coffey geotechnics
SPECIALISTS MANAGING THE EARTH

30M12-338

FOUNDATION INVESTIGATION AND DESIGN REPORT
PROPOSED WIDENING OF CLARK BOULEVARD UNDERPASS
OVER HIGHWAY 410
BRAMPTON, REGION OF PEEL

AECOM

GEOTMARK00170AA
September 22, 2011

Coffey Geotechnics Inc.
351 Steelcase Rd. W., Unit 10 Markham Ontario L3R 4H9 Canada

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

September 22, 2011

AECOM
300 Town Centre Blvd., Suite 300
Markham, Ontario
L3T 7W3

Attention: Mr. Travis Brown

Dear Mr. Brown:

RE: – Foundation Investigation Report Proposed Widening of Clark Boulevard Underpass over Highway 410, Brampton, Region of Peel

Please find attached the results of our foundation investigation and design report relating to the above noted site.

If you have any comments or enquiries please contact the undersigned.

For and on behalf of Coffey Geotechnics Inc.

Chris Pressdee
Geotechnics Manager, Markham

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

FOUNDATION INVESTIGATION REPORT PROPOSED WIDENING OF CLARK BOULEVARD UNDERPASS OVER HIGHWAY 410 BRAMPTON, REGION OF PEEL

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CONTENTS

Foundation Investigation Report
Proposed Widening of Clark Boulevard Underpass over Highway 410,
Brampton, Region of Peel

1 INTRODUCTION

This project involves the widening of Clark Boulevard from Rutherford Road to 500 m east of Dixie Road in Brampton, Ontario. Coffey Geotechnics Inc. (Coffey) was retained by AECOM to perform the foundation investigation for the proposed bridge widening of the Clark Boulevard underpass over Highway 410.

The existing bridge is a 109 m long and 21 m wide, two-span structure, with a skew angle of 62° relative to the centreline of Highway 410.

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

An earlier foundation investigation report prepared by the Pavement and Foundation Design Section of the Ministry of Transportation Ontario (MTO) for the detailed design of the existing bridge is included in this report to provide a more thorough summary of the Site conditions. This previous report, dated January 8, 1982, can be found in Appendix E.

The findings of the investigations are presented in this report.

2 SITE DESCRIPTION AND PHYSIOGRAPHY

The Site is located in the physiographic region known as the "Peel Plain" according to "The Physiography of Southern Ontario". The region has a gradual and fairly uniform slope towards Lake Ontario and is well-drained by Credit River, Oakville and Etobicoke Creeks, which have cut deep valleys into the overburden. As a result, there is no large undrained depression, swamp or bog in the whole area, although in many of the interstream areas, the drainage is still imperfect.

The subsurface conditions at the project location generally consist of glacial till overlying grey shale (Georgian Bay Formation) bedrock.

3 FIELD AND LABORATORY WORK

The fieldwork for the foundation investigation was performed between June 16 and August 17, 2010, and consisted of drilling and sampling of eleven (11) boreholes (Boreholes 101 through 110 and 102A). Groundwork Drilling Inc. of Etobicoke, Ontario carried out the drilling, testing and sampling work of all boreholes. Fieldwork was conducted under the direction and supervision of technical staff from Coffey. Upon completion, each borehole was backfilled with a mixture of bentonite/cement, as per MTO procedures.

Borehole 102 was terminated at 5.0 m (El. 210.8 m) below the existing ground level due to auger refusal and Borehole 102A was subsequently advanced at a horizontal distance of about 1 m from Borehole 102 to confirm the refusal depth. Borehole 102A was advanced to 4.6 m by straight augering where sampling began.

The borehole information from the current investigation is summarized in Table 3.1.

Table 3.1 – Borehole Information

Borehole No.	Ground Elevation (m)	Depth (m)	Location	Boring Method	Depth/Elevation of Tip of Piezometer (m)
101	222.1	14.7	West Abutment	Solid Stem Auger	14.6 / 207.5
102	215.8	5.0	West Abutment	Solid Stem Auger	N/A
102A	215.8	13.9	West Abutment	Solid Stem Auger, NQ Coring	9.1 / 206.7
103	215.2	9.4	Centre Pier	Solid Stem Auger	9.1 / 206.1
104	215.9	9.3	Centre Pier	Solid Stem Auger	9.1 / 206.8
105	216.5	14.8	East Abutment	Solid Stem Auger, NQ Coring	14.8 / 201.7
106	222.9	15.5	East Abutment	Solid Stem Auger	15.2 / 207.7
107	216.9	6.6	West Approach	Solid Stem Auger	N/A
108	216.1	6.3	West Approach	Solid Stem Auger	N/A
109	216.7	6.2	East Approach	Solid Stem Auger	N/A
110	217.6	11.4	East Approach	Solid Stem Auger	11.3 / 206.3

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 Coffey's technical staff using an existing benchmark.

Sampling in the boreholes was effected at frequent intervals of depth by the Standard Penetration Test method (SPT), in general accordance with ASTM D1586. The 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 granular (cohesionless) soils (gravels, sands and coarse silts) or the consistency of cohesive soils (clays and clayey silts).

Water level observations in the open boreholes (or casing) were made during the drilling and at completion of each borehole.

Piezometers were installed at the bottom of selected boreholes to determine the groundwater levels over a prolonged period of time, without interference from surface water.

The soil and rock core samples were transported to our geotechnical laboratory in Toronto for further examination and classification. A laboratory testing programme, consisting of natural moisture content determinations, Atterberg Limits tests and grain size analyses, was performed on selected representative samples. Two (2) rock core samples from Boreholes 102A and 105 were forwarded to the laboratory of Queen's University in Kingston, Ontario, where the samples were tested for their unconfined compressive strength (UCS) and density. The results of the laboratory tests are presented on the appropriate Record of Borehole Sheets.

4 SUMMARIZED SUBSURFACE CONDITIONS

In general, the subsurface conditions at the project site consist of fill and embankment fills, surficial granular deposits (at some locations) and glacial till deposits (both non-cohesive and cohesive) overlying grey shale bedrock of the Georgian Bay Formation.

All the boreholes contacted fill and embankment fill which were found to extend to depths/elevations 0.7 – 8.4 m / 215.4 – 212.5 m. Below the fill materials, Boreholes 101, 106 and 107 contacted upper granular deposits at El. 215.2 to 212.9 m. Borehole 107 was terminated in the upper granular deposit at a depth of 6.6 m or El. 210.3 m. The remaining boreholes encountered below the fill, embankment fill and the upper granular deposits, a glacial deposit, which ranges in composition from silty sand/sandy silt to clayey silt/silty clay till. The glacial till was encountered at depths ranging between 0.7 and 11.4 m below the ground surface or at El. 215.4 and 210.7 m. Boreholes 101, 103, 104, 106, 108 and 109 were terminated in this deposit at depths 6.2 – 15.5 m or at El. 210.5 – 205.8 m, after penetrating it for a vertical distance of 3.3 to 8.3 m. In Boreholes 102A, 105 and 110 where the boreholes were extended deeper, the surface of the bedrock was contacted at 10.7 m or at El. 206.9 – 205.1 m. Bedrock was proven by NQ coring in Boreholes 102A and 105 upon auger refusal.

Boreholes 1, 2, and 3 advanced for the previous investigation encountered a sand and gravel deposit to the full depth of the investigation in Borehole 1 and clayey silt to silty clay till deposit in Boreholes 2 and 3 from the ground surface at the time of investigation (i.e. 1981). In combination with the current investigation, the sand and gravel encountered in Borehole 1 can be classified as part of the upper granular deposits encountered in Boreholes 101, 106 and 107.

Subsurface conditions at the site are discussed in the following sections. Details of the stratigraphy encountered in the boreholes from the current investigation are presented on the Records of Borehole Sheets and in the soil strata drawings in Appendix A. Details from the previous investigation can be found within the report attached in Appendix E. The following paragraphs are only meant to complement these data.

4.1 Topsoil

A layer of topsoil ranging from 0.1 to 0.2 m in thickness was contacted in Boreholes 102 and 107 to 110 at ground surface.

4.2 Fill

4.2.1 Pavement Structure

Pavement structure was encountered in Boreholes 101 and 106, which were drilled on top of the embankment of Clark Boulevard. The pavement structure consists of 220 and 260 mm thick asphalt with 0.5 m thick granular sand and gravel base course and a 0.3 m to 0.6 m thick sand sub-base course.

Standard Penetration tests performed within the granular base and sub-base course layers yielded N-values between 11 and 38 blows/0.3 m, indicating the pavement fill is compact to dense in relative density.

4.2.2 Embankment Fill

Boreholes 101, 106, 107, 108, 109 and 110 contacted fill which constituted the embankment of Clark Boulevard. This embankment fill was found to extend to depths ranging from 0.7 m to 8.4 m below the top of the embankment, or to El. 215.4 – 212.5 m.

In general, the embankment fill at the borehole locations was found to consist of clayey silt with trace to some sand and gravel. Based on recorded Standard Penetration test results (N-values) which range from 4 to 48 blows/0.3 m, the consistency of the embankment fill material can be described as firm to hard. Occasional higher N-values were also recorded but these are believed to be due to the presence of cobbles in the fill.

Sand fill and gravelly sand fill were contacted in Borehole 108 and top portion of Borehole 107 to depths of 0.7 m and 0.8 m, respectively. From recorded N-values of 13 and 21 blows/0.3 m, the relative density of this material is described as compact.

4.2.3 Other Fill

In Boreholes 102, 103, 104 and 105, fill materials (i.e. unrelated to pavement structure or embankment fill) were contacted. These fill materials were found to extend to depths of 1.0 m – 3.7 m (typically 1.0 m – 1.3 m) below the ground surface or to El. 214.9 – 212.8 m. The composition of these fill materials was found to widely range from granular to cohesive (clayey) soils. Based on N-values which range from 2 to 43 blows/0.3 m, the relative density of the granular type fills can be described as very loose to dense while the consistency of the cohesive fills can be described as stiff to hard.

4.3 Upper Granular Deposits

Boreholes 1, 101, 106 and 107 contacted granular deposits at o.g. (original grade) level or below the embankment fill at Elevations between 215.8 m and 212.9 m. These upper granular deposits extended to El. 213.0 – 210.3 m and are described in the following paragraphs.

4.3.1 Sand and Gravel / Gravelly Sand

A sand and gravel to gravelly sand deposit was encountered in Boreholes 1 and 101 at Elevations 215.8 m and 215.2 m. In Borehole 101, this deposit was found to be 3.0 m thick while in Borehole 1 it extended to the full depth of the borehole (i.e. 4.2 m), where auger refusal was encountered.

The grain-size distribution of three (3) samples retrieved from this deposit in Borehole 1 is presented in Figure 1 in Appendix E, as summarized below:

Gravel: 47 – 66 %

Sand: 21 – 41 %

Silt: 5 – 11 %

Clay: 2 – 5 %

This deposit is a granular (i.e. non-cohesive) material.

N-values obtained from the Standard Penetration tests performed within this layer range between 28 and in excess of 100 blows/0.3 m, indicating that this granular (i.e. non-cohesive) material is compact to very dense, but typically very dense.

4.3.2 Silty Sand to Sand

Boreholes 101, 106 and 107 contacted a layer of silty sand to sand at depths between 4.0 m and 9.9 m below the ground surface (between El. 214.5 and 212.2 m). Traces of gravel and clay and occasional clayey silt pockets were found within the deposit. Borehole 107 was terminated within the silty sand to sand deposit at 6.6 m depth (El. 210.3 m).

Grain size distribution of five (5) samples retrieved from the deposit was determined in the laboratory. The results are shown in Figure B-1 in Appendix B and are summarized as follows:

Gravel: 0 – 24 %

Sand: 39 – 87 %

Silt & Clay: 13 – 37 %

Standard Penetration tests performed within this deposit yielded N-values between 39 and in excess of 100 blows/0.3 m. These results indicate that this granular (i.e. non-cohesive) deposit is dense to very dense, typically very dense.

4.4 Glacial Till

A glacial till deposit was contacted in all boreholes except at Boreholes 1 and 107 which were terminated within the overlying granular deposits. The glacial till deposit can be subdivided into two (2) major groups: a basically non-cohesive (granular) silty sand to sandy silt till and a basically cohesive clayey silt to silty clay till.

4.4.1 Silty Sand to Sandy Silt Till

Boreholes 103, 108 and 109 encountered a silty sand to sandy silt till deposit below the fill at El. 215.4 to 213.7 m. In Borehole 103, this deposit was contacted from 1.3 to 5.0 m, below the existing grade, and is underlain by cohesive till while Boreholes 108 and 109 were terminated within this deposit at 6.3 m and 6.2 m below the ground surface (El. 209.8 and 210.5 m), respectively.

Grain-size distribution of five (5) samples retrieved from the deposit is presented in Figure B-2, Appendix B.

Gravel: 6 – 20 %

Sand: 35 – 58 %

Silt: 24 – 47 %

Clay: 5 – 10 %

This glacial deposit is a heterogeneous mixture of gravel, sand, silt and clay with silt and sand being major constituents. Cobbles and boulders should be expected within this deposit due to its mode of deposition.

Standard Penetration tests performed in this granular (i.e. non-cohesive) till deposit yielded N-values between 35 and in excess of 100 blows/0.3 m, indicating the deposit is dense to very dense.

A silty sand layer within the upper zone of the silty sand to sandy silt till deposit was contacted in Borehole 109. The grain-size distribution of a sample retrieved from this layer is shown in Figure B-3 in Appendix B and summarized below.

Gravel: 10 %

Sand: 78 %

Silt & Clay: 12 %

4.4.2 Clayey Silt to Silty Clay Till

All boreholes except for Boreholes 1, 107, 108 and 109 encountered a clayey silt to silty clay till deposit at depths between 0 and 11.4 m below the ground level at the time of investigation or Elevations 215.0 m and 210.2 m. This deposit extends to a depth of 10.7 m in Boreholes 102A, 105 and 110, where it is underlain by bedrock while in Boreholes 2, 3, 101, 102, 103, 104 and 106, the clayey silt to silty clay till extended to the remaining depths of the boreholes. Boreholes 2, 3, 101 and 102 were terminated in this deposit due to auger refusal, probably on a boulder (e.g. Borehole 2) or possibly on bedrock (e.g. Boreholes 3 and 101).

Grain-size analyses were performed on 13 samples (including four samples from MTO's 1981-82 investigation)* retrieved from the deposit and the distribution is summarized as follows (See Figure B-4 in Appendix B and Figure 2 in Appendix E):

Gravel: 1 – 34 %

Sand: 10 – 43 %

Silt: 27 – 50 %

Clay: 7 – 29 %

*There are actually five samples with grain-size analyses from the MTO 1981-82 investigation. However, one of them appears to contain error and was not included here-in.

This glacial deposit is a heterogeneous mixture of clayey silt with some sand and gravel size particles. It shows a very wide range of grain size distribution, as indicated by the grain-size distribution in Figure B-4. Cobbles and boulders can be expected within this deposit due to its mode of deposition, as well as refusal in the boreholes (e.g. Borehole 102).

Atterberg Limits tests were also performed on eleven (11) samples retrieved from the deposit. The results are shown in Figure B-5 in Appendix B and Figure 3 in Appendix E.

Liquid Limit: 14 – 37 %

Plastic Limit: 10 – 20 %

Plasticity Index: 3 – 17

These results indicate a cohesive soil of low plasticity.

N-values recorded from Standard Penetration tests performed within the deposit range from 32 to in excess of 100 blows/0.3 m, indicating that the deposit has a hard consistency.

4.5 Bedrock

A grey shale bedrock was encountered in Boreholes 102A, 105 and 110 and was proven by NQ coring in Boreholes 102A and 105, as indicated in the table below. Bedrock condition was not mentioned in the previous MTO report of 1982 (i.e. bedrock was not reached).

Table 4.5.1 – Bedrock Elevation and Condition

Borehole	Ground Elevation (m)	Inferred Bedrock Depth/Elevation (m)	T.C.R (%) *	R.Q.D. (%) **
101	222.1	Not encountered Auger Refusal at 14.7 / 207.4 m		
102	215.8	Not encountered		
102A	215.8	10.7 / 205.1	79 – 100	21 – 29
103	215.2	Not encountered		
104	215.9	Not encountered		

Borehole	Ground Elevation (m)	Inferred Bedrock Depth/Elevation (m)	T.C.R (%) *	R.Q.D. (%) **
105	216.5	10.7 / 205.8	93 – 100	54 – 83
106	222.9	Not encountered		
107	216.9	Not encountered		
108	216.1	Not encountered		
109	216.7	Not encountered		
110	217.6	10.7 / 206.9 *** (not proven)		

* T.C.R. = Total Core Recovery
** R.Q.D. = Rock Quality Designation
*** inferred bedrock

Boreholes 102A and 105 were advanced into the bedrock for a vertical distance of about 3.2 m and 4.1 m by NQ coring. As shown in the table above, the percentage of recovery was 79 to 100% while the RQD values vary from 21 to 83%. These results indicate rock quality from poor to good. Borehole 110 was advanced into the inferred bedrock by about 0.7 m by augering.

Two (2) unconfined compression tests were performed on selected intact rock core samples from Boreholes 102A and 105. Tests yielded unconfined compression strengths (U.C.S.) of 51.2 MPa and 11.5 MPa, as shown on Table 4.5.2. From these results, the strength of the shale bedrock can be classified as weak to medium strong.

The laboratory testing results for rock core samples are attached in Appendix B.

Table 4.5.2 – Unconfined Compression Test Data

Borehole & Sample No.	Approximate Depth (m)	Approximate Elevation (m)	Density (kN/m3)	Unconfined Compressive Strength (MPa)
102A-RC7	12.9	202.9	25.3	51.2
105-RC15	14.4	202.1	25.3	11.5

4.6 Groundwater Conditions

Groundwater levels were observed in open boreholes while drilling and upon completion of each borehole. These results may not represent the stabilized groundwater conditions, especially in Boreholes 102A and 105 where NQ coring was used (i.e. water introduced into the boreholes). The observations made in the boreholes are summarized in Table 4.6.1 and presented on the Record of Borehole Sheets in Appendix A.

Table 4.6.1 - Groundwater conditions

Borehole	Ground Elevation (m)	Depth / Elevation of the Tip of Piezometer (m)	Date	Water Level Depth / Elevation (m)
101	222.1	14.6 / 207.5	July 23, 2010 (32 days after completion)	7.5 / 214.6
102	215.8	N/A	June 18, 2010 (on completion)	3.4 / 212.4**

Borehole	Ground Elevation (m)	Depth / Elevation of the Tip of Piezometer (m)	Date	Water Level Depth / Elevation (m)
102A*	215.8	9.1 / 206.7	July 23, 2010 (15 days after completion)	1.7 / 214.1
103	215.2	9.1 / 206.1	July 23, 2010 (36 days after completion)	7.0 / 208.2**
104	215.9	9.1 / 206.8	July 23, 2010 (37 days after completion)	2.8 / 213.1
105*	216.5	14.8 / 201.7	July 23, 2010 (35 days after completion)	2.6 / 213.9
106	222.9	15.2 / 207.7	July 23, 2010 (31 days after completion)	9.4 / 213.5
107	216.9	N/A	June 29, 2010 (on completion)	2.4 / 214.5**
108	216.1	N/A	August 17, 2010 (on completion)	4.0 / 212.1**
109	216.7	N/A	June 29, 2010 (on completion)	3.0 / 213.7**
110	217.6	11.3 / 206.3	July 23, 2010 (30 days after completion)	7.5 / 210.1**
1	215.8	N/A	December 8, 1981 (overnight water level reading)	0.9 / 214.9
2	215.0	N/A	December 8, 1981 (overnight water level reading)	5.5 / 209.5**
3	214.5	N/A	December 9, 1081 (overnight water level reading)	0.6 / 213.9

*water used for coring
**not stabilized

In the eight piezometers installed across the site, groundwater levels were measured 15 to 37 days after completion but typically 30 – 37 days. The recorded groundwater levels in the piezometers range from El. 214.6 to 208.2 m, showing a wide variation. The lower water levels of El. 210.1 and 208.2 m were recorded in Boreholes 110 and 103, respectively. In the remaining six piezometers, the recorded levels range from El. 214.6 to 212.1 m. Based on these data, at the time of the investigation, the groundwater table across the site was generally between El. 214.5 and 213.5 m.

It should be noted that the observed groundwater levels represent the conditions at the time of our investigation and they are subject to fluctuations, both seasonally and in response to major weather events.

For and on behalf of Coffey Geotechnics Inc.

for Winnie Chan, E.I.T.

