 1997 02 24 we had recommended that if integral abutments were selected for this structure, then pile socketing into the bedrock will be required. Since the pile length will range from 5m to 5.4m and the upper 3m of the piles will be within the preaugured holes and fill, the lower portion of the piles are not considered fixed. As a result the pile tips may become unstable. To prevent pile tips movement, the piles should be either socketed into the bedrock or rock point, such as Oslo point should be used.
- A note on the drawing suggests placing "dry sand" within the preaugured holes. The sand should be "Uniformly graded" Ottawa sand or equivalent. The gradation of the sand should meet the following criteria:

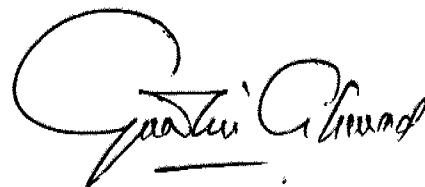
NSSP - Backfill to Integral Abutment-Augured Hole

The annular space between the preaugured oversize hole and the pile shall be backfilled with uniformly graded sand. The gradation for the uniformly graded sand shall be as follows:

MTO SIEVE DESIGNATION	PERCENTAGE PASSING BY MASS
2 mm (#10)	100
600 μm (#30)	80 - 100
425 μm (#40)	40 - 80
250 μm (#60)	5 - 25
150 μm (#100)	0 - 6

Alternatively, commercially available material which meets the above gradation may be considered instead of Ottawa sand.

If you have any questions please advise.

