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SPECIALISTS MANAGING THE EARTH

FOUNDATION INVESTIGATION AND DESIGN REPORTS – COBOURG CREEK EAST STRUCTURE WIDENING HIGHWAY 401, COBOURG, ONTARIO, G.W.P. NO. 205-00-01, GEOCRES 30M16-42

AECOM

TRANETOB10134AA-AG
August 09, 2011

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

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SPECIALISTS MANAGING THE EARTH

August 09, 2011

AECOM
5080 Commerce Boulevard
Mississauga, ON L4W 4P2

Attention: Ms. Peggy Baleka

Dear Madam:

RE: Foundation Investigation and Design Reports, Cobourg Creek East Structure Widening Highway 401, Cobourg, Ontario G.W.P. No. 205-00-01

Coffey Geotechnics Inc (Coffey) is pleased to present the Foundation Investigation and Design Reports for the proposed Cobourg Creek East structure widening on Highway 401, Cobourg, Ontario.

Please call us on 416 213 1255 should you require further clarification on any aspects of the reports.

For and on behalf of Coffey Geotechnics Inc.

Delfa Sarabia, M.Eng.

Senior Geotechnical Engineer

Ramon Miranda, P.Eng.

Principal

Zuhtu Ozden, P.Eng.

Senior Principal

Distribution: Original held by Coffey Geotechnics Inc.

1 hard copy to AECOM

1 hard copy to MTO Project Manager

1 hard copy to MTO Pavements and Foundation Section

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FOUNDATION INVESTIGATION REPORT COBOURG CREEK EAST STRUCTURE WIDENING, HIGHWAY 401 COBOURG, ONTARIO G.W.P. NO. 205-00-01, GEOCRES 30M16-42

AECOM

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CONTENTS

FOUNDATION INVESTIGATION REPORT
COBOURG CREEK EAST STRUCTURE WIDENING
HIGHWAY 401, COBOURG, ONTARIO
G.W.P. 205-00-01

1 INTRODUCTION

At the request of AECOM, Coffey Geotechnics Inc. (Coffey) has prepared this foundation investigation report for the proposed Cobourg Creek East structure widening on Highway 401, Cobourg, Ontario. The work was carried out as part of the Highway 401 Expansion (6-Laning) from Burnham Street to approximately 2.0 km east of Nagle Road, within the Town of Cobourg and Township of Hamilton, Ontario. The foundation investigation was generally carried out in accordance with Coffey proposal (Reference PO 9236, dated May 25, 2009) and the requirements of the RFP.

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

This report provides factual information concerning subsurface conditions, in situ test results and laboratory test results, based on the foundation investigation undertaken.

2 SITE DESCRIPTION AND PHYSIOGRAPHY

2.1 Site Description

The Site is located at Station 17+915 on Highway 401, just west of County Road 45 near Cobourg, Ontario.

Direction of flow in Cobourg Creek is southerly. The width of the creek was about 10 m and the water depth in the creek was about 1 m at the time of our investigation. These probably correspond to normal to low flow conditions. The GA drawings prepared in 1958 show the high and average water levels in the creek at El. 94.7 m (310.6 ft) and El. 93.1 m (305.5 ft).

The existing structure over Cobourg Creek East is a three-span structural steel girder bridge with a concrete deck and asphalt wearing surface. The structural steel girders are supported on concrete piers and abutments constructed at a 24 degree skew to the road alignment.

The existing approach embankments and creek bank, which are approximately 2 to 5 m high, do not exhibit any apparent signs of slope instability or excessive erosion. As well, in the immediate vicinity of the existing bridge, there are no signs of excessive settlements/unusual cracking or deformations in the pavement.

Photographs of the Site are presented in Appendix E.

2.2 Physiography

According to "The Physiography of Southern Ontario" by L.J. Chapman and D.F. Putnam, 1984, the Cobourg Creek East structure widening is located within the physiographic region known as the Iroquois Plain. The Iroquois Plain was previously inundated by a body of water known as Lake Iroquois, the forerunner of the present Lake Ontario. Iroquois Plain at Cobourg is about three and a half mile in width and has peculiar belted pattern. The land within the project area is almost flat and is covered by glaciolacustrine deposits overlying sandy glacial till deposits.

The bedrock underlying the project area is known to consist of limestone, dolostone, shale, arkose and sandstone from the Simcoe Group of the Middle Ordovician (Bedrock Geology of Ontario, Southern Sheet, Map 2544).

3 METHOD OF INVESTIGATION

3.1 Fieldwork

The fieldwork for the investigation was carried out between June 2010 and September 2010 and comprised of drilling sixteen boreholes (B1 to B16) at the locations shown on the Borehole Location Plan, Drawing 1. Table 1 below presents a summary of the borehole details.

Table 1: Borehole Details

Borehole No./ Location	Station	Offset from Hwy 401 C/L	Existing Ground Elevation (m)	Drilled/Tested Depth (m)
B3 West Abutment	17+903	20 m Left of C/L	96.3	23.7
B4 West Abutment	17+882	17 m Right of C/L	96.6	12.7/14.8
B9 East Abutment	17+947	19 m Left of C/L	96.7	25.1
B10 East Abutment	17+930	19 m Right of C/L	96.3	25.1
B5 West Pier	17+910	26 m Left of C/L	94.7	16.5
B6 West Pier	17+899	18 m Right of C/L	94.0	12.7/16.3
B7 East Pier	17+931	19 m Left of C/L	93.6	17.4
B8 East Pier	17+920	18 m Right of C/L	93.9	19.1
B1 West Approach Embankment	17+882	20 m Left of C/L	96.2	17.2/17.7
B2 West Approach Embankment	17+871	19 m Right of C/L	96.7	12.7/14.8
B11 East Approach Embankment	17+967	19 m Left of C/L	96.9	18.4
B12 East Approach Embankment	17+950	20 m Right of C/L	97.2	20.0
B13 West Roadway Protection	17+891	12 m Left of C/L	97.2	8.1

Borehole No./ Location	Station	Offset from Hwy 401 C/L	Existing Ground Elevation (m)	Drilled/Tested Depth (m)
B14 West Roadway Protection	17+875	11 m Right of C/L	97.1	25.5
B15 East Roadway Protection	17+954	12 m Left of C/L	97.3	8.1
B16 East Roadway Protection	17+941	11 m Right of C/L	97.2	9.6

The borehole drilling was carried out by three different drilling subcontractors (Eastern Soil Investigation Limited, Strong Soil Search and Kodiak Drilling). The boreholes were drilled using either a track mounted (Bombardier or Mini-Mole) or truck mounted drilling rig. Each borehole was advanced using a solid flight auger or hollow stem augers within the soil materials, to depths of about 8.1 m to 22.1 m below the ground surface. Standard Penetration Tests (SPTs) were carried out in the overburden at selected depth intervals, to assess the soil strength and obtain samples for logging and testing purposes. SPTs were carried out 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 outside diameter (OD) 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 (or cohesionless) soils (gravels, sands and silts) or the consistency of cohesive soils (clays and clayey soils). Thin walled tube samples were collected within cohesive soils.

In Boreholes B3, B9, B10 and B14, rock was cored by at least 3 m to a maximum depth of 25.5 m below the ground surface, using NQ coring technique. Rock core samples were boxed and colour photographed.

The soil and rock samples were described in the field, placed in appropriate containers, labelled and transported to our Etobicoke geotechnical laboratory where the samples underwent further detailed visual examination and samples were selected for geotechnical laboratory testing.

Dynamic Cone Penetration Tests (DCPT) were carried out in Boreholes B1, B2, B4 and B6 below the drilled depths. The DCPT consists of driving an uncased 50 mm diameter cone, attached to A-size drill rods, with a driving energy of 475 kJ (63.5 kg hammer free falling for a distance of 0.76 m) per blow, continuously. The number of blows for each 0.3 m of penetration is recorded, providing an indication of the relative changes in the soil density and/or consistency with respect to depth.

Groundwater levels and inflows observed in the open boreholes during drilling were recorded. In Boreholes B7 and B10, a piezometer was installed to enable long term groundwater level monitoring. The remaining boreholes were grouted upon their completion using a cement/bentonite mixture as per MTO procedures.

The borehole locations were located on Site using existing site features. The borehole location coordinates and ground elevations were subsequently measured by the client's surveyors and were provided to Coffey.

A Coffey representative was present during the drilling operations to direct sampling and testing, record test results and log materials encountered.

Appendix A presents the Record of Borehole Sheets and rock core photographs.

3.2 Laboratory Testing

Soil and rock samples obtained during the investigation were taken to our Etobicoke laboratory. The following tests were performed on selected soil samples:

- Natural moisture content tests;
- Unit weight tests;
- Grain size analyses (sieve);
- Grain size analyses (sieve and hydrometer tests);
- Atterberg Limits tests; and
- Consolidation tests.

The results of natural moisture content and unit weight tests are presented on the Record of Borehole Sheets in Appendix A. Appendix B presents laboratory test results sheets.

4 SUBSURFACE CONDITIONS

Detailed descriptions of the materials encountered in the boreholes are presented on the Record of Borehole Sheets in Appendix A, which also includes rock core photographs. Explanation of Terms Used in Report is presented in Appendix F.

Records of boreholes, drilled at the Site by MTO in 1957 for the existing bridge are presented in Appendix C.

Drawings 2 and 3 present generalized subsurface profiles along the proposed Cobourg Creek East structure widening.

In general, below topsoil and fill (including pavement fill and embankment fill), the Site is underlain by native soils consisting of upper granular soils, clayey silt and lower granular soils, and bedrock. The bedrock encountered in the boreholes consist of limestone and the surface of the bedrock was inferred or proven at depths ranging from 17.4 to 22.1 m below the existing ground surface or at Elevations 76.2 to 74.2 m.

The Record of Borehole Sheets and sections indicate the subsurface conditions only at the borehole locations. Note that the material boundaries indicated on the logs are approximate and based on visual observations. These boundaries typically represent a transition from one material type to another and should not be regarded as an exact plane of geological change. It should be pointed out that the subsurface conditions may vary across this Site.

The following summarizes the surface conditions encountered in the boreholes.

4.1 Topsoil

Topsoil was encountered at the ground surface in Boreholes B1 to B12 which were drilled at the bottom of the existing bridge approach embankment and near the existing bridge piers. The thickness of topsoil at these borehole locations ranged from 0.1 m to 0.2 m.

Note that in our experience, the thickness of organic rich soils frequently varies in between and beyond borehole locations.

4.2 Fill

The fill encountered in the boreholes can be classified into three categories, namely pavement fill, embankment fill (in boreholes drilled from the road embankment – i.e. Boreholes B1, B2, B3, B4, and B9 to B16), and construction fill (Boreholes B5, B6, B7 and B8).

4.2.1 Pavement Fill

In Boreholes B13 to B16 drilled on the road level, 240 to 300 mm thick asphalt over 100 mm to 560 mm thick granular pavement fill (sand and sand and gravel) was encountered from the surface.

Standard Penetration Tests yielded SPT N-values of 14 to 47 blows/0.3 m within the granular pavement fill layer, indicating compact to dense condition.

4.2.2 Embankment Fill

Below the topsoil and pavement fill, embankment fill was encountered in all the boreholes except for Boreholes B5, B6, B7 and B8. The embankment fill in this area generally showed two distinct layers namely, sandy silt to silty sandy, sand and gravel typically within the upper zone and clayey silt within the lower zone. The embankment fill was found to extend to depths of 4.4 to 5.5 m below the existing road surface or to Elevations ranging between 92.3 m and 91.7 m (probably representing the original ground surface level before the existing bridge was constructed minus the stripped material thickness).

Upper Embankment Fill

The boreholes indicated that the upper zone of embankment fill, about 1.0 m to 4.6 m thick, typically consists of silty sand to sandy silt with some gravel and clay content, but ranges from sandy gravel to sandy silt. As such it is classified as a granular (non-cohesive) material. Clayey pockets and trace rootlets were encountered within this layer. Grain size distribution analyses carried out on four samples taken from the upper zone of the embankment fill indicate the following distribution, as shown in Figure B1, in Appendix B.

Gravel:	7 – 66 %
Sand:	27 – 48 %
Silt and Clay:	7 – 45 %

SPT N-values of 5 to 61 blows/0.3 m were recorded within this upper zone of the embankment fill indicating varying relative densities from very loose to very dense condition, with typical N-values between 10 and 25 blows/0.3 m (i.e. generally compact).

Lower Embankment Fill

The lower zones of the embankment fill typically consist of clayey silt with some sand and traces of gravel.

The boreholes indicated the presence of organics, wood pieces, rootlets and cobbles within this layer. The thickness of the layer was found to be 0 to 4.2 m at the borehole locations. Grain size distribution analyses carried out on five samples taken from the lower zone of the embankment fill indicate the following distribution, as shown in Figure B2, in Appendix B.

Gravel:	0 – 4 %
Sand:	8 – 21 %
Silt:	52 – 60 %
Clay:	24 – 28 %

The results of Atterberg Limits testing carried out on three sample of the clayey silt fill showed the following, as presented in Figure B3, in Appendix B.

Liquid Limit:	24 – 26 %
Plastic Limit:	15 – 17 %
Plasticity Index:	8 – 9 %

The clayey silt fill is considered to have a low degree of plasticity.

SPT N-values of 4 to 28 blows/0.3 m were recorded within the clayey silt fill, indicating varying densities consistency from firm to very stiff.

One unit weight test was carried out on a clayey silt fill sample, B1 SS5, and yielded a bulk unit weight of 21.5 kN/m^3.

Construction Fill

Boreholes B5, B6, B7 and B8 were put down from the pier support level at Elevations 94.7 to 93.6 m and contacted other fill than embankment fill, probably fills placed during the construction of the existing bridge. In Boreholes B5, B7 and B8, this fill was found to extend to depths of 0.6 to 2.4 m or to Elevations 93.0 to 91.8 m, while in Borehole B6 it extended to 5.3 m below the ground surface or to Elevation 88.7 m.

Similar to the upper zone of the embankment fill, this fill was found to consist typically of silty sand to sandy silt with some gravel and clay size particles and occasional clayey zones and zones containing organics.

Grain size distribution analysis carried out on one sample taken from this fill, from Borehole B6 at a depth of about 3.3 m, indicate the following distribution, as shown in Figure B4, in Appendix B.

Gravel:	70 %
Sand:	23 %
Silt and Clay:	7%

4.3 Upper Granular Soils – Sand, Sandy Gravel and Sand & Gravel

Underneath the embankment fill, granular (non-cohesive) deposits described as typically sand, sandy gravel and sand and gravel were encountered at Elevations 93.0 to 88.7 m with thicknesses of about 0.3 to 3.4 m. These upper granular soils were not encountered in Borehole B10. Coarser materials such as cobbles were also encountered within these deposits.

The following are the grain size distributions of the selected six samples taken from these upper granular deposits, as presented in Figure B5, in Appendix B.

Gravel:	49 – 74 %
Sand:	23 – 48 %
Silt and Clay:	3 – 8 %

Finer granular materials (silty sand) were encountered in Boreholes B6 and B12. The following are the grain size distributions of the silty sand samples from Boreholes B6 and B12, as presented in Figure B6, in Appendix B.

Gravel:	7 – 8 %
Sand:	53 – 65 %
Silt and Clay:	28 – 39 %

Standard Penetration Tests yielded SPT N-values of 16 to in excess of 100 blows/0.3 m within these upper granular soils, indicating varying relative densities from compact to very dense. SPT blow counts corresponding to compact conditions were taken within the upper zone of the deposit just below the embankment fill. An isolated low blow count (8 blows/0.3 m) was recorded in Borehole B12 just below the embankment fill.

4.4 Clayey Silt

Underneath the upper granular deposit is a clayey silt deposit which was encountered at Elevations 91.8 to 87.3 m, with thicknesses ranging from 8.4 to 10.7 m. Boreholes B2, B4, B6, B13, B15 and B16 were terminated within this deposit at depths of 8.1 to 12.7 m below the existing ground surface or at Elevations 89.2 to 81.3 m.

The following are the grain size distributions of the selected nineteen samples taken from this deposit, as shown in Figure B7, in Appendix B.

Gravel:	0 – 1 %
Sand:	1 – 16 %
Silt:	61 – 91 %
Clay:	5 – 35 %

A thin silty sand layer was encountered within this deposit in Borehole B1 at a depth of about 7.8 m and the following is the grain size distribution, also presented in Figure B8, in Appendix B.

Gravel:	0 %
Sand:	56 %
Silt:	37 %
Clay:	7 %

Atterberg Limits tests conducted on the sixteen samples taken from this deposit indicated the following results, also shown in Figure B9, in Appendix B.

Liquid Limit:	17 – 28 %
Plastic Limit:	13 – 20 %
Plasticity Index:	4 – 8 %

The clayey silt is considered to have a low plasticity (i.e. a CL-ML to CL material).

Typical SPT N-values of 5 to 33 blows/0.3 m were recorded within this deposit. A relatively low N-value of 2 blows/0.3 m was also recorded in Borehole B6 (SS10). But this is believed to have been caused by disturbance during the drilling operations. Relatively higher N-values of 38 to 85 blows/0.3 m were recorded within this deposit on either near the interface between this deposit and the upper granular soils or the lower granular soils. These higher values are not considered to be representative of the condition of the deposit as they were influenced by the sand, gravel and cobble present near the interfaces between this deposit and the upper and lower granular soils. Field vane testing recorded typical shear strength values of 36 to greater than 100 kPa. Relatively low shear strength values of 24 and 28 kPa were recorded in Borehole B8 at 7.4 m and 8.4 m depths. Based on the SPT and field vane testing, the clayey silt deposit is considered to have a consistency of firm to very stiff, except within the upper and lower zones (i.e. interface with the upper and lower granular soil) as discussed above. Sensitivity values ranging from 2.3 to 4 and an isolated value of 5.7 were recorded within this deposit, indicating that the clayey silt has generally a low to medium sensitivity.

Consolidation tests were carried out on two samples from this deposit (samples B7 TW11 and B12 TW13). The test results are presented in Figures B10 and B11, in Appendix B. The results showed that the deposit is considered to have a possible pre-consolidation pressure in excess of the existing overburden pressure, $P'_c - P'_0$ of the order of 85 to 160 kPa or over consolidation ratio, OCR, equal to 1.9 to 2.3. A compression index, C_c, of about 0.3 and recompression index, C_r, of about 0.03 were calculated from the results. Coefficient of consolidation, c_v, of about 0.003 cm^2/sec was also calculated from the results.

We also assessed the consolidation characteristics of the clayey silt by plotting undrained shear strength, overburden pressure, P'_0 and 0.23 x P'_0 versus depth. The plots are presented in Appendix C. Figure C1 presents the measured undrained shear strengths versus elevation from all the boreholes drilled on the existing embankment and on the creek banks that is outside the existing embankment. Figures C2 and C3 present the distribution of undrained shear strength, overburden pressure, P'_0, and 0.23 x P'_0 versus the elevation for Boreholes B7 and B12, where the clayey silt was found to be relatively thick. It is commonly acknowledged that with Ontario clays if the measured undrained shear strength are in excess of 0.23 P'_0 line, the deposit may be over consolidated. Based on this criterion, the clayey silt deposit appears to be over consolidated, which is in concert with the results of the consolidation tests discussed above.

4.5 Lower Granular Soils – Sand, Silty Sand, Sand & Gravel and Silty Sand Till

Granular (non-cohesive) deposits were encountered below the clayey silt deposit. The granular deposits which consist of interglacial deposits of sand, silty sand, sand and gravel and a glacial deposit of silty sand till were encountered at Elevations 81.9 to 80.1 m with thicknesses of about 4.4 m to 7.6 m. Cobbles and boulders were encountered within these lower granular deposits, as auger grinding and SPT spoon bouncing were observed during drilling within these materials. Boreholes B1, B5, B11 and B12 were terminated within these deposits at depths of 16.5 to 20.0 m below the existing ground surface or at Elevations 79.0 to 77.2 m.

4.5.1 Interglacial Deposits

The grain size distributions of the three selected samples described as interglacial sandy gravel, gravel and sand and gravel deposits are presented in Figure B12, in Appendix B. These show the following grain size distribution.

Gravel:	49 – 55 %
Sand:	31 – 43 %
Silt and Clay:	7 – 20 %

4.5.2 Glacial Deposits

The grain size distribution of the four samples from silty sand till and sand till are presented below and also presented in Figure B13, in Appendix B.

Gravel:	2 – 27 %
Sand:	47 – 89 %
Silt and Clay:	9 – 42 %

Typical SPT N-values of 31 to in excess of 100 blows/0.3 m were recorded within these deposits, indicating dense to very dense relative densities. Relatively lower SPT blow counts or 17 to 24 blows/0.3 m were recorded in Boreholes B3, B10 and B12, just below the clayey silt deposit, indicating isolated compact conditions.

4.6 Bedrock

Bedrock was proven by coring in Boreholes B3, B9, B10 and B14, where the top of bedrock was encountered at depths of 20.6 to 22.1 m below the existing ground surface or at Elevations 75.7 to 74.2 m. The rock was cored to 3.0 m to 3.6 m depths below the top of bedrock surface. Boreholes B7 and B8 encountered auger refusal on possible bedrock at depths of 17.4 m and 19.1 m below the existing ground surface or at Elevations 76.2 and 74.8 m, respectively. From these observations, the surface of the bedrock seems to be dipping mildly from west to east, but conditions could be different beneath the watercourse itself.

Based on the rock cores recovered, the bedrock is described as a grey limestone with some dolomatic and argillaceous seam/zones. The recorded total core recovery (TCR) ranged from 80 to 100 %. Rock Quality Designation (RQD) values of 28 to 80 % were recorded. Based on these values, the rock mass quality can be described as poor to good but typically poor within the upper 1.5 m zone (fist core run) and fair to good below.

4.7 Groundwater Conditions

Groundwater levels were observed in the open boreholes while drilling and upon completion of each borehole. In Boreholes B3, B9, B10 and B14, rock coring was carried out where water was introduced into the borehole. Therefore, groundwater levels measured on completion of the boreholes may not be reliable.

Piezometers were installed in Boreholes B7 and B10 to monitor the groundwater levels over a prolonged period without interference from surface water. The groundwater levels observed during the investigation are presented on the Record of Borehole Sheets in Appendix A and are summarized in the following table.

Table 2: Groundwater Level Observations

Borehole No.	Date of Water Level Measurement	Measured Water Level Depth/Elevation (m)	Comments
B1	Jun 17 2010	3.4 / 92.8*	measured upon borehole completion
B2	Sep 9 2010	4.6 / 92.1*	measured upon borehole completion
B3	Jun 18 2010	5.0 / 91.3*	measured before rock coring
B4	Sep 8 2010	3.1 / 93.5*	measured upon borehole completion
B5	Jun 24 2010	3.8 / 90.9*	measured upon borehole completion
B6	Sep 7 2010	3.7 / 90.3*	measured upon borehole completion
B7	Aug 19 2010 Oct 15 2010	1.0 / 92.6 0.9 / 92.7	measured within the piezometer installed (installed June 16, 2010)
B8	Jul 15 2010	1.2 / 92.7*	measured upon borehole completion
B9	Jun 15 2010	6.7 / 90.0*	measured upon borehole completion
B10	Jul 15 2010 Aug 19 2010 Oct 15 2010	3.8 / 92.5* 3.7 / 92.6 3.4 / 92.9	First reading measured upon borehole completion and succeeding readings measured within the piezometer installed
B11	Jun 14 2010	6.4 / 90.5*	measured upon borehole completion
B12	Jul 13 2010	3.5 / 93.7*	measured upon borehole completion
B13	Aug 12 2010	5.3 / 91.9*	measured upon borehole completion
B14	Sep 8 2010	4.0 / 93.1*	measured upon borehole completion
B15	Aug 12 2010	4.7 / 92.6*	measured upon borehole completion
B16	Aug 27 2010	4.6 / 92.6*	measured upon borehole completion

Note: * Groundwater level measured not stabilized.

Based on the moisture condition of the soil samples and results of the piezometer readings, the Site groundwater level at the time of our investigations was at about Elevation 93.0 to 92.5 m.

The 1957 MTO investigation (as shown in Appendix D) indicated a water level at Elevation 93.1 m (305.48 ft) in May 1957 and a high water level at Elevation 94.7 m (310.6 ft).

It should be noted that groundwater levels are subject to variations due to the influence of rainfall, temperature, local drainage, seasons and other factors. There may also be potential for development of perched groundwater tables following periods of rainfall and groundwater may rise to the ground surface. In addition the water level in the watercourse would influence the groundwater level at the Site.

For and on behalf of Coffey Geotechnics Inc.

for **Delfa Sarabia, M.Eng.**

Senior Geotechnical Engineer

Ramon Miranda, P.Eng.

Manager, Transportation

LICENSED PROFESSIONAL ENGINEER
RAMON MIRANDA
PROVINCE OF ONTARIO

Zuhtu Ozden, P.Eng.

Senior Principal

REGISTERED PROFESSIONAL ENGINEER
Z. S. OZDEN
PROVINCE OF ONTARIO

Drawings

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS REFER TO RECORD OF BOREHOLE SHEETS.

METRIC

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

CONT No.

GWP: 205-00-01

SHEET

COBOURG CREEK EAST STRUCTURE WIDENING
BOREHOLE LOCATION PLAN

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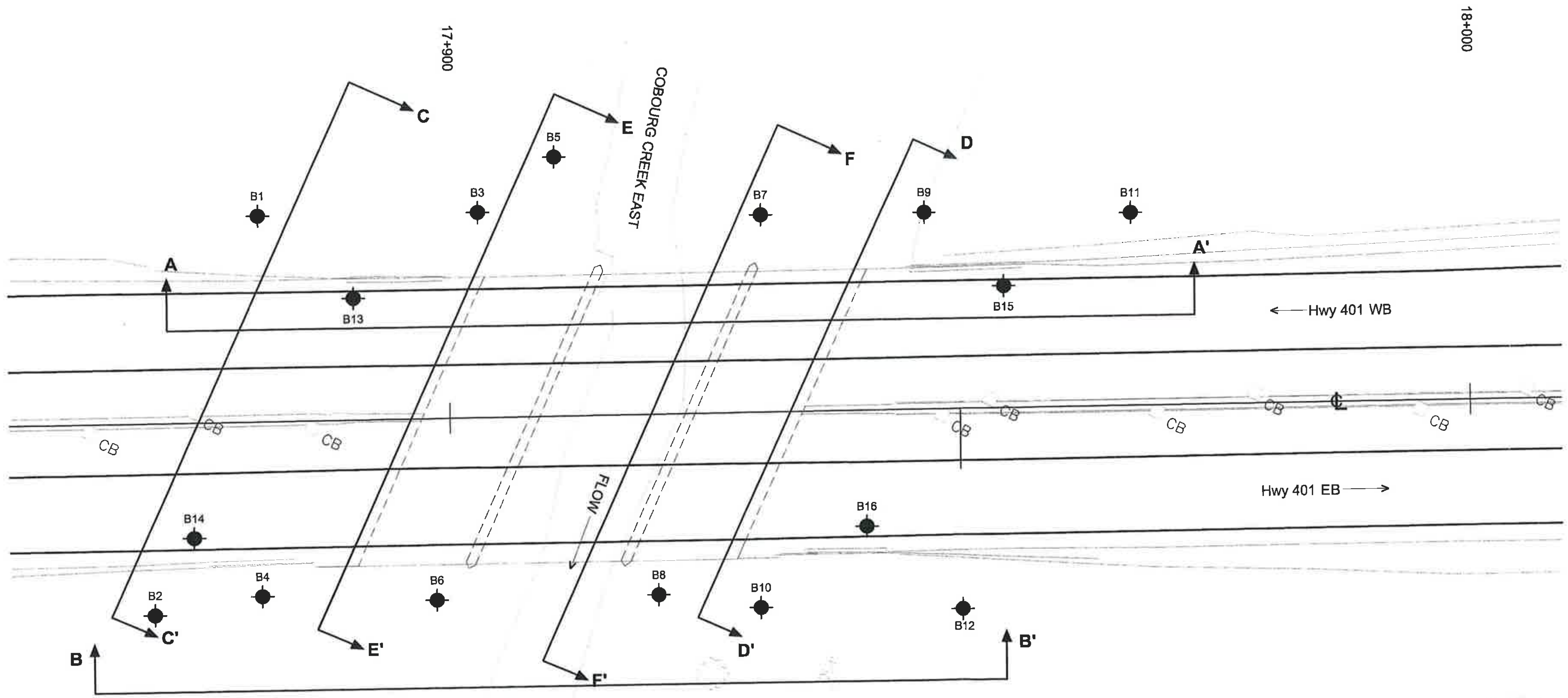

PLAN
SCALE
25 m 0 25 m

KEY PLAN
N.T.S.

LEGEND

Borehole

Section (see Drawings 2, 3 & 4)

No.	ELEVATION	EASTING	NORTHING
B1	96.2	410607.2	4872375.2
B2	96.7	410614.8	4872335.3
B3	96.3	410626.5	4872384.4
B4	96.6	410623.5	4872341.4
B5	94.7	410631.0	4872392.4
B6	94.0	410639.1	4872348.1
B7	93.6	410651.8	4872395.6
B8	93.9	410658.6	4872357.5
B9	96.7	410666.3	4872402.5
B10	96.3	410668.3	4872360.6
B11	96.9	410684.7	4872410.9
B12	97.2	410686.4	4872368.7
B13	97.2	410619.1	4872371.7
B14	97.1	410615.0	4872343.8
B15	97.3	410676.5	4872399.2
B16	97.2	410674.3	4872372.2

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

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

REVISIONS	DATE	BY	DESCRIPTION

Geocres No. 30M16-42

TRANETOB10434AA			DIST
SUBM'D	CHECKED	DATE Apr. 26, 2011	SITE
DRAW N SH	CHECKED RM	APPROVED ZO	DWG 1

METRIC

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

CONT No.
GWP: 205-00-01

COBOURG CREEK EAST STRUCTURE WIDENING
SOIL STRATA 1 OF 3

SHEET

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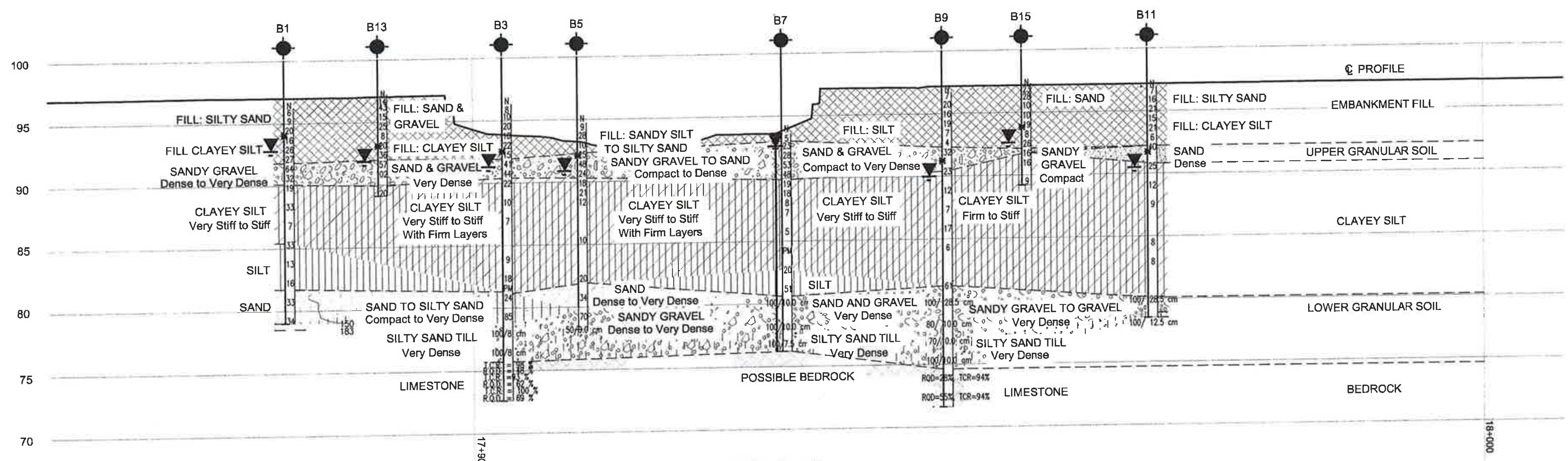


SECTION A-A'

HORIZONTAL SCALE

25 m 0 25 m

KEY PLAN
N.T.S.