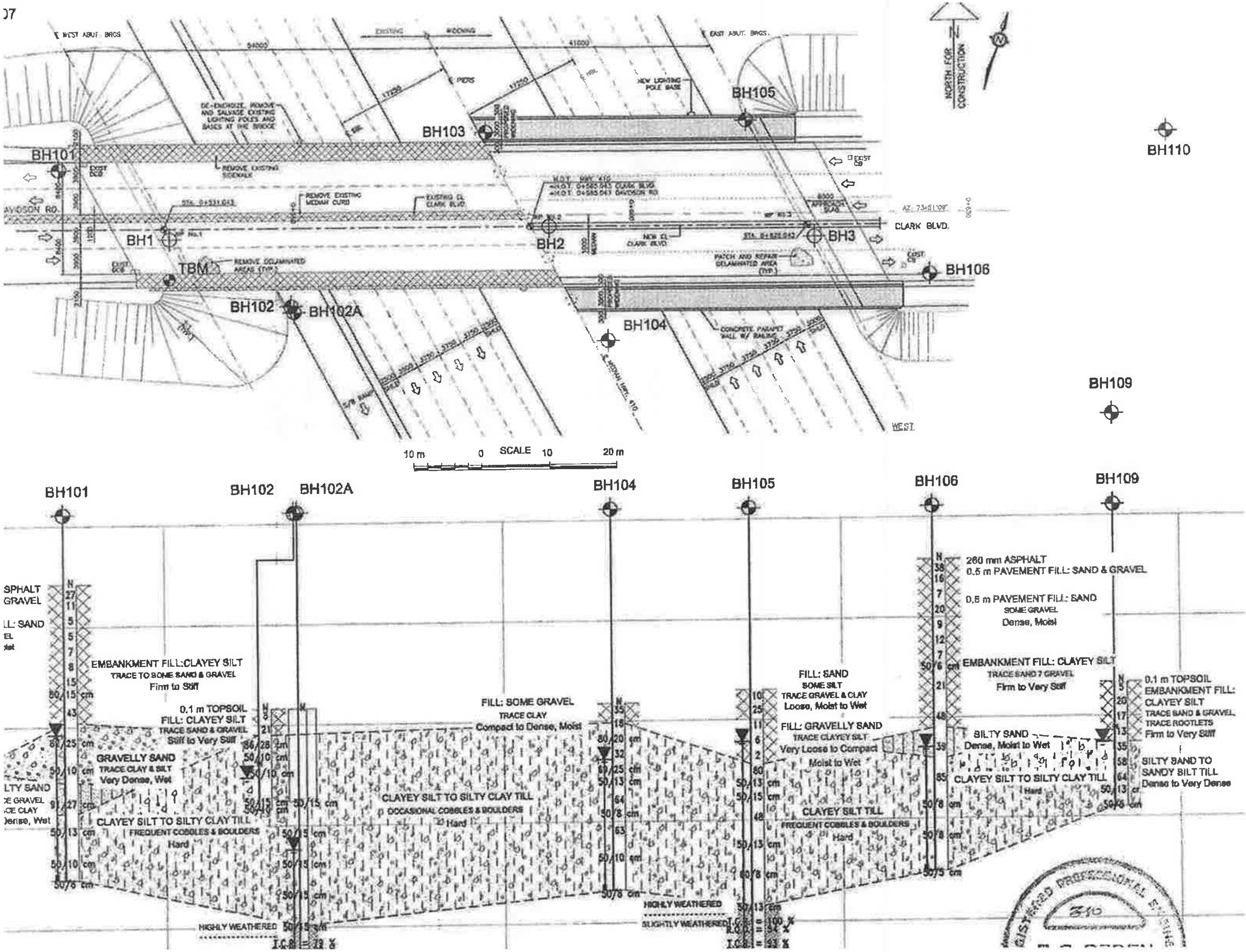
LICENSED PROFESSIONAL ENGINEER
RAMON MIRAN...
PROVINCE OF ONTARIO

Ramon Miranda, P.Eng.

REGISTERED PROFESSIONAL ENGINEER
Z. S. OZDEN
PROVINCE OF ONTARIO

Zuhtu Ozden, P.Eng.

Drawing



07
WEST ABUT. BRGS.
EAST ABUT. BRGS.
EXISTING
WIDENING
NORTH FOR CONSTRUCTION
BH101
BH102
BH102A
BH103
BH104
BH105
BH106
BH109
BH110
BH1
BH2
BH3
TBM
DAVIDSON RD.
CLARK BLVD.
NEW LIGHTING POLE BASE
REMOVE EXISTING SIDEWALK
REMOVE EXISTING MEDIAN CURB
EXISTING CL CLARK BLVD.
NEW CL CLARK BLVD.
REMOVE DELAMINATED AREAS (TYP.)
PATCH AND REPAIR DELAMINATED AREA (TYP.)
CONCRETE PARAPET WALL W/ RAILING
WEST
SCALE
10 m
0
10
20 m
SPHALT
GRAVEL
LL: SAND
EMBANKMENT FILL:CLAYEY SILT
TRACE TO SOME SAND & GRAVEL
Firm to Stiff
0.1 m TOPSOIL
FILL: CLAYEY SILT
TRACE SAND & GRAVEL
Stiff to Very Stiff
GRAVELLY SAND
TRACE CLAY & SILT
Very Dense, Wet
LTY SAND
CLAYEY SILT TO SILTY CLAY TILL
FREQUENT COBBLES & BOULDERS
Hard
FILL: SOME GRAVEL
TRACE CLAY
Compact to Dense, Moist
CLAYEY SILT TO SILTY CLAY TILL
OCCASIONAL COBBLES & BOULDERS
Hard
FILL: SAND
SOME SILT
TRACE GRAVEL & CLAY
Loose, Moist to Wet
FILL: GRAVELLY SAND
TRACE CLAYEY SILT
Very Loose to Compact
Moist to Wet
CLAYEY SILT TILL
FREQUENT COBBLES & BOULDERS
Hard
260 mm ASPHALT
0.5 m PAVEMENT FILL: SAND & GRAVEL
0.5 m PAVEMENT FILL: SAND
SOME GRAVEL
Dense, Moist
EMBANKMENT FILL: CLAYEY SILT
Firm to Very Stiff
SILTY SAND
Dense, Moist to Wet
CLAYEY SILT TO SILTY CLAY TILL
Hard
0.1 m TOPSOIL
EMBANKMENT FILL:
CLAYEY SILT
TRACE SAND & GRAVEL,
TRACE ROOTLETS
Firm to Very Stiff
SILTY SAND TO
SANDY SILT TILL
Dense to Very Dense
HIGHLY WEATHERED
SLIGHTLY WEATHERED

Appendix A

Record Borehole Sheets

GEOTMARK00170AA: Clark Boulevard Widening, Brampton, Ontario

RECORD OF BOREHOLE No 101

METRIC

GWP ______ LOCATION See Borehole Location Plan ORIGINATED BY AA

DIST ______ HWY ______ BOREHOLE TYPE Solid Stem Auger COMPILED BY WC

DATUM Geodetic DATE 6/21/2010 CHECKED BY ZO

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION
222.1	GROUND SURFACE
0.0	220 mm **ASPHALT** 0.5 m **PAVEMENT FILL:** Sand and Gravel **PAVEMENT FILL:** Sand, some gravel brown, compact, moist
220.8 / 1.3	**EMBANKMENT FILL:** Clayey Silt tr. to some sand and gravel occ. silt seams, tr. rootlets brown, firm to stiff freq. cobbles, hard
215.2 / 6.9	**GRAVELLY SAND** tr. clay and silt brown, v. dense, wet
212.2 / 9.9	**SILTY SAND** tr. gravel, tr. clay grey, v. dense, wet
210.7 / 11.4	**CLAYEY SILT TO SILTY CLAY TILL** tr. to some shale fragments grey, hard
207.4 / 14.7	End of Borehole.

SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	27	
2	SS	11	
3	SS	5	
4	SS	5	
5	SS	7	
6	SS	8	
7	SS	15	
8	SS	50 / 15 cm	
9	SS	43	
10	SS	52 / 25 cm	
11	SS	50 / 10 cm	
12	SS	91 / 27 cm	3 64 27 6
13	SS	50 / 13 c	34 28 27 11
14	SS	50 / 10 cm	19 29 34 18
15	SS	50 / 8 cm	Auger Refusal @ 14.7 m

ELEVATION SCALE: 222, 221, 220, 219, 218, 217, 216, 215, 214, 213, 212, 211, 210, 209, 208

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/m^3

Continued Next Page

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

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

GEOTMARK00170AA: Clark Boulevard Widening, Brampton, Ontario

RECORD OF BOREHOLE No 101

METRIC

GWP ____ LOCATION See Borehole Location Plan ORIGINATED BY AA

DIST ____ HWY ____ BOREHOLE TYPE Solid Stem Auger COMPILED BY WC

DATUM Geodetic DATE 6/21/2010 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	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
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
207.1											
	Water level @ 7.8 m (not stabilized)* upon completion. Piezometer installed to 14.6 m. Date / Measured Water Level July 23, 2010 / 7.5 m										

Numbers refer to Sensitivity — (%) STRAIN AT FAILURE

GEOTMARK00170AA: Clark Boulevard Widening, Brampton, Ontario

RECORD OF BOREHOLE No 102

METRIC

GWP ______ LOCATION See Borehole Location Plan ORIGINATED BY AA

DIST ______ HWY ______ BOREHOLE TYPE Solid Stem Auger COMPILED BY WC

DATUM Geodetic DATE 6/18/2010 CHECKED BY ZO

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
215.6	GROUND SURFACE						
0.0	0.1 m TOPSOIL FILL: Clayey Silt mixed with topsoil, tr. sand and gravel brown, stiff to v. stiff	1	SS	8			
		2	SS	21		215	
214.5 1.3	CLAYEY SILT TO SILTY CLAY TILL freq. cobbles and boulders brown, hard	3	SS	88 / 28 cm		214	Auger grinding below 1.5 m 1 28 50 21
	cobbles and boulders	4	SS	50 / 10 cm			
						213	
	cobbles and boulders	5	SS	50 / 10 cm	▼*		
						212	
	cobbles and boulders	6	SS	50 / 15 cm			Augering hard
210.8	shale fragments	7	SS	50 / 10 cm		211	Auger refusal @ 4.9 m
5.0	End of Borehole. Water level @ 3.4 m (not stabilized)* upon completion.						

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/m^3

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

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

GEOTMARK00170AA: Clark Boulevard Widening, Brampton, Ontario

RECORD OF BOREHOLE No 103

METRIC

GWP ______ LOCATION See Borehole Location Plan ORIGINATED BY AA

DIST ______ HWY ______ BOREHOLE TYPE Solid Stem Auger COMPILED BY WC

DATUM Geodetic DATE 6/17/2010 CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
215.2	GROUND SURFACE				
0.0	FILL: Sand, some gravel tr. clay, brown, compact, moist	1	SS	20	
214.5 / 0.7	FILL: Clayey Silt tr. sand and gravel brown, hard	2	SS	43	
213.9 / 1.3	SILTY SAND TO SANDY SILT TILL freq. cobbles brown, v. dense, moist	3	SS	50 / 13 cm	Auger grinding below 1.3 m
		4	SS	50 / 13 cm	20 50 24 6
		5	SS	50 / 13 cm	
		6	SS	50 / 13 cm	14 35 43 8
		7	SS	50 / 13 cm	
210.2 / 5.0	CLAYEY SILT TO SILTY CLAY TILL freq. cobbles, grey, hard	8	SS	50 / 13 cm	
		9	SS	50 / 15 cm	
		10	SS	78 / 28 cm	22 34 27 17
205.8 / 9.4		11	SS	50 / 13 cm	

End of Borehole.
Borehole was dry upon completion.

Piezometer installed to 9.1 m.

Date / Measured Water Level
July 23, 2010 / 7.0 m

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

ELEVATION SCALE 215 214 213 212 211 210 209 208 207 206

+3, ×3: Numbers refer to Sensitivity

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

GEOTMARK00170AA: Clark Boulevard Widening, Brampton, Ontario

RECORD OF BOREHOLE No 104

METRIC

GWP	LOCATION See Borehole Location Plan	ORIGINATED BY AA
DIST HWY	BOREHOLE TYPE Solid Stem Auger	COMPILED BY WC
DATUM Geodetic	DATE 6/16/2010	CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
215.9	GROUND SURFACE					
0.0	FILL: Sand, some gravel tr. clay, brown, compact to dense, moist	1	SS	35		
	topsoil inclusion	2	SS	18	215	
214.9 / 1.0	CLAYEY SILT TO SILTY CLAY TILL occ. cobbles and boulders hard	3	SS	80 / 20 cm	214	Auger grinding on poss. cobbles
		4	SS	32	213	
		5	SS	69 / 25 cm		17 19 37 27 Auger grinding below 3.1 m
		6	SS	50 / 13 cm	212	
	brown / grey	7	SS	64	211	32 17 30 21
		8	SS	50 / 8 cm	210	
	shale fragments	9	SS	63	209	
		10	SS	50 / 10 cm	208	
206.8 / 9.1		11	SS	50 / 8 cm	207	

End of Borehole.
Borehole was dry upon completion.
Borehole caved-in @ 9.1 m upon completion.

Piezometer installed to 9.1 m

Date / Measured Water Level
July 23, 2010 / 2.8 m

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

GEOTMARK00170AA: Clark Boulevard Widening, Brampton, Ontario

RECORD OF BOREHOLE No 105

METRIC

GWP	LOCATION See Borehole Location Plan		ORIGINATED BY AA
DIST	HWY	BOREHOLE TYPE Solid Stem Auger, NQ Coring	COMPILED BY WC
DATUM Geodetic		DATE 6/18/2010	CHECKED BY ZO

SOIL PROFILE

ELEV. / DEPTH	DESCRIPTION
216.5	GROUND SURFACE
216.5 / 0.0	FILL: Sand, some silt tr. gravel and clay brown, loose, moist to wet
215.8 / 0.7	FILL: Clayey Silt tr. oragnics, tr. gravel brown, v. stiff
215.2 / 1.3	FILL: Gravelly Sand tr. clayey silt pockets brown, v. loose to compact moist to wet
212.8 / 3.7	CLAYEY SILT TO SILTY CLAY TILL freq. cobbles and boulders grey, hard
205.8 / 10.7	highly weathered BEDROCK grey shale, slightly weathered interbedded with thin layers of limestone and siltstone
201.7 / 14.8	

Continued Next Page

SAMPLES

NUMBER	TYPE	"N" VALUES
1	SS	10
2	SS	25
3	SS	11
4	SS	6
5	SS	2
6	SS	80
7	SS	50 / 13 cm
8	SS	50 / 15 cm
9	SS	48
10	SS	50 / 13 cm
11	SS	50 / 8 cm
12	SS	50 / 13 cm
13	RC	C.R. = 100 %, R.Q.D. = 54 %
14	RC	C.R. = 98 %, R.Q.D. = 58 %
15	RC	C.R. = 100 %, R.Q.D. = 83 %

GROUND WATER CONDITIONS · ELEVATION SCALE (216 – 202)

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

Elevation (approx.)	Remark
215.7	Spoon wet below 0.8 m
214.2	Augering hard
212.7	16 17 41 26
212.0	Auger grinding @ 4.5 m
209.0	20 28 32 20
202.0	U.C.S. = 11.5 MPa

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

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

GEOTMARK00170AA: Clark Boulevard Widening, Brampton, Ontario

RECORD OF BOREHOLE No 105

METRIC

GWP ______ LOCATION See Borehole Location Plan ORIGINATED BY AA

DIST ______ HWY ______ BOREHOLE TYPE Solid Stem Auger, NQ Coring COMPILED BY WC

DATUM Geodetic DATE 6/18/2010 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH (kPa) O UNCONFINED + FIELD VANE ● 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
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
201.5	End of Borehole. Piezometer installed to 14.8 m. Date / Measured Water Level July 23, 2010 / 2.6 m										

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

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

Ministry of Transportation
Ontario

Foundation Design

GEOTMARK00170AA: Clark Boulevard Widening, Brampton, Ontario

RECORD OF BOREHOLE No 106

METRIC

GWP ______ LOCATION See Borehole Location Plan ORIGINATED BY AA

DIST ______ HWY ______ BOREHOLE TYPE Solid Stem Auger COMPILED BY WC

DATUM Geodetic DATE 6/22/2010 CHECKED BY ZO

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
222.9	GROUND SURFACE				
0.0	260 mm ASPHALT 0.5 m PAVEMENT FILL: Sand and Gravel PAVEMENT FILL: Sand, some gravel brown, dense, moist	1	SS	38	
		2	SS	16	
221.8 1.1	EMBANKMENT FILL: Clayey Silt tr. sand and gravel brown, firm to v. stiff sandy silt layer/lense	3	SS	7	
		4	SS	20	
		5	SS	9	
		6	SS	12	
		7	SS	7	
		8	SS	50 / 6 cm	
		9	SS	21	
		10	SS	48	possible gravel and cobbles
214.5 8.4	SILTY SAND occ. clayey silt pockets brown, dense, moist to wet	11	SS	39	24 39 29 8
213.0 9.9	CLAYEY SILT TO SILTY CLAY TILL brown, hard	12	SS	85	
		13	SS	50 / 8 cm	25 38 28 9
		14	SS	50 / 8 cm	
207.9					

Columns also include: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (222, 221, 220, 219, 218, 217, 216, 215, 214, 213, 212, 211, 210, 209, 208); 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.

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

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

Ministry of Transportation
Ontario

GEOTMARK00170AA: Clark Boulevard Widening, Brampton, Ontario

RECORD OF BOREHOLE No 106

METRIC

GWP ______ LOCATION See Borehole Location Plan ORIGINATED BY AA

DIST ______ HWY ______ BOREHOLE TYPE Solid Stem Auger COMPILED BY WC

DATUM Geodetic DATE 6/22/2010 CHECKED BY ZO

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● 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
207.9 15.0	**CLAYEY SILT TO SILTY CLAY TILL** brown, hard										
207.4			15	SS	50 / 5 cm						22 23 32 23
15.5	End of Borehole. Water level @ 3.7 m (not stabilized)' upon completion. Piezometer installed to 15.2 m. Date / Measured Water Level July 23, 2010 / 9.4 m										

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

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

GEOTMARK00170AA: Clark Boulevard Widening, Brampton, Ontario

RECORD OF BOREHOLE No 107

METRIC

GWP ______ LOCATION See Borehole Location Plan ORIGINATED BY AA
DIST ______ HWY ______ BOREHOLE TYPE Solid Stem Auger COMPILED BY WC
DATUM Geodetic DATE 6/29/2010 CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
216.9	GROUND SURFACE					
0.0	0.1 m TOPSOIL **EMBANKMENT FILL:** Gravelly Sand brown, compact, moist	1	SS	13		
216.1 0.8	**EMBANKMENT FILL:** Clayey Silt tr. to some sand, tr. gravel brown, firm to stiff	2	SS	18	216	
		3	SS	9	215	
		4	SS	6	214	
		5	SS	4		
212.9 4.0	gravelly **SILTY SAND TO SAND** tr. clay, tr. to some gravel grey, v. dense, moist to wet	6	SS	89	213	24 53 (23)
		7	SS	50 / 15 cm	212	9 70 (21)
		8	SS	72	211	
210.3 6.6		9	SS	88		0 87 (13)
	End of Borehole. Water level @ 2.4 m (not stabilized)* upon completion. Borehole caved-in @ 4.8 m 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) ○ 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 $15 \oplus 5$ (20, 10) (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

GEOTMARK00170AA: Clark Boulevard Widening, Brampton, Ontario

RECORD OF BOREHOLE No 108

METRIC

GWP	LOCATION	See Borehole Location Plan	ORIGINATED BY AA
DIST HWY	BOREHOLE TYPE	Solid Stem Auger	COMPILED BY WC
DATUM Geodetic	DATE	8/17/2010	CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
216.1	GROUND SURFACE					216					
0.0	0.2 m TOPSOIL EMBANKMENT FILL: Sand, some silt brown, compact to v. dense, moist	1	SS	21							
215.4 0.7	SILTY SAND TO SANDY SILT TILL occ. cobbles and boulders brown, v. dense, wet	2	SS	50 / 13 cm		215					
		3	SS	53		214					
		4	SS	50 / 15 cm				20	38	35	7
		5	SS	50 / 10 cm		213					
		6	SS	50 / 13 cm	▼*	212		12	58	25	5
		7	SS	50 / 5 cm		211					
		8	SS	50 / 13 cm							
209.8	brown grey, shale fragments	9	SS	50 / 15 cm		210					
6.3	End of Borehole. Water level @ 4.0 m (not stabilized)* upon completion. Borehole caved-in @ 5.8 m.										

+ 3, × 3: Numbers refer to Sensitivity

(%) STRAIN AT FAILURE

GEOTMARK00170AA: Clark Boulevard Widening, Brampton, Ontario

RECORD OF BOREHOLE No 109

METRIC

GWP ______ LOCATION See Borehole Location Plan ORIGINATED BY AA

DIST ______ HWY ______ BOREHOLE TYPE Solid Stem Auger COMPILED BY WC

DATUM Geodetic DATE 6/29/2010 CHECKED BY ZO

ELEV DEPTH	SOIL PROFILE DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
216.7	GROUND SURFACE						
0.0	0.1 m **TOPSOIL** **EMBANKMENT FILL:** Clayey Silt tr. sand and gravel, tr. rootlets brown, firm to v. stiff	1	SS	5		216	
		2	SS	20			
		3	SS	17		215	
		4	SS	13		214	
213.7 / 3.0	**SILTY SAND TO SANDY SILT TILL** brown, dense to v. dense, wet; silty sand	5	SS	35	▼*		10 78 (12)
		6	SS	56		213	
		7	SS	64		212	6 37 47 10
		8	SS	50 / 13 cm		211	
210.5 / 6.2		9	SS	50 / 5 cm			
	End of Borehole. Water level @ 3.0 m (not stabilized)* upon completion. Borehole caved-in @ 2.7 m upon completion.						

