coffey geotechnics
SPECIALISTS MANAGING THE EARTH

FOUNDATION INVESTIGATION AND DESIGN REPORTS HIGHWAY 401 / MOULINETTE ROAD BRIDGE, CITY OF CORNWALL, ONTARIO G.W.P. 256-00-01, SITE #31-163, GEOCRES NO. 31G-231

AECOM

Project: SPT1227A
August 31, 2009

Coffey Geotechnics Inc.
20 Meteor Drive Etobicoke Ontario M9W 1A4 Canada

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SPECIALISTS MANAGING THE EARTH

August 31, 2009

AECOM
5080 Commerce Boulevard
Mississauga, Ontario
L4W 4P2

Attention: Mr. Bruce Dickey, P.Eng.

Dear Sirs:

RE: Foundation Investigation and Design Reports, Highway 401, Moulinette Road Bridge, City of Cornwall, Ontario G.W.P. 256-00-01; Site 31-163, GEOCRES No. 31G-231

Please find attached the Foundation Investigation and Design Reports relating to the above noted site.

For and on behalf of Coffey Geotechnics Inc.

Ramon Miranda, P.Eng.
Manager, Transportation Division

Attachment A: Attachments

FOUNDATION INVESTIGATION REPORT HIGHWAY 401 / MOULINETTE ROAD BRIDGE, CITY OF CORNWALL, ONTARIO G.W.P. 256-00-01, SITE #31-163, GEOCRES NO. 31G-231

AECOM

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CONTENTS

FOUNDATION INVESTIGATION REPORT
HIGHWAY 401/MOULINETTE ROAD BRIDGE
CITY OF CORNWALL, ONTARIO
G.W.P. 256-00-01; SITE 31-163

1 INTRODUCTION

The existing bridge which carries Moulinette Road over Highway 401 in the City of Cornwall is to be replaced with a new structure. Coffey Geotechnics Inc. (Coffey) was retained by AECOM to carry out a foundation investigation at the site of the proposed bridge (MTO Site No. 31-163).

The existing Bridge is a four-span bridge with a total length of about 70 m. It is our understanding that the existing bridge will be replaced by a two-span bridge with a similar length. The new bridge will be located to the west of the existing bridge.

The purpose of the investigation was to obtain information about the subsurface conditions at the proposed bridge site by means of boreholes, and to determine the engineering characteristics of the subsurface soils by means of field and laboratory tests.

The findings of the investigation are presented in this report.

2 SITE DESCRIPTION AND GEOLOGY

This Project site is located at the intersection of Moulinette Road with Highway 401 in the City of Cornwall.

According to the Physiography of Southern Ontario by L.J. Chapman and D.F. Putnam, 1984, this project site is located within the Physiographic Region known as the Glengarry Till Plain.

The till has a medium texture and contains a high proportion of limestone mixed with materials derived from the Precambrian rocks to sandstone of the Nepean Formation. The outstanding characteristic of the soils in the region is stoniness. The till itself is very stony, and on the crests of the ridges and drumlins, which suffered wave action in the Champlain Sea, there are boulder pavements. The action of the wave also built numerous bars of sand and gravel deposits. There are, also, large undrained depressions in which peat and mucks are found.

According to Southern Ontario Geological Highway Map (Map 2418), the bedrock underlying this area consists of Middle Ordovician limestone and shale.

From an MTO drawing (dated February 1962) provided to use by AECOM, the existing Moulinette Road underpass site is underlain by glacial sandy silt to silty sand tills extending to bedrock at about Elevation 70 m.

The existing approach embankments, which are approximately 6 to 7 m high, exhibit neither apparent signs of instability not excessive erosion. In the immediate vicinity of the existing bridge abutment no signs of excessive settlements were noted (e.g. cracking/deformations in the pavement).

3 INVESTIGATION PROCEDURES

The fieldwork for the proposed bridge was performed during the period of December 2008 through February 2009. The fieldwork consisted of drilling and sampling of ten boreholes (Boreholes M1 through M10). The following table summarizes the borehole locations and drilling depths.

Table 3.1: Borehole Locations and Drilling Depths

Borehole No.	Location	Depth of Borehole Below Existing Ground Surface (m)	Piezometer
M1 & M1A	9+969 (Abutment-north)	4.8/7.4	1 Piezometer
M2 & M2A	9+973 (Abutment-north)	5.7/9.2	-
M3	10+000 (Pier-centre)	7.8	-
M4	10+000 (Pier-centre)	8.2	-
M5	10+032 (Abutment-south)	9.2	-
M6	10+023.5 (Abutment-south)	12.2	1 Piezometer
M7	9+951 (Approach-north)	7.4	-
M8	10+050 (Approach-south)	8.4	-
M9	9+948 (Approach-north)	9.8	-
M10	10+051 (Approach-south)	11.3	-

Marathon Drilling of Ottawa, Ontario carried out the drilling, testing and sampling work, under the direction and supervision of a Professional Engineer from Coffey. The boreholes were advanced using truck/track mounted drilling rigs, outfitted with tools and equipment for soil sampling and testing. The boreholes were advanced using three different methods (i.e. continuous flight hollow-stem augers, wash boring in the overburden and coring) depending on the ground conditions.

Samples in the boreholes were taken at frequent intervals of depth by the Standard Penetration Test method (SPT), in general accordance with ASTM D1586. This test consists of freely dropping a 63.5 kg hammer a vertical distance of 0.76 m to drive a 51 mm O.D. split barrel (SS – split-spoon) sampler into the ground. The number of blows of the hammer required to drive the sampler into the relatively undisturbed ground by a vertical distance of 0.30 m is recorded as the Standard Penetration Resistance or the N-value of the soil which is indicative of the compactness condition of cohesionless granular soils (gravels, sands and silts) or the consistency of cohesive soils (clays and clayey silts).

Groundwater conditions in the boreholes were observed during drilling and upon completion in the open boreholes. In addition, a piezometer was installed in each of Boreholes M1 and M6 to enable groundwater level monitoring in the boreholes over a prolonged period of time without interference from surface water. The remaining boreholes were grouted upon their completion using a cement/bentonite mixture as per MTO procedures.

The borehole locations were established in the field by Coffey engineering staff, in relation to the existing features. The locations were then tied in and the geodetic elevations of the ground at the borehole locations were determined by the client's surveyors. This survey information was provided to us.

The soil samples were transported to our geotechnical laboratory in Toronto for further examination and classification. A laboratory testing programme, natural moisture content, grain size analyses, and Atterberg

Limits tests, was performed on selected representative soil samples. The results of the laboratory tests are presented on the appropriate Record of Borehole Sheets (Appendix A) and also in Appendix B.

4 SUBSURFACE CONDITIONS

The sub-surface conditions were explored at ten (10) boreholes (see Table 3.1 in Section 3) for this project. The plan locations of the boreholes are shown on Drawing No. 1 while a stratigraphic section and profiles are presented on Drawing Nos 2, 3 and 4. Details of sub-surface conditions encountered at each borehole location for the investigation, including the results of in-situ testing, groundwater observations and laboratory test results, are presented on the Record of Borehole Sheets in Appendix A. Detailed laboratory test results are enclosed in Appendix B.

For the boreholes, drilled from the Highway 401 level or adjacent to it, ground elevations ranged from 81.1 to 83.6 m (Boreholes M1 through M8). Boreholes M9 and M10 were drilled from the top of the existing highway embankment, from Elevations 88.3 and 88.6 m, respectively.

Beneath a 0.1 to 0.2 m thick topsoil and 0.6 to 1.2 m thick fill layer (in Boreholes M2, M6, M7 and M8), in general, the most of the boreholes show the presence of between 0.6 and 2.1 m thick surficial native soils which consist of silt, clayey silt, silty sand, organic silt, and peat, while Boreholes M9 and M10 encountered about 7 m of embankment fill. Underlying the fill and surficial soils, all boreholes contacted at Elevations ranging between 81.4 and 80.3 m, a sandy silt to silty sand till deposit to the full extent of the investigation.

Details of the subsurface conditions encountered in the boreholes are presented on the Record of Borehole Sheets in Appendix A. The following paragraphs are only meant to amplify and complement these data.

4.1 Topsoil

A 0.1 to 0.2 m thick surficial topsoil layer was contacted in each borehole, except for Boreholes M9 and M10 (which were drilled from the paved portion of Moulinette Road embankment). In Borehole M9 a compressed 0.2 m thick topsoil and organic silt layer was contacted underlying the embankment fill.

4.2 Fill

Boreholes M2, M6, M7 and M8, drilled from the base of the Moulinette Road embankment, contacted an about 0.6 m to 3.0 m thick fill material extending to depths of 0.8 m (Borehole M7) to 3.1 m (Borehole M8). The fill was found to extend to El. 81.6 and 80.5 m. The fill was generally found to consist of silt to sandy silt with traces of clay and gravel or organics. These deposits are considered to be fine-grained granular (basically non-cohesive) materials. N-values recorded range from 6 to 11 blows/0.3 m, indicating a loose to compact condition.

In Boreholes M2 and M6, the basically silt fill is underlain at a depth of 0.8 m below the ground surface by a granular fill which consists of sand & gravel or sandy gravel. This granular fill was found to be 0.7 to 1.0 m thick.

The grain-size distribution of a sample from the granular fill is given in Figure B-1, in Appendix B. This indicates the following grain-size distribution.

Gravel:	40%
Sand:	40%
Silt & Clay:	20%

Standard Penetration tests performed in the 0.7 m to 1.0 m thick granular fill yielded N-values of 23 and 43 blows/0.3 m. These results indicate that the relative density of the granular fill can be described as compact to dense. These results indicate that the granular fill received a systematic compaction when the fill was first placed. This deposit is considered to be granular (i.e. non-cohesive) material.

Boreholes M9 and M10, which were advanced from the top of the road embankment contacted fill materials to depths of 6.8 m (El. 81.5 m) and 6.9 m (El. 81.7 m), respectively. In addition, in Borehole M10, a gravelly sand layer was contacted below 6.9 m (El. 81.7 m) to a depth of 8.0 m or to El. 80.6 m, which was identified as possible fill. These two boreholes were drilled from the top of the asphalt paved road surface and contacted a 0.2 m thick layer of asphaltic concrete, underlain by granular pavement fill to a depth of 1.2 m below the asphalt road surface. In Borehole M10, a granular soil consisting of gravelly sand was contacted from 1.2 m to 2.9 m depth. From the recorded N-values of 32 and 41 blows/0.3 m, it appears to be well-compacted (i.e. dense relative density).

Underlying these granular soils the embankment fill was found to consist of sandy silt to silty sand with traces to some gravel and traces of clay. This fill material appeared to be derived from the indigenous till materials underlying the site. In general, the recorded N-values range from 14 to 30 blows/0.3 m indicating a compact condition and that the fill has received some systematic compaction, except for an about 1 m thick zone in Borehole M9, where an N-value of 4 blows/0.3 m was recorded, indicating a very loose condition. The sandy silt to silty sand embankment fill is underlain in both boreholes by a 0.4 to 0.7 m thick sand and gravel to gravelly sand fill layer to Elevations of 81.5 and 81.7 m. These were probably placed after stripping the site when the bridge was first built, to facilitate the compaction of the overlying fill. From recorded N-values of 22 to 37 blows/0.3 m, this granular embankment fill appeared to be well-compacted with a relative density of compact to dense.

4.3 Organic Silt to Clayey Silt and Peat

Underlying the topsoil in Boreholes M4 and M5 and the surficial fill in Borehole M7, an organic silt to clayey silt layer was contacted at depths of 0.2/0.2/0.8 m, respectively or at between El. 82.5 and 81.2 m. The thickness of this deposit was found to range from 0.6 to 0.9 m and the deposit extended to El. 80.6 to 81.6 m. Relatively high organic content and the presence of rootlets and pieces of wood were noted in the material. The measured moisture contents of these materials vary between 26% and 56%.

The recorded N-values range from 5 to 7 blows/0.3 m, which indicate a loose compactness condition of the organic silt materials in Boreholes M4 and M7, and firm consistency of the organic clayey silt in Borehole M5.

Underlaying the organic clayey silt, Borehole M5 contacted a 0.7 m thick peat layer that extended between El. 81.6 m and El. 80.9 m. The peat material generally consisted of fibrous organic matter demonstrating high compressibility. The measured natural moisture contents of two samples from this deposit are 200% and 239%.

4.4 Silt to Silty Sand

Boreholes M1 and M3 contacted a 0.6 m thick silt layer below the topsoil. This deposit extended to El. 80.3 m in Borehole M1 and to El. 80.4 m in Borehole M3. The silt is a fine-grained granular soil with traces of gravel and clay.

N-values recorded in the surficial silt deposit are 4 blows/0.3 m (Borehole M3) and 8 blows/0.3 m (Borehole M1) which indicate a very loose to loose condition.

A 0.2 to 0.4 m thick sandy silt to silt layer was also contacted in Boreholes M9 and M10, underlying the embankment fill and topsoil at El. 81.3 and 80.6 m, respectively.

4.5 Clayey Silt

Borehole M5 encountered a 0.5 m thick clayey silt layer at a depth of 1.8 m or at El. 80.9 m, below the surficial organic clayey silt and peat layers. The deposit contained trace organics and rootlets. This is a cohesive material and based on an N-value of 5 blows/0.3 m, which was recorded, the consistency of the deposit is described as firm.

4.6 Silty Sand to Sandy Silt Till

Underlying the fill materials or the native surficial soil deposits, all boreholes contacted a glacial till deposit at depths ranging from 0.8 m to 8.2 m (El. 80.3 m to El. 81.4 m). The following table summarizes the top elevations of the deposit, as encountered in the boreholes.

Table 4.6.1: Depth/elevation of top of glacial till deposit

Borehole No.	Depth Below Ground Surface/Elevation of the Top of the Deposit(m)
M1	0.8 / 80.3
M2	1.5 / 81.4
M3	0.8 / 80.4
M4	0.8 / 80.6
M5	2.3 / 80.4
M6	1.8 / 80.5
M7	1.5 / 80.9
M8	3.1 / 80.5
M9	7.4/80.9
M19	8.2/80.4

All Boreholes were terminated in this deposit upon encountering practical refusal on the augers. The deposit consists of a heterogeneous mixture of sandy silt to silty sand with traces to some clay. This is a basically fine-grained granular (i.e. non-cohesive) soil type. Due to the clay content and some cementation, the upper portion of the deposit exhibits some cohesiveness or cementation to depths of about 3.5 m to 4.0 m. With the decreasing clay content, the cohesiveness and cementation of the deposit becomes less prominent below these depths. The lower portion of the deposit was found to contain cobbles and boulders which necessitated advancing holes in this deposit in some boreholes by coring (Boreholes M4 and M6). As well Boreholes M1 and M2 encountered refusal to augering which necessitated drilling additional boreholes at a later date (i.e. Boreholes M1A and M2A). The presence of cobbles and boulders should be expected to occur throughout in the till deposits, due to their mode of deposition.

The grain-size distribution of nine samples from the slightly cohesive upper portion of the till deposit is given in Figure B-2 in an envelope form, which shows the following grain-size distribution:

Gravel:	6-27%
Sand:	35-48%
Silt:	26-36%
Clay:	11-13%

The Atterberg limits test performed on a sample from Borehole M4 is given in Figure B-3 in Appendix B. The test yielded the following index values:

Liquid Limit:	17%
Plastic Limit:	14%
Plasticity Index:	3

These results indicate an ML material.

Another sample that was tested (Borehole 7, Sample 3) yielded a Liquid Limit of 13%, and the sample was found to be non plastic (i.e. plastic limit could not be determined).

The grain-size distribution of four samples from the non-cohesive lower portion of the till deposit is given in Figure B-4 in an envelope form, which shows the following grain-size distribution:

Gravel:	13-28%
Sand:	37-53%
Silt:	26-32%
Clay:	3-10%

Sand and gravel or sandy gravel interbeds within the sandy silt to silty sand till deposit were observed in Boreholes M3 and M6, at depths of about 7.6 m (El. 73.6 m) and 6.3 m (El. 76 m), respectively. The grain-size distribution of one sample from this deposit is given in Figure B-5, which shows the following grain-size distribution:

Gravel:	28%
Sand:	53%
Silt:	16%
Clay:	3%

Standard Penetration tests performed in the sandy silt to silty sand till deposit gave N-values which range from 5 blows/0.3 m to in excess of 100 blows per 0.1 m indicating a compact to very dense relative density with occasional loose zones. Generally, the deposit is compact in the upper portion and becomes dense to very dense below about El. 77 m to 76 m.

4.7 Groundwater Conditions

Groundwater conditions were observed in the open boreholes while drilling and upon completion of each borehole. These short term observations may not represent the stabilized groundwater levels. In the deep boreholes, where NQ coring and wash boring were used (i.e. water introduced into the boreholes) the on-completion water levels are unlikely to be reliable. The observations made in the boreholes are shown on the individual Record of Borehole Sheets in Appendix A and are summarized in the following table. To enable us to monitor groundwater levels over a prolonged period of time without interference from surface water, piezometers were installed in Boreholes M1 and M6.

Table 4.7.1: Groundwater condition

Borehole No	Ground Surface Elevation (m)	Depth/Elevation of the Tip of Piezometer (m)	Water Level Measurement Depth/Elevation (m)	Date	Piezometers
M1	81.1	4.7/76.4	0.9/80.2	Jan. 28/09	Yes
M2	82.9		1.6*/81.3	Dec. 10/08	
M3	81.2		1.4*/79.8	Dec. 19/08	
M4	81.4		0.9*/80.5	Dec. 19/08	
M5	82.7		1.4*/81.3	Jan. 06/09	
M6	82.3	11.6/70.7	1.4/80.9	Jan. 28/09	Yes
M7	82.4		2.3*/80.1	Dec. 10/08	
M8	83.6		1.5*/82.1	Jan. 05/09	
M9	88.3		Caved @ 7.3 m/El. 80.3*	Feb. 25/09	
M10	88.6		Caved @ 5.5 m/El. 83.1 m		

*groundwater table not stabilized

From the measured values, it is our opinion that groundwater level was about 1 m below original grade at the time of investigation or at about El. 80 m at the Highway 401 level and at about El. 81 m beyond the highway level (i.e. near the abutments). Based on the piezometer readings and recorded water levels in the open boreholes, it is our opinion that these elevations represent the true water level at the site, emanating from within the glacial till deposit. Higher water levels are also possible due to the accumulation of the surface water in the surficial deposits, overlying the glacial till, including the fill deposits.