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS AND DYNAMIC CONE PENETRATION TESTS REFER TO RECORD OF BOREHOLE SHEETS.

LEGEND

Borehole

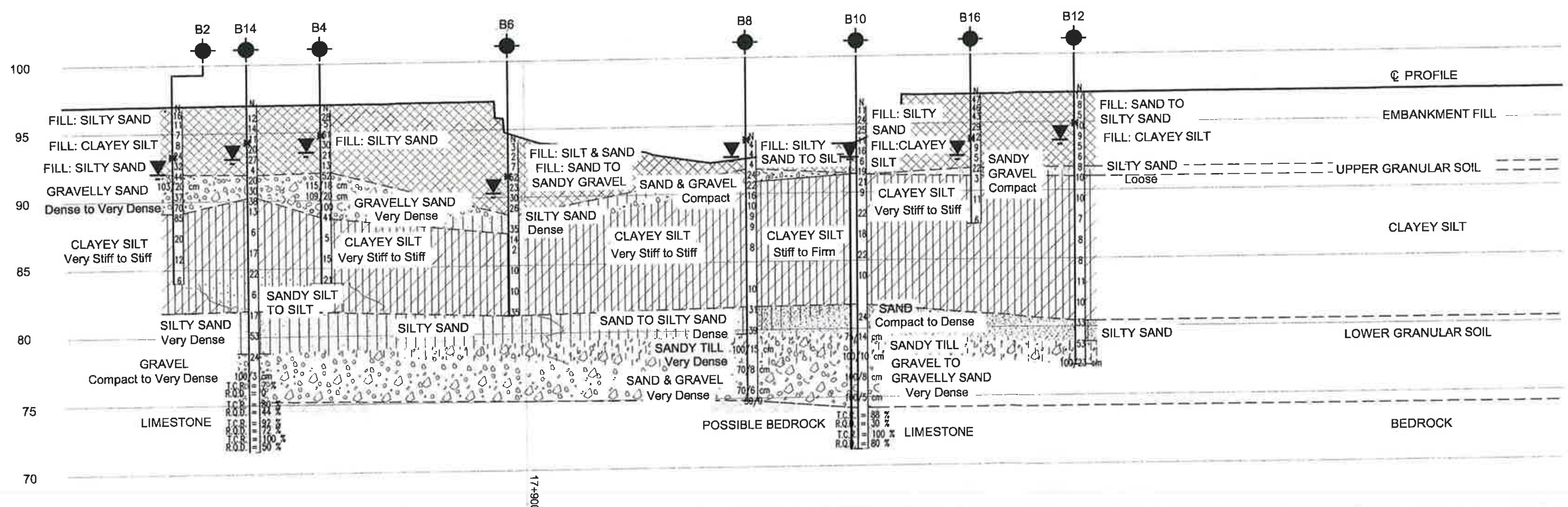
Water Level in Piezometer

N Blows/0.3m (Std. Pen. Test, 475 J/blow)

Piezometer

Water Level at Time of Investigation
(W. L. NOT STABILIZED)

No.	ELEVATION	EASTING	NORTHING
B1	96.2	410607.2	4872375.2
B2	96.7	410614.8	4872335.3
B3	96.3	410626.5	4872384.4
B4	96.6	410623.5	4872341.4
B5	94.7	410631.0	4872392.4
B6	94.0	410639.1	4872348.1
B7	93.6	410651.8	4872395.6
B8	93.9	410658.6	4872357.5
B9	96.7	410666.3	4872402.5
B10	96.3	410668.3	4872360.6
B11	96.9	410684.7	4872410.9
B12	97.2	410686.4	4872368.7
B13	97.2	410619.1	4872371.7
B14	97.1	410615.0	4872343.8
B15	97.3	410676.5	4872399.2
B16	97.2	410674.3	4872372.2



SECTION B-B'

HORIZONTAL SCALE

25 m 0 25 m

REGISTERED PROFESSIONAL ENGINEER
Z. S. OZDEN
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
RAMON MIRANDA
PROVINCE OF ONTARIO

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

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

REVISIONS	DATE	BY	DESCRIPTION

Geocres No. 30M16-42

TRANETOB10434AA			DIST
SUBM'D	CHECKED	DATE Mar 07, 2011	SITE
DRAWN SH	CHECKED RM	APPROVED ZO	DWG 2

METRIC

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

CONT No.
GWP: 205-00-01

COBOURG CREEK EAST STRUCTURE WIDENING
SOIL STRATA 2 OF 3

SHEET

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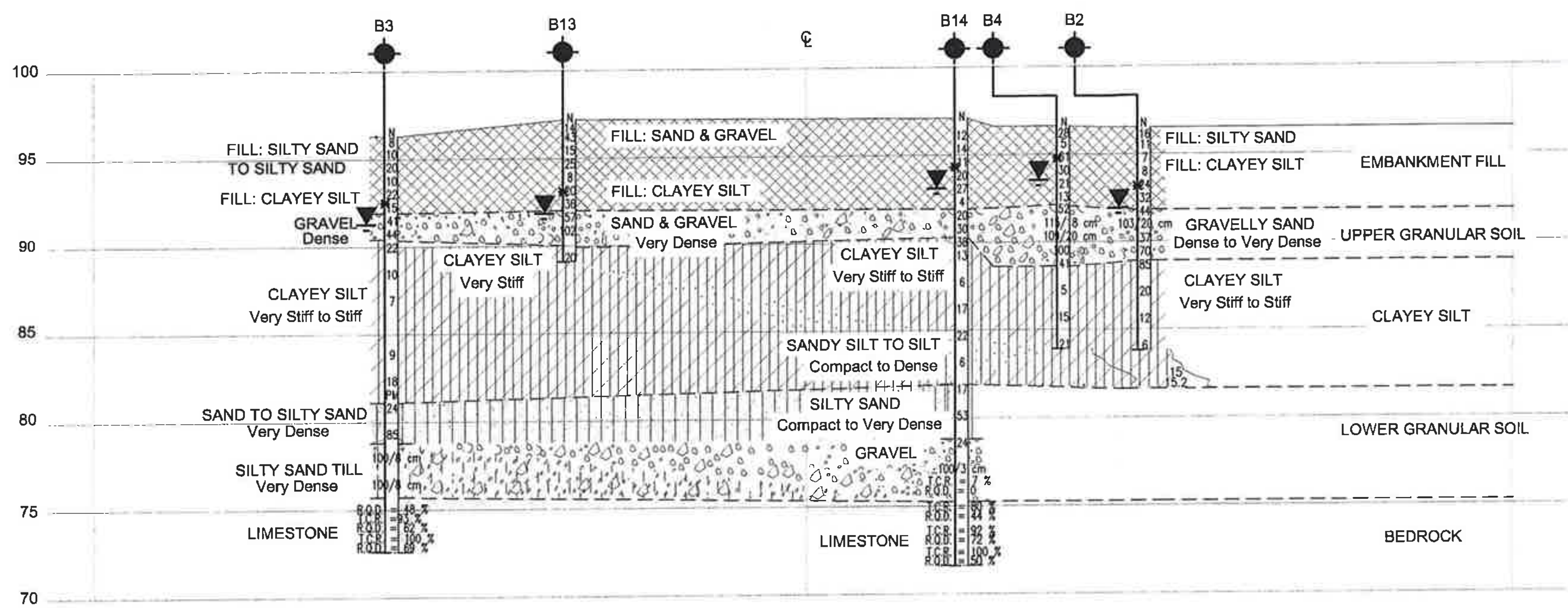
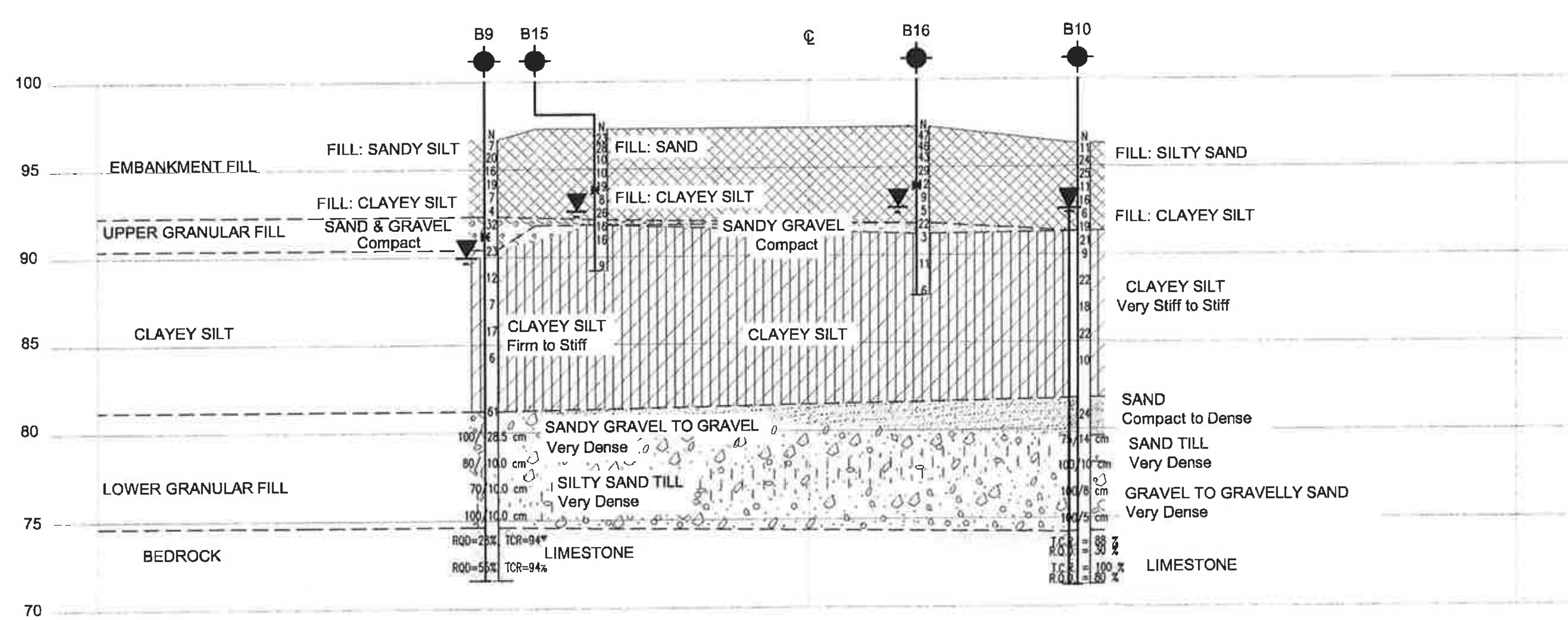


SECTION C-C'
HORIZONTAL SCALE
25 m 0 25 m



SECTION D-D'
HORIZONTAL SCALE
25 m 0 25 m

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS AND DYNAMIC CONE PENETRATION TESTS REFER TO RECORD OF BOREHOLE SHEETS.

KEY PLAN
N.T.S.

LEGEND

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

No.	ELEVATION	EASTING	NORTHING
B2	96.7	410614.8	4872335.3
B3	96.3	410626.5	4872384.4
B4	96.6	410623.5	4872341.4
B9	96.7	410666.3	4872402.5
B10	96.3	410668.3	4872360.6
B13	97.2	410619.1	4872371.7
B14	97.1	410615.0	4872343.8
B15	97.3	410676.5	4872399.2
B16	97.2	410674.3	4872372.2

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

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

REGISTERED PROFESSIONAL ENGINEER
Z. S. OZDEN
Aug 9/11
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
RAMON MIRANDA
Aug 9/11
PROVINCE OF ONTARIO

REVISIONS	DATE	BY	DESCRIPTION

Geocres No. 30M16-42

TRANETOB10434AA			DIST
SUBM'D	CHECKED	DATE Mar 04, 2011	SITE
DRAWN SH	CHECKED RM	APPROVED ZO	DWG 3

CONT No.
GWP: 205-00-01

SHEET

COBOURG CREEK EAST STRUCTURE WIDENING
SOIL STRATA 3 OF 3

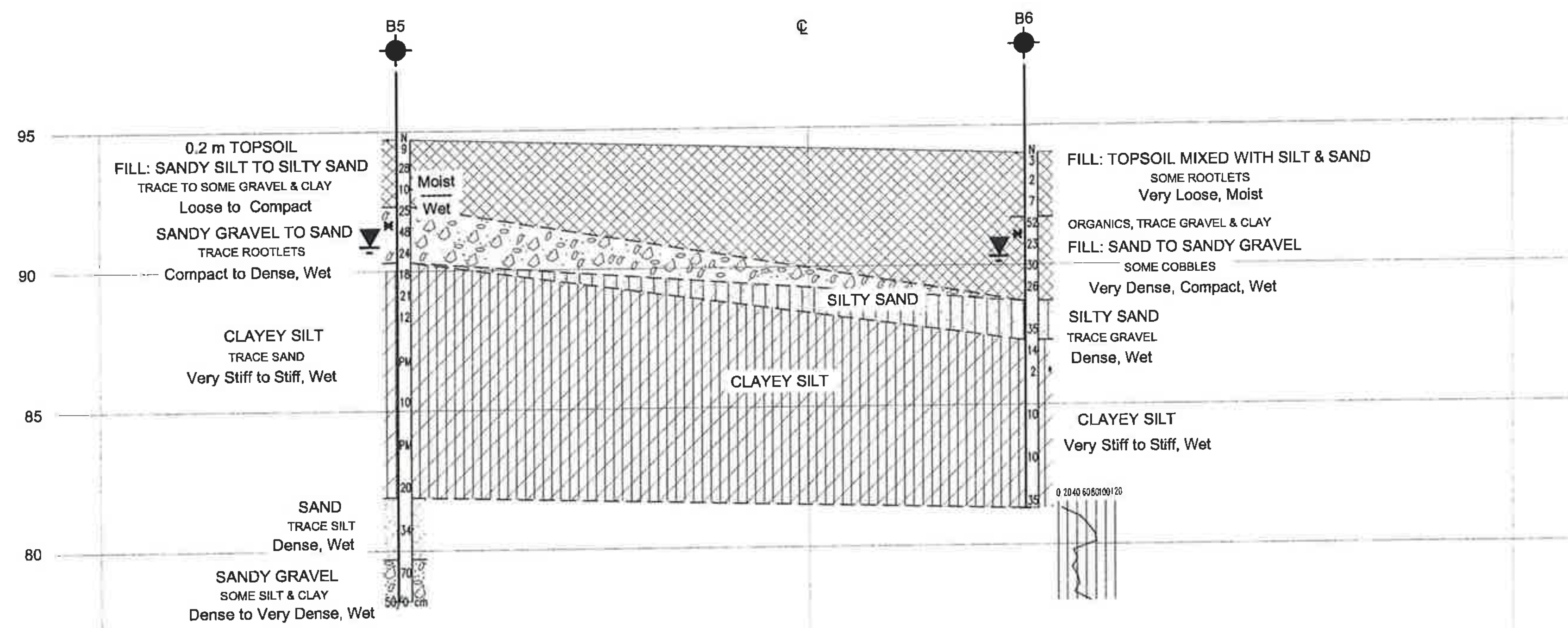
coffey geotechnics
SPECIALISTS MANAGING THE EARTH

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS REFER TO RECORD OF BOREHOLE SHEETS.

METRIC

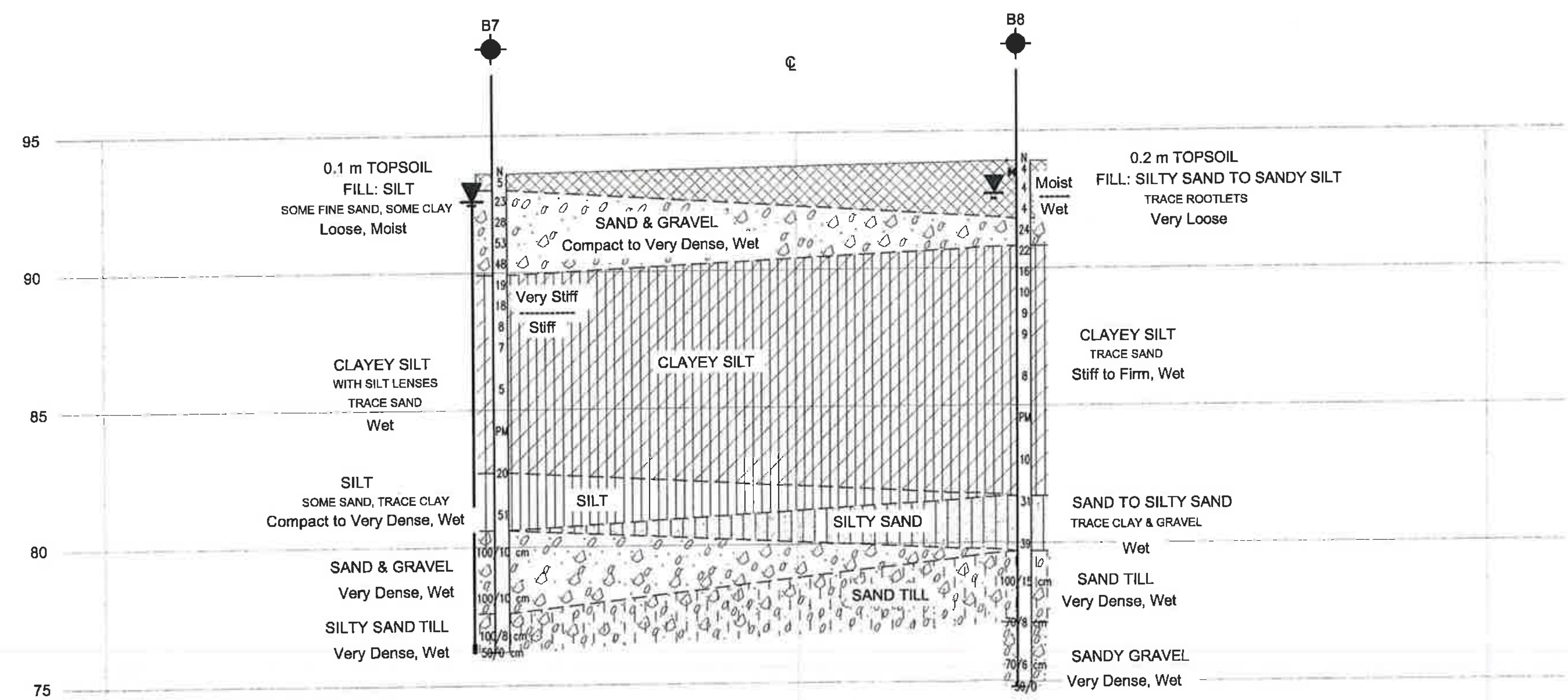
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES + METRES.



SECTION E-E'
SCALE

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS AND DYNAMIC CONE PENETRATION TESTS REFER TO RECORD OF BOREHOLE SHEETS.



SECTION F-F'
SCALE

KEY PLAN
N.T.S.

LEGEND

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

No.	ELEVATION	EASTING	NORTHING
B5	94.7	410631.0	4872392.4
B6	94.0	410639.1	4872348.1
B7	93.6	410651.8	4872395.6
B8	93.9	410658.6	4872357.5

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

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

REGISTERED PROFESSIONAL ENGINEER
Z. S. OZDEN
Aug 9/11
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
RAMON MIRANDA
Aug 9/11
PROVINCE OF ONTARIO

REVISIONS	DATE	BY	DESCRIPTION

Geocres No. 30M16-42

				DIST
	TRANETOB10434AA			
SUBM'D	CHECKED	DATE	Apr.26, 2011	SITE
DRAWN SH	CHECKED RM	APPROVED	ZO	DWG 4

Appendix A

Record of Borehole Sheets and Rock Core Photographs

Ministry of Transportation
Ontario

Foundation Design

TRANETOB10434AA: Highway 401

RECORD OF BOREHOLE No B1

METRIC

GWP	G.W.P 205-00-01	LOCATION	Station 17+882, 20 m Lt of C/L (E 410607.2, N 4872375.2)	ORIGINATED BY	RK
DIST	HWY 401	BOREHOLE TYPE	Hollow Stem Auger, DCPT	COMPILED BY	SK
DATUM	Geodetic	DATE	8/17/2010	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	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT (kN/m³)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
96.2	GROUND SURFACE						
0.0	0.1 m TOPSOIL EMBANKMENT FILL: Sandy Silt to Silty Sand some clayey pockets, tr. gravel brown to grey, loose to compact, moist to wet	1	SS	6			
		2	SS	9			
	wood pieces	3	SS	20			
94.2 2.0	EMBANKMENT FILL: Clayey Silt tr. to some sand, brown greyish brown, v. stiff, moist to wet	4	SS	16			
		5	SS	26	▼	21.5	3 15 58 24
		6	SS	27			
91.8 4.4	SANDY GRAVEL grey, dense to v. dense, wet	7	SS	64			spoon wet 74 23 (3)
		8	SS	32			spoon wet
90.1 6.1	CLAYEY SILT with thin sand layer, tr. gravel grey, v. stiff to stiff, wet	9	SS	19			spoon wet
	some silty sand layer	10	SS	33			0 56 37 7
		11	SS	7			1 4 62 33
85.4 10.8	SILT tr. fine sand, grey dense to compact, wet	12	SS	33			0 4 91 5
		13	SS	13			
		14	SS	16			
81.7 14.5	SAND some silt, grey, dense, wet						
81.2							

Continued Next Page

+3, ×3 Numbers refer to Sensitivity

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

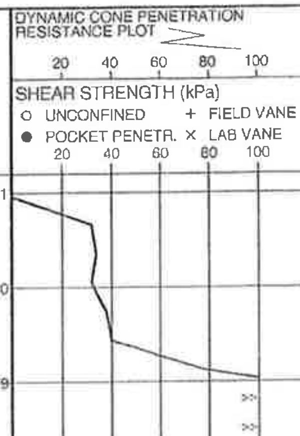
TRANETOB10434AA: Highway 401

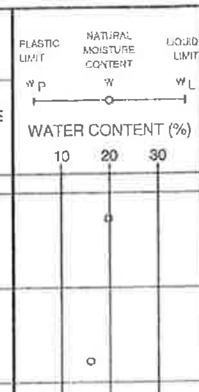
RECORD OF BOREHOLE No B1

METRIC

GWP	G.W.P 205-00-01	LOCATION	Station 17+882, 20 m Lt of C/L (E 410607.2, N 4872375.2)	ORIGINATED BY	RK
DIST		HWY 401	BOREHOLE TYPE Hollow Stem Auger, DCPT	COMPILED BY	SK
DATUM	Geodetic	DATE	6/17/2010	CHECKED BY	ZO

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH (kPa)	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
81.2 15.0	**SAND** some silt, grey, dense, wet		15	SS	33		81				1.2 m sand backed-in the hollow stem
							80				
			16	SS	34						
79.0 17.2	End of Borehole Water Level @ 3.4 m (not stabilized)* before flushing the borehole upon completion Borehole caved in @ 10.7 m upon completion						79				
76.5 17.7	End of DCPT										





+ ³ × ³ Numbers refer to Sensitivity

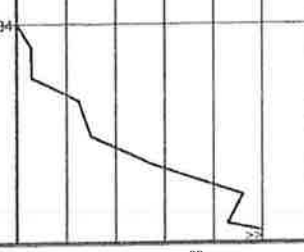
20 15 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No B2

METRIC

GWP G.W.P 205-00-01	LOCATION Station 17+871, 19 m Rt of C/L (E 410614.8, N 4872335.3)		ORIGINATED BY WC
DIST HWY 401	BOREHOLE TYPE Solid Stem Auger, DCPT		COMPILED BY WC
DATUM Geodetic	DATE 9/8/2010 9/9/2010		CHECKED BY ZO

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT (kN/m³)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
96.7	GROUND SURFACE								
0.0	0.2 m TOPSOIL **EMBANKMENT FILL:** Silty Sand tr. rootlet, brown, compact moist to wet	1	SS	16		96			
		2	SS	11					7 48 34 11
95.2 / 1.5	**EMBANKMENT FILL:** Clayey Silt some gravel and sand brown, firm to stiff moist / wet mixed with silty sand	3	SS	7		95			
		4	SS	8		94			
		5	SS	24					
92.9 / 3.8	**EMBANKMENT FILL:** Silty Sand tr. gravel, brown, dense, wet	6	SS	32		93			
92.0 / 4.7	**GRAVELLY SAND** tr. silt and clayey silt pockets grey, dense to v. dense, wet sandy gravel with cobbles	7	SS	44	▽*	92			
		8	SS	103 / 20 cm		91			Spoon wet below 5.3 m
		9	SS	37		90			62 32 (6)
		10	SS	70					
89.1 / 7.6	**CLAYEY SILT** tr. cobble in the upper zone tr. sand, grey, v. stiff to stiff, wet	11	SS	85		89		21.7	High SPT due to cobble 0 3 67 30
		12	SS	20		88, 87			
		13	SS	12		86, 85			
		14	SS	6					SS14 not reliable due to cave-in
84.0 / 12.7	End of Borehole Water level @ 4.6 m (not stabilized)* upon completion					84			
81.8									



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+³, ×³ Numbers refer to Sensitivity

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

Ministry of Transportation Ontario

TRANETOB10434AA: Highway 401

RECORD OF BOREHOLE No B2

METRIC

GWP G.W.P 205-00-01 | LOCATION Station 17+871, 19 m Rt of C/L (E 410614.8, N 4872335.3) | ORIGINATED BY WC

DIST ____ HWY 401 | BOREHOLE TYPE Solid Stem Auger, DCPT | COMPILED BY WC

DATUM Geodetic | DATE 9/8/2010 9/9/2010 | 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) 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
81.7 14.9	End of DCPT										

+ 3, × 3: Numbers refer to Sensitivity

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

Ministry of Transportation Ontario

TRANETOB10434AA: Highway 401

RECORD OF BOREHOLE No B3

METRIC

GWP G.W.P 205-00-01 | LOCATION Station 17+903, 20 m Lt of C/L (E 410626.5, N 4872384.4) | ORIGINATED BY RK

DIST ____ HWY 401 | BOREHOLE TYPE Hollow Stem Auger, NQ Coring | COMPILED BY SK

DATUM Geodetic | DATE 6/18/2010 | CHECKED BY ZO

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
96.3	GROUND SURFACE					
0.0	0.2 m TOPSOIL					
	EMBANKMENT FILL: Sandy Silt to Silty Sand, tr. gravel and clay, with rootlets in upper 1m, brown to grey, loose to compact, moist	1	SS	8		
		2	SS	10		
	wet	3	SS	20		
	clayey silt layer	4	SS	10		0 21 52 27
	tr. organics	5	SS	22		
92.5 / 3.8	**EMBANKMENT FILL:** Clayey Silt, some sand and tr. gravel, grey, stiff, wet	6	SS	15		spoon wet; 2 17 54 27
91.9 / 4.4	GRAVEL, some sand, some cobbles, grey, dense, wet	7	SS	41	▼*	auger grinding
		8	SS	44		auger grinding
90.4 / 5.9	CLAYEY SILT, tr. sand, grey, v. stiff to stiff with firm layers	9	SS	22		
		10	SS	10		
		11	SS	7		0 3 62 35
		12	TW	PM		no recovery in shelby tube
		13	TW	PM		no recovery in shelby tube
		14	SS	9		
	some sand	15	SS	18		
81.3		16	TW	PM		

Elevation scale: 96, 95, 94, 93, 92, 91, 90, 89, 88, 87, 86, 85, 84, 83, 82

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

Field vane values: 3.3 (approx. elev. 86.3); 3.0 (approx. elev. 83.2)

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 above, 10 below) (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

Foundation Design

TRANETOB10434AA; Highway 401

RECORD OF BOREHOLE No B3

METRIC

GWP G.W.P 205-00-01 LOCATION Station 17+903, 20 m Lt of C/L (E 410626.5, N 4872384.4) ORIGINATED BY RK

DIST HWY 401 BOREHOLE TYPE Hollow Stem Auger, NQ Coring COMPILED BY SK

DATUM Geodetic DATE 6/18/2010 CHECKED BY ZO

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
81.3 15.0	**CLAYEY SILT** tr. sand, grey, v. stiff to stiff with firm layers				81	
81.1 15.2	**SAND TO SILTY SAND** some gravel and cobbles grey, wet compact / v. dense	17	SS	24	80	
		18	SS	85	79	1.2 m sand backed-in the hollow stem
78.8 17.5	**SILTY SAND TILL** grey, v. dense, wet	19	SS	100/7.8 cm	78	18 55 20 7
		20	SS	100/7.8 cm	77 / 76	NQ coring begins @ 19.9 m.
75.7 20.6	**BEDROCK** grey limestone	21	RC	RQD=48% TCR=98%	75	
		22	RC	RQD=62% TCR=93%	74	
		23	RC	RQD=69% TCR=100%	73	
72.6 23.7	End of Borehole Water Level @ 5.0 m (not stabilized)* before rock coring Borehole caved in 5.6 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
GROUND WATER CONDITIONS

+3, ×3: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

TRANETOB10434AA: Highway 401

RECORD OF BOREHOLE No B4

METRIC

GWP G.W.P 205-00-01	LOCATION Station 17+882, 17 m Rt of C/L (E 410623.5, N 4872341.4)		ORIGINATED BY WC
DIST	HWY 401	BOREHOLE TYPE Solid Stem Auger, DCPT	COMPILED BY WC
DATUM Geodetic	DATE 9/8/2010		CHECKED BY ZO

Column headings: 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)

Soil profile

ELEV / DEPTH	DESCRIPTION
96.6	GROUND SURFACE
0.0	**0.1 m TOPSOIL**
	EMBANKMENT FILL: Silty Sand tr. asphalt and rootlets in upper 0.5 m with gravel, some clay pockets loose to v. dense; clayey silt; moist; wet; with organics
92.0 / 4.6	**GRAVELLY SAND** some silt, grey v. dense, wet
88.6 / 8.0	**CLAYEY SILT** tr. sand and gravel grey, v. stiff to stiff, wet
83.9 / 12.7	End of Borehole. Water level @ 3.1 m (not stabilized)* upon completion
81.8 / 14.8	

Samples

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%)
1	AS	28	
2	SS	5	
3	SS	61	
4	SS	30	
5	SS	21	
6	SS	13	20 46 25 9
7	SS	52	spoon wet below 4.6 m; Auger grinding @ 4.9 m
8	SS	115 / 18 cm	
9	SS	109 / 20 cm	
10	SS	100	1.8 m cave-in @ 6.9 m
11	SS	41	
12	SS	5	SPT and sample not representative due to cave-in
13	SS	15	1 2 73 24
14	SS	21	

Elevation scale: 96, 95, 94, 93, 92, 91, 90, 89, 88, 87, 86, 85, 84

Continued Next Page

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

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

Ministry of Transportation
Ontario

TRANETOB10434AA: Highway 401

RECORD OF BOREHOLE No B4

METRIC

GWP: G.W.P 205-00-01	LOCATION: Station 17+882, 17 m Rt of C/L (E 410623.5, N 4872341.4)	ORIGINATED BY: WC
DIST:	HWY: 401	BOREHOLE TYPE: Solid Stem Auger, DCPT	COMPILED BY: WC
DATUM: Geodetic		DATE: 9/8/2010	CHECKED BY: ZO

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH (kPa): O UNCONFINED, + FIELD VANE, ● POCKET PENETR., X LAB VANE (20 40 60 80 100)	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
81.6	End of DCPT										

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

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

RECORD OF BOREHOLE No B5

1 OF 2 **METRIC**

GWP G.W.P 205-00-01 LOCATION Station 17+910, 26 m Lt of C/L (E 410631.0, N 4872392.4) ORIGINATED BY RK

DIST ____ HWY 401 BOREHOLE TYPE Hollow Stem Auger COMPILED BY RK

DATUM Geodetic DATE 6/23/2010 6/24/2010 CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
94.7	GROUND SURFACE					
0.0	0.2 m TOPSOIL FILL: Sandy Silt to Silty Sand tr. to some gravel and clay brown to grey, loose to compact	1	SS	9		
		2	SS	28		
	moist / wet	3	SS	10		
	some clay some rootlets, black					
92.3 2.4	SANDY GRAVEL TO SAND tr. rootlets in upper 0.5 m grey, compact to dense, wet	4	SS	25		
		5	SS	48		64 32 (4) auger grinding
					▼*	
		6	SS	24		
90.3 4.4	CLAYEY SILT tr. sand, grey v. stiff to stiff, wet	7	SS	18		
		8	SS	21		
		9	SS	12		
	clay layer	10	TW	PM		
		11	SS	10		0 4 69 27
		12	TW	PM		
		13	SS	20		Unit weight 21.2 kN/m³; 0 10 72 18
81.9 12.8	SAND tr. silt, grey, dense, wet	14	SS	34		sand back up 1.0 m inside the hollow stem
79.7						

Field vane / shear strength annotations: 67 (approx. elev. 88), 29 (approx. elev. 85).