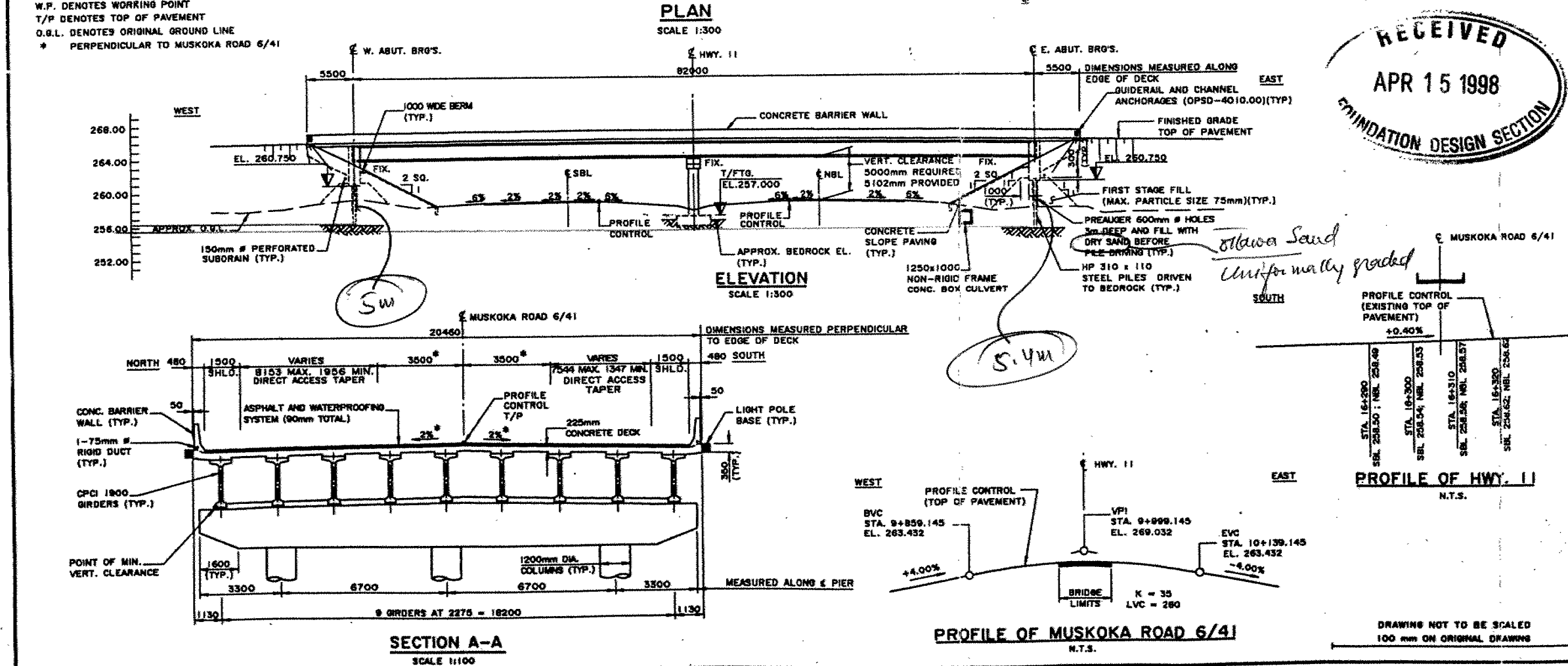


K.S.Q. Ahmad, P. Eng.
Foundation Engineer

For

T.C. Kim, P. Eng.
Senior Foundation Engineer

cc: P. Furst
I. Hussain
T. Kazmierowski





memorandum

To: Peter Stuart, P. Eng.
Senior Structural Engineer
Structural Section
Northern Region

1997 02 24

From: Pavements and Foundation Section
Room 315, Central Building
Downsview, Ontario

Re: Preliminary General Arrangement Drawing
Doe Lake Road Underpass
W.P. 429-94-02, Site 42-142
Highway # 11, District 52, Huntsville

This is in response to your memo of February 10, 1997. We have reviewed the General Arrangement Drawing for the above mentioned project. Our comments and recommendations are as follows:

The original scheme for this project was for spread footing constructed on granular pad. We understand that, it is now proposed to construct the abutment on H-piles. The bridge structure can be supported on steel H-piles driven to bedrock. Since the bedrock is shallow at this site (4.8m below the proposed pile cap at the west abutment), the integral abutment may not be feasible at this site. Consideration may be given to socketing the piles into the bedrock. However, this should be discussed with the structural office for the comments and recommendations.

The recommended bearing resistance of H-piles founded on bedrock are as follows:

	<u>HP 310X110</u>	<u>HP 310X79</u>
Factored Axial Resistance @ ULS	1600 kN/pile	1150 kN/pile

The serviceability Limit State (SLS) will not govern for piles founded on bedrock.

The horizontal subgrade reaction may be calculated using the following soil parameters:

West Abutment

El. (m) From-To	WL.El. (m)	Soil Type	ϕ' (Deg)	Qu (kPa)	γ (kN/m ³)
256.7-255.0	256.4	Non-Cohesive	30	0	20.0
Below 255.0		Bedrock	0	1500	22.5

East Abutment

El. (m) From-To	WL.El. (m)	Soil Type	ϕ' (Deg)	Qu (kPa)	γ (kN/m ³)
256.5-252.1	256.4	Non-Cohesive	30	0	20.0
Below 252.1		Bedrock	0	1500	22.5

where:


ϕ' = angle of internal friction

Qu = unconfined compressive strength in kPa

γ = unit weight in kN/m³

We understand that Oslo points will be used for the piles. The bedrock surface (particularly at the west abutment) is not steep. Instead of using Oslo points the pile tips should be reinforced with driving shoes as detailed in OPSD 3301. If the piles are socketed into the bedrock then no pile tip reinforcement will be required.

If you have any further questions, please advise.



K.S.Q. Ahmad, P. Eng.
Foundation Engineer

For

T.C. Kim, P. Eng.
Senior Foundation Engineer

cc: I. Hussain

memorandum



To: P. Furst, P. Eng.
Head, Structural Section
Northern Region

Attn: P. Stuart, P. Eng.