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

(%) STRAIN AT FAILURE

GEOTMARK00170AA: Clark Boulevard Widening, Brampton, Ontario

RECORD OF BOREHOLE No 110

METRIC

GWP	LOCATION See Borehole Location Plan	ORIGINATED BY AA
DIST HWY	BOREHOLE TYPE Solid Stem Auger	COMPILED BY WC
DATUM Geodetic	DATE 6/23/2010	CHECKED BY ZO

SOIL PROFILE: ELEV DEPTH / DESCRIPTION / STRAT PLOT — SAMPLES: NUMBER / TYPE / "N" VALUES — GROUND WATER CONDITIONS — ELEVATION SCALE — DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● 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

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS
217.6	GROUND SURFACE				
0.0	0.2 m TOPSOIL				
	EMBANKMENT FILL: Clayey Silt tr. gravel, tr. to some sand brown, hard, moist	1	AS		
		2	SS	47	
		3	SS	39	
	silty sand	4	SS	50 / 13 cm	poss. cobbles or boulder
		5	SS	18	
	tr. organic	6	SS	22	
212.5 5.1	**CLAYEY SILT TO SILTY CLAY TILL** freq. cobbles and boulders grey, hard	7	SS	82	
		8	SS	50 / 5 cm	
		9	SS	50 / 13 cm	
		10	SS	50 / 5 cm	
206.9 10.7	**BEDROCK** highly weathered grey shale	11	SS	50 / 3 cm	
206.2 11.4		12	AS		

Elevation scale: 217, 216, 215, 214, 213, 212, 211, 210, 209, 208, 207

End of Borehole.
Water level @ 7.8 m (not stabilized)* upon completion.

Piezometer installed to 11.3 m.

Date / Measured Water Level
July 23, 2010 / 7.5 m

+³, ×³: Numbers refer to Sensitivity

20 15 ⊕ 5 10 (%) STRAIN AT FAILURE

GEOTMARK00170AA: Clark Boulevard Widening, Brampton, Ontario

RECORD OF BOREHOLE No 102A

METRIC

GWP	LOCATION See Borehole Location Plan		ORIGINATED BY AA
DIST	HWY	BOREHOLE TYPE Solid Stem Auger, NQ Coring	COMPILED BY WC
DATUM Geodetic		DATE 7/8/2010	CHECKED BY ZO

ELEV. DEPTH	SOIL PROFILE: DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
215.8 / 0.0	GROUND SURFACE					
	Straight Augering				215, 214, 213, 212	
211.2 / 4.6	**CLAYEY SILT TO SILTY CLAY TILL** occ. cobbles and boulders brown, hard	1	SS	50 / 15 cm	211	Auger grinding below 4.6 m
	cobbles and boulders	2	SS	50 / 15 cm	210	Spoon wet below 6.1 m
		3	SS	50 / 15 cm	209, 208	
		4	SS	50 / 15 cm	207, 206	
205.1 / 10.7	highly weathered	5	SS	50 / 15 cm	205	
	BEDROCK grey shale with highly weathered bedding partings	6	RC	T.C.R. = 7[illegible] % R.Q.D. = 2[illegible] %	204	
		7	RC	T.C.R. = 100 % R.Q.D. = 2[illegible] %	203	U.C.S. = 51.2 MPa
201.9 / 13.9	End of Borehole.				202	

Piezometer installed to 9.1 m.

Date / Measured Water Level
July 23, 2010 / 1.7 m

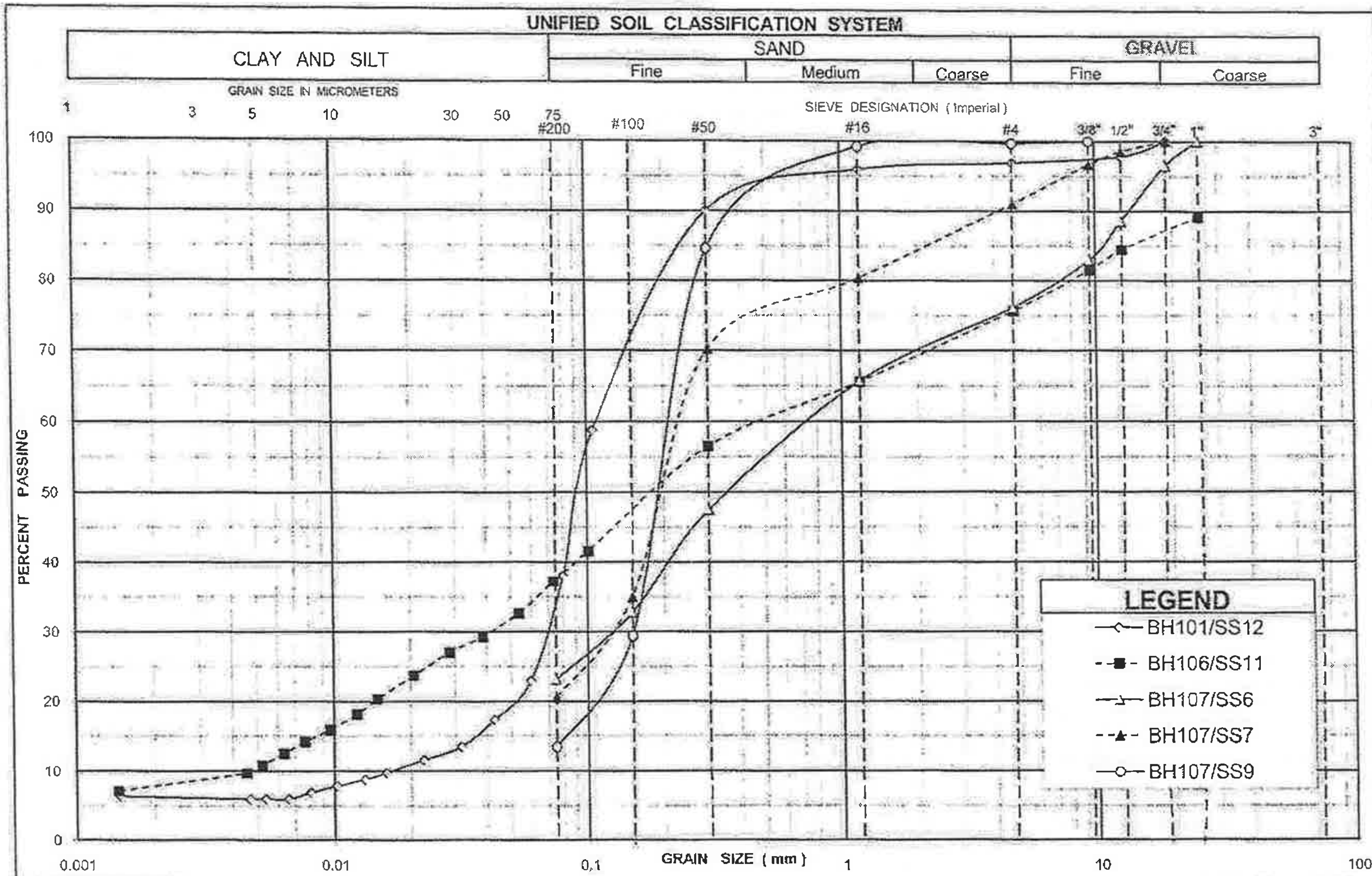
Column headings: STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT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

^{3}, $\times^3$: Numbers refer to Sensitivity

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

Appendix B

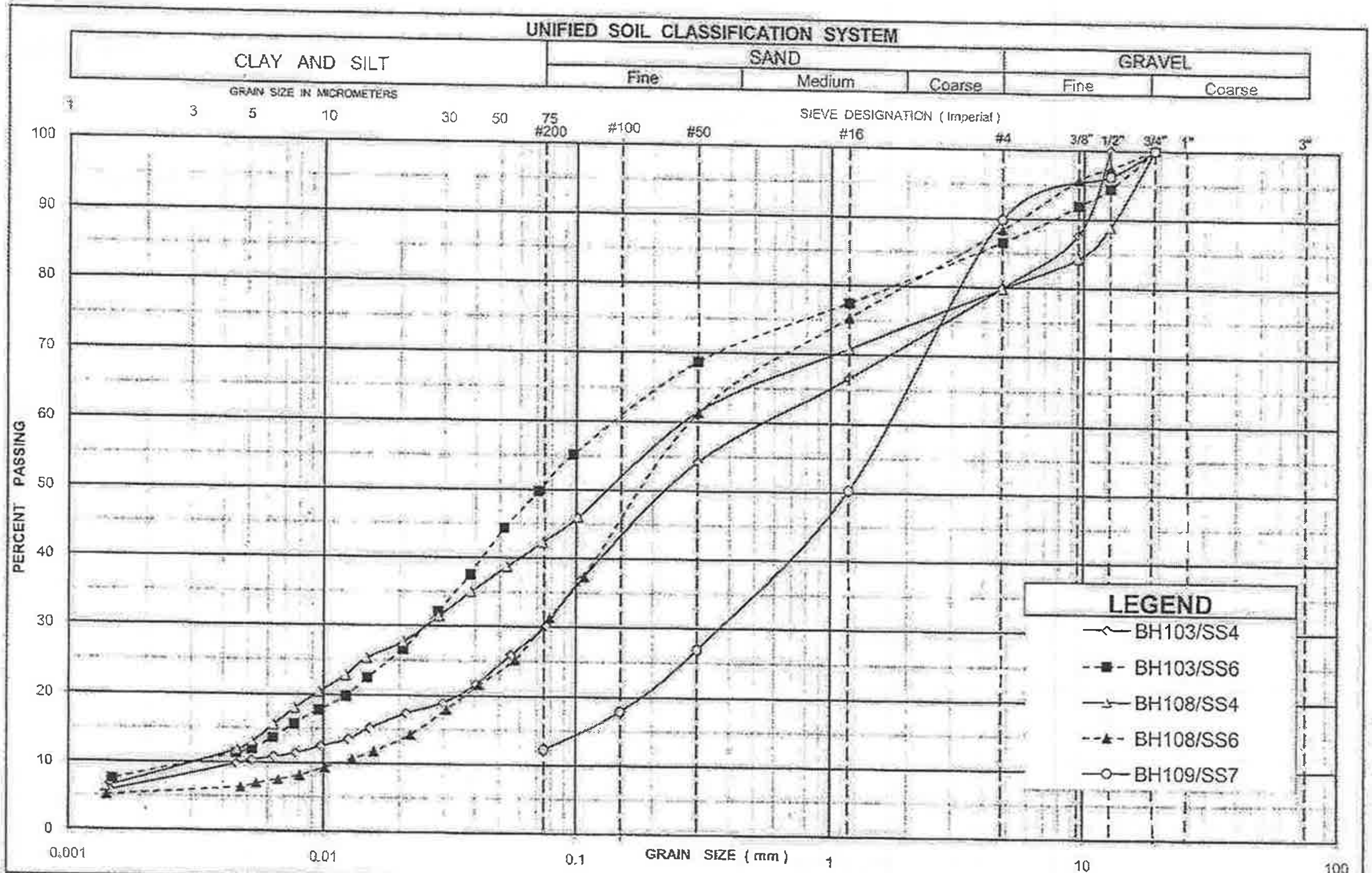
Laboratory Test Results



coffey geotechnics
SPECIALISTS MANAGING THE EARTH

GRAIN SIZE DISTRIBUTION
SILTY SAND TO SAND, trace gravel and clay

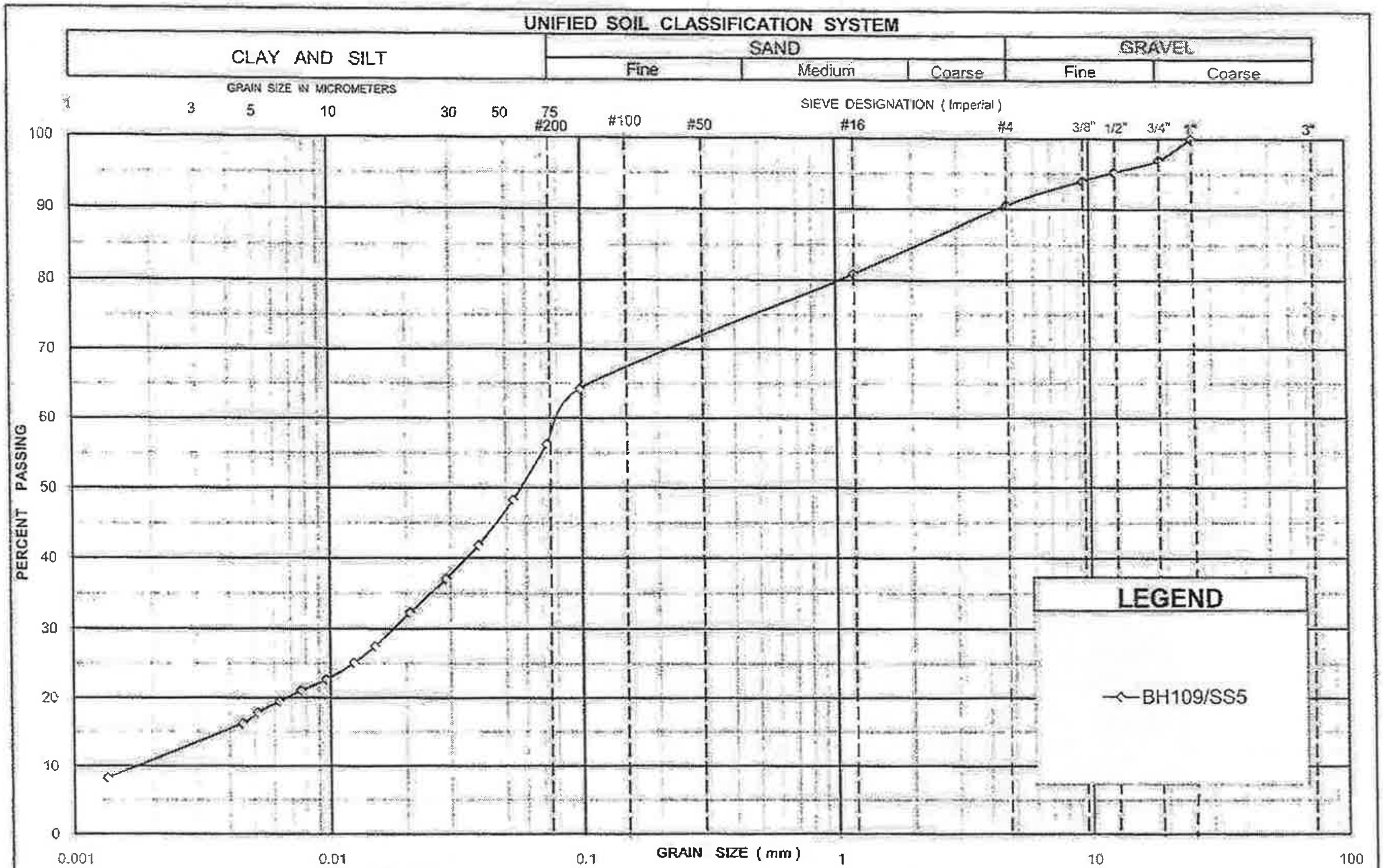
FIGURE NO.: B-1
PROJECT NO: GEOTMARK00170AA
DATE: JULY. 2010



coffey geotechnics
SPECIALISTS MANAGING THE EARTH

GRAIN SIZE DISTRIBUTION
SILTY SAND TO SANDY SILT TILL

FIGURE NO.:	B-2
PROJECT NO:	GEOTMARK00170AA
DATE:	JULY. 2010



coffey geotechnics
SPECIALISTS MANAGING THE EARTH

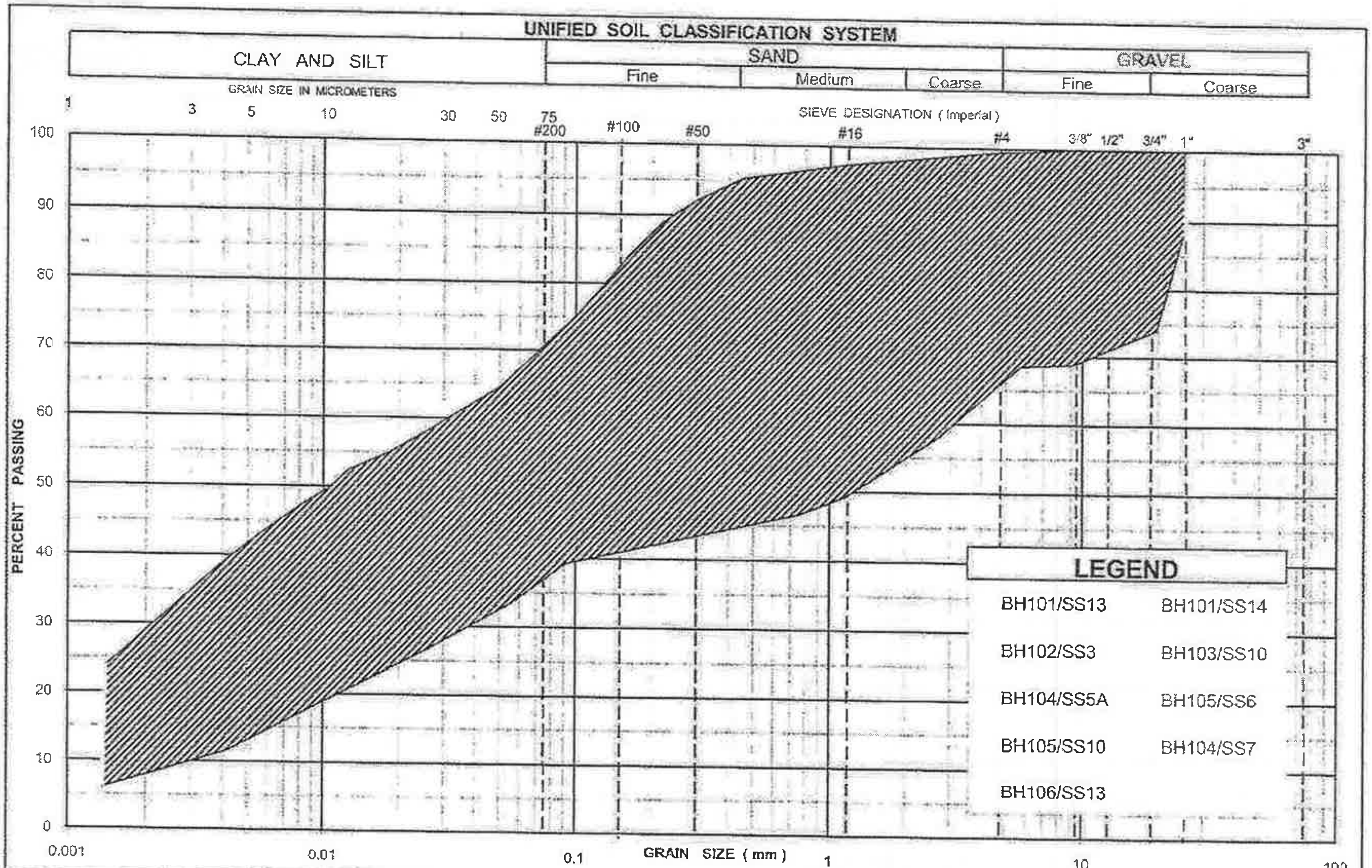
GRAIN SIZE DISTRIBUTION

Silty Sand layer within SILTY SAND TO SANDY SILT TILL

FIGURE NO.: B-3

PROJECT NO: GEOTMARK00170AA

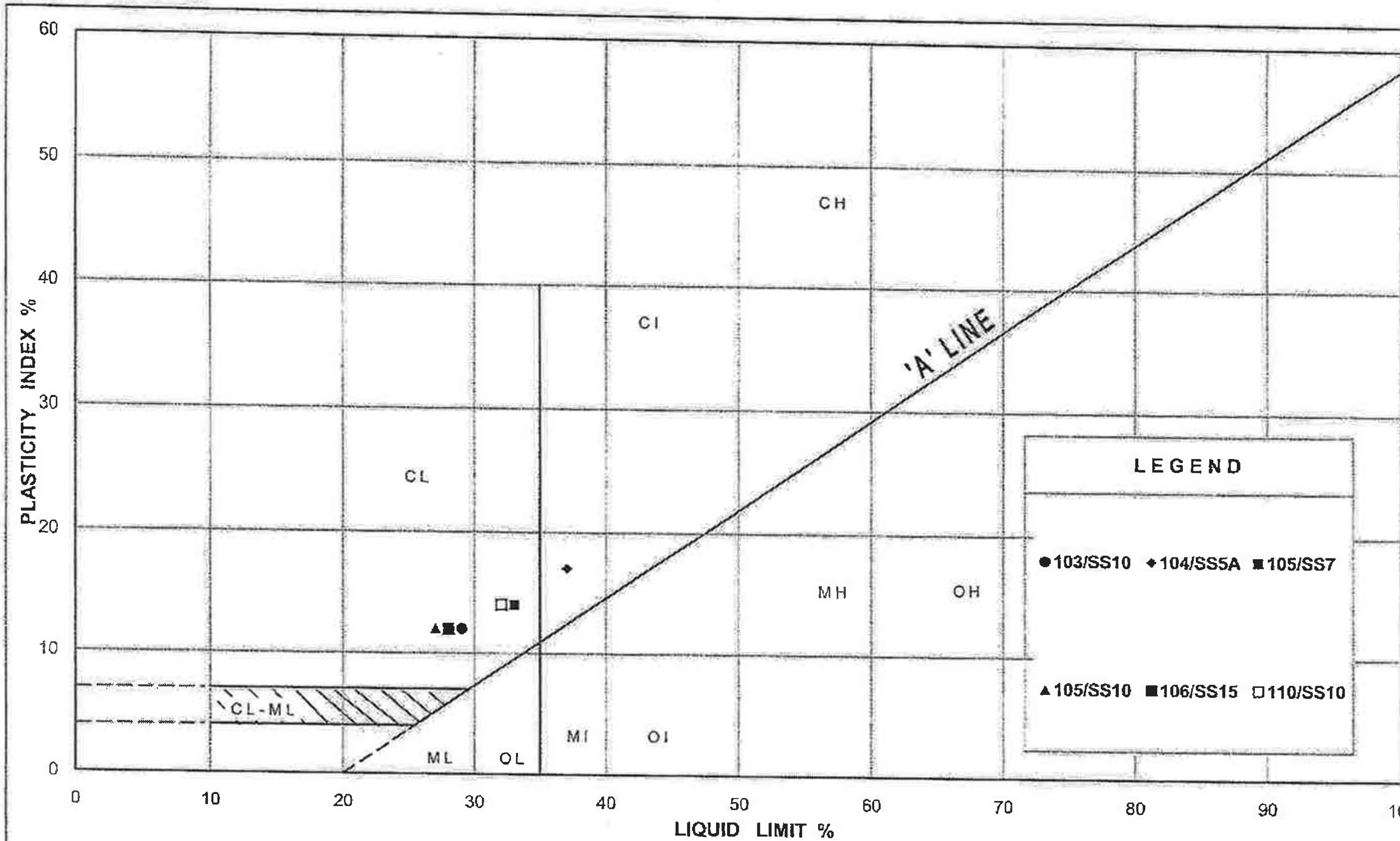
DATE: JULY, 2010



coffey geotechnics
SPECIALISTS MANAGING THE EARTH

GRAIN SIZE DISTRIBUTION
CLAYEY SILT TO SILTY CLAY TILL

FIGURE NO.:	B-4
PROJECT NO:	GEOTMARK00170AA
DATE:	JULY, 2010



coffey geotechnics SPECIALISTS MANAGING THE EARTH	PLASTICITY CHART **CLAYEY SILT TO SILTY CLAY TILL**	FIGURE No. B-5 REF. No. GEOTMARK00170AA DATE JULY, 2010