It should be pointed out that the water observed levels represent the conditions at the time of our investigations and that they would be subject to fluctuations, both seasonally and in response to major weather events.

For and on behalf of Coffey Geotechnics Inc.

Eva Papp, P.Eng.

LICENSED PROFESSIONAL ENGINEER
RAMON MIRANDA
PROVINCE OF ONTARIO

Ramon Miranda, P.Eng.

REGISTERED PROFESSIONAL ENGINEER
Z. S. OZDEN
PROVINCE OF ONTARIO

Zuhtu Ozden, P.Eng.

Drawings

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS REFER TO RECORD OF BOREHOLE SHEETS.

METRIC

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

CONT No.
WP: 256-00-01

SHEET

MOULINETTE ROAD AND HIGHWAY 401 BOREHOLE LOCATION PLAN

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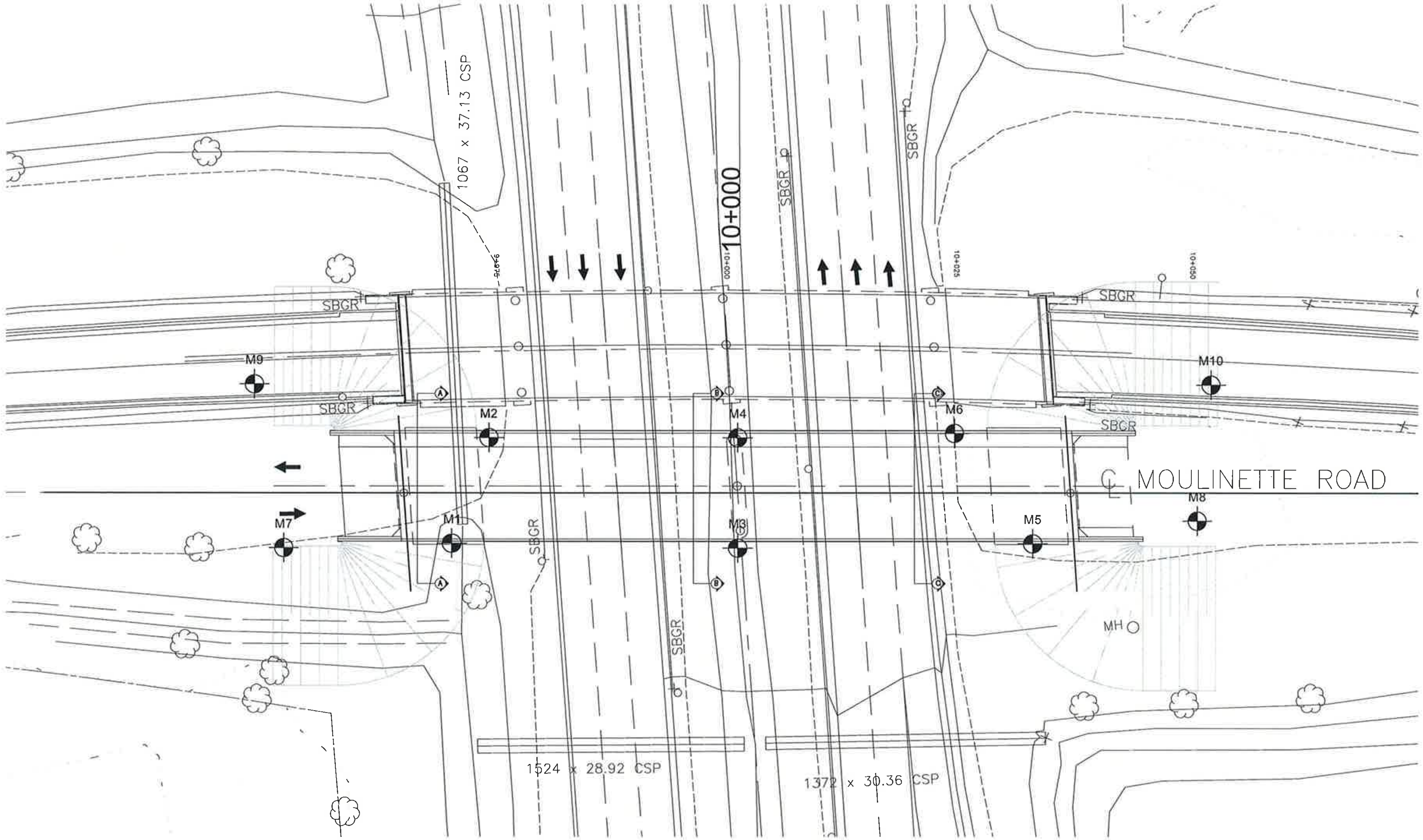
SPECIALISTS MANAGING THE EARTH



PLAN

10m 0 10 20m
SCALE

KEY PLAN
N.T.S.

LEGEND

Borehole

No.	ELEV.	STATION No.	OFFSET
M1	81.1	9+969	5.5m Rt C/L
M2	82.9	9+973	6.0m Lt C/L
M3	81.2	10+000	6.0m Rt C/L
M4	81.4	10+000	6.0m Lt C/L
M5	82.7	10+032	5.5m Rt C/L
M6	82.3	10+023.5	6.5m Lt C/L
M7	82.4	9+951	6.0m Rt C/L
M8	83.6	10+050	3.0m Rt C/L
M9	88.3	9+948	11.8m Lt C/L
M10	88.6	10+052	11.8m Lt C/L

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

NOTE: This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

LICENSED PROFESSIONAL ENGINEER
Aug 25/09
RAMON MIRANDA
PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER
Aug 25/09
Z. S. OZDEN
ZO
PROVINCE OF ONTARIO

REV.	DATE	BY	DESCRIPTION

Geocres No. 31G-231

SPT 1227A			DIST
SUBM'D	CHECKED	DATE Mar. 2009	SITE
DRAWN PHK	CHECKED RM	APPROVED ZO	DWG 1

METRIC

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS REFER TO RECORD OF BOREHOLE SHEETS.

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

CONT No.

WP: 256-00-01

MOULINETTE ROAD AND HIGHWAY 401
PROFILE

SHEET

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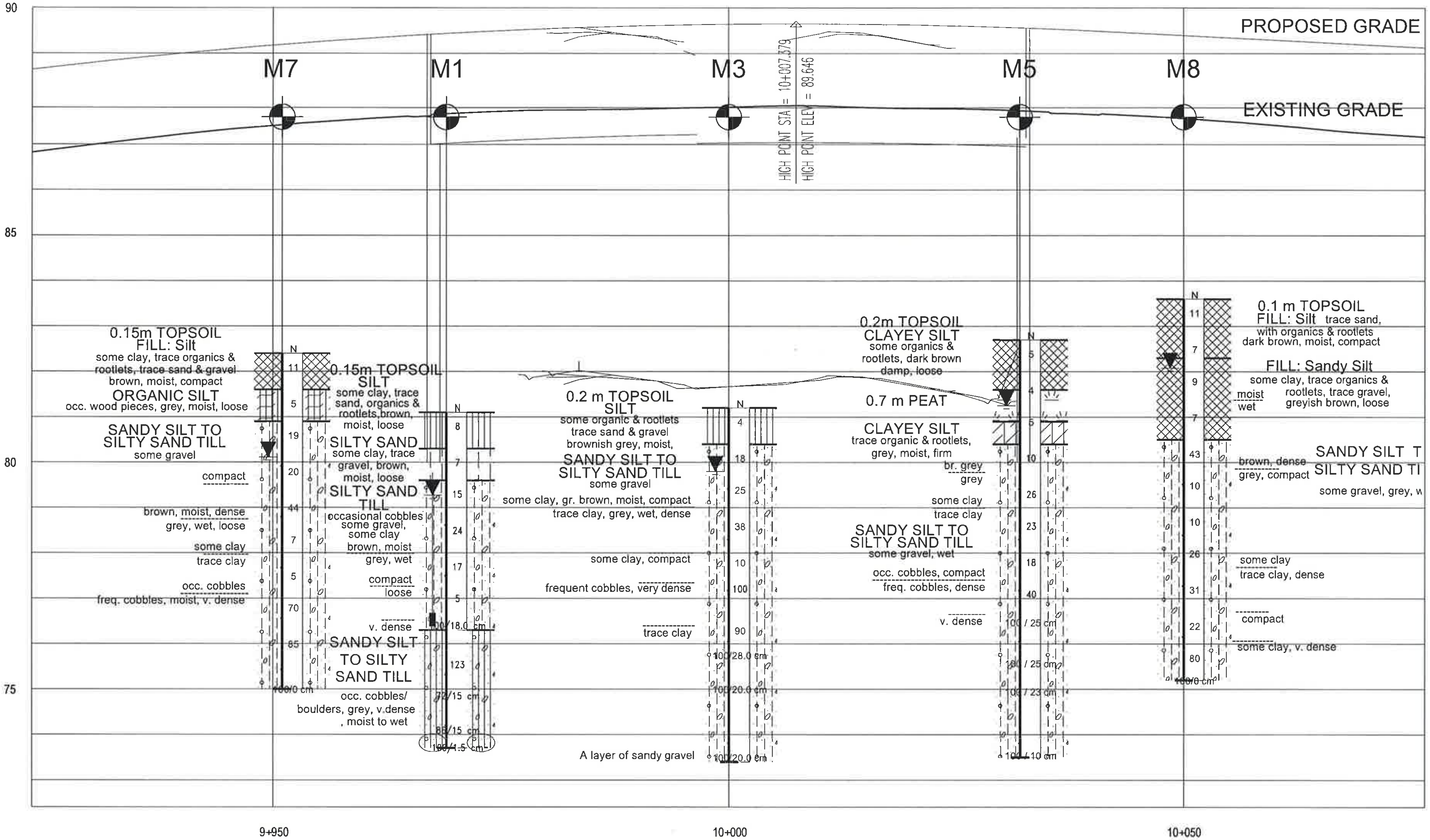
SPECIALISTS MANAGING THE EARTH



PROFILE

HORIZONTAL SCALE 5m 0 5 10m

KEY PLAN
N.T.S.

LEGEND

- Borehole & Cone
- N Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Water Level at Time of Investigation (W. L. NOT STABILIZED)
- Water Level in Piezometer
- Piezometer

No.	ELEV.	STATION No.	OFFSET
M1	81.1	9+969	5.5m Rt C/L
M3	81.2	10+000	6.0m Rt C/L
M5	82.7	10+032	5.5m Rt C/L
M7	82.4	9+951	6.0m Rt C/L
M8	83.6	10+050	3.0m Rt C/L

NOTE
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LICENSED PROFESSIONAL ENGINEER Aug 25/09 RAMON MIRANDA PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER Aug 25/09 Z. S. OZDEN PROVINCE OF ONTARIO

REV.	DATE	BY	DESCRIPTION

Geocres No. 31G-231

SPT 1227A			DIST
SUBM'D	CHECKED	DATE Mar. 2009	SITE
DRAWN PHK	CHECKED RM	APPROVED ZO	DWG 2

METRIC

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

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS REFER TO RECORD OF BOREHOLE SHEETS.

CONT No.

WP: 256-00-01

MOULINETTE ROAD AND HIGHWAY 401

PROFILE

SHEET

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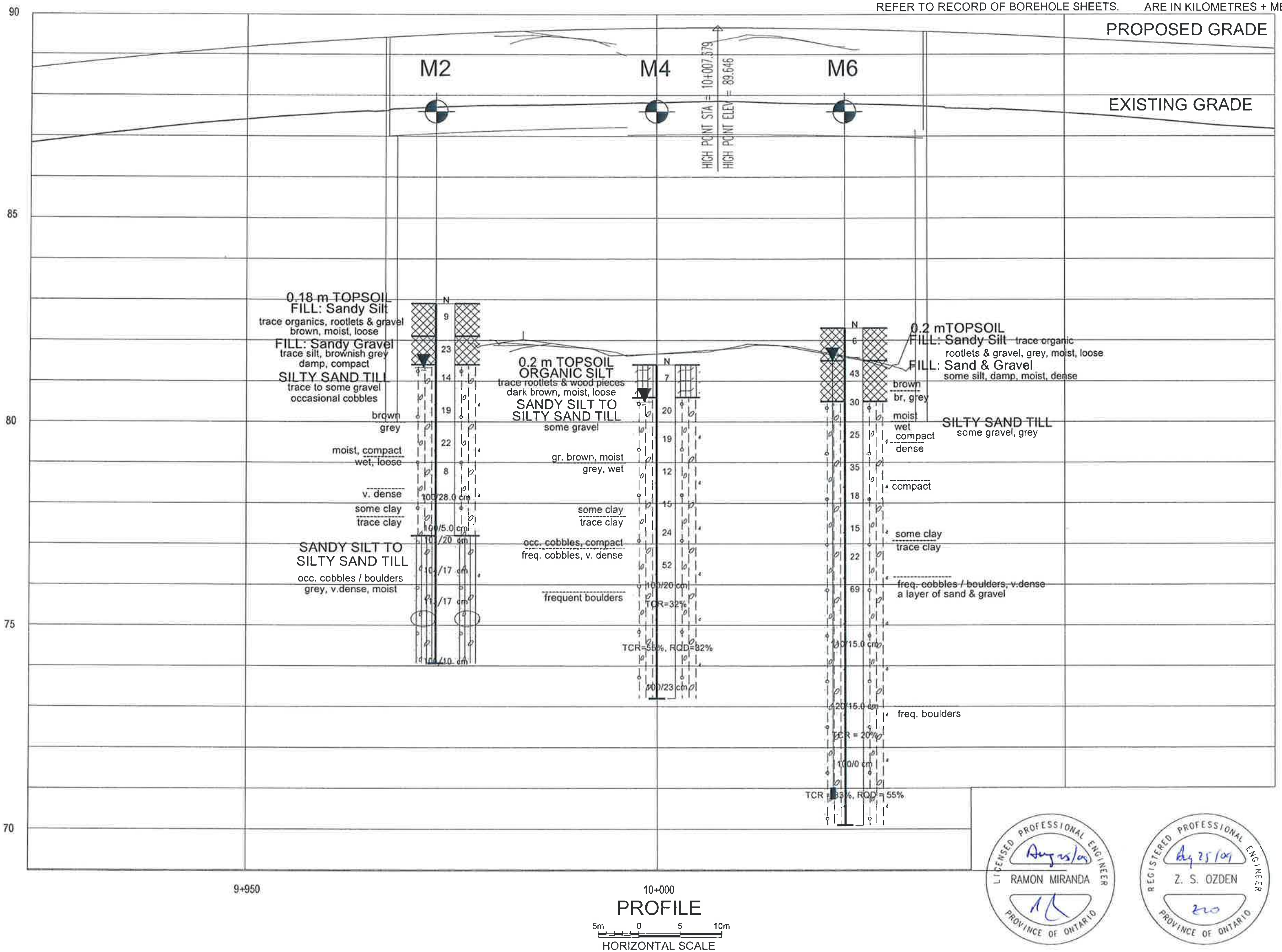

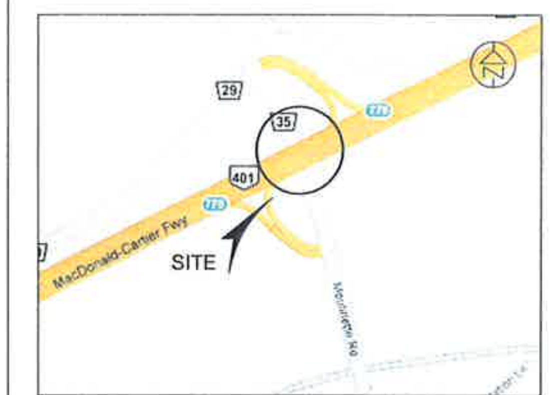

KEY PLAN

N.T.S.

LEGEND

- Borehole & Cone
- N — Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Water Level at Time of Investigation (W. L. NOT STABILIZED)
- Water Level in Piezometer
- Piezometer

No.	ELEV.	STATION No.	OFFSET
M2	82.9	9+973	6.0m Lt C/L
M4	81.4	10+000	6.0m Lt C/L
M6	82.3	10+023	6.5m Lt C/L

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LICENSED PROFESSIONAL ENGINEER
Aug 25/09
RAMON MIRANDA
PROVINCE OF ONTARIO

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Z. S. OZDEN
PROVINCE OF ONTARIO

REV.	DATE	BY	DESCRIPTION

Geocres No. 31G-231

SPT 1227A			DIST
SUBM'D	CHECKED	DATE Mar. 2009	SITE
DRAWN PHK	CHECKED RM	APPROVED ZO	DWG 3

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS REFER TO RECORD OF BOREHOLE SHEETS.