From: Pavements and Foundations Section
Room 315, Central Building

Subject: Advance Foundation Recommendations
Doe Lake Road Interchange Bridge
Highway 11, Township of Gravenhurst
W.P. 429-94-00, Site No. 42-142
District 52, Huntsville

Date: 95 09 07

The foundation investigation for the above-noted project has recently been complete. This memorandum provides a summary of the subsurface conditions encountered at the site and preliminary foundation recommendations necessary for design to proceed. The final report will be prepared when the E-plans are available.

The site is located on Hwy 11, just south of the north entrance to Gravenhurst, in the Township of Gravenhurst, District of Muskoka. The existing highway embankment at this location is about 1-2 m high. The NBL and SBL are separated by a ditch area in the median. The private properties on both sides of the highway are fenced off and are heavily vegetated with tall trees. It is proposed to construct an interchange for Muskoka Roads 41 and 6 (Doe Lake Road) at Hwy 11. Based on a preliminary drawing provided by your office, it is understood that a two span structure will be constructed with the central pier located in the median of the existing highway. The profile grade of the structure will be approximately 7.3 m above the existing highway.

The field work was carried out between 95 08 02 and 95 08 04 and consisted of six sampled boreholes and three probe-holes (dynamic cone penetration test only). The boreholes were advanced by a track mounted augering machine to depths ranging from 2.8 to 7.0 m. Disturbed subsoil samples were retrieved by a split spoon sampler in accordance with the Standard Penetration Test. Rock coring was carried out in four boreholes (BHs 3, 4, 6 and 7) to prove bedrock surface. The borehole locations are shown on the attached plan. Details of the subsurface conditions and laboratory test results are shown in the Record of Boreholes sheets attached.

The subsurface stratigraphy revealed from the field investigation typically comprised sand and silt overlying bedrock. A surficial layer of topsoil was also encountered in all boreholes. Bedrock was generally contacted at shallow depths. It sloped from El.

255.5 \pm m at the west abutment location to El. 256.8 \pm m at the central pier and dipped down to El. 252-255 \pm m at the east abutment. Depth to bedrock varies from 3-4 \pm m at the abutments to about 0.6 m at the central pier location. The thickness of the topsoil varies but typically between 0.3 to 1.5 m. The sand and silt layer is typically in a compact state of denseness. Bedrock is a slightly weathered to unweathered interlayered Biotite-Hornblende Gneiss, Amphibolite, Marble and Pegmatite. An unconfined compression test was carried out on a rock core sample in the Marble layer and the unconfined compressive strength obtained was 36.1 MPa. The Marble is therefore classified as medium strong while the rest of the rock is considered strong.

Groundwater level was measured in the open boreholes during the investigation. It is generally between El. 256.5 \pm m to El. 257 \pm m. The unstabilized water level measured at BH 9 was El. 255.8. Seasonal fluctuations in ground water level are expected.

The following are the foundation recommendations pertaining to the design and construction of the structure.

FOUNDATION

According to the investigation results, competent subsoil or bedrock exists at relatively shallow depths. The foundation for the structure may therefore be founded on conventional spread footings to achieve a cost effective design.

West Abutment -

At this location, the sand and silt layer is overlain by a 1.5 \pm m thick topsoil stratum. It is recommended to remove the topsoil to El. 257 \pm m and backfill the excavation with granular material to form a pad for placement of footings, as illustrated in Figure 1. Footings should be perched as high as possible within the fill on a minimum 3 m thick granular pad. The granular pad should be constructed to a minimum 1 m edge distance from the top of the footing to the crest of the pad and with 1H:1V slopes. The granular 'A' material must be placed and compacted to achieve 100% of the Proctor maximum density as outlined in OPSS 501-08-02 (Method A). For the purpose of the O.H.B.D.C., the following bearing capacities can be used in the design:

Factored Capacity at U.L.S. = 900 kPa
Bearing Capacity at S.L.S. = 350 kPa

A 1.8 m earth cover should be provided for frost protection purpose.