Appendix C

Site Photographs

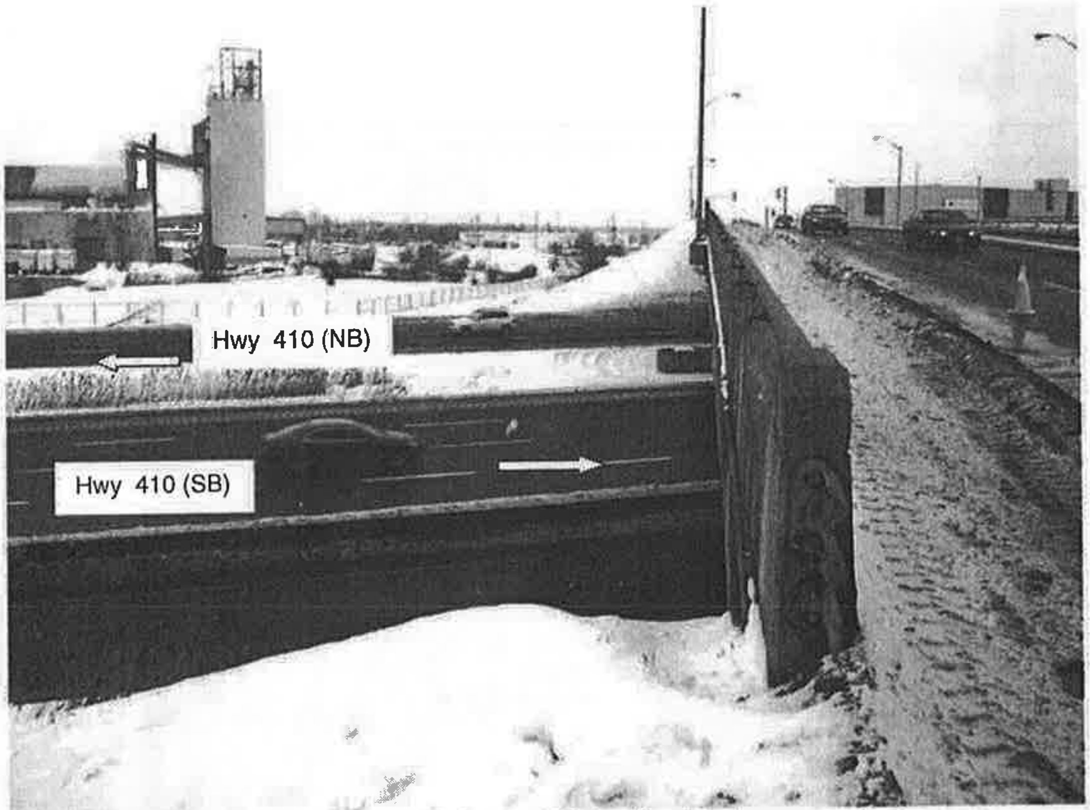
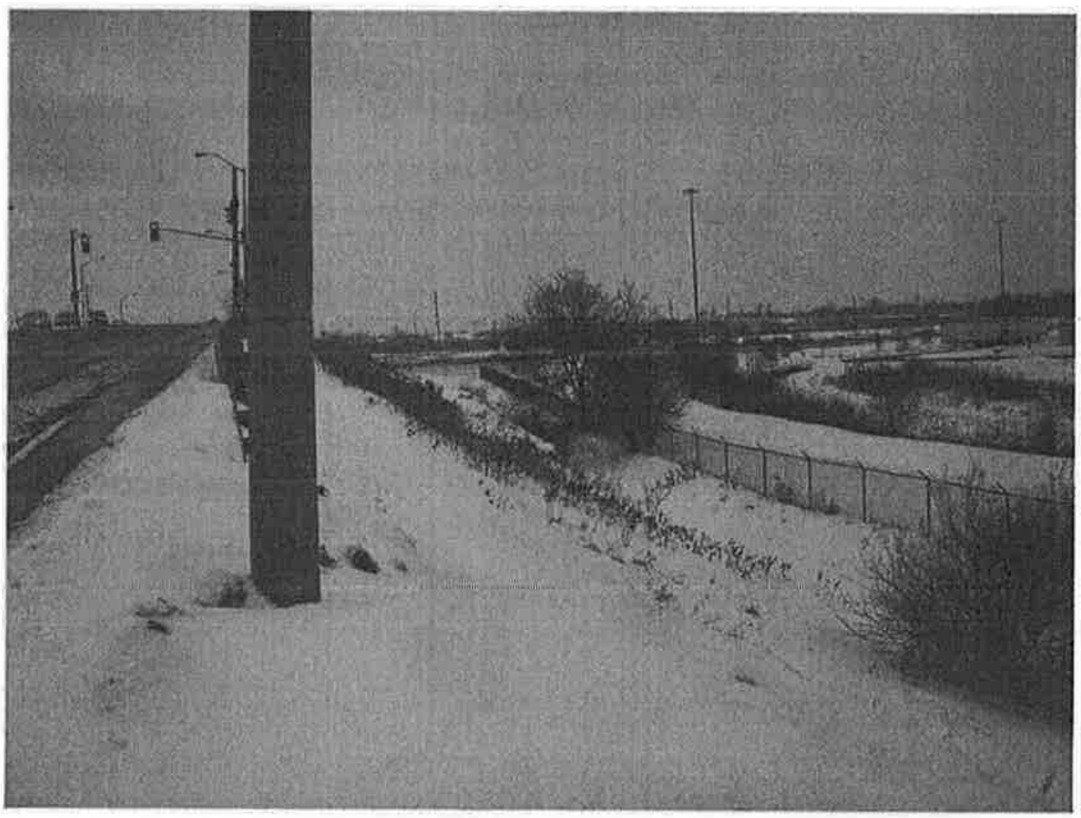


Figure C-1 – Clark Boulevard over Highway 410, looking East

Figure C-2 – Northwest slope of the approach embankment of Clark Boulevard

Figure C-3 – Clark Boulevard over Highway 410, looking West



Appendix D

Rock Core Photographs

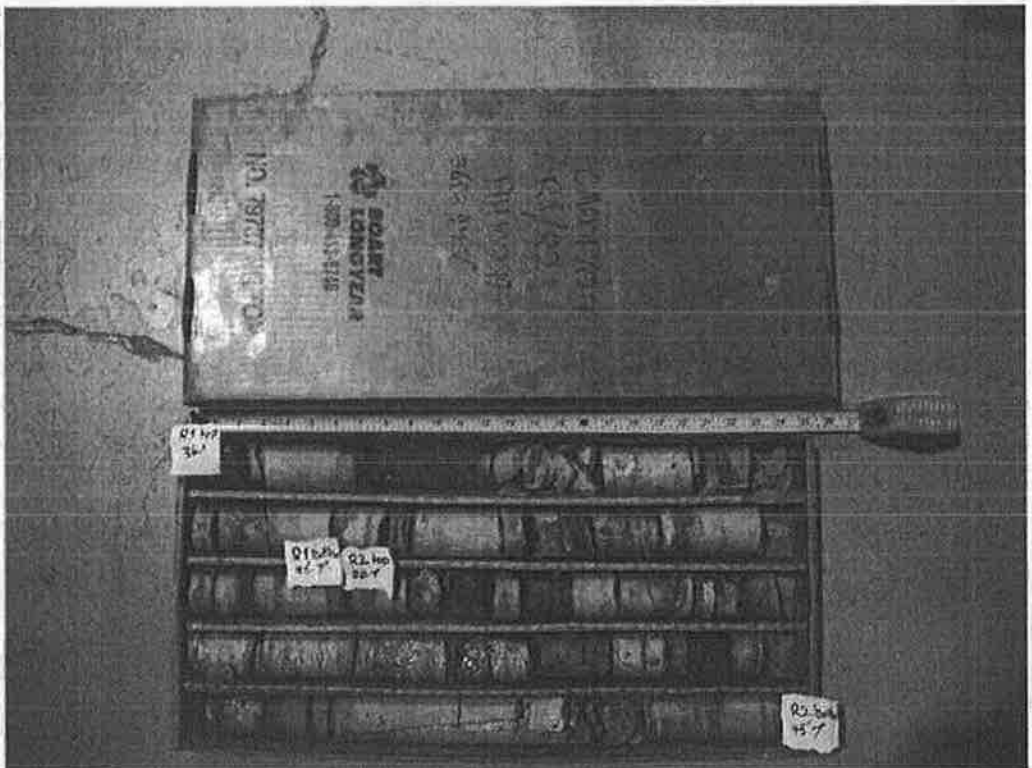
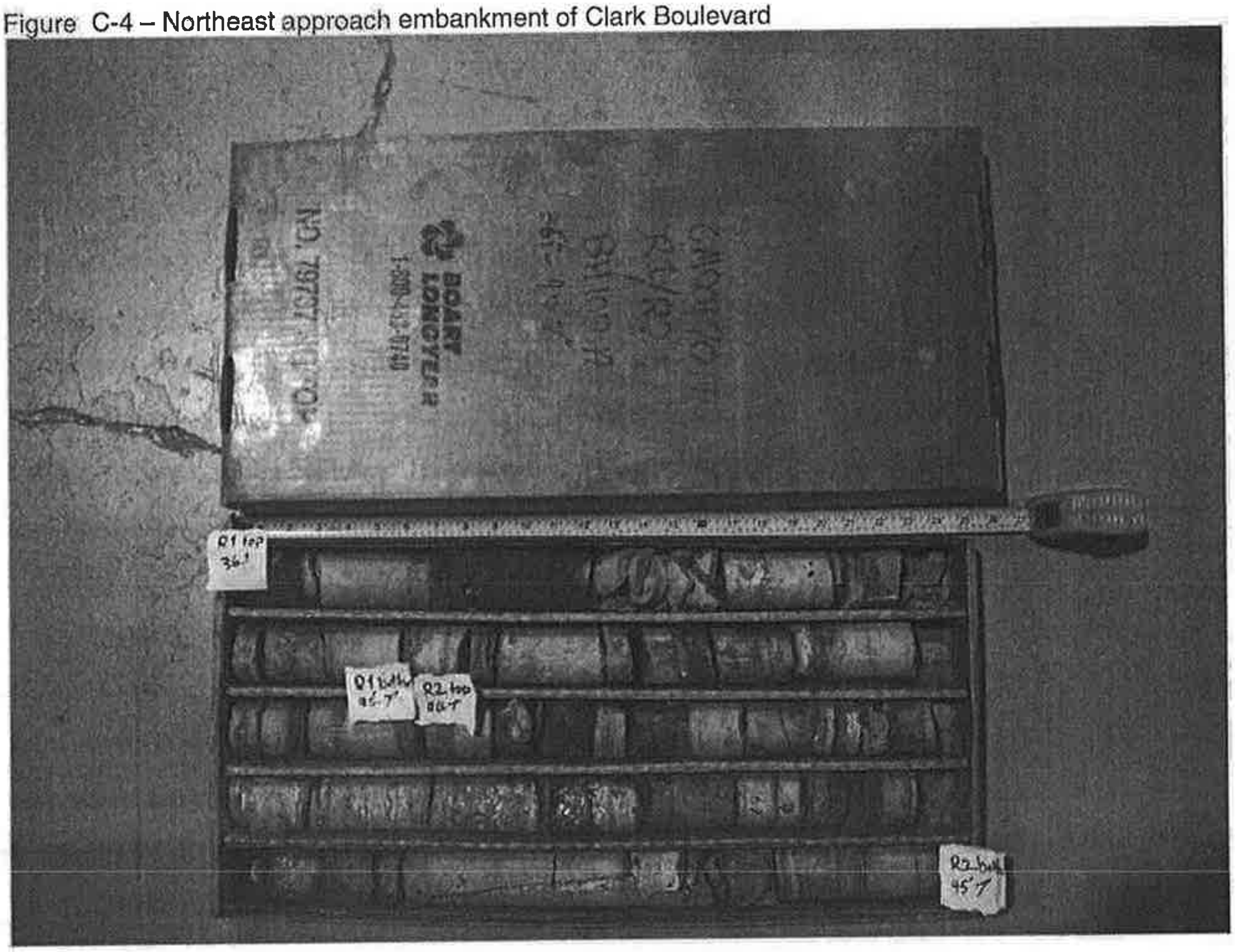


Figure D-1 – Rock Core from Borehole 102A

Figure D-2 – Rock Core from Borehole 105 (Box #1)

Figure C-4 – Northeast approach embankment of Clark Boulevard



Appendix E

Previous (1981-82) Investigation Report by MTO

Cont 83-39

Ministry of
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foundation investigation and design report

Cont. 83-39

ENGINEERING MATERIALS OFFICE
PAVEMENT & FOUNDATION DESIGN SECTION

WP 21-79-04 DIST 6

HWY 410 STR SITE 24-145- 471

Clark Boulevard Underpass

CONT 83-39

DISTRIBUTION

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R. D. Gunter
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R. Fitzbiggon (Cover only)
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DATE Jan. 8/82

FOUNDATION INVESTIGATION REPORT
For
Clark Boulevard Underpass
W. P. 21-79-04, Site 24-145-471
Highway 410, District 6, Toronto

INTRODUCTION

This report summarizes the factual information obtained from a foundation investigation program performed at the above-mentioned structural site and provides detailed recommendations pertaining to the structure foundations and related earthworks. The fieldwork was carried out between 81 12 07 and 81 12 08 and consisted of advancing three sampled boreholes, one accompanied by a dynamic cone penetration test, for depths ranging from 4.2 to 8.4 metres.

SITE DESCRIPTION AND GEOLOGY

The site is located at the existing intersection of Heart Lake Road and Clark Boulevard/Davidson Road some 650 metres south of Highway 7, in the City of Brampton, Municipality of Peel.

Land use in the area has recently changed from predominately farming to industrial subdivision development. Topography across the site is generally flat with ground surface sloping gently towards Lake Ontario.

The site is located in the physiographic region known as the "Peel Plain". The characteristic deposit, in the vicinity of the area under investigation, is composed of cohesive glacial till, whose thickness varies from 3 to 15 metres. The overburden is underlain by shale bedrock. This physiographic region is well drained by Credit, Oakville and Etobicoke Creeks, which have cut deep valleys into the overburden. There is, therefore, no large undrained depression, swamp or bog areas, although in many of the interstream areas drainage is still imperfect.

The shale bedrock is of the Meaford-Dundas formation, Ordovician Period.

SUBSURFACE CONDITIONS

Although variable in composition, generally competent subsurface conditions were encountered across the site. Surficial deposits varied from a sand and gravel at the west abutment location to a cohesive glacial till deposit at the pier and east abutment locations.

The boundaries between the various soil types, insitu and laboratory test results, as well as stabilized ground water levels, are shown on the attached Record of Borehole Sheets. The locations and elevations of the borings, along with a profile showing an estimated stratigraphical section based on borehole data, are shown on Drawing No. 217904-A.

The various soil types encountered are briefly described in the following paragraphs.

Sand and Gravel

Underlying the west abutment location and explored to a maximum depth of 4.2 metres is a surficial deposit of sand and gravel with traces of silt and clay. Typical grain size distribution curves for this granular deposit are shown on Figure 1. Based on augering operations, cobble and boulder sized fragments are well dispersed throughout this deposit and may account for refusal to augering at a depth of 4.2 metres.

Interpretation of standard penetration test 'N' values generally in excess of 100 blows per foot, suggests a denseness ranging from compact to very dense, but predominately very dense throughout.

Silty Clay, Sand and Gravel (Glacial Till)

Immediately underlying the pier and east abutment locations and explored for depths ranging from 6.7 to 8.4 metres is a cohesive till deposit consisting of silty clay with varying amounts of sand and gravel. Gradation of this till deposit became coarser (increased sand and gravel contents) at the east abutment location compared to the pier location, as shown by the two distinct sets of grain size distribution curves on Figure 2. Cobbles and boulders were also encountered towards the base of this deposit.

In addition to gradation, the plasticity of the fill decreased with increasing sand content. Results of Atterberg limit and water content testing are plotted on the Plasticity Chart, Figure 3. These results indicate the cohesive matrix of the till deposit to range from an inorganic silty clay of moderate plasticity (CL-CI) to slight plasticity (CL-ML).

Based on interpretation of 'N' values and augering operations, the consistency of this over-consolidated till deposit is assessed as being very stiff to hard, but generally hard below elevation 213.

Groundwater Conditions

Overnight stabilized water level readings taken in open boreholes were found to correspond to elevations of 215, 214, and 209.5. The higher water levels were recorded in boreholes close to existing roadway ditches and probably reflect a higher localized water table. However, due to the impermeable nature of the till at the pier location, the lowest water level may not reflect a true stabilized condition. Based on results of previous work carried out in the area, it is felt that groundwater will approximate elevation 213.5, with normal seasonal fluctuations occuring depending on the time of year.

DISCUSSION AND RECOMMENDATIONS

In order to carry Clark Boulevard over the proposed Highway 410, a two span 95 x 20 metre underpass structure is contemplated. Design requirements will necessitate realignment of Clark Boulevard to the south, shifting of Heart Lake Road at the crossing to the west, and construction of approach fill to a maximum height in the order of 9 metres.

In consideration of the variable but competent subsoil conditions across the site, recommendations pertaining to the foundations of the new structure and related earthworks are summarized as follows.

Foundations for perched abutments can be founded on spread footings located on a well compacted Granular 'A' core within the approaches as per current M. T. C. Standards. All surficial organic and/or softened material within the planned limits of the approaches must be subexcavated prior to placement of the wall-compacted granular core. For spread footings founded on a Granular 'A' core and constructed to current M. T. C. standards, an allowable capacity at the S. L. S. Type II of 280 kPa and a factored capacity at the U. L. S. of 750 kPa may be used for design purposes.

Pier elements can be founded on spread footings located at or below elevation 213.5 for an allowable capacity at the S. L. S. Type II of 400 kPa and a factored capacity of the U. L. S. of 950 kPa.

Earth pressures against the abutment wall should be computed as per Subsection 6.6.1.2.2 of the O. H. B. D. C. Manual.

Resistance to sliding of the abutment footings can be calculated assuming a coefficient of friction of 0.6 between the underside of the concrete footing and the Granular 'A' core.

The underside of all footing elements should be provided with a minimum 1.3 metres of earth cover for frost protection purposes.

No major dewatering difficulties are anticipated for pier footing excavations in consideration of the relatively low permeability of the glacial till deposit. Localized seepage into excavations can be controlled by perimeter ditches and pumping from corner sumps.

No stability problems are anticipated for permanent embankment slopes constructed to a 2:1 geometry.

MISCELLANEOUS

The fieldwork for this investigation was carried out under the supervision of Mr. H. Sturm, Engineer-in-Training, utilizing equipment owned and operated by Atcost Soil Investigation, Toronto. This report was written by Mr. T. J.Kazmierowski, Foundations Engineer and reviewed by Mr. M. Devata, Senior Foundations Engineer.