Continued Next Page

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

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

Ministry of Transportation
Ontario

Foundation Design

TRANETOB10434AA: Highway 401

RECORD OF BOREHOLE No B5

METRIC

GWP	G.W.P 205-00-01	LOCATION	Station 17+910, 26 m Lt of C/L (E 410631.0, N 4872392.4)	ORIGINATED BY	RK
DIST	HWY 401	BOREHOLE TYPE	Hollow Stem Auger	COMPILED BY	RK
DATUM	Geodetic	DATE	6/23/2010 6/24/2010	CHECKED BY	ZO

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● 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
79.7 15.0	**SANDY GRAVEL** some silt and clay, grey dense to v. dense, wet		15	SS	70		79		○ ○		49 31 (20) auger grinding
78.2 16.5			16	SS	50 / 0 cm						no sample retrieved auger refusal on possible boulder
	End of Borehole Auger refusal on possible boulder Water Level @ 3.8 m (not stabilized)* upon completion Borehole caved-in @ 7.0 m upon completion										

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

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

Ministry of Transportation Ontario

TRANETOB10434AA: Highway 401

RECORD OF BOREHOLE No B6

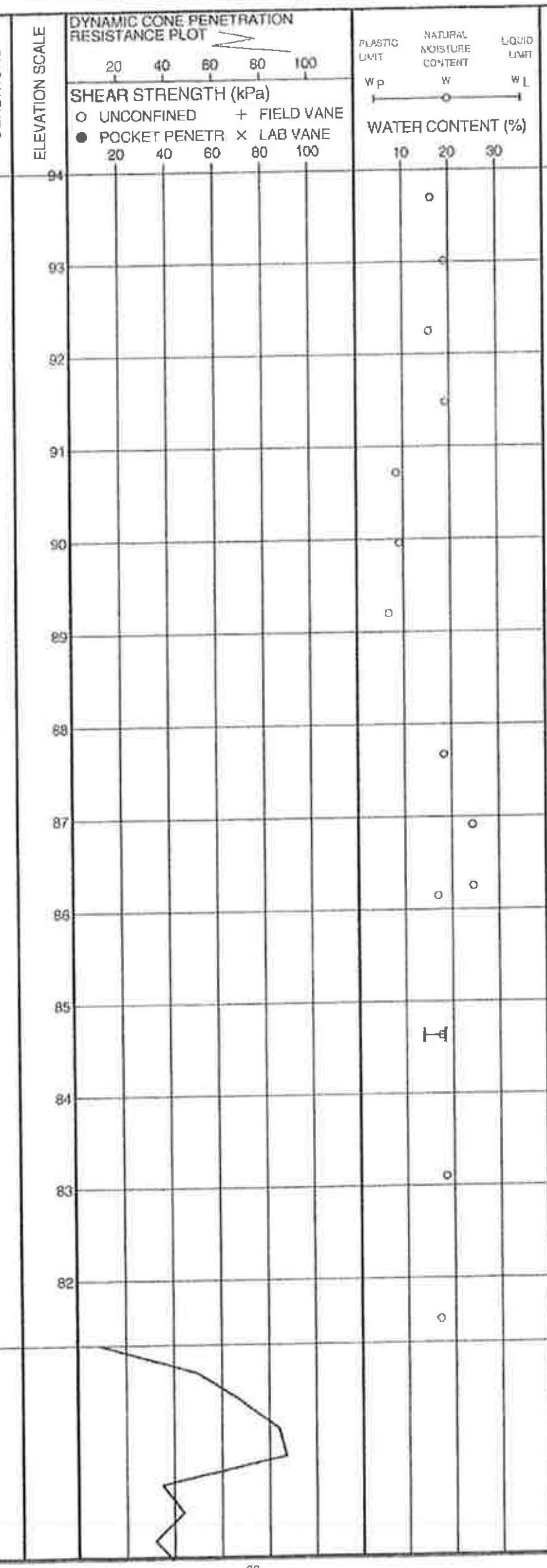
METRIC

GWP G.W.P 205-00-01 LOCATION Station 17+899, 18 m Rt of C/L (E 410639.1, N 4872348.1) ORIGINATED BY WC

DIST HWY 401 BOREHOLE TYPE Hollow and Solid Stem Augers, DCPT COMPILED BY WC

DATUM Geodetic DATE 9/7/2010 CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
94.0	GROUND SURFACE				
0.0	FILL: Topsoil mixed with Silt and Sand some rootlets, grey and dark brown v. loose to loose, moist	1	SS	3	
		2	SS	2	
	organics, tr. gravel and clay	3	SS	7	Spoon wet @ 2.3 m
91.7 2.3	FILL: Sand to Sandy Gravel some cobbles, grey v. dense to compact, wet	4	SS	52	
		5	SS	23	70 23 (7) Cobbles stuck in auger, switched to s.s. augers @ 3.1 m
	with organic pockets	6	SS	30	
		7	SS	26	Spoon wet below 4.6 m
88.7 5.3	SILTY SAND tr. gravel, grey, dense, wet				Significant soil back-up @ 5.3 m Borehole relocated to 1 m from original location. Straight augering to 6.1 m
		8	SS	35	
87.3 6.7	CLAYEY SILT with sand lenses, tr. gravel grey, v. stiff to stiff, wet	9	SS	14	7 65 23 5
		10	SS	2 *	* SPT not reliable due to cave-in
		11	SS	10	1 15 61 23
		12	SS	10	
		13	SS	35	High SPT due to gravel
81.3 12.7	End of Borehole Water level @ 3.7 m (not stabilized)* upon completion				



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+3 ×3 Numbers refer to Sensitivity   (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

Foundation Design

TRANETOB10434AA: Highway 401

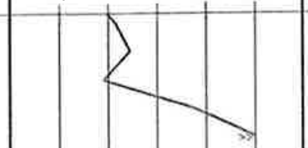
RECORD OF BOREHOLE No B6

METRIC

GWP G.W.P 205-00-01 | LOCATION Station 17+899, 18 m Rt of C/L (E 410639.1, N 4872348.1) | ORIGINATED BY WC

DIST ____ HWY 401 | BOREHOLE TYPE Hollow and Solid Stem Augers, DCPT | COMPILED BY WC

DATUM Geodetic | DATE 9/7/2010 | 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						
79.0											
77.7 16.3	End of DCPT										

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

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

Ministry of Transportation Ontario

Foundation Design

TRANETOB10434AA: Highway 401

RECORD OF BOREHOLE No B7

METRIC

GWP G.W.P 205-00-01 | LOCATION Station 17+931, 19 m Lt of C/L (E 410651.8, N 4872395.6) | ORIGINATED BY RK

DIST ____ HWY 401 | BOREHOLE TYPE Hollow Stem Auger | COMPILED BY SK

DATUM Geodetic | DATE 6/16/2010 6/17/2010 | CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
93.6	GROUND SURFACE					
0.0	0.1 m TOPSOIL FILL: Silt some fine sand, some clay, with rootlets brown, loose, moist	1	SS	5	93	
93.0 0.6	SAND AND GRAVEL grey, compact to v. dense, wet	2	SS	23		spoon wet
		3	SS	28	92	auger grinding spoon wet 49 48 (3) auger grinding
		4	SS	53	91	
		5	SS	48	90	
89.9 3.7	CLAYEY SILT with silt lenses, tr. sand grey, wet v. stiff / stiff	6	SS	19		0 1 70 29
		7	SS	18	89	
		8	SS	8	88	
		9	SS	7	87	
		10	SS	5	86	
		11	TW	PM	85, 84	Unit weight 20.8 kN/m³; consolidation test
82.7 10.9	SILT some sand, tr. clay grey, compact to v. dense, wet with cobbles	12	SS	20	83, 82	
		13	SS	51	81	auger grinding
80.6 13.0	SAND AND GRAVEL grey, v. dense, wet	14	SS	100/7.10	80, 79	auger grinding
78.6						

Field vane shear strength readings: 4.0 (approx. elev. 86.7), 2.0 (approx. elev. 85.0), 4.0 (approx. elev. 83.5)

Continued Next Page

+3, ×3 Numbers refer to Sensitivity

15 ⊕ 5 (20/10) (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

Foundation Design

TRANETOB10434AA: Highway 401

RECORD OF BOREHOLE No B7

2 OF 2

METRIC

GWP G.W.P 205-00-01	LOCATION Station 17+931, 19 m Lt of C/L (E 410651.8, N 4872395.6)		ORIGINATED BY RK
DIST HWY 401	BOREHOLE TYPE Hollow Stem Auger		COMPILED BY SK
DATUM Geodetic	DATE 6/16/2010	6/17/2010	CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH (kPa)	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
78.6 15.0	**SAND AND GRAVEL** grey, v. dense, wet		15	SS	100 / 10 cm		78		o	23.8	50 43 (7)
77.6 16.0	**SILTY SAND TILL** grey, v. dense, wet		16	SS	100 / 8 cm		77		o		0.9 m sand backed in the hollow stem auger grinding
76.2 17.4	End of Borehole Auger refusal on possible bedrock Piezometer installed @ 17.4 m upon completion Date / Measured Water Level August 19, 2010 / 1.0 m October 15, 2010 / 0.9 m		17	SS	50 / 0 cm						auger refusal and spoon bouncing

+ ³ × ³ Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

Foundation Design

TRANETOB10434AA. Highway 401

RECORD OF BOREHOLE No B8

METRIC

GWP G.W.P 205-00-01 LOCATION Station 17+920, 18 m Rt of C/L (E 410658.6, N 4872357.5) ORIGINATED BY RK

DIST HWY 401 BOREHOLE TYPE Hollow Stem Auger COMPILED BY WC

DATUM Geodetic DATE 7/15/2010 CHECKED BY ZO

ELEV. DEPTH	SOIL PROFILE DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
93.9	GROUND SURFACE				
0.0	0.2 m TOPSOIL; moist	1	SS	4	
	FILL: Silty Sand to Sandy Silt, tr. to some clay, tr. gravel, tr. rootlets, v. loose; wet	2	SS	4	spoon wet below 0.8 m
	with organics	3	SS	4	auger grinding @ 1.8 m
91.8 / 2.1	SAND AND GRAVEL, grey, compact, wet	4	SS	24	50 42 (8)
90.8 / 3.1	CLAYEY SILT, tr. sand, grey, stiff to firm, wet	5	SS	22	0 16 64 20
		6	SS	16	
	v. stiff	7	SS	10	
		8	SS	9	0 4 66 30
		9	SS	9	
		10	SS	8	
		11	TW	PM	
		12	SS	10	
81.7 / 12.2	SAND TO SILTY SAND, tr. clay and gravel, dense, wet	13	SS	31	
		14	SS	39	
79.7 / 14.2	SAND TILL, v. dense, wet				auger grinding @ 14.3 due to poss. boulder
78.9					

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (93, 92, 91, 90, 89, 88, 87, 86, 85, 84, 83, 82, 81, 80, 79); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH (kPa) O UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³.

Field vane values: 20, 2.5 (at approx. 87); 2.1 (at approx. 85.5).

Continued Next Page

+³ ×³ Numbers refer to Sensitivity   20 15 6 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No B8

METRIC

GWP G.W.P 205-00-01	LOCATION Station 17+920, 18 m Rt of C/L (E 410658.6, N 4872357.5)	ORIGINATED BY RK
DIST ___ HWY 401	BOREHOLE TYPE Hollow Stem Auger	COMPILED BY WC
DATUM Geodetic	DATE 7/15/2010	CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
78.9 / 15.0	**SAND TILL** v. dense, wet	15	SS	100 / 15 cm		o	2 89 (9) auger grinding from 15.5 to 17.4 m; spoon bounces @ 16.8 m
					78		
77.1 / 16.8	**SANDY GRAVEL** grey, v. dense, wet	16	SS	70 / 8 cm	77	o	
					76		
		17	SS	70 / 8 cm		o	spoon bounces @ 18.3 m
					75		
74.8 / 19.1	End of Borehole Water level @ 1.2 m (not stabilized)* upon completion Borehole caved in @ 7.9 m upon completion Borehole moved three times due to presence of boulders between 1.8 and 2.1 m	18	SS	50 / 0			auger refusal @ 19.1 m

Column headings: SOIL PROFILE; STRAT PLOT; 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. X LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³

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

Ministry of Transportation Ontario

Foundation Design

TRANETOB10434AA: Highway 401

RECORD OF BOREHOLE No B9

METRIC

GWP G.W.P 205-00-01 LOCATION Station 17+847, 19 m Lt of C/L (E 410666.3, N 4872402.5) ORIGINATED BY RK

DIST HWY 401 BOREHOLE TYPE Hollow Stem Auger, NQ Coring COMPILED BY RK

DATUM Geodetic DATE 6/15/2010 CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
96.7	GROUND SURFACE					
0.0	0.2 m **TOPSOIL**	1	SS	7		
	EMBANKMENT FILL: Clayey Silt some sand, brown, loose, moist					
95.9 / 0.8	**EMBANKMENT FILL:** Sandy Silt some gravel and clay brown, compact, moist	2	SS	20		
		3	SS	16		
94.6 / 2.1	**EMBANKMENT FILL:** Clayey Silt some sand, tr. gravel, some rootlets grey, v. stiff to firm, moist to wet	4	SS	19		
		5	SS	7		
		6	SS	4		
92.3 / 4.4	**SANDY GRAVEL** some cobbles grey, dense to compact, wet	7	SS	32		spoon wet
90.4 / 6.3	**CLAYEY SILT** grey, firm to stiff, wet	8	SS	23	▼	
		9	SS	12		1 3 70 26
		10	SS	7		
		11	SS	17		
		12	SS	6		
		13	TW	PM		no sample from shelby tube is retrieved at 13.7 m depth
81.7						

Elevation scale: 96, 95, 94, 93, 92, 91, 90, 89, 88, 87, 86, 85, 84, 83, 82

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

Field vane: 2.5 (at approx. 85.4); 2.5 (at approx. 83.8)

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

UNIT WEIGHT γ kN/m³

Continued Next Page

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

RECORD OF BOREHOLE No B9

METRIC

GWP G.W.P 205-00-01 | LOCATION Station 17+947, 19 m Lt of C/L (E 410666.3, N 4872402.5) | ORIGINATED BY RK

DIST ____ HWY 401 | BOREHOLE TYPE Hollow Stem Auger, NQ Coring | COMPILED BY RK

DATUM Geodetic | DATE 6/15/2010 | CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
81.7 / 15.0	**CLAYEY SILT** grey, firm to stiff, wet	14	SS	61	
81.2 / 15.5	**SANDY GRAVEL TO GRAVEL** grey, v. dense, wet	15	SS	100 / 29 cm	sand backed up 1.2 m inside hollow stem; 55 37 (8)
		16	SS	80 / 10 cm	auger grinding at 17.0, 17.3 and 17.6 m
					auger grinding
76.9 / 19.8	**SILTY SAND TILL** grey, v. dense, wet	17	SS	70/10 cm	
		18	SS	100 / 10 cm	
74.6 / 22.1	**BEDROCK** grey limestone	19	RC	RQD=28% TCR=94%	
		20	RC	RQD=55% TCR=94%	
71.6 / 25.1	End of Borehole Water Level @ 6.7 m (not stabilized)* upon completion Borehole caved in @ 13.7 m upon completion				

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

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

TRANETOB10434AA: Highway 401

RECORD OF BOREHOLE No B10

METRIC

GWP G.W.P 205-00-01 | LOCATION Station 17+930, 19 m Rt of C/L (E 410688.3, N 4872360.6) | ORIGINATED BY RK

DIST ___ HWY 401 | BOREHOLE TYPE Hollow Stem Auger, NQ Coring | COMPILED BY WC

DATUM Geodetic | DATE 7/14/2010 7/15/2010 | CHECKED BY ZO

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
96.3	GROUND SURFACE				
0.0	0.2 m TOPSOIL	1	SS	11	
	EMBANKMENT FILL: Silty Sand tr. to some gravel, tr. rootlets compact, grey, moist	2	SS	24	auger grinding @ 1.2 m
		3	SS	25	
94.0 / 2.3	EMBANKMENT FILL: Clayey Silt tr. sand, tr. gravel, tr. rootlets brown to grey, stiff to firm	4	SS	11	
	moist / wet	5	SS	16	4 8 60 28
		6	SS	6	
	grey to dark grey, tr. organics; silty sand, dark grey with organics and wood pieces	7	SS	19	
91.3 / 5.0	CLAYEY SILT with gravel, grey v. stiff to stiff, wet	8	SS	21	auger grinding @ 5.3 and 5.8 m spoon wet below 5.3 m
		9	SS	9	
88.7 / 7.6	CLAYEY SILT with fine grained sand interbeds grey, v. stiff to stiff, grey, wet	10	SS	22	
		11	SS	18	
		12	SS	22	
		13	SS	10	0 4 67 29
		14	TW	PM	
81.8 / 14.5	SAND fine grained, tr. to some silt, tr. gravel				

Chart headings: GROUND WATER CONDITIONS; ELEVATION SCALE (96 to 82); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH (kPa) O UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³

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

Ministry of Transportation Ontario

Foundation Design

TRANETOB10434AA: Highway 401

RECORD OF BOREHOLE No B10

METRIC

GWP G.W.P 205-00-01 LOCATION Station 17+930, 19 m Rt of C/L (E 410668.3, N 4872360.6) ORIGINATED BY RK

DIST HWY 401 BOREHOLE TYPE Hollow Stem Auger, NQ Coring COMPILED BY WC

DATUM Geodetic DATE 7/14/2010 7/15/2010 CHECKED BY ZO

ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
81.3					81	
	SAND fine grained, tr. to some silt, tr. gravel grey, compact to v. dense, wet	15	SS	24	80	auger grinding @ 16.2 m
79.6 16.7	SAND TILL grey, v. dense, wet	16	SS	75 / 14 cm	79	auger grinding @ 16.8 to 18.0 m; 27 54 16 3
78.1 18.2	GRAVEL TO SANDY GRAVEL grey, v. dense, wet	17	SS	100 / 10 cm	78	spoon bounces @ 18.3 m; quick gel used to prevent soil back-up. 0.8 m of soil back-up @ 18.3 m.
		18	SS	100 / 8 cm	77, 76	auger grinding @ 18.6 to 19.8 m; start of wash boring @ 20.1 m
		19	SS	100 / 5 cm	75	
74.2 22.1	BEDROCK grey limestone	20	RC	RQD=30 TCR=88	74, 73	Start of NQ coring @ 22.1 m
		21	RC	RQD=80 TCR=100	72	
71.2 25.1	End of Borehole Piezometer installed @ 18.3 m upon completion. Date / Measured water level July 15, 2010 / 3.8 m (on completion) August 19, 2010 / 3.7 m October 15, 2010 / 3.4 m					

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

UNIT WEIGHT γ kN/m³

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

Ministry of Transportation Ontario

TRANETOB10434AA: Highway 401

RECORD OF BOREHOLE No B11

METRIC

GWP G.W.P 205-00-01 LOCATION Station 17+967, 19 m Lt of C/L (E 410684.7, N 4872410.9) ORIGINATED BY RK

DIST HWY 401 BOREHOLE TYPE Hollow Stem Auger COMPILED BY RK

DATUM Geodetic DATE 6/14/2010 CHECKED BY ZO

ELEV DEPTH	SOIL PROFILE DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
96.9	GROUND SURFACE					
0.0	0.2 m TOPSOIL EMBANKMENT FILL: Silty Sand interbedded with clayey silt tr. rootlets in upper layer some decayed wood, brown to grey loose to compact, moist	1	SS	7		
		2	SS	16		
		3	SS	21		
94.8 2.1	EMBANKMENT FILL: Clayey Silt to Silty Clay tr. to some sand brown to grey, stiff to v. stiff with firm layers moist to wet	4	SS	15		
		5	SS	21		
		6	SS	6		
92.2 4.7	SAND some gravel, tr. to some clay grey, dense, wet	7	SS	40		
90.8 6.1	CLAYEY SILT tr. sand grey, stiff to firm, wet v. stiff	8	SS	25	▼	spoon wet 0 10 67 23
		9	SS	12		
		10	SS	9		0 4 64 32
		11	TW	PM		
		12	SS	8		
		13	SS	8		
81.9						

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

ELEVATION SCALE: 96 95 94 93 92 91 90 89 88 87 86 85 84 83 82

Shear strength annotations: 2.5 (El. ~87), 2.4 (El. ~84), 2.5 (El. ~82.5)

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

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

TRANETOB10434AA: Highway 401

RECORD OF BOREHOLE No B11

METRIC

GWP G.W.P 205-00-01 LOCATION Station 17+967, 19 m Lt of C/L (E 410684.7, N 4872410.9) ORIGINATED BY RK

DIST HWY 401 BOREHOLE TYPE Hollow Stem Auger COMPILED BY RK

DATUM Geodetic DATE 6/14/2010 CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)
81.9 / 15.0	**CLAYEY SILT** tr. sand grey, stiff to firm, wet	14	TW	PM	
80.1 / 16.8	**SANDY GRAVEL** grey, v. dense, wet	15	SS	100 / 29 cm	o
78.9 / 18.0	**SANDY SILT TO SILTY SAND** some gravel, tr. clay grey, v. dense, wet	16	SS	100 / 13 cm	o
78.5 / 18.4	End of Borehole Water Level @ 6.4 m (not stabilized)* upon completion Borehole caved in @ 8.8 m upon completion				

Column headings: SOIL PROFILE; STRAT PLOT; 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. X LAB VANE (20 40 60 80 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Elevation scale: 81, 80, 79

+ ³, × ³ Numbers refer to Sensitivity

15 20 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

Foundation Design

TRANETOB10434AA: Highway 401

RECORD OF BOREHOLE No B12

 METRIC

GWP: G.W.P 205-00-01 | LOCATION: Station 17+950, 20 m Rt of C/L (E 410686.4, N 4872368.7) | ORIGINATED BY: RK

DIST: ___ HWY: 401 | BOREHOLE TYPE: Hollow Stem Auger | COMPILED BY: WC

DATUM: Geodetic | DATE: 7/13/2010 | CHECKED BY: ZO

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
97.2	GROUND SURFACE				
0.0	0.1 m TOPSOIL	1	SS	17	
	EMBANKMENT FILL: Sand to Silty Sand, tr. to some gravel, tr. rootlets, brown to reddish brown, compact to loose, moist / wet / some organics, dark grey	2	SS	8	auger grinding @ 0.6 m
		3	SS	5	auger grinding @ 2.0 m
		4	SS	10	
94.2 / 3.0	**EMBANKMENT FILL:** Clayey Silt, tr. to some sand, tr. gravel and rootlets, grey, stiff to firm, wet	5	SS	9	
		6	SS	5	
		7	SS	6	start of hollow stem auger @ 4.6 m
91.7 / 5.5	**SILTY SAND** tr. to some gravel, tr. rootlets, some organics, dark grey, loose, wet	8	SS	8	8 53 26 13
91.0 / 6.2	**CLAYEY SILT** tr. sand, grey, stiff to firm, wet	9	SS	10	
		10	SS	10	auger grinding @ 7.3 m; 1 3 68 28
		11	SS	7	
		12	SS	8	
		13	TW	PM	Unit weight 19.7 kN/m³; consolidation test
		14	SS	8	
		15	SS	11	1.5 m of soil backup @ 13.7 m
82.2					

Shear strength (field vane) values plotted: 24, 21, 24.

Continued Next Page

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

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

RECORD OF BOREHOLE No B12

METRIC

GWP: G.W.P 205-00-01	LOCATION: Station 17+950, 20 m Rt of C/L (E 410686.4, N 4872368.7)	ORIGINATED BY: RK	
DIST:	HWY: 401	BOREHOLE TYPE: Hollow Stem Auger	COMPILED BY: WC
DATUM: Geodetic	DATE: 7/13/2010	CHECKED BY: ZO	

ELEV. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
82.2 / 15.0	CLAYEY SILT, tr. sand, grey, stiff to firm, wet; some sand	16	SS	10	82	
					81	
80.4 / 16.8	SILTY SAND, fine grained, tr. clay, grey, dense, wet	17	SS	33	80	auger grinding @ 17.4 m
					79	
78.9 / 18.3	**SILTY SAND TILL**, some gravel, tr. silt, v. dense	18	SS	53		11 47 (42)
					78	auger grinding @ 19.2 m
	gravel, tr. coarse grained sand	19	SS	100 / 23 cm		
77.2 / 20.0	End of Borehole Water level @ 3.5 m (not stabilized)* upon completion Borehole caved in @ 6.4 m upon completion					

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH (kPa): O UNCONFINED, + FIELD VANE, ● POCKET PENETR., × LAB VANE: 20 40 60 80 100

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

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m^3

+ ³, × ³ Numbers refer to Sensitivity

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

Ministry of Transportation Ontario

Foundation Design

TRANETOB10434AA: Highway 401

RECORD OF BOREHOLE No B13

1 OF 1

METRIC

GWP G.W.P 205-00-01 LOCATION Station 17+891, 12 m Lt of C/L (E 410619.1, N 4872371.7) ORIGINATED BY RK

DIST HWY 401 BOREHOLE TYPE Hollow Stem Auger COMPILED BY WC

DATUM Geodetic DATE 8/12/2010 CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
97.2	GROUND SURFACE					
0.0	250 mm **ASPHALT** 0.1 m **GRANULAR FILL** **EMBANKMENT FILL:** Sand some gravel, brown, compact, moist	1	SS	14		
96.6 0.6						
96.3 0.9	**EMBANKMENT FILL:** Silty Clay to Clayey Silt some sand, tr. gravel brown, hard, moist	2	SS	43		Auger grinding occasionally below 0.8 m
	EMBANKMENT FILL: Sand and Gravel some cobbles, brown compact with loose zones, moist	3	SS	15		
		4	SS	25		50 41 (9)
		5	SS	8		
93.2 4.0	**EMBANKMENT FILL:** Clayey Silt tr. sand, tr. gravel, tr. rootlets brown, v. stiff, moist	6	SS	20		
	silty sand	7	SS	36		
92.0 5.2	**SAND AND GRAVEL** with cobbles, grey, v. dense, wet	8	SS	57	▼*	spoon wet below 5.3 m 57 36 (7)
		9	SS	102		
89.9 7.3	**CLAYEY SILT** tr. sand, grey, v. stiff, wet	10	SS	20		
89.1 8.1	End of Borehole Water level @ 5.3 m (not stabilized)* upon completion Borehole caved-in @ 5.8 m upon completion					

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (97, 96, 95, 94, 93, 92, 91, 90); 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

Ministry of Transportation Ontario

Foundation Design

TRANETOB10434AA: Highway 401

RECORD OF BOREHOLE No B14

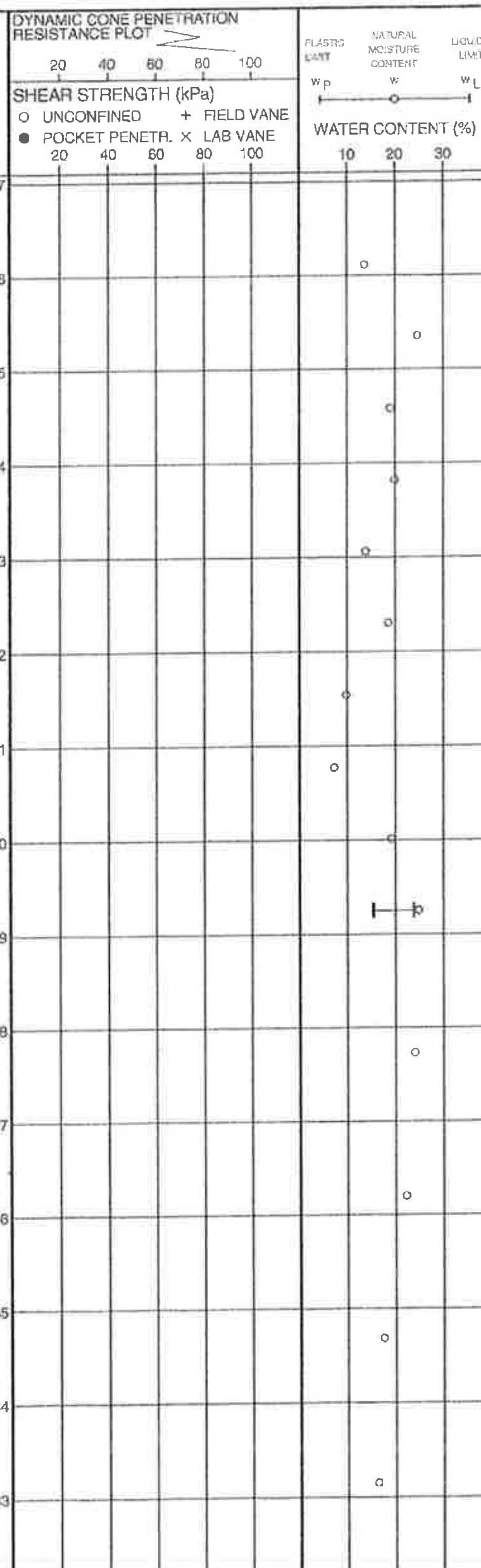
METRIC

GWP: G.W.P 205-00-01 | LOCATION: Station 17+875, 11 m Rt of C/L (E 410615.0, N 4872343.8) | ORIGINATED BY: GJ