Pier -

At this location, bedrock is encountered at shallow depth. It is therefore recommended to place the footings directly on bedrock. For the purpose of O.H.B.D.C., the following bearing capacities can be used.

Factored Capacity at U.L.S. = 3500 kPa
Bearing Capacity at S.L.S. does not govern for 'unyielding soils'

A slightly lower bearing capacity is assigned to the bedrock due to the weaker marble layer present in the rock. The highest founding elevations for the footing, as estimated from the investigation results are as follows:

South End	El. 256.5 m
Centre	El. 257.0 m
North End	El. 256.8 m

The above values are for preliminary estimating purposes. Actual founding elevations have to be verified during construction. The rock surface at the footing base should be inspected and all loosened or highly fractured rock should be removed prior to placement of concrete. For footings founded on bedrock, frost cover is not required.

East Abutment -

At this location, bedrock dips from El. 255 m to El. 252 m in a northerly direction. Placing the footings on bedrock is impractical. It is therefore recommended to adopt the same design as the West Abutment and place the footing on a compacted Granular 'A' pad. The plan area of the granular pad should be cleared of topsoil. This would involve subexcavation to El. 256.5 \pm m and backfilling with granular material. As recommended above for the West Abutment, the following bearing capacities can be used:

Factored Capacity at U.L.S. = 900 kPa
Bearing Capacity at S.L.S. = 350 kPa

A 1.8 m earth cover should be provided for frost protection purpose.

Sliding Resistance -

The computation of the sliding resistance of the foundation shall be carried out in accordance with Section 6-8.4.2 of the O.H.B.D.C. $\phi = 35^\circ$ can be used for sliding within the granular fill and $\delta = 30^\circ$ can be used for sliding at footing/granular fill interface. The same values may be used for footings on bedrock provided that the rock

surface is relatively rough.

BACKFILL

Backfill to abutments or retaining walls should consist of granular materials such as Granular 'A' and 'B', or rock fill. Computation of earth pressure shall be in accordance with Section 6.7.4 of the O.H.B.D.C. Unfactored properties for backfill materials are provided in the following table:

<u>Material</u>	ϕ	γ
Granular 'A'	35°	22.8 kN/m ³
Granular 'B'	30°	21.2 kN/m ³
Rock Fill	35°	19.0 kN/m ³


ABUTMENT SLOPES

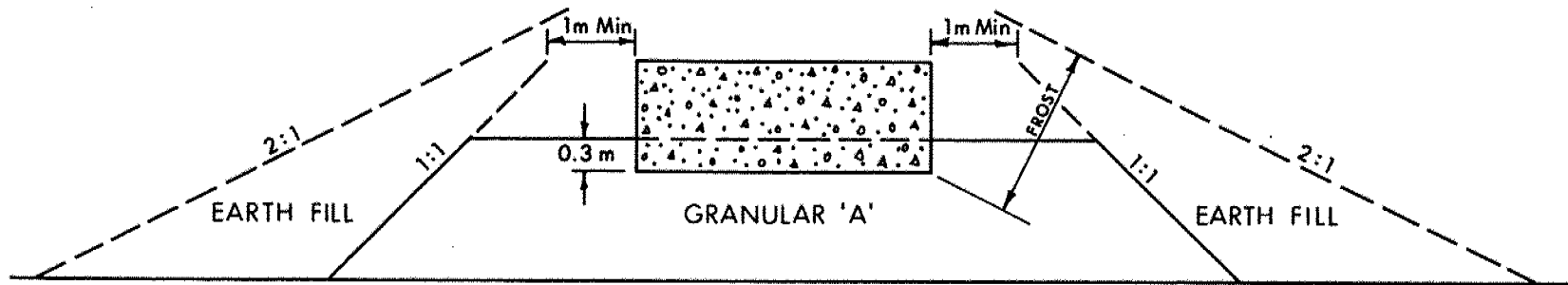
The maximum fill height is about 8 m. It is recommended that the fill slopes be formed at a gradient of 2H:1V or flatter, up to a maximum height of 10 m. Topsoil and any soft or unsuitable material should be removed prior to filling. Normal slope protection such as vegetation cover shall be established as soon as possible after completion of the slope formation in order to control surficial erosion.