REGISTERED PROFESSIONAL ENGINEER
T. J. KAZMIEROWSKI
PROVINCE OF ONTARIO

T. J. Kazmierowski, P. Eng.,
Foundations Engineer

M. Devata, P. Eng.,
Senior Foundations Engineer

APPENDIX

Ministry of Transportation and Communications
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RECORD OF BOREHOLE No 1 — METRIC

W P 21-79-04 LOCATION Co-ords. N 4 839 963.0; E 285 971.2 ORIGINATED BY H. S.
DIST 6 HWY 410 BOREHOLE TYPE Hollow Stem Auger & Cone Test COMPILED BY H. S.
DATUM Geodetic DATE 81 12 07 CHECKED BY

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	PLASTIC LIMIT W_p NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
215.8	Ground Surface										
0.0	Brown Sand and Gravel Trace of silt and clay Cobbles and boulders throughout Compact to Very Dense					▽	215				
			1	SS	28						
			2	SS	75/2 cm		214				47 39 11 3
								100/15 cm			
			3	SS	93		213				66 21 8 5
			4	SS	100/10 cm						
							212				
211.6			5	SS	100/6 cm						52 41 5 2
4.2	Refusal to Augering End of Borehole		6	SS	100/2 cm						

$+^3, \times^5$: Numbers refer to Sensitivity

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

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications Ontario

RECORD OF BOREHOLE No 2

METRIC

W P 21-76-04 LOCATION Coords. N 4 839 975.0; E 286 022.0 ORIGINATED BY H. S.

DIST 6 HWY 410 BOREHOLE TYPE Solid Stem Augers COMPILED BY H. S.

DATUM Geodetic DATE 81 12 07 CHECKED BY [illegible]

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	SAMPLES – TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
215.0 / 0.0	Ground Surface						
	(Glacial Till)	1	SS	20		214	
	Silty clay of low plasticity	2	SS	36			10 10 (80)
	Brown / Grey					213	
	Trace of gravel and sand	3	SS	134			
	Very stiff	4	SS	48		212	13 10 48 29
	to						
	hard	5	SS	41		211	
		6	SS	66		210	
	gravel and cobbles				▽		
						209	
		7	SS	171/18 cm			
208.3	cobbles & boulders	8	SS	125/15 cm			
6.7	Refusal to augering End of Borehole						

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH: o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE

PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L — WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ

$+^3, \times^5$: Numbers refer to Sensitivity

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

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications Ontario

RECORD OF BOREHOLE No 3

METRIC

W P 21-79-04 LOCATION Co-ords N 4 839 986.0; E 286 056.5 ORIGINATED BY H. S.

DIST 6 HWY 410 BOREHOLE TYPE Solid Stem Augers COMPILED BY H. S.

DATUM Geodetic DATE 81 12 07 - 08 CHECKED BY [signature]

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%) Wp W WL	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
214.5	Ground Surface							
0.0	(Glacial Till)				▽	214		
	Silty clay of slight plasticity	1	SS	39				9 43 39 9
	and					213		
	Sand	2	SS	50			o H	18 35 39 8
		3	SS	149		212		
	Brown / Gray	4	SS	153		211	o ⊢	
	- Varying amounts of gravel	5	SS	111		210		
	HARD	6	SS	180			o ⊢	22 36 35 7
						209		
	Cobbles and Boulders	7	SS	95		208		
		8	SS	100/13 cm		207		
206.1		9	SS	100/2 cm				
8.4	Refusal to augering End of Borehole							

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH: o UNCONFINED + FIELD VANE • QUICK TRIAXIAL × LAB VANE

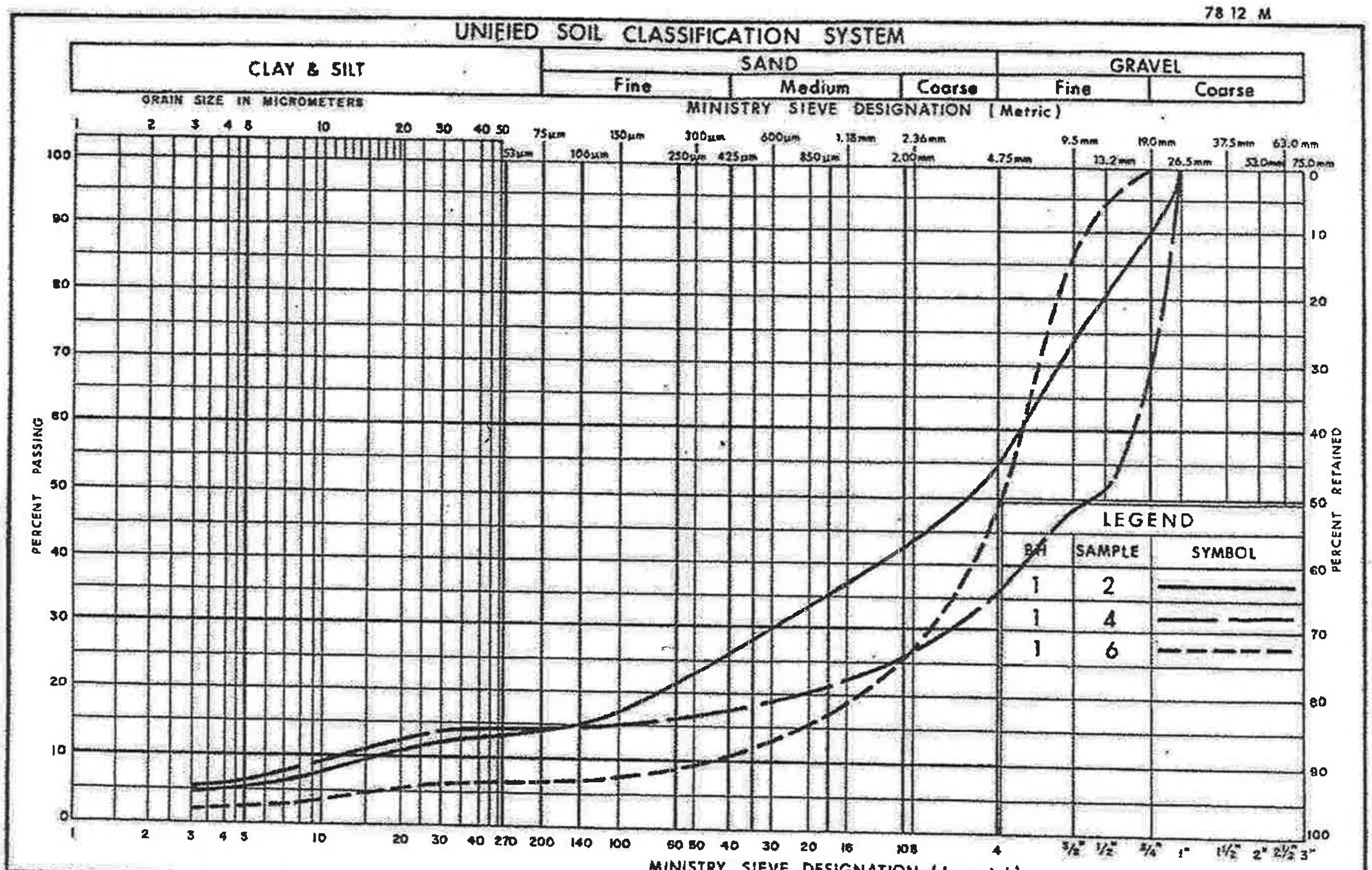
PLASTIC LIMIT Wp, NATURAL MOISTURE CONTENT W, LIQUID LIMIT WL

UNIT WEIGHT γ

$+^3, \times^5$: Numbers refer to Sensitivity

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

OFFICE REPORT ON SOIL EXPLORATION



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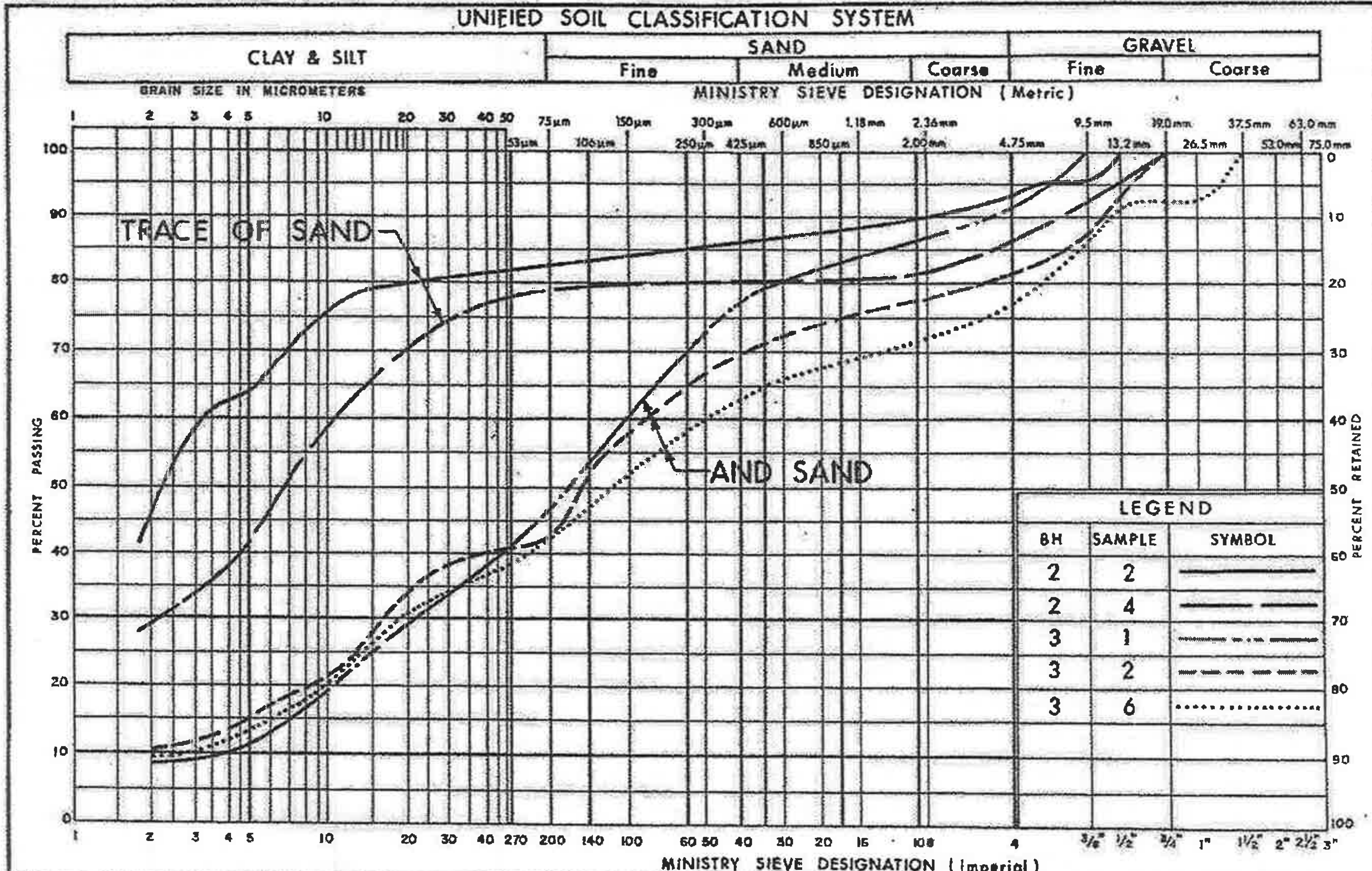
GRAIN SIZE DISTRIBUTION

SAND & GRAVEL, TRACE OF SILT & CLAY

FIG No 1

WP 21-79-04

78 12 M



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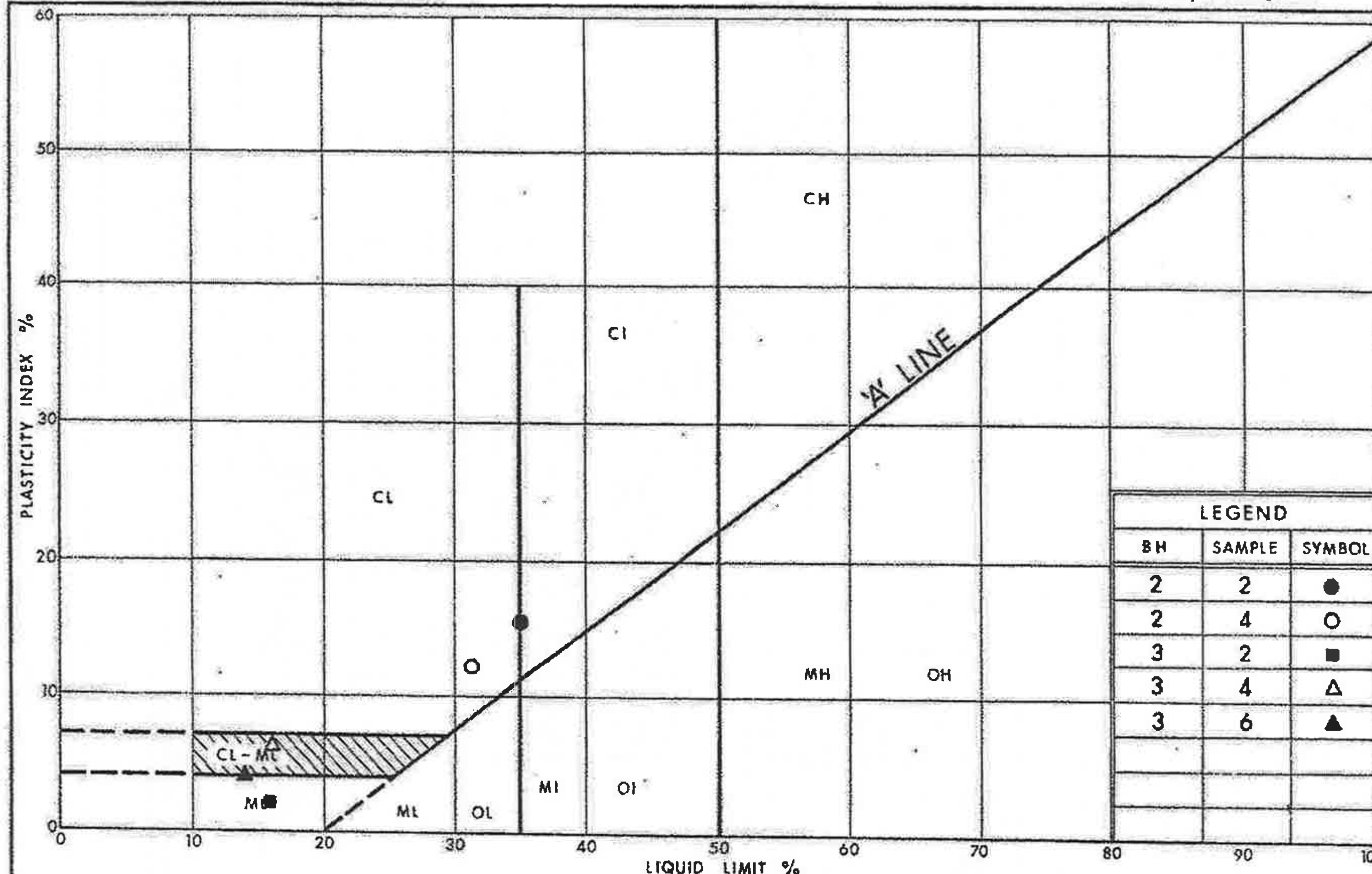
GRAIN SIZE DISTRIBUTION

SILTY CLAY (Glacial Till)

FIG No 2

W P 21-79-04

Oct 75, FF-S-21



LEGEND		
BH	SAMPLE	SYMBOL
2	2	●
2	4	○
3	2	■
3	4	△
3	6	▲

Ontario
Ministry of Transportation and Communications

PLASTICITY CHART
SILTY CLAY Matrix (Glacial Till)

FIG No 3
W P 21-79-04

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS $\bar{N}$.

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

<u>CONSISTENCY:</u> 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*

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

N (BLOWS/0.3 m)	0 - 5	5 - 10	10 - 30	30 - 50	>50
	VERY LOOSE	*LOOSE*	*COMPACT*	*DENSE*	*VERY DENSE*

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

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

<u>MODIFIED RECOVERY:</u> SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (R Q D), FOR MODIFIED RECOVERY, IS:

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

<u>JOINTING AND BEDDING:</u>

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

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	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
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

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

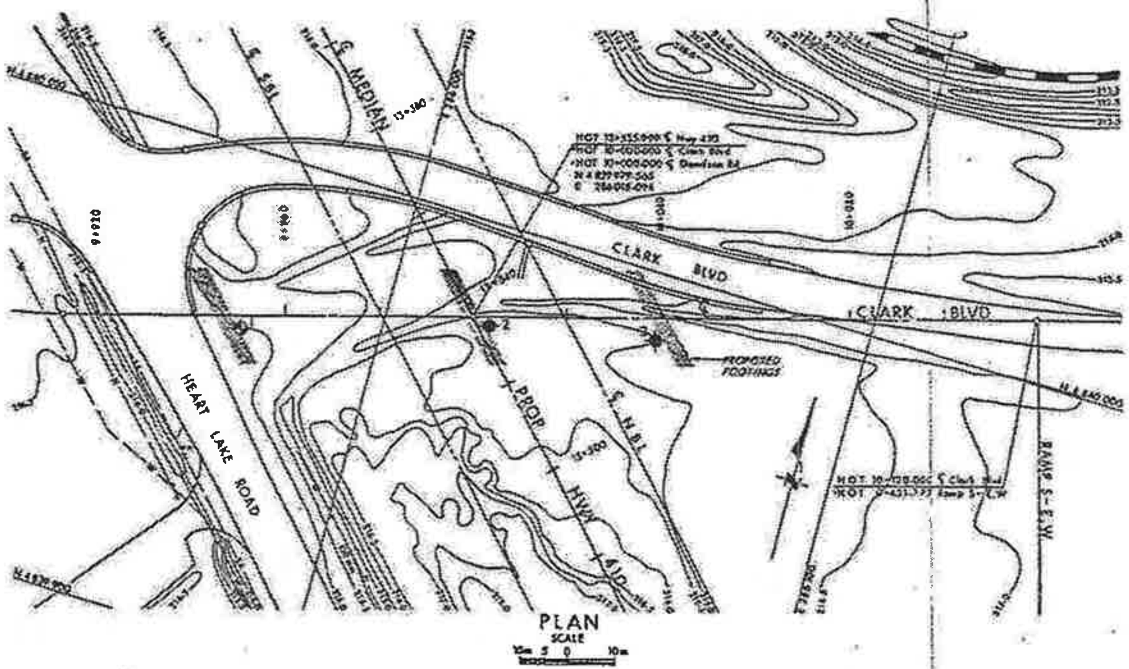
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES + METRES.

CONT No
WP No 21-79-04

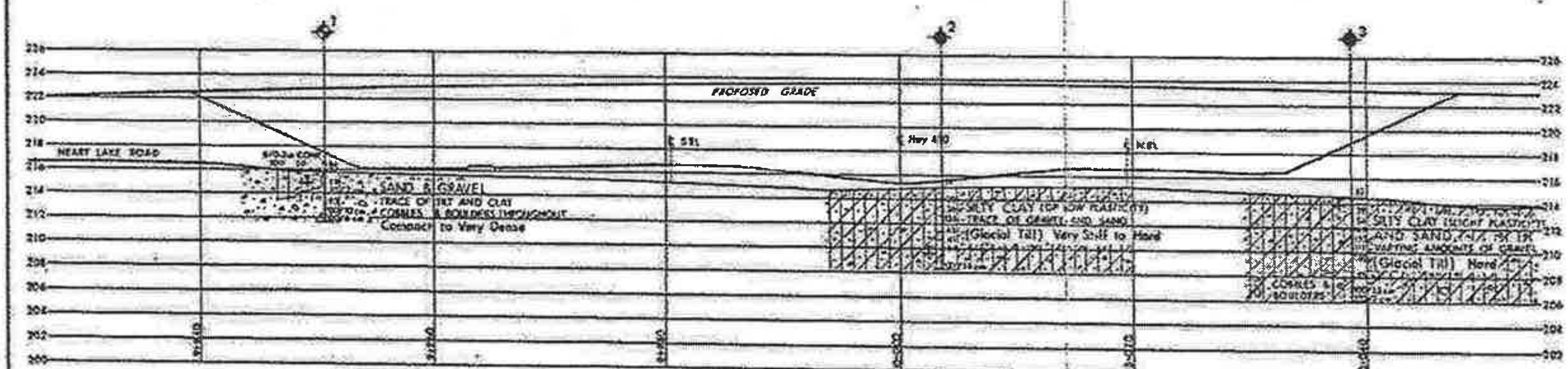
CLARK BLVD

SHEET

BORE HOLE LOCATIONS & SOIL STRATA



PLAN
SCALE



℄ PROFILE CLARK BLVD
SCALE

NOTE:
Contour lines do not accurately represent existing ground surface due to recent grading operations.