METRIC

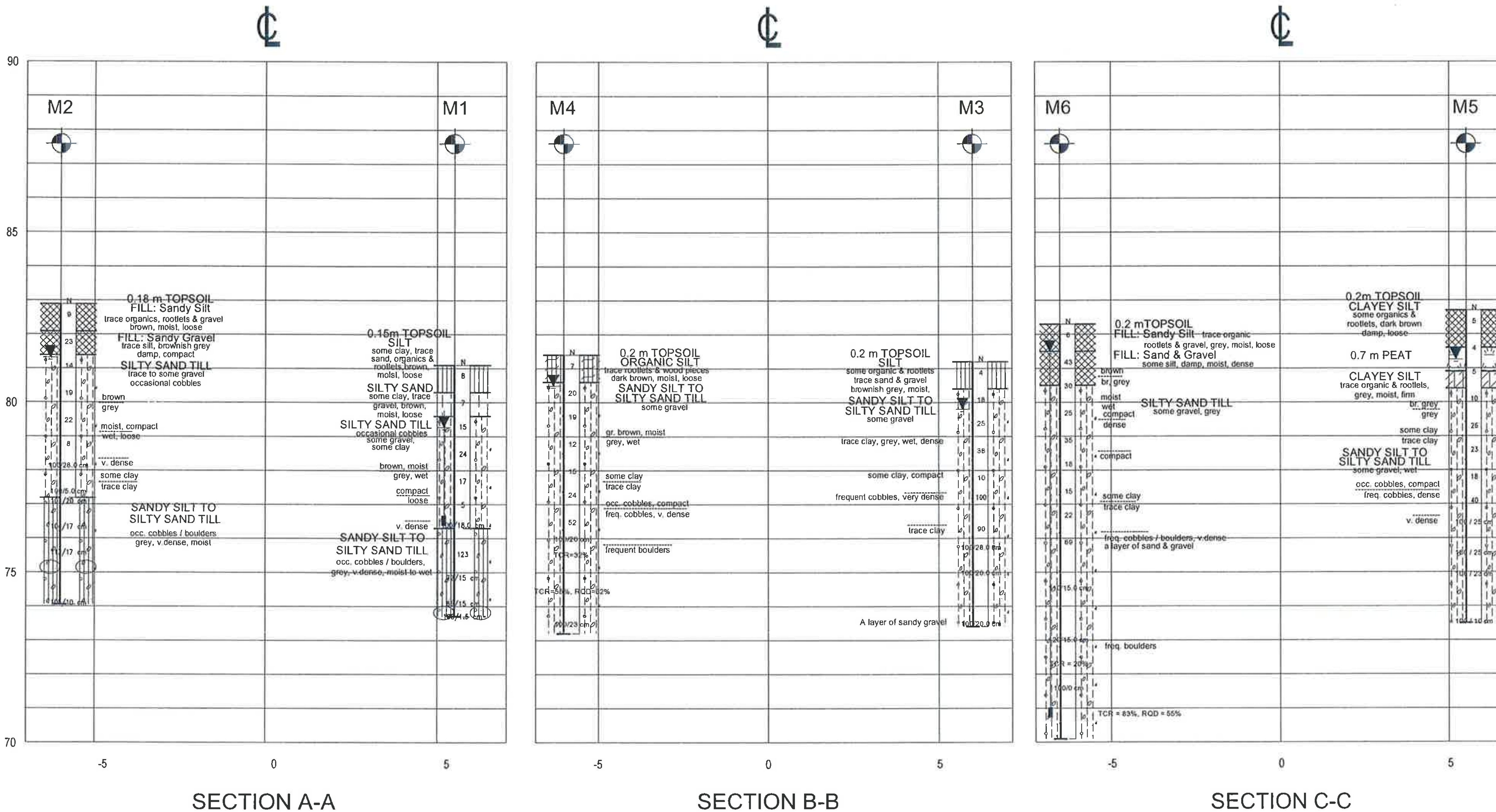
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES + METRES.

CONT No.
GWP: 256-00-01

SHEET

MOULINETTE ROAD AND HIGHWAY 401
SECTIONS

coffey geotechnics
SPECIALISTS MANAGING THE EARTH



SECTION A-A

SECTION B-B

SECTION C-C

SECTIONS

SCALE

KEY PLAN
N.T.S.

LEGEND

- Borehole & Cone
- N Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Water Level at Time of Investigation (W. L. NOT STABILIZED)
- Water Level in Piezometer
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No.	ELEV.	STATION No.	OFFSET
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NOTE

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

NOTE: This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

REV.	DATE	BY	DESCRIPTION

Geocres No. 31G-231

SPT 1227A			DIST
SUBM'D	CHECKED	DATE Mar. 2009	SITE
DRAWN PHK	CHECKED RM	APPROVED ZO	DWG 4

Appendix A

Record of Borehole Sheets

Ministry of Transportation Ontario

Foundation Design

SPT1227A

RECORD OF BOREHOLE No M1

1 OF 1 **METRIC**

GWP 256-00-00 LOCATION Sta: 9+969; 5.5 m Rt. C/L of Proposed Moulinette Rd ORIGINATED BY SK

DIST HWY HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS

DATUM Geodetic DATE 12/11/2008 CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
81.1	GROUND SURFACE				81		
0.0	0.15 m **TOPSOIL** **SILT** some clay trace sand, organics & rootlets brown, moist, loose	1	SS	8			
80.3 0.8	**SANDY SILT TO SILTY SAND TILL** occasional cobbles some gravel, trace clay loose	2	SS	7	80		L.L.= 12.7 ; P.L. = Non Plastic 11 44 33 12
	compac	3	SS	15	79		17 42 29 12
	brown, moist	4	SS	24	78		
	grey, wet compact	5	SS	17			
	loose	6	SS	5	77		
76.3 4.8	v. dense	7	SS	100/2.5 cm			

End of Borehole
Auger refusal @ 4.8 m on boulder
Hole advanced by moving 0.3 m south of original position to get through a boulder (See log M1A)
Water level @ 1.8 m (not stabilized)* upon completion
Hole caved-in @ 3.3 m upon completion
Piezometer installed to 4.7 m
Water level in Piezometer
Jan 22, 2009 0.9 m
Jan 28, 2009 0.9 m

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE; ● POCKET PENETR. X LAB VANE; 20 40 60 80 100. PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30. UNIT WEIGHT γ kN/m^3. STRAT PLOT. GROUND WATER CONDITIONS.

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

(%) STRAIN AT FAILURE

SPT1227A

RECORD OF BOREHOLE No M1A

METRIC

GWP 256-00-00 LOCATION Sta: 9+969.3 ; 5.5 m Rt. C/L of Proposed Moulinette Rd ORIGINATED BY SK

DIST HWY HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS

DATUM Geodetic DATE 2/24/2009 CHECKED BY ZO

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
81.1	GROUND SURFACE				81	
0.0	not sampled See Record of Borehole M1				80, 79, 78, 77	occ. auger grinding after 3 m
76.3 4.8	**SANDY SILT to SILTY SAND TILL** occ. cobbles/boulders grey, v.dense, moist to wet	8	SS	123	76, 75	boulder encoutered and augering took 20 min to get through
		9	SS	72/15 cm		
		10	SS	86/15 cm	74	
		11	SS	100/1.5 cm		
73.7 7.4	End of Borehole Auger refusal @ 7.4 m Water level @ 1.8 m (not stabilized)* and borehole caved-in @ 4.4 m upon completion					

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE; 20 40 60 80 100

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

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m^3

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

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

RECORD OF BOREHOLE No M2

METRIC

GWP 256-00-00 LOCATION Sta: 9+973 ; 6.0 m Lt. C/L of Proposed Moulinette Rd ORIGINATED BY SK

DIST HWY HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS

DATUM Geodetic DATE 12/10/2008 CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
82.9	GROUND SURFACE						
0.0	0.2 m **TOPSOIL** **FILL:** Sandy Silt trace organics, rootlets & gravel brown, moist, loose	1	SS	9			
82.1 / 0.8	**FILL:** Sandy Gravel trace silt, brownish grey damp, compact	2	SS	23		82	
81.4 / 1.5	**SILTY SAND TILL** trace to some gravel occasional cobbles	3	SS	14	▼*	81	24 38 27 11
	brown	4	SS	19		80	
	grey; moist, compact	5	SS	22			17 40 31 12
	wet, loose	6	SS	8		79	
	some clay; trace clay, dense	7	SS	58/12 cm		78	27 37 26 10
77.2 / 5.7		8	SS	148/20 cm			

End of Borehole
Auger refusal @ 5.7 m on boulder
Hole advanced by moving 0.5 m west of original position to get through a boulder (See log M2A)
Water level @ 1.6 m (not stabilized)* upon completion
Hole caved-in @ 3.6 m upon completion

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● POCKET PENETR., × LAB VANE (20 40 60 80 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³

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

RECORD OF BOREHOLE No M2A

METRIC

GWP 256-00-00		LOCATION Sta: 9+973 ; 5.5 m Lt. C/L of Proposed Moulinette Rd	ORIGINATED BY SK
DIST	HWY HWY 401	BOREHOLE TYPE Hollow Stem Augers	COMPILED BY SS
DATUM Geodetic		DATE 2/24/2009	CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
82.9	GROUND SURFACE							
0.0	not sampled See Record of Borehole M2				▽	82, 81, 80, 79, 78, 77		augering slow below 4.5 m boulder @ 5.8 m encountered and augering took 15 min to get through
76.8 6.1	**SANDY SILT to SILTY SAND TILL** occ. cobbles/boulders grey, v.dense, moist	9	SS	107/20 cm			o	
		10	SS	104/17 cm		76	o	
		11	SS	113/17 cm		75, 74	o	boulder encountered and augering took 20 min to get through
73.7 9.2		12	SS	100/10 cm			o	
	End of Borehole Water level @ 1.5 m (not stabilized)* and borehole caved-in @ 4.6 m upon completion							

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● POCKET PENETR., X LAB VANE (20 40 60 80 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; STRAT PLOT; UNIT WEIGHT γ kN/m³

+³, ×³: Numbers refer to Sensitivity   20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

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RECORD OF BOREHOLE No M3

METRIC

GWP 256-00-00 LOCATION Sta: 10+000 ; 6.0 m Rt. C/L of Proposed Moulinette Rd ORIGINATED BY SK

DIST HWY HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS

DATUM Geodetic DATE 12/19/2008 CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	GR	SA	SI	CL
81.2	GROUND SURFACE								
0.0	0.2 m **TOPSOIL** **SILT** some organic &rootlets trace sand & gravel brownish grey, moist, loose	1	SS	4	81				
80.4 / 0.8	**SANDY SILT TO SILTY SAND TILL** some gravel, trace clay	2	SS	18	80				
		3	SS	25					
	greyish brown, moist, compact				79				
	grey, wet, dense	4	SS	38		27	35	26	12
	compact	5	SS	10	78				
	frequent cobbles, very dense	6	SS	92/23 cm	77				
		7	SS	90		16	46	31	7
		8	SS	113	76				
		9	SS	100/20 cm	75				
					74				
73.4 / 7.8	A layer of sandy gravel	10	SS	100/20 cm					

End of Borehole
Auger refusal @ 7.8 m probably on boulder
Hole caved-in @ 4.3 m upon completion
Water level @ 1.4 m (not stabilized)* upon completion

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● POCKET PENETR., X LAB VANE, 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%)

+ 3, × 3: Numbers refer to Sensitivity

20 / 15–5 / 10: (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No M4

METRIC

GWP 256-00-00 LOCATION Sta: 10+000 ; 6.0 m Lt. C/L of Proposed Moulinette Rd ORIGINATED BY SK

DIST HWY HWY 401 BOREHOLE TYPE Hollow Stem Augering, Wash Boring & NQ Coring COMPILED BY SS

DATUM Geodetic DATE 12/18/2008 12/19/2008 CHECKED BY ZO

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	GR	SA	SI	CL
81.4	GROUND SURFACE									
0.0	0.2 m TOPSOIL ORGANIC SILT trace rootlets & wood pieces dark brown, moist, loose	1	SS	7		81				
80.6 / 0.8	SANDY SILT TO SILTY SAND TILL some gravel, trace clay	2	SS	20	▼*	80				
	gr. brown, moist	3	SS	19			10	42	36	12
	grey, wet	4	SS	12		79				
		5	SS	15		78	26	36	27	11
	occ. cobbles, compact	6	SS	24		77				
	freq. cobbles, v. dense	7	SS	52			23	39	30	8
		8	SS	100/20 cm		76				
	frequent boulders	9	RC	TCR=32%		75				
		10	RC	TCR=55% RQD=32%		74				
73.2		11	SS	100/23 cm						
8.2	End of Borehole Auger refusal @ 5.5 m and hole advanced by NW Casing and Wash Boring Hole caved-in @ 4.0 m upon completion Water level @ 0.9 m (not stabilized)* upon completion									

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE — 20 40 60 80 100

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

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m^3

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

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

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

RECORD OF BOREHOLE No M5

METRIC

GWP 256-00-00 LOCATION Sta: 10+032 ; 5.5 m Rt. C/L of Proposed Moulinette Rd ORIGINATED BY SK

DIST HWY HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS

DATUM Geodetic DATE 12/17/2008 1/6/2009 CHECKED BY ZO

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
82.7	GROUND SURFACE					
0.0	0.2 m TOPSOIL ORGANIC CLAYEY SILT some rootlets, dark brown damp, firm	1	SS	5		
81.6 1.1	PEAT dark brown/black, soft	2	SS	4	▼ *	
80.9 1.8	CLAYEY SILT trace organic & rootlets grey, moist, firm	3	SS	5		
80.4 2.3	SANDY SILT TO SILTY SAND TILL some gravel, trace clay, wet brown to grey	4	SS	10		13 44 30 13
	grey	5	SS	26		
		6	SS	23		
	occ. cobbles, compact	7	SS	18		6 48 33 13
	freq. cobbles, dense	8	SS	40		
	v. dense	9	SS	100 / 25 cm		
		10	SS	100 / 25 cm		
		11	SS	100 / 23 cm		
73.5 9.2		12	SS	100 / 10 cm		

Elevation scale: 82, 81, 80, 79, 78, 77, 76, 75, 74

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE: 20 40 60 80 100

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

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m³

56.2

209.5

238.9

End of Borehole
Water level @ 1.4 m (not stabilized)* upon completion
Hole caved-in @ 3.5 m upon completion

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

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

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RECORD OF BOREHOLE No M6

METRIC

GWP	256-00-00
DIST	
HWY	HWY 401
DATUM	Geodetic
LOCATION	Sta: 10+023.5 ; 6.5 m Lt. C/L of Proposed Moulinette Rd
BOREHOLE TYPE	Hollow Stem Augering, Wash Boring & NQ Coring
DATE	12/23/2008 12/24/2008
ORIGINATED BY	SK
COMPILED BY	SS
CHECKED BY	ZO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
82.3	GROUND SURFACE				
0.0	0.2 m TOPSOIL FILL: Sandy Silt trace organic, rootlets & gravel grey, moist, loose	1	SS	6	
81.5 0.8	FILL: Sand & Gravel some silt, damp, moist, dense brown	2	SS	43	40 40 (20)
	br, grey	3	SS	30	
80.5 1.8	SANDY SILT TO SILTY SAND TILL some gravel, trace clay, grey moist wet compact	4	SS	25	
	dense	5	SS	35	13 46 32 9
	compact	6	SS	18	
		7	SS	15	
		8	SS	22	
	freq. cobbles / boulders, v.dense a layer of sand & gravel	9	SS	69	28 53 16 3
		10	SS	110/15 c	
	freq. boulders	11	SS	120/15 c	
		12	RC	TCR = 20%	
		~~13~~	~~SS~~	~~100/8 cm~~	No Recovery
		14	RC	TCR = 83% RQD = 55%	
70.1 12.2	End of Borehole				

Elevation scale: 82, 81, 80, 79, 78, 77, 76, 75, 74, 73, 72, 71

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m^3

End of Borehole
Auger refusal @ 9.2 m and hole advanced by NW Casing and Wash Boring
Hole caved-in @ 4.1 m upon completion
Water level @ 0.8 m (not stabilized)* upon completion
Piezometer installed to 11.6 m
Water Level in Piezometer
Jan 22, 2009 1.4 m
Jan 28, 2009 1.4 m

$+^3, \times^3$: Numbers refer to Sensitivity; 20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

SPT1227A

Foundation Design

RECORD OF BOREHOLE No M7

1 OF 1 **METRIC**

GWP 256-00-00 LOCATION Sta: 9+951 ; 6.0 m Rt. C/L of Proposed Moulinette Rd ORIGINATED BY SK

DIST HWY HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS

DATUM Geodetic DATE 12/10/2008 CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE
82.4	GROUND SURFACE					
0.0	0.15 m TOPSOIL FILL: Silt, some clay trace organics & rootlets trace sand & gravel brown, moist, compact	1	SS	11		82
81.6 0.8	ORGANIC SILT occ. wood pieces grey, moist, loose	2	SS	5		81
80.9 1.5	SANDY SILT TO SILTY SAND TILL some gravel, trace clay	3	SS	19	▼*	
	compact	4	SS	20		80
	brown, moist, dense	5	SS	44		79
	grey, wet, loose	6	SS	7		78
	occ. cobbles	7	SS	5		
	freq. cobbles, moist, v. dense	8	SS	70		77
		9	SS	85		76
75.0 7.4	End of Borehole Auger refusal @ 7.4 m probably on boulder Hole caved-in @ 3.4 m upon completion Water level @ 2.3 m (not stabilized)* upon completion	~~10~~	~~SS~~	~~100/0 cm~~		

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE 20 40 60 80 100

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

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

+³, ×³: Numbers refer to Sensitivity 15-⊕-5 (20, 10) (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

SPT1227A

RECORD OF BOREHOLE No M8

METRIC

GWP 256-00-00 LOCATION Sta: 10+050 ; 3.0 m Rt. C/L of Proposed Moulinette Rd ORIGINATED BY SK