CONSTRUCTION CONSIDERATIONS

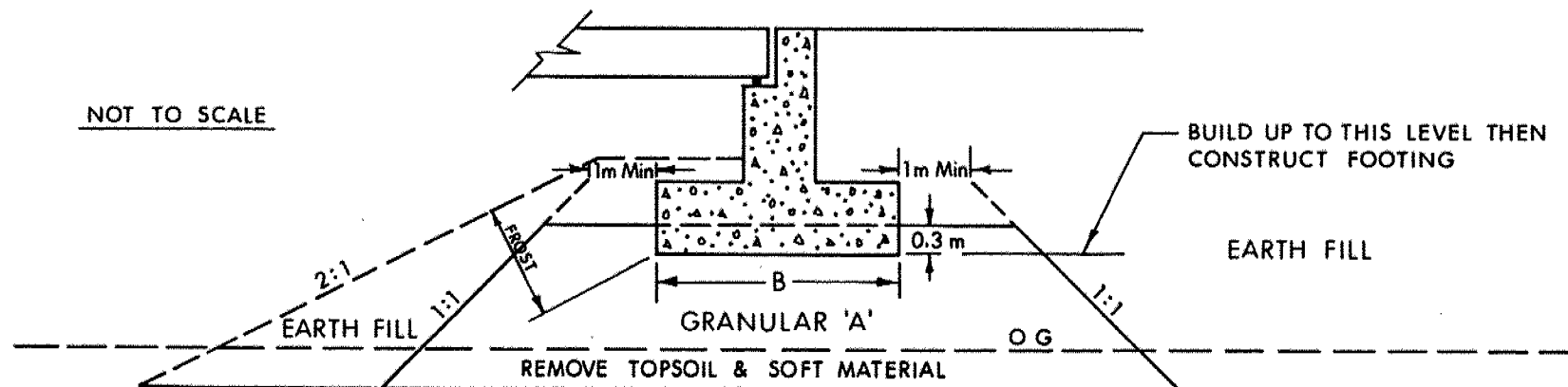
Temporary excavations of up to 2 ±m is required for footing construction. Cut slopes can be formed at a gradient of 1H:1V. Some dewatering may be required. Unwatering can be carried out by conventional sump pumping techniques.

We believe that the above is sufficient for your present purpose. Should you require further information, please contact our office.


D. Kwok, P. Eng.
Project Foundation Engineer
for
T. Kim, P. Eng.
Senior Foundation Engineer



X SECTION



LONGITUDINAL SECTION

NOTES:

- 1- REMOVE TOPSOIL &/OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' & EARTH FILL.
- 2- PLACE GRANULAR 'A' & EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT M T O STANDARDS.
- 3- CONSTRUCT CONCRETE FOOTING.
- 4 - PLACE REMAINDER OF GRANULAR 'A' & EARTH FILL AS REQUIRED.



Ontario

Ministry of
Transportation

ABUTMENT ON COMPACTED FILL
SHOWING GRANULAR 'A' CORE

FIG No 1

W P 429-94-00

RECORD OF BOREHOLE No 1

1 OF 1

METRIC

W.P. 429-94-00 LOCATION Sta. 9+953.2 O/S 9.8 m Rt. Musk.Rd. 41&6 ORIGINATED BY DK
DIST 52 HWY 11 BOREHOLE TYPE H.S. Auger, Cone Test COMPILED BY DK
DATUM Geodetic DATE 95 08 02 CHECKED BY TK