KEY PLAN
REG MUN OF PEEL
CITY OF BRAMPTON
SITE

LEGEND

- Bore Hole
- Dynamic Cone Penetration Test (Cone)
- Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- WL at time of investigation 1981 12

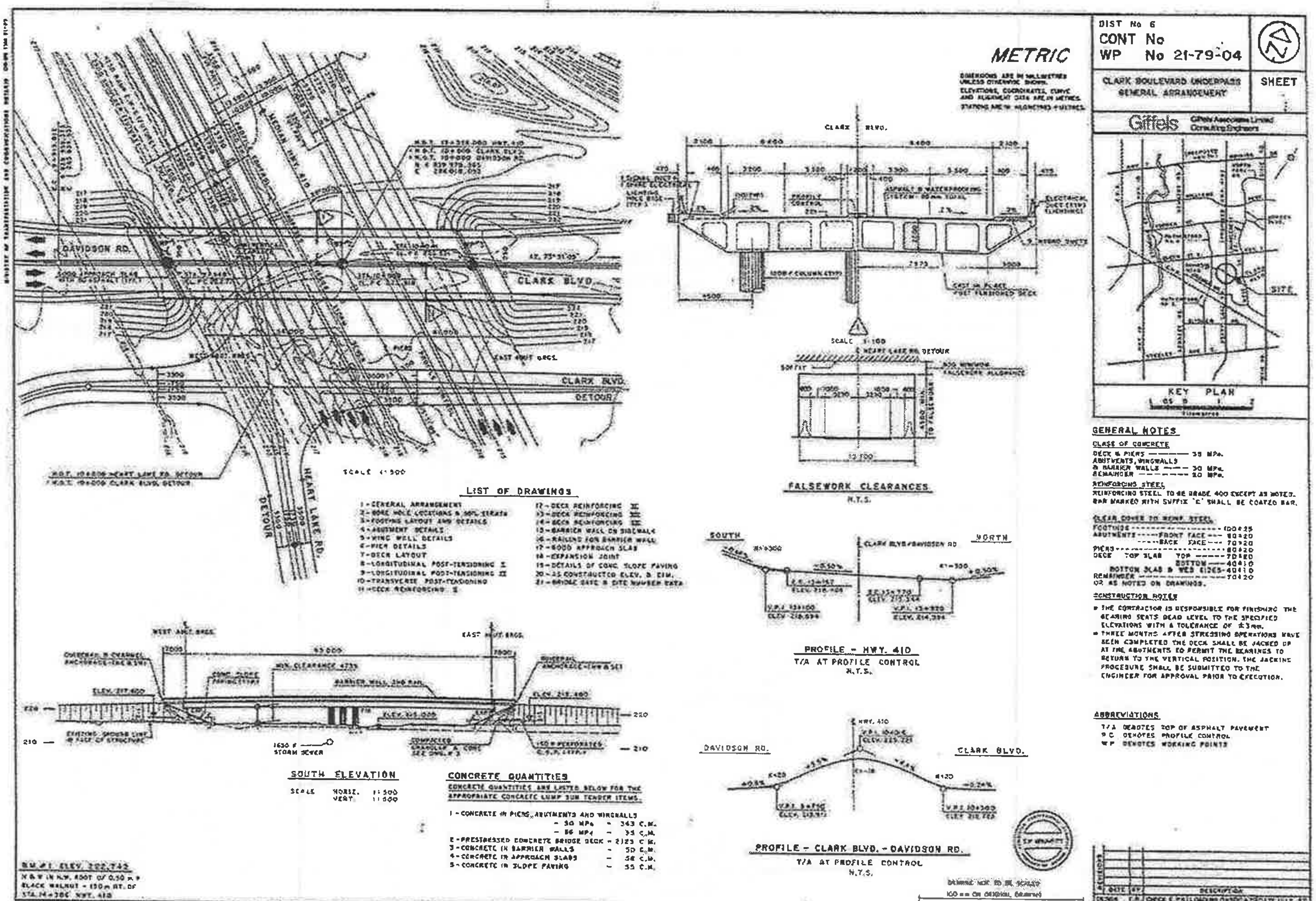
No	ELEVATION	CO-ORDINATES NORTH	EAST
1	215.8	4 839 963.0	285 971.2
2	215.0	4 839 979.0	286 022.0
3	215.5	4 839 986.0	286 056.3

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

REVISIONS
DATE | BY | DESCRIPTION

Geocres No 30M12-146

Hwy No Prop 410


DIST No 6
CONT No
WP No 21-79-04
SHEET
CLARK BOULEVARD UNDERPASS
GENERAL ARRANGEMENT
Giffels
METRIC
DAVIDSON RD.
CLARK BLVD.
CLARK BLVD. DETOUR
HEART LAKE RD.
DETOUR
SCALE 1:500
KEY PLAN
SITE
SCALE 1:100
FALSEWORK CLEARANCES
N.T.S.
SOUTH
NORTH
PROFILE - HWY. 410
T/A AT PROFILE CONTROL
N.T.S.
DAVIDSON RD.
CLARK BLVD.
PROFILE - CLARK BLVD. - DAVIDSON RD.
T/A AT PROFILE CONTROL
N.T.S.
WEST ABUT. BRGS.
EAST ABUT. BRGS.
63 000
SOUTH ELEVATION
SCALE HORIZ. 1:500
VERT. 1:500
1650 ø STORM SEWER
LIST OF DRAWINGS
1 - GENERAL ARRANGEMENT
2 - BORE HOLE LOCATIONS & SOIL STRATA
3 - FOOTING LAYOUT AND DETAILS
4 - ABUTMENT DETAILS
5 - WING WELL DETAILS
6 - PIER DETAILS
7 - DECK LAYOUT
8 - LONGITUDINAL POST-TENSIONING I
9 - LONGITUDINAL POST-TENSIONING II
10 - TRANSVERSE POST-TENSIONING
11 - DECK REINFORCING I
12 - DECK REINFORCING II
13 - DECK REINFORCING III
14 - DECK REINFORCING IV
15 - BARRIER WALL ON SIDEWALK
16 - RAILING FOR BARRIER WALL
17 - GOOD APPROACH SLAB
18 - EXPANSION JOINT
19 - DETAILS OF CONC. SLOPE PAVING
20 - AS CONSTRUCTED ELEV. & DIM.
21 - BRIDGE SITE & SITE NUMBER DATA
CONCRETE QUANTITIES
CONCRETE QUANTITIES ARE LISTED BELOW FOR THE APPROPRIATE CONCRETE LUMP SUM TENDER ITEMS.
1 - CONCRETE IN PIERS, ABUTMENTS AND WINGWALLS
- 30 MPa - 343 C.M.
- 35 MPa - 35 C.M.
2 - PRESTRESSED CONCRETE BRIDGE DECK - 2123 C.M.
3 - CONCRETE IN BARRIER WALLS - 50 C.M.
4 - CONCRETE IN APPROACH SLABS - 58 C.M.
5 - CONCRETE IN SLOPE PAVING - 55 C.M.
GENERAL NOTES
CLASS OF CONCRETE
DECK & PIERS ———— 35 MPa.
ABUTMENTS, WINGWALLS & BARRIER WALLS ——— 30 MPa.
REMAINDER ———— 20 MPa.
REINFORCING STEEL
REINFORCING STEEL TO BE GRADE 400 EXCEPT AS NOTED. BAR MARKED WITH SUFFIX 'C' SHALL BE COATED BAR.
CLEAR COVER TO REINF. STEEL
FOOTINGS ———— 100±25
ABUTMENTS ——— FRONT FACE ——— 80±20
——— BACK FACE ——— 70±20
PIERS ———— 80±20
DECK TOP SLAB TOP ———— 70±20
BOTTOM ——— 40±10
BOTTOM SLAB & WEB SIDES - 40±10
REMAINDER ———— 70±20
OR AS NOTED ON DRAWINGS.
CONSTRUCTION NOTES
• THE CONTRACTOR IS RESPONSIBLE FOR FINISHING THE BEARING SEATS DEAD LEVEL TO THE SPECIFIED ELEVATIONS WITH A TOLERANCE OF ±3mm.
• THREE MONTHS AFTER STRESSING OPERATIONS HAVE BEEN COMPLETED THE DECK SHALL BE JACKED UP AT THE ABUTMENTS TO PERMIT THE BEARINGS TO RETURN TO THE VERTICAL POSITION. THE JACKING PROCEDURE SHALL BE SUBMITTED TO THE ENGINEER FOR APPROVAL PRIOR TO EXECUTION.
ABBREVIATIONS
T/A DENOTES TOP OF ASPHALT PAVEMENT
P.C. DENOTES PROFILE CONTROL
W.P. DENOTES WORKING POINTS
REVISIONS
DESCRIPTION

CONT No	
WP No 21-79-04	
CLARK BOULEVARD UNDERPASS FOOTING LAYOUT AND DETAILS	SHEET

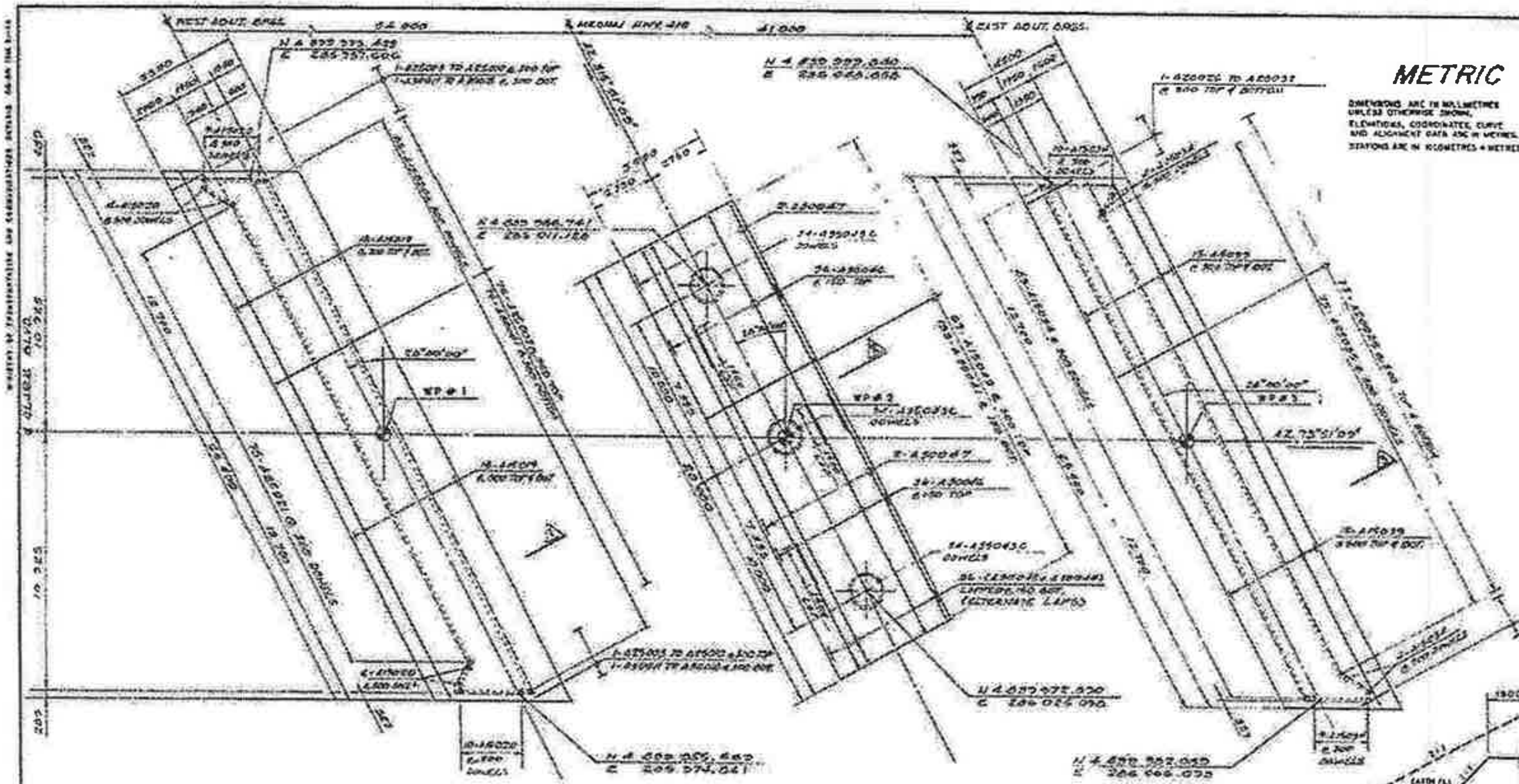
Giffels — Giffels Associates Limited, Consulting Engineers

METRIC

DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE SHOWN. ELEVATIONS, COORDINATES, CURVE AND ALIGNMENT DATA ARE IN METRES. STATIONS ARE IN KILOMETRES + METRES.

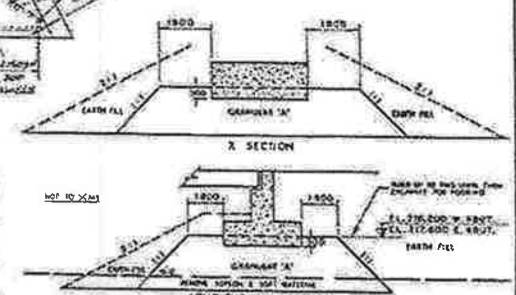
NOTES:

- FOR COLUMN LAYOUT SEE DWG No 6
- COMPACTED GRANULAR 'A' CORE
  1- REMOVE TOPSOIL & / OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' & EARTH FILL.
  2- PLACE GRANULAR 'A' & EARTH FILL TO ELEVATION SHOWN, COMPACTED ACCORDING TO CURRENT M.T.C. STANDARDS.
  3- EXCAVATE COMPACTED GRANULAR 'A' & EARTH FILL FOR FOOTING.



FOUNDATION LAYOUT
SCALE 1:100

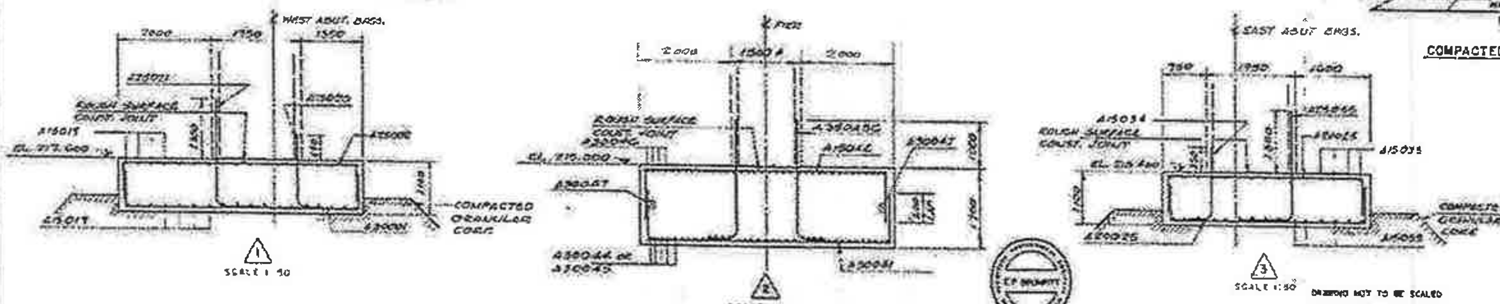
WORKING POINT DATA			
W P	STA. @ ℄ CLARK BLVD.	PROVINCIAL COORDINATES	
		NORTH	EAST
1	9+948	4 839 964.547	285 986.223
2	10+000	4 839 979.563	286 018.093
3	10+041	4 839 990.488	286 037.476



X SECTION

LONGITUDINAL SECTION

COMPACTED GRANULAR 'A' CORE FOR ABUTMENTS



DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

Appendix F

Explanation of Terms Used in the 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 $\bar{N}$.

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
$\sigma_1, \sigma_2, \sigma_3$	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'	$-^{\circ}$	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	$-^{\circ}$	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

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

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

FOUNDATION DESIGN REPORT PROPOSED WIDENING OF CLARK BOULEVARD UNDERPASS OVER HIGHWAY 410 BRAMPTON, REGION OF PEEL

AECOM

GEOTMARK00170AA
September 22, 2011

Coffey Geotechnics Inc.
351 Steelcase Rd. W., Unit 10 Markham Ontario L3R 4H9 Canada

CONTENTS

Foundation Design Report
Proposed Widening of Clark Boulevard Underpass over Highway 410,
Brampton, Region of Peel

5 DISCUSSION AND RECOMMENDATIONS

The existing Highway 410 underpass structure at Clark Boulevard is to be widened. The existing structure is a two-span bridge with a total length of about 109 m. It is proposed to widen both the west and east abutments and also the central pier, on both the north and south sides. A widening of approximately 3.6 m is anticipated to the north and 3.6 m to the south. According to the GEOCRES published by the MTO, the existing bridge is supported by spread footings both at the abutments and the pier location.

The subsurface conditions were explored at ten (10) locations by means of eleven (11) boreholes (see Table 3.1 in Section 3 of the foundation investigation section of this report) during this investigation. An earlier investigation carried out in 1981-82 by MTO for the existing bridge consisted of three boreholes. The original grade (o.g.) has an elevation of 215 ± m. In general, the native soils at the project site consist of a 1.5 m to 4.5 m thick upper granular layer at four of the borehole locations. Underlying these granular soils and/or the original ground surface throughout the site is a major glacial till deposit, which is in turn underlain by shale bedrock. The surface of the shale bedrock was recorded at three deep boreholes at El. 206.9 to 205.1 m.

The groundwater level at the time of investigation was observed to be between El. 214.6 – 208.2 m, based on the measurements of water levels taken within the piezometers installed. Based on these data, however, it is our opinion that at the time of our investigation, the groundwater table across the site was generally between El. 214.5 and 213.5 m. It should be noted that the groundwater levels are subject to seasonal fluctuations and in response to major weather events.

5.1 Foundations

We understand that the existing approximately 109 m long bridge is supported on spread footings based on the drawing available at the MTO (see Appendix E). Details of the proposed construction method for the proposed widening (i.e. lane closure or staged construction) and the construction sequence will be developed during the detailed design phase of the project.

Based on the available subsurface information, we have considered a number of foundation options varying from normal spread footings to deep foundations which include drilled caissons, driven piles and micropiles, as discussed below.

5.1.1 Abutments

We understand that the existing abutments are supported on spread footings with compacted Granular 'A' pad over the glacial till deposit. Based on an old General Arrangement (GA) drawing (given in Appendix E and which is also available in MTO GEOCRES information system, along with several other drawings), the foundation levels (i.e. bottom of footing) are at about El. 216.5 m (west abutment), and El. 217.3 m (east abutment). These aspects will however need to be verified for both the abutments and the pier footings.

The use of same foundation type is normally recommended for most bridge structure widenings. In this case, this is possible but for the abutments, since compacted Granular 'A' pads were used for the existing foundations, considerable shoring effort may be required to retain the existing embankment and the existing foundations.

The use of augered and cast-in-place concrete foundations (drilled caissons) can also be considered as a feasible foundation option for this project.

The use of driven piles can also be considered. However, driving of piles into the hard clayey silt to silty clay till and the very dense silty sand to sandy silt till would likely present difficulties and considerable vibrations. Also, cobbles and boulders may pose some risks on possible shallow refusal of the driven piles on boulders. For these reasons, as discussed later, the use of driven piles is not recommended.

Micropiles could also be considered for this project due to possible space and/or overhead restrictions, immediately adjacent to or beneath the existing bridge.

When designing foundations it must be remembered that total settlements experienced by the new foundations will translate into differential settlements between the existing bridge structure and the widened section.

The advantages and disadvantages of various foundation types at the support locations are summarized in Appendix G. The following paragraphs present a further discussion on these options.

5.1.1.1 Spread Footing Foundations on Compacted Granular Fill

The abutments can be supported on spread footing foundations founded on compacted Granular 'A' pads over sufficiently competent natural soils (i.e. upper granular deposits and primarily glacial tills). The engineered fill would consist of Granular 'A' type material, compacted in thin layers to at least 100% of the material's Standard Proctor Maximum Dry Density (SPMDD). The thickness of the Granular 'A' pad supporting the spread footing foundations should be at least 1.5 m. Prior to the placement of the engineered fill, the upper variable, weak and otherwise unsuitable zones of the existing subgrade will need to be stripped to the surface of the competent stratum. The suggested highest subgrade elevations at the borehole locations are given in Table 5.1.1.1.1.

Table 5.1.1.1.1 Spread Footing Foundations on Compacted Granular Fill for Abutments

Location	Borehole	Existing Ground Elevation* (m)	Recommended Stripping (base of granular pad) Depth (m)	Recommended Stripping (base of granular pad) Elevation (m)	Soil Type
West Abutment	101	222.1	7.0	215.1	Gravelly Sand
	102 & 102A	215.8	1.3	214.5	Clayey Silt to Silty Clay Till
	1	215.8	0.8	215.0	Sand and Gravel
East Abutment	105	216.5	3.8	212.7	Clayey Silt to Silty Clay Till

Location	Borehole	Existing Ground Elevation* (m)	Recommended Stripping (base of granular pad) Depth (m)	Recommended Stripping (base of granular pad) Elevation (m)	Soil Type
East Abutment	106	222.9	8.4	214.5	Silty Sand
	3	214.5	0.3	214.2	Clayey Silt to Silty Clay Till

* at the time of investigation

After stripping, the exposed subgrade must be inspected and approved by the Geotechnical Engineer. The approved subgrade may then need to be proof-rolled, depending on the Site and subgrade conditions, as directed by the Geotechnical Engineer.

The construction of the Granular 'A' pad and of the earth fill should meet the minimum requirements as per MTO standard, as shown in Appendix I. The Granular 'A' pad supporting the spread footing foundations should be at least 1.5 m thick.

From the available information, it appears that the existing footings were designed for a bearing resistance at S.L.S. of 280 kPa and a factored vertical resistance at U.L.S. of 750 kPa.

For footings satisfying these requirements, similar resistances can be used for the widening, namely a factored vertical bearing resistance at U.L.S. equal to 750 kPa and a bearing resistance at S.L.S. of 300 kPa (for 25 mm settlement). This should, however, be reviewed before the embankment and foundation details are finalized. The unfactored horizontal resistance against sliding between concrete and properly compacted Granular 'A' fill can be calculated using an angle of friction of 35 degrees.