DIST HWY HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS

DATUM Geodetic DATE 1/5/2009 CHECKED BY ZO

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS
83.6	GROUND SURFACE				
0.0	0.1 m TOPSOIL FILL: Silt, trace sand with organics & rootlets dark brown, moist, compact	1	SS	11	
		2	SS	7	
82.3 1.3	FILL: Sandy Silt, some clay trace organics & rootlets trace gravel, greyish brown, loose	3	SS	9	▼*
	moist / wet	4	SS	7	
80.5 3.1	SANDY SILT TO SILTY SAND TILL some gravel, trace clay, grey, wet brown, dense	5	SS	43	
	grey, compact	6	SS	10	
		7	SS	10	
		8	SS	26	
	trace clay, dense	9	SS	31	
	compact	10	SS	22	
	very dense	11	SS	80	
75.2 8.4		12	SS	100/0 cm	

End of Borehole
Auger refusal @ 8.4 m probably on boulder
Hole caved-in @ 4.8 m upon completion
Water level @ 1.5 m upon completion (not stabilized)*

ELEVATION SCALE: 83, 82, 81, 80, 79, 78, 77, 76

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE — 20 40 60 80 100

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

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m³

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

$+^3, \times^3$: Numbers refer to Sensitivity — 20 / 15 ◆ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

SPT1227A

Foundation Design

RECORD OF BOREHOLE No M9

METRIC

GWP 256-00-00 LOCATION Sta: 9+948 ; 11.8 m Lt. C/L of Proposed Moulinette Rd ORIGINATED BY SK

DIST HWY HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS

DATUM Geodetic DATE 2/25/2009 2/25/2009 CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS
88.3	GROUND SURFACE				
0.0	0.20 m Asphaltic Concrete **GRANULAR PAVEMENT FILL** sand & gravel brown, damp	1	AS		
		2	SS	138	N-value not reliable due to frost
87.1 / 1.2	**SANDY SILT to SILTY SAND (EMBANKMENT FILL)** trace to some gravel brown, compact, damp to moist	3	SS	22	
		4	SS	16	
		5	SS	14	
		6	SS	19	
		7	SS	15	
	v. loose moist to wet	8	SS	4	
81.9 / 6.4	**SAND AND GRAVEL (FILL)** brown, compact, moist	9	SS	22	
81.5 / 6.8	**TOPSOIL & ORG. SILT** dk. brown/black				
81.3 / 7.0	**SANDY SILT to SILT** brown, compact, moist	10	SS	11	
80.9 / 7.4	**SANDY SILT to SILTY SAND TILL** compact, moist	11	SS	21	
	brown / grey	12	SS	24	
78.6 / 9.8	End of Borehole Borehole dry (not stabilized)* upon completion Hole caved-in @ 7.3 m upon completion				

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (88, 87, 86, 85, 84, 83, 82, 81, 80, 79); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa O UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

RECORD OF BOREHOLE No M10

1 OF 1 **METRIC**

GWP 256-00-00 LOCATION Sta: 10+052 ; 11.8 m Lt. C/L of Proposed Moulinette Rd ORIGINATED BY SK

DIST HWY HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS

DATUM Geodetic DATE 2/25/2009 2/25/2009 CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
88.6	GROUND SURFACE				
0.0	0.20 m **Asphaltic Concrete** **GRANULAR PAVEMENT FILL** sand & gravel brown, damp	1	AS		
		2	SS	122	N-value not reliable due to frost
87.4 / 1.2	**GRAVELLY SAND** (possible trench backfill) brown, dense, damp	3	SS	32	
		4	SS	41	
85.7 / 2.9	**SANDY SILT to SILTY SAND (EMBANKMENT FILL)** trace to some gravel brown/grey, compact, damp to moist	5	SS	30	
		6	SS	14	
		7	SS	17	
		8	SS	14	
82.4 / 6.2	**GRAVELLY SAND (EMBANKMENT FILL)** greyish brown, dense, damp	9	SS	37	
81.7 / 6.9	**GRAVELLY SAND** darkish grey occ. sea shell remains, compact wet (possible fill)	10	SS	24	
		11	SS	13	
80.6 / 8.0	**SANDY SILT**, tr. gravel, tr. org.				
80.4 / 8.2	**SANDY SILT to SILTY SAND TILL** grey, moist to wet; compact / loose	12	SS	19	
		13	SS	8	
		14	SS	8	
77.3 / 11.3	End of Borehole Borehole dry upon completion (not stabilized)* Hole caved-in @ 5.5 m upon completion				

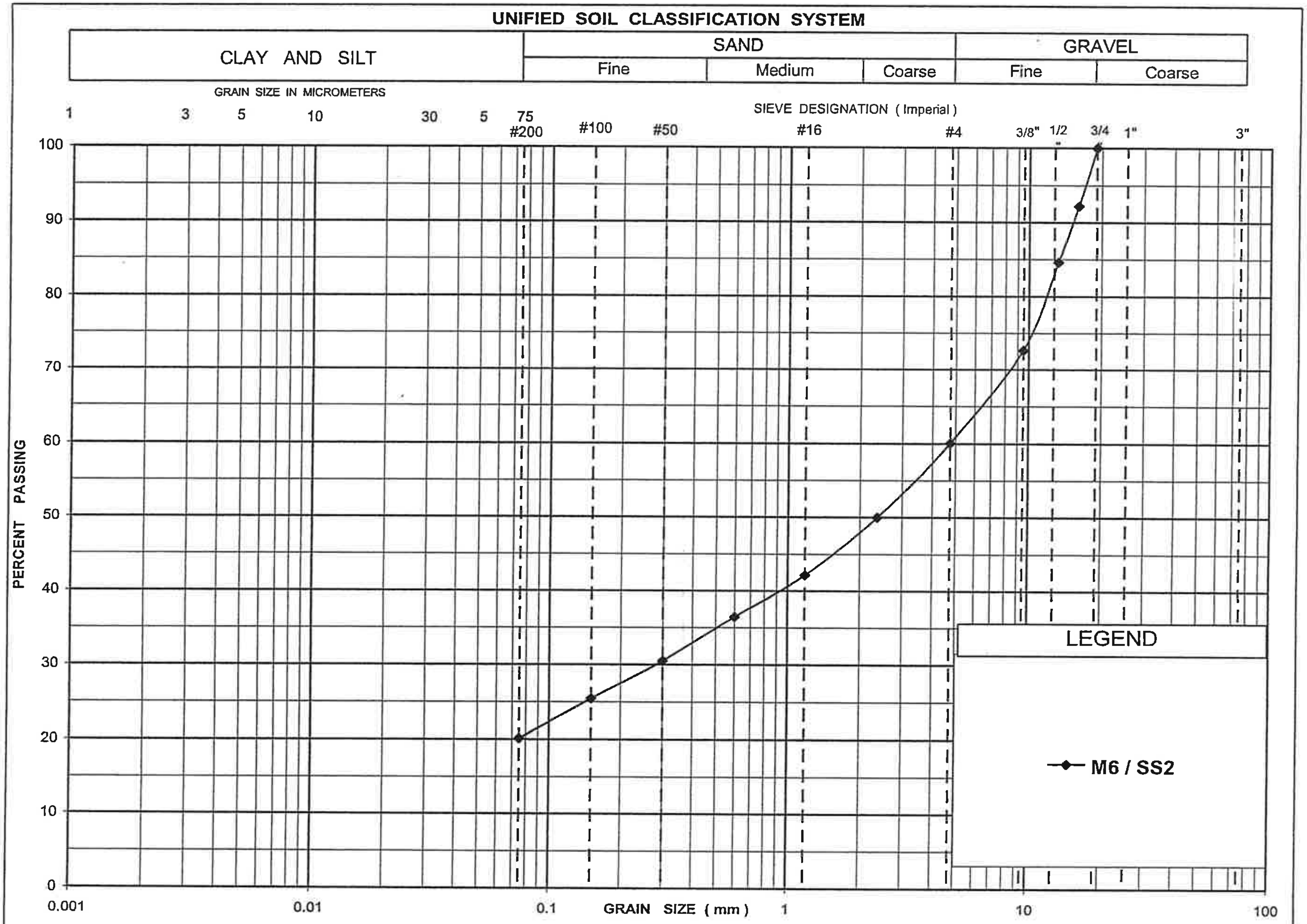
Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (88, 87, 86, 85, 84, 83, 82, 81, 80, 79, 78); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³

+ ³, × ³ : Numbers refer to Sensitivity

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

Appendix B

Laboratory Test Results

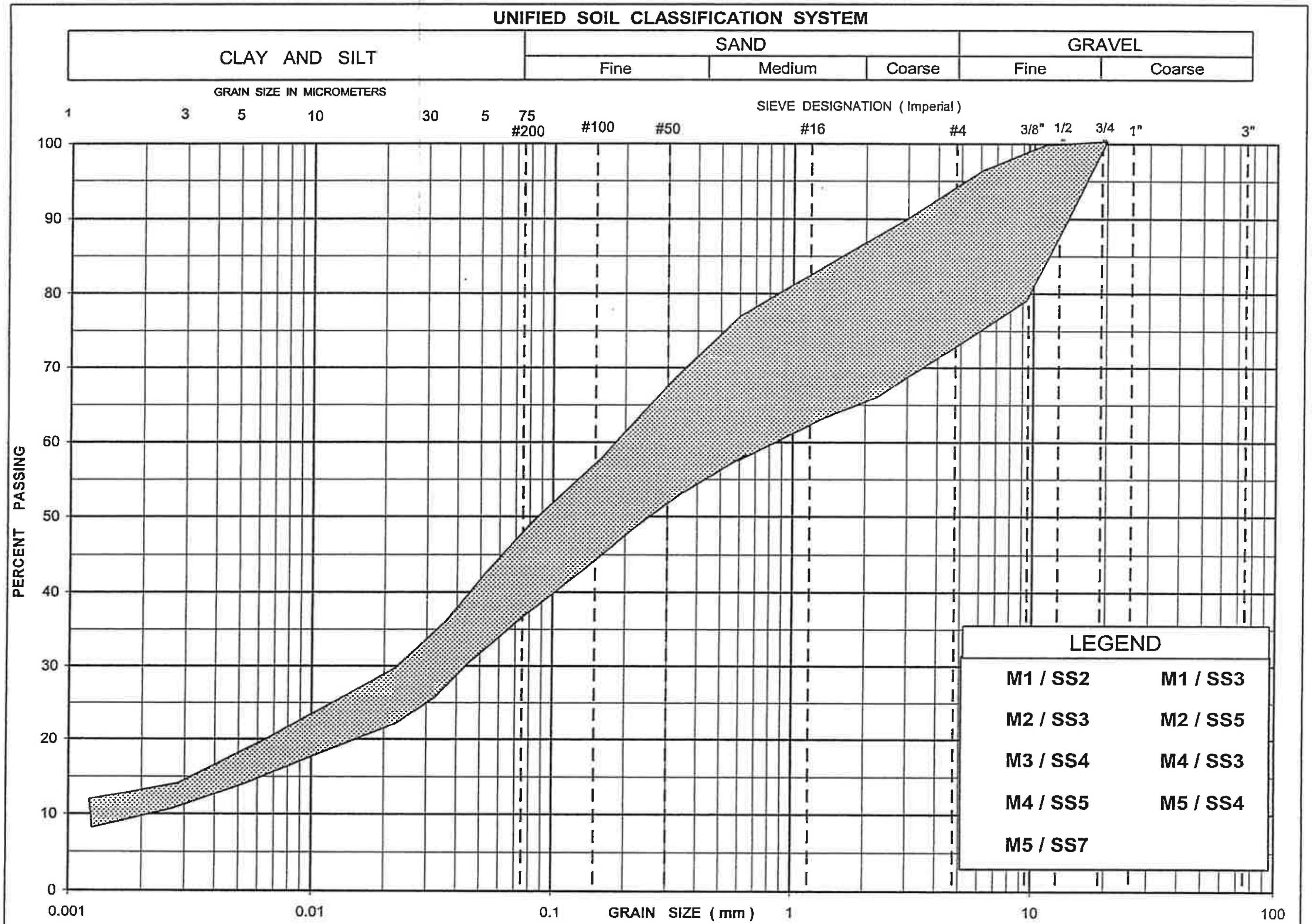


coffey geotechnics
SPECIALISTS MANAGING THE EARTH

GRAIN SIZE DISTRIBUTION

GRANULAR FILL: Sand & Gravel, some silt

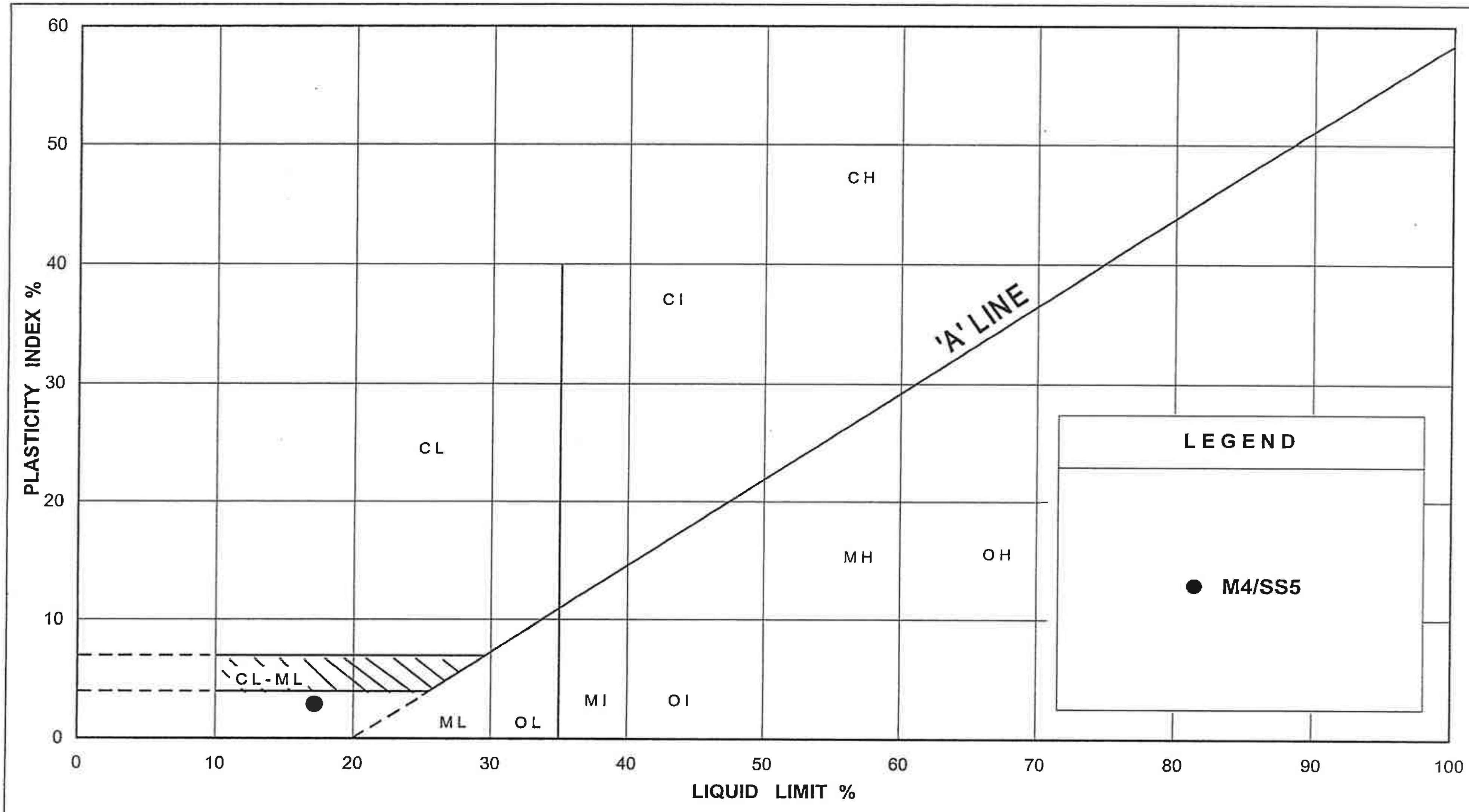
FIGURE No.	B-1
REF. No.	SPT 1227A
DATE	February 2009



coffey geotechnics
SPECIALISTS MANAGING THE EARTH

GRAIN SIZE DISTRIBUTION
SANDY SILT to SILTY SAND TILL

FIGURE No.	B-2
REF. No.	SPT 1227A
DATE	January 2009



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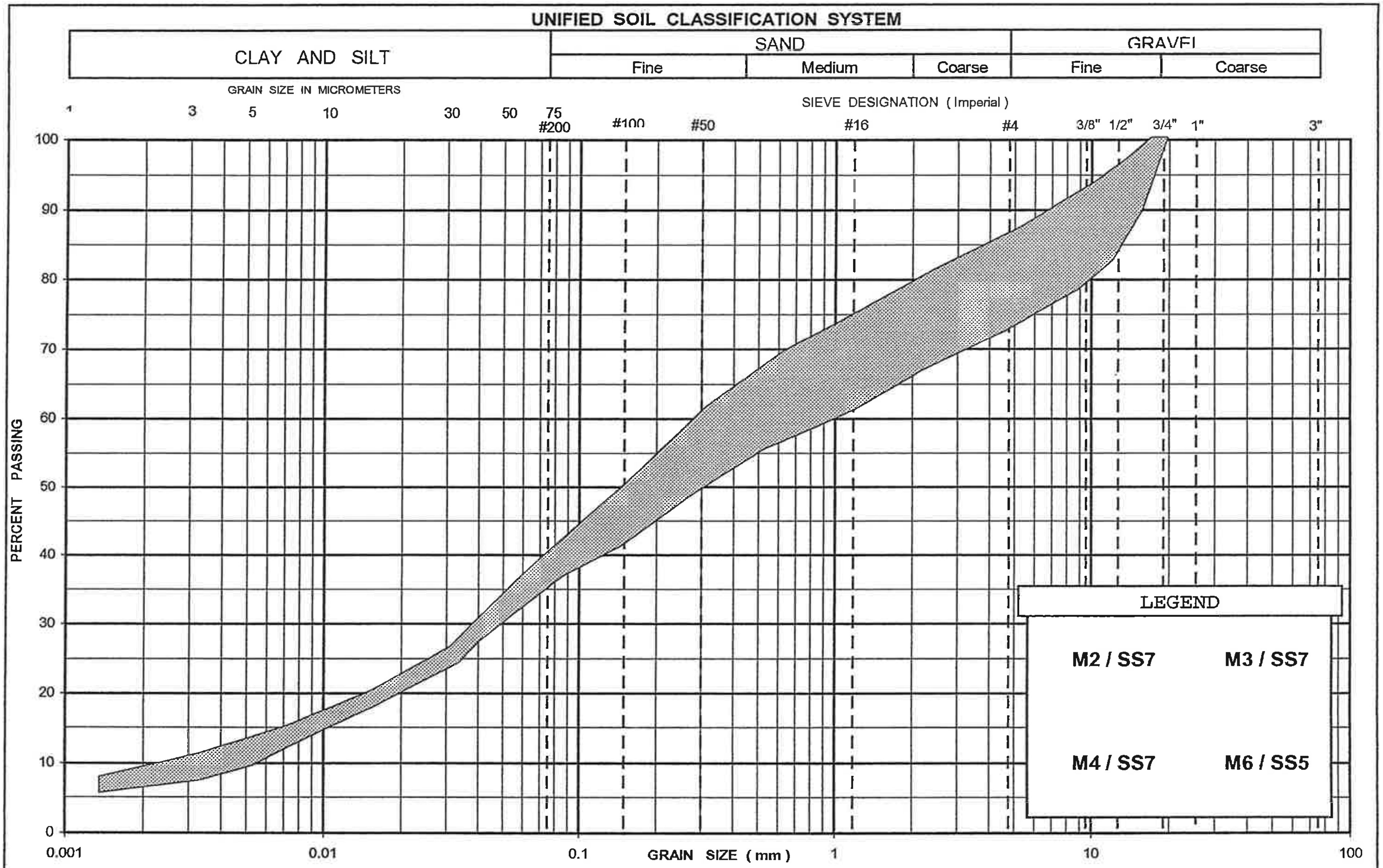
PLASTICITY CHART