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100	W _p	W	W _L		
258.4	Ground Surface																
0.0	Silty Sand with Organics Some Root hairs Brown, Compact (Topsoil)		1	SS	14											0 53 45 2	
258.7			2	SS	23												
1.7	Sand and Silt Brown and Grey, Compact		3	SS	21											0 42 54 4	
255.6																	
2.8	End of Borehole * 95 08 03 ** Auger Refusal on Probable Bedrock	**															

RECORD OF BOREHOLE No 2

1 OF 1

METRIC

W.P. 429-94-00 LOCATION Sta. 9+957.7 C.L. Musk.Rd. 41&6 ORIGINATED BY DK
DIST 52 HWY 11 BOREHOLE TYPE Cone Test COMPILED BY DK
DATUM Geodetic DATE 95 08 02 CHECKED BY TK

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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
258.5	Ground Surface																
0.0	Probable Silty Sand With Organics (Topsoil)																
257.0																	
1.5	Probable Sand and Silt																
255.6																	
2.9	End of Borehole Probable Bedrock																

RECORD OF BOREHOLE No 3

1 OF 1

METRIC

W.P. 429-94-00 LOCATION Sta. 9+962.8 O/S 8.6 m Lt. Musk. Rd. 41&6 ORIGINATED BY DK
DIST 52 HWY 11 BOREHOLE TYPE H.S. Auger, NQ Core Barrel, Cone Test COMPILED BY DK
DATUM Geodetic DATE 95 08 02 CHECKED BY TK

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100	W _p	W	W _L		
258.9	Ground Surface																
0.0	Silty Sand with Organics Trace of Roothair Brown to Dark Brown, Loose (Topsoil)		1	SS	9												
257.4																	
1.5	Trace Clay Occasional Roothair		2	SS	10												
	Sand and Silt		3	SS	17												
	Grey, Compact		4	SS	11												
254.9	Boulders																
4.0			5	RC	REC	100%										RQD 100%	
	Bedrock		6	RC	REC	100%											
			7	RC	REC	100%											
251.9																	
7.0	End of Borehole																
	* 95 08 03																

RECORD OF BOREHOLE No 4

1 OF 1

METRIC

W.P. 429-94-00 LOCATION Sta. 9+996.7 O/S 9.4 m Rt. Musk. Rd. 41&6 ORIGINATED BY DK
DIST 52 HWY 11 BOREHOLE TYPE H.S. Auger, NQ Core Barrel, Cone Test COMPILED BY DK
DATUM Geodetic DATE 95 08 03 CHECKED BY TK

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20	40	60	80	100	W _p	W		
257.2	Ground Surface															
0.0	Silty Sand with Organics Brown, Very Loose (Topsoil)		1	SS	3											
256.5																
0.7			2	RC	REC	90%										RQD 19%
	Bedrock		3	RC	REC	100%										RQD 79%
			4	RC	REC	100%										RQD 100%
			5	RC	REC	100%										RQD 97%
253.4																
3.8	End of Borehole															
	* 95 08 04															

RECORD OF BOREHOLE No 6

1 OF 1

METRIC

W.P. 429-94-00 LOCATION Sta. 10+003.9 O/S 9.0 m Lt. Musk Rd. 41&6 ORIGINATED BY DK
DIST 52 HWY 11 BOREHOLE TYPE H.S. Auger, NQ Core Barrel, Cone Test COMPILED BY DK
DATUM Geodetic DATE 95 08 03 CHECKED BY TK

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20	40	60	80	100	W _p	W			W _L
257.6	Ground Surface																
0.0	Silty Sand with Organics (Topsoli)		1	SS	8												
0.3	Sand and Silt Brown, Loose																
256.8																	
0.8	Bedrock		2	RC	REC	71%										RQD 25%	
			3	RC	REC	62%											RQD 13%
			4	RC	REC	100%											RQD 100%
			5	RC	REC	100%											RQD 93%
253.6																	
4.0	End of Borehole																
	* 95 08 04																