DIST: | HWY: 401 | BOREHOLE TYPE: Solid Stem Auger, NQ Coring | COMPILED BY: WC

DATUM: Geodetic | DATE: 9/7/2010 9/8/2010 | CHECKED BY: ZO

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
97.1	GROUND SURFACE				
0.0	250 mm ASPHALT 0.4 m GRANULAR FILL EMBANKMENT FILL: Silty Sand to Sandy Silt some clay, tr. gravel brown to grey, compact, moist	1	AS		
		2	SS	12	
95.6 / 1.5	EMBANKMENT FILL: Clayey Silt some sand, some organics tr. wood pieces and rootlets brown and grey. v. stiff/compact, moist	3	SS	14	
		4	SS	11	
		5	SS	20	
		6	SS	27	
	silty sand, tr. rootlets black, loose, wet	7	SS	4	
91.9 / 5.2	SAND AND GRAVEL some silt, grey, compact, wet	8	SS	20	
		9	SS	30	
90.3 / 6.8	CLAYEY SILT with sandy seams and layers grey, v. stiff to stiff, wet hard	10	SS	38	0 10 72 18
		11	SS	13	0 3 70 27
		12	SS	6	
		13	SS	17	
84.9 / 12.2	SANDY SILT TO SILT tr. to some clay grey, compact to loose, wet	14	SS	22	
		15	SS	6	
82.1					



GROUND WATER CONDITIONS; ELEVATION SCALE: 97 96 95 94 93 92 91 90 89 88 87 86 85 84 83; UNIT WEIGHT γ kN/m^3

Continued Next Page

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

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

Ministry of Transportation
Ontario

Foundation Design

TRANETOB10434AA: Highway 401

RECORD OF BOREHOLE No B14

METRIC

GWP G.W.P 205-00-01 | LOCATION Station 17+875, 11 m Rt of C/L (E 410615.0, N 4872343.8) | ORIGINATED BY GJ

DIST ____ HWY 401 | BOREHOLE TYPE Solid Stem Auger, NQ Coring | COMPILED BY WC

DATUM Geodetic | DATE 9/7/2010 9/8/2010 | CHECKED BY ZO

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%)
82.1 / 15.0	**SANDY SILT TO SILT** tr. to some clay, grey, compact to loose, wet				82		
81.9 / 15.2	**SILTY SAND** tr gravel, grey, compact to v. dense, wet	16	SS	17		o	
					81		
		17	SS	53	80	o	
					79		
78.8 / 18.3	**GRAVEL** some sand, grey, wet; compact to dense / v. dense	18	SS	24		o	
					78		auger grinding from 19.2 to 19.8 m
		19	SS	100 / 3 cm	77	o	
		20	RC	RQD=0% TCR=7%	76		
75.2 / 21.9	**BEDROCK** grey limestone	21	RC	RQD=44% TCR=80%	75		
					74		
		22	RC	RQD=72% TCR=92%	73		
		23	RC	RQD=50% TCR=100%	72		
71.6 / 25.5	End of Borehole. Water level @ 4.0 m (not stabilized)* upon completion. Borehole caved-in @ 5.5 m upon completion						

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

+³ ×³ Numbers refer to Sensitivity

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

Ministry of Transportation
Ontario

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TRANETOB10434AA: Highway 401

RECORD OF BOREHOLE No B15

1 OF 1

METRIC

GWP G.W.P 205-00-01 LOCATION Station 17+954, 12 m Lt of C/L (E 410676.5, N 4872399.2) ORIGINATED BY RK

DIST HWY 401 BOREHOLE TYPE Hollow Stem Auger COMPILED BY WC

DATUM Geodetic DATE 8/12/2010 CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
97.3	GROUND SURFACE					
0.0	300 mm ASPHALT 0.2 m GRANULAR FILL EMBANKMENT FILL: Sand some gravel, brown, compact, moist	1	SS	23		Auger grinding @ 0.4 m
		2	SS	28		
96.3 1.0	EMBANKMENT FILL: Clayey Silt tr. gravel, tr. rootlets, tr. wood pieces brown to grey, compact, moist	3	SS	10		
		4	SS	10		
94.3 3.0	EMBANKMENT FILL: Clayey Silt with sand lenses brown to grey, v. stiff to stiff moist wet	5	SS	19		0 17 55 28
		6	SS	8		
92.6 4.7	EMBANKMENT FILL: Silty Sand with gravel, cobbles and boulders some clay, corn organics, tr. rootlets dark grey, compact, wet	7	SS	26	▼*	spoon wet below 4.6 m
92.1 5.2	SANDY GRAVEL grey, compact, wet					
91.8 5.5	CLAYEY SILT with fine grained sand lenses grey, v. stiff to stiff, wet	8	SS	16		
		9	SS	16		0 4 69 27
		10	SS	9		
89.2 8.1	End of Borehole Water level @ 4.7 m (not stabilized)* upon completion Borehole caved in @ 5.2 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

ELEVATION SCALE 97 96 95 94 93 92 91 90

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

Ministry of Transportation Ontario

TRANETOB10434AA: Highway 401

RECORD OF BOREHOLE No B16

METRIC

GWP G.W.P 205-00-01 LOCATION Station 17+941, 11 m Rt of C/L (E 410674.3, N 4872372.2) ORIGINATED BY RK

DIST HWY 401 BOREHOLE TYPE Solid Stem Auger COMPILED BY WC

DATUM Geodetic DATE 8/27/2010 CHECKED BY ZO

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
97.2	GROUND SURFACE						
0.0	240 mm **ASPHALT** **GRANULAR FILL:** Sand and Gravel grey, dense, damp	1	SS	47			
96.4 / 0.8	**EMBANKMENT FILL:** Sand and Gravel some gravel, some cobbles brown to grey, dense to compact, moist	2	SS	46			
		3	SS	43			66 27 5 2
		4	SS	29			
94.2 / 3.0	**EMBANKMENT FILL:** Silty Clay to Clayey Silt tr. sand, tr. rootlets, with organic pockets brown to grey, stiff	5	SS	12			
		6	SS	9			
	moist / wet	7	SS	5	▼*		
91.7 / 5.5	**SAND** with gravel some clay, some organics dark grey, compact, wet	8	SS	22			
91.1 / 6.1	**CLAYEY SILT** grey, firm to stiff, wet	9	SS	3			
	with fine grained sand interbeds	10	SS	11			
		11	SS	6			
87.6 / 9.6	End of Borehole Water level @ 4.6 m (not stabilized)* upon completion Borehole caved in @ 6.6 m upon completion						

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

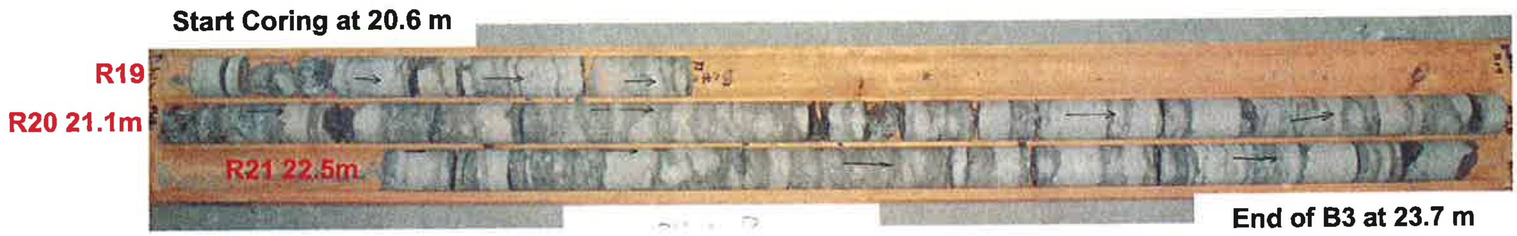
SHEAR STRENGTH (kPa) O UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE 20 40 60 80 100

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

UNIT WEIGHT γ kN/m³

ELEVATION SCALE 97 96 95 94 93 92 91 90 89 88

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



BOREHOLE B3

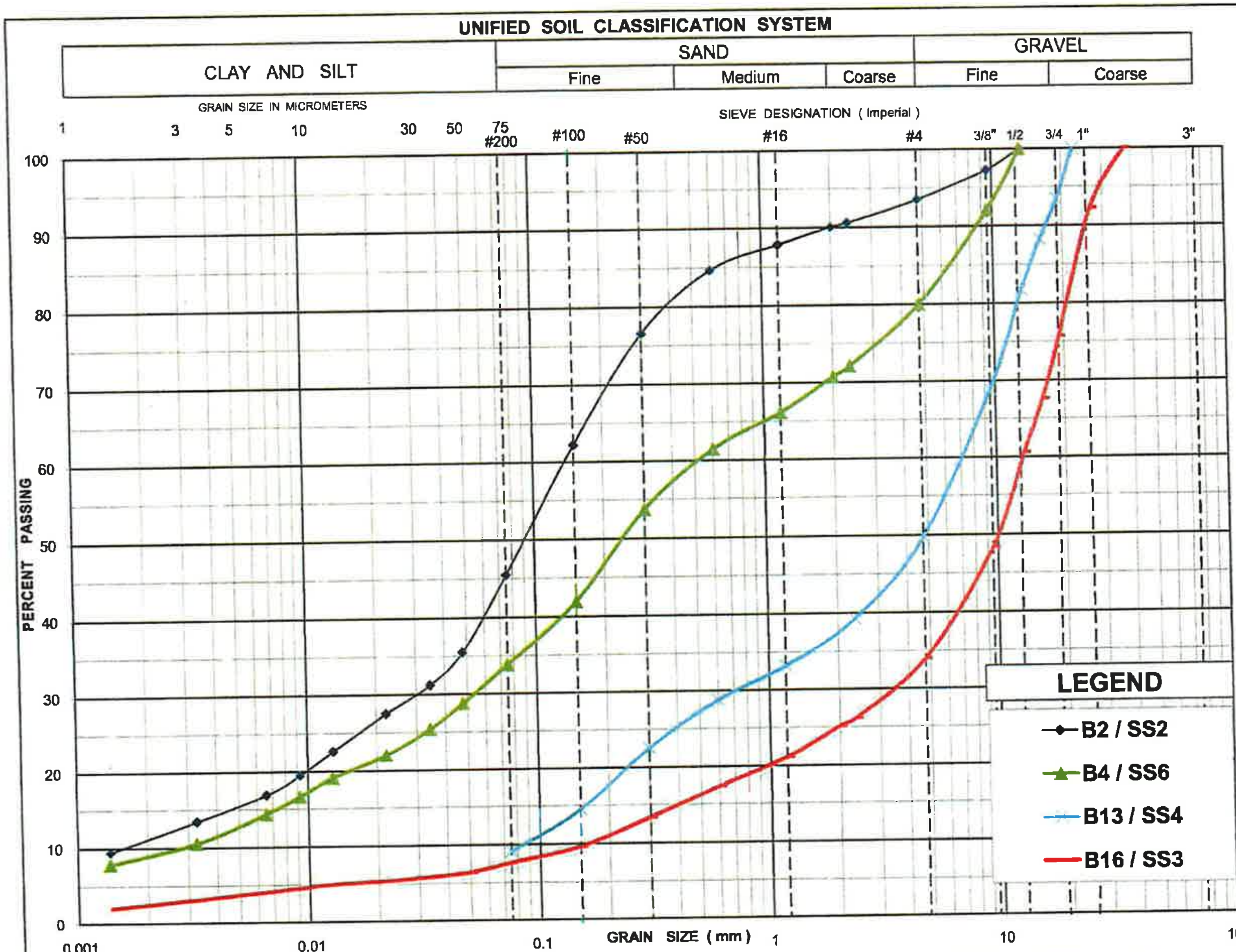
BOREHOLE B9

BOREHOLE B10

BOREHOLE B14

Appendix B

Laboratory Test Results



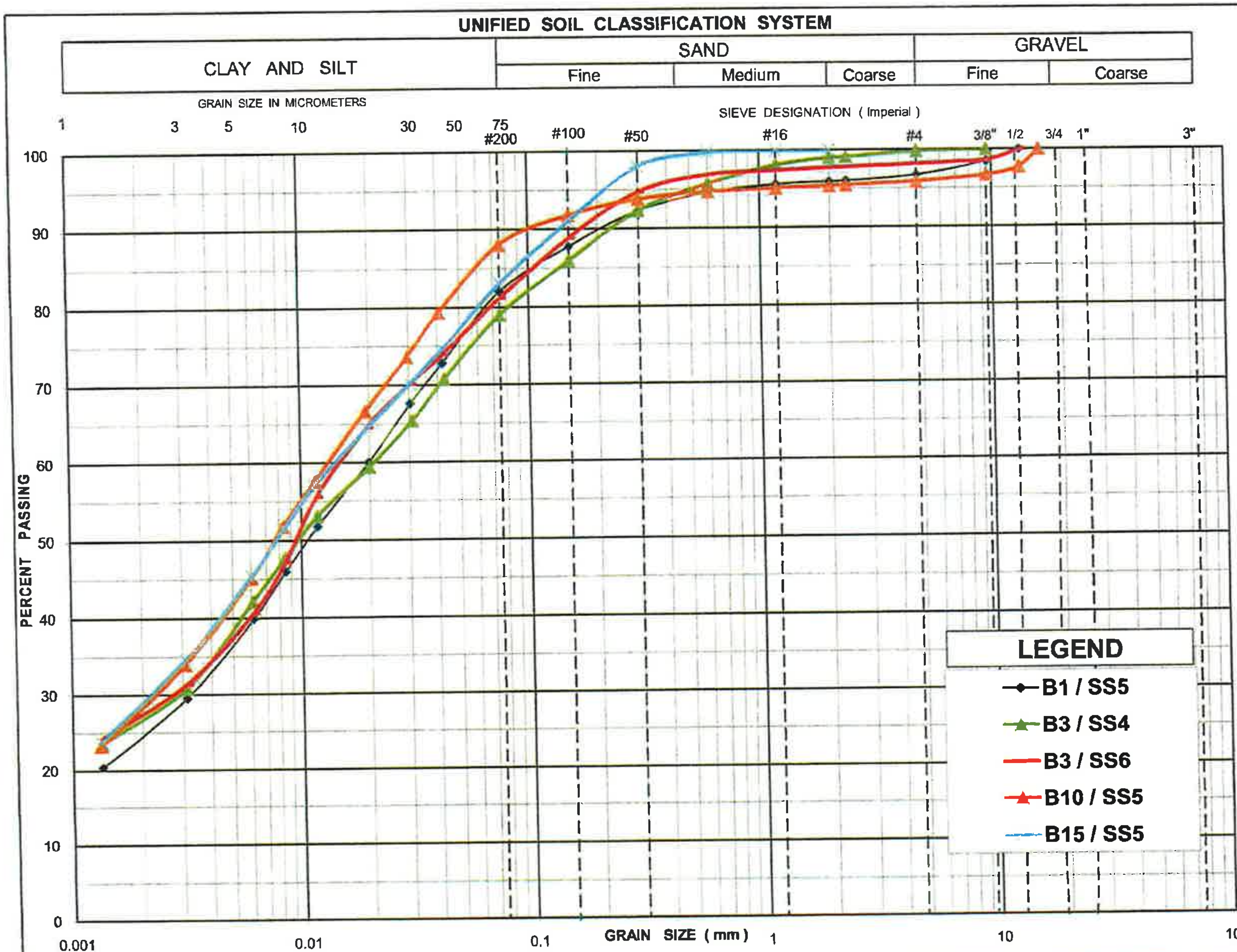
coffey geotechnics
SPECIALISTS MANAGING THE EARTH

GRAIN SIZE DISTRIBUTION
Upper Embankment Fill - Silty Sand to Sandy Gravel

FIGURE NO.: B1

PROJECT NO: TRANETOB10434AA

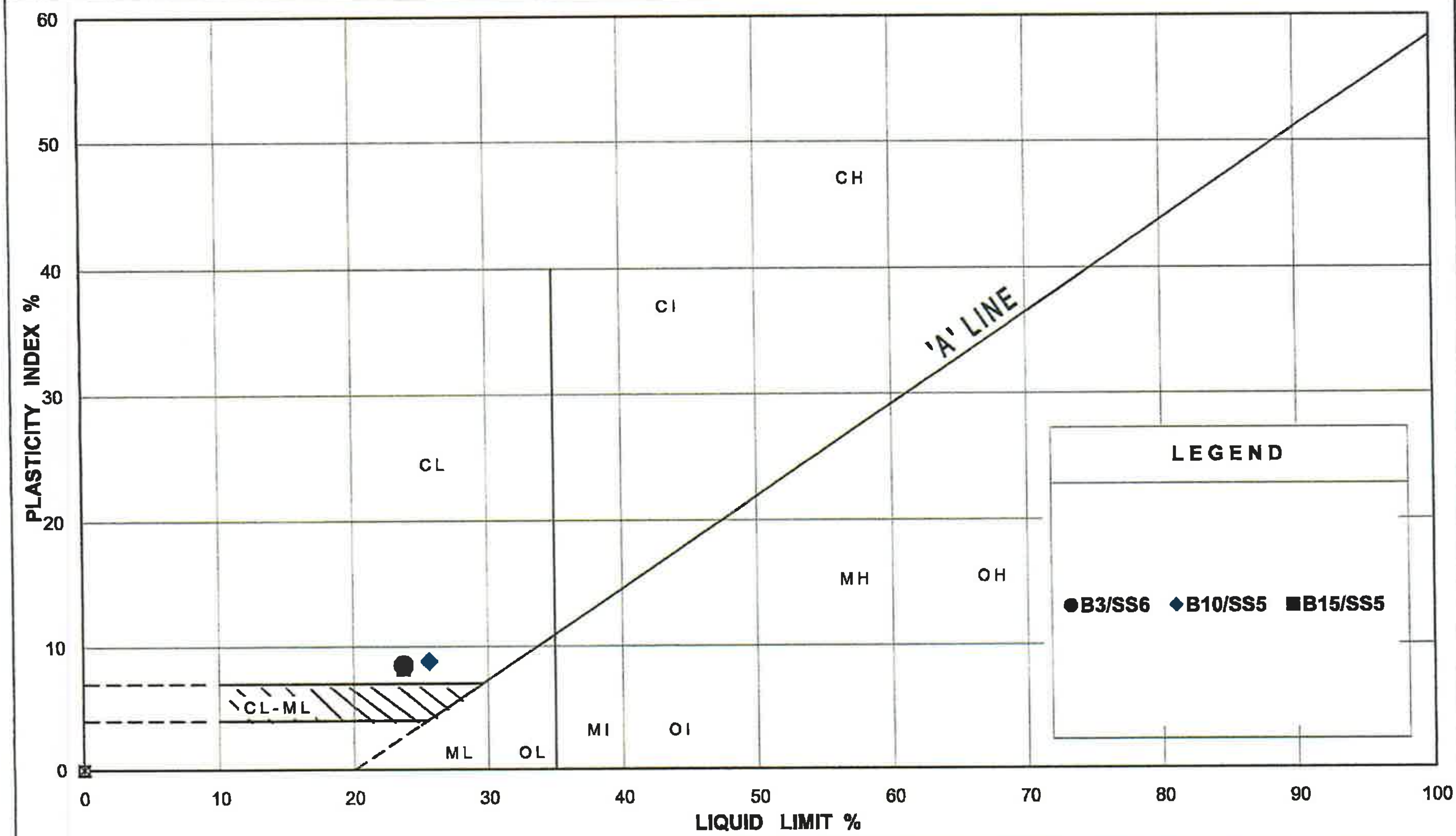
DATE: Nov, 2010



coffey geotechnics
SPECIALISTS MANAGING THE EARTH

GRAIN SIZE DISTRIBUTION
Lower Embankment Fill - Clayey Silt

FIGURE NO.:	B2
PROJECT NO:	TRANETOB10434AA
DATE:	Nov, 2010

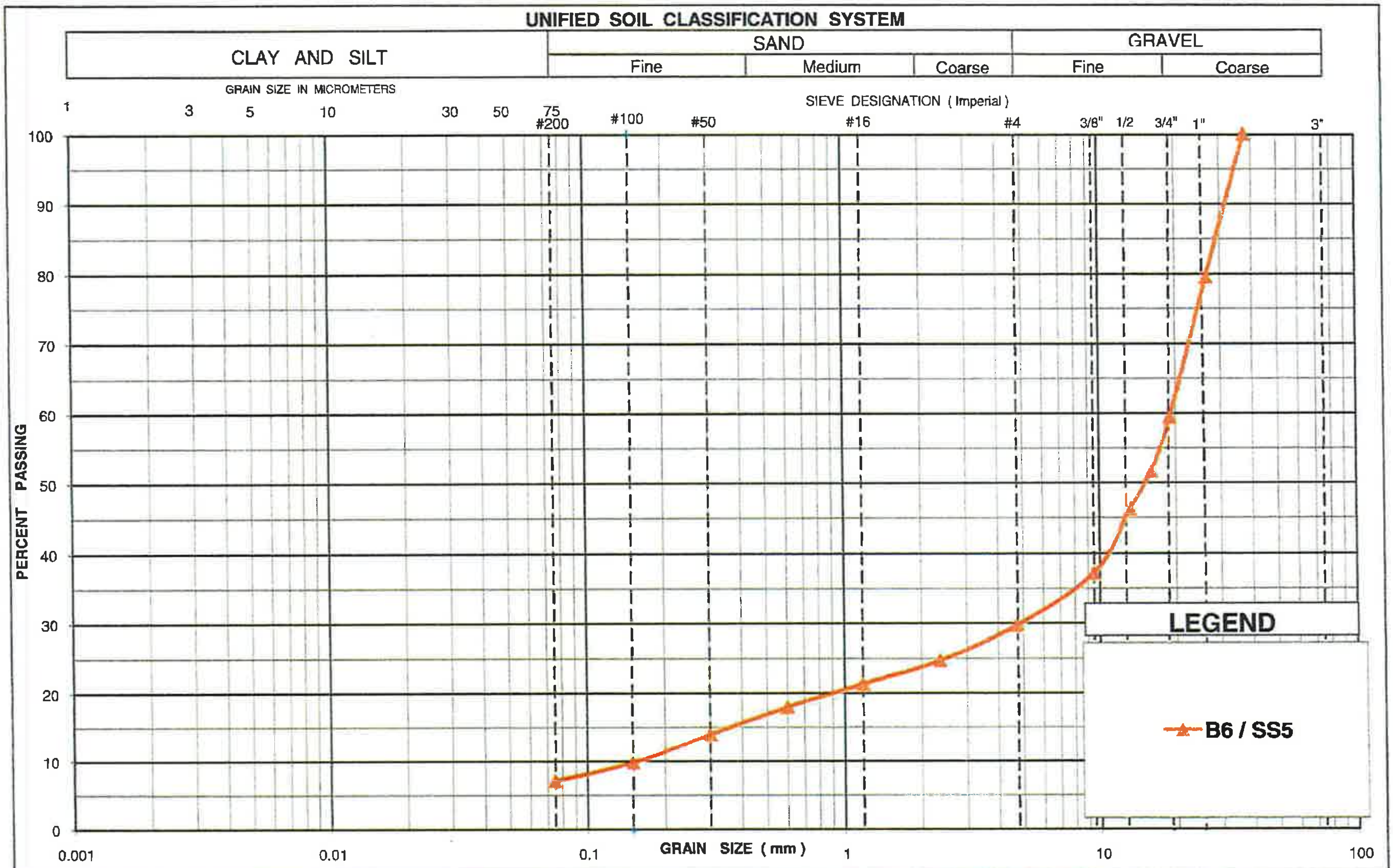


coffey geotechnics
SPECIALISTS MANAGING THE EARTH

PLASTICITY CHART

Lower Embankment Fill - Clayey Silt

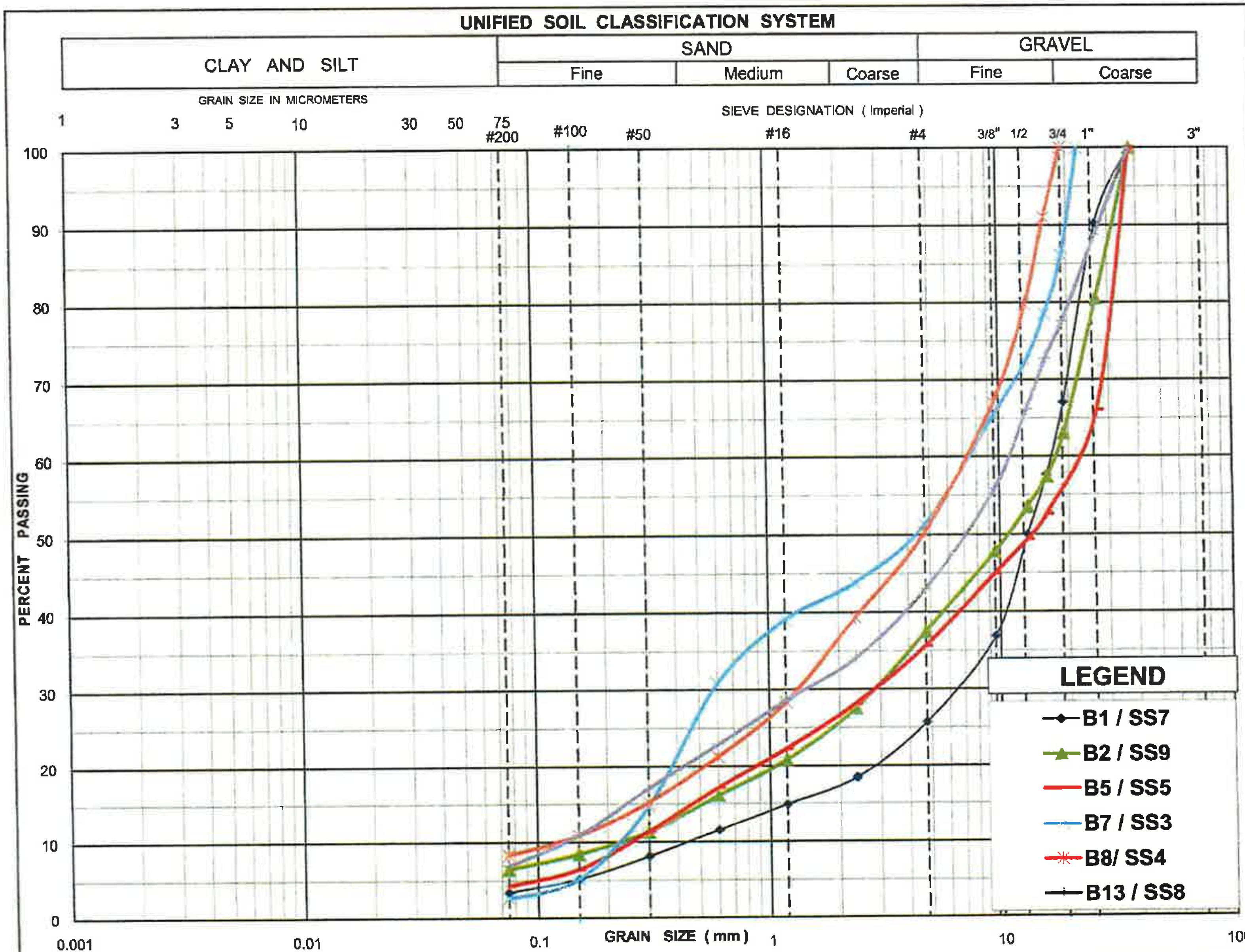
FIGURE No.	B3
REF. No.	TRANETOB10434AA
DATE	NOVEMBER, 2010



coffey geotechnics
SPECIALISTS MANAGING THE EARTH

GRAIN SIZE DISTRIBUTION
Construction Fill - Sandy Gravel

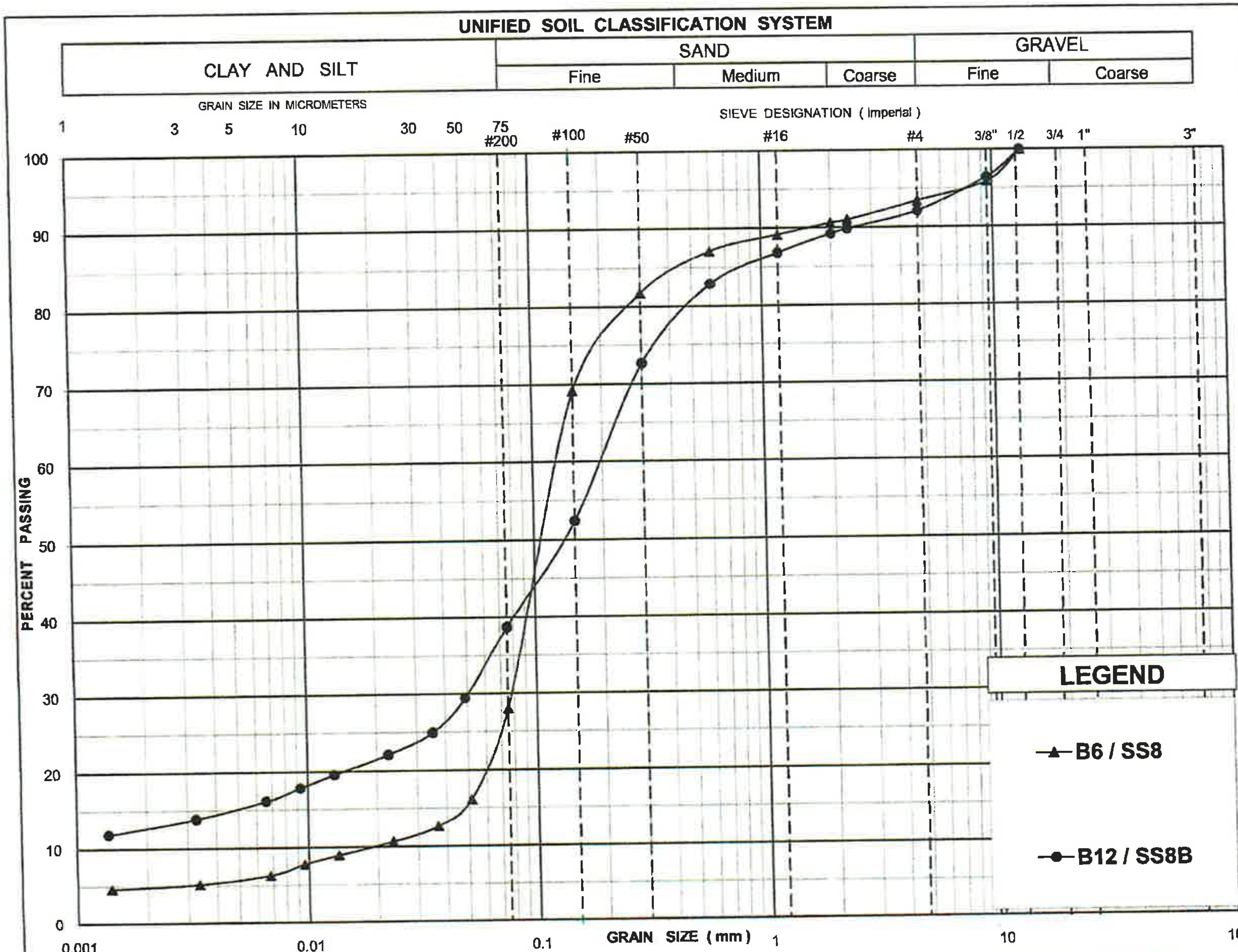
FIGURE NO.:	B4
PROJECT NO:	TRANETOB10434AA
DATE:	Nov 2010