SANDY SILT to SILTY SAND TILL

FIGURE No.	B-3
REF. No.	SPT 1227A
DATE	FEBRUARY 2009



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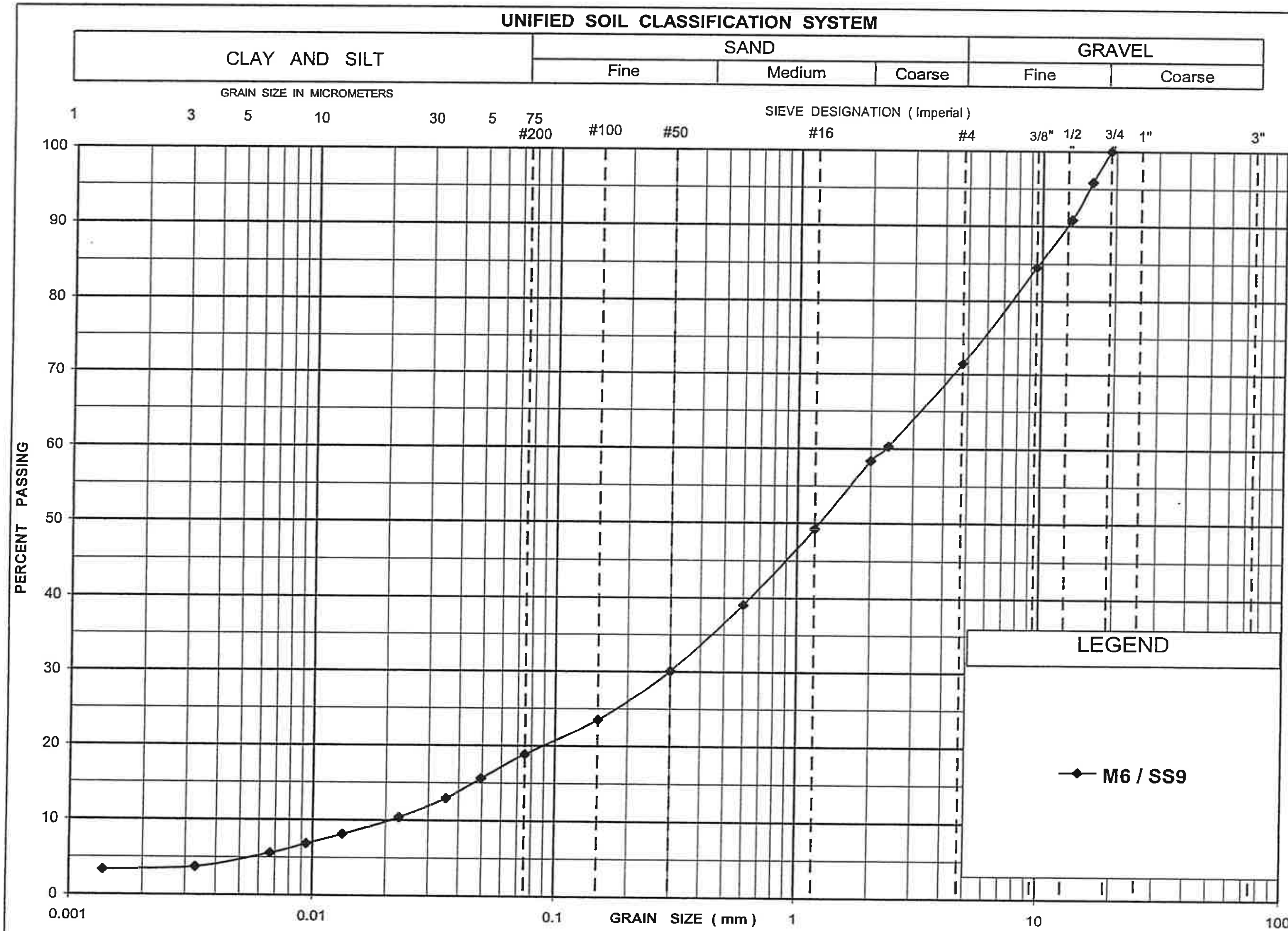
GRAIN SIZE DISTRIBUTION SILTY SAND, some gravel, trace clay	FIGURE No. B-4
	REF. No. SPT 1227A
	DATE FEBRUARY 2009



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GRAIN SIZE DISTRIBUTION

SAND & GRAVEL INTERBED in Till deposit

FIGURE No. B-5	
REF. No.	SPT 1227A
DATE	February 2009

Appendix C

Site Photographs

Photo (01): Moulinette Bridge North of Highway 401 High Fill (New Alignment), Looking towards South

Photo (02): Moulinette Bridge North of Highway 401 High Fill (New Alignment), Looking towards South

Photo (03): North Pier of Moulinette Bridge, Looking towards West

Photo (04): North Abutment of Moulinette Bridge, Looking towards North

Photo (05): East Side (New Alignment) Moulinette Bridge at Highway 401, Looking towards South

Photo (06): Moulinette Bridge at Highway 401, Looking towards South

Photo (07): West Side of Moulinette Bridge at Highway 401, Looking towards South

Photo (08): Moulinette Bridge North of Highway 401, Looking towards South

Photo (09): Moulinette Bridge South of Highway 401, Looking towards South

Photo (10): Moulinette Bridge South of Highway 401, Looking towards North

Appendix D

Explanation of Terms Used in Report

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 $\overline{N}$.

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475J 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 COMPOSION AND STRUCUTRAL 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*

JOINT 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	TW ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	TW ADVANCED MANUALLY
TW	THINWALL OPEN	FS	FOIL SAMPLE

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
$\sigma_l, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ε	%	LINEAR STRAIN
$\varepsilon_1, \varepsilon_2, \varepsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICALL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
c_c	1	COMPRESSION INDEX
c_s	1	SWELLING INDEX
c_a	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'	$-^o$	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	$-^o$	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = c_u / τ_r

PHYSICAL PROPERTIES OF SOIL

P_s	kg/m^3	DENSITY OF SOLID PARTICLES	e	1,%	VOID RATIO	e_{min}	1,%	VOID RATIO IN DENSEST STATE
i_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES	n	1,%	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
P_w	kg/m^3	DENSITY OF WATER	w	1,%	WATER CONTENT	D	mm	GRAIN DIAMETER
i_w	kN/m^3	UNIT WEIGHT OF WATER	s_r	%	DEGREE OF SATURATION	D_n	mm	N PERCENT – DIAMETER
P	kg/m^3	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
i	kN/m^3	UNIT WEIGHT OF SOIL	w_P	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
P_d	kg/m^3	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
i_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_P	%	PLASTICITY INDEX = (W_L – W_L)	v	m/s	DISCHARGE VELOCITY
P_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = (W – W_P)/ I_P	i	1	HYDAULIC GRADIENT
i_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = (W_L – W) / 1_P	k	m/s	HYDRAULIC CONDUCTIVITY
P'	kg/m^3	DENSITY OF SUBMERED SOIL	e_{max}	1,%	VOID RATIO IN LOOSEST STATE	j	kN/m^3	SEEPAGE FORCE
i'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

FOUNDATION DESIGN REPORT HIGHWAY 401/MOULINETTE ROAD BRIDGE, CITY OF CORNWALL, ONTARIO G.W.P. 256-00-01, SITE # 31-163, GEOCRES NO. 31G-231

AECOM

Project: SPT1227A
August 31, 2009

Coffey Geotechnics Inc.
20 Meteor Drive Etobicoke Ontario M9W 1A4 Canada

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FOUNDATION DESIGN REPORT
HIGHWAY 401/MOULINETTE ROAD BRIDGE
CITYOF CORNWALL, ONTARIO
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5 DISCUSSION AND RECOMMENDATIONS

The existing bridge, which carries Moulinette Road over Highway 401 near Cornwall, is a two-lane, four-span structure, with a total length of about 70 m. We understand that the existing bridge is a circa 1960's structure supported on driven steel 12BP53 (HP310x79) H-piles. The piles appear to have been designed for loads of 40 to 50 tons/pile or about 356 to 445 kN/pile. The existing structure will be replaced with a two-span bridge of similar total length and width. The anticipated height of the new structure is about 8 m over the existing Highway 401 elevation. We understand that the new bridge will be located west (about 16 m centre to centre)of the existing bridge and that the clear distance between the two structures will be 3 m. The existing bridge will carry the Moulinette Road traffic during the construction of the new structure.

A total of ten boreholes were put down at the site, which showed, in general, beneath some fill and/or topsoil, the presence of 0.2 to 0.6 m thick surficial silt and clayey silt in most of the boreholes, as well the presence of surficial organic soils in two of the boreholes (i.e. Boreholes M4 and M5). These surficial soils are underlain by a major glacial deposit, consisting of sandy silt to silty sand till to the termination of the boreholes at El. 78.6 and 70.1 m. One of the boreholes drilled by MTO at the site of existing bridge site indicates the presence of bedrock at El. 230.0 ft (approximately 70.1 m). From this and the refusal elevations in the present boreholes and the presence of rock fragments in the lower zones, the surface of the bedrock at the site may be at about El. 70 m. The groundwater table at the time of our investigation was generally recorded at about 1 m below the o.g. levels, but would be subject to fluctuations.

5.1 Foundations

5.1.1 Abutments

The bridge incorporates an integral abutment design and thus the use of driven H-piles is the preferred foundation for abutment support.

The use of spread footings was looked into but the foundations will need to extend to considerable depths below the groundwater table. As well, deep excavations are not desirable immediately adjacent to the existing bridge. For these reasons spread footing foundations are not considered to be a good alternative. In addition, because of the proximity to the existing bridge, supporting the foundation elements on perched granular pad is considered impractical for this project.

The borehole data (i.e. Boreholes M1, M2, M5, M6, M9 and M10) show that the use of driven steel piles is suitable to support the abutments.

The advantages and disadvantages of various foundation support types at the abutment locations are summarized in Appendix E.

The following paragraphs present a further discussion on these options.

5.1.1.1 Spread Footing Foundations on Natural Soil

If necessary, the abutments can be supported footing foundations placed on the undisturbed dense to very dense glacial till. The following table summarizes the recommended resistances and footing depths.

5.1.1.1 Spread Footing Foundations for Abutments

Location	Applicable Boreholes	Recommended Footing Elevation (Bottom of Footing) (m)	Recommended SLS (kPa)	Recommended ULS (kPa)	Subgrade Soils
North Abutment	M1, M1A, M2 & M2A	76.5	500	800	Very dense sandy silt to silty sand till
South Abutment	M5 & M6	76.4	500	800	Dense to very dense sandy silt to silty sand till

The factored bearing resistance at ULS given in the above table incorporates a resistance factor of 0.5 as per Canadian Highway Bridge Design Code (CHBDC) S6-06. The serviceability condition is based on the premise that the maximum total and differential settlements will not exceed 25 mm and 20 mm, respectively. This can be achieved provided that the founding subgrade is undisturbed during the construction.

Under inclined loading conditions, the bearing resistance at ULS should be reduced in accordance with CHBDC S6-06.

Allowance should be made to place a 120 mm thick concrete mud mat (i.e. skim coat) in all footing excavations as soon as possible (not more than four hours) after excavation. All footing excavations should be inspected and approved by the geotechnical engineer prior to pouring the concrete mud mat.

For frost protection, the foundations should have a permanent earth cover of at least 1.7 m or equivalent artificial insulation.

As can be seen from above table, deep excavations will be required, which may extend below the water table. These will necessitate shoring and probably dewatering. In addition, the excavations can be expected to extend to variable depths. As well, spread footing foundations are not suited for the support of integral abutments. For these reasons the use of spread footing foundations is not recommended, including the use of foundations supported on engineered granular pad.

5.1.1.2 Drilled Caisson Foundations

The use of augered and cast-in-place concrete foundations (drilled caissons) is feasible but is not recommended for the support of the abutments.

Caissons extended at least 1.2 m into the very dense glacial till can be designed for a geotechnical resistance of 2000 kPa at SLS and 3000 kPa at ULS. These values include both side friction and end bearing contributions and are applicable for most commonly used sizes in Ontario (i.e. 0.76 to 1.8 m diameter). The following table summarizes the anticipated caisson bottom elevations at the borehole locations.

Table 5.1.1.2.1: Recommended Caisson Resistances

Area Reference/ Borehole No.	Existing Ground Surface Elevation (m)	Recommended Caisson Bottom Depth/Elevation (m)	Recommended SLS (kPa)	Recommended ULS (kPa)	Subgrade Soil
North Abutment/ M1 & M1A	81.1	6.1/75.0	2000	3000	v. dense sandy silt to silty sand till
North Abutment/M2 & M2A	82.9	6.1/76.8	2000	3000	v. dense sandy silt to silty sand till
South Abutment/M6	82.3	7.6/74.7	2000	3000	v. dense sandy silt to silty sand till
South Abutment/M5	82.7	7.0/75.7	2000	3000	v. dense sandy silt to silty sand till

*Below existing ground surface

The anticipated caisson depths/elevations at the borehole locations, as given in the table above can be used for design purposes, with interpolation in between and beyond the borehole locations. Actual caisson depths in the field would be decided during their installation, ensuring at least 1.2 m socket into very dense, undisturbed till.

As caisson foundations are not well-suited for the support of the abutments, they will not be discussed any further in this section. However, a more detailed discussion on their use at this project site is given in Section 5.1.2 of this report.

5.1.1.3 Steel H-piles

Driven steel-H-piles is the recommended option for the support of the proposed bridge abutments. In an event, the design of the new bridge entails the use of driven H-piles to support the abutments, since integral abutments are to be implemented. The borehole data indicate that the geotechnical conditions are suitable for the use of driven steel H-piles at the abutment locations. Steel H-piles are preferable to other types of driven piles, such as precast concrete piles, steel tube piles, etc, since steel H-piles are low displacement piles in comparison with precast concrete or steel tube piles. It is recommended that a steel H-pile with a relatively heavy section, such as HP 310 x 110, be used to prevent damage to the pile during the anticipated heavy driving conditions and due to the presence of cobbles and boulders in the till.

Steel H-piles (HP310 x 110) driven to refusal in the very dense silty sand to sandy silt till can be designed for MTO's standard values of 1800 kN/pile for U.L.S. and 1600 kN/pile for S.L.S., for very dense till soils. The following table summarizes the estimated pile tip elevations at the borehole locations.

Table 5.1.1.3.1: Estimated Tip Elevations for Steel H-Pile Foundation

Location	Borehole No.	Existing Ground Elevation at Borehole Location (m)	Estimated Pile Tip Depth/Elevations (m)
North Abutment	M1 & M1A M2 & M2A	81.1 82.9	6.3*/74.8 6.4*/76.5
South Abutment	M5 M6	82.7 82.3	7.9*/74.8 8.5*/73.8

*below existing ground surface.

The estimated pile tip elevations in between or beyond the borehole locations can be estimated by interpolation in between and beyond the borehole location or alternatively, an average single elevation can be quoted as follows, north abutment: El. 75.5 m, south abutment: El. 74.3 m.

According to AECOM, the elevation for the pile tops will be approximately 84.0 m and therefore length of the piles based on the borehole data can be expected to range from about 7.5 m to about 10.2 m. However, the actual pile lengths may vary. We recommend that consideration be given to this aspect when ordering the piles.

The piles should be driven into the competent glacial till deposit using a suitably heavy hammer capable of delivering a suitable rated energy. The possibility of piles encountering potential cobbles and boulders in the till should be anticipated. In view of this, as well as the very dense nature of the till, the tips of the piles should be stiffened as per OPSD-3000.100, Type I, to minimize damage to the piles in anticipation of heavy driving conditions. Care must be taken to avoid overdriving and damaging the pile tip (i.e., the structural capacity of the piles should not be exceeded).