RECORD OF BOREHOLE No 7

1 OF 1

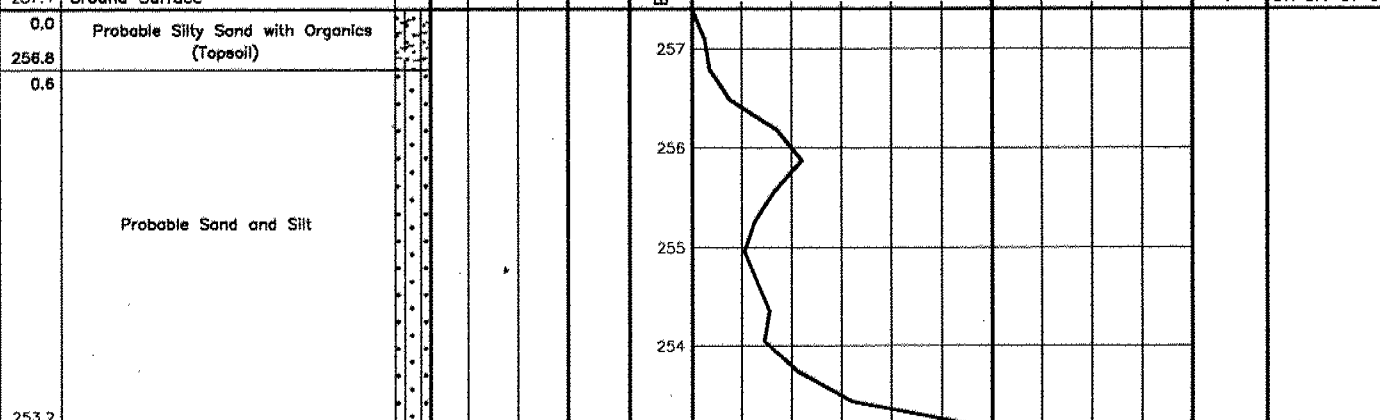
METRIC

W.P. 429-94-00 LOCATION Sta. 10+038.2 O/S 8.5 m Rt. Musk. Rd. 41&6 ORIGINATED BY DK
DIST 52 HWY 11 BOREHOLE TYPE H.S. Auger, NQ Core Barrel, Cone Test COMPILED BY DK
DATUM Geodetic DATE 95 08 03 - 95 08 04 CHECKED BY TK

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 10 20 30 40 50	PLASTIC LIMIT w _p NATURAL MOISTURE CONTENT w LIQUID LIMIT w _L	WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES							
257.2	Ground Surface											
0.0	Silty Sand with Organics Dark Brown, Compact (Topsoil)		1	SS	19		256					
1.0	Sand and Silt Brown and Grey Compact		2	SS	19		255					
255.1	----- Boulders											
2.1	Bedrock											
251.9												
5.3	End of Borehole • 95 08 04											

METRIC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ KN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI C
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	W _P W W _L	WATER CONTENT (%) 10 20 30			
257.4	Ground Surface						SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE 10 20 30 40 50						

[illegible]