coffey geotechnics
SPECIALISTS MANAGING THE EARTH

GRAIN SIZE DISTRIBUTION
Upper Granular Soil - Sand, Sandy Gravel, Sand & Gravel

FIGURE NO.:	B5
PROJECT NO:	TRANETOB10434AA
DATE:	NOV, 2010



coffey geotechnics
SPECIALISTS MANAGING THE EARTH

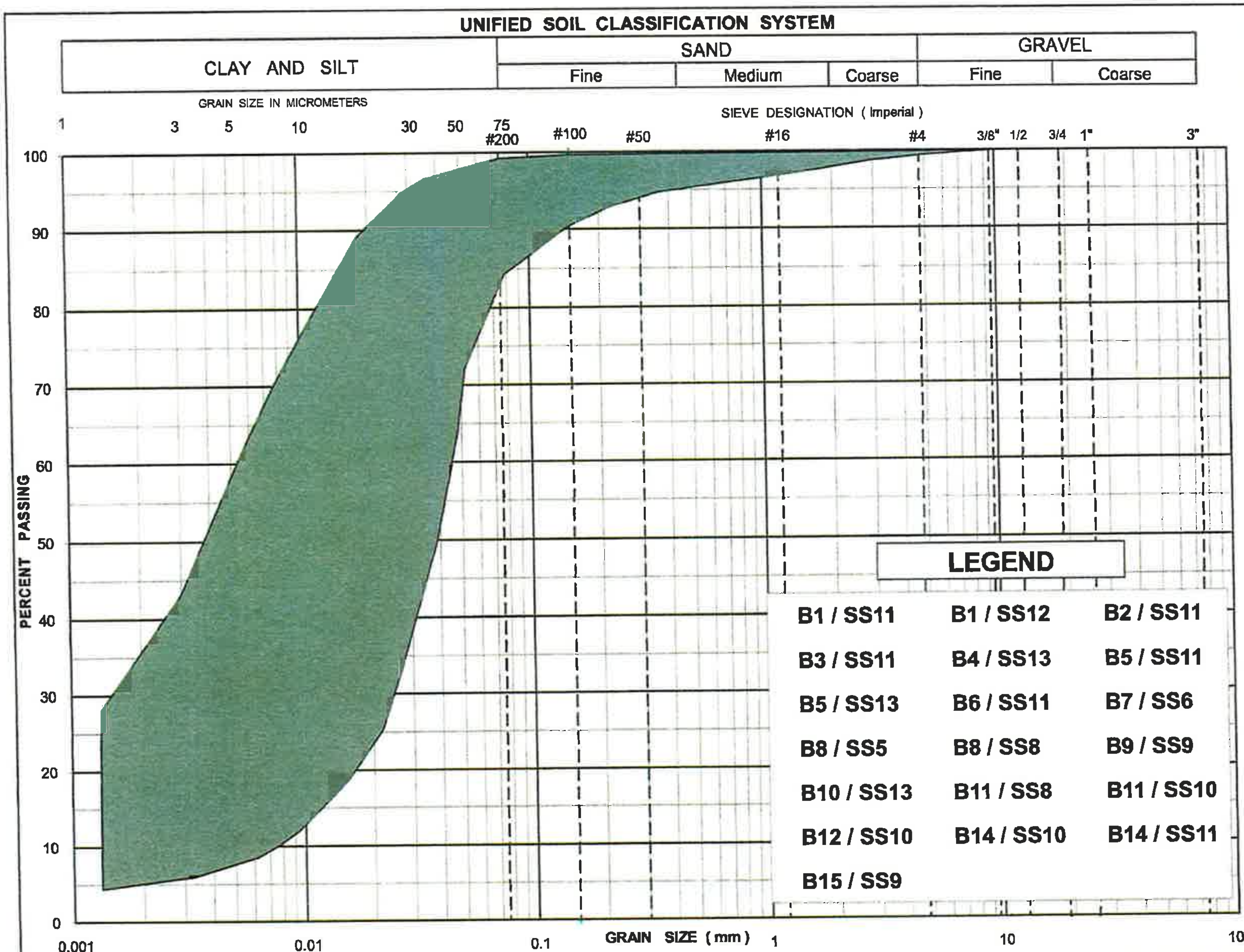
GRAIN SIZE DISTRIBUTION

Upper Granular Soil - Silty Sand

FIGURE NO.: B6

PROJECT NO: TRANETOB10434AA

DATE: NOV, 2010

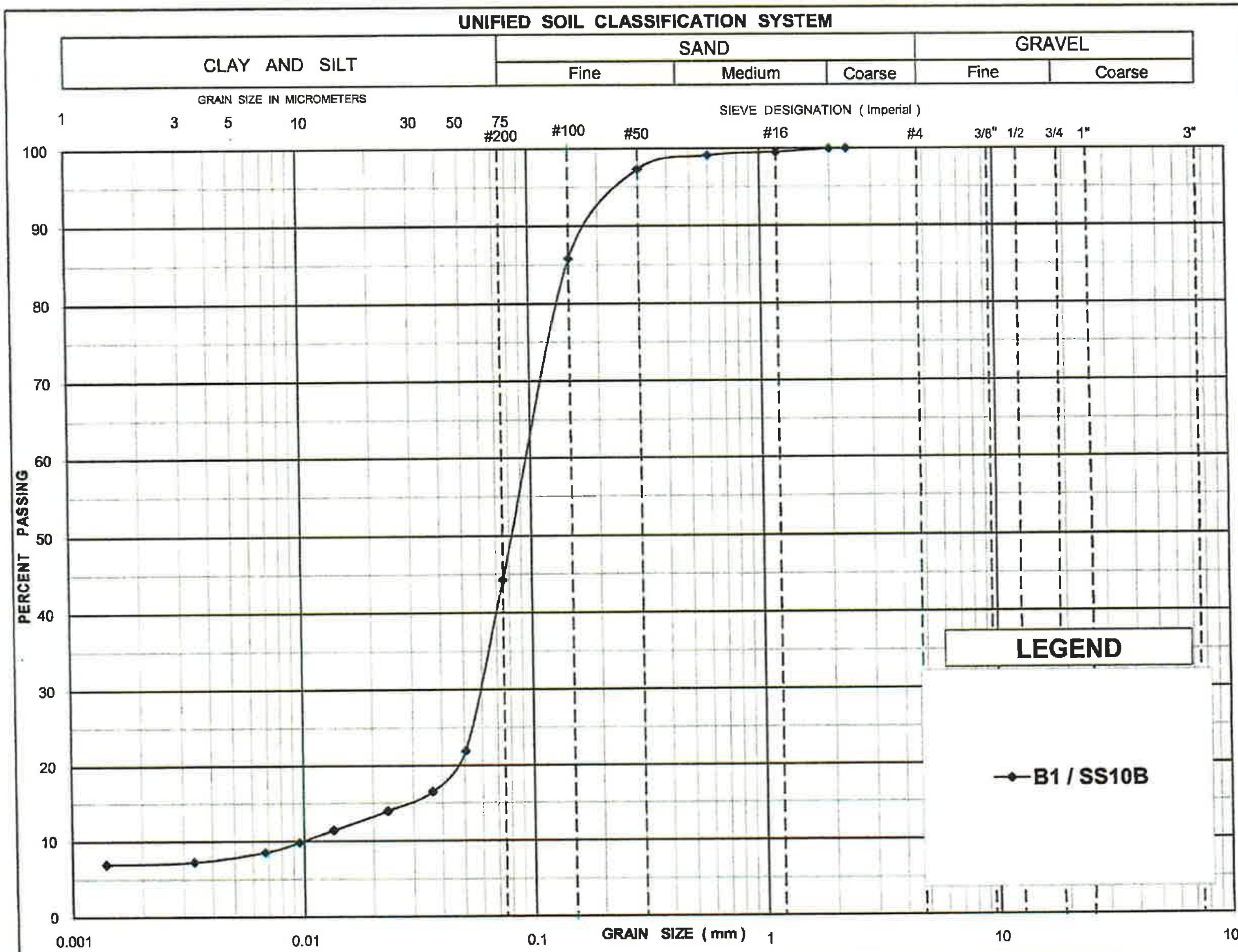


coffey geotechnics
SPECIALISTS MANAGING THE EARTH

GRAIN SIZE DISTRIBUTION

Clayey Silt

FIGURE NO.:	B7
PROJECT NO:	TRANETOB10434AA
DATE:	Nov, 2010



coffey geotechnics
SPECIALISTS MANAGING THE EARTH

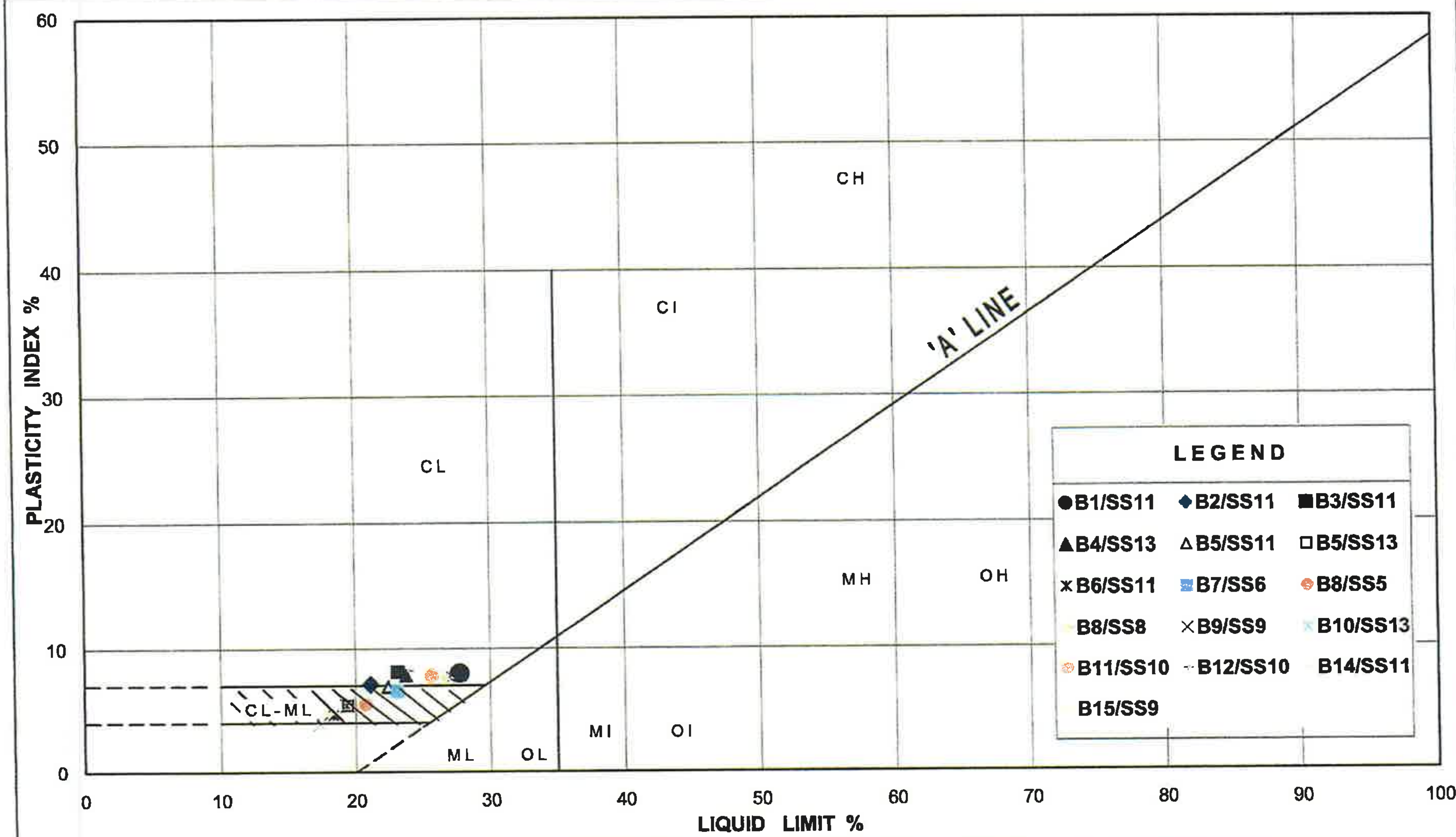
GRAIN SIZE DISTRIBUTION

Silty Sand Layer in Clay Silt

FIGURE NO.: B8

PROJECT NO: TRANETOB10434AA

DATE: Nov, 2010

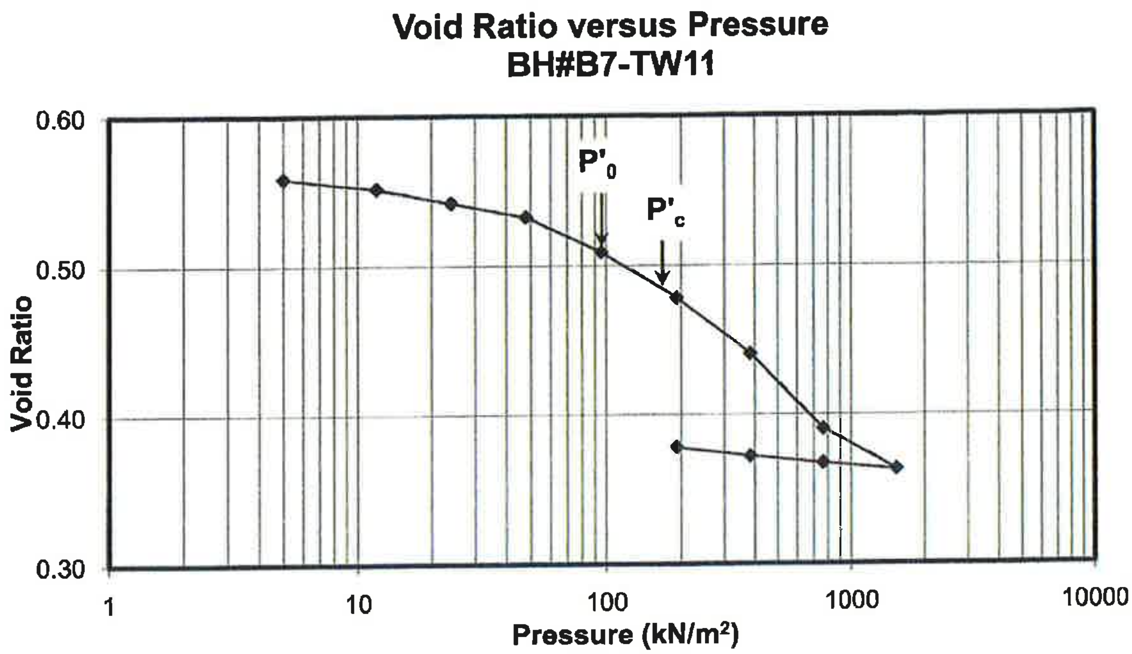


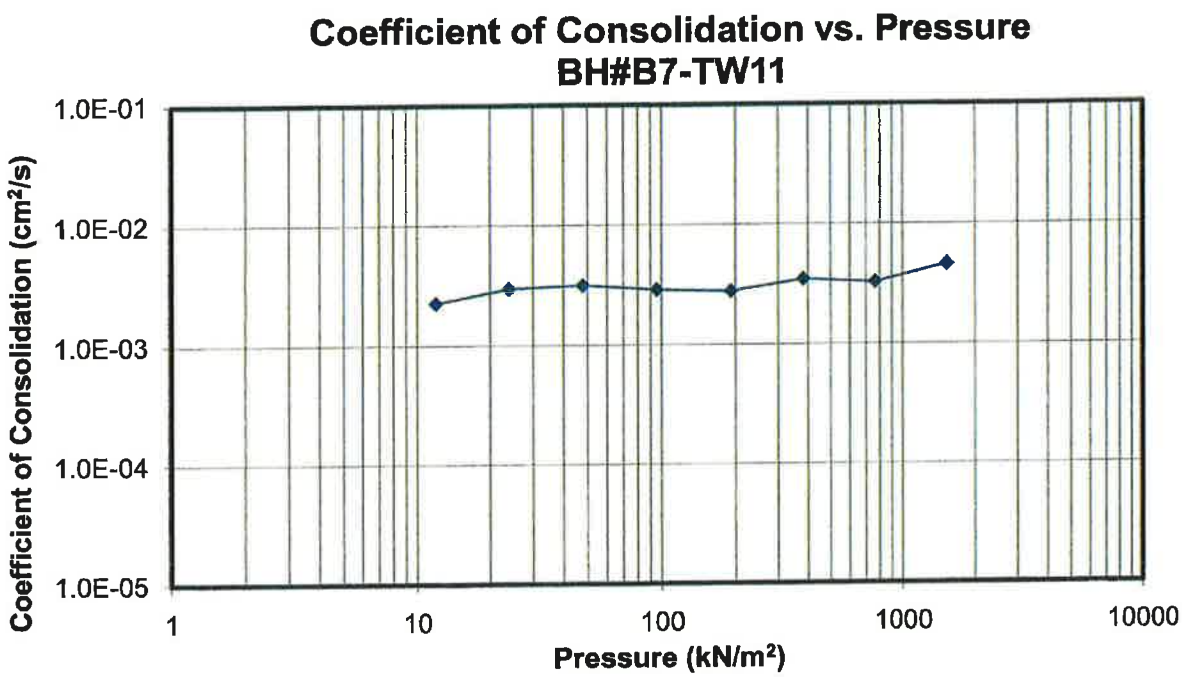
coffey geotechnics
SPECIALISTS MANAGING THE EARTH

PLASTICITY CHART

Clayey Silt

FIGURE No.	B9
REF. No.	TRANETOB10434AA
DATE	NOVEMBER, 2010

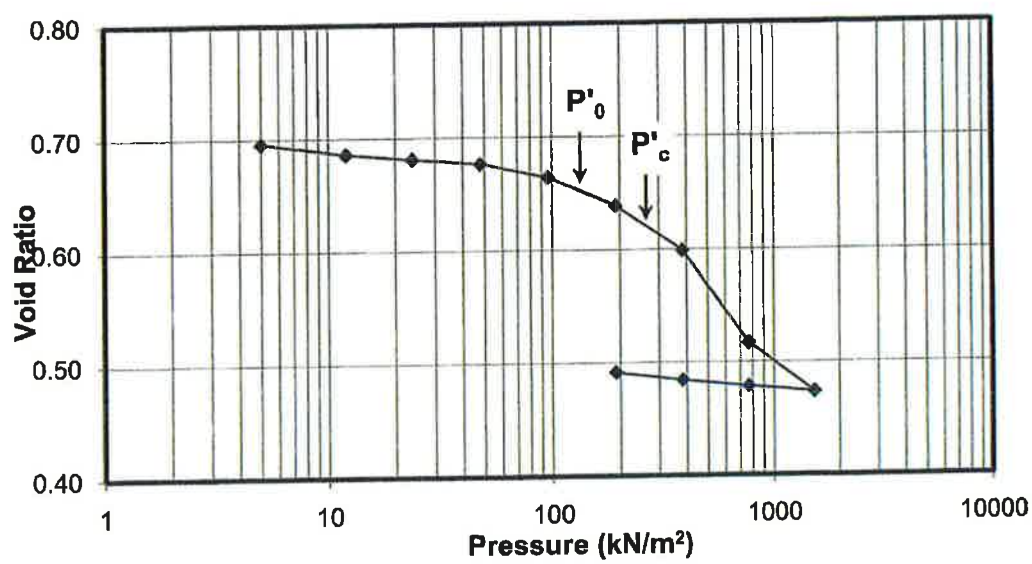




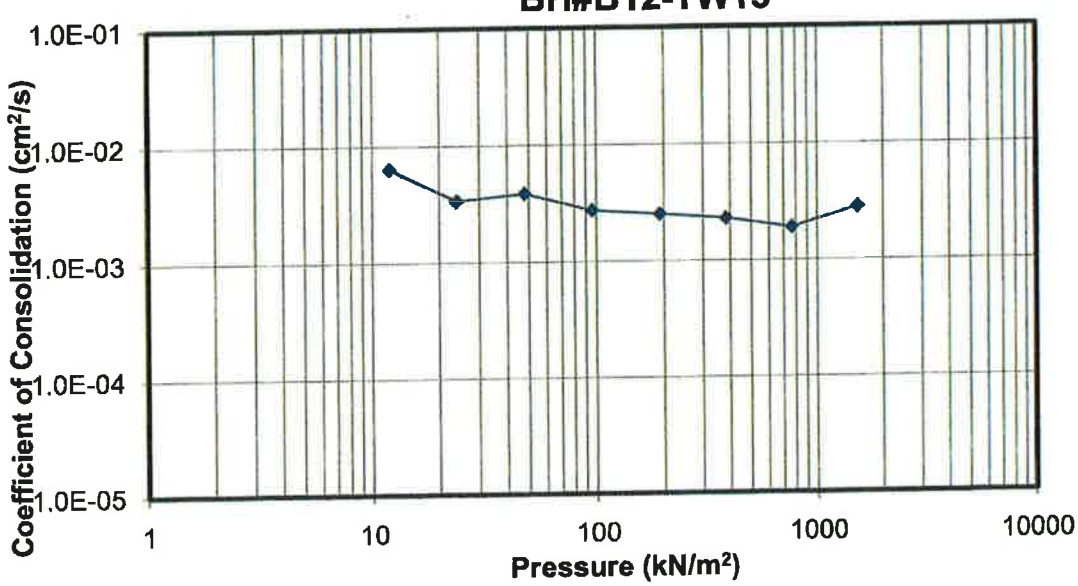
drawn	DS	coffey geotechnics SPECIALISTS MANAGING THE EARTH	client:	AECOM
approved	ZO		project:	HIGHWAY 401 EXPANSION COBOURG CREEK EAST STRUCTURE WIDENING
date	Jan-11			
scale	as shown		title:	CONSOLIDATION TEST RESULT - B7 TW11
original size	Letter		project no: TRANETOB10434AA	figure no: B10

F:\GEOT\transprt\ACTIVE\PROJECT\2010\10434 - TRANETOB10434AA - Hwy 401, Burnham to Nagle\foundation reports\Bridge\lab figures\[fig B10 B11 consolidation.xls]B11

Void Ratio versus Pressure
BH#B12-TW13

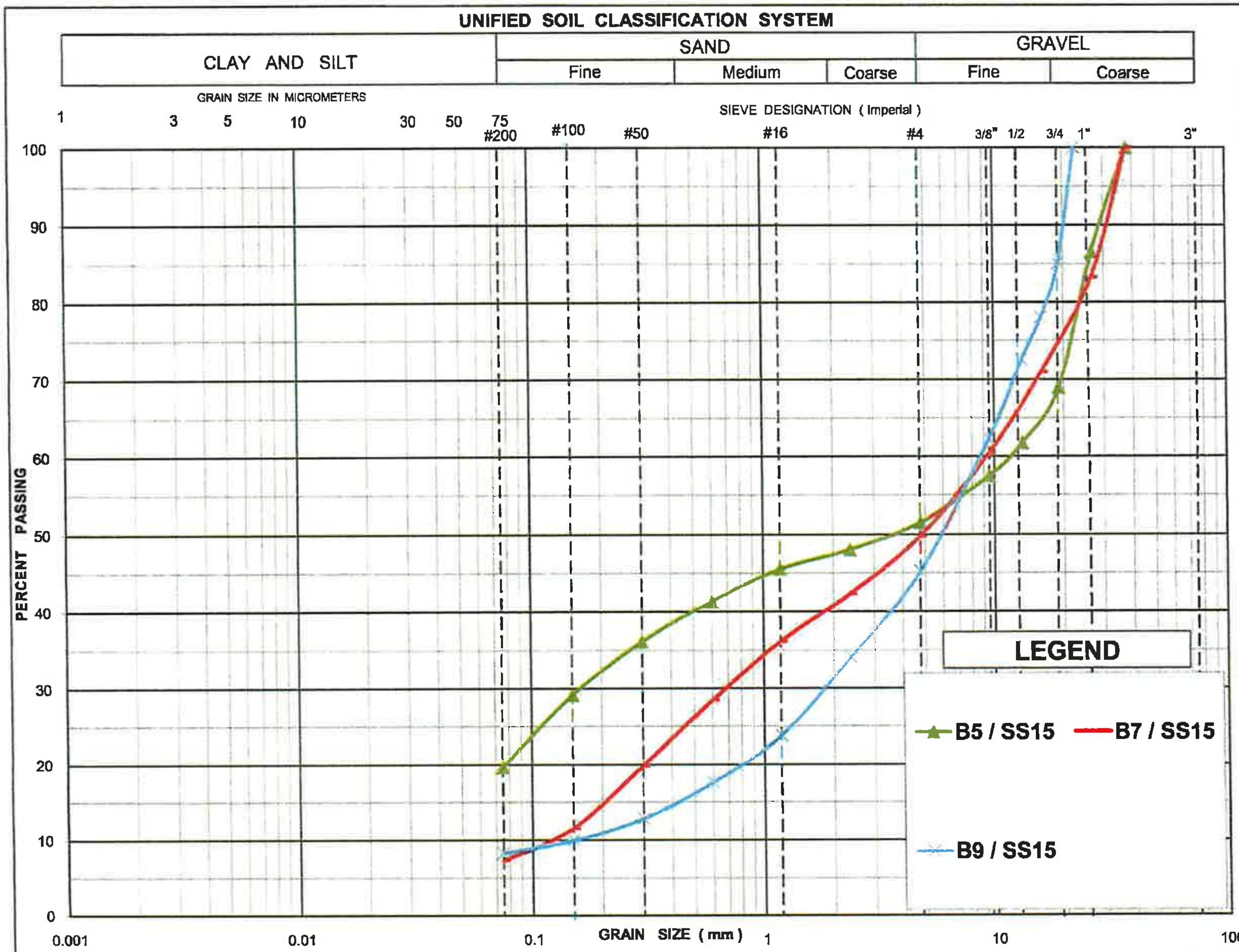


Coefficient of Consolidation vs. Pressure
BH#B12-TW13



drawn	DS	coffey geotechnics SPECIALISTS MANAGING THE EARTH	client:	AECOM
approved	ZO		project:	HIGHWAY 401 EXPANSION COBOURG CREEK EAST STRUCTURE WIDENING
date	Jan-11		title:	CONSOLIDATION TEST RESULT - B12 TW13
scale	as shown		project no:	TRANETOB10434AA
original size	Letter		figure no:	B11

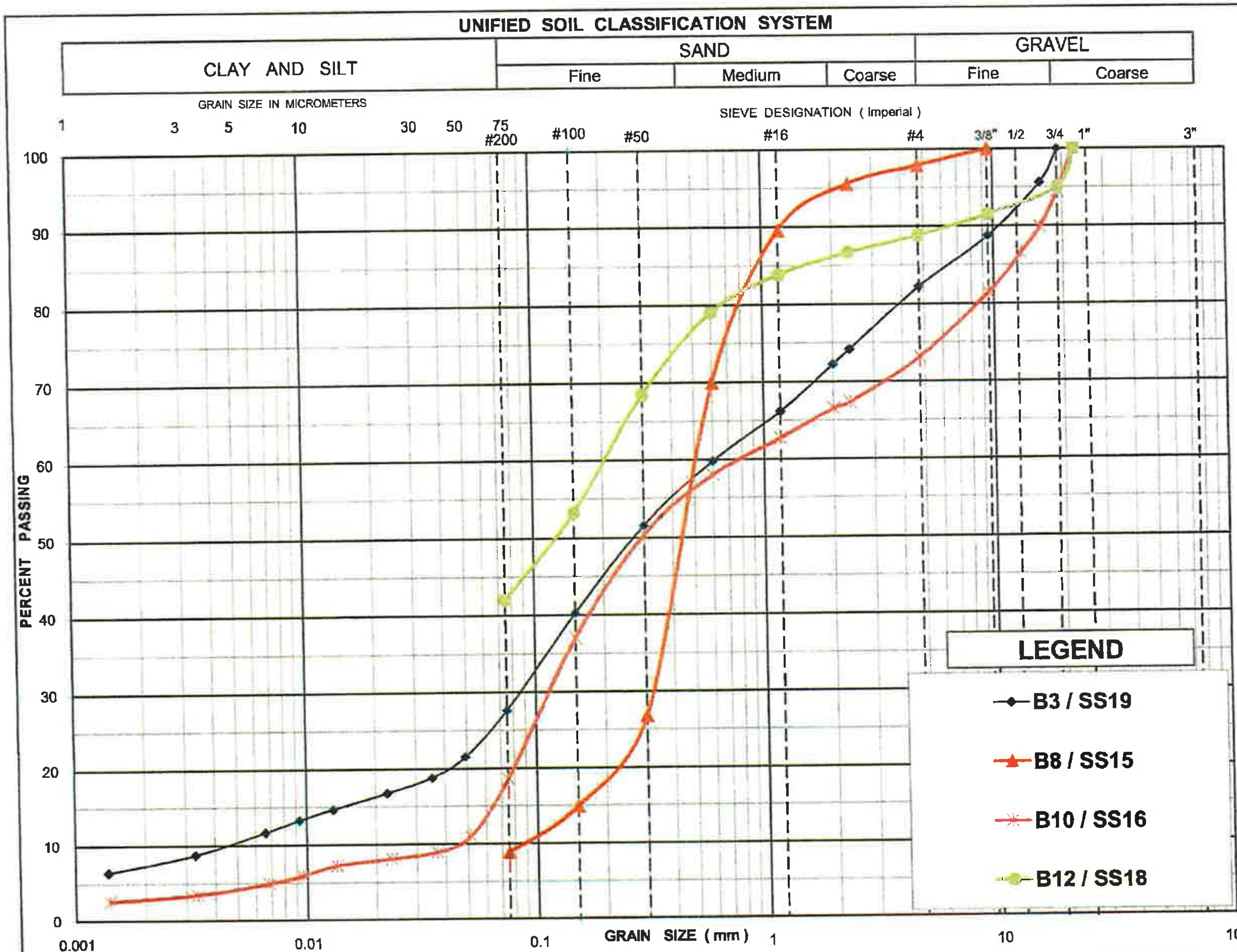
F:\GEOT\transprt\ACTIVE\PROJECT\2010\10434 - TRANETOB10434AA - Hwy 401, Burnham to Nagle\foundation reports\Bridge\lab figures\[fig B10 B11 consolidation.xls]B11



coffey geotechnics
SPECIALISTS MANAGING THE EARTH

GRAIN SIZE DISTRIBUTION
Lower Granular Soil - Sandy Gravel, Gravel and Sand & Gravel