For frost protection, the footing should have a permanent earth cover of at least 1.2 m.

As was mentioned before, this approach will necessitate careful construction techniques, including extensive shoring, as will be discussed in Section 5.4 of this report.

5.1.1.2 Drilled Caisson Foundations

The use of augered and cast-in-place concrete foundations (drilled caissons) can be considered as a feasible foundation option for the abutments. From the reliability viewpoint, drilled caisson foundation is a favourable option for this project, provided that sufficient space is available to install the caissons immediately adjacent to the existing bridge.

Caissons extended at least 2.0 m into the sufficiently hard or very dense till (N>50 blows/0.3 m) can be designed for a factored vertical geotechnical resistance of 2500 kPa at U.L.S. and a value equal to 1600 kPa can be used for S.L.S. These resistance values can be increased with further penetrations (socketing) into the competent till overburden. Intermediate values are given in the table presented (Table 5.1.1.2.1). This latter approach is however not recommended as further penetration into the overburden increases risks due to groundwater, as will be further elaborated in this report. If higher resistance values must be used then consideration can be given to extending the caisson units into the underlying bedrock. For caissons extended to the surface of the relatively sound bedrock, the following resistance values can be utilized:

- Factored Geotechnical Resistance at U.L.S. = 3000 kPa;
- S.L.S. need not be considered.

These values assume an at least 0.6 m into the bedrock or more, until sufficiently competent bedrock is reached. They can be increased further with further socket depths but this is not likely to be necessary for this particular project.

The following table summarizes the anticipated caisson bottom elevations at the borehole locations.

Table 5.1.1.2.1 – Caisson Foundations for Abutment Widening

Location	Borehole	Recommended Bottom of Caisson Elevation (m)	Recommended S.L.S. (kPa)	Recommended Factored U.L.S. (kPa)	Subgrade Material
West Abutment North Side	101	209.5 208.5 206.0*	1600 1800 Need not be considered	2500 2700 3000	Clayey Silt to Silty Clay Till Relatively Sound Shale Bedrock
West Abutment South Side	1	**			
	102 & 102A	210.0 209.0 204.4	1600 1800 Need not be considered	2500 2700 3000	Clayey Silt to Silty Clay Till Relatively Sound Shale Bedrock
East Abutment North Side	105	209.2 208.0 205.0	1600 1800 Need not be considered	2500 2700 3000	Clayey Silt to Silty Clay Till Relatively Sound Shale Bedrock
	3	210.0 209.0 ***	1600 1800	2500 2700	Clayey Silt to Silty Clay Till Relatively Sound Shale Bedrock
East Abutment South Side	3	210.0 209.0 ***	1600 1800	2500 2700	Clayey Silt to Silty Clay Till Relatively Sound Shale Bedrock

Location	Borehole	Recommended Bottom of Caisson Elevation (m)	Recommended S.L.S. (kPa)	Recommended Factored U.L.S. (kPa)	Subgrade Material
East Abutment South Side	106	209.5 208.5 ***	1600 1800	2500 2700	Clayey Silt to Silty Clay Till Relatively Sound Shale Bedrock

* Estimated Bedrock was not reached
**Not suitable to El. 211.6 m where the borehole was terminated.
***Bedrock was not reached.

These design values are applicable to commonly used caisson sizes in Ontario (i.e. between 0.76 m and 1.8 m diameter) provided the minimum caisson length is 6.0 m below the bottom of the pile cap. However, the use of relatively smaller caisson sizes (i.e. between 0.76 m and 1.35 m diameter) would be preferable as these are relatively easier and more efficient to install, especially in confined areas (e.g. the present case). 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 S.L.S. 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 S.L.S. Similarly, if a 1.2 m diameter caisson is used, then the caisson resistance at S.L.S. would be $(1.2/2)^2$x3.1416 x1600 kN/m^2 =1810 kN/caisson.

As was mentioned before, these resistance values assume a minimum of 2.0 m socket into the competent till or 0.6 m into the sufficiently sound shale bedrock, depending on the design value used. Proper penetration into the competent overburden or sound shale bedrock 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. As well, if caissons are to be constructed, it would be prudent to put down additional (deeper) boreholes, extending into the bedrock, to minimize surprises during the construction.

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

During the installation of the caissons, dewatering may be required due to the presence of non-cohesive soil types below the groundwater table. As well, problems may arise due to the possible presence of cobbles, boulders and shale fragments in the till or hard (limestone) layers in the bedrock. Some dewatering is expected to be necessary to intercept and remove surface water and to pump out any perched water. Temporary steel casing (liner) will be required during the construction of the caisson holes to prevent caving. In particular, Boreholes 1 and 101 show the presence of coarse granular soils (i.e. gravelly sand and sand and gravel) below water table. Installing the caisson holes through these coarse granular soil layers may require additional precautions to prevent the caving of the hole, such as extending the casing simultaneously as the hole is augered (or if necessary ahead of the auger) and use of bentonite slurry, etc. The casing/liner 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 to prevent the deterioration of the base of the caisson. As well, if cohesionless till or interbedded sand/silt layers are encountered, further dewatering and/or extending the caisson further deeper (if necessary to bedrock) to prevent pouring the concrete on unstable base. 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 possible presence of cobbles, boulders and shale fragments in the glacial till deposit. An NSSP should be issued to alert the Contractor of cobbles, boulders, shale fragments in the overburden, as well as possible dewatering requirements.

The tremie concrete method can be used, if desired or required, to reduce the degree of dewatering during the installation of the caisson foundations. Based on the borehole data, however, the use of the tremie concereting method is unlikely to be necessary.

The anticipated caisson elevations at the borehole locations, as given in the table above, can be used with interpolation in between and beyond the borehole locations. Actual caisson depths in the field would be decided during their installation, ensuring sufficient socket into the bearing stratum (i.e. till or bedrock). As was mentioned before, additional boreholes will reduce uncertainties in this regard.

5.1.1.3 Driven Pile Foundations

Driven pile foundations, including steel H-piles, are not recommended due to the prevailing soil conditions (i.e. hard and very dense glacial till with frequent cobbles and boulders), as well as the vibrations induced by pile driving adjacent to the existing structure. Due to the variable consistency/relative density of the overburden, the piles may have unpredictable lengths, as well as not reaching sufficient depths (i.e. piles having not enough length). This may be somewhat alleviated by pre-augering, but it is not desirable situation. As well, very heavy driving can be anticipated. The vibrations induced by the driving of piles may damage the existing foundations or may induce settlement or instability of the existing foundations. It is our opinion therefore that the use of driven piles including steel H-piles, is not a good choice from reliability point of view.

5.1.1.4 Micropile Foundations

An alternative which may be considered is the use of micropiles to support the abutments. Under normal circumstances, micropiles are less cost effective than caissons or spread footings, but in this case they may be an attractive solution if overhead restriction and/or insufficient working space present problems for the construction of the caissons and due to shoring requirements for spread footings.

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. Due to the small pile diameter (typically 160 mm to 260 mm), the end-bearing contribution in micropiles is generally neglected. The grout/ground bond strength achieved is influenced primarily by the surrounding soil or rock and grouting method used, i.e., pressure grouting or gravity feed. The role of the drilling method is also influential, although less well quantified.

The axial resistance of micropile for this project would depend on the diameter, penetration length and the type of reinforcement. The lateral resistances would also depend on the diameter, as well as, to a lesser extent, on the socket length.

The use of micropiles is generally less economical than spread footing foundations or caissons due to the required numbers of micropiles to achieve similar geotechnical resistance to conventional foundations. However, it is advantageous if low overhead and/or site access conditions occur and/or interference of new foundation support with the existing foundations is a concern. As was mentioned before, geotechnical resistances will also depend on such factors as diameter, method of installation, socket lengths, etc. Typically, the geotechnical resistance is calculated by multiplying the circumferential area (i.e. circumference x length) by bond strength.

Axial resistances of up to about 900 kN/micropile are available (at U.L.S. and S.L.S. will typically not govern). In this present case, up to a similar resistance would be available depending on the diameter and penetration. For high resistances, sufficient penetration into the sound bedrock would be required. The lateral resistances would also depend on the diameter, as well as, to a lesser extent, on the socket length into the bedrock.

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

In summary, with the prevailing conditions, the use of caissons or spread footings matching the existing foundation design appear to present more attractive options, considering reliability and cost.

Depending on the type of foundation and shoring used, monitoring (including instrumentation) during construction will likely be necessary.

5.1.2 Central Pier

It is our understanding that the existing central pier located at the centre median of Highway 410 is supported by spread footings founded on glacial till. Based on an old GA drawing (see Appendix E), the founding level of the central pier footing is at El. 213.3 m, but should be verified. As mentioned before, the use of same foundation type is generally recommended for most bridge structure widening for better performance of the structure.

Boreholes 103 and 104, drilled within the median of Highway 410, indicated that fill materials, such as sand and clayey silt, are present from the ground surface to 1.0 m – 1.3 m depth (El. 214.9 – 213.9 m). This fill material (possible backfill from original construction) is underlain by very dense/hard glacial till. The boreholes were terminated within the glacial till deposit due to practical refusal (N>100 blows/0.3 m), at El. 205.8 and 206.6 m, respectively. In Borehole 103, the glacial till was found to consist of an essentially cohesionless material (i.e. silty sand to sandy silt till) with N-values in excess of 100 blows/0.3 m (i.e. very dense) to El. 210.2 m, while below this elevation in Borehole 103 and in Borehole 104, the material was found to be a primarily cohesive till (i.e. clayey silt to silty clay) with N-values ranging from 32 to generally in excess of 100 blows/0.3 m. Borehole 2, put down at the o.g. level by MTO in 1981, contacted a clayey till deposit of low plasticity, which is very stiff near the ground surface (N=20 blows/0.3 m) becoming hard (N=36 to in excess of 100 blows/0.3 m at about El. 213.5 m or at 1.5 m below the o.g. level.

Spread footings can be considered as the preferred option for the central pier as it is the same foundation type as the existing foundation.

The use of augered and cast-in-place concrete foundations (drilled caissons) can also be considered as a feasible foundation option for this project, provided that the existing pier foundation and the superstructure will not interfere with the installations of the caissons. This may be an attractive option, especially if caissons are to be used for abutment support.

The use of driven piles is not recommended, as the driving of the piles into hard clayey silt till or very dense silty sand to sandy silt till would be very difficult. Also, cobbles and boulders may pose some risks on possible shallow refusal of the driven piles on boulders. In addition, the vibrations induced from the driving of piles may also cause settlement or damage to the existing foundations as well as having insufficient pile lengths.

Micropiles could also be considered for this project if the construction of normal spread footings and caissons pose a problem due to overhead height restriction beneath the bridge and/or due to space restrictions.

When designing foundations it must be remembered that total settlements experienced by the new foundations will translate into differential settlements between the existing bridge structure and the widened section.

The advantages and disadvantages of various foundation types at the support locations are summarized in Appendix G. The following paragraphs present a further discussion on these options.

5.1.2.1 Spread Footing Foundations

The use of spread footing foundations for the support of the central pier is one of the recommended options. The recommended geotechnical resistances and highest founding elevations are summarized in the following table:

Table 5.1.2.1.1 Spread Footing Foundations for Central Pier

Location	Borehole	Existing Ground Elevation* (m)	Recommended Highest Founding Elevation (m)	Recommended S.L.S (kPa)	Recommende dU.LS. (kPa)	Soil Type
Central Pier	103	215.2	213.8	400	950	Silty Sand to Sandy Silt Till
	104	215.9	214.4	400	950	Clayey Silt to Silty Clay Till
	2	215.0	213.6	400	950	Clayey Silt to Silty Clay Till

* at the time of investigation

As was mentioned before, however, the presently available data indicate that the elevation at the existing foundation (underside of footing) is at El. 213.3 m and therefore we recommend that the foundations for the widening be placed at the same or similar elevation (i.e. El. 213.3 m ± 0.1 m).

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

The recommended S.L.S. value (i.e. 400 kPa) is based on a settlement value of not exceeding 25 mm (provided that the subgrade is not unduly disturbed during the construction). As was mentioned before, however, this settlement will lead to differential settlement of the same magnitude between the new and the existing foundations. This should be taken into consideration in the design and construction of the widenings.

All footing excavations and bearing surfaces must be inspected, evaluated and approved by a Geotechnical Engineer who is familiar with the findings of this investigation. Allowance should be made to place a 120 mm thick concrete mud mat (i.e. skim coat) in the footing excavations as soon as possible (not more than three hours) after excavation. The footing excavation should be inspected and approved by the Geotechnical Engineer prior to pouring the concrete mud mat.

For frost protection, the footing should have a permanent earth cover of at least 1.2 m.

5.1.2.2 Drilled Caisson Foundations

The use of augered and cast-in-place concrete foundations (drilled caissons) can be considered as a feasible option for the pier foundations, especially if caissons are to be used to support the abutments. From the reliability viewpoint, drilled caisson foundation is a favourable option for this project, provided that sufficient space is available to install the caisson immediately adjacent to the existing bridge.

As was mentioned before in Section 5.1.1.2, the use of drilled caissons entail two (2) options, namely caissons extended a sufficient distance into the competent overburden or caissons extended into the bedrock.

Caissons extended at least 2.0 m into the very dense or hard till with a minimum N-value of 50 blows/0.3 m can be designed for the following geotechnical resistances, provided that a minimum caisson length of 5.0 m below the bottom of pile cap can be maintained:

- Factored Geotechnical Resistance at U.L.S. = 2200 kPa;
- Geotechnical Resistance at S.L.S. = 1500 kPa.

If the bottom elevation for the pile cap is to match that of the existing footing (i.e. El. 213.3 m), with the minimum pile length requirement of 5.0 m, the caissons will need to be extended to El. 208.3 m (i.e. 213.3 – 5.0 m) or below. As the till in Borehole 103 was found to consist of a basically non-cohesive material to El. 210.2 m, some dewatering may be required to facilitate the installation of the caissons, as the groundwater table was measured at about El. 214 m.

Alternatively, the caissons can be extended deeper into the underlying bedrock. As was discussed in Section 5.1.1.2, caissons extended to the surface of the relatively sound bedrock (at least 0.6 m below the bedrock surface) can be designed for the following resistances:

- Factored Geotechnical Resistances at U.L.S. = 3000 kPa;
- S.L.S. need not be considered.

Higher resistances can be utilized with further socketing into the bedrock but such values are not likely to be needed.

If caissons socketed into the bedrock are to be utilized, you may wish to verify the depths to the surface of the bedrock by advancing additional boreholes, as the present boreholes were not extended into the bedrock (i.e. terminated before reaching the bedrock).

The recommended caisson design values are applicable to commonly used caisson sizes in Ontario (i.e. between 0.76 m and 1.8 m diameter). However, the use of relatively smaller caisson sizes (i.e. between 0.76 m and 1.35 m) would be preferable as these are relatively easier and more efficient to install. For example, a 0.76 m diameter caisson will have a base area of $r^2\pi=(0.76/2)^2x3.1416=0.45\ m^2$. When designed for a U.L.S. value of 3000 kPa, the caisson would be capable of carrying an axial load of $0.45\ m^2$ x 3000 kN/m^2 = 1350 kN/caisson at U.L.S. Similarly, if a 1.2 m diameter caisson is used, then the caisson resistance at U.L.S. would be $(1.2/2)^2$x3.1416 x3000 kN/m^2 =3390 kN/caisson.

Proper penetration into the sufficiently competent overburden/bedrock 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.

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).

Difficulties may arise during the installation of the caissons due to the presence of granular (non-cohesive) soil types below the groundwater table (e.g. Borehole 103), necessitating dewatering, as well as due to the possible presence of cobbles, boulders and shale fragments in the till, along with possible hard layers in the shale bedrock. Some dewatering is also expected to be necessary to intercept and remove surface water and to pump out any perched water. Dewatering of interbedded pervious (i.e. granular) soils within the glacial till deposit may also be necessary. Temporary steel casing (liner) will be required during the construction of the caisson holes to prevent caving. The casing/liner 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 to prevent the deterioration of the base of the caisson. 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 possible presence of cobbles, boulders and shale fragments in the glacial till deposit. An NSSP should be issued to alert the Contractor of the presence of cobbles, boulders, shale fragments in the overburden, as well as possible dewatering requirements.

The tremie concrete method can be used, if desired or required, to reduce the degree of dewatering during the installation of the caisson foundations. Based on the borehole data, however, the use of the tremie concereting method is unlikely to be necessary.

Socketing into the bedrock may possibly (though unlikely) have to be advanced by rock coring or churn drilling since the shale bedrock at the site may contain relatively strong layers.

5.1.2.3 Driven Pile Foundations

Driven pile foundations, including steel H-piles, are not recommended due to the anticipated soil conditions (i.e. hard and very dense glacial till with frequent cobbles and boulders), and the fact that piles will unlikely be driven to sufficient depths to satisfy minimum length requirements without pre-augering. As well, the vibrations induced by the heavy driving of the piles may induce settlement or instability of the existing foundations. It is our opinion that the use of driven piles including steel H-piles, is not a good choice from reliability point of view.

5.1.2.4 Micropile Foundations

An alternative which may be considered is the use of micropiles to support the central pier. Under normal circumstances, micropiles are less cost effective than caissons or spread footings, but for this present case they may present an attractive solution if overhead restrictions under the existing bridge present problems for the construction of the caissons and due to shoring requirements for spread footings.

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. Due to the small pile diameter (typically 160 mm to 260 mm), the end-bearing contribution in micropiles is generally neglected. The grout/ground bond strength achieved is influenced primarily by the surrounding soil or rock 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 900 kN/micropile are available (at U.L.S. and S.L.S. will typically not govern). In this present case, up to a similar resistance would be available depending on the diameter and penetration into the sound bedrock. The lateral resistances would also depend on the diameter, as well as, to a lesser extent, on the socket length into the bedrock.

The use of micropiles is generally less economical than spread footing foundations and caissons due to the required numbers of micropile to achieve similar geotechnical resistance to conventional foundations.

The axial and horizontal resistances of micropiles and other details regarding the design of micropiles can be discussed with specialist contractor and we will be pleased to further comment on this type of support should you wish us to do so.

In summary, the use of spread footing foundations matching the existing foundations or caissons appear to be the two more favourable solutions with respect to reliability and cost, with the prevailing subsurface conditions.

Depending on the choice of foundations and shoring method, monitoring which may include instrumentation may be necessary during the construction.

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 H.

Free-draining backfill materials (i.e. Granular 'A' or Granular 'B', OPSS 1010) and the provision of drain 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

Coefficients of Lateral Earth Pressure:

Level Backfill	Backfill Sloping at 3H:1V	Backfill Sloping at 2H:1V
K_a=0.27	K_a=0.34	K_a=0.40
K_o=0.43	K_o=0.56	K_o=0.62

Compacted Granular 'B' Type I

Angle of Internal Friction ϕ=32° (unfactored)

Unit Weight = 21 kN/m^3

Coefficient of Lateral Earth Pressure:

Level Backfill	Backfill Sloping at 3H:1V	Backfill Sloping at 2H:1V
Ka=0.31	Ka=0.42	Ka=0.54
Ko=0.47	Ko=0.66	Ko=0.76

NOTE:

K_a is the coefficient of active earth pressure.

K_o is the coefficient of earth pressure at rest.

These values are based on the assumption that the backfill behind the retaining structure is free-draining and adequate drainage is provided.

The earth pressure coefficient adopted will depend on whether the retaining structure is restrained or some movement can occur such that the active state of earth pressure can develop. The effect of compaction during construction should also be taken into account in the selection of the appropriate earth pressure coefficients in accordance with Clause 6.9.2 of CAN/CSA-S6-06 CHBDC. The use of vibratory compaction equipment behind the retaining walls should be restricted in size as per current MTO and municipal practice. Vibration generated by traffic should also be considered in the selection of appropriate earth pressure coefficients.

5.3 Seismic Design Data

Brampton is considered to be in Velocity-related seismic zone (Z_v) 1 according to Table A3.1.1 of the CAN/CSA-S6-06 CHBDC. Subsection 4.6.1 of the CHBDC indicates that the seismic analysis is not required for bridges in this zone.

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.

OPSS 539 – Protection Systems

SP902S01 – Excavation and Backfilling to Structure

The boreholes show that the excavations can be expected to extend through some fill materials including topsoil, underlain by glacial tills which generally consist of clayey silt to silty clay tills along with silty sand to sandy silt tills. The presence of surficial interglacial granular deposits was also noted overlying the glacial tills in some of the boreholes. Three of the boreholes were extended relatively deeper to contact the bedrock at Elevations ranging between 206.9 and 205.1 m or at about 8 m to typically 10 m below o.g. levels. The overburden materials can be classified as follows:

Granular Embankment (Pavement) Fill	Type 2 soil
Fill	Type 3 soil above water table
	Type 4 soil below water table
Upper Granular Deposits	Type 3 soil above water table
	Type 4 soil below water table
Silty Sand to Sandy Silt Till	Type 2 soil above water table
	Type 4 soil below water table
Clayey Silt to Silty Clay Till	Type 1 above the water table
	Type 3 below the water table

At the time of our investigation, the groundwater table at the site was generally contacted at about El. 214 m. Depending on the conditions at the time of the construction, dewatering can therefore be expected for excavations extending below this elevation. In addition, some dewatering may be necessary to intercept and remove the surface water, as well as dewatering due to a possible perched water table.