The actual pile tip elevations and the driving of the piles in the field should be controlled by a recognized pile driving formula such as the Hiley Formula, in accordance with MTO standard SS103-11. Normally, in accordance with MTO practice, the estimated ultimate resistance of the piles by the Hiley Formula can be calculated by multiplying the recommended axial resistance at U.L.S. by a factor of 2 (i.e., 1800 x 2), giving an ultimate geotechnical resistance of 3600 kN. In accordance with the above criterion, we recommend that the piles be driven to about 1.5 m above the estimated pile tip elevations, and driving should then be monitored and controlled by employing the Hiley Dynamic Pile Driving Formula in accordance with MTO Standard SS103-11, using an ultimate geotechnical resistance of 3600 kN per pile, subject to the approval of the QVE.

If the piles encounter refusal before sufficiently penetrating into the competent sandy silt to silty sand till deposit, then pile capacities may need to be revisited and alternative measures sought. Therefore, pile driving records should be kept and if refusal is encountered above the recommended bearing zone, a geotechnical engineer should review the driving records to assess the axial resistance. As well, the Structural Engineer should be consulted for minimum pile length requirements. It is also possible that the piles may be driven some distance below the estimated pile tip elevations to achieve the desired capacity.

All pile driving should be carried out in accordance with SP903S01. Re-striking should be done as per SP903S01. At least 10% of the piles (but not less than two piles) driven at each support element should be re-tapped not less than 24 hours after the driving of the pile, as per SP903S01, to check that relaxation has not occurred. If it has, then all the piles should be re-tapped.

In addition, it may be necessary to stagger the driving of the piles, if heaving is observed. The use of light-weight (e.g. HP 310 x 79) piles is not recommended as lighter piles are more vulnerable to damage. If premature refusal is encountered, allowance may have to be made to resort to pre-augering, if necessary, as well as to reduce the axial resistance and uplift capacity of the piles. Any decision regarding pre-augering should be made in consultation with the Design Engineer, since pre-augering will lead to a loss in lateral resistances and also possibly in axial resistances. An NSSP should be included in the contract documents to alert the contractor of the possible presence of cobbles and boulders and possible heavy driving requirements through the very dense strata, possible pre-augering as well as the high water table within the cohesionless soils.

For frost protection, all pile caps should have a permanent earth cover of at least 1.7 m.

Oversize materials (e.g. greater than 75 mm nominal diameter) should not be used in the embankment fills through which piles would be driven.

Eccentric loading on piles and the required pile spacing should be considered as per the most recent Canadian Highway Bridge Design Code. Reference may be made to Section C6-8.7.1 of the Canadian Highway Bridge Design Code (S6-06), CHBDC, for assessing lateral pile resistance.

In cohesionless soils, the coefficient of horizontal subgrade reaction can be estimated from:

$k_s=n_h z/d$

Where k_s = coefficient of horizontal subgrade reaction

z = depth

d = pile width

n_h = coefficient related to soil density as given in Table 5.1.1.3.2.

Also as presented in the same table are estimated values for angle of internal friction and bulk unit weights.

Where the soil is primarily cohesive, the undrained shear strength of the soil is given. In this case,

$$k_s=67\ c_u/d$$

Where k_s = coefficient of horizontal subgrade reaction

c_u = undrained shear strength

d = width of pile

Table 5.1.1.3.2

Area Reference/ Borehole No.	Applicable Elevation (m)	Soil Type	Bulk Unit Weight (kN/m^3)	Angle of Internal Friction (ϕ) Degrees	Recommended n_h Value (kN/m^3)	Recommended Undrained Shear Strength, c_u (kPa)	Groundwater Elevation (m)
North Abutment/ M1 & M1A	81.0-80.3 80.3-79.6 79.6-76.5 76.5-73.7	loose silt loose till compact till v. dense till	16.0 20.0 21.0 22.0	27 29 30 33	2,000 1,300 4,400 11,000		80.2
North Abutment M2 & M2A	82.7-82.1 82.1-81.4 81.4-78.4 78.4-73.7	Loose fill Compact granular fill Compact till v. dense till	16.0 21.0 21.0 22.0	27 33 31 33	2,000 6,600 4,400 11,000		80.0
South Abutment M5	82.5-80.9 80.9-80.4 80.4-77.5 77.5-73.5	organic soils firm clayey silt compact till v. dense till	13.0 16.0 20.5 22.0	20 - 30 33	400 - 4,400 11,000	20	80.9

Area Reference/ Borehole No.	Applicable Elevation (m)	Soil Type	Bulk Unit Weight (kN/m^3)	Angle of Internal Friction (ϕ) Degrees	Recommended n_h Value (kN/m^3)	Recommended Undrained Shear Strength, c_u (kPa)	Groundwater Elevation (m)
South Abutment M6	82.1-81.5 81.5-80.5 80.5-76.0 76.0-70.1	loose fill dense granular fill compact till v. dense till	17.5 21.0 21.0 22.0	27 33 32 33	2.000 11.000 4,400 11,000		80.9

For preliminary design purposes, the recommended horizontal resistances for HP 310 x 110 steel H-piles are as follows:

Horizontal Resistance at ULS = 110 kN/pile

Horizontal Resistance at SLS = 40 kN/pile

In accordance with MTO requirements (MTO Structural Office Standard), piles for integral abutments require a 3 m long flex zone. In essence, the current MTO standard for the flex zone consists of an annular space in between two concentric corrugated steel pipes (CSP's). One of the CSP's surrounds the H-pile (i.e. has a diameter of about 600 mm surrounding the pile, while the second CSP has a somewhat larger diameter; typically 800 mm for a 310 mm H-pile). The annular space in between the CSP's is the 3 m long flex zone. In accordance with current MTO practice, this space between the CSP's can be left void. After the pile is driven, the space between the H-pile and the inner CSP is filled with sand. This double CSP scheme is typically used for false abutments.

If a false abutment is not provided, in accordance with MTO structural office requirements (Report SO-96-01), the flex zone can be provided by augering a 600 mm diameter hole 3000 mm deep and filling with uniform sand. A special provision should be included in the contract specifying the gradation of the sand as follows:

Sieve Size	Percentage Passing
2 mm	100 %
600 µm	80-100 %
425 µm	40-80 %
250 µm	4-25 %
150 µm	0-6 %

A special provision should be provided in the contract for the supply and installation of the CSP's

5.1.2 Central Pier Foundations

The new bridge will be a two-span structure with a central pier located within the existing median of Highway 401.

Boreholes M3 and M4 drilled within the median area indicate beneath an approximately 0.2 m thick veneer of topsoil, the presence of silt with some organic inclusions (Borehole M3) and organic silt (Borehole M4). These deposits are in a loose condition and extend to a depth of 0.8 m below the ground surface or to El. 80.4-80.6 m. They are underlain by a sandy silt to silty sand till with occasional cobbles near the surface, with increasing frequency of cobbles and boulders with increasing depth. The boreholes were extended in this glacial till deposit to about 8 m below the ground surface or to El. 73.4-73.2 m to practical refusal on the augers (on inferred boulders). The recorded N-values are variable ranging from 10 to 38 blows/0.3 m to a depth of about 4 m or to about El. 77 m (typically 12 to 25 blows/0.3 m), with higher N-values (i.e. in excess of 50 blows/0.3 m) below about El. 77 m. The groundwater table was recorded about one meter below the ground surface or at about El. 80.0 to 80.5 m.

These conditions indicate that if normal spread footings are to be utilized they must be extended to about El. 77 or about 2.5 m below the normally desirable founding level of about 2 m below the existing grade in the median area. This means excavations extending several metres below the estimated groundwater level. As such, the use of spread footing foundations at this site is not recommended. Deep foundation option must therefore be resorted to. The feasible options are driven steel H-piles, micropiles and drilled caisson foundations as discussed later in this section of the report. A summary of various foundation options for the central pier foundation support is given in Appendix E.

5.1.2.1 Spread Footing Foundation on Natural Soil

The central pier can be supported on spread footing foundation placed on the undisturbed dense to very dense sandy silt to silty sand till. The depth to the surface of the sufficiently competent till from the existing ground surface is given in the following table.

Borehole No.	Existing Ground Elevation (m)	Recommended Highest Founding Level Below Existing Ground (m)	Elevation (m)	SLS (kPa)	ULS (kPa)	Subgrade Soil
M3	81.2	4.2	77.0	500	800	V. dense Sandy silt to silty sand till
M4	81.4	4.4	77.0	500	800	V. dense Sandy silt to silty sand till

The factored bearing resistance at ULS given in the above table incorporates a resistance factor of 0.5 as per Canadian Highway Bridge Design Code (CHBDC) S6-06. The serviceability condition is based on the premise that the maximum total and differential settlements will not exceed 25 mm and 20 mm, respectively. This can be achieved provided that the founding subgrade is undisturbed during the construction.

Under inclined loading conditions, the bearing resistance at ULS should be reduced in accordance with CHBDC S6-06.

As can be seen from the above table, relatively deep excavations extending to below water table will be required. As the footings should be constructed in the dry, dewatering as well as a temporary shoring system will be required (due to the proximity to the existing structure).

For frost protection the footing should have a permanent earth cover of at least 1.7 m.

Allowance should be made to place a 120 mm thick concrete mud mat (i.e. skim coat) in the footing excavation as soon as possible (not more than four hours) after excavation. The footing excavation should be inspected and approved by the geotechnical engineer prior to pouring the concrete mud mat.

Spread footing foundation is not recommended for this bridge due to the extensive excavation and dewatering requirements.

5.1.2.2 Steel H-piles

The boreholes show that the geotechnical conditions in the central pier area are generally suitable for the use of driven H-piles. Borehole M3 encountered very dense soil at about El. 77 m and refusal to augering at El. 73.4 m, where the borehole was terminated. In Borehole M4, refusal to further augering was encountered at about El. 76 m and the borehole was extended by about 2 m to El. 74 m by coring through boulders and the borehole was then terminated at about El. 73 m. Since the existing grade is at about 81.3 m and assuming that the pile top elevations will be at about El. 79.5 m, the pile lengths can be very short (i.e. between about 3.5 and 6 m), due to the presence of frequent cobbles and boulders as revealed by Borehole M4 and the possible presence of cobbles and boulders in the till deposit in Borehole M3.

These conditions indicate that the pile lengths will likely be much shorter than conventional and there may be problems during the driving of the piles, especially if boulders are encountered at relatively high elevations. Assuming that the piles can be driven to about El. 75.5 m, a minimum pile length of 4 m will be provided (assuming that the pile top elevation will be 79.5 m). In this instance, due to the anticipated short and irregular pile lengths, we recommend that the pile resistances be lowered to less than conventional resistances. However, since MTO's standard design values are 1600 kN/pile for SLS and 1800 kN/pile for ULS, we recommend that this aspect be taken into consideration by the Structural Engineers in their design by increasing the applied load factors.

In our opinion, since the pile lengths are quite short, even with the relatively low resistances (or higher factored structural loads), H-pile option should present a cost-effective solution.

For the reasons cited (i.e. very short pile lengths and the presence of cobbles of boulders, as well as the presence of very dense zones in the till) the use of other types of driven piles including timber piles, concrete piles and steel tube piles is not recommended.

The following table summarizes the anticipated pile tip elevations for HP 310x110 steel H-piles.

Table 5.1.2.2.1: Estimated Tip Elevations for HP310x110 Steel H-piles at Central Pier Area

Borehole No.	Existing Ground Elevations at Borehole Location (m)	Probable Pile Top Elevation	Estimated Pile Tip Elevation (m)	Estimated Pile Length Below Probable Pile Top Elevation (m)
M3	81.2	79.5	75.5-75.0	4.0-4.5
M4	81.4	79.5	75.5-75.0	4.0-4.5

As mentioned before, since the anticipated pile lengths are very short, we recommend that the pile resistances be lowered to take this aspect into consideration. Since standard MTO resistances for HP310x110 steel H-piles driven to practical refusal in the very dense till deposits are high, this can be achieved by upward factoring of structural loads on the central pier.

The pile tip elevations provided in Table 5.1.2.2.1 are for estimating purposes only. Due to potentially variable soil conditions, the actual pile tip elevations may vary. The contract should allow for some variations in pile length and this aspect should be taken into consideration when ordering the piles. The piles should be driven into the competent glacial till deposit using a suitable hammer. The possibility of piles encountering potential cobbles and boulders in the till should be anticipated. In view of this, as well as the very dense nature of the till, the tips of the piles should be stiffened to minimize damage to the piles in anticipation of heavy driving conditions, as per OPSD 3000.100 Type I. Care must be taken to avoid overdriving and damaging the pile tip (i.e., the structural capacity of the piles should not be exceeded).

The actual pile tip elevations and the driving of the piles in the field should be controlled by a recognized pile driving formula, such as the Hiley Formula. Normally, in accordance with MTO practice, the estimated ultimate resistance of the piles by the Hiley Formula can be calculated by multiplying the recommended axial resistance at U.L.S. by a factor of 2 (i.e., 1800 x 2), giving a geotechnical resistance of 3600 kN. With the above criterion, we recommend that the piles be driven to about 1.5 m above the estimated pile tip elevations, and the driving should then be monitored and controlled by employing the Hiley Dynamic Pile Driving Formula in accordance with MTO Standard SS103-11, using an ultimate geotechnical resistance of 3600 kN per pile, subject to the approval of the QVE.

If the piles encounter refusal before sufficiently penetrating into the competent sandy silt to silty sand till deposit, then pile capacities may need to be revisited and alternative measures sought. Therefore, pile driving records should be kept and if refusal is met above the recommended bearing zone, a geotechnical engineer should review the driving records to assess the axial resistance. As well, the Structural Engineer should be consulted for minimum pile length requirements. Short pile lengths would reduce uplift resistance, if uplift resistance is a factor for the central pier this may be a concern for the structural engineer. It is also possible that the piles may be driven some distance below the estimated pile tip elevations to achieve the desired capacity.

All pile driving should be carried out in accordance with SP903S01. Re-striking should be done as per SP903S01. At least 10% of the piles (but not less than two piles) driven should be re-tapped not less than 24 hours after the driving of the pile, as per SP903S01, to check that relaxation has not occurred. If it has, then all the piles should be re-tapped.

In addition, it may be necessary to stagger the driving of the piles, if heaving is observed. The use of light-weight (e.g. HP 310 x 79) piles is not recommended as lighter piles are more vulnerable to damage. If premature refusal is encountered, allowance may have to be made to resort to pre-augering or boulder removal, if necessary, as well as to reduce the axial resistance and uplift capacity of the piles. Any decision regarding disturbing the soil should be made in consultation with the Design Engineer, since pre-augering or removal of boulders with a backhoe will lead to a loss in lateral resistances and also possibly in axial resistances. An NSSP should be provided to alert the contractor of the possible presence of cobbles and boulders, the high water table within the cohesionless soils, the possible heavy driving requirements through the very dense strata, as well as possible pre-augering; reference should be made to SP 903S01.

For frost protection, all pile caps should have a permanent earth cover of at least 1.7 m.

The piles should be provided with reinforced tips, as per OPSD 3000.100, Type I.

As mentioned in Section 5.1.1, eccentric loading on piles and the required pile spacing should be considered as per the latest edition of Canadian Highway Bridge Design Code.

The following table contains the recommended soil parameters for the calculation of coefficient of horizontal subgrade reaction.

Table 5.1.2.2.2: Soil Parameters

Reference Borehole	Applicable Elevation (m)	Soil Type	Bulk Unit Weight (kN/m^3)	Angle of Internal Friction (ϕ) Degrees	Recommended n_h Value (kN/m^3)	Groundwater elevation (m)
M3	81.0-80.4 80.4-77.3 77.3-73.4	Silt Compact till v. dense till	16.0 21.0 22.0	24 31 34	1,000 4,400 11,000	80.0
M4	81.2-80.6 80.6-76.7 76.7-73.2	Organic silt Compact till v. dense till	14.0 21.0 22.0	20 31 34	500 4,400 11,000	80.2

5.1.2.3 Caisson Foundations

Augered and cast-in-place concrete foundations (drilled caissons) can be considered for the central pier.

For caissons socketed into the very dense sandy silt to silty sand till by 1.0 m, geotechnical design resistance values of 1600 kPa at SLS and 2400 kPa at ULS can be used. These design values are applicable to commonly used caisson sizes in Ontario (i.e. between 0.76 and 1.8 m diameter) provided the minimum caisson length is 3.0 m below the bottom of the pile cap. However, the use of relatively smaller caisson sizes (i.e. between 0.76 and 1.5 m) would be preferable as these are relatively easier and more efficient to install. For example, a 0.9 m diameter caisson will have a base area of $r^2\pi=(0.9/2)^2x3.1416=0.64\ m^2$. When designed for a SLS value of 1600 kPa, the caisson would be capable of carrying an axial load of 0.64 m^2 x 1600 kN/m^2 = 1024 kN/caisson at SLS. Similarly, if a 1.2 m diameter caisson is used, then the caisson resistance at SLS would be $(1.2/2)^2x3.1416x1600=1810$ kN/caisson.

As was mentioned before, these resistance values assume a minimum of 1.0 m socket into the very dense till. This aspect must be verified during the installation of the caissons by the geotechnical engineer appointed by the QVE, who would also inspect the base of the caissons and approve them. We recommend that an NSSP be issued to cover this requirement.

At the location of Boreholes M3 and M4, the anticipated caisson depths (below existing grade) and base elevations are 5.2 m/76.0 m and 5.9 m/75.5 m, respectively, in order to provide a minimum socket of 1.0 m into the very dense till.

The minimum caisson diameter is 0.76 m to enable the cleaning and inspection of the base of the caisson. The clear distance between any two adjacent caissons should be at least two diameters (edge to edge).

Dewatering may be required to facilitate the installation of the caisson units, especially since the till is a basically non-cohesive soil and to preserve the geotechnical resistance of the soil. As well, difficulties can be expected due to the presence of cobbles and boulders in the till.