FIGURE NO.:	B12
PROJECT NO:	TRANETOB10434AA
DATE:	Nov, 2010



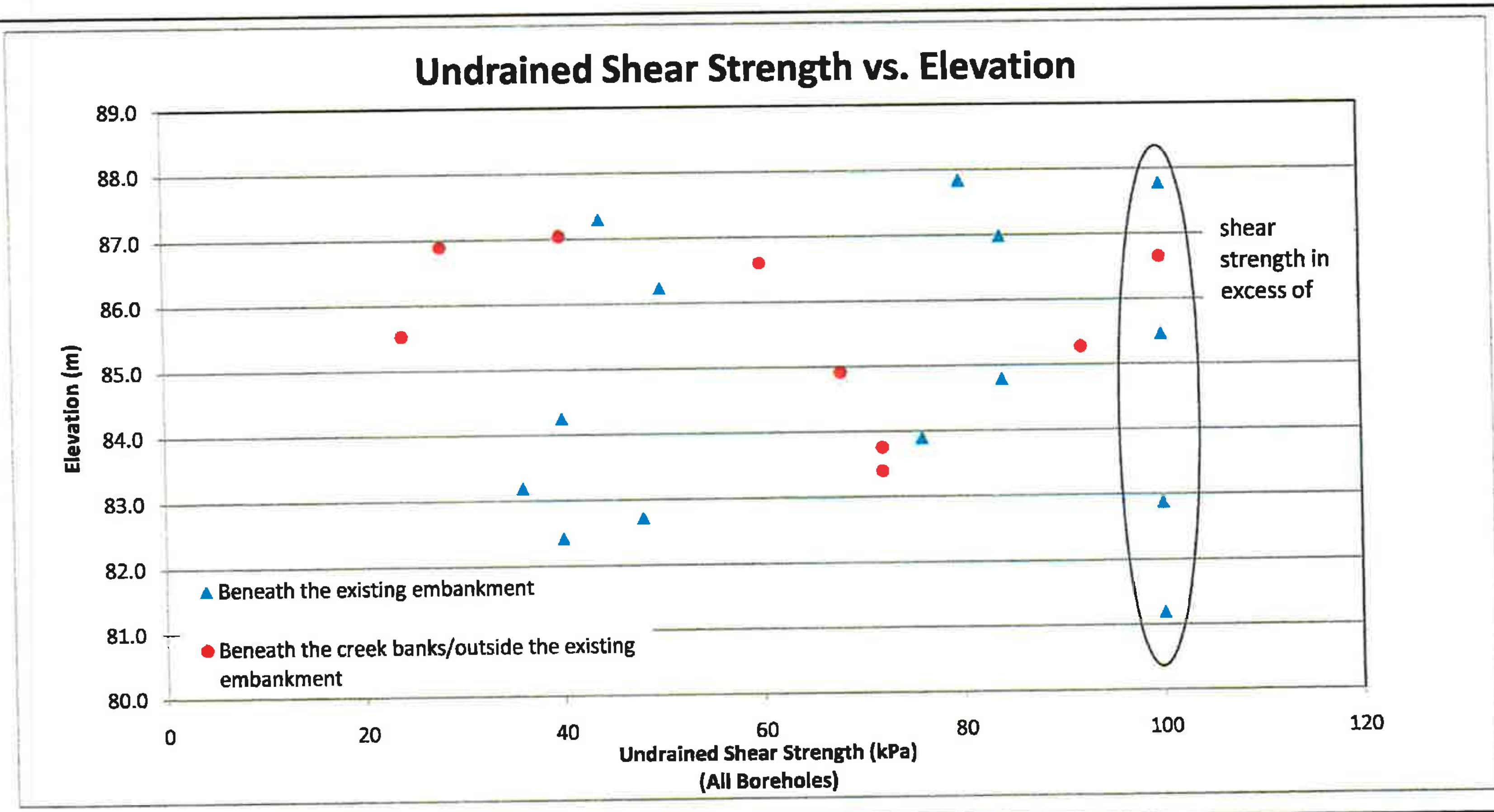
coffey geotechnics
SPECIALISTS MANAGING THE EARTH

GRAIN SIZE DISTRIBUTION
Lower Granular Soil - Silty Sand Till and Sand Till

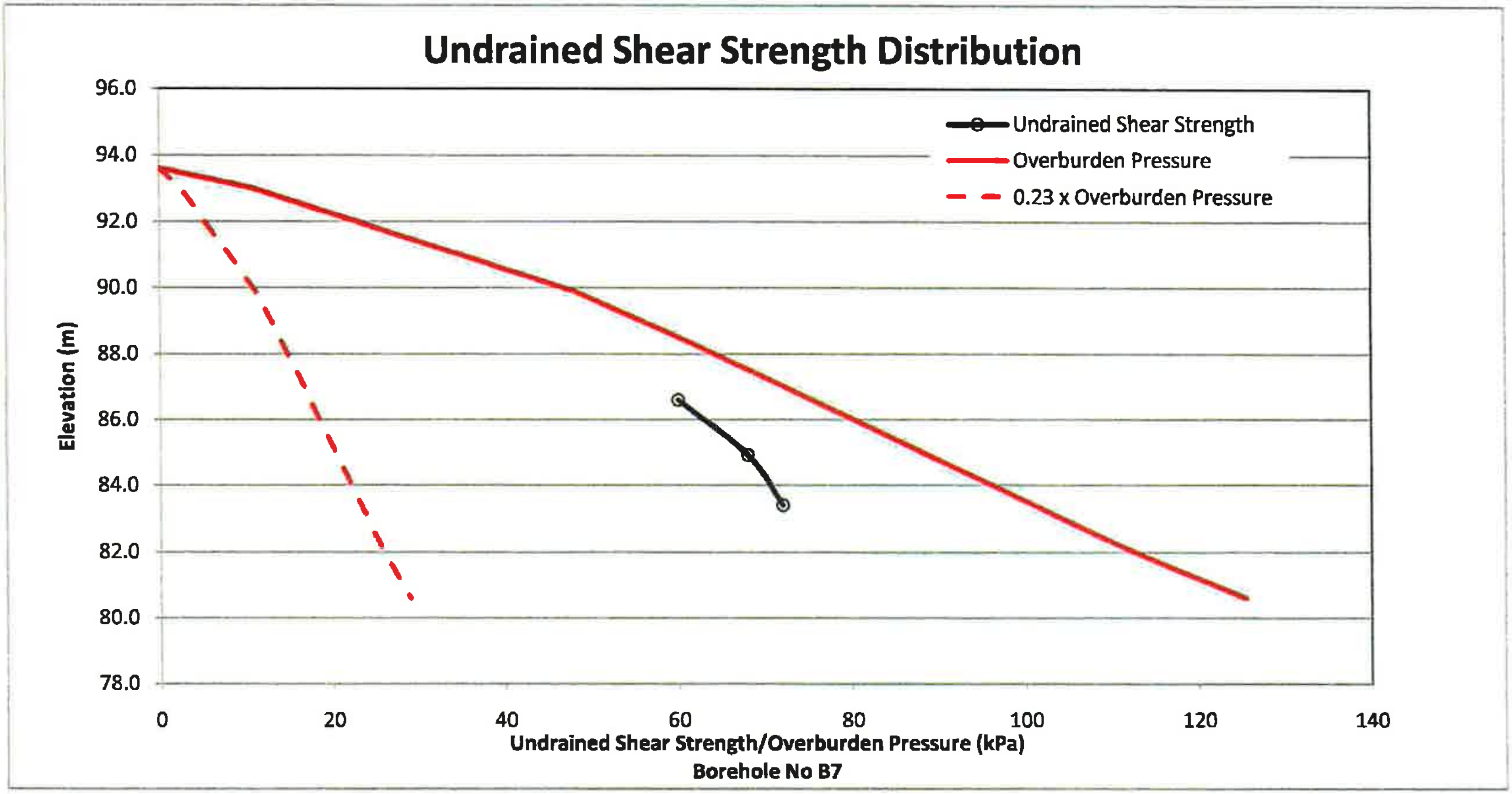
FIGURE NO.:	B13
PROJECT NO:	TRANETOB10434AA
DATE:	Nov, 2010

Appendix C

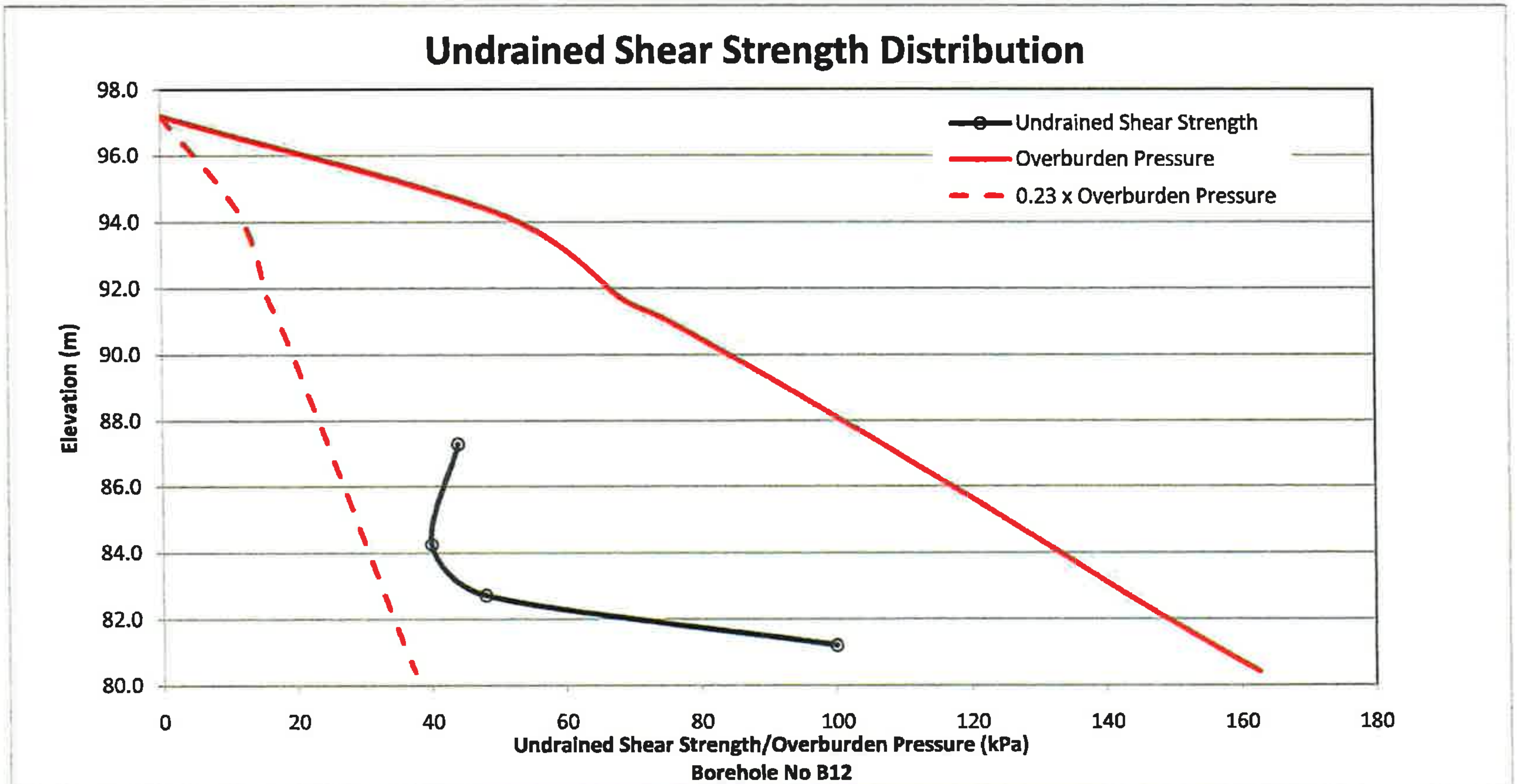
Undrained Shear Strength – Clayey silt



drawn	DS	coffey geotechnics SPECIALISTS MANAGING THE EARTH	client:	AECOM
approved	ZO		project:	Highway 401 Expansion Cobourg Creek East Structure Widening
date	Feb 2011			
scale	As Shown		title:	Undrained Shear Strength vs Elevation
original	Letter		project no: TRANETOB10434AA	drawing no: C1



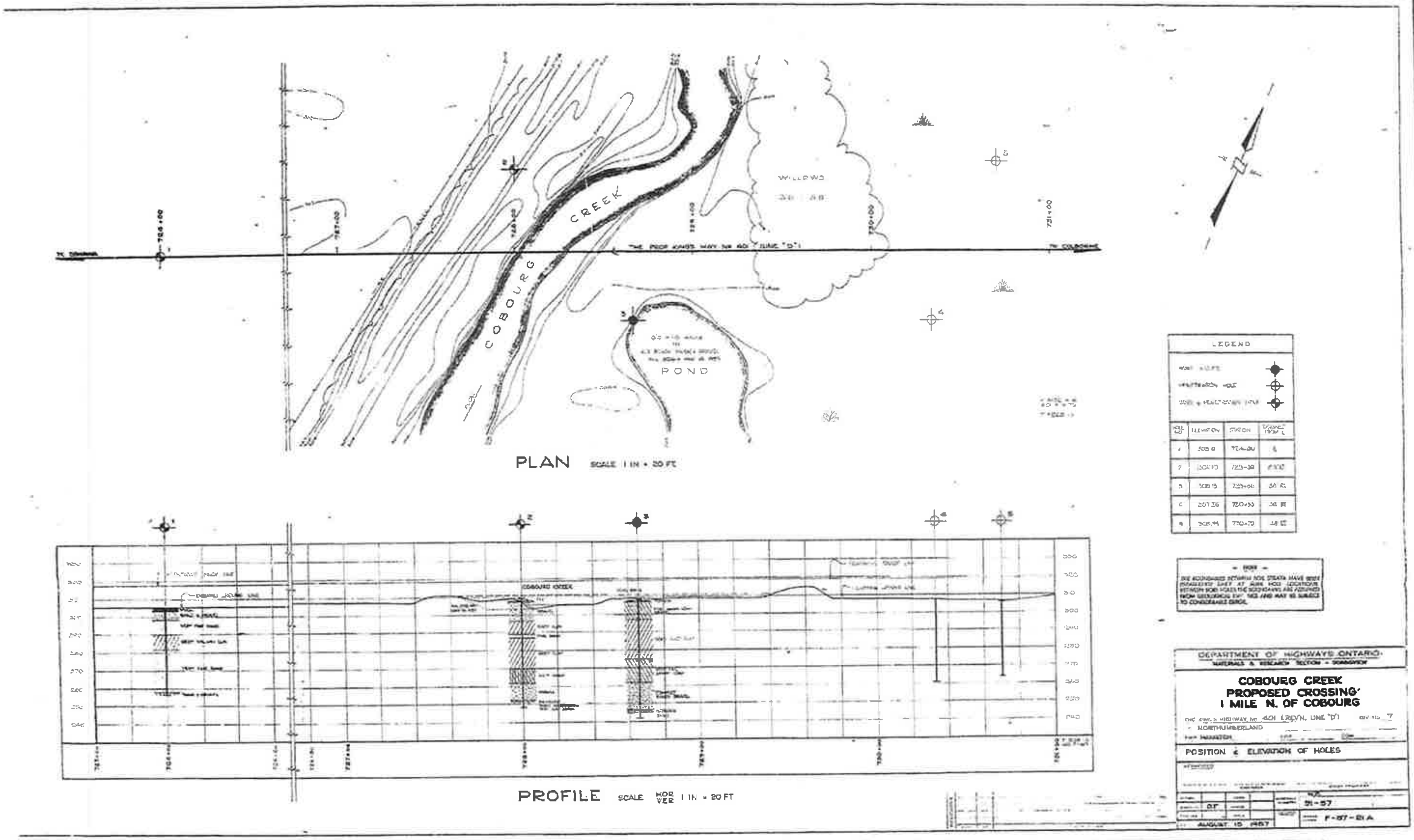
drawn	DS	coffey geotechnics SPECIALISTS MANAGING THE EARTH	client:	AECOM
approved	ZO		project:	Highway 401 Expansion Cobourg Creek East Structure Widening
date	Feb 2011			
scale	As Shown		title:	Undrained Shear Strength Distribution - B7
original	Letter		project no: TRANETOB10434AA	drawing no: C2



drawn	DS	coffey geotechnics SPECIALISTS MANAGING THE EARTH	client:	AECOM
approved	ZO		project:	Highway 401 Expansion Cobourg Creek East Structure Widening
date	Feb 2011			
scale	As Shown		title:	Undrained Shear Strength Distribution - B12
original	Letter		project no: TRANETOB10434AA	drawing no: C3

Appendix D

Record of Previous Boreholes by MTO


COBOURG CREEK
WILLOWS
POND
THE PROP. KING'S HWY. No 401 (LINE "D")
TO COBOURG
724+00
727+00
728+00
729+00
730+00
731+00
PLAN SCALE 1 IN = 20 FT
COBOURG CREEK
PROFILE SCALE HOR VER 1 IN = 20 FT
LEGEND
DEPARTMENT OF HIGHWAYS ONTARIO
COBOURG CREEK
PROPOSED CROSSING
1 MILE N. OF COBOURG
NORTHUMBERLAND
POSITION & ELEVATION OF HOLES
51-57
F-57-21A
AUGUST 15 1957

FL-129 (REV 56)

HDR

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & RESEARCH BRANCH - FOUNDATIONS SECTION - DOWNSVIEW

OFFICE REPORT ON SOIL EXPLORATION

DRILL RIG 54-1 OPERATION BORE & PENET'N JOB F-57-21 W.P. 31-57 BORING 1 STA. 724+00 ℄
CASING BX (standard samplers to fit unless noted) DATUM GEODETIC DATE REPORT AUG 1957
SAMPLER HAMMER WT. 250 LBS DROP 19 INCHES COMPILED BY AL CHECKED BY DATE BORING JUNE 19 1957

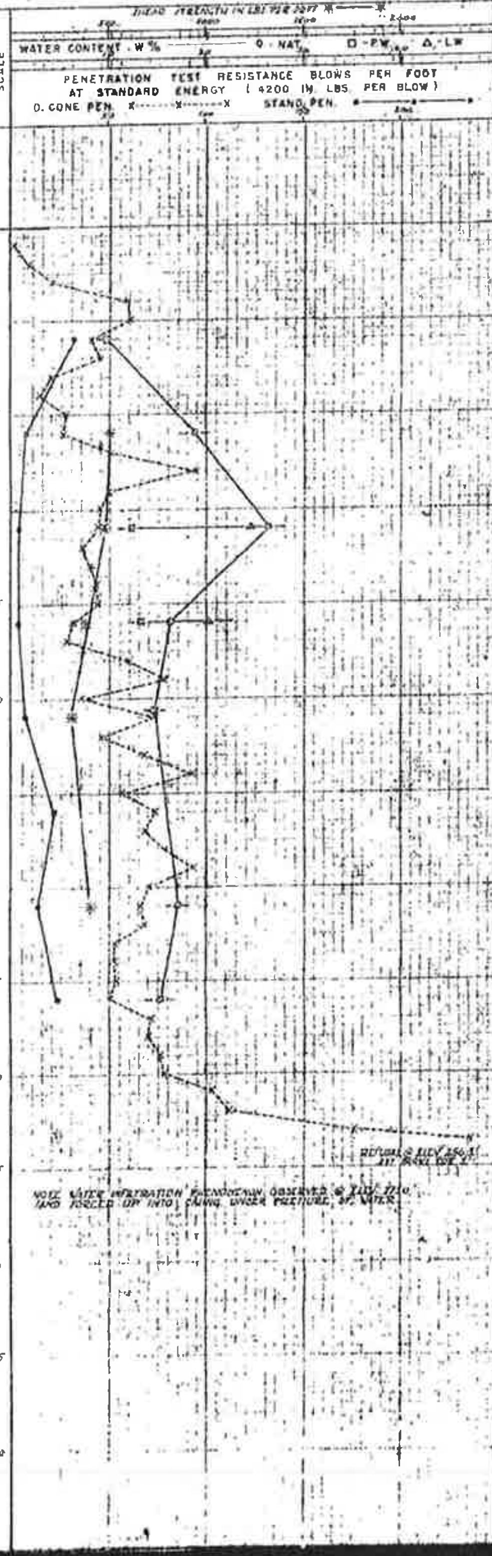
ABBREVIATIONS
V - INSITU VANE SHEAR TEST; M - MECHANICAL ANALYSIS; U - UNCONFINED COMPRESSION; Q_C - TRIAXIAL CONSOLIDATED QUICK; Q - TRIAXIAL QUICK; S - TRIAXIAL SLOW; WL - WATER LEVEL IN CASING; WT - WATER TABLE IN SOIL; K - PERMIABILITY; C - CONSOLIDATION; CA - CASING; γ - UNIT WEIGHT

SAMPLE TYPES
C.S. - CHUNK; D.O. - DRIVE OPEN; D.F. - DRIVE FOOT VALVE; T.O. - THIN WALLED OPEN; S.S. - SLEEVE SAMPLE; R.S. - PISTON SAMPLE; W.S. - WASHED SAMPLE; R.C. - ROCK CORE

SAMPLE CONDITION
DISTURBED; FAIR; GOOD; LOST

SOIL PROFILE

ELEVATION / DEPTH	DESCRIPTION
305.0'	GROUND LEVEL
303.0' / 2.0'	MUCK
298.0' / 7.0'	SAND & GRAVEL
290.0' / 15.0'	VERY FINE SAND
282.0' / 23.0'	GREY MEDIUM CLAY
259.0' / 47.0'	VERY FINE SAND
256.5' / 48.5'	SAND & GRAVEL
	END of BOREHOLE



SAMPLES

OTHER TESTS	CONDITION	TYPE	NO.	PENETRATION RESISTANCE	ELEV. RECOV. %
γ-110.0 p.c.f.	GOOD	T.O. 2"	1	31	100
γ-130.5 p.c.f.	GOOD	T.O. 2"	2	7	87.5
γ-126.4 p.c.f.	GOOD	T.O. 2"	3	3	100
γ-123.1 p.c.f.	GOOD	T.O. 2"	4	3	100
γ-127.8 p.c.f.	GOOD	T.O. 2"	5	7	91.7
	LOST	T.O. 2"	6	22	-
γ-131.3 p.c.f.	GOOD	T.O. 2"	7	14	62.5
γ-109.3 p.c.f.	GOOD	T.O. 2"	8	24	33.3
	LOST	T.O. 2"	9	PUSHED	-
		W/S			

TL-129 (REV 56)

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & RESEARCH BRANCH - FOUNDATIONS SECTION - DOWNSVIEW

OFFICE REPORT ON SOIL EXPLORATION

DRILL RIG _5/1_ OPERATION _BORE & PENETN_ JOB _T-57-21_ W.P. _51-57_ BORING _2_ STA _720+00 (45' LT.)_

CASING _BX_ (standard samplers to fit unless noted) DATUM _GEODETIC_ DATE REPORT _AUG 1957_

SAMPLER HAMMER WT _250_ LBS DROP _12_ INCHES COMPILED BY _A.L._ CHECKED BY ___ DATE BORING _JUNE 21 1957_

ABBREVIATIONS

V - INSITU VANE SHEAR TEST
M - MECHANICAL ANALYSIS
U - UNCONFINED COMPRESSION
Q_C - TRIAXIAL CONSOLIDATED QUICK
Q - TRIAXIAL QUICK
S - TRIAXIAL SLOW
WL WATER LEVEL IN CASING
WT - WATER TABLE IN SOIL
K - PERMIABILITY
C - CONSOLIDATION
CA - CASING
γ - UNIT WEIGHT

SAMPLE TYPES

C.S. - CHUNK
D.O. - DRIVE OPEN
D.F. - DRIVE FOOT VALVE
T.O. - THIN WALLED OPEN
S.S. - SLEEVE SAMPLE
P.S. - PISTON SAMPLE
W.S. - WASHED SAMPLE
R.C. - ROCK CORE

SAMPLE CONDITION

- DISTURBED
- FAIR
- GOOD
- LOST

SOIL PROFILE

ELEVATION DEPTH	WATER CONDITIONS	DESCRIPTION
309.20'		GROUND LEVEL
307.20' / 2.0'		TILL
296.20' / 13.0'	∇ @ ELEV 304.20 JUNE 21 1957	GRAVEL
289.20' / 20.0'		GREY CLAY
287.20' / 22.0'		FINE SAND
272.20' / 37.0'		GREY CLAY
262.20' / 47.0'		SILTY LOAM
253.20' / 56.0'		GRAVEL
248.20' / 61.0'		BEDROCK SHALE INTERBEDDED WITH CLAY BANDS
		END OF BOREHOLE

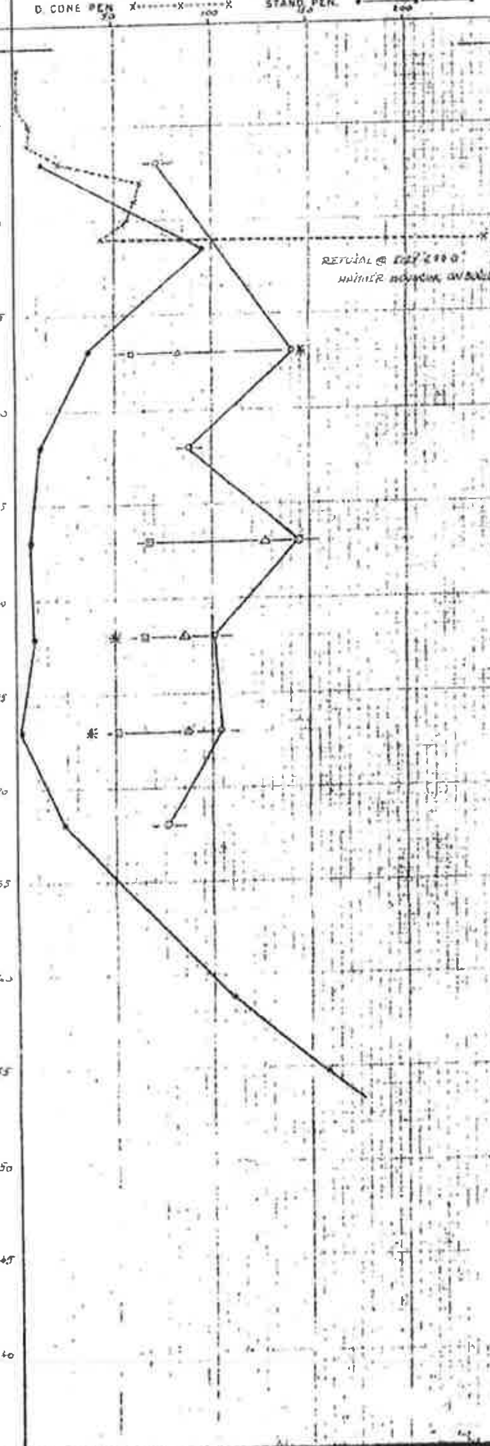
ELEVATION SCALE: 305, 300, 295, 290, 285, 280, 275, 270, 265, 260, 255, 250, 245, 240

SHEAR STRENGTH IN LBS PER SQ FT: 500, 1000, 1500, 2000

WATER CONTENT W% — O - NAT. □ - PW. △ - LW.: 10, 20, 30

PENETRATION TEST RESISTANCE BLOWS PER FOOT AT STANDARD ENERGY (4200 IN. LBS PER BLOW)

D. CONE PEN. x----x----x STAND. PEN. •——•: 50, 100



REFUSAL @ ELEV 299.0' HAMMER BOUNCING ON BOULDER

SAMPLES

OTHER TESTS	CONDITION	TYPE	NO.	PENETRATION RESISTANCE	ELEV RECOV. %
M + Hd, pcf	GOOD	T.O. 2"	1	12	416
	LOST	D.O.	2	95	
γ 1208 pcf	GOOD	T.O. 2"	3	16	[illegible]
γ 1267	GOOD	T.O. 2"	4	[illegible]	[illegible]
	DISTURBED	T.O. 2"	[illegible]	6	[illegible]
γ 1253 pcf	GOOD	T.O. 2"	6	[illegible]	50
γ 1253 pcf	GOOD	T.O. 2"	7	[illegible]	[illegible]
γ 1364 pcf	GOOD	T.O. 2"	8	23	
		V.S.			
	LOST	D.O.	9	>100	
		V.S.			
	DISTURBED	D.O.	10	>100	[illegible]
		AXT R.C.	11		100

END OF CASING @ ELEV 248.20'

TL-129 (REV 56) H.O.R.