The excavations for the construction of the foundations for the widening will likely involve shoring.

If the proposed pier foundations are to match that of the existing pier foundation, the excavations will likely be extended to about El. 213.0 m since the information available to us (which should be verified) indicate that the existing pier is supported on a spread footing foundation with a bottom elevation of 213.3 m. In this case, the excavation will extend to about 2 m below the existing grades and the required shoring effort should be reasonably straightforward. As the excavation will be extended to below the water levels recorded in most of the boreholes (i.e. generally El. 214 m ±), dewatering should be anticipated. Depending on how quickly the excavation is carried out and concrete poured after excavating to final subgrade level, pumping from perimeter ditches and filtered sumps may suffice if the soil is basically a low permeability cohesive material (i.e. clayey silt to silty clay till, as in Boreholes 2 and 104). If and where, however a basically granular type of soil, such as the upper granular soils or the silty sand to sandy silt till (e.g. Borehole 103) is encountered, then more extensive dewatering would be needed, possibly requiring deep wells and/or well points.

In the case of abutment foundations, more extensive shoring can be expected. As was discussed in Section 5.1.1, if the new foundations are to match the existing foundations, then compacted Granular 'A' support must be provided. To implement this, whereas the footing elevation will be at about El. 217 m, excavations will have to extend considerably deeper (i.e. to about El. 215.1 to 212.7 m) in order to remove the existing soils to the surface of the suitable natural subgrade. This means that the shoring will need to support not only the existing footings, but also the underlying soils to a depth of about 2 m to 4 m below the underside of the footing. In addition, the shoring must be of an unyielding type such that no support loss is experienced by the existing foundations. As well, the shoring will probably be an L-shape support in order to retain portions of the existing embankment itself. In any event, the shoring construction sequence must be very carefully thought and designed; otherwise, considerable damage could occur to the existing structure. Furthermore, the section of the shoring below the bottom of the existing foundation level should not be withdrawn (e.g. steel sheet piling), as the existing foundations may be damaged during the withdrawal process; as well, a gap will be left when the shoring elements are withdrawn and this gap will eventually be filled with yielding soil which may cause lateral yield and settlements. As well, the units of the shoring may have to be of a permanent nature. For example, if timber lagging is used, this may eventually decompose and create voids.

In the case of a caisson or micropile type support, the excavation will probably be extended to about the footing level and probably no deeper, thus requiring less robust support in comparison with spread footing foundations. This aspect may render the use of caisson type foundations more attractive in comparison with spread footing type foundations.

The shoring system should be designed by a Professional Engineer, experienced in this type of work. All shoring should be in accordance with OPSS 539.

In Ontario, shoring typically consists of soldier pile and timber lagging or steel sheet piling (with or without bracing / rakers). In this instance, tiebacks will likely be required. Sheet piling may be objectionable for this project as it would likely induce vibrations, which could be detrimental to the existing structure. The soldier piles, if utilized, may need to be extended onto the shale bedrock. Tiebacks would extend into the shale bedrock, depending on the depth of shoring/height of the soils to be retained. We will be pleased to further discuss such a system, if you wish us to do so.

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 1a for shoring supporting excavations below the existing foundation levels, and 1b for the remaining types. 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 OPSS 539.

Table 5.4.1: Recommended Unfactored Parameters for Temporary Shoring Design

Soil Type	K_a	K_o	K_p	γ (kN/m^3)
Topsoil	0.55	0.72	1.8	14.0
Granular Pavement Fill	0.32	0.49	3.1	21.0
Approach Embankment Fill	0.41	0.60	2.2	18.0
Sand / Silty Sand	0.32	0.49	3.1	20.5
Gravelly Sand	0.31	0.47	3.2	21.5
Cohesive Till	0.33	0.50	3.0	20.5
Granular Till	0.29	0.45	3.4	21.5
Weathered Shale	0.26	0.42	3.6	21.5
Shale	0.25	0.40	3.8	22.0

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

5.5 Embankment Widening

Based on the available data, foundation failures are not anticipated for widened approach embankments with side slopes of 2H:1V or flatter, provided that all unsuitable soils are removed as per MTO standards, including all topsoil and other unsuitable, weak or very loose soils.

Based on the findings of the boreholes and provided that unsuitable soils will be properly stripped under the footprint of the embankment (i.e. both existing and the widened section), the anticipated foundation settlements (including the settlement of the existing embankment) under the stresses induced by the proposed 3.6 m widening on both sides are about 15 mm, while another 5 mm of settlement of the widened embankment can occur under its own weight, bringing the total settlement to about 20 mm. Based on the borehole data, the anticipated settlements of the existing embankments and that of the natural subgrade under the additional stresses imposed by the widening 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 three months. In our opinion, settlements of these orders of magnitude require neither surcharging nor preloading. We recommend however paving of the roadway (i.e. placing of the asphaltic concrete) be delayed by as much as practicable, but not less than about three weeks after the final grades are reached, to effect some of these settlements in order to minimize the possibility of inducing cracks in the flexible pavement after construction.

The existing embankment side slopes will need to be properly benched as per MTO standards (OPSD 208.010) where the embankment is to be widened. The materials used for the construction of the embankment fills should consist of approved, acceptable earth fill (e.g. select subgrade materials-OPSS 1010). Fill used for construction of the widening should be in accordance with OPSS 212 and fill placement should meet or exceed the requirements of OPSS 501 and OPSS 206. Construction should be in accordance with OPSS 206. Quality assurance should be provided as per MTO Standard 501.08 (OPSS 501).

5.6 Frost Protection

Design frost protection depth for the general area is 1.2 m. Therefore, a permanent soil cover of at least 1.2 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 J, are an integral part of this report.

For and on behalf of Coffey Geotechnics Inc.

for **Winnie Chan, E.I.T.**

Ramon Miranda, P.Eng.

Zuhtu Ozden, P.Eng.

Appendix G

Summary of Foundation Alternatives

Summary of Foundation Alternatives for Abutment Widening

Foundation Type	Advantage/ Disadvantage	Risks/Consequences	Relative Costs	Recommendations
Spread Footings on Compacted Granular 'A' Fill	Cost effective Moderate cost Will match existing foundations. May require extensive excavations and will require shoring which will increase costs	The shoring must be carefully designed and executed. Monitoring during the construction may be required due to the closeness to the existing structure.	Moderate cost	A suitable option provided shoring is properly designed and constructed.
Driven Piles	Steel H-piles are more suitable being low displacement piles. Vibration monitoring is essential.	Cobbles, boulders and shale fragments may be encountered during the installation, which may present problems. Preaugering may be required to achieve minimum pile lengths. Vibrations created during pile driving may cause damage to the existing bridge.	Moderate cost	Not recommended based on reliability.
Drilled and cast-in-place Concrete piles (drilled caissons)	Less vibration created than driven piles. Will require less sophisticated shoring effort in comparison with spread footings	The presence of cobbles, boulders and shale fragments may present problems during the installation of drilled caisson foundations. Low overhead under the existing bridge and confined area for the installation of caissons as well as existing embankment fills (requiring shoring) will present problems for caisson installation.	Moderate cost	A feasible option. Preferred option for the bridge widening from reliability point of view, provided possible low overhead conditions and space restrictions will not create problems during the construction.
Micropile Foundations	Minimizes vibrations and dewatering. Can be installed in low overhead conditions	Cost effectiveness is a main concern	Expensive due to special equipment / material and specialist contractor	A feasible option but more expensive than other options but can be considered if space restrictions for construction preclude the use of spread footings and caissons.

Summary of Foundation Alternatives for Central Pier Widening

Foundation Type	Advantage/ Disadvantage	Risks/Consequences	Relative Costs	Recommendations
Normal Spread Footings	Cost effective Moderate cost. Will match existing foundations Will require shoring and dewatering	Will require shoring and dewatering Monitoring during the construction may be required due to the closeness to the existing structures.	Low to Moderate cost	Preferred Option from economics point of view
Driven Piles	Steel H-piles are more suitable being low displacement piles. Vibration monitoring is essential.	Cobbles, boulders and shale fragments may be encountered during the installation, which may present problems. Preaugering will be required to achieve minimum pile lengths. Vibrations created during pile driving may cause damage to the existing bridge.	Moderate cost	Not recommended based on reliability.
Drilled and cast-in-place Concrete piles (drilled caissons)	Less vibrations created than driven piles. Will require less stringent shoring and dewatering in comparison with normal spread footing foundations	The presence of cobbles, boulders and shale fragments may present problems during the installation of drilled caisson foundations. Low overhead conditions under the existing bridge and confined area for the installation of the caissons may present some problems for caisson installation.	Moderate cost	A feasible option which may be an attractive (cost effective) solution if caissons are used to support the abutments.
Micropile Foundations	Minimizes vibrations, dewatering and shoring. Can be installed in low overhead conditions.	Cost effectiveness is a main concern	Expensive due to special equipment / material and specialist contractor	A feasible option but more expensive than drilled caissons.

Appendix H

List of Standard Drawings and Specifications

List of Standard Drawings and Specifications

OPSD

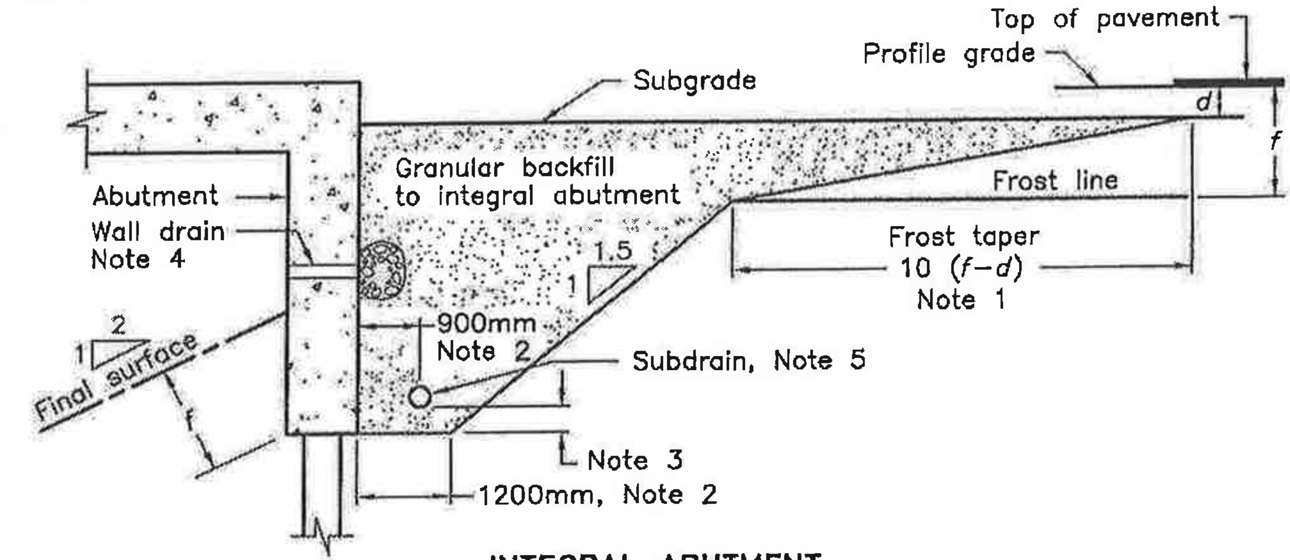
- 3101.150 WALLS, ABUTMENT, BACKFILL MINIMUM GRANULAR REQUIREMENT
- 208.010 BENCHING OF EARTH SLOPES

OPSS

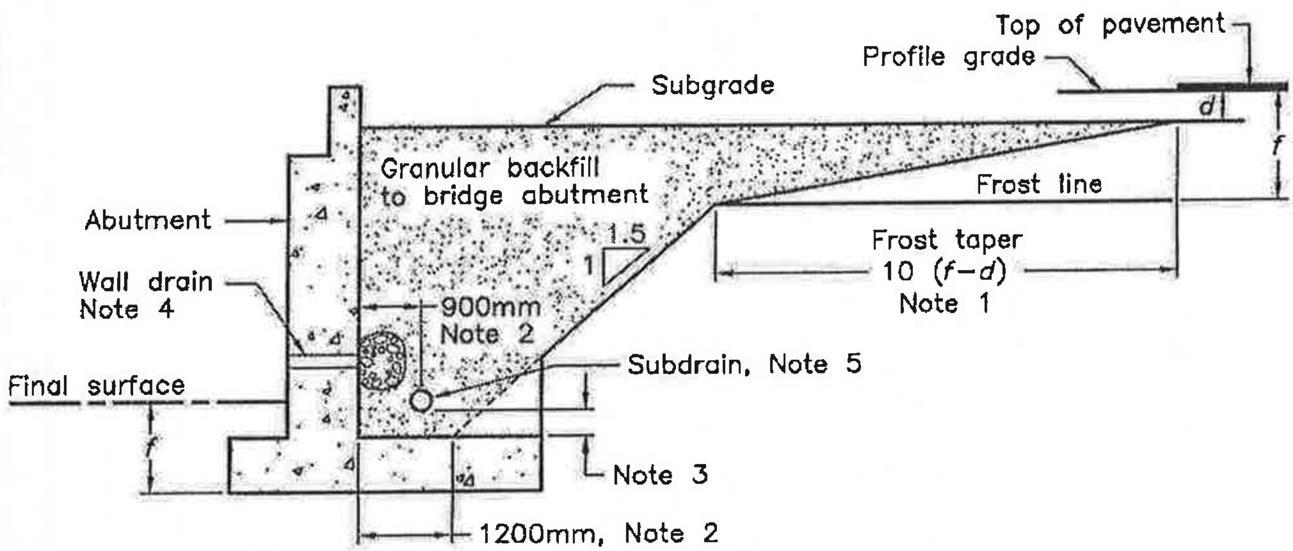
- 539 TEMPORARY PROTECTION
- 1010 MATERIAL SPECIFICATION FOR AGGREGATES - BASE, SUBBASE, SELECT SUBGRADE, AND BACKFILL MATERIAL
- 206 CONSTRUCTION SPECIFICATION FOR GRADING
- 212 CONSTRUCTION SPECIFICATION FOR. BORROW
- 501 CONSTRUCTION SPECIFICATION FOR COMPACTING

SP

- 902S01 EXCAVATION AND BACKFILLING TO STRUCTURES



INTEGRAL ABUTMENT



ABUTMENT

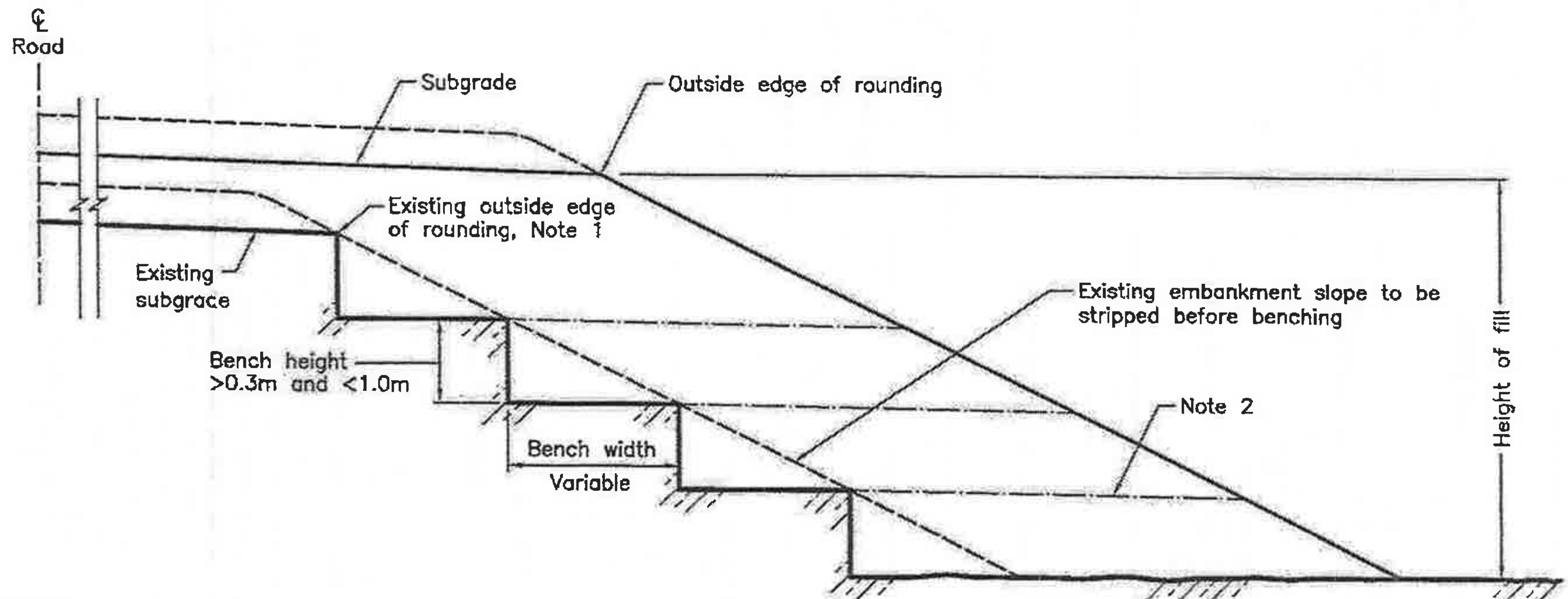
NOTES:

1 *d* = depth of combined base and subbase courses
f = frost penetration depth 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 backfill unless interrupted by the retaining wall or wingwall.
B Sections shown are parallel to centreline of roadway.
C Subdrain shall be installed with a 2% gradient behind wall.
D All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING | Nov 2010 | Rev 1

WALLS
ABUTMENT, BACKFILL
MINIMUM GRANULAR REQUIREMENT

OPSD 3101.150



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.

2 Benches are to be excavated one level at a time and the fill placed and compacted before the next bench is excavated.

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

B All dimensions are in metres unless otherwise shown.

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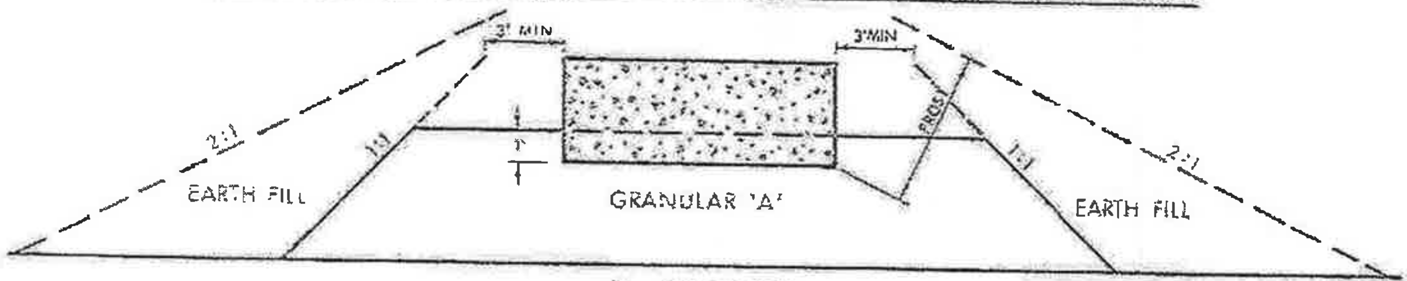
BENCHING OF EARTH SLOPES		OPSD 208.010

Appendix I

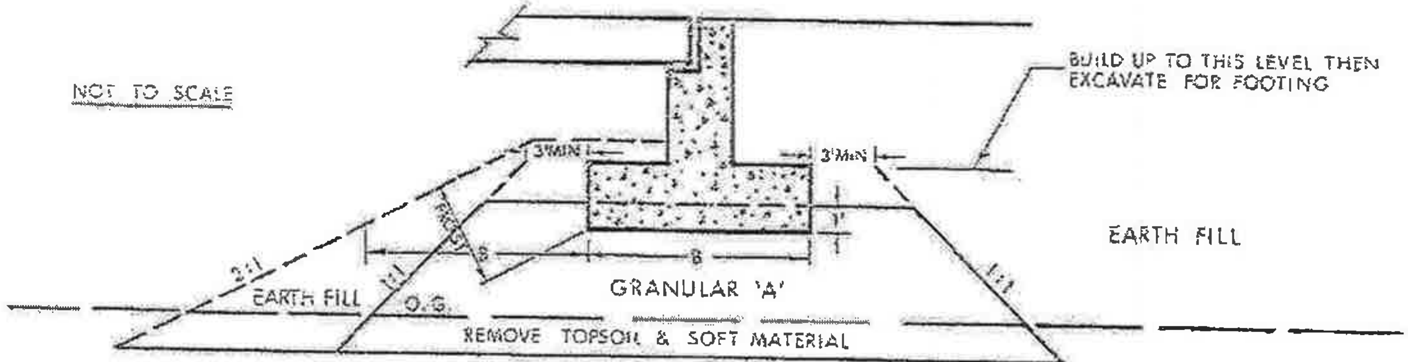
MTO Drawing

ABUTMENT ON COMPACTED FILL SHOWING GRANULAR 'A' CORE



X SECTION

NOT TO SCALE



LONGITUDINAL SECTION

NOTES:

1 - REMOVE TOPSOIL &/OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' & EARTH FILL.

2 - PLACE GRANULAR 'A' & EARTH FILL TO TOP OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT M.T.C. STANDARDS.

3 - EXCAVATE COMPACTED GRANULAR 'A' & EARTH FILL FOR FOOTING.

Appendix J

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.

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