Difficulties may arise during the installation of the caissons due to the basically cohesionless nature of the till below the groundwater table, as well as the presence of cobbles and boulders in the till. Some dewatering is expected to be necessary to intercept and remove surface water and to pump out any perched water. As well, dewatering may be required to prevent the disturbance of the base of the caisson before pouring the concrete. Temporary steel casing will be required during the construction of the caisson holes to prevent caving. The casing would be withdrawn as the concrete is poured, ensuring a sufficient head of concrete in the casing to prevent 'necking.' Concrete must be poured expeditiously after the preparation and approval of the base of the caisson base of the caisson to prevent its disturbance due to hydrostatic uplift. Even though these are standard aspects of caisson installation operations, we recommend that they be 'red-flagged' in the contract documents to reduce the possibility of claims for 'extras' by the contractor, including the presence of cobbles and boulders in the glacial till deposit. An NSSP should be issued to alert the contractor of boulders and cobbles and cohesionless soils submerged below the groundwater table.

5.1.2.4 Micropiles

Another alternative which may be considered is the use of micropiles to support the central pier.

A micropile is constructed by drilling a borehole, placing reinforcement, and grouting the hole. Micropiles can withstand axial and/or lateral loads, and may be considered a substitute for conventional piles or as one component in a composite soil/pile mass, depending upon the design concept employed. Micropiles are installed by methods that cause minimal disturbance to adjacent structures, soil, and the environment. They can be installed in access-restrictive environments and in most soil and rock types and ground conditions. Micropiles can be installed at any angle below the horizontal using the same type of equipment used for ground anchor and grouting projects. Since the installation procedure causes minimal vibration and noise and can be used in conditions of low headroom, micropiles are often used to enhance the support of existing structures. Micropile structural capacities, by comparison, rely on high capacity steel elements to resist most or all of the applied loads. These steel elements have been reported to occupy as much as one-half of the whole volume. The special drilling and grouting methods used in micropile installation allow for high grout/ground bond values along the grout&round interface. The grout transfers the load through friction from the reinforcement to the ground in the micropile bond zone in a manner similar to that of ground anchors. Due to the small pile diameter (typically 160 to 260 mm), any end-bearing contribution in micropiles is generally neglected. The grout/ground bond strength achieved is influenced primarily by the ground type and grouting method used, i.e., pressure grouting or gravity feed. The role of the drilling method is also influential, although less well quantified.

Axial resistances of up to about 750 kN/micropile are available (at ULS) depending on the diameter and penetration into the very dense till. The lateral resistances would also depend on the diameter and penetration length into the very dense till.

As mentioned before, the use of micropiles may be less economical than caissons due to the fact that the installation requires a more specialized installer for the micropiles than the many contractors who are able to routinely install caissons. However, it is advantageous if low overhead is necessity and/or interference of new foundation support (i.e. caisson) with the existing pile foundations. As was mentioned before, geotechnical resistances will also depend on such factors as diameter, method of installation, micropile lengths, etc. Typically, the geotechnical resistance is calculated by multiplying the circumferential area (i.e. circumference x length) by bond strength. For preliminary estimating purposes, the bond strength between the micropile and the very dense till can be taken as 250 to 350 kPa. A special provision will need to be developed for this project.

The axial and horizontal resistances of micropiles and other details regarding the design of micropiles can be discussed with specialist contractor and will be pleased to expand on this further should you wish to pursue this option.

5.2 Lateral Earth Pressures

Backfill behind abutments should consist of non-frost susceptible, free-draining granular materials in accordance with the Ontario Ministry of Transportation Standards and the requirements of OPSD 3101.150, as given in Appendix F.

Free-draining backfill materials (i.e. Granular ‘A’ or Granular ‘B’) and the provision of drains pipes and weep holes, etc., should prevent hydrostatic pressure build-up. Computation of earth pressures should be in accordance with CHBDC S6-06. For design purposes, the following parameters (unfactored) can be used.

Compacted Granular ‘A’ and Granular ‘B’ Type II

Angle of Internal Friction, ϕ = 35° (unfactored)
Unit Weight = 22 kN/m^3
Coefficient of Lateral Earth Pressure:

K_a = 0.27	K_b = 0.35
K_o = 0.43	K^* = 0.45

Compacted Granular ‘B’ Type I

Angle of Internal Friction, ϕ = 32° (unfactored)
Unit Weight = 21 kN/m^3
Coefficient of Lateral Earth Pressure:

K_a = 0.31	K_b = 0.41
K_o = 0.47	K^* = 0.57

Where K_b is the ‘intermediate’ earth pressure coefficient for a partially restrained structure. This case occurs when some movement (yield) of the retaining structure occurs but not in a sufficient magnitude to fully mobilize an active condition (as such an intermediate condition between K_o and K_a occurs).

K* is the earth pressure coefficient for a soil loading a fully-restrained structure, including compaction surcharge effects.

These values are based on the assumption that the backfill behind the retaining structure is free-draining and adequate drainage is provided. As well, it is assumed that the ground behind the retaining structure is level.

The earth pressure coefficient adopted will depend on whether the retaining structure is restrained or movements can be allowed such that the active state of earth pressure can develop. If the abutment is restrained and does not allow lateral yielding, then at rest pressures should be used in accordance with CHBDC S6-06. The effect of compaction should also be taken into account in the selection of the appropriate earth pressure coefficients in accordance with Section 6.9 of CHBDC S6-06.

For unrestrained wing walls (if any), the intermediate earth pressure coefficient K_b may be adopted. In the determination of degree of wall displacement or rotation to mobilize the fully active earth pressure state, Section C6.9 of the CHBDC S6-06 Commentary can be consulted.

Vibratory equipment for use behind abutments and retaining walls should be restricted in size as per current MTO practice.

5.2.1 Seismic Design Data

5.2.1.1 Site Coefficient

The subsurface conditions encountered at the site are represented by Soil Profile Type I (see Clause 4.4.6.2 of CHBDC CAN/CSA-S6-06). For seismic design, therefore, in accordance with Clause 4.4.6.1 site coefficient, S, for the site is 1.0.

5.2.1.2 Seismic Zone and Zonal Acceleration Ratio (A)

Table A3.1.1 of the CHBDC provides a zonal Acceleration Ratio (A) of 0.20 and Velocity Related Seismic Zone (Z_v) of 2 for Cornwall. As site coefficient (S) is 1.0, and the zonal acceleration is 0.20, the design zonal acceleration ratio for the site can be taken as A=0.20.

5.2.1.3 Seismic Earth Pressures

Seismic (earthquake) loading should be taken into account in the design in accordance with Section 4.6 of the CHBDC.

In accordance with Sections 4.6.4 and C.4.6.4 of the CHBDC and its Commentary, the horizontal seismic coefficient, k_h, used in the calculation of the seismic active pressure coefficient, is taken as kh=0.20. The seismic active earth pressure coefficient is also dependent on the vertical component of the earthquake acceleration coefficient, k_v. Three discrete values of vertical acceleration coefficient are typically selected analysis, corresponding to $k_v = +2/3\ k_h$, $k_v = 0$, and $k_v = -2/3\ k_h$.

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

Seismic Active Pressure Coefficients

Active Earth Pressure Coefficient	Granular 'A' (ϕ = 35° - unfactored)	Granular 'B' Type II (ϕ = 32° - unfactored)
Non-Seismic, K_a	0.27	0.31
Seismic, K_{AE}	0.42	0.47

In the calculation of K_{AE}, the effect of the friction between the wall and the soil is not considered (i.e.δ=0).

5.2.1.4 Liquefaction Potential

The proposed structures will be supported by deep foundations (driven piles and cassions) founded in/on dense tills. The founding soils are considered not liquefiable.

The liquefaction potential of the soils below the approach embankments under seismic loading has been considered using the empirical method outlined in Section C4.6.2 of the CHBDC Commentary, which correlates the cyclic resistance ratio of the soils with their normalized penetration resistance and fines content. Based on this assessment, and assuming a ground surface acceleration of 0.20 g, a factor of safety of greater than 1.0 against liquefaction is obtained for magnitude 7.5 earthquake events under the approach embankment.

5.3 Approach Embankments

Based on the information provided to us by AECOM, the grade at the north and south abutment locations will be raised by about 2.0 m above the grade of the existing embankments or to about El. 90.2 m and that this will involve a grade raise of up to about 7 to 8 m over the existing grades (o.g.).

Based on the available borehole data, foundation failures are not anticipated for approach embankments of up to 7 to 8 m in height constructed with normal 2H:1V side slopes or flatter, provided that all organic, soft/very loose or otherwise unsuitable materials will be removed as per MTO standards, prior to placing the embankment fills, as per standard MTO procedures. The anticipated stripping depths/elevations at the borehole locations are as follows:

Borehole No.	Existing Ground Elevation at the Borehole Location (m)	Recommended Stripping Elevation/Depth (m)
M1	81.1	80.9/0.2
M2	82.9	82.6/0.3
M5	82.7	80.8/1.9
M6	82.3	82.0/0.3
M7	82.4	80.9/1.5
M8	83.6	83.3/0.3

After stripping, the exposed subgrade should be inspected, approved and properly compacted (i.e. proof rolled) from the surface, using a heavy compactor. If necessary, the groundwater table should be lowered to at least 0.7 m in below the subgrade level, before any proofrolling and the application of significant compaction effort. This dewatering can be achieved by gravity drainage and pumping from strategically placed sumps and, if necessary, ditches.

Assuming properly compacted, acceptable inorganic earth fill materials are utilized, 2 horizontal to 1 vertical side slopes can be used for the construction of the approach fills. Proper erosion control measures should be implemented by seed and cover (OPSS 572) or sodding (OPSS 571).

The existing embankments side slopes should be properly benched as per MTO standards (OPSD 208.010) where the new embankment fills are to abut into the existing.

The materials used for the construction of the embankment fills should consist of approved, acceptable earth fill (e.g. Select Subgrade Materials - OPSS1010). Fill used for construction of the embankments should be in accordance with OPSS 212 and fill placement should meet or exceed the requirements of

SP 105S10 and OPSS 206. Construction should be in accordance with SP 206S03. Quality assurance should be provided as per MTO standard 501.08.

Based on the findings of the boreholes, the anticipated foundation settlements under the stresses generated by the approximately 7 to 8 m grade raise are approximately 40 mm, while another 40 mm of settlement can occur due to settlement of the new embankment fill under its own weight, bringing the total anticipated settlements to about 80 mm. The anticipated total settlements are therefore not more than 80 mm, which, in our opinion, necessitate neither surcharging nor preloading, especially since some of these settlements would take place immediately after construction. The foundation settlements should be substantially completed within a period of about three months while the settlement due to the own weight of the embankment will depend on the type of soil used to build the embankment (e.g. the settlement of granular soils will be relatively rapid while clayey soils will settle more slowly). Assuming an average SSM type soil, the settlement of the embankment under its own weight should also be substantially completed within about three months. We recommend that in order to minimize differential settlements immediately adjacent to the new bridge structure, the approach embankments be constructed to the subgrade elevation (i.e. bottom of granular pavement fill) prior to driving the piles. The grade in the area of the pile driving can then be lowered to the desired elevation for pile driving. This will effect some of the settlements prior to paving of the road. As well the paving of the road itself should be delayed by about four weeks, after the placement of the granular pavement fills, if possible.

In addition, the construction of the new embankment immediately adjacent to the existing embankment will cause settlement of the existing embankment. Assuming that the new bridge will be 3.0 m away from the existing (i.e. clear distance between the structures) and that the new embankment will be 2.0 m higher than the existing, based on the results of Boreholes M9 and M10, the settlement of the existing embankment due to stress superposition is about 40 mm at about midway point of the embankment slope, gradually decreasing in magnitude towards the shoulder of the existing embankment. This amount of settlement near the western edge of the paved portion of the road is not expected to cause significant problems with the performance of the existing road; especially since the road will be removed after the construction (i.e. will be used as a temporary road to maintain traffic on Moulinette Road during the construction). It is, however, recommended that any excessive differential settlements should be observed during the construction and if necessary they can be rectified (e.g. any cracking of the pavement), due to stress superposition, and or due to vibrations during pile driving. An NSSP may be issued for this aspect to alert the Contractor.

5.4 Construction Comments

All excavations, shoring and backfilling should be carried out in conformance with the Occupational Health and Safety Act (OHSA), Regulation 213/91, as well as the following specifications.

SP 105S19 – Protection Systems

SP 902S01 – Excavation and Backfilling to Structures.

The boreholes show that the excavations can be expected to extend through some fill material and surficial clayey silt, sandy silt and gravelly sand deposits which are underlain by silty sand to sandy silt till layers. These soils can be classified as follows:

Granular Embankment (Pavement) Fill	Type 3 soil
Embankment Fill (Sandy silt to silty sand, gravelly sand)	Type 3 soil
Other Fill (Clayey silt to sandy silt)	Type 4 soil
Organic Soils	Type 4 soil
Clayey Silt To Sandy Silt	Type 3 soil above water level Type 4 soil below water level
Glacial Till (Dense to very dense)	Type 2 soil above water table Type 4 soil below water table, if the soil was not dewatered
Glacial Till (Loose to compact)	Type 3 soil above water table Type 4 soil below water table, if the soil was not dewatered

If at the central pier location normal spread footings are to be utilized, then dewatering will be required since the groundwater table at the time of our investigation was typically about one meter below the o.g. levels. As excavations must be carried out in the dry, aggressive dewatering will be required. This may consist of vacuum well points or deep wells/deep filtered sumps along with perimeter ditches (to intercept and dispose of surface/perched water). Based on the information provided to us by AECOM, the foundations of the existing bridge including the central pier, are supported on driven steel H-piles. If this is the case, then construction dewatering should not have a major detrimental effect of the performance of the foundations of the bridge. If however spread footing were used then dewatering should be carried out in a manner so as not affect the performance of the foundations of the existing bridge, since dewatering may cause a settlement of the central pier foundation of the existing bridge. The edge of the footing closest to the proposed bridge would undergo greater settlements due to dewatering, thus causing some rotation. If the dewatering is properly designed and executed, the maximum settlement of the on the side closer to the new bridge construction should not exceed 6 mm and should therefore in our opinion not be detrimental to the overall performance of the structure. In addition, due to the proximity to the existing bridge pier, shoring will likely be required on the east side of the excavation. The shoring system should be designed by a Professional Engineer, experienced in this type of work. All shoring should be in accordance with SP 105S19.

Consideration can be given to issuing an NSSP red flagging to the contractor that dewatering and shoring should be designed and carried out in a manner so that performance of the existing pier foundation will not be adversely affected, especially if the existing and the proposed bridge are both supported on spread footings.

As was mentioned before, if caisson foundations are used to support the central pier, some dewatering may be required due to the essentially non-cohesive nature of the till, together with the recorded high water table to retain the integrity of the base of the caisson excavations.

Some minor dewatering will also be required to facilitate stripping and the construction of the new embankment fills, which, should it be necessary, can consist of gravity drainage and pumping from strategically placed sumps. Dewatering is not expected to influence the wetlands located next to the Moulinette Road Bridge or vice versa

Temporary support will be required along the west side of the abutments and approach embankments of the existing structure. This likely consists of shoring. In Ontario, shoring is typically in the form of soldier piles and lagging. In this instance, tiebacks will also likely be required. Alternatively, horizontal support can be provided from the east side of the existing embankment (e.g. dead-man type support). The soldier piles can be expected to extend into the very dense till. Tiebacks would also extend through the existing embankment fill into the very dense till. There is some evidence that bedrock may be present at the site at about El. 70 m.

The shoring system should be designed so that the lateral movement of any portion of the roadway protection system will not exceed the established criterion for the structural performance level. In this case, the required performance level is considered 2. The shoring system should be designed by a Professional Engineer, experienced in this type of work. As mentioned before all shoring should be in accordance with SP 105S19.

Table 5.4.1: Recommended Unfactored Parameters for Temporary Shoring Design

Soil Type	K_a	K_o	K_p	γ (kN/m^3)
Granular Embankment Fill (typically upper 1.2 m)	0.32	0.49	3.1	21.0
Lower Embankment Fill	0.33	0.50	3.0	20.5
Other Fill	0.38	0.55	2.7	18.0
Organic Soils & Topsoil	0.55	0.72	1.0	14.0
Surficial Clayey Silt/Sandy silt/Silt	0.45	0.62	2.2	17.0
Silty Sand to Sandy Silt Till (loose to compact)	0.33	0.50	3.0	20.5
Silty Sand to Sandy Silt Till (dense to very dense)	0.29	0.45	3.4	22.0

It should be pointed out that the presence of cobbles and boulders can be expected within the overburden, as well possibly in the embankment fill. These can be expected to cause problems during the installation of shoring units. This aspect should be ‘red-flagged’ in the contract documents.

As was mentioned before, materials that may impede the driving of the piles should not be used in the affected areas.

It is also recommended that as a precaution, it would be prudent to monitor the vibrations during the driving of the piles.

5.5 Frost Protection

Design frost protection depth for the general area is 1.70 m. Therefore, a permanent soil cover of 1.70 m or its thermal equivalent of artificial insulation is required for frost protection of foundations, including pile caps. In case of rip-rap (rock fill), only one-half of the rock fill thickness should be assumed to be effective in providing frost protection.

6 CLOSURE

The Limitations of Report, as quoted in Appendix G, are an integral part of this report.

For and on behalf of Coffey Geotechnics Inc.