+3, x5: Numbers refer to Sensitivity

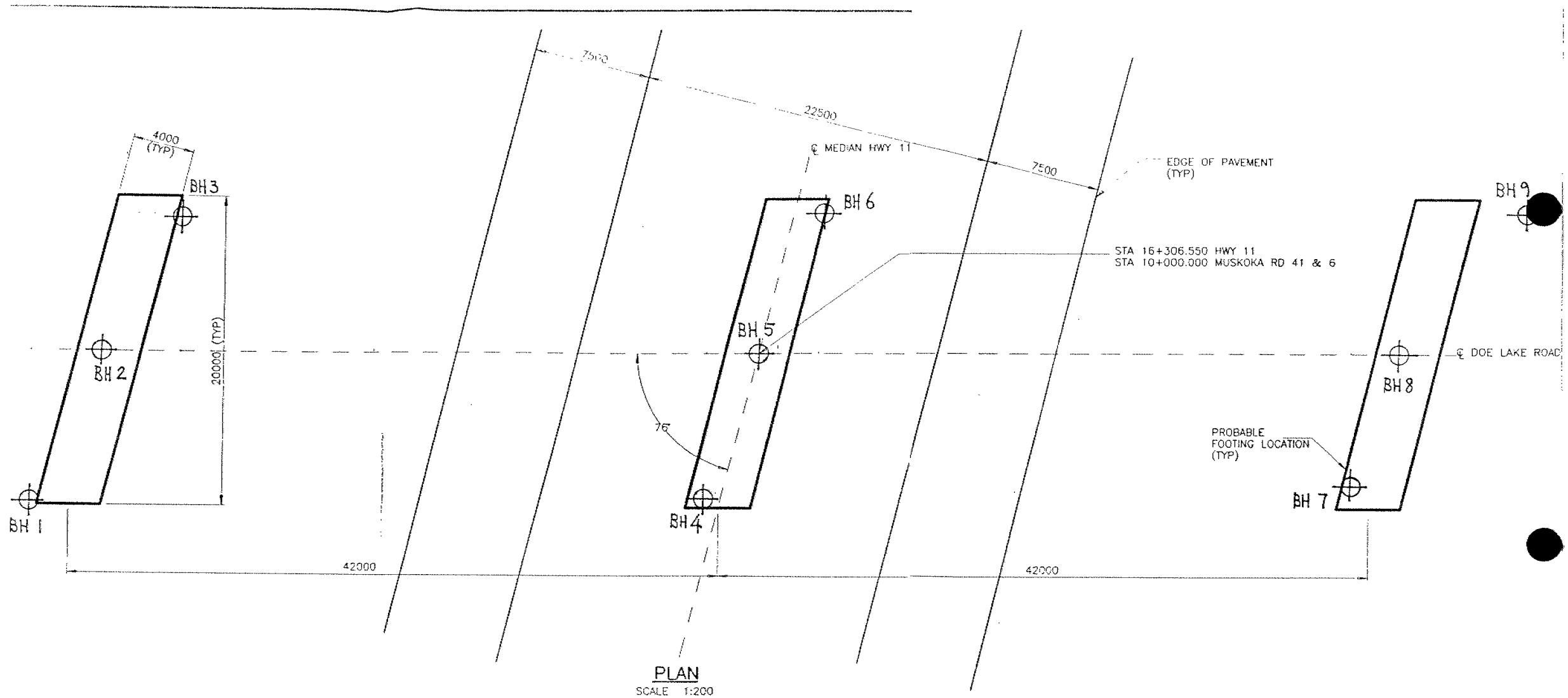
RECORD OF BOREHOLE No 9

1 OF 1

METRIC

W.P. 429-94-00 LOCATION Sta. 10+049.2 O/S 9.0 m Musk. Rd. 41&6 ORIGINATED BY DK
DIST 52 HWY 11 BOREHOLE TYPE H.S. Auger, Cone Test COMPILED BY DK
DATUM Geodetic DATE 95 08 04 CHECKED BY TK

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 10 20 30 40 50	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES								
257.4	Ground Surface												
0.0	Silty Sand with Organics Scattered Rootlets Brown, Compact (Topsoil)												
0.9	Brown Grey Sand and Silt Loose to Compact Trace Gravel, Dense Cobbles & Boulders		1	SS	27								0 59 32 9
			2	SS	14								
			3	SS	14								
			4	SS	4								
			5	SS	9								
			6	SS	50								7 46 45 2
252.1													
5.3	End of Borehole * Unstabilized water level measured upon completion ** Auger refusal on probable bedrock												



*DOE LAKE ROAD INTERCHANGE
HWY 11, SITE 42-142
JULY 1995*



VIEW NORTH



VIEW WEST

*DOE LAKE ROAD INTERCHANGE
HWY 11, SITE 42-142
JULY 1995*



VIEW EAST