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & RESEARCH BRANCH - FOUNDATIONS SECTION - DOWNSVIEW

OFFICE REPORT ON SOIL EXPLORATION

DRILL RIG 54-1 OPERATION BORE JOB 7-57-21 W.P. 51-57 BORING 3 STA 728+00(38'RT)

CASING BX (standard samplers to fit unless noted) DATUM GEODETIC DATE REPORT AUG 1957

SAMPLER HAMMER WT 250 LBS. DROP 19 INCHES COMPILED BY A.L. CHECKED BY DATE BORING JUNE 27 1957

ABBREVIATIONS

V - INSITU VANE SHEAR TEST
M - MECHANICAL ANALYSIS
U - UNCONFINED COMPRESSION
Q_C - TRIAXIAL CONSOLIDATED QUICK
Q - TRIAXIAL QUICK
S - TRIAXIAL SLOW
WL - WATER LEVEL IN CASING
W.T. - WATER TABLE IN SOIL
K - PERMIABILITY
C - CONSOLIDATION
CA - CASING
γ - UNIT WEIGHT

SAMPLE TYPES

C.S - CHUNK
D.O - DRIVE OPEN
DF - DRIVE FOOT VALVE
T.O - THIN WALLED OPEN
S.S - SLEEVE SAMPLE
P.S - PISTON SAMPLE
W.S - WASHED SAMPLE
R.C - ROCK CORE

SAMPLE CONDITION

- DISTURBED
- FAIR
- GOOD
- LOST

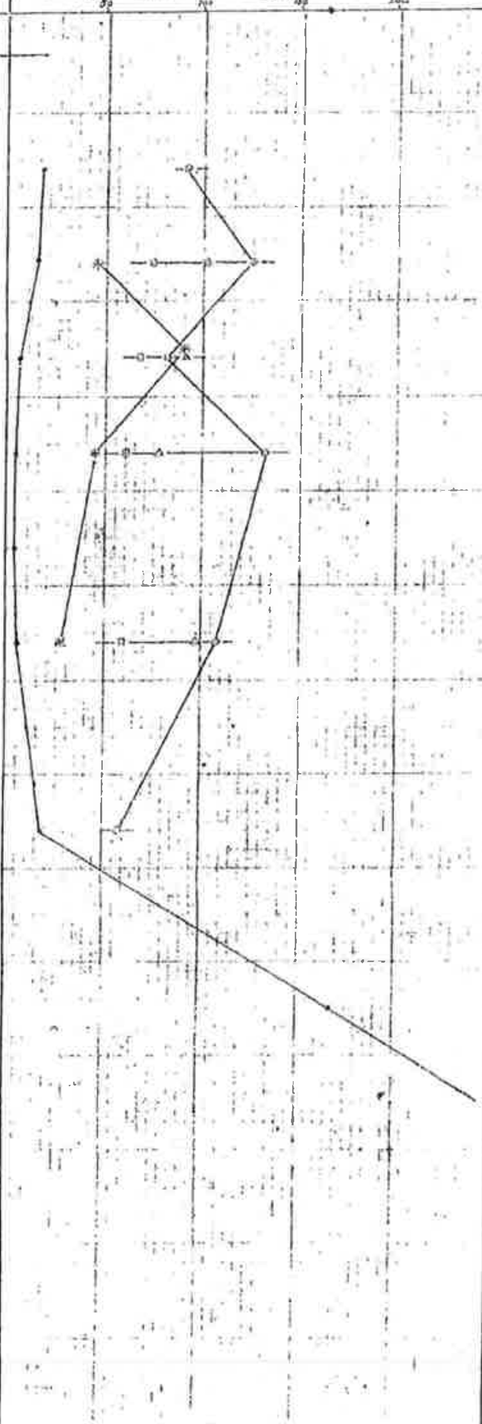
SOIL PROFILE

ELEVATION DEPTH	DESCRIPTION	ELEVATION SCALE
308.15'	GROUND LEVEL	
307.4' / .75'	TOPSOIL	
	FINE SANDY LOAM GRAVEL	305
297.15' / 11.0'		300
	SOFT GREY CLAY	295, 290, 285, 280
273.15' / 35.0'		275
	VERY FINE SANDY LOAM	270, 265
262.15' / 46.0'		
	"COMPACT" SANDY GRAVEL	260, 255, 250
247.15' / 61.0'	BEDROCK	
	SHALE	245
242.15' / 66.0'	END of BOREHOLE	240

SHEAR STRENGTH IN LBS PER SQ FT 500 1000 1500 2000

WATER CONTENT W% ○ NAT. □ - PW △ - LW

PENETRATION TEST RESISTANCE BLOWS PER FOOT AT STANDARD ENERGY (4200 IN. LBS PER BLOW) D. CONE PEN. X---X---X (STAND. PEN. ●—●—● 50 100 150 200



CASING BLOWS (ACTUAL): 1, 1, 4, 5, 9, 15, 84, 50, 36, 12, 16, 24, 21, 15, 10, 9, 3, 6, 5, 3, 4, 6, 13, 11, 16, 4, 10, 13, 2, 7, 5, 6, 6, 7, 7, 10, 10/1183, 70/112, 14, 11, 16, 29, 100, 321, 310, 215, 310

CASING DRIVEN IN TO 247.15' — END of CASING (ELEV. 247.15')

SAMPLES

ELEV.	OTHER TESTS	CONDITION	TYPE	NO	PENETRATION RESISTANCE	RECOV. %
308.15'						
303.15'	γ=130 pcf	GOOD	TO 2"	1	16	91.6
298.15'	γ=147.3 pcf	GOOD	SS	2	14	100
293.15'	γ=106.6 pcf	GOOD	PS 2"	3	5	58.3
288.15'	γ=112.4 pcf	GOOD	PS 2"	4	3	83+
283.15'		LOST	PS 2"		3	-
278.15'	γ=118.8 pcf	GOOD	PS 2"	5	5	41.6
268.15'		GOOD	PS 2"	6	18	41.6
			V.S.			
258.15'		LOST	DO	7	3/100	-
			V.S.			
253.15'		LOST	DO	8	3/50	-
			V.S.			
247.15'		GOOD	2XT RC	9		100

DEPTH: 3, 8, 13, 18, 23, 28, 33, 38, 43, 48, 53, 58, 63, 68

TL-129 (REV. 56) HDR

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & RESEARCH BRANCH - FOUNDATIONS SECTION - DOWNSVIEW

OFFICE REPORT ON SOIL EXPLORATION

DRILL RIG 5A-1 OPERATION PENET'N JOB 7-57-21 W.P. 31-57 BORING 4 STA. 730+33 (38' RT.)

CASING BX (standard samplers to fit unless noted) DATUM GEODETIC DATE REPORT AUG. 1957

SAMPLER HAMMER WT. 250 LBS. DROP 19 INCHES COMPILED BY AL CHECKED BY ___ DATE BORING JUNE 26 1957

ABBREVIATIONS

V - INSITU VANE SHEAR TEST Q - TRIAXIAL QUICK K - PERMIABILITY
M - MECHANICAL ANALYSIS S - TRIAXIAL SLOW C - CONSOLIDATION
U - UNCONFINED COMPRESSION WL - WATER LEVEL IN CASING CA - CASING
Qc - TRIAXIAL CONSOLIDATED QUICK WT - WATER TABLE IN SOIL γ - UNIT WEIGHT

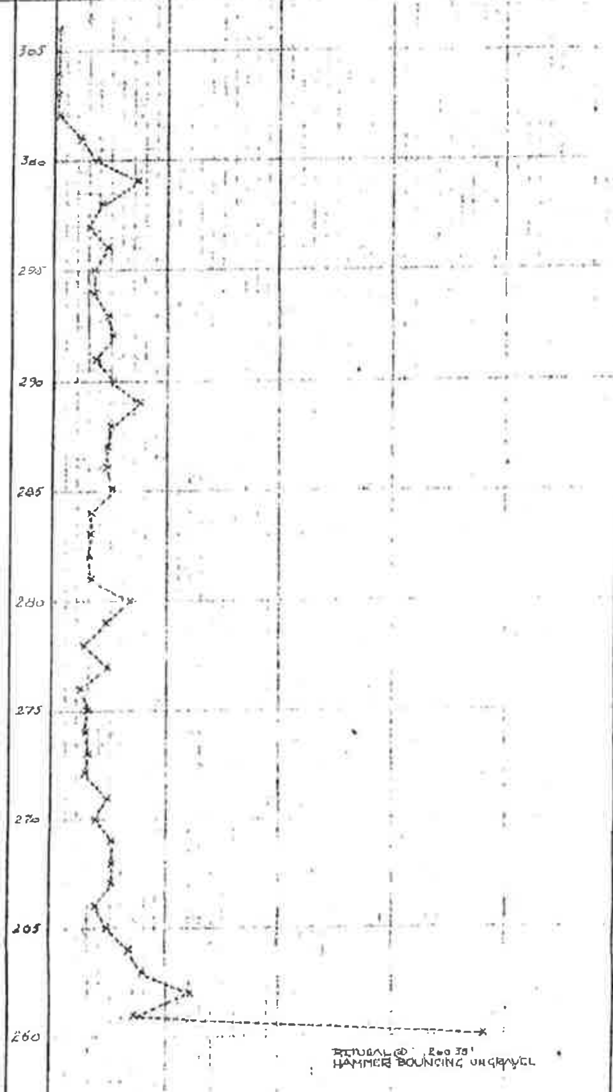
SAMPLE TYPES

CS - CHUNK SS - SLEEVE SAMPLE
DO - DRIVE OPEN PS - PISTON SAMPLE
OF - DRIVE FOOT VALVE WS - WASHED SAMPLE
TO - THIN WALLED OPEN RC - ROCK CORE

SAMPLE CONDITION

- DISTURBED
- FAIR
- GOOD
- LOST

SOIL PROFILE						SAMPLES					
ELEVATION DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT PLOT	ELEVATION SCALE	WATER CONTENT W% — O - NAT □ - PW Δ - LW; PENETRATION TEST RESISTANCE BLOWS PER FOOT AT STANDARD ENERGY (4200 IN. LBS. PER BLOW) D. CONE PEN. X-----X-----X STAND. PEN. •——•——• (50, 100, 150, 200)	CASING BLOWS (ACTUAL)	OTHER TESTS	CONDITION	TYPE	NO.	PENETRATION RESISTANCE	ELEV. RECOV. %
307.35'		GROUND LEVEL									



TL-129 (REV. 56)

DEPARTMENT OF HIGHWAYS – ONTARIO

MATERIALS & RESEARCH BRANCH – FOUNDATIONS SECTION – DOWNSVIEW

OFFICE REPORT ON SOIL EXPLORATION

DRILL RIG 54-1 OPERATION PENIT'N JOB T-57-21 W.P. 51-57 BORING 5 STA. 73+57 (ALT.)

CASING BX (standard samplers to fit unless noted.) DATUM GEODETIC DATE REPORT AUG. 1957

SAMPLER HAMMER WT. 350 LBS. DROP 12 INCHES COMPILED BY A.L. CHECKED BY — DATE BORING JUNE 26, 1957

ABBREVIATIONS

V - INSITU VANE SHEAR TEST
M - MECHANICAL ANALYSIS
U - UNCONFINED COMPRESSION
Q_C - TRIAXIAL CONSOLIDATED QUICK
Q - TRIAXIAL QUICK
S - TRIAXIAL SLOW
WL - WATER LEVEL IN CASING
WT - WATER TABLE IN SOIL
K - PERMIABILITY
C - CONSOLIDATION
CA. - CASING
γ - UNIT WEIGHT

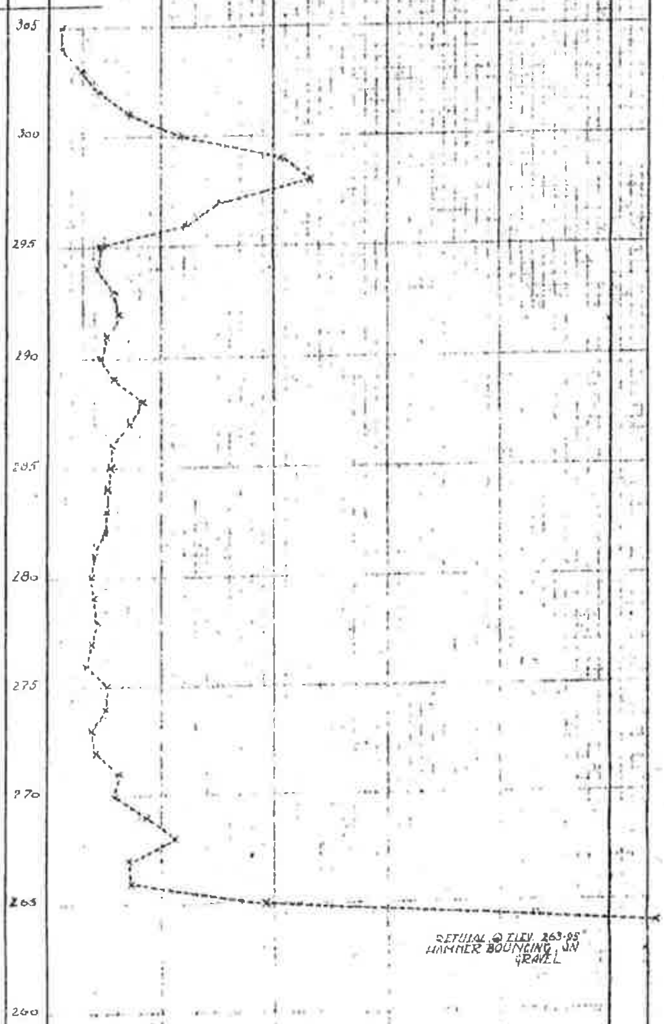
SAMPLE TYPES

CS - CHUNK
DO - DRIVE OPEN
DF - DRIVE FOOT VALVE
TO - THIN WALLED OPEN
SS - SLEEVE SAMPLE
PS - PISTON SAMPLE
WS - WASHED SAMPLE
RC - ROCK CORE

SAMPLE CONDITION

DISTURBED
FAIR
GOOD
LOST

SOIL PROFILE					WATER CONTENT W% — O - NAT. □ - P.W. △ - L.W. PENETRATION TEST RESISTANCE BLOWS PER FOOT AT STANDARD ENERGY (4200 IN. LBS. PER BLOW) D. CONE PEN. X-----X-----X 50 100 STAND. PEN. •———• 150 200	CASING BLOWS (ACTUAL)	OTHER TESTS	SAMPLES				
ELEVATION - DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT PLOT	ELEVATION SCALE				CONDITION	TYPE	NO.	PENETRATION RESISTANCE	ELEV. RECOV. %
305.95		GROUND LEVEL										



Appendix E

Site Photographs

Photograph 1. Existing Cobourg Creek East Bridge (looking west from south side)

Photograph 2. Existing Cobourg Creek East Bridge (looking north from south side)

Appendix F

Explanation of Terms Used in Report

EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

C_u (kPa)	0 – 12	12 – 25	25 – 50	50 – 100	100 – 200	>200
	VERY SOFT	*SOFT*	*FIRM*	*STIFF*	*VERY STIFF*	*HARD*

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

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

ROCKS ARE DESCRIBED BY THEIR COMPOSION AND STRUCUTRAL FEATURES AND/OR STRENGTH.

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

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

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

JOINT AND BEDDING:

SPACING	60mm	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
COBOURG CREEK EAST
STRUCTURE WIDENING
HIGHWAY 401, COBOURG, ONTARIO
G.W.P. 205-00-01, GEOCRES 30M16-42

AECOM

TRANETOB10434AA-AG
August 09, 2011

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

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FOUNDATION DESIGN REPORT

COBOURG CREEK EAST STRUCTURE WIDENING

HIGHWAY 401, COBOURG, ONTARIO

G.W.P. 205-00-01

5 DISCUSSION AND RECOMMENDATIONS

5.1 General

As part of the Highway 401 Expansion (6-Laning) from Burnham Street to approximately 2.0 km east of Nagle Road, within the Town of Cobourg and Township of Hamilton, Ontario, the existing Cobourg Creek East structure will be widened by about 8.2 m on the north side and 6.3 m on the south side. The existing structure is a three span structural steel girder bridge with a concrete deck and asphalt wearing surface. The structural steel girders are supported on concrete piers and abutments constructed at a 24 degree skew to the highway alignment. The existing grade of Hwy 401 at the existing bridge location is about Elevation 97.2 m while the grade at the proposed north and south extensions behind the existing abutments is about Elevation 97.2 to 96.2 m. The embankment grade will therefore be raised by up to 1 m.

The sixteen boreholes drilled for the site and the 1957 MTO boreholes indicate that the site is generally underlain by topsoil, fill (including pavement fill, embankment fill and other fill), native soils consisting of upper granular soils, clayey silt and lower granular soils, which are in turn underlain by bedrock. The embankment fill was found to extend to depths of 4.4 to 5.5 m below the ground surface or to Elevation 92.3 to 91.7 m and generally consist of granular soil within the upper zone and clayey silt fill within the lower zone. The upper embankment fill was described as generally in a compact condition with very loose to loose zones and the lower embankment fill as having a firm to very stiff consistency. The other fill encountered near the creek banks was found to extend to depths of 0.6 to 5.3 m or to Elevation 93.0 to 88.7 m and was described to consist typically of silty sand to sandy silt, in a compact condition with very loose to loose condition within the upper zones. Below the fill layers, native upper granular soils were found and these were described as typically sand, sandy gravel and sand & gravel, with thicknesses of about 0.3 to 3.4 m and in a compact to very dense condition. A firm to very stiff clayey silt deposit was found below the native upper granular soils at Elevation 91.8 to 87.3 m, with a thickness ranging from 8.4 to 10.7 m. Native lower granular deposit was encountered at Elevations 81.9 to 80.1 m and were described as dense to very dense sand, silty sand, sand & gravel and silty sand till with thicknesses ranging from 4.4 to 7.6 m. The top of the bedrock was found to be at depths ranging from 17.4 to 22.1 m below the existing ground surface or at Elevations 76.2 to 74.2 m. Based on the rock cores recovered, the bedrock was described as grey limestone with some dolomatic and argillaceous seams/zones. The site groundwater level at the time of our investigations was at about Elevation 93.0 to 92.5 m (about the same level as the water level in the watercourse) but would be subject to seasonal variations and variations in response to major weather events, as well as variations in the level of water in the watercourse.

5.2 Bridge Foundations

According to the information available to us at present, the existing structure is supported on driven steel H-piles (12BP53). The 1957 investigation by MTO reported the following conclusions regarding this site, "For practical bearing values it will be convenient to support the foundations on end bearing piles. ...it is expected that pile refusal will be met in the compact sand and gravel layer somewhere about Elevation 260 feet." Elevation 260 feet corresponds to about Elevation 79 m, where the presence of dense to very dense lower granular deposits can be expected.

We have considered both shallow and deep foundation alternatives to support the widening, as discussed in the following paragraphs.

The use of normal spread footings is considered unsuitable due to the insufficient bearing resistance of the upper soils and settlements of the clayey silt deposit.

The use of driven steel H-piles and steel tube piles can be considered to support the proposed widening and will be discussed in more details in the following sections.

Drilled and cast in place concrete (caisson) foundations, extended to the underlying competent granular deposit (i.e. dense to very dense lower granular soil), to support the structure can be considered. This however is not recommended because the groundwater is at relatively high elevations and can present construction problems and excessively disturb the base of the caissons in these basically cohesionless soils. However, caissons can be socketed into the underlying bedrock and this option may present an acceptable alternative, minimizing vibrations during construction.

Auger press piles can be extended into the lower granular soils, below the groundwater table but these offer little resistance to lateral loads and will not be economical especially since the construction will be carried out in a watercourse setting. Therefore, this foundation option is not recommended based on reliability and cost.

Expanded base (Franki-type) concrete piles and driven concrete piles are not considered to represent a practical and cost effective solution for this project and as such they are not recommended.

A summary of foundation alternatives is given in a tabular form in Appendix G.

From a geotechnical point of view, the preferred option is the use of steel H-piles (especially since the existing structure is also supported on driven steel H-piles), provided that sufficient precautions are taken to prevent damage to the existing structure due to vibrations created during pile driving.

5.2.1 Shallow Foundations

The site is underlain by up to 5.5 m thick fill. The existing fill is considered unsuitable to support the proposed widening of the existing structure. The construction of spread footing foundations extended into the upper granular soils will require considerable dewatering which will be difficult adjacent to a large body of water (i.e. the watercourse). Furthermore, the underlying clayey silt deposit will settle in excess of the normally accepted settlement for this type of structure. Spread footings supported on Granular 'A' pads can be considered but excessive settlement of the underlying clayey silt deposit will occur. For these reasons, the use of spread footing foundations is not recommended.

5.2.2 Deep Foundations

5.2.2.1 Driven Steel H-Piles

Based on the borehole information, the use of a low displacement pile type, such as a steel H-pile with a heavy section (e.g. HP310x110 or HP310x125) with Titus Standard 'H' Bearing Pile Points (or equivalent) is more suitable under the prevailing conditions compared to other pile types (e.g. steel tube piles, Steel H-piles with lighter sections or precast concrete piles). The H-piles are also suitable for integral or semi integral abutment design, if considered.

The HP310x110 or HP310x125 steel H-piles can be driven to practical refusal in the dense to very dense sand, silty sand, sand & gravel and silty sand till (i.e. lower granular soils). The top of the lower granular soil deposit was encountered at about 12.2 to 16.8 m below the existing ground surface (Elevation 81.9 to 80.1 m). The following table summarizes the approximate pile tip elevations that may be assumed for design purposes for HP310x110 or HP310x125 steel H-piles.

Table 3: Estimated Pile tip Elevations for HP310x110 and HP310x125 steel H-piles

Support Location	Reference Borehole	Estimated Pile Tip Elevation (m)	Soil Deposit
North West Abutment	B3	77.5	Silty sand till
North West Pier	B5	78.0	Sandy gravel
North East Pier	B7	78.0	Sand and gravel
North East Abutment	B9	78.0	Sandy gravel to gravel
South West Abutment	B14	77.0	Gravel
South West Pier	B6	77.0*	N/A
South East Pier	B8	77.0	Sand till / Sandy gravel
South East Abutment	B10	77.5	Gravel to sandy gravel

*Estimated only. Borehole not deep enough.

The following axial resistances are estimated for HP310x110 steel H-piles:

- Factored Axial Resistance at U.L.S. = 1700 kN/pile
- Axial Resistance at S.L.S. = 1100 kN/pile

These values can be increased by 50 kN/pile for HP310x125

For HP 310x125 steel H-piles, following geotechnical resistances can be used.

- Factored Axial Resistance at U.L.S. = 1750 kN/pile
- Axial Resistance at S.L.S. = 1150 kN/pile

Above geotechnical resistance values are increased from HP 310x110 steel H piles by increasing steel area.

As will be discussed later in the report, the grade raise will cause a settlement which will in turn cause a downdrag on the piles. This is due to the fact that the clayey silt deposit can be expected to be subject to consolidation settlements. As the soils surrounding the piles settle, the piles will be dragged down, thus inducing additional loads on the piles due to a phenomenon known as negative skin friction/adhesion. As per Section 6.8.4 of the Canadian Highway Bridge Design Code (CHBDC CAN/CSA, S6-06), downdrag on the piles is considered as a load. For this project, the unfactored downdrag can be taken as 240 kN/pile. However, this may not be necessary depending on the grade raise design. Load factor typically used for this purpose is 1.25. The downdrag acting on the piles can be reduced by the application of bituminous or other viscous coating to the pile surface before the installation. But this is costly and is not recommended as it is not cost effective for this project. A surcharge/preload process can also be considered but this too is considered impractical.

The driving of the piles in the field should be monitored by a recognized pile driving formula such as the Hiley Formula. The estimated ultimate resistance of the piles by the Hiley Formula can be calculated by dividing the recommended axial resistance at ULS by a resistance factor of 0.5, as per current MTO practice. For example, for a ULS value of 1700 kN the resistance sought would be 1700/0.5=3400 kN. In accordance with the above criterion, we recommend that the piles be driven to about 2.5 m above the recommended pile tip elevation and then the driving monitored by employing the Hiley Dynamic Formula, in accordance with MTO Standard Drawing SS103-11.

All pile driving should be carried out in accordance with OPSS 903. Re-striking should be done as per SP903S01. After each pile is installed, an elevation should be taken of the pile top or on a suitable mark on the side of the pile. This elevation should be checked periodically to confirm that the pile has not heaved as a result of the driving of adjacent piles. Piles that are heaved must be re-driven to the required resistance as required by the engineer. At least 10 % of the piles (but not less than two piles) driven at each support element should be re-tapped not less than 24 hours after the driving of the pile, as per SP903S01, to check that relaxation has not occurred. If it has, then all the piles should be re-tapped.

In addition, it may be necessary to stagger the driving of the piles, if heaving is observed. Consideration should also be given to provide an NSSP to alert the Contractor of the possible presence of cobbles and boulders and possible heavy driving requirements through the very dense strata. It is also possible that the piles may be driven some distance below the estimated pile tip elevations to achieve the desired capacity. These aspects should be taken into consideration when ordering the piles.

Assuming that the structure widening is to be supported on driven H-piles, we do not anticipate significant structural/foundation damage to the existing structure due to the vibrations caused by pile driving for the new structure. We recommend, however that a vibration specialist and/or pile driving contractor be consulted on this matter. As well vibration monitoring will need to be conducted during pile driving. An NSSP with regards to vibration monitoring should be considered.

In our experience, the very dense lower granular soil deposit under upward hydrostatic pressure will likely exhibit a high rebound in pile driving and this may create problems in achieving the required resistance in accordance with the Hiley Formula. Furthermore, the additional energy required to achieve the relatively higher resistances will likely create increased vibrations. The piles will also likely be driven about 1 to 1.5 m below the existing piles.

Battered piles from the existing structure may interfere with the new piles and vice versa. This aspect should be taken into consideration in the design and during the construction.

Any settlement of the new structure can be expected to translate into differential settlements between the existing and the new structures and this should be taken into consideration in the design and construction. We recommend installing construction joints between the new and the existing structures.

For frost protection, all pile caps should have a permanent earth cover of at least 1.5 m.

Eccentric loading on piles and the required pile spacing should be considered as per the latest CHBDC CAN/CSA-S6-06 and the Canadian Foundation Engineering Manual (CFEM). Reference can be made to Section C6.8.7.1 of the CHBDC CAN/CSA-S6-06 for assessing lateral pile resistance.

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

$$k_s = n_h z/d$$

Where k_s = coefficient of horizontal subgrade reaction

z = depth

d = pile width

n_h = coefficient related to soil density as given in Table 4.

Where the soil is primarily cohesive, the coefficient of horizontal subgrade reaction can be estimated from:

$$k_s = 67c_u/d$$

Where k_s = coefficient of horizontal subgrade reaction

c_u = undrained shear strength as given in Table 4

d = pile width.

Also presented in the Table 4 are estimated values for angle of internal friction and bulk unit weights.

For estimating purposes, the contribution to lateral pile resistance from the existing fill should be ignored.

Table 4: Recommended n_h values, c_u, angle of internal friction and bulk unit weights

Area Reference / Borehole No.	Applicable Elevation (m)	Soil Type	Bulk Unit Weight (kN/m^3)	Angle of Internal Friction (ϕ) degrees	Recommended n_h value (MN/m^3)	Recommended undrained shear strength, c_u (kPa)
North West Abutment / B3	94.0-93.6	E.Fill:Clayey silt*	18.0			50
	93.6-92.5	E.Fill:Sandy silt to silty sand*	20.0	30	3.0	
	92.5-91.9	E.Fill: Clayey silt*	18.5			70
	91.9-90.4	Gravel	22.0	36	11.0	
	90.4-88.0	Clayey silt	18.5			100
	88.0-83.0	Clayey silt	18.0			50
	83.0-81.1	Clayey silt	18.5			120
	81.1-79.6	Sand to silty sand	20.0	31	4.4	
	79.6-78.8	Sand to silty sand	20.5	33	10.0	
	78.8-75.5	Silty sand till	22.0	35	11.0	
North West Pier / B5	94.0-92.3	E.Fill:Sandy silt to silty sand*	18.0	30	3	
	92.3-90.3	Sandy gravel to sand	22.0	33	4.4	
	90.3-88.0	Clayey silt	18.5			100
	88.0-83.0	Clayey silt	18.0			70
	83.0-81.9	Clayey silt	18.5			100
	81.9-79.7	Sand	21.0	33	11.0	
	79.7-78.2	Sandy gravel	22.0	35	11.0	
North East Pier / B7	93.4-93.0	Fill: Silt*	18.0	29	2.0	
	93.0-89.9	Sand and gravel	22.0	34	8.0	
	89.9-88.0	Clayey silt	18.5			100
	88.0-82.7	Clayey silt	18.0			70
	82.7-80.6	Silt	19.0	31	6.0	
	80.6-77.6	Sand and gravel	22.0	35	11.0	
	77.6-76.2	Silty sand till	22.0	35	11.0	
North East Abutment / B9	94.0-92.3	E.Fill:Clayey silt*	19.0			50
	92.3-90.4	Sandy gravel	21.5	33	8.0	
	90.4-89.0	Clayey silt	18.5			100
	89.0-81.2	Clayey silt	18.0			70
	81.2-76.9	Sandy gravel to gravel	22.0	36	11.0	
	76.9-74.6	Silty sand till	22.0	35	11.0	
South West Abutment / B4 and B14	94.0-92.7	E.Fill: Clayey silt*	19.0			70
	92.7-91.9	E.Fill: Clayey silt*	18.0			40
	91.9-90.0	Gravelly sand	21.5	35	9.0	
	90.0-89.0	Clayey silt	18.5			100
	89.0-85.0	Clayey silt	18.0			70
	85.0-83.9	Clayey silt	18.5			100
	83.9-81.9	Sandy silt to silt	19.0	30	3.0	
	81.9-78.0	Silty sand	20.5	32	4.4	
	78.0-75.2	Gravel	21.5	36	11.0	
South West Pier / B6	94.0-91.7	Fill: Topsoil mixed w. silty sand*	16.0	24	1.0	
	91.7-88.7	Fill: Sand to sandy gravel*	20.5	31	4.0	
	88.7-87.3	Silty sand	21.0	33	8.0	
	87.3-85.0	Clayey silt	18.5			100
	85.0-82.0	Clayey silt	18.0			70
	82.0-80.0	Clayey silt	18.5			100

Area Reference / Borehole No.	Applicable Elevation (m)	Soil Type	Bulk Unit Weight (kN/m³)	Angle of Internal Friction (ϕ) degrees	Recommended n_h value (MN/m³)	Recommended undrained shear strength, c_u (kPa)
South East Pier / B8	93.7-91.8	Fill: Silty sand to sandy silt*	17.0	27	1.0	
	91.8-90.8	Sand and gravel	21.5	33	4.4	
	90.8-89.3	Clayey silt	18.5			120
	89.3-87.4	Clayey silt	18.0			100
	87.4-84.0	Clayey silt	17.5			30
	84.0-81.7	Clayey silt	18.5			100
	81.7-79.7	Sand to silty sand	21.0	34	11.0	
	79.7-77.1	Sand till	22.0	35	11.0	
	77.1-74.8	Sandy gravel	22.0	36	11.0	
South East Abutment / B10	94.0-91.3	E. Fill: Clayey Silt*	18.0			50
	91.3-84.0	Clayey silt	18.5			100
	84.0-81.8	Clayey silt	18.0			80
	81.8-79.6	Sand	20.0	32	8.0	
	79.6-78.1	Sand till	22.0	35	11.0	
	78.1-74.2	Gravel to sandy gravel	22.0	36	11.0	

Note: *The contribution to lateral pile resistance from the existing embankment fill (E.Fill) should be ignored if the pile perimeter is not surrounded by existing fill of at least 5 m distance from the pile (i.e. no lateral support from existing fill).

For preliminary estimation purposes, the lateral pile resistance can be taken as follows for HP310x110 steel H-piles:

Horizontal Resistance at ULS = 140 kN/pile

Horizontal Resistance at SLS = 60 kN/pile

Lateral resistance of the piles can be supplemented by the horizontal components of battered piles. In this instance, we recommend that the batter be limited to no more than 4:1, as in practice greater batter is difficult to install.

In selecting pile locations (especially in the case of battered piles), the locations and pile tip positions of the piles supporting the existing structure should be taken into consideration.

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

A possible risk with this approach is the vibration induced during pile driving, which may cause damage to the existing structure.

5.2.2.2 Steel Tube Piles

Steel tube piles will provide lower resistances in comparison with H-piles as they will not drive as deep, but the lower resistances may be compensated by relatively shorter piles. Steel tube piles have the advantage that they can be inspected after driving and prior to pouring of concrete for possible damage that they may have incurred while driving. They should have sufficient wall thickness and base plate thickness to minimize potential damage caused by the expected hard driving conditions. The end plates should not be wider than the base plate area of the piles (i.e. should not project beyond the circumference of the pile) so that adhesion/friction is not adversely affected. The tube piles will need to be filled with concrete after their installation and inspection for possible damage.