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Appendix E

Summary of Foundation Alternatives for Abutments and Central Pier

Summary of Foundation Alternatives for Abutments

Foundation Type	Advantage/ Disadvantage	Risks/Consequences	Relative Costs	Recommendations
Normal Spread Footings	Cost effective. Not suitable for integral abutment design. Will require extensive excavations.	High risk due to extensive excavation immediately adjacent to existing bridge.	Low to moderate Cost	Not recommended due to extensive excavation, very close to the existing structure as well as being not suitable for integral abutment design.
Spread footings on compacted Granular 'A' pad	Impractical to implement considering the closeness of the existing bridge structure. Not suitable for integral abutments.	Moderate bearing resistance and moderate to high settlements can be expected. Considered impractical for this project due to close proximity to the existing bridge structure.	Moderate Cost	Not recommended based on economics, practicality and reliability.
Steel H-piles	Low displacement piles; relatively short but adequate depth; suitable for integral abutment design.	Boulders may be encountered during the installation, which may present problems.	Moderate cost	Recommended based on suitability, economics and reliability. .
Steel Tube Piles	Higher displacement piles in comparison with steel H-piles. Not suitable for integral abutment design.	The presence of boulders and intermittent very dense zones in the glacial till deposit may present problems during their installation.	Moderate cost	Not suitable for integral abutment design and they are considered less reliable than Steel H-piles for this project. Not recommended.

Foundation Type	Advantage/ Disadvantage	Risks/Consequences	Relative Costs	Recommendations
Drilled and cast-in-place Concrete piles (drilled caissons)	Less vibration created than driven piles. Less reliable than driven H-piles. Not suitable for integral abutment design.	The presence of boulders may present problems during the installation of drilled caisson foundations.	Moderate cost	Not suitable for integral abutment design. Not recommended.

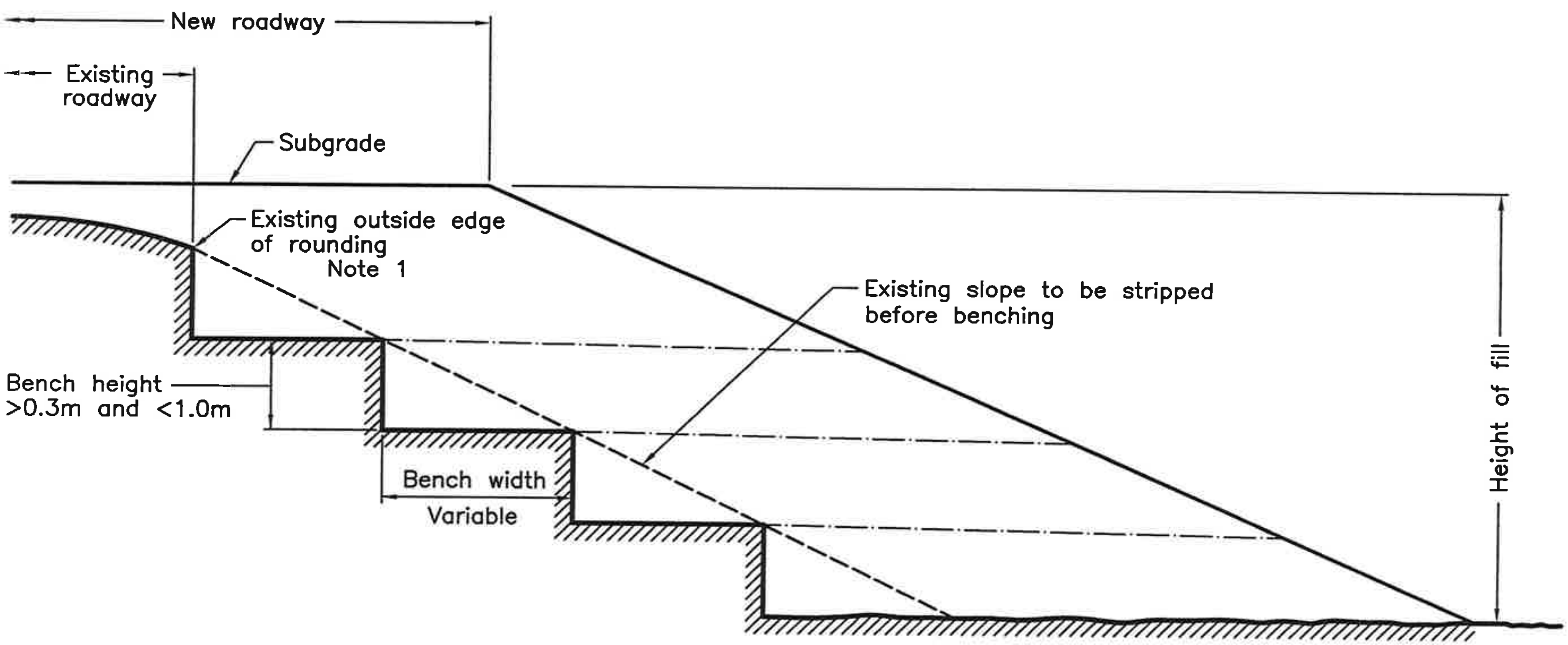
Summary of Foundation Alternatives at Central Pier

Foundation Type	Advantage/ Disadvantage	Risks/Consequences	Relative Costs	Recommendations
Normal Spread Footings	Least costly, but will necessitate a rather deep excavation extending below the groundwater table, immediately adjacent to the existing pier foundation. Will require shoring and dewatering.	Possible deformation of the existing pier foundation, due to adjacent excavation.	Economical	Not recommended due to extensive excavation (partially extending below the groundwater table), close to the existing structure.
Expanded Base (Frankie -type) Concrete Piles	Not well suited for the prevailing overburden conditions.	Extreme vibrations which may cause damage to the existing bridge structure.	Expensive	Not recommended
Auger Press Concrete Piles	Not very suitable for the prevailing subsurface conditions.	May not provide adequate lateral resistance. Boulders may increase costs.	Expensive	Not recommended based on economics and reliability.
Timber Piles	Prone to damage during driving due to boulders and very dense zones in the till. Will not provide adequate axial resistance.	Damaged piles may go undetected. The piles may be too short.	Economical	Not recommended along a major highway based on reliability.
Driven Concrete Piles	High displacement piles, not suitable for the subsurface conditions at the site.	Can be damaged during driving due to the pressure of cobbles and boulders.	Expensive	Not recommended based on cost and reliability.

Foundation Type	Advantage/ Disadvantage	Risks/Consequences	Relative Costs	Recommendations
Steel H-piles	Low displacement piles are well suited for the glacial till deposit underlying the site. However the piles will be shorter than normally accepted industry standards and as such relatively low axial and uplift resistances will be available. Problems may arise due to the presence of cobbles and boulders in the till deposit. No shoring required, minimizes dewatering.	The piles will be short and may be extremely short if boulders are encountered during their driving.	Moderate	Can be considered but piles will be short and will be rather risky if boulders are encountered (i.e. piles will be too short).
Steel Tube Piles	Higher displacement piles in comparison with Steel H-piles; vulnerable to damage due to the presence of cobbles and boulders and very dense zones in the glacial till. Less suitable than H-piles. No shoring required, minimizes dewatering.	Considered unsuitable for the prevailing subsurface conditions, may be too short.	Moderate	Not recommended based on reliability.
Drilled and Cast-in-place Concrete Piles (Drilled Caissons)	Minimizes vibrations; no shoring required; some dewatering will be required, provides suitable resistances.	Some problems may arise during the construction due to hydrostatic uplift and the presence of cobbles and boulders.	Moderate to Expensive	A feasible option.
Micropile Foundations	Minimizes vibrations and dewatering. Less economical than drilled caissons.	Problems may arise during the construction due to the presence of cobbles and boulders.	Expensive due to less competitive pricing.	A feasible option but more expensive than drilled caissons.

Appendix F

OPSD



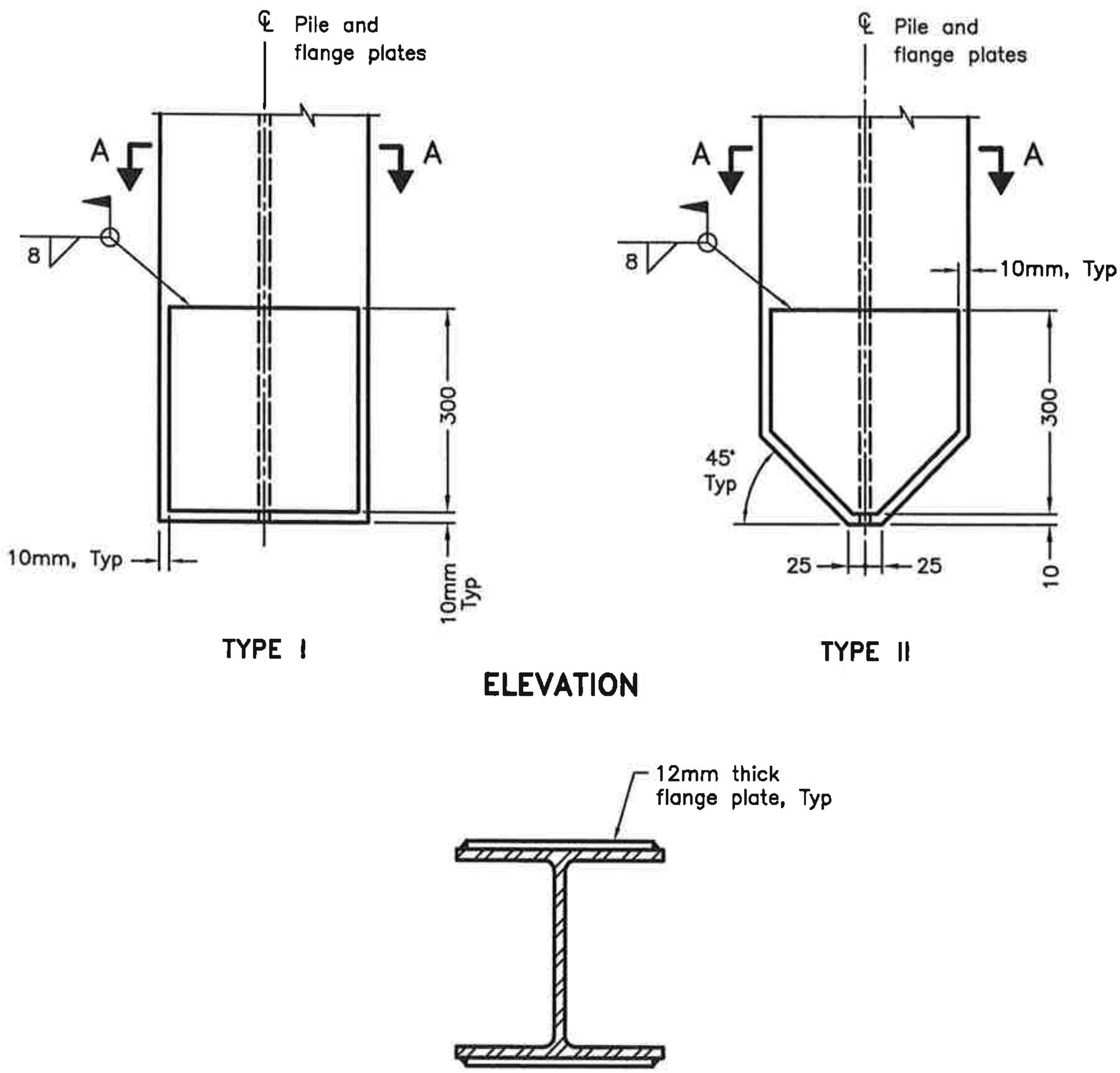
NOTES:

1 When the subgrade is below the existing outside edge of rounding, benching shall be carried out below the point where the subgrade intersects the existing slope.

A Benching is not required on existing slopes flatter than 3H:1V.

B Benches are to be excavated one level at a time and the compacted fill brought up before the next benching level is excavated.

ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2003	Rev 1
BENCHING OF EARTH SLOPES	**OPSD – 208.010**	



PILE DRIVING SHOE

SECTION A–A

NOTES:

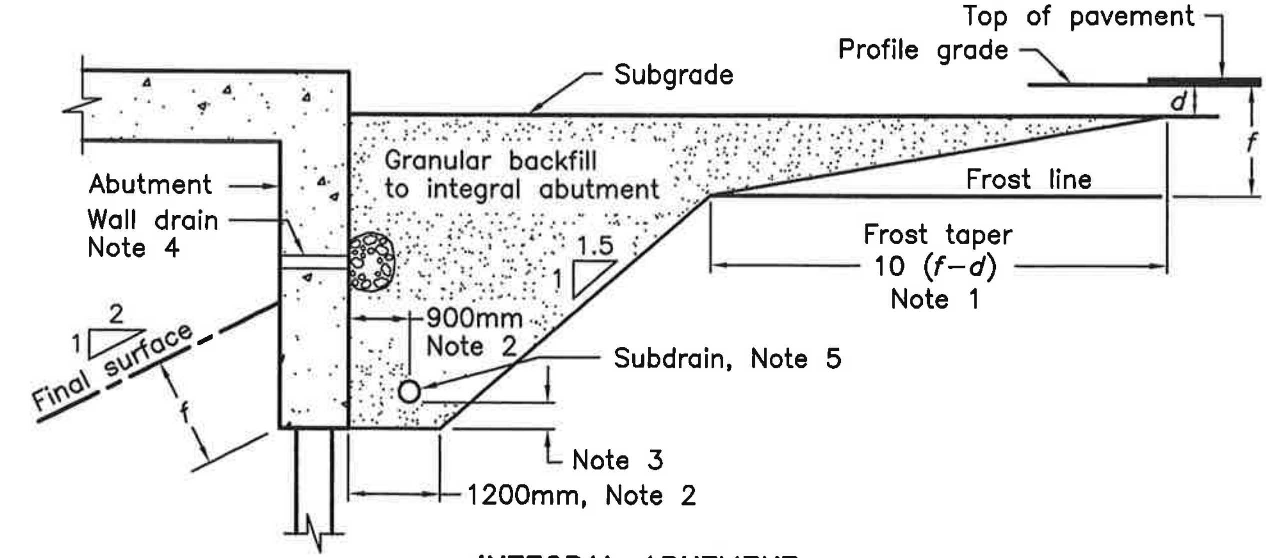
A Flange plates shall be according to CSA–G40.20/G40.21, Grade 300W.

B Welding shall be according to CSA–W59.

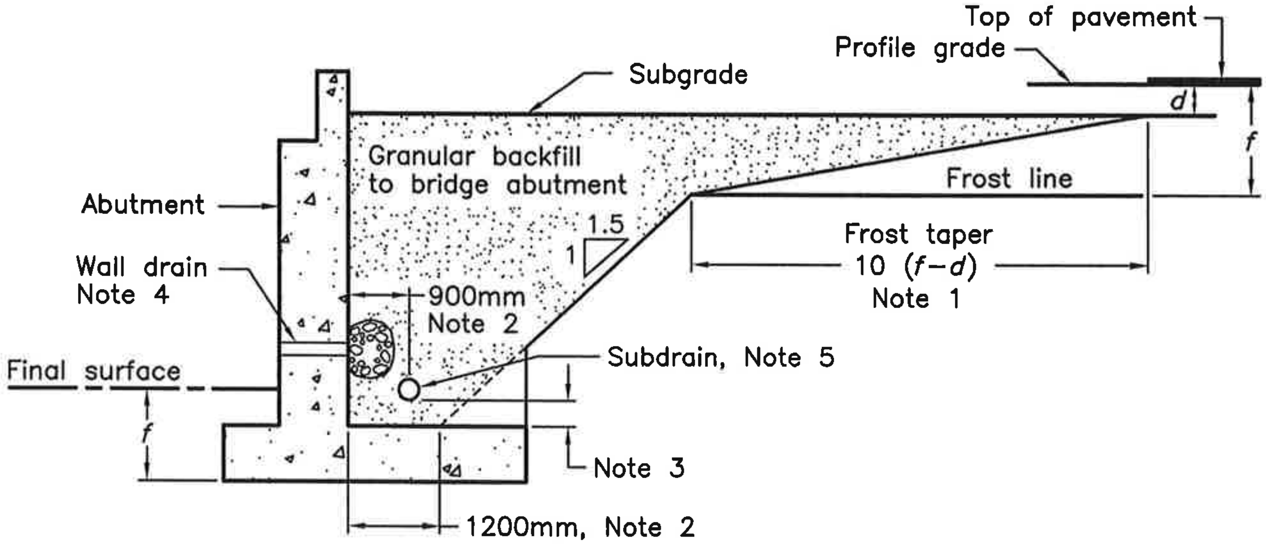
C Driving shoe Type I shall be used unless Type II is specified.

D All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2005	Rev 1
FOUNDATION PILES STEEL H–PILE DRIVING SHOE	OPSD – 3000.100	



INTEGRAL ABUTMENT



ABUTMENT

NOTES:

1 *d* = depth of combined base and subbase courses.
 f = roadbed depth of frost penetration as specified.
2 Dimensions perpendicular to back face of abutment.
3 Height to be consistent with positive drainage of subdrain as specified.
4 Where specified, wall drains shall be installed according to OPSD−3190.100.
5 150mm dia perforated pipe subdrain wrapped with geotextile.
A Lateral limits of granular backfill to bridge abutment to be inside face to inside face of retaining wall or wingwall. Frost taper shall extend the full width of the fill unless interrupted by the retaining wall or wingwall.
B Sections shown are parallel to centreline of roadway.
C Subdrain to be installed with a 2% gradient behind wall.
D All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2005	Rev 0
WALLS ABUTMENT, BACKFILL MINIMUM GRANULAR REQUIREMENT	OPSD − 3101.150	

Appendix G

Limitations of Report

LIMITATIONS OF REPORT

This report is intended solely for the Client named. The material in it reflects our best judgment in light of the information available to Coffey Geotechnics Inc. (Coffey) at the time of preparation. Unless otherwise agreed in writing by Coffey, it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity, it is written to be read in its entirety.

The conclusions and recommendations given in this report are based on information determined at the testhole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the testhole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the testhole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. Coffey accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.