Steel tube piles of 300 mm nominal diameter (e.g. 324 mm x 9.4 mm) driven into the very dense lower granular soil deposit can be expected to provide a Factored Axial Resistance at U.L.S. of 1050 kN/pile and an Axial Resistance at S.L.S. of 700 kN/pile. The following table summarizes the approximate pile tip elevations that may be assumed for design purposes for steel tube piles.

Table 5: Estimated Pile tip Elevations for steel tube piles

Support Location	Reference Borehole	Estimated Pile Tip Elevation (m)	Soil Deposit
North West Abutment	B3	78.5	Silty sand till
North West Pier	B5	78.5	Sandy gravel
North East Pier	B7	79.0	Sand and gravel
North East Abutment	B9	79.0	Sandy gravel to gravel
South West Abutment	B14	77.3	Gravel
South West Pier	B6	77.3*	N/A
South East Pier	B8	78.0	Sand till
South East Abutment	B10	78.5	Gravel to sandy gravel

*Estimated only. Borehole not deep enough.

Recommendations for pile installation and soil parameters for the calculation of the lateral resistance/deflection of the piles were given in the previous section of this report.

Pile lengths may be different than the estimated values shown above. Therefore, this aspect need to be considered in the contract documents and when ordering piles. If battered piles are required, the batter should be limited to 4:1 as discussed in the previous section.

As was mentioned before, steel H-piles are better suited for this project, in comparison with steel tube piles, in view of the fact that steel H-piles are low displacement piles.

5.2.2.3 Timber piles and Driven Concrete Piles

Timber piles are not suitable for the subsurface conditions encountered at the site. As well, they are seldom used for significant structures, due to durability, especially when a major highway is involved, such as Highway 401.

Driven concrete piles are high displacement piles and as such they are not well suited for the prevailing subsurface conditions. In additions, they are locally not cost effective.

5.2.2.4 Caisson Foundations

The following are the recommended geotechnical resistances for the design of caissons (cast-in-place concrete piles).

Table 6: Recommended Caisson Resistances

Support Location	Reference Borehole	Recommended Caisson Base Elevation (m)	Factored Axial Resistance at U.L.S. (kPa)	Axial Resistance at S.L.S. (kPa)	Soil Deposit
North West Abutment	B3	78.5	3000	2000	Silty sand till
North West Pier	B5	79.0	3000	2000	Sandy gravel
North East Pier	B7	79.5	3000	2000	Sand and gravel
North East Abutment	B9	79.8	3000	2000	Sandy gravel to gravel
South West Abutment	B14	77.0	3000	2000	Gravel
South West Pier	B6	77.5*	3000	2000	N/A
South East Pier	B8	78.0	3000	2000	Sand till
South East Abutment	B10	79.0	3000	2000	Sand till

*Estimated only. Borehole not deep enough.

During the installation, caissons would require the use of temporary steel casings to enable the bases to be properly cleaned of any disturbed soils and to enable the inspection and approval of the base by the engineer. The casing would be carefully withdrawn as the concrete is poured. Higher resistances would be available at greater depths but this is not recommended because of extensive dewatering requirements.

The presence of cohesionless soils at the proposed base elevations together with the recorded high water levels (i.e. upward gradient) would necessitate special construction measures including extensive dewatering during the installation of the caissons. This would be difficult to achieve, especially since the pervious granular soil aquifers (i.e both upper and lower granular deposits) may be connected to the water in the watercourse. As well as being very costly, extensive dewatering may cause damage to the existing structure. For these reasons, the use of caisson with base elevations within the basically granular soils is not recommended for this project.

However, caisson foundations will minimize vibrations, in comparison with driven piles. If the use of caissons is desirable for this reason, consideration should be given to extending them into underlying bedrock. Caissons, socketed at least 0.5 m into the relatively sound bedrock, can be designed for the following resistances.

- Factored Axial Resistance at U.L.S. = 4000 kPa
- Axial Resistance at S.L.S. = Need not be considered

These values are applicable to commonly used caisson sizes in Ontario (i.e. between 0.76 and 1.8 m diameter). However, the use of relatively smaller caisson sizes (i.e. between 0.76 and 1.35 m diameter) would be preferable as these are relatively easier and more efficient to install especially in confined areas. For example, a 0.9 m diameter caisson will have a base area of $A = \pi r^2 = \pi(0.9/2)^2 = 0.64\ m^2$. When designed for a factored U.L.S. of 3000 kPa, the caisson would be capable of carrying an axial load of $0.64\ m^2 \times 3000$ kPa = 1920 kN/caisson. Higher resistances would be available with increased depth of socketing, but this is unlikely to be necessary.

The following table summarizes the anticipated bedrock surface and caisson bottom elevations.

Table 7: Caisson Foundation Bedrock Elevations

Support Location	Reference Borehole	Approximate Top of Bedrock Elevation (m)	Approximated Bottom of Caisson Elevation (m)
North West Abutment	B3	75.7	75.1
North East Abutment	B9	74.6	74.0
South East Abutment	B10	74.2	73.7
South West Abutment	B14	75.2	74.6

Proper penetration into the competent limestone bedrock must be verified, including the quality of the bedrock at the base, as per OPSS 903.

The minimum caisson diameter should be 0.76 m to enable the proper 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, problems will likely be experienced due to the presence of non-cohesive soil types (i.e. both the upper and especially the lower granular soils) below the groundwater table. As well, problems may arise due to the presence of cobbles and boulders in the overburden as well as hard dolostone/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 addition, dewatering may be required to install the caissons through the coarse granular deposits (especially the basal granular soils). Installing caisson holes through these coarse granular layers may require additional precautions to prevent the caving of the holes, such as extending the casing simultaneously as the hole is augered (or if necessary ahead of the auger) and the use of bentonite slurry, etc. When the caisson extends to the bedrock the casing/liner would be withdrawn as the concrete is poured, ensuring a sufficient head of concrete in the casing to prevent 'necking'. Even though most of these are standard aspects of caisson installation operations, we recommend that they are 'red flagged' in the contract documents to reduce the possibility of claims for 'extras' by the Contractor, including the possible presence of cobbles, boulders in the overburden and hard layers in the bedrock. An NSSP should be issued to alert the Contrator of these aspects, 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 caissons, although this is unlikely be necessary if the casing is properly screwed into the bedrock.

Driven steel H-piles are believed to have been used to support the existing bridge. One disadvantage of using caissons extended into bedrock to support the widening may be foundation support at different elevations by probably about 2 to 3 m. Another aspect is to make sure that the caissons will not interfere with the existing piles, especially if there are battered piles in that direction. As well, the installation of caissons through the lower granular soils should proceed in a manner so that no loss of ground occurs, which may adversely affect the existing piles, leading to settlement of the existing piles.

If caisson foundations are to be adopted this should be further discussed with a contractor specializing in this field and also discussed with Coffey Geotechnics Inc.

5.2.2.5 Micropile Foundations

An alternative which may be considered is the use of micropiles to support the widening.

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 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 driven H-piles 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.

5.3 Approach Embankments

The presently existing grade of Hwy 401 at the existing bridge location is at about Elevation 97.2 m, while the grade at the proposed north and south extensions behind the existing abutments is at about Elevation 97.2 to 96.2 m. The grade will therefore be raised to maximum 1 m. Based on the original ground surface elevations at the boreholes drilled by MTO in 1957 and the cross section drawings of the existing Highway 401, the construction of the existing approach embankments was assumed to have extended beyond the existing Highway 401 alignment.

Based on the borehole data, generally the existing embankment fill materials behind the abutments of the existing bridge appear to have received some degree of systematic compaction during their placement, with the exception of occasional loose zones which were typically contacted in the lower portions (i.e. near the interface with the o.g.). Based on this and considering the fact that the grade raise will be limited to no more than 2 m in height, it is in our opinion that the existing fill will generally not need stripping and embankment fill needed to raise the grade can generally be placed directly on top of existing fills, provided that any topsoil cover and any other unsuitable soils that may become evident during the stripping and benching process are removed prior to placement of embankment fill.

The following are recommended for site preparation:

- Strip surface vegetation, tree roots, topsoil, other organics, and other unsuitable and/or loose/soft materials. For example, the existing fills encountered at Boreholes B6 and B8 will need to be removed to Elevation 92.0 m and replaced.

- Where feasible, proof roll the exposed surface. (e.g. where the footprint of the new embankment extends beyond that of the existing, near the toe).
- If localized soft spots or excessive heave occurs during proof rolling, further excavate and replace with suitable fill.

The removal of the organic soils should be carried within an envelope given by an imaginary slope no steeper than 1H:1V from the toe of the proposed embankment, as per MTO standard procedures. After stripping, the exposed subgrade should be inspected and approved. It should be compacted from the surface using a suitable compactor.

Some topsoil was encountered in the boreholes drilled towards the bottom of the embankments. For preliminary estimating purposes, the thickness of topsoil to be removed can be taken as 0.2 m in these areas. However, the thickness of the topsoil and/or other organic soils could vary at the o.g. level, especially near the watercourse.

Proper benching of the existing embankment slope should be implemented during the construction of the approach embankments, as per MTO procedures and in accordance with OPSD 208.010.

Foundation failures are not anticipated for the approach embankments of this height (i.e. approximately 3 to 5 m high from the original ground surface) constructed with normal 2H:1V side slopes or flatter, assuming that the soil conditions at and beneath the watercourse are similar to those encountered in the boreholes.

The materials used for the construction of the embankment fills should consist of approved, acceptable earth fill (e.g. Select Subgrade Materials - OPSS1010). Fill used for construction of the embankments should be in accordance with OPSS 212 and fill placement should meet or exceed the requirements of SP 105S10 and OPSS 206. Construction should be in accordance with SP 206S03. In general, the fills should be placed in suitable lift thicknesses not exceeding 300 mm when loose placed and each lift should be uniformly compacted to at least 95 % of the material's Standard Proctor Maximum Dry Density (SPMDD). We recommend that the fill within 0.5 m of the subgrade level should be compacted to 100 % SMDD.

Based on the cross sectional drawings presented in Appendix H and the borehole data, our calculations show that the new embankment loadings would likely result in a settlement of the order of 20 mm due to the settlement of foundation soils, including the existing embankment fills. About one-third of this settlement should take place within one month, with the majority of the remaining settlement to occur within one year. In addition, the settlement of the new embankment fills under their own weight can be expected to occur. If the embankment is constructed to MTO standards, however, this should not exceed 5 mm. The settlement due to the own weight of the new 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 new embankment under its own weight should also be substantially completed within about one month.

As settlements of these magnitude (i.e. 25 mm or less) are not excessive, neither surcharging nor preloading is considered necessary for the approach embankments.

Proper erosion control measures should be implemented both during the construction and permanently. This can be achieved by prompt seed and cover (OPSS 572) or sodding (OPSS 571).

5.4 Backfill behind the abutments

Backfill behind abutments should consist of non-frost susceptible, free-draining granular materials in accordance with the MTO Standards and the requirements of OPSD 3101.150.

Free-draining backfill materials such as Granular 'A' or Granular 'B' Type I, II or III can be used. To maintain free draining characteristics in these granular till materials, the maximum percentage passing the No. 200 sieve (76 μm) should be limited to 5 %. Drains pipes, weep holes and the like should be incorporated to reduce hydrostatic pressure build-up.

5.4.1 Lateral Earth Pressures

Computation of earth pressures should be in accordance with CHBDC. For design purposes, the following parameters (unfactored) can be used.

Compacted Granular 'A' and Granular 'B' Type II

Angle of Internal Friction, ϕ = 35° (unfactored)
Unit Weight = 22 kN/m^3
Coefficient of Lateral Earth Pressure:
K_a = 0.27　　K_b = 0.35
K_o = 0.43　　K^* = 0.45

Compacted Granular 'B' Type I and Type III

Angle of Internal Friction, ϕ = 32° (unfactored)
Unit Weight = 21 kN/m^3
Coefficient of Lateral Earth Pressure:
K_a = 0.31　　K_b = 0.41
K_o = 0.47　　K^* = 0.57

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

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

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

The earth pressure coefficient adopted will depend on whether the retaining structure is restrained or movements can be allowed such that the active state of earth pressure can develop. If the abutment is restrained and does not allow lateral yielding, then at rest pressures should be used in accordance with CHBDC CAN/CSA-S6-06. This is likely to be the case for this project as rigid frame structure is under consideration. Vibrations generated by the highway traffic should also be taken into consideration in the selection of appropriate earth pressure coefficients. The effect of compaction should also be taken into account in the selection of the appropriate earth pressure coefficients in accordance with Section 6.9 of CHBDC CAN/CSA-S6-06.

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

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

5.4.2 Seismic Design Data

5.4.2.1 Site Coefficient

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

5.4.2.2 Seismic Zone and Zonal Acceleration Ratio (A)

Table A3.1.1 of the CHBDC provides a zonal Acceleration Ratio (A) of 0.05 and Velocity Related Seismic Zone (Z_v) of 1 for Cobourg. As site coefficient (S) is 1.0, and the zonal acceleration is 0.20, the design zonal acceleration ratio for the site can be taken as A=0.05.

5.4.2.3 Seismic Earth Pressures

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

In accordance with Sections 4.6.4 and C.4.6.4 of the CHBDC and its Commentary, the horizontal seismic coefficient, k_h, used in the calculation of the seismic active pressure coefficient, is taken as k_h=0.05. The seismic active earth pressure coefficient is also dependent on the vertical component of the earthquake acceleration coefficient, k_v. Three discrete values of vertical acceleration coefficient are typically selected analysis, corresponding to $k_v = +2/3\ k_h$, $k_v = 0$, and $k_v = -2/3\ k_h$.

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

Seismic Active Pressure Coefficients

Active Earth Pressure Coefficient	Granular 'A' (ϕ = 35° - unfactored)	Granular 'B' Type II (ϕ = 32° - unfactored)
Non-Seismic, K_a	0.27	0.31
Seismic, K_{AE}	0.29	0.33

In the calculation of K_{AE}, the effect of the friction between the wall and the soil are assumed as half of soil friction angle (i.e.δ=0.5 x ϕ).

5.4.2.4 Liquefaction Potential

If the proposed structure is supported by deep foundations in the very dense granular soil or bedrock, the materials below the pile tip elevation are considered not liquefiable.

The compact cohesionless fill and stiff cohesive fill materials within the approach fill may not liquefy under earthquake loading. However, some soft portion of the cohesive fill or loose portion of cohesionless fill may be settled under earthquake loading. Similar settlement can be also occurred within the upper granular soil and clayey silt deposit depending on their relative density/consistency under earthquake excitation.

5.5 Construction Comments

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

OPSS 539 – Construction Specification for Temporary Protection Systems

OPSS 902 – Construction Specification for Excavating and Backfilling-Structures.

In accordance with OHSA, the soils can be classified as follows

Fill	Type 3 soil above water level;	Type 4 soil below water level
Clayey Silt (stiff, very stiff or hard)	Type 2 soil above water level;	Type 3 soil below water level
Clayey Silt (firm)	Type 3 soil above water level;	Type 4 soil below water level
Other Native soils	Type 3 soil above water level;	Type 4 soil below water level

Excavations within the existing fill and native soils should be possible using heavy equipment such as a hydraulic excavator. Cobbles and boulders are expected within the native granular deposits. It is assumed that excavations for the site would include stripping of unsuitable soils from the existing embankment and excessive seepage into the excavation is not anticipated and the seepage can be handled by gravity drainage and pumping from open sumps. For excavations extending to not more than about 0.7 m below the ground surface, it will be possible to effect the dewatering by gravity drainage and pumping from filtered sumps. For excavation extending into the upper granular soils in excess of 0.7 m, more aggressive dewatering, including pumping from deep wells may be required.

The excavated soils free from topsoil and organics can be used as general construction backfill where it can be compacted with smooth drum or pad-foot type rollers. Loose lifts of soil, which are to be compacted, should not exceed 300 mm. On site verification of the excavated fill for re-use as backfill by a suitably qualified personnel during construction would be required. The excavated soils, with the exception of upper granular soils, are not considered to be free draining. Where free draining backfill is required, imported granular fill such as OPSS Granular B should be used. Note that the excavated soils are subject to moisture content increase during wet weather which would make these materials too wet for adequate compaction. Stockpiles should therefore be compacted at the surface or be covered with tarpaulins to help minimize moisture uptake.

Temporary support may be necessary to retain the existing embankment fills during construction. However, the existing fill is only about maximum 1 m higher than the widening areas. It would be more convenient to slope the ground rather than shore it for the duration of the construction. Temporary slope of 1H:1V can be used provided no relatively high loadings (e.g. heavy machinery, stockpiles etc.) are present at the crest of the slope.

Shoring will likely to be required to create work area to install deep foundations. Locally, temporary shoring systems generally consist of support provided by conventional soldier piles and timber lagging. Occasionally, driven sheet piling is also used, but this is unlikely be feasible for this project (i.e. closeness to the existing structure and vibrations generated).

The soldier piles can be designed as cantilever structures or supported by raker footings. Anchor system can also be employed depending on the depth of soil to be retained and the required performance criterion. The anchors will likely be extended into the bedrock, in which case an unfactored bond resistance of 600 kPa in the bedrock can be adopted and S.L.S. will not govern. The resistance from the upper 0.5 m of the bedrock should be ignored.

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 to be 1b or 2 depending on the details of the retained structure or embankment (including offset from traffic to the shoring).

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

Table 8: Recommended Unfactored Parameters for Temporary Shoring Design

Soil Type	Ka	Ko	Kp	γ (kN/m^3)
Embankment Fill	0.41	0.60	2.2	18.0
Upper Granular Soils	0.32	0.49	3.1	21.0
Clayey Silt – firm to stiff	0.40	0.58	2.4	18.0
Clayey Silt – very stiff to hard	0.35	0.51	2.9	18.5
Lower Granular Deposit – compact	0.32	0.49	3.1	21.5
Lower Granular Deposit – dense to v.dense	0.29	0.45	3.4	22.0

It should be pointed out that the presence of cobbles and boulders can be expected within the native granular deposits. These can be expected to cause problem during the installation of shoring units. This aspect should be 'red flagged' in the contract document.

The construction of the foundations at pier locations should be carried out in a dry condition, therefore use of a stream diversion or cofferdam at each widening location needs to be considered. Based on the GA drawing provided to us by AECOM, pile cap bottom at the pier locations will extend to about El. 91.5 m.

Based on the boreholes drilled adjacent to the pier locations (Boreholes B5, B6, B7 and B8), the proposed excavation for placing new pile cap will be carried out within the fill and surficial granular soils. A clayey silt deposit underlying the surficial granular soils was encountered in Boreholes B5, B7 and B8 at about El. 90 to 91 m, while the clayey silt layer was encountered relatively deeper in Borehole B6 at El. 87.3 m.

Due to the expected high permeability of surficial granular soil and fill materials and anticipated seepage through the excavation side and bottom, the use of portable cofferdams such as Aqua- Barrier (Nilex, water inflated dams), Portadam (Portadam Inc) and typical concrete jersey barrier with sand bags would only be applicable for this project for the control/diversion of surface water.

The use of a tight interlocking sheet pile cofferdam in the form of single or double row of steel sheet piles can be considered for the control of seepage from the surficial granular soils overlying the relatively impervious clayey silt deposit. The steel sheet piles are commonly used because of their structural strength, water tightness, interlocking characteristics and ability to be driven to sufficient depths in most types of ground. The use of a lighter weight sheet piles such as fiber reinforced polymers (FRP) sheet piles and plastic sheet piles are not recommended for this project because of the prevailing subsoil conditions at the site.

The sheet pile cofferdam should be designed by a Professional Engineer, experienced in this type of work. For the preliminary design of the sheet pile wall, the above mentioned soil design parameters (Table 8) can be used to calculate the required sheet pile penetration depth.

Sheet piles are normally driven into the ground using vibration. However, pushing the sheet piles instead of using vibration to install the sheet piles has become a common practice in areas where minimum ground and noise disturbance is required. Penetration of steel sheet pile through the compact to very dense surficial granular soil and fill without causing damage to the existing structure should be verified with Contractor. In addition vibrations, if any, should be monitored.

Removal of the sheet piles must be planned and executed with the same degree of care as its installation to minimize the disturbance of the ground around the new pier foundations. Depending on the design, there may be a need to cut-off the sheet piles below ground surface, rather than full withdrawal.

5.6 Frost Protection

Design frost protection depth for the site is 1.5 m. A minimum 1.5 m thick permanent soil cover or equivalent thermal insulation is required for all of footings 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 presented in Appendix K are integral part of the report.

For and on behalf of Coffey Geotechnics Inc.

for **Delfa Sarabia, M.Eng.**

Senior Geotechnical Engineer

Ramon Miranda, P.Eng.

Principal

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Appendix G

Summary of Foundation Alternatives

Summary of Foundation Alternatives

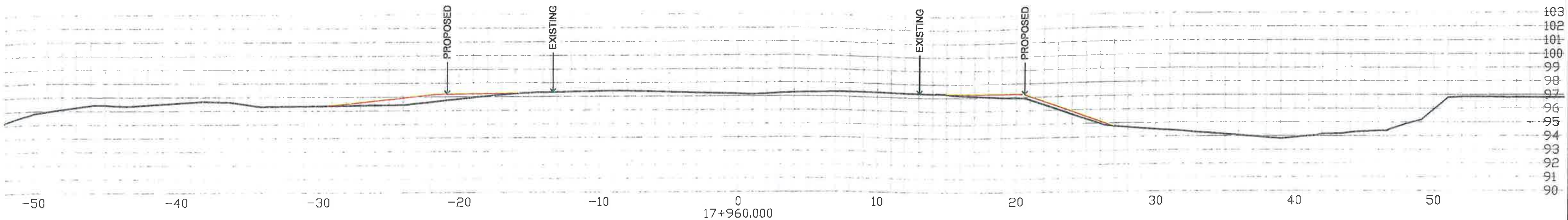
Foundations Type	Advantage/ Disadvantage	Risks/ Consequences	Relative Costs	Recommendations
Normal Spread Footings	Moderate Cost Will require considerable shoring effort Insufficient bearing resistance Does not match existing foundation type	Excessive Settlements Susceptible to instability through scouring	Low to Moderate	Not recommended based on reliability
Spread Footings on Compacted Granular 'A' pad	Moderate cost Does not match existing foundation type. Will require considerable shoring effort	Larger than normal settlements Unlikely to be practical for this project	Low to Moderate	Not recommended based on reliability
Expanded Base (Franki - Type) Concrete Piles	Costly	Vibrations induced during construction will likely damage the existing bridge	High Cost	Not recommended based on cost and reliability
Auger-Press Concrete Piles	Costly Require specialized installer contractor	Difficult to install adjacent to the existing body of water	High Cost	Not recommended based on economics
Driven Concrete Piles	Costly At present, seldom used in Ontario Not well suited for the prevailing subsurface conditions	Susceptible to damage during transport, handling and driving	High Cost	Not recommended based on cost and reliability
Timber Piles	Not suitable for a structure supporting a major highway	Maybe locally damaged while driving through the upper granular soils at some locations Will terminate at higher elevations than existing piles	Inexpensive	Not recommended based on reliability
Micropiles	Minimizes vibrations and dewatering Can be installed in low overhead and restricted space conditions	Cost effectiveness is a main concern	Expensive due to special equipment/ material and specialist contractor	A feasible option but more expensive than some of the other options Can be considered if space restrictions preclude the use of caissons extending into the bedrock and/or if the use of driven H-piles is objectionable due to vibrations.

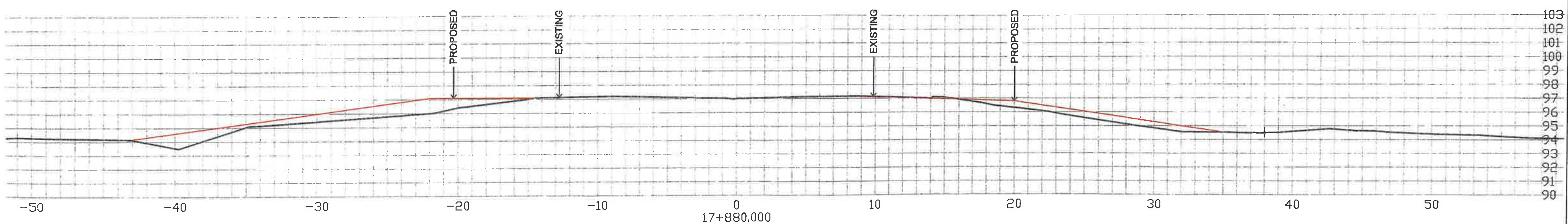
Foundations Type	Advantage/ Disadvantage	Risks/ Consequences	Relative Costs	Recommendations
Drilled and cast-in-place concrete piles (drilled caissons) socketed into bedrock	Less vibrations than driven piles Space restrictions may create problems during their installation	The presence of cobbles, boulders and hard zones in the bedrock may cause problems during the construction of drilled caisson foundations Installing caissons through the water bearing granular soils may present problems	Moderate to costly	A feasible option from reliability point of view but somewhat more expensive than driven steel H-piles
Driven Steel Tube Piles	As they are relatively high displacement piles, they are less reliable than steel H-piles Moderate cost Space restrictions may create problems during their installation	Will induce high vibrations which may cause damage to the existing structure Cobbles and boulders may create problems during pile driving	Moderate Cost	Can be considered as an alternative to steel H-piles but are considered to be less reliable
Driven Steel H-piles	Being low displacement piles, represent a preferred option for the prevailing subsurface conditions Space restrictions may create problems during the installation Vibrations created during installation may be objectionable	Vibrations may cause damage to the existing structure cobbles and boulders may create problems during pile drilling	Moderate Cost	A preferred option based on reliability, suitability and relative cost but vibration monitoring and analysis are essential.

Appendix H

Existing Highway 401 Cross Sections

and Proposed Grade Raise at Structure Location





Existing Hwy 401 Cross Sections &
Proposed Grade Raise at Structure Locations

Appendix I

List of SPs, OPSSs and OPSDs

List of SPs, OPSSs and OPSDs referenced in the report

SP 105S10 Construction Specification for Compaction

SP 206S03 Grading, Earth and Rock Excavation, Excavation for Pavement Widening

OPSS 206 Construction Specification for Grading

OPSS 212 Construction Specification for Borrow

OPSS 539 Construction Specification for Temporary Protection Systems

OPSS 571 Construction Specification for Sodding

OPSS 572 Construction Specification for Seed and Cover

OPSS 902 Construction Specification for Excavating and Backfilling - Structure

OPSS 903 Construction Specification for Deep Foundations

OPSS 1010 Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

OPSD 208.010 Benching of Earth Slopes

OPSD 3101.150 Walls, Abutment, Backfill, Minimum Granular Requirement

Appendix J

NSSPs

H-PILES – HP 310X110 - Item No.

Special Provision

The requirements of OPSS 903, November, 2009 shall govern this specification with the following amendments:

903.07.02 Driven Piles

The Contractor shall note that there is a possibility of the presence of cobbles, boulders and rock slabs in the area where piles are to be installed, and heavy pile driving requirements through the very dense strata. If cobbles and boulders are encountered, the Contractor shall employ the necessary measures to comply with the requirements of OPSS 903. The Contractor shall avoid overdriving and damaging the pile tip, i.e. the structural capacity of the piles shall not be exceeded.

903.10 BASIS FOR PAYMENT

903.10.02 H-Piles - Item

Subsection 903.10.02 is amended by the addition of the following paragraphs:

If obstacles such as cobbles, boulders, rock slabs, and heavy pile driving conditions are encountered there will be no additional cost to the Contract.

<u>VIBRATION MONITORING</u> - Item No.

Special Provision

The requirements of OPSS 903, November, 2009 shall govern this specification with the following amendments:

903.07.02.07

The vibration monitoring equipment shall be placed on the existing structure such that it will not be disturbed. The location should be as close as possible to the piling works.

The vibrations at the existing structure shall not exceed 100 mm/s (peak particle velocity).

The Contractor shall take readings on the first pile in each pile group (i.e. at each corner of the abutment), starting with the pile furthest away from the existing structure. As a minimum, the readings should be taken and recorded during the first 3 m of driving and during seating of the pile onto the bearing layer.

The results shall be certified by the Quality Verification Engineer (QVE) as being accurate and meeting the requirements of the specification. The results shall be submitted to the Contract Administrator prior to continuing with the remaining piles. As a minimum, the pile number, location set criteria and driving log must be submitted with vibration monitoring results.

If the results are acceptable, the Contractor may continue with the remaining piles with readings taken during driving of each pile. Subsequent vibration readings should be taken for each pile during final seating. The results of the subsequent piles should be certified by the Quality Verification Engineer as being accurate and meeting the requirements of the specifications. The results shall be submitted to the Contract Administrator at the end of each day.

If the readings are not within the limits stated above, the Contractor must alter his driving procedures until the vibrations on the existing structure are within acceptable levels. The above process must be repeated for each pile.

Basis of Payment

The above tender item shall be paid on a lump sum basis. The contract price for the above tender item shall include full compensation for all materials, labour and equipment necessary to carry out the work.

CAISSON PILES - Item No.

Special Provision

The requirements of OPSS 903, November, 2009 shall govern this specification with the following amendments:

903.07.03 Caisson Piles

903.07.03.01 General

Subsection 903.07.03.01 is amended by the addition of the following paragraphs:

The Contractor shall note that dewatering may be required to facilitate the installation of the caisson units, especially in cohesionless soils below groundwater table. The Contractor shall be prepared to employ sufficient dewatering procedures to successfully advance the caisson hole and to prevent the loosening and disturbance due to groundwater inflow. Temporary steel liner will be required during the construction of the caisson holes to prevent caving. The liner shall be withdrawn as the concrete is poured, ensuring a sufficient head of concrete in the liner to prevent 'necking'. Concrete must be poured expeditiously after the preparation and approval of the base of the caisson to prevent its disturbance due to hydrostatic uplift.

The Contractor should also note that there is a possibility of the presence of cobbles and boulders in overburden, as well as rock slabs (especially immediately above the bedrock) along with possible hard layers in the bedrock in the area where caisson piles are to be installed. If these obstacles are encountered, the Contractor shall employ the necessary measures to comply with the requirements of OPSS 903.

If caissons socketed into bedrock are to be used, then we recommend that the following provision should also be included in the Contract Documents.

The Contractor should note that the caisson resistances are provided based on a minimum of 0.5 m socket into the relatively sound bedrock. This aspect shall be verified during that installation of the caissons by the geotechnical engineer appointed by the QVE. If the augering proves to be extremely hard and there seem to be no further penetration, the QVE shall make a field decision to discontinue further advance into the bedrock provided that he/she is confident that the required resistances are achieved.

903.10 BASIS FOR PAYMENT

903.10.02 Caisson Piles - Item

Subsection 903.10.02 is amended by the addition of the following paragraphs:

If cobbles, boulders, rock slabs and hard layers in the bedrock are encountered and/or dewatering is required for the installation of the caisson piles, there will be no additional cost to the Contract.

Appendix K

Limitations of Report

LIMITATIONS OF REPORT

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

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

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

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